# Totland to Colwell Bay Landslide Assessment

December 2013

Isle of Wight Council

Mott MacDonald

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Mott MacDonald, Mott MacDonald House, 8-10 Sydenham Road, Croydon CR0 2EE, United Kingdom
**T** +44 (0)20 8774 2000 **F** +44 (0)20 8681 5706 **W** www.mottmac.com

Mott MacDonald

# Issue and revision record

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# Contents

# Executive Summary

Mott MacDonald has undertaken a study on behalf of the Isle of Wight Council to assess;

- the modes of failure of a recent landslide at Totland, on the north-west coast of the island;
- the risk of future failures within the landslide and the rest of the frontage;
- potential management options to reduce the risk of ground movement and;
- consider the potential scenarios for Funding through the Environment Agency's Partnership Funding procedures.

To carry out this assessment a desk study was compiled, a geomorphological survey undertaken, and a cost benefit appraisal was conducted to determine the potential suitability of management schemes and the funding of these.

This study has concluded that the failures have occurred along a basal failure plane at approximately-4m OD (the promenade level is +3m OD). The main cause is believed to be groundwater within the cliff following the above average precipitation levels for the winter of 2012.This increased pore pressures within the interbedded sand and clay cliffs, leading to a reduction in shear strength and the resultant failure below the sheet piles associated with the seawall.

From site walkovers in September and October 2013, Mott MacDonald has observed instability and in particular mudslides along the rest of the frontage that was studied, between the recent landslide and Totland Pier. This is attributed to the erosion of the toe due to overtopping waves and seepage lines associated with the Venus Beds above the How Ledge Limestone which can be traced throughout the cliff. There was also evidence of water run-out from the cliffs and bulging of the toe or slumping onto the concrete promenade behind the seawall - indicating high levels of instability within the rest of the cliff frontage and a correspondingly high risk of future failure, similar to the large scale event of the 2012 winter.

Mitigation methods would need to be implemented to reduce the risk of further failures along this section of coastline, which is a popular amenity connecting the two towns of Colwell and Totland for both local people and tourists. Mott MacDonald has suggested some mitigation measures that could be implemented to reduce the risk of landsliding along the frontage, and also stabilise the now failed section. Options include:

- A rock revetment to be installed along the toe of the failed section
- A rock revetment to be installed along the toe of the failed section and along the rest of the frontage to the Totland Pier
- A rock revetment to be installed along the toe of the failed section and a new upstand wall, to prevent scour by overtopping, built at the rear of the promenade along the rest of the frontage to Totland Pier.

To determine the economic viability of these options Mott MacDonald undertook an economic assessment to compare the costs of the schemes with the value of the properties that would be protected from cliff retreat (benefits). From these analyses it was concluded that there was a greater economic benefit if the stability of the whole of the frontage was improved compared to just mitigating the recent landslide. The most advantageous option would consist of a rock revetment to protect the toe of the recent landslide, a upstand wall along the rest of the frontage and drainage being installed along the whole cliff system. The Present Value Cost of this preferred option would be £2,100,000 (including 60% Optimism Bias). However, it might be possible to gain funding contributions from the Environment Agency worth around 34%, leading to required additional funding of about £1,394,000.

After this initial assessment Mott McDonald believes there is a reasonable case for the development of a mitigation scheme along this stretch of the Isle of Wight coastline. The first stage, following agreement with the Isle of Wight Council would be to apply to the Environment Agency for funding of a Project Appraisal report through the FRM7 process in order to then refine design options and benefits assessments, and consider other potential funding contributors in more detail and key environmental matters.

As part of this PAR process it is recommended that a further ground investigation is undertaken to gain a better understanding of the geotechnical properties of the soil, understand the hydrogeological conditions and confirm the stratigraphy and depth of the failure plane. These investigations will assist with the development of the options evaluation and a more accurate understanding of the costs of the scheme and the potential funding available.

# 1 Introduction

## 1.1 Terms of Reference

Along the coast of the Isle of Wight between Totland and Colwell there has been a significant coastal landslip which started on the 26th December 2012, and has continued moving since then. The Isle of Wight Council commissioned Mott MacDonald (hereafter referred to as MM) in September 2013 to undertake a technical assessment of the causes of the coastal landslide and develop a proposal for remediation options for the stretch of coastline between Totland and Colwell on the North West coast of the Isle of Wight. MM has produced an assessment of the condition and stability of the coastline and have proposed preferred management solutions.

The key objectives met in this report are to:

- Understand the reasons for the cliff failure;
- Identify the risks from coastal erosion and coastal slope stability and provide an understanding of the ground behaviour for the next 100 years;
- Develop a range of management options for the short to long term management of cliff stability and coastal protection;
- Ensure that the preferred options are technically, environmentally, socially and economically feasible; and
- Ensure where possible, opportunities for environment and economic enhancements are considered.

These objectives have been met through:

- **Assessment of the cliff failure** through the development of a desk study, site investigation and ground models (sections 1-5 in this report)
- **Options Appraisals** to assess the suitability of the proposed management options based on the assessment of failure and the site asset condition survey (Section 6 in this report).
- **Economic Appraisals** to calculate the benefits and costs of each scheme and determine the amount of partnership funding available (Section 6 in this report)

## 1.2 Scope of Report

This report combines the findings from the desk study, the condition assessment of the frontage between Totland and Colwell Bay and geomorphological mapping undertaken during September and October 2013 as well as using previous geotechnical investigations by Card Geotechnics (2003) to present failure mechanisms for the landslide event at Totland and to identify the risks of further ground movement over the next 100 years within the landslip and over the wider frontage area. From this assessment a range of management options have been assessed considering the protection of the whole frontage and just the landslide section itself. These options are deemed to be technically feasible, economically, environmentally and socially acceptable.

