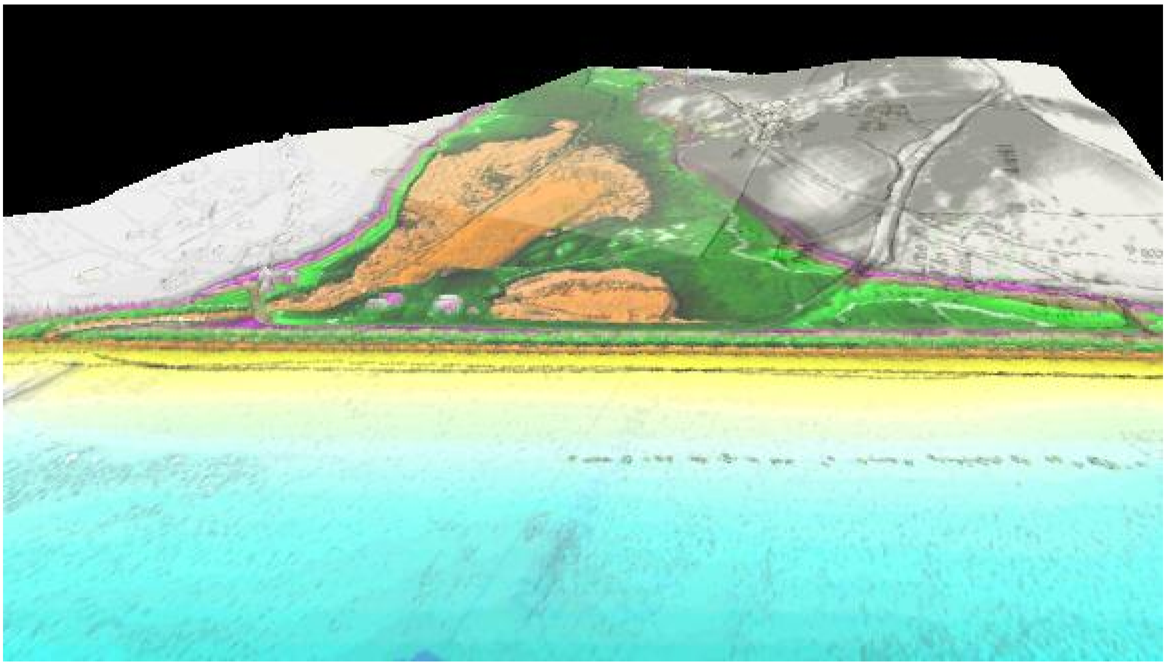

Newgale Shingle Bank
Vulnerability Assessment

Pembrokeshire County Council

December 2014
Final Report
PB2500

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CONTENTS

1 INTRODUCTION

1.1 The purpose of the study and the approach

The large shingle bank at Newgale (Figure 1.1) is situated along the northern stretch of coastline forming part of the shoreline of St Brides Bay. The shingle bank has been overtopped on many occasions in the past, with shingle being washed over the crest onto the main A487 road, resulting in closure of the road and, under more severe conditions, extensive flooding of the low lying valley behind. Earlier this year, during the severe storms that occurred, the shingle ridge was again overtopped and the road closed whilst clearance operations were undertaken.

There has been growing concern over the increasing vulnerability of the shingle ridge and the implications this has at both a strategic level, in terms of the transport network and the future viability of property within the area, and in terms of shorter term management of the ridge. The purpose of this study is to examine the present day vulnerability of the ridge and to assess the degree to which, and over what period of time, the bank may continue to be managed in its current form. A critical part of the study is to consider how historic changes to the bank and future sea level rise affects the situation.

The study builds on previous assessments of coastal management for the area, which are outlined in this section **(Section 1)**. The report then goes on to provide a brief description of the site, the physical factors influencing change and the broader scale changes that have occurred **(Section 2)**. An analysis of beach profiles has been carried out in the context of this change and the present day management approach is discussed in **Section 3**.

The study then considers the vulnerability to bank failure and overtopping, which is assessed for present day and future conditions **(Section 4).** Conclusions and recommendations are made in **Section 5**.

Figure 1.1 Location plan and plates showing the shingle bank

1.2 Background to Management

During the early 1990s a study was undertaken examining the behaviour of the shingle ridge. This study concluded that, unless there was significant strengthening of the shingle bank (which was considered not to be practical), there would be increasing pressure for roll back and an increased occurrence of overtopping and failure of the bank.

These conclusions were reiterated by the first generation of SMP policy for the area and this conclusion was also drawn by the policy stated in the second generation of West of Wales SMP (SMP2). The SMP2 policy for the frontage is managed realignment (MR) and eventually allowing the shingle ridge to develop naturally or No Active Intervention (NAI), as shown in Table 1 below.

Table 1 SMP2 Policies

2.11	Newgale Sands north	MR	MR	NAI	Manage shingle on the road but with the long term intent of allowing the shingle ridge to behave naturally.
2.12	Newgale village	MR	MR	MR	Manage the cliffs and position of the stream to sustain the upper village.

At present, the situation is being managed, accepting that the shingle ridge will be overtopped on occasion and that when this occurs shingle is lost from the crest of the bank blocking the road. The events in 2014 were a relatively extreme case where the road was closed as it was flooded and blocked by shingle. The coastal water level, based on tide records from Fishguard and Milford Haven, reached a level estimated to be in the order of a 1:20 to 1:25 year extreme water level event (see Section 2). However, the road is also closed more frequently during less extreme conditions.

In 2001 an initial study was undertaken that highlighted the risk as a consequence of both wave height and water level. With extreme wave heights, overtopping and bank failure can occur even on relatively frequent water levels. With more extreme water levels overtopping or ridge failure can occur on relatively low wave heights.

The current response, after an event, is to reclaim shingle from the back face of the bank and the road and to attempt to rebuild the crest and front face of the ridge. This does provide some additional protection. However, research has shown in other areas that such action does not fully restore the natural resilience of a shingle bank. While this is not fully understood, it is considered that a naturally sorted ridge, with sediment within the profile being arranged by wave action, provides a ridge that is able to better withstand subsequent wave action.

Coupled to this, there are more progressive pressures on the ridge, attempting to move the ridge back over time. Where this is resisted or where attempts are made to restore the bank and beach to a previous, more forward position, the shingle ridge is placed in a more vulnerable position. Clearly with sea level rise, where increased water levels allow more wave energy to work on the backshore ridge, the pressure for roll back increases.

It is against this background of both natural change and the intent of future management that current actions are being taken. At some point in time further restoration of the ridge will become unsustainable as the benefits of providing increasingly limited additional or temporary protection will no longer be effective. This study aims to examine the existing pressures on the shingle ridge and assist in understanding when such a situation may develop.

A change in the management of the shingle ridge will necessitate significant changes in the use of land behind; both in terms of the use of the road and the ability to maintain acceptable standards of risk to property.

2 DESCRIPTION OF THE SITE AND BEHAVIOUR

2.1 General Description

Newgale is situated in the northwest corner of St Brides Bay (Figure 2.1) on the western coast of Pembrokeshire. The Bay faces out to the west and south west and is exposed to waves from these the dominant directions. The shoreline of the bay is exposed to both locally wind generated waves as well as long period swell waves which develop out in the Atlantic. The whole bay, with its gradually shoaling nearshore area, is strongly aligned to the direction of these waves.

The shoreline of St Brides Bay generally comprises a wide sand lower intertidal foreshore and in areas where there are low lying valleys at the shoreline, such as Newgale, the shore is backed by a shingle ridge.

At Newgale, this shingle ridge has formed across the low lying valley of the Brandy Brook. The Brook runs through the centre of the valley, exiting to the sea at the northern side of the valley where its channel forces its way behind the shingle ridge. At present NRW manage the course of the Brandy Brook, clearing the river to allow drainage of the main valley.

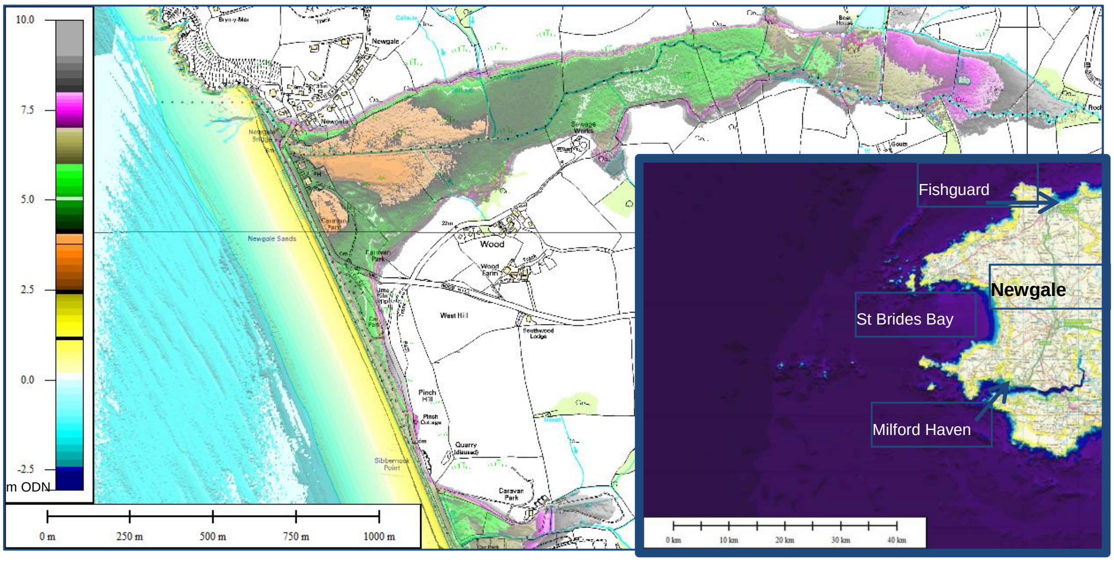


Figure 2.1 Layout of the low lying valley, the village of Newgale and the main road.