# 2 Site Description

## 2.1 Location

Totland and Colwell Bays are located on the north-west coast of the Isle of Wight. The site of the landslide is located below Fort Warden near Warden Point. The National Grid reference for the approximate centre of the site is SZ 3240 8770, Figure 2.1. A more detailed location map identifying the position of the landslide and site boundaries is presented on aerial imagery in Figure 2.2. Note this image does not show the failed landslide areas as the imagery is dated January 2004, before the landslip occurred.

Figure 2.1: Site Location


Source: Streetmap (2013)

## 2.2 The Landslip Problem

The seawall between Totland and Colwell was pushed up to 20m seaward due to a landslide that occurred on the cliffs behind the wall on the 26th December 2012. The movement caused numerous gaps in the seawall (Figure 2.3), allowing material to be subsequently eroded behind and beneath the wall, leading to further collapse and exacerbating the problem. The Public Right of Way footpath T34 between Totland and Colwell has been closed, which is an inconvenience to the local community and businesses that rely on the footpath for customers and tourists who use it regularly, especially in the summer months.

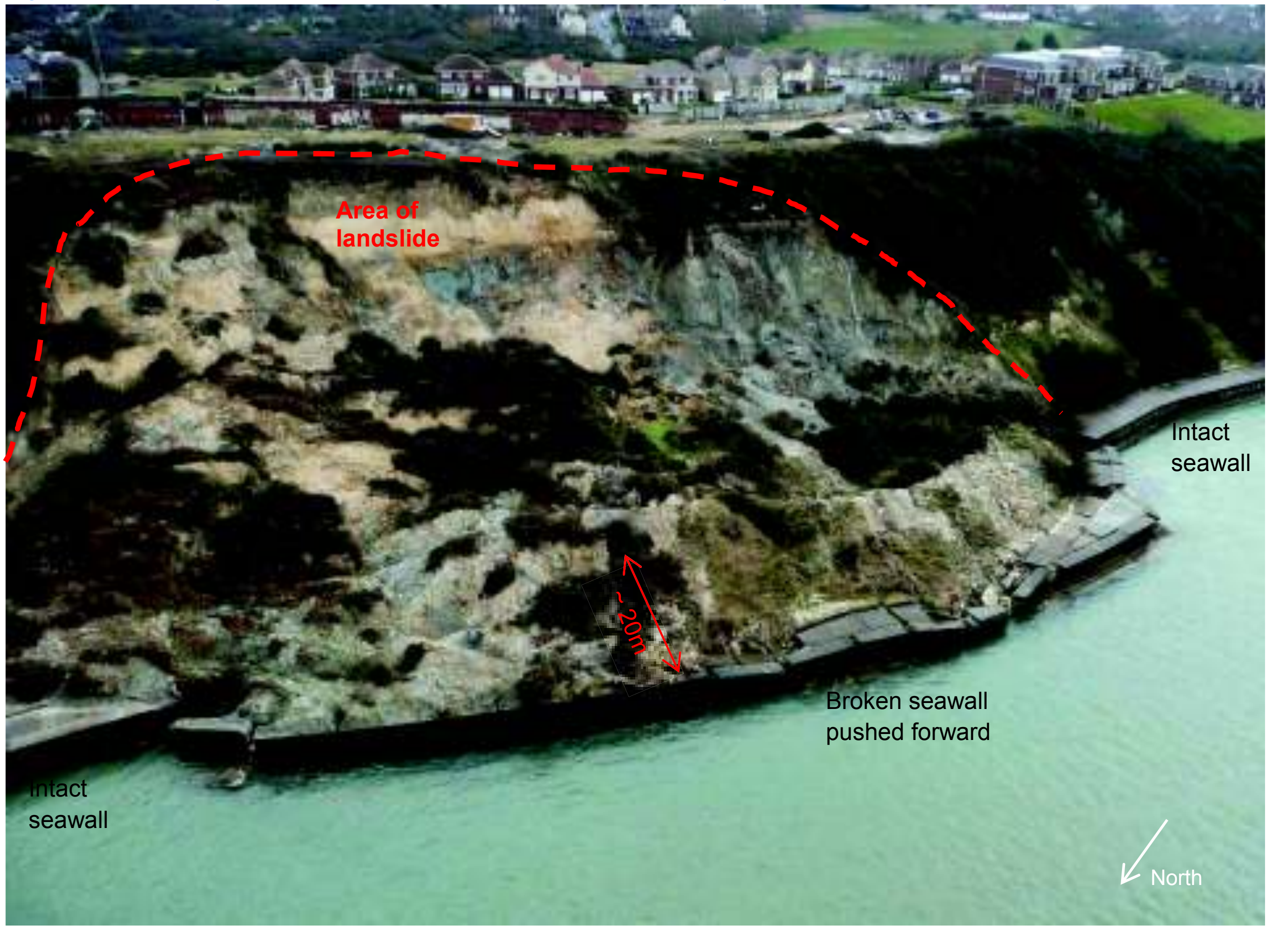
A laser scan cliff survey carried out by Channel Coast Observatory (CCO) highlights the change in volume of the cliff between January 2012 and January 2013 (Figure 2.4), from this image the true extent of the landslide is highlighted, as well as the areas where the material has slipped from the top of the cliff (red areas) and been deposited on the toe (blue areas).