The main village of Newgale is situated on the rising land to the northern end of the bay, but there is a row of properties, car parks and a camping site behind the road within the low lying area of the valley. Further to the south, where the A487 runs down to the coast, the coast road (the Welsh Road) continues along the coast to the south, set back behind the natural shingle ridge protection. This minor road leads through to Nolton Haven and further south to Broad Haven.

2.2 Water Levels

Water levels have been determined by the SMP2 based on Admiralty Tide Tables (normal tidal conditions) and from estimates of extreme water levels at the time of producing the SMP. These are set out in Table 2.1 below.

Table 2.1. Water levels determined by SMP2.

Water levels: Significant variation between Milford Haven and Fishguard with tidal ranges of 6.3m and 4m respectively. High water levels at Solva and Little Haven would suggest extremes relate more to Fishguard; however the tidal range would suggest values closer to those given for Milford Haven.

	Tide m. AOD				Extreme Levels (return period) m AOD			
Location	MLWS	MLWN	MHWN	MHWS	10 yr	50 yr	100 yr	200 yr
Fishguard	-1.64	-0.44	0.96	2.36	3.21	3.38	3.49	3.56
Solva	-2.40	-0.80	1.10	2.40				
Little Haven	-2.55	-0.95	1.15	2.65				
Milford Haven	-3.01	-1.21	1.49	3.29	4.31	4.44	4.56	4.64

Normal tide levels for Newgale are taken as being those applying to Little Haven.

Since the development of the SMP, extreme water levels have been reassessed by the Environment Agency/ Natural Resources Wales and these revised levels (including those specifically for Newgale) are presented in Table 2.2.

Table 2.2. Revised extreme water levels.

	Tide m. AOD				Extreme Levels (return period) m AOD			
Location	MLWS	MLWN	MHWN	MHWS	10 yr	50 yr	100 yr	200 yr
Fishguard	-1.64	-0.44	0.96	2.36	3.31	3.46	3.52	3.58
Solva	-2.40	-0.80	1.10	2.40	3.72	3.88	3.94	4
Newgale					**3.77**	**3.93**	**4**	**4.06**
Little Haven	-2.55	-0.95	1.15	2.65	3.83	4	4.06	4.13
Milford Haven	-3.01	-1.21	1.49	3.29	4.4	4.59	4.67	4.75

2.2.1 Recorded Water Levels 2014

During the early part of 2014 a series of storms hit the coast of Pembrokeshire. Tidal records for Fishguard and Milford Haven have been examined and the four most severe water level conditions are recorded below in Table 2.3, assessed against predicted return periods for each water level.

Table 2.3. Recorded water level at Fishguard and Milford Haven

	Fishguard		Milford Haven	
Date	m ODN	Return period	m ODN	Return period
3rd. January 2014	3.36	T10/20	4.49	T10/20
1st. February 2014	3.26	T5	4.39	T10
3rd. February 2014	3.36	T10/20	4.44	T10/20
2nd. March 2014	3.26	T5	4.39	T10

It is highlighted that there was, in terms of return periods, significant variation between records at the Standard Ports and that, depending on wind direction and meteorological differences, there might have been variation within St Brides Bay. It is also noted that the impact at different sites in the area critically depended on wave conditions.

2.3 Wave Climate

Wave conditions inshore depend on the offshore wave height and water levels at the shoreline, determined by wave breaking. The Posford Haskoning report 2001, referred to an analysis undertaken by EA Wales in relating offshore wave conditions to wave heights in the nearshore area. This is shown in Figure 2.2.

The dominant wave directions are from the southwest and west, although the area is exposed to waves generated from the broader sector, from the south through to the northwest. Newgale is protected by the St David's headland from the worst wave conditions from more northerly directions. The graph (Figure 2.2) shows that the more extreme wave heights inshore (in excess of 2.5m wave heights) can be generated by relatively low (6m wave height) waves from the two principle directions. More significant wave heights offshore are required to drive extreme wave heights from a southerly direction. As reported in SMP1, the fifty year offshore wave height for the area is in the order of 25m.

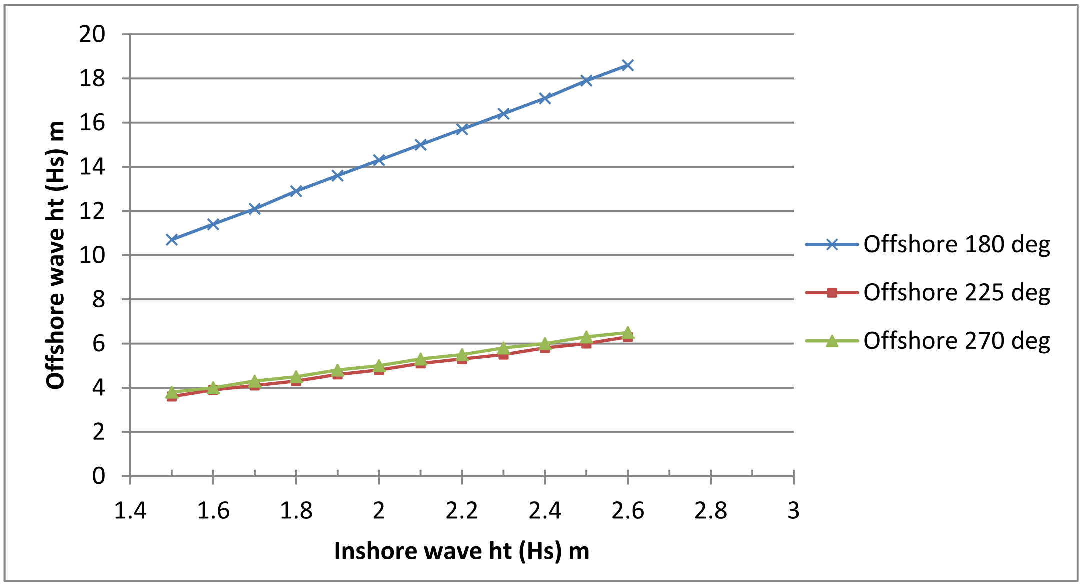


Figure 2.2. Relationship between offshore and inshore wave conditions

For this study, inshore wave heights between 1.5m and 3m have been considered as being representative of a range between normal storm and extreme storm conditions that impact on the shoreline. No attempt has been made to assess joint probabilities of wave height and extreme water levels. However, from the graph above, it seems quite realistic to assume that a 3m inshore depth limited wave could be associated with a high energy event driving significant elevation (surge) in water levels.

Wave period can be significant in determining both the effect of waves overtopping and the influence of waves in changing the beach and shingle ridge. Wave periods of 15 seconds can be associated with longer swell waves in the offshore area. More typically under storm conditions waves may occur at the shoreline with a wave period anywhere between 4 seconds and 12 seconds. This study has, were appropriate considered 9 second and 11 second waves in assessing overtopping and beach behaviours.

2.4 Geomorphological Behaviour

2.4.1 Survey Data

Survey information for the frontage has been taken from:

- Lidar - in 2006 and post storms in 2014.
- Land surveys – 2001 and 2011.

The land surveys have not always been undertaken covering the same profiles of the beach and while the 2011 survey points have been able to be related directly to Lidar, there is some uncertainty associated with the 2001 survey points. These points have had to be adjusted based on comparable fixed points taken from OS 10,000 mapping and Mastermap information. Notwithstanding this, the 2001 survey provides a realistic assessment of beach change for the area.

2.4.2 General Assessment from Lidar

Figure 2.3 shows a comparison of general ground levels based on Lidar a) 2006, b) 2014.

Figure 2.3 a) shows the uniform sweep of the frontage, generally in alignment with the nearshore wave direction. To the north the Brandy Brook squeezes out behind the shingle ridge with the land rising behind. Over the central section of the frontage the shingle ridge is formed across the wide low lying valley and to the south the shingle ridge lies to the front of rising land to the rear.

At this broad scale the narrow crest of the ridge is seen to be formed quite consistently across the whole area, rising steeply from the wider, more shallow gradient of the lower foreshore. Significant areas within the valley behind are seen to be at or below the level of more extreme water levels, as reported in Table 2.3.

It would be anticipated that under normal conditions there is little sediment drift, possibly a slight net drift towards the north.

In the storms of 2014, the lower beach was stripped of sediment in places down to harder material, quite possibly resulting in erosion to the lower foreshore platform. In some areas beach levels dropped by over 1m. The relative position of the 0m ODN contour is shown in the comparison between a) and b) on Figure 2.3.

On the 2014 plot (Figure 2.3 b) two significant low water banks (probably formed of harder material) are seen. With the near normal wave direction, these features may have subtly influenced movement of sediment within the lower and upper beach, resulting in sediment being moved in different directions along the frontage. While there was a general depletion of sediment, this amplified this process in specific areas.