Furthermore, as is presented in Figure 2.4, there is minor volume change along the frontage during 2012, highlighted by the red colour. This may be due to vegetation growth or be indicative of smaller scale preparatory landslides.

Figure 2.2: Aerial image with the key locations labelled.


Source: Google Earth Pro (Licenced) Image Dated January 2004.

Figure 2.3: Photograph of the landslide and the distance moved by the seawall.




Source: HM Coastguard, 2013

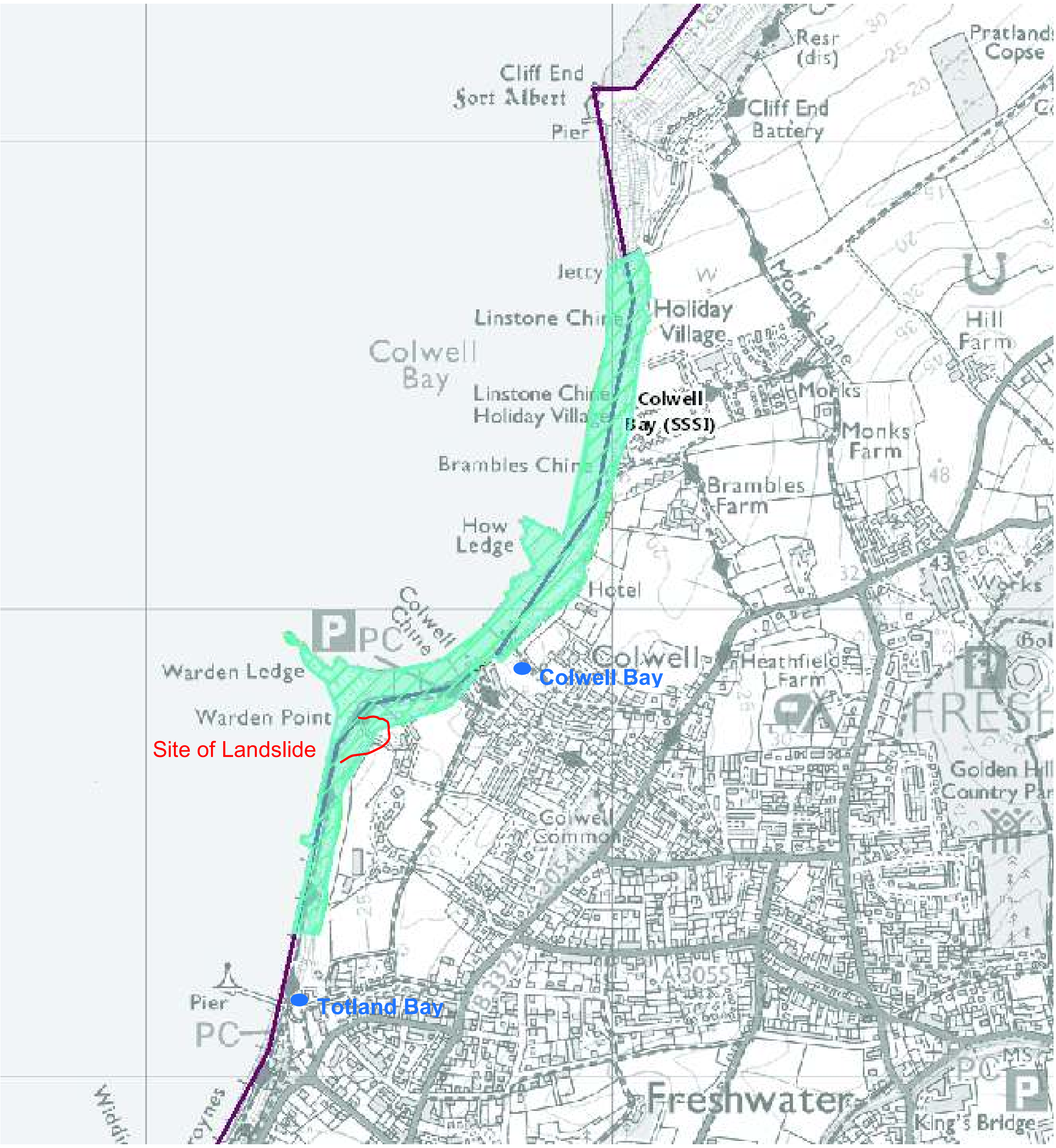
## 2.3 Land Designations

The cliffs backing the frontage were classified on 18th December 1995 within the Wildlife and Countryside Act (1981) designation as a Site of Specific Scientific Interest (SSSI) based on the geology (Figure 2.5). The site was classified due to the presence of the Headon Hill formation of which three members are designated stratotypes (international sections) based on the presence of flora and fauna: Colwell Bay Member, Linstone Chine Member, and Cliff End Member. The Venus Beds within the Colwell Bay Member are of particular reference because they contain abundant and diverse molluscan fauna of near full marine aspect. The overall rarity of well-preserved plant fossils from the lower Headon Hill formation is considered to be particularly important for interpretation of vegetation types and environmental conditions throughout the Hampshire Basin during the last Eocene. The marine environment is also designated as the Solent Maritime Special Area of Conservation (SAC) and the Solent and Southampton Water Special Protection Area (SPA) and Ramsar site.

Figure 2.4: The change in cliff volume between 2012 and January 2013. Red values indicate a loss in volume and blue indicate an increase


Source: Channel Coast Observatory (2013)

Figure 2.5: Extent of Colwell Bay SSSI




Source: Natural England (2013) http://www.sssi.naturalengland.org.uk/Special/sssi/sssi_details.cfm?sssi_id=1004379

## 2.4 Land Use

On the cliff top, directly above the landslip site is Fort Warden, which was originally a military fortification during the 19th and 20th Century. The remaining gun emplacement and Victorian brickwork loophole walls are now protected and are Grade II listed under the Planning (Listed Buildings and Conservation Areas) Act 1990 for their historic interest (English Heritage, 2013) (Figure 2.6). In 2003 development of the area around Fort Warden called the Fort Warden Estate commenced, through Roseberry Homes. There are currently 16 houses and 21 apartments on the cliff top with permission for 21 detached houses and 72 apartments (planning code TCP/17112R/P429/9).

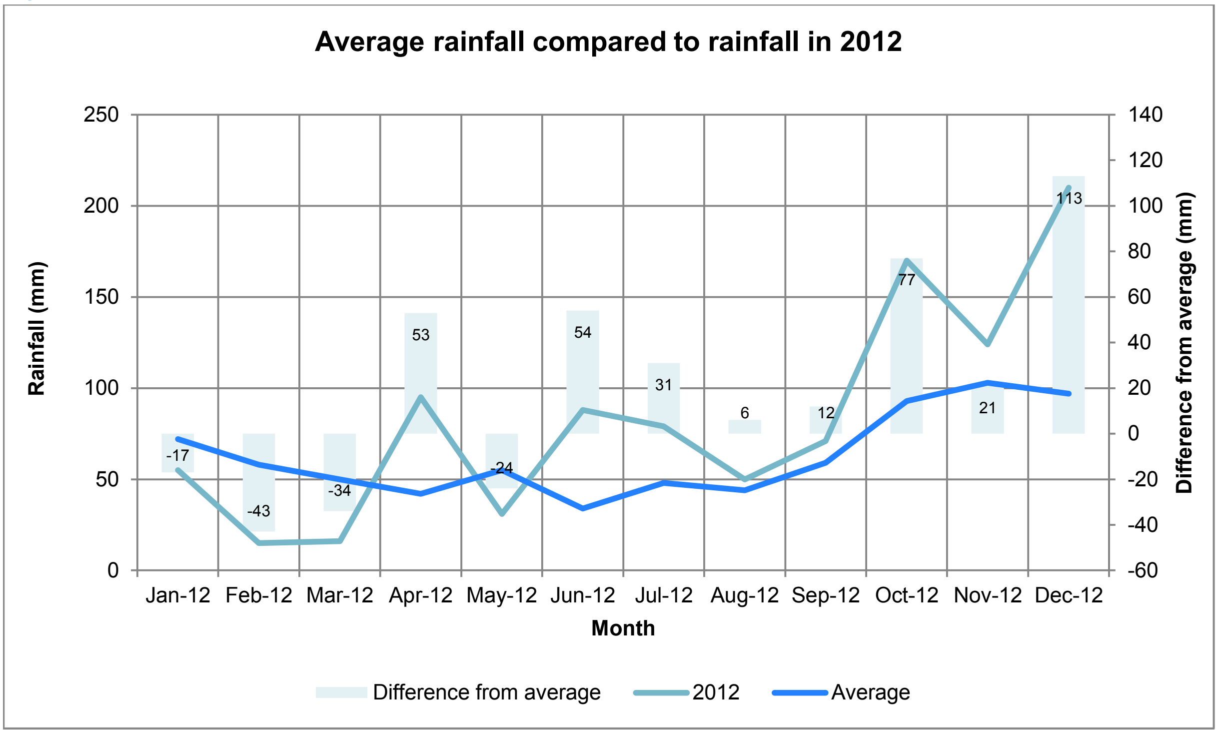
Figure 2.6: The Victorian Loophole Wall

Source: Photo courtesy of Fort Warden Heights, 2013 http://fortwardenheights.com/page3.htm URL accessed 25/11/13

## 2.5 Climate

The annual rainfall is less on the Isle of Wight than mainland UK; the annual average temperature is 13°c and the mean annual rainfall is 700mm (Wight Farm Holidays, 2013). However, the year of 2012 was a very wet year; the Met Office (2013) stated it was the 2nd wettest year on record for England. On the Isle of Wight the monthly rainfall was above average for the last 6 months of the year, particularly in the autumn months (Figure 2.7). This scenario is significant although further assessment of overall antecedent conditions would be recommended to understand the extent of this significance.