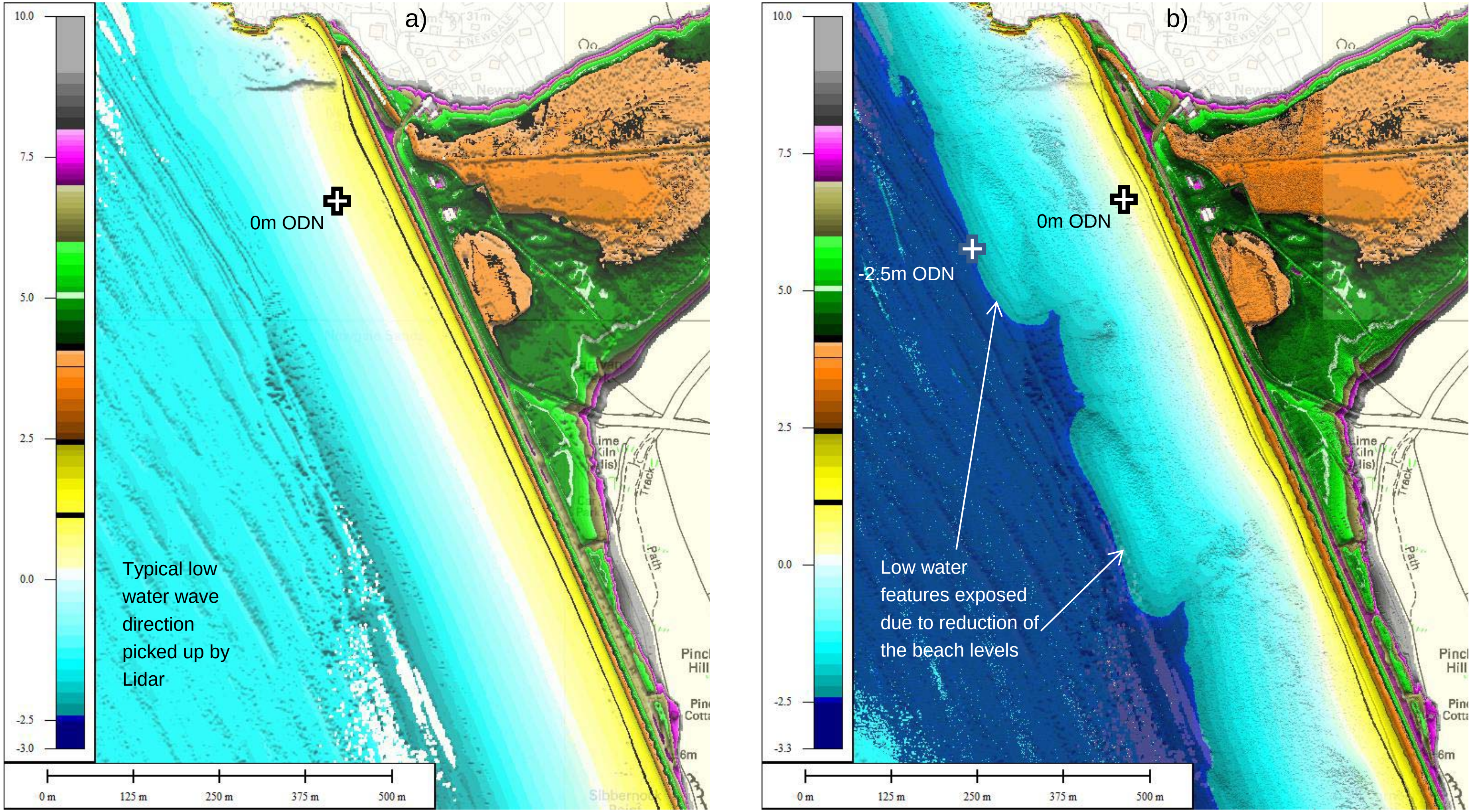


Figure 2.3. Comparison of ground levels between a) 2006 and b) 2014

A series of profiles have been examined along the frontage based on the survey information set out above. The position of profiles is shown in Figure 2.4.

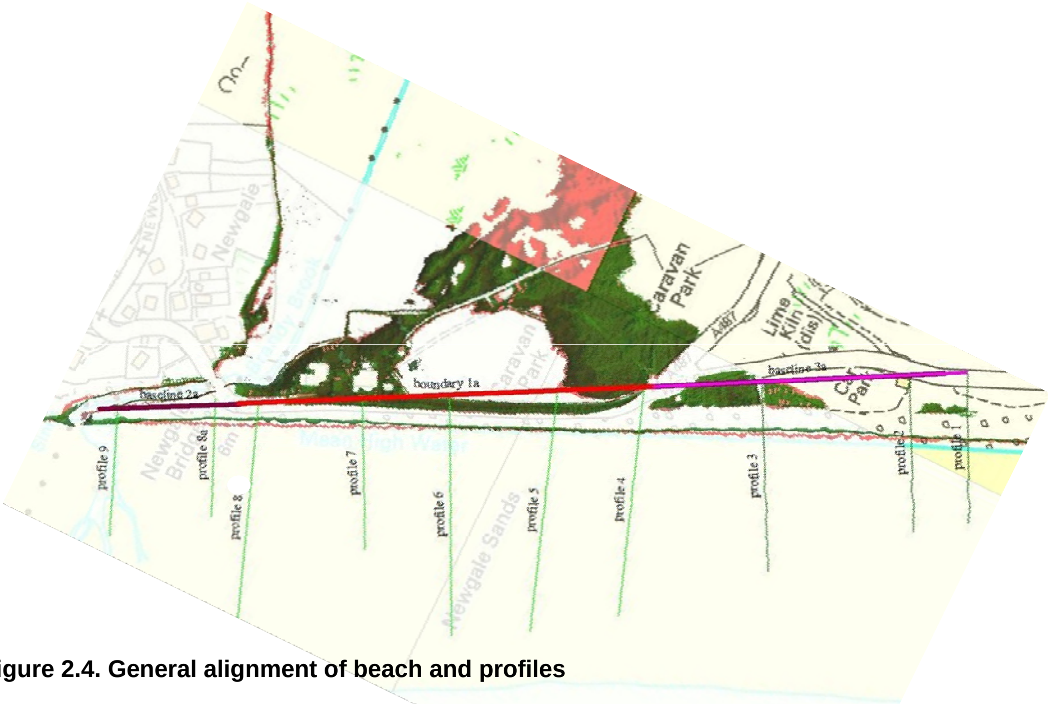


Figure 2.4. General alignment of beach and profiles

Three representative profiles shapes are presented in Figure 2.5 showing the general shape of the beach following the storms in 2014 at:

a) Profile 8, the northern section where the road turns to cross the bridge before climbing through the main village of Newgale,

b) Profile 5, to the southern end of the managed beach section where the road runs up hill and inland; and,

c) Profile 2, further south, to the southern end of the car park along the coastal road towards Nolton Haven.

The profiles have all been plotted at approximately the same scale, showing typically 100m of ridge and beach profile. The actual profiles are taken from a common baseline to allow incorporation of land survey information.

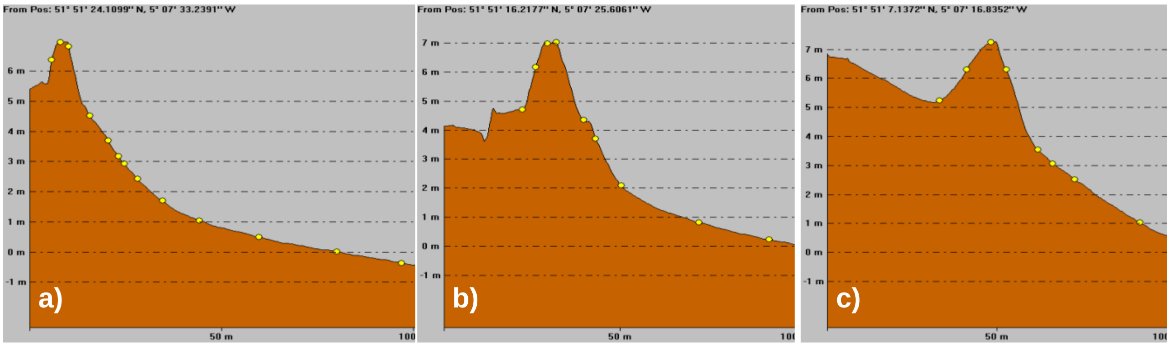


Figure 2.5. Variation in beach and ridge profiles - a) Profile 8, b) Profile 5, c) Profile 2

The shape and nature of the three profiles changes along the frontage with:

- Profile 8, established as a narrow ridge perched at the crest of the underlying beach and valley face. The valley behind tends to run up to the higher ground to the north.

- Profile 5, representative of the slighter wider shingle ridge in front of the main low lying valley.

- Profile 2, showing the shingle ridge perched on the general beach and coastal slope behind.

The local behaviour of each profile and its sensitivity to overtopping and breach are discussed later in Sections 3 and 4 of the report. However, at each section the beach and ridge represented above has been seen to respond in a different manner with respect to sediment movement and coastal processes. This may be seen by the change in the general pattern of retreat, shown by the comparison of change at the 5m ODN contour (shown as 2006 – red and 2014 - green) shown in detail in Figure 2.6 below.

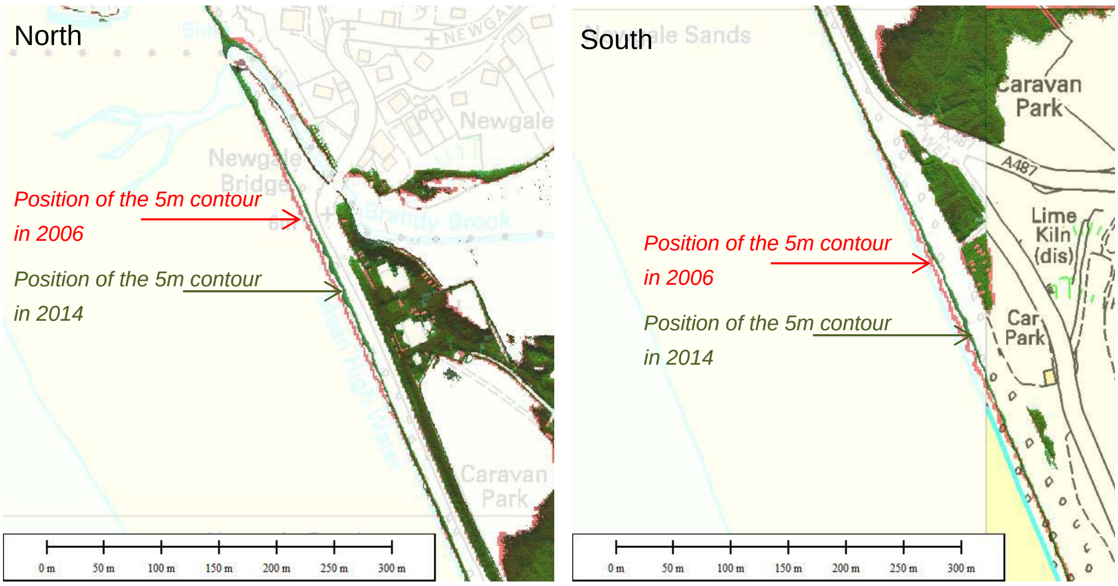


Figure 2.6. Differential change in shingle ridge 2006 compared to 2014.