Figure 2.7: Rainfall in 2012




Source: Data obtained from the Isle of Wight Council to create this graph

There is evidence that the occurrence of landslide activity can be attributed to so-called ‘wet year’ sequences, i.e. those cumulative number of years with effective rainfall greater than the mean value (DEFRA, 2002). Recognised ‘wet year’ sequences date from 1874-1877, 1914-1917, 1927-1930, 1935-1937, 1958-1960, 1965-1970, 1979-1982 and 1993-1998 (Card Geotechnics Limited, 2003). Hence a large failure which was recorded between 1862 and 1898 may have been coincident with the wet year sequence of 1874-1877; but there is no historical evidence of failures between 1898 and 1908, coincident with no wet year sequence being recorded over this time (Card Geotechnics Limited, 2003). Furthermore several small slips occurring along the cliff edge in 1961 may be attributed to the wet year sequence 1958-1960 (Evans Grant, 1994). The most significant recent wet winter on the island was 2000/1, when there was a large amount of surficial movement recorded on the Island (Hutchinson and Bromhead, 2002).

## 2.6 Geology

The cliffs are formed from the Oligocene age Headon and Osbourne Beds which form the lower part of the Headon Hill Formation, which is part of the Solent Group (Insole et al, 1988); these beds are according to Hutchinson and Bromhead (2001) the most prone to failure. The Headon Hill Formation consists of thinly bedded marls and clays with subordinate bands of limestone and sand. Marker beds identified that the dip is between 1.2 and 1.5 degrees northwards with the effective dip at the coast being near horizontal (Card Geotechnics Limited, 2003).This gentle northwards dip promotes seepage erosion and landsliding (SCOPAC, 2004).

### 2.6.1 Previous Ground Investigations

Four previous Ground Investigations have been undertaken near the site (Table 2.1) as a result of installation of sea defences and residential developments on the cliff top.

Table 2.1: Previous ground investigations

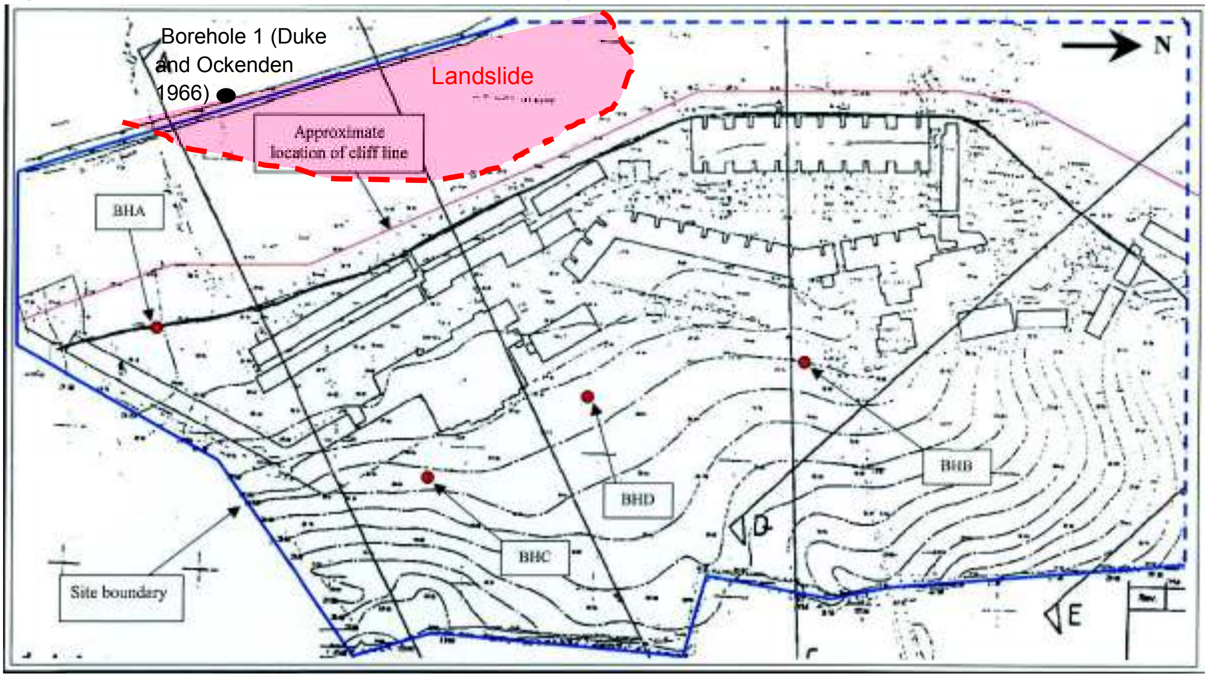
| Date | Type | Company | Client | Average Depth |
|---|---|---|---|---|
| June 1966 | Borehole (x3) | Duke and Ockenden Limited | Isle of Wight Council | 6m |
| April 1994 | Borehole (x5) | Evans Grant | - | 15-25m |
| September 2002 | Shallow Trial Pit | Card Geotechnics Limited | Roseberry Homes Limited | - |
| March 2003 | Cable Percussive Boreholes (x2) | Card Geotechnics Limited | Roseberry Homes Limited | 25-30m |

Source: Card Geotechnics Limited; Isle of Wight Council

The borehole logs from these investigations (Figure 2.8) have helped determine the geological strata (examined in greater detail below) but also provide ground condition information within the cliff. This is important to the investigation because it helps explain the methods of failure and determine where failure planes may be.

The borehole BHB drilled by Card Geotechnics in 2003 has been used to develop the ground models in the upper portions of the cliff as its closest to the landslide area. The 1966 borehole 1 by Duke and Ockenden has been used for the lower stratigraphy since the Card Geotechnics boreholes do not extend beyond the base of the cliff. The levels assumed for the beach level are taken from the 1966 Defence works drawings as being -0.03m. The location of the boreholes in relation to the landslide is presented in Figure 2.8.

Figure 2.8: The position of Boreholes undertaken by Card Geotechnics Limited




Source: Adapted image from Card Geotechnics Limited (2003) report

The geological features and typical engineering geological descriptions along with the description of the strata from boreholes extracted at Fort Warden by Card Geotechnics (2003) are presented in Table 2.2.

Table 2.2: Characteristic of the Geological units in stratigraphic order from the top of the cliff face

| Unit | Thickness | Geological Description | In-situ description |
|---|---|---|---|
| Linstone Chine Member (Upper Headon) | 1-3.3m | Medium dense light brown slightly silty fine to medium SAND | The Linstone Chine Member underlies superficial deposits at between 1m and 3.3m depth. The soils in the upper part comprise uniform, medium dense, slightly silty, orange-brown, fine to medium sand. A thin clay layer with a plasticity index of 13% is between 2 m and 2.2 m depth overlying silty fine sand. |
| Colwell Bay Member (Middle Headon) | 10.6m | Stiff Laminated blue-green mottled brown slightly sandy CLAY with thin subordinate dense blue grey silty sand layers. Marine and estuarine shell beds are present towards the base of the Colwell bay member (Venus and Neritina Beds) | The full thickness (10.6m) of the Colwell Bay Member has been identified at the site. Insole (1980), suggests that the lowermost 0.7 m, which is of a similar lithology to the Colwell Bay Member, does in fact belong to the underlying Totland Bay Member. Card Geotechnics Limited (2003) took the How Ledge Limestone as the upper boundary of the Totland Bay Member. The significance of the strata within this unit is that low permeability clay forms a cap and base (Neritina Beds) with confined water bearing silty sand strata (in particular the Venus Beds) between. In addition to this, the capping beds form an aquiclude to groundwater within the permeable sands of the overlying Linstone Chine Member.<br>As the sequence is traced downwards, stiff blue clay predominates; attaining a total thickness of 7.5 m and |

| Unit | Thick ness | Geological Description | In-situ description |
|---|---|---|---|
| | | | containing cyclically deposited sandy partings and minor beds of silty sand. The plasticity index of these beds varied between 18%-22% and 32%. At approximately 9.5 m depth, the Venus Beds are encountered and the strata changes to predominantly dark silty sands with occasional clay laminae and shelly bands with a plasticity index of 19%. The lowermost unit, the Neritina Beds, are found at 12.5 m and are composed of blue clay containing occasional shell fragments with a plasticity index of 36% and 46% clay fraction. The Neritina Beds grade into, and are interlaminated with, brown calcareous silt towards the base of the member and at 13.9 m there is a change in lithologies between the Colwell Bay Member and the underlying Totland Bay Member. |
| Totland Bay Member (Lower Headon) – including the How Ledge Limestone | 18.6m | Inter-bedded stiff laminated green-grey mottled brown slightly sandy CLAY and very dense homogenous light brown silty SAND. In addition the presence of a massive and strong limestone bed at the top of the sequence (How Ledge Limestone) and within the lower half of the sequence. | The Totland Bay Member is very similar to the overlying Colwell Bay Member containing repetitive sequences of water bearing sand and impermeable clay. The main difference between the units is the presence of two massive and strong limestone beds. The limestones are very resistant to shear failures and may form additional low permeability boundaries within the unit.<br>The uppermost unit within the Totland Bay Member is the How Ledge Limestone which can be seen as a small discontinuous scarp running across the western facing cliff line with a very low angle apparent dip to the north. The How Ledge Limestone is a massive sandy and shelly limestone which is marly (clayey) in its' upper part. At the site it has a comparatively uniform thickness of approximately 1.5 m.<br>Beneath the How Ledge Limestone, a series of clay and silty sand beds, each approximately 1 m to 2 m thick, were encountered. A thin, 0.5 m to 0.6 m thick, limestone bed was found, underneath which the interbedded clays and sands of the Totland Bay Member continue. |

Source: This is a consolidation of information from the Fort Warden, Totland, Isle of Wight - Report on Cliff Stability (page 18-21) by Card Geotechnics Limited, 2003.

The inter-bedded clay and sand strata and the presence of seepage lines, leads to changes in pore water pressure and resultant shear strength. This is believed to influence the cliffs stability and determine the resultant ground conditions.

## 2.7 Hydrogeology and Hydrology

The cliffs form a topographic high with a gentle slope towards the east, indicating that the main surface water flow would be expected to be west to east, away from the cliffs. From site observations by MM, it was noted that there was an unlined ditch on the northern side of Fort Warden, which was dry at the time of mapping but may represent a drainage pathway. Numerous broken clay pipes which are considered to be surface drains were also noted along the cliff frontage within the Linstone Chine and Colwell Bay Members. Often these were blocked and thus rendered ineffective but they highlight that surface drains have been directed towards the cliff face. It is considered that this drainage was installed in 1925 as part of the works done to prevent or reduce future cliff top instability (Lewis and Duvivier, 1973; Posford Duvivier, 1989, 1991, 1993; HR Wallingford, 1999).

Mott MacDonald identified on a walkover that on the scarp of the upper cliff face there were occasional seepage lines and small areas of standing water between the Venus Beds and the How Ledge limestone band often forming the head of mudslides within the recent landslide and also along the frontage studied. For further information from the Mott MacDonald walkover refer to Section 4.