It may be seen that over the very northern end of the frontage there has been relatively little change in the 5m contour, although in detail the bank was overtopping and shingle was deposited in the stream behind.

Over the northern section of the managed ridge, there was a significant retreat of the whole ridge profile as shown by the distinguishable difference in the 5m contour line. In front of the Caravan Park to the southern end of the managed frontage there was little overall change in the 5m contour and there is some evidence that sediment was moved probably from both the north and the south, to feed sediment into this general area. There was, however, a change in detail of the profile within this area as discussed in Section 3.

Further south, again there was a substantial retreat of the profile at the 5m contour but with some accretion at the toe of the bank and overwash of sediment to the back face.

2.5 Overview

The frontage appears to be relatively well aligned to the general wave direction over the foreshore area, however, during more major events this can change slightly. Although the main response to storms is for the bank to roll back in land, there can also be some variation in this process, which can increase the vulnerability of specific sections of the frontage.

During a major event, such as those that occurred in the early part of 2014, the lower foreshore can be exposed. This may result in slow erosion of the harder foreshore platform such that, despite the anticipated rebuilding of sand over the lower foreshore, there is a more general retreat of the whole intertidal slope.

It would appear, and be expected, that the shingle ridge is, in effect, perched upon this lower platform of glacial clay infilling the main valley and extending as an intertidal slope. As the overall coastal profile retreats, the shingle bank also tends to move back within the low lying area.

Significantly, the coastal slope appears to have only a veneer of mobile sediment (sand and shingle) over the lower foreshore. While some additional sediment would be provided by the reduction of the main coastal platform, this supply will be relatively limited. Sediment, primarily sand, will be brought in by normal wave action from the nearshore area which would tend to rebuild the lower foreshore. However, it is anticipated that little new shingle will be made available to replenish the shingle ridge.

Shingle banks tend to move back both gradually (rollback), and through failure and reconstitution. In the first instance, this occurs where sediment is typically removed from the front face and deposited on the rear face of the ridge due to partial overwash, causing a slow retreat of the ridge.

In the latter case, the crest and a significant bulk of the ridge fails, forming an overwash fan behind the ridge. Under natural processes, as the ridge retreats, it is gradually reformed over the overwash fan, slowly reconstituting the sediment within the active ridge. This collapse, retreat and reforming tends to move the ridge back in a stepwise manner. If not managed this process would be happening along much of the Newgale frontage. Where the intervention takes place along the road section, overwash sediment is being artificially restored to the front face of the ridge in an effort to maintain the ridge in its current forward position.

The next section considers this by examining the changes that have occurred to different profiles and sections of the frontage.

3 ANALYSIS OF PROFILES

Eleven cross section profiles have been analysed covering the frontage as shown previously in Figure 2.4. These profiles have generally been taken where there is land survey data for either 2001 or 2011. Profiles 1 to 3 cover the largely unmanaged section to the south. Profiles 4 to 9 (including profiles 7a and 8a, where land survey profiles had to be interpolated) cover the managed section of the frontage.

A summary of the rate of change seen in each profile is shown in relation to the 5m ODN contour in Table 3.1 and this information is plotted in Figure 3.1 below. The 5m ODN contour was chosen as being representative of the profile above normal water levels and therefore better reflects the longer term behaviour of the ridge as a whole.

Clearly the 2014 storm events were exceptional and will have had a more marked impact on the shoreline. However, these storms are seen as part of the longer term process influencing the behaviour of the shingle bank, in the same way as periods when little change occurs. It is recognised however that the average rate of change taken between the two Lidar surveys (2006 to 2014) has to be treated with caution and does not necessarily reflect the long term rate for sections of the frontage.

Table 3.1. Rates of change at the 5m contour between surveys.

	Rate of change between surveys m/yr (+ retreat, - accretion)			
profile	**01 to '06**	**06 to '11**	**11 to '14**	**av 2006 to 2014**
Pr 1	-0.04	No survey	No survey	**0.23**
Pr 2	0.06	0.14	0.43	**0.25**
Pr 3	0.00	0.18	0.97	**0.48**
Pr 4	0.10	-0.48	0.23	**-0.21**
Pr 5	-0.14	0.18	-0.30	**0.00**
Pr 6	0.20	0.04	0.27	**0.13**
Pr 7	-0.02	0.20	1.40	**0.65**
Pr 7a	No survey	-0.02	1.73	**0.64**
Pr 8	0.22	-0.12	1.87	**0.63**
Pr 8a	-0.26	0.26	1.47	**0.71**
Pr 9	-0.30	0.52	0.10	**0.36**

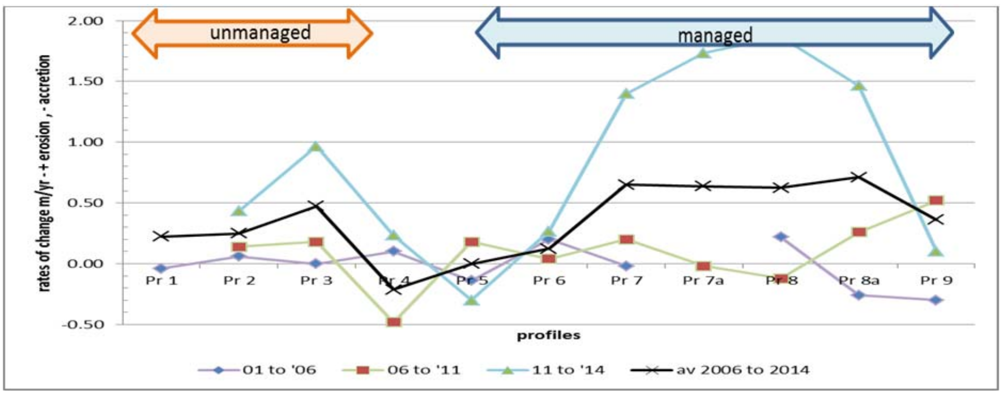
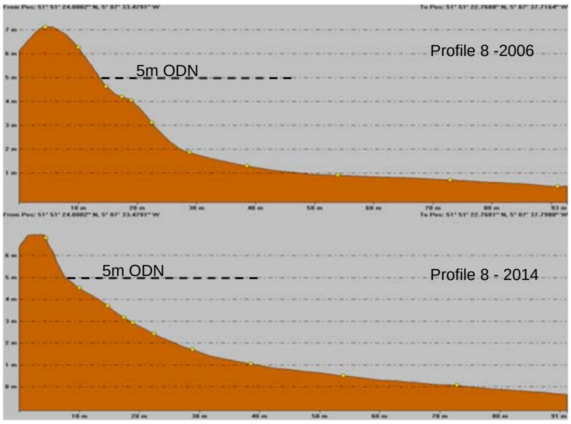


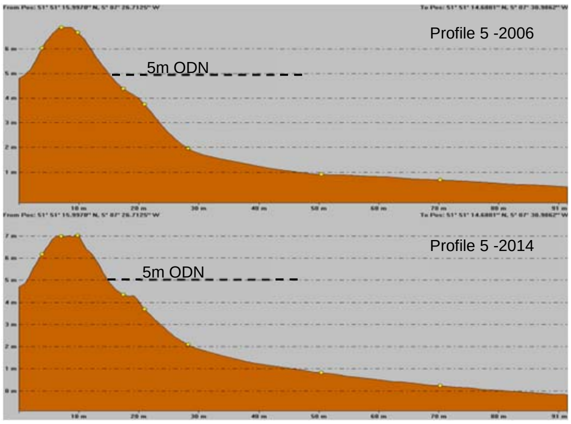
Figure 3.1. Variation in rate of change by profile.

It can be seen that there is quite significant variation along the frontage. Generally over the northern section there has tended to be more retreat, obviously but not exclusively due to the 2014 storms. This process has continued despite re-profiling of the area. In the centre of the frontage there has been less variation, even taking account of the 2014 storms and at profiles 4 and 5, there has been little overall change in position. Over the unmanaged section to the south, there continues to be a general pattern of retreat

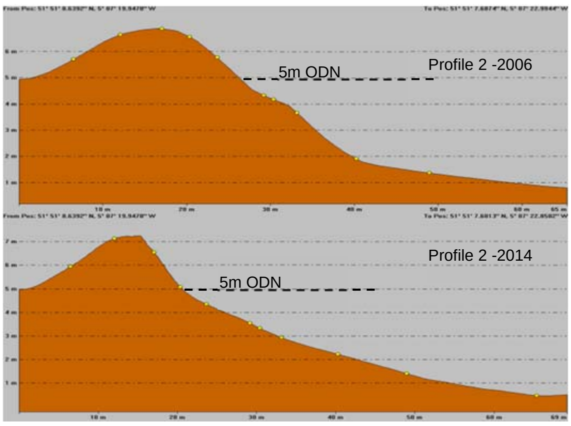
With respect to the change that occurred over the 2014 storms compared to Lidar 2006:



Profile 8 shows a significant cutting back of the main beach face and significant thinning of the upper ridge. (Figure 3.2 a)



At profile 5, the actual crest of the bank has set back slightly with some overtopping and overwash to the back face. Both the front and back faces are steeper, despite the ridge appearing to retain its integrity. There has been some draw down of sediment at the toe of the ridge. (Figure 3.2 b)



Around profile 2, there has been significant cutting back of the front face of the upper ridge and some overwash of sediment to the back face. There has been some draw down of sediment to form a more gradual transition between the toe of the ridge and the foreshore. (Figure 3.2 c)

Figure 3.2 comparison between 2006 (upper) cross section and 2014 (lower).

3.1 Management of the Ridge

Over the period considered above, there has been on-going management of the ridge. Typically this takes the form of clearing the road and areas behind the road of shingle and re-profiling the crest of the bank with material won from this action. As such the analysis given above does include these interventions along the managed section of the frontage. In particular, the 2014 Lidar survey does show the reformed ridge crest.

The Council have attempted to maintain a consistent width and level at the crest and the desired profile is shown in Figure 3.3.

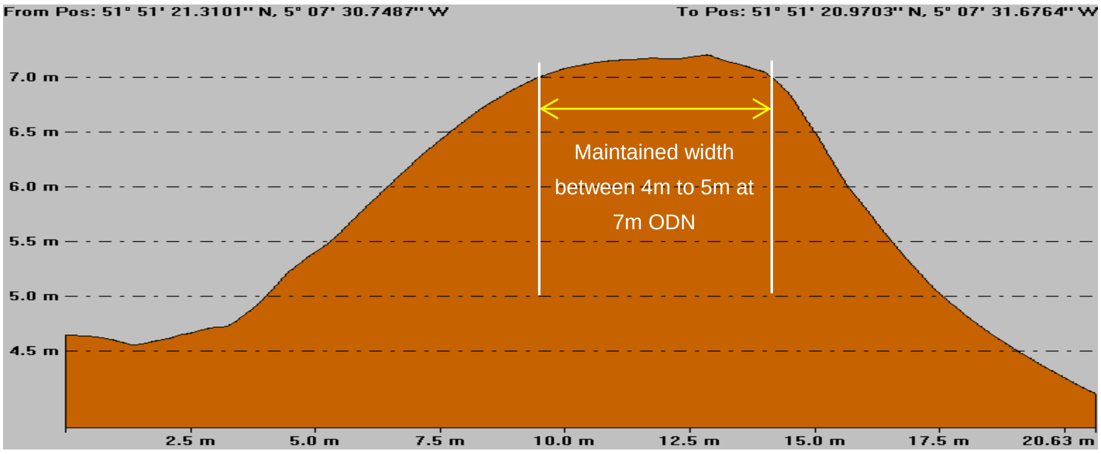


Figure 3.3. Maintained crest shape, based on Profile 7.

With some sections of the ridge this appears not to have been possible, notably at Profile 8 where the crest is shown to be only around 3m wide. In other areas the crest level remains below 7m ODN.

The managed profile crest does however attempt to match that of the more natural ridge further to the south.

Although reinforcing the crest potentially reduces overtopping during more regular storm conditions, it does not address the fundamental problem of loss of overall bulk of the ridge. The works can result in over steepening both the front and back face of the ridge. Furthermore, as discussed earlier, it is recognised that artificially placed sediment does not respond in the same manner as naturally sorted shingle and is therefore more vulnerable to subsequent movement by wave action.

The next section considers the overall stability of the shingle ridge with respect to change in form and overtopping.

4 SHINGLE RIDGE VULNERABILITY

Two approaches have been taken in examining the vulnerability of the ridge. In the first, an analysis has been carried out on the form of the ridge, considering how the ridge deforms under different wave and water level conditions. In the second, critical profiles have been considered using the overtopping model AMAZON. This considers, in relation to a defined profile, the degree to which wave overtopping occurs.

In both analyses a standard matrix of wave heights and water levels has been taken, as shown in Figure 4.1. The range of water levels have been chosen to reflect both present day return periods and cover the range of extreme values likely to occur in the future with sea level rise (SLR). For comparison, Figure 4.1 sets out the return period of extreme water levels and how these return periods change with sea level rise.

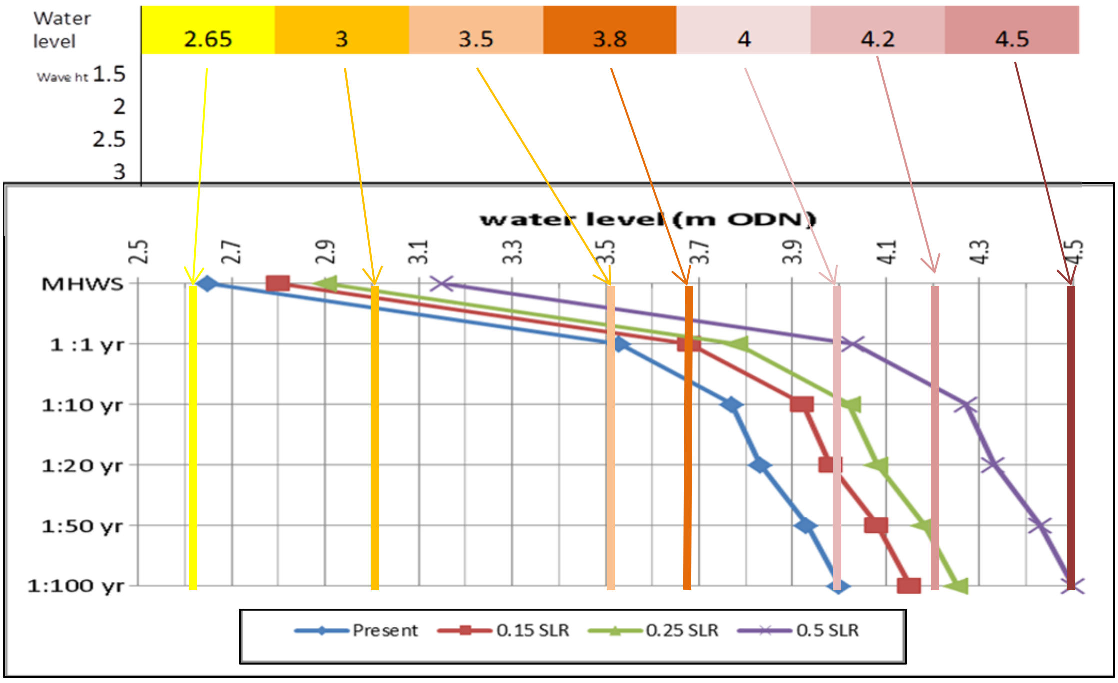


Figure 4.1. Matrix of wave heights and water levels.

From this analysis it can be seen that a level of 3.8m ODN, which has a return period of around 1:20 years now, would have a return period of closer to 1:10 year assuming a 0.15m SLR and a 1:1 year return period assuming a 0.25m SLR. Similarly a water level of 4m, which at present has a return period of 1:100 year, would typically have a return period of 1:1 year assuming a 0.5m SLR.

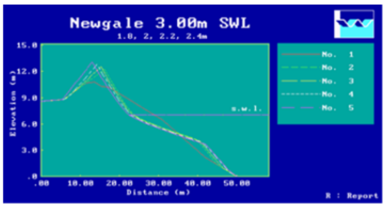
Extreme water levels over the storms during 2014 ranged between T10 (1:10 year) and T20 based on the recorded levels from Fishguard and Milford Haven (Table 2.1). This equates to a level around 3.8m ODN at Newgale.

4.1 Shingle Bank Behaviour Analysis

Two independent forms of analysis have been undertaken. The first uses the empirically based model Breakwat. This model was originally developed to consider the behaviour of rock armour breakwaters. It has however proved a useful way of considering how shingle banks deform under different water level and wave height conditions as shown in Figure 4.2. From the analysis of profiles described in Section 3, five profiles have been considered covering areas where different changes may have occurred.

- Profile 8a considers the wider section of the bank at the northern end of the frontage.
- Profiles 7a and 8 consider the section of the bank where greatest change occurred.
- Profile 5 and 6 consider the southern section of the managed section of the bank, where little retreat of the bank occurred.
- Profile 3 represents the more natural bank to the south of the frontage.

The changes seen in the bank for different water levels and wave conditions run in Breakwat for each profile have been classified as set out in Figure 4.2. This colour system is used in developing the vulnerability matrices report below (Figure 4.3).



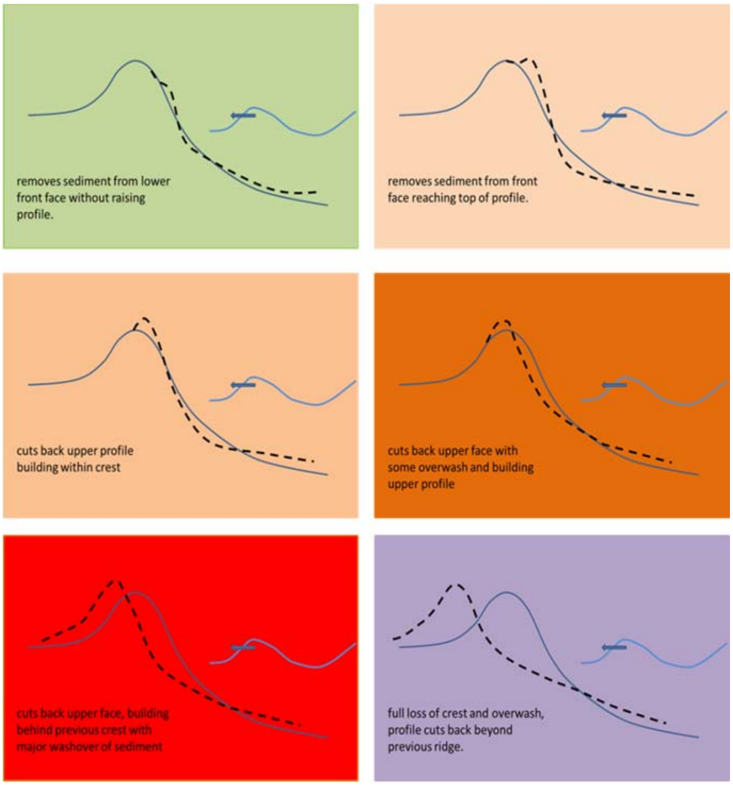


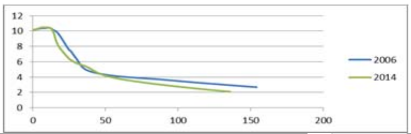
Figure 4.2 Example of Breakwat analysis and classification of response.