Card Geotechnics (2003) made a similar observation noting seepage points 'particularly above the scarp of the limestone bands at the junction of the sandy and clayey beds.'

These observations were supported by the findings in the boreholes drilled as part of further ground investigation. Water was found to be under sub-artesian pressure at the base of the Linstone Chine Member at a depth of 1m and 2m, and a standing water level of 1.6m was recorded after a period of 24 hours (Card Geotechnics Limited, 2003). Further seepages were recorded in the Venus Beds at a depth of 7.5-8m, with the level of standing water recorded at 7.4m after a period of 24 hours (Card Geotechnics Limited, 2003). It is these seepage lines which are believed to be associated with the surficial failures observed. Finally water was recorded at a depth of 27.5m and 25m in the two boreholes, which was interpreted as being in continuity with the sea level (Card Geotechnics, 2003). In addition a study by Evans Grant (1994) found that ground water seepages at many depths at the junctions of clay and sand beds, however, there were no significant rises in water level over 20 minute standing periods. Therefore it could be argued that water may be a strong influencing factor on the cliff stability, due to erosion along seepage lines but also due to changing pore water pressure regimes.

## 2.8 Coastal Regime

The frontage between Totland and Colwell is exposed both to tidal currents and modified open sea waves, including swell. Maximum significant wave heights of up to 2.36m (Webber, 1969; Posford Duvivier, 1990, 2000; HR Wallingford, 1999) might occur at a 1 in 50 to 1 in 100 year frequency. Waves can propagate directly from the west on to the frontage, while currents run parallel to the shoreline approximately north-south.

Net longshore sediment supplies are low due to the presence of headlands between the bays, therefore both bays act as a small pocket beaches. It is thought no larger material is transported around the headland at Fort Warden from Totland Bay in to Colwell Bay. As the bays are relatively closed systems, they receive sediment inputs only from erosion of local cliffs. Much of the material yielded is too fine to remain on beaches and is transported seaward, where tidal currents may transport it south-westward of the Needles or north-eastwards into the Western Solent (SCOPAC, 2004).

# 3 Coastal Defences

### 3.1 Coastal Defence Development

The coastal defences from Totland Bay to Colwell Bay consist of a variety of seawall sections with rock and timber groynes along some sections. Parts of the seawall have additional rock armour along the toe to provide increased protection against scour along the most exposed sections of the seawall.

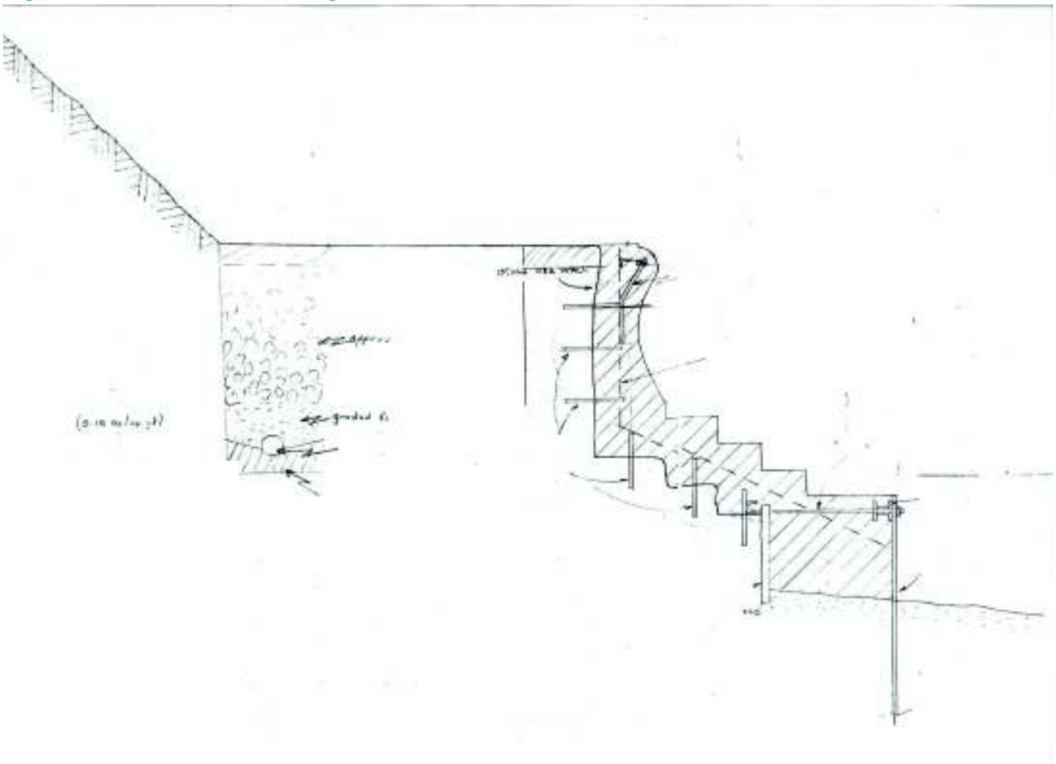
The first seawall at the site was installed in the early 20$^{th}$ century (date unknown). It was "approximately 13ft. high above beach level and comprises cyclopean stone blocks with a vertical face (with one step) 9"6' in height, surmounting a sloped steel apron, the toe of which is protected by a horizontal timber planking held by timber piles at 10ft. centres. Behind the seawall there is a 12ft. wide concrete surfaced promenade." (Report of County Surveyor- Coast Protection Works Scheme, 1966). However by 1966 "much of the stone apron [had] perished" and the wall was "being undermined in places" (Report of County Surveyor- Coast Protection Works Scheme, 1966).