Summary

- Little change
- Change occurs within ridge
- Overwash towards failure
- Failure and collapse

4.1.1 Vulnerability Matrices (Breakwat)

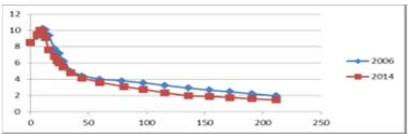
Profile 8a.



2006	Water levels						
Wave Ht.	2.65	3	3.5	3.8	4	4.2	4.5
1.5							
2							
2.5							
3							

2014	Water levels						
Wave Ht.	2.65	3	3.5	3.8	4	4.2	4.5
1.5							
2							
2.5							
3							

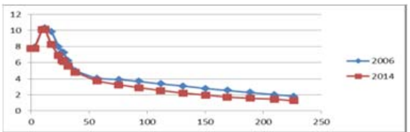
Profile 8



2006	water level						
wave ht	2.65	3	3.5	3.8	4	4.2	4.5
1.5							
2							
2.5							
3							

2014	water level						
wave ht	2.65	3	3.5	3.8	4	4.2	4.5
1.5							
2							
2.5							
3							

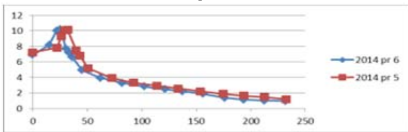
Profile 7a



2006	water level						
wave ht	2.65	3	3.5	3.8	4	4.2	4.5
1.5							
2							
2.5							
3							

2014	water level						
wave ht	2.65	3	3.5	3.8	4	4.2	4.5
1.5							
2							
2.5							
3							

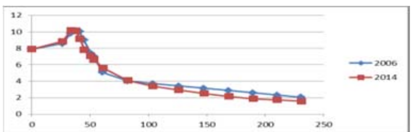
Profiles 5 and 6 (2014 similar to 2006)



PR5 2014	water level						
wave ht	2.65	3	3.5	3.8	4	4.2	4.5
1.5							
2							
2.5							
3							

PR6 2014	water level						
wave ht	2.65	3	3.5	3.8	4	4.2	4.5
1.5							
2							
2.5							
3							

Profile 3



2006	water level						
wave ht	2.65	3	3.5	3.8	4	4.2	4.5
1.5							
2							
2.5							
3							

2014	water level						
wave ht	2.65	3	3.5	3.8	4	4.2	4.5
1.5							
2							
2.5							
3							

Figure 4.3. Vulnerability matrices showing start of failure and collapse conditions.

The following conclusions may be drawn from the relative vulnerability matrices:

At profile 8a (Figure 4.4), it can be seen that there is a wide berm at the crest running back to the stream behind. Here, and as seen during the 2014 storms, the Breakwat model predicts that starting from the 2006 profile, with a water level of 3.8m ODN shingle is likely to be moved away from the front face and moved onto the crest. Analysis of the 2014 profile suggests that while the ridge, under similar storm conditions is slightly more vulnerable, sediment is still likely to remain within the existing ridge. While the profile, therefore becomes more vulnerable, even with increased water levels (i.e. more severe events or with sea level rise) there is still sufficient width (bulk) within the bank to resist full collapse.

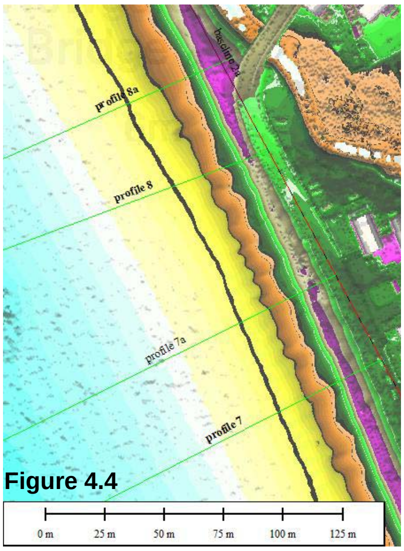


Figure 4.4

Profiles 7a and 8 (Figure 4.4) cover part of the most vulnerable section of the frontage. While the trigger water level at either section remains the same (comparing 2006 profile and that of 2014) at around a 3.5m ODN level, both sections have become more vulnerable, even with efforts to re-establish the crest. In the case of profile 8, the 2006 cross section could potentially withstand waves up to 2m on a water level of 3.8m ODN but starts to fail on higher water levels. With the narrower ridge, the resilience of the bank remains relatively similar but becomes significantly more vulnerable to even minor increase in water level above 3.8m ODN. Significant failure might be expected with waves of 2m on a 4m ODN water level and, indeed, greater damage may now occur with higher waves on water levels as low as 3.5m ODN.

The increase in vulnerability (comparing 2006 and 2014) is even clearer for profile 7a. With the present profile full failure might now be expected with wave heights greater than 2m on water levels above 3.8m ODN.

Typically these profiles might be considered to have a standard of failure at around a 1:1 year water level (Figure 4.1 – 3.5m ODN) with major failure occurring at around a 1:10 to 1:20 year water level. With sea level rise, the standard of failure might be expected to be exceeded more frequently that once a year, with total failure occurring possibly every year with a sea level rise of 0.25m.

Profiles 5 and 6 showed only minor change as a result of the 2014 storms, although to a degree this might be a result of the subsequent management activities, re-profiling the cross section. Only the 2014 profile has therefore been considered in Figure 4.3.

The position of these profiles is shown in Figure 4. 5.

Both profile 5 and 6 behave in a similar manner and despite apparently having been less affected by the 2014 storms show greater vulnerability to the start of overwash than either profile 7 or 8. This section of the frontage is therefore likely to be an area where more overwash of shingle occurs on less extreme events. It is seen from Figure 4.3, however, that complete failure of the bank in these areas occurs on similar conditions to those that cause failure further to the north. In effect, it can be concluded that at this section of the frontage the bank is less fragile. The bank may overwash on lower extremes but is only likely to fail completely on the upper extreme conditions.

Typically, the standard of initial failure is around the 3m ODN water level (a level that occurs several times a year). However, full failure might occur more probably around a 4m water level (i.e. the 1:50 to 1:100 year event).

Profile 3, shown on Figure 4.5, represents a more naturally developing section of the frontage. For this profile it may be seen from Figure 4.3 that significant overwash occurs at a water level of 3.5m ODN (with higher wave height) but that complete failure is not indicated to occur even under the most severe conditions tested.

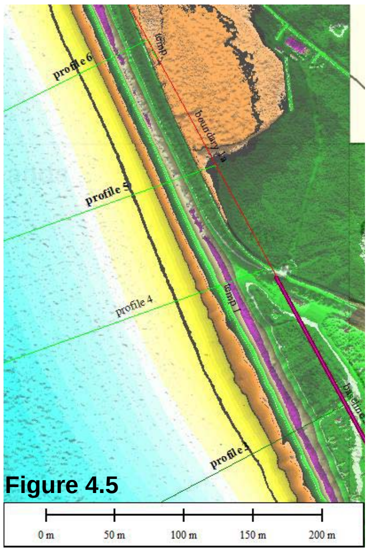


Figure 4.5

From the analysis, there is a suggestion that as the bank has changed and has rolled back, it has increased its resilience. It is suggested that the ability for the bank to adapt under storm conditions is apparent as it has moved into a new position where it is better able to resist overwash.

This frontage, from the profile shape, is not that obviously different from the managed profiles discussed above. However, in part because the profile is allowed to develop naturally and in part because the land levels behind the bank are higher, the profile is more resilient.

4.1.2 Bank Failure Assessment

In the above analysis it is recognised that there is a high degree of judgement applied to the results. While Breakwat does appear to give a realistic and good comparative assessment of bank behaviour, it cannot be considered as a rigorous absolute representation of the movement of shingle.

As a test of Breakwat an alternative form of analysis has been undertaken based on research undertaken by Bradbury (2000) and Obhrai (2008). This approach is similarly based on empirical analysis of actual shingle ridge behaviours. The approach provides a "Failure / No Failure" response to different wave and water level conditions, determining a threshold condition. The analysis was carried out for profiles 3, 6 and 8 and a comparison of results is shown in Figure 4.6 below.

It may be seen that the two approaches give very similar results. Breakwat does allow some further assessment of when overwash might be initiated (as shown in red). As discussed above:

- For profile 8 the fragility is highlighted with little change required to initiate failure.

- For profile 6 there is slightly greater resilience but ultimately failure occurs under similar conditions.
- Profile 3 shows greater overall resilience.

Profile 8

PR8 2006	water level						
wave ht	2.65	3	3.5	3.8	4	4.2	4.5
1.5	SAFE	SAFE	SAFE	SAFE	SAFE	SAFE	SAFE
2	SAFE	SAFE	SAFE	SAFE	SAFE	SAFE	SAFE
2.5	SAFE	SAFE	SAFE	SAFE	SAFE	SAFE	FAIL
3	SAFE	SAFE	SAFE	FAIL	FAIL	FAIL	FAIL

PR8 2014	water level						
wave ht	2.65	3	3.5	3.8	4	4.2	4.5
1.5	SAFE	SAFE	SAFE	SAFE	SAFE	SAFE	SAFE
2	SAFE	SAFE	SAFE	SAFE	SAFE	SAFE	FAIL
2.5	SAFE	SAFE	SAFE	FAIL	FAIL	FAIL	FAIL
3	SAFE	SAFE	FAIL	FAIL	FAIL	FAIL	FAIL

Profile 6

pr6 2014	water level						
wave ht	2.65	3	3.5	3.8	4	4.2	4.5
1.5	SAFE	SAFE	SAFE	SAFE	SAFE	SAFE	SAFE
2	SAFE	SAFE	SAFE	SAFE	SAFE	SAFE	SAFE
2.5	SAFE	SAFE	SAFE	SAFE	SAFE	FAIL	FAIL
3	SAFE	SAFE	SAFE	FAIL	FAIL	FAIL	FAIL

Profile 3

PR3 2006	water levels						
wave ht	2.65	3	3.5	3.8	4	4.2	4.5
1.5	SAFE	SAFE	SAFE	SAFE	SAFE	SAFE	SAFE
2	SAFE	SAFE	SAFE	SAFE	SAFE	SAFE	SAFE
2.5	SAFE	SAFE	SAFE	SAFE	SAFE	SAFE	SAFE
3	SAFE	SAFE	SAFE	SAFE	SAFE	SAFE	FAIL

PR3 2014	water levels						
wave ht	2.65	3	3.5	3.8	4	4.2	4.5
1.5	SAFE	SAFE	SAFE	SAFE	SAFE	SAFE	SAFE
2	SAFE	SAFE	SAFE	SAFE	SAFE	SAFE	SAFE
2.5	SAFE	SAFE	SAFE	SAFE	SAFE	SAFE	SAFE
3	SAFE	SAFE	SAFE	SAFE	SAFE	SAFE	FAIL

Key

Breakwat			Bradbury	
	overwash		FAILS	fails
	failure			

Figure 4.6 Comparison of Breakwat and Bradbury

4.1.3 Sensitivity to Wave Period

All the above analyses were undertaken based on a 9 second wave period. The opportunity was also taken to examine difference in failure thresholds based on an 11 second wave. This is shown in Table 4.1 for profile 8 and in Table 4.2 for profile 6. It may be seen that, in particular, a greater influence tends to be with respect to lower water levels. Effectively the greater wave length and power is able to compensate for the lower water levels.

Table 4.1. Comparison of failure threshold with wave period – Profile 8

wave period 9 sec							
PR8 2014	watwer level						
wave ht	2.65	3	3.5	3.8	4	4.2	4.5
1.5	SAFE	SAFE	SAFE	SAFE	SAFE	SAFE	SAFE
2	SAFE	SAFE	SAFE	SAFE	SAFE	SAFE	FAIL
2.5	SAFE	SAFE	SAFE	FAIL	FAIL	FAIL	FAIL
3	SAFE	SAFE	FAIL	FAIL	FAIL	FAIL	FAIL
wave period 11 sec							
PR8 2014	water level						
wave ht	2.65	3	3.5	3.8	4	4.2	4.5
1.5	SAFE	SAFE	SAFE	SAFE	SAFE	SAFE	SAFE
2	SAFE	SAFE	SAFE	SAFE	SAFE	SAFE	FAIL
2.5	SAFE	SAFE	SAFE	FAIL	FAIL	FAIL	FAIL
3	SAFE	FAIL	FAIL	FAIL	FAIL	FAIL	FAIL

Table 4.2. Comparison of failure threshold with wave period – Profile 6

wave period 9 sec							
PR6 2014	water level						
wave ht	2.65	3	3.5	3.8	4	4.2	4.5
1.5	SAFE	SAFE	SAFE	SAFE	SAFE	SAFE	SAFE
2	SAFE	SAFE	SAFE	SAFE	SAFE	SAFE	SAFE
2.5	SAFE	SAFE	SAFE	SAFE	SAFE	FAIL	FAIL
3	SAFE	SAFE	SAFE	FAIL	FAIL	FAIL	FAIL
wave period 11 sec							
PR6 2014	water level						
wave ht	2.65	3	3.5	3.8	4	4.2	4.5
1.5	SAFE	SAFE	SAFE	SAFE	SAFE	SAFE	SAFE
2	SAFE	SAFE	SAFE	SAFE	SAFE	SAFE	FAIL
2.5	SAFE	SAFE	SAFE	SAFE	FAIL	FAIL	FAIL
3	SAFE	SAFE	FAIL	FAIL	FAIL	FAIL	FAIL

4.2 Wave Overtopping.

Wave overtopping analysis has been undertaken using the Amazon model. The model ran a series of wave and water level conditions, matching those used for the vulnerability analysis. Based on the above, two profiles were considered – profiles 8 and 6.

Typically a one hour storm was run and average and peak overtopping rates were determined for two output positions on each profile. Position 1 was taken just below the crest of the bank and position 2 taken just beyond the crest, reflecting the actual overtopping that might occur.

Normally for flood risk studies the average overtopping rate is considered to determine flow over a defence. In this study, it is recognised that overtopping by individual waves may be significant, tending to move sediment and lowering the crest such that subsequent overtopping may increase with further damage occurring. As such peak flows have been considered.

The EurOtop manual refers to average flows that may be unsafe for trained staff, for vehicles and in terms of buildings. Typically the upper limits of average overtopping rates is quoted as being between 10 l/sec and 50 l/sec. A nominal value of 25l/sec has been taken as potentially resulting in initial damage and overwash of the shingle bank.

In Table 4.3 a comparison of critical overtopping rates is shown for the two profiles. Within each table the difference in overtopping due to a 9 sec and 11 sec wave period is considered and for profile 8 a comparison is drawn between the 2006 profile and that of 2014.

For reference the respective vulnerability diagrams from the Breakwat analysis (Figure 4.3) have been included in the table for further comparison.

It may be seen from the table that overtopping rates increase generally with wave period, in particular on higher water levels.

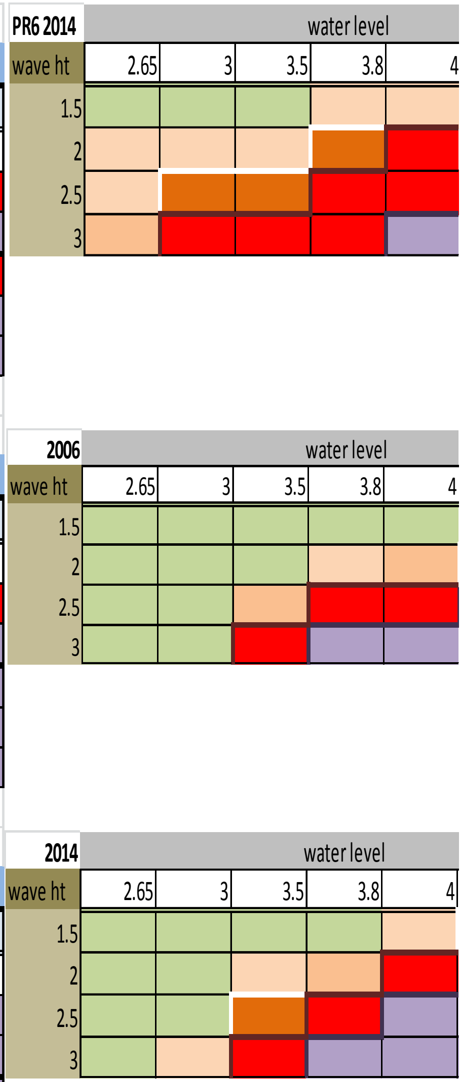
The assessment between bank failure and overtopping seems quite consistent over the higher water levels, with respect wave height. However, the table indicates that overtopping could cause loss of the crest on lower water levels due to overtopping.

Table 4.3. Comparison of overtopping rates

profile 6 - 2014 (litres/sec)					
PR6 2014	water levels				
wave ht	2.65	3	3.5	3.8	4
wave ht 2m (9sec)	0	0	0	0	4
wave ht 2.5m (9sec)	0	0	3	16	45
wave ht 3m (9sec)	22	19	66	180	303
wave ht 2m (11sec)	0	0	7	41	96
wave ht 2.5m (11sec	0	5	78	268	442
wave ht 3m (11sec)	26	98	447	758	988

profile 8 - 2006 (litres/ sec)					
PR8 2006	water levels				
wave ht	2.65	3	3.5	3.8	4
wave ht 2m (9sec)	0	0	0	0	0
wave ht 2.5m (9sec)	0	0	1	8	49
wave ht 3m (9sec)	182	201	245	371	498
wave ht 2m (11sec)	0	0	0	13	109
wave ht 2.5m (11sec	0	1	109	327	517
wave ht 3m (11sec)	137	264	595	888	1120

profile 8 - 2014 (litres/ sec)					
PR8 2014	water levels				
wave ht	2.65	3	3.5	3.8	4
wave ht 2m (9sec)	0	0	0	0	2
wave ht 2.5m (9sec)	0	0	2	60	172
wave ht 3m (9sec)	39	61	252	466	645
wave ht 2m (11sec)	0	0	4	140	319
wave ht 2.5m (11sec	0	8	293	627	842
wave ht 3m (11sec)	135	394	925	1290	1524



Comparison with Breakwat analysis

Of particular note, it is indicated for both profiles that significant overtopping can occur on higher wave height (3m) even on normal spring tides.

While potentially the crest level of the bank is the most critical feature in determining the onset of significant overtopping, the whole profile of the ridge and the level of the foreshore appears to determine the degree and severity of overtopping. In general, with reference to Profile 8; at lower water levels and larger wave heights, the steeper 2014 profile does reduce overtopping but at higher water levels the rate of overtopping is increased. This can be seen (Figure 4.6) also in relation to the marked increase in overtopping on the 2014 profile occurring at around the 3.8m ODN water level.

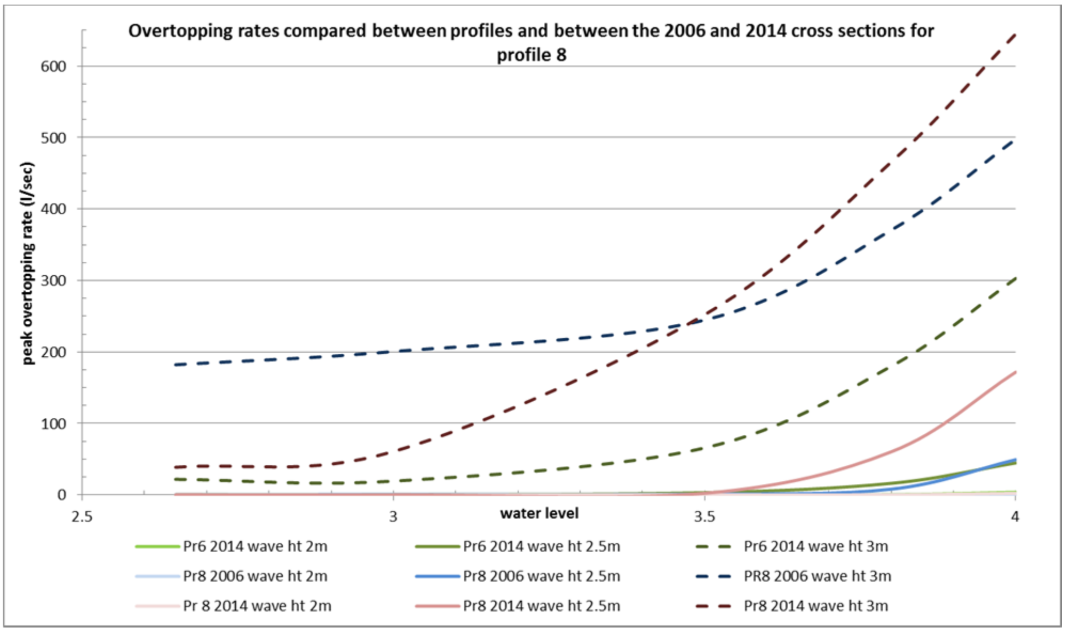


Figure 4.6. Comparison of overtopping rates

In summary, Profile 6 appears to have a standard of defence against significant overtopping of around a 1:10 year return period of water level, under more moderate wave conditions, although significant overtopping may occur at lower water levels with higher wave heights.

The standard of defence for typical wave conditions has changed (from 2006) significantly for profile 8 from around a 1:10 year return period water level to more typically a 1:1 year return period water level with the managed profile of 2014. It is also noted that the profile becomes significantly more vulnerable to overtopping as water levels increase.

5 CONCLUSIONS AND RECOMMENDATIONS.

5.1 Summary of Findings.

5.1.1 Geomorphology

The study has undertaken a brief review of the typical coastal processes and geomorphology of the frontage based on available survey data. This confirms generally the trend for movement back of the shingle ridge over the whole frontage, although locally different behaviours are seen with respect to separate sections of the coast.

Considering the unmanaged ridge to the south of the area the typical rate of retreat is in the order of 0.25m per year. This did increase due to the storms of 2014, with a potential retreat of some 3m during the storm period. However, it is noted that the average rate has to be seen as a combination of periods of little change and then rapid movement during more occasional storm events.

Along the northern part of frontage in front of the road, average retreat rates are in the order of 0.6m per year although this was primarily down to the storms of 2014. The frontage has been managed over the period between 2001 (the earliest survey) and 2014 and this, it has to be assumed has held the bank in a more forward position. The frontage, by being managed in this way, is therefore considered to be increasingly vulnerable to major storm events attempting to reset the ridge to its more natural alignment. The retreat, assumed to be due to the 2014 storm, measured at the 5m ODN contour, may have been as much as 6m in some locations.

In between these two sections, toward the southern area of the managed shingle ridge, significantly less retreat occurred over the storms. Over profiles 4, 5 and 6 there has been little long term retreat and, at profile 5, the 5m ODN contour seems to have actually accreted. There was however movement and overwash at the crest of this profile. It is suggested that during storms there may be changes in the net northern drift pattern such that this central section of the frontage possibly receives sediment from both the south and the north.

The longer term change that might be expected to occur with sea level rise is for the whole ridge and overlying coastal slope to move in land. This obviously depends on the resistance of the underlying coastal slope and its response to increased water levels.

With a lower beach slope of around 1:50, a rise in sea level of 0.15m (potentially over the next 20 years) would mean the shingle ridge would attempt to move back by some 7.5m, a rate of 0.38m/ year. With sea level rise over the next 50 years, which could be between 0.25m and 0.5m, the ridge would attempt to move back by some 12.5m (a rate of 0.25m/ year) to 25m (a rate of 0.5m/ year). These rates of change are not inconsistent with rates currently observed over the frontage. At profile 8, with a retreat of 7.5m, the crest of the ridge would be along the western edge of the road.

5.1.2 Vulnerability

Within the above context the study has examined the vulnerability of the ridge both over the managed sections and the natural section with respect to wave overtopping and shingle movement. The findings are summarised by different sections of the frontage below.

Northern managed section.
In terms of bank stability for the northern managed section of the frontage, the bank might be considered to have a standard of initial failure at around a 1:1 year water level (3.5m ODN) with major failure occurring at around a 1:10 to 1:20 year water level. With sea level rise, the standard of failure might be expected to be exceeded more frequently than once a year, with total failure occurring possibly every year assuming a sea level rise of 0.25m. The analysis of overtopping provides a similar picture, with significant overtopping likely to occur now with the 2014 profile at around a 1:1 year return period water level, with this standard possibly having reduced from around the 1:10 year return period of water level based on the 2006 profile.

It is also noted that the frontage has become more fragile with potentially less resilience between the event that might initiate overtopping being only marginally less than that which might cause substantial failure. This is highlighted in the vulnerability diagram in Figure 5.1.

2014	water level						
wave ht	2.65	3	3.5	3.8	4	4.2	4.5
1.5							
2					initial failure		
2.5					full failure		
3							

Figure 5.1. Vulnerability of the northern managed section.

Central managed section
Despite the lower retreat rates seen over this area, this section of the frontage is likely to be an area where more overwash of shingle occurs on less extreme events. It is seen from Figure 5.2, however, that complete failure of the bank in these areas occurs on similar conditions that cause failure further to the north.

PR6 2014	water level						
wave ht	2.65	3	3.5	3.8	4	4.2	4.5
1.5							
2							
2.5				initial failure			
3					full failure		

Figure 5.2. Vulnerability of the central managed section.

The standard of initial failure could occur around the 3m ODN water level (a level that occurs several times a year). However, full failure is more likely to occur around a 4m water level (i.e. the 1:50 to 1:100 year event). The standard of defence against significant overtopping is around a 1:10 year return period of water level, under more moderate wave conditions, although significant overtopping may occur at lower water levels with higher wave heights.

Southern unmanaged section
This is a more naturally developing section of the frontage. Figure 5.3 shows that overwash occurs at a water level of 3.5m ODN (with higher wave height) but that complete failure is not indicated to occur even under the most severe conditions tested.

2006	water level						
wave ht	2.65	3	3.5	3.8	4	4.2	4.5
1.5							
2							
2.5				Over wash			
3						initail failure	

Figure 5.3. Vulnerability of the southern unmanaged section.

It has also been noted that following the natural re-profiling and setting back of the bank by the 2014 storms, the bank in this area is more resilient against failure.

5.1.3 Management

At present the Council respond to overwash and failure of the bank by removing shingle from the front face and rebuilding the crest to a minimum crest height and width. This approach has been taken over at least the last 20 years.

The analysis concludes that while this approach does address the immediate problem and does restore the bank to a condition whereby it is capable of providing a defence against normal storm conditions, the standard and resilience of the shingle ridge is continuing to deteriorate.

Based on the natural section of coast, the typical rate of retreat is in the order of 0.25m/ year. In undertaking management to restore the bank to a pre-storm position, the bank has been unable to adapt and its position and shape (coupled to the fact that sediment has not been naturally sorted) makes the bank increasingly vulnerable.

It is concluded that:

- The work undertaken following the 2014 storms has restored the bank to a condition that might reasonably be expected to resist the typical annual storm conditions without major failure. However, above this level of storm, particularly over the northern managed frontage, the bank is increasingly likely to fail fully, with significant deposition of shingle in the road and potential increased flood risk.

- Depending obviously on the occurrence of storms in any particular year, it is considered that current management might still be capable of maintaining at least a 1:1 year standard of defence along the frontage over potentially the next ten years.

- Within twenty years, it would be anticipated that substantial bank failure would be occurring several times a year, irrespective of on-going management.

6 RECOMMENDATIONS

The study confirms that without significant strengthening of the shingle bank, there would be increasing pressure for roll back and increased occurrence of overtopping and failure of the bank.

It is considered that re-profiling the bank, to the currently used profiles is, at present sensible in maintaining a minimal standard of defence along the frontage. Even so there is an increased risk that a storm event of less severity than those experienced in 2014 would cause substantial bank failure. It is recommended that this management continues over the short term, while future land use management issues are addressed.

It is considered however that over quite possibly the next ten years this approach will no longer be sustainable.

It is, therefore, recommended that consideration is given to how the transport system will be affected in the event that the shingle bank is allowed to roll back and that plans are immediately put in place to address these issues.

It is further recommended that discussions are held with businesses, property owners and members of the community to raise awareness of the situation. As part of this, it is recommended that consideration is given to developing an adaptation plan. This should consider the potentially different ways in which the retreating shingle bank could be managed (in the absence of the road).

While not forming a specific recommendation, some improvement might be achieved, if only in delaying slightly the anticipated loss of the road, if some recycling of shingle were to be undertaken. It is only sensible for this to occur if it is in association with positive action being taken in addressing the longer term issues.

=0=0=0=