New sea defences were installed in 1966 to account for landsliding/undermining. Along the section of the landslide new 13ft piles were "driven through three foot of beach material and four feet of compacted sand and gravel and three feet into the firm clay base" with the top 3ft above beach level; "a reinforced concrete encasement of the existing wall face" in approximately 5m sections was constructed; "a reinforced concrete protective toe and stepped apron" were added and it was decided to "fill all the cavities with concrete" (Report of County Surveyor- Coast Protection Works Scheme, 1966) (Figure 3.1). The rest of the frontage had the same design; however the piles were reduced to 10ft long.

Cliff drainage has been present at selected points since 1925 to prevent or reduce future cliff top instability (Lewis and Duvivier, 1973; Posford Duvivier, 1989, 1991, 1993; HR Wallingford, 1999).

In 1993 further remedial works were undertaken, and rock groynes to the south of the landslide area were constructed. In the area of the landslip, rock was placed along the toe of the seawall. It is assumed this was placed in order to prevent scour of the beach in front of the wall, which may be an indication that beach levels in 1993, were lower than those in 1966 (Appendix A).

Figure 3.1: Technical drawing of the seawall installed in1966.



Source: Reed, J., 1966, Isle of Wight Council

It was noted in the site walkover that there was a small wall (<50cm) along the rear of the promenade. This was not located across the whole frontage and construction details are unknown. This may have been a later addition to reduce scour from the overtopping waves, however, it was noted to be ineffective. Defence Condition and Failure Assessment

## 3.2 Defence and Condition Failure Assessment

The defence condition and failure assessment (presented in Appendix A) was undertaken by MM on the 20th September 2013. Defence condition surveys have been previously carried out for this area as part of the Isle of Wight Shoreline Management Plan 2 (Isle of Wight Council, 2010). The West Wight Coastal Defence Strategy Study is currently in development.

The SMP2 referenced the section of Totland Bay and Colwell Bay as section IW45. Along this section all defences were graded as either 2 “Good” or 3 “Fair”, with the exception of a section of seawall and groynes south of Totland Pier.

Table 3.2 below summarises the condition assessment and details for the defences at the location of the landslip (IW45/005):

Table 3.1: SMP Defence Condition and Residual Life at location of 2013 Landslip (Section IW45/005)

| Structure | Description | Condition Grade | Residual Life |
|---|---|---|---|
| Seawall | Seawall constructed in 1960's. Steel sheet pile toe with stepped concrete apron. Concrete wall with overhung coping at +3m ODN. | 2 "Good" | 15-25 Years |
| Seawall Toe (Rock) | Rock toe protection constructed in 1993 to exposed section of wall at the headland. | 2 "Good" | 15-25 Years |
| Rock Groynes | Shore perpendicular rock groyne constructed in 1993 | 2 "Good" | 15-25 Years |
| Timber Groynes | Shore perpendicular timber groynes (possibly constructed 1976) | 3 "Fair" | 8-12 ears |

Source: Isle of Wight Shoreline Management Plan 2 – Appendix C2: Defence Appraisal

The results of the assessment carried out by MM in September 2013 according to the Environment Agency's Condition Assessment Manual (Environment Agency, 2006), are presented in Table 3.2, with the detailed defence condition tables in Appendix A. The defence sections discussed are labelled on Figure 3.2.

Table 3.2: Results of the Conditions Assessment carried out in September 2013

| Defence section | SMP Policy | Defence Type | Condition Grade |
|---|---|---|---|
| Totland Bay | Hold the line | Concrete Seawall, Rock toe protection and Rock Groynes | Good |
| Warden Point Section 1 – Landslide Failure | Hold the Line | Concrete Seawall, Rock Toe Protection | Failed |
| Warden Point Section 2 – Non-Failure Section | Hold the Line | Concrete Seawall | Good |
| Colwell Bay | Hold the Line | Concrete Seawall, Rock Toe Protection, Timber Groyne | Good. Timber Groyne, fair. |

Figure 3.2: The location of the defence sections assessed in the Defence Condition Assessment


From the condition assessment work, it is assumed that the general condition of the seawall that failed was also in a good condition prior to the landslide event, and it would have been unlikely to demonstrate any visual signs of failure until movement started to occur. The wall in the failed section now offers limited protection to the cliff toe and significant gaps between the seawall units are present (Figure 3.3). In these gaps erosion of the material behind the seawall will occur, particularly during storms. However, the seawall encasement (in 5m sections) did not fail or crack during the movement, but it is unsure if the upper encasement was fixed to the toe as some units show signs of separation (at the wall and stepped apron interface).

Figure 3.3: A section of the seawall fronting the landslide in September 2013 when the defence condition survey was carried out.


Along the rest of the frontage, although the wall is intact, overtopping of the seawall is common. This has led to the scour of the slope behind the wall (Figure 3.4). This could lead to instabilities because material is removed from the toe, which alters the equilibrium of the slope, and leads to the steepening of the cliff. Therefore, the slopes are at a greater risk of failure.

Figure 3.4: Overtopping of the seawall and scour caused at the base of the cliff.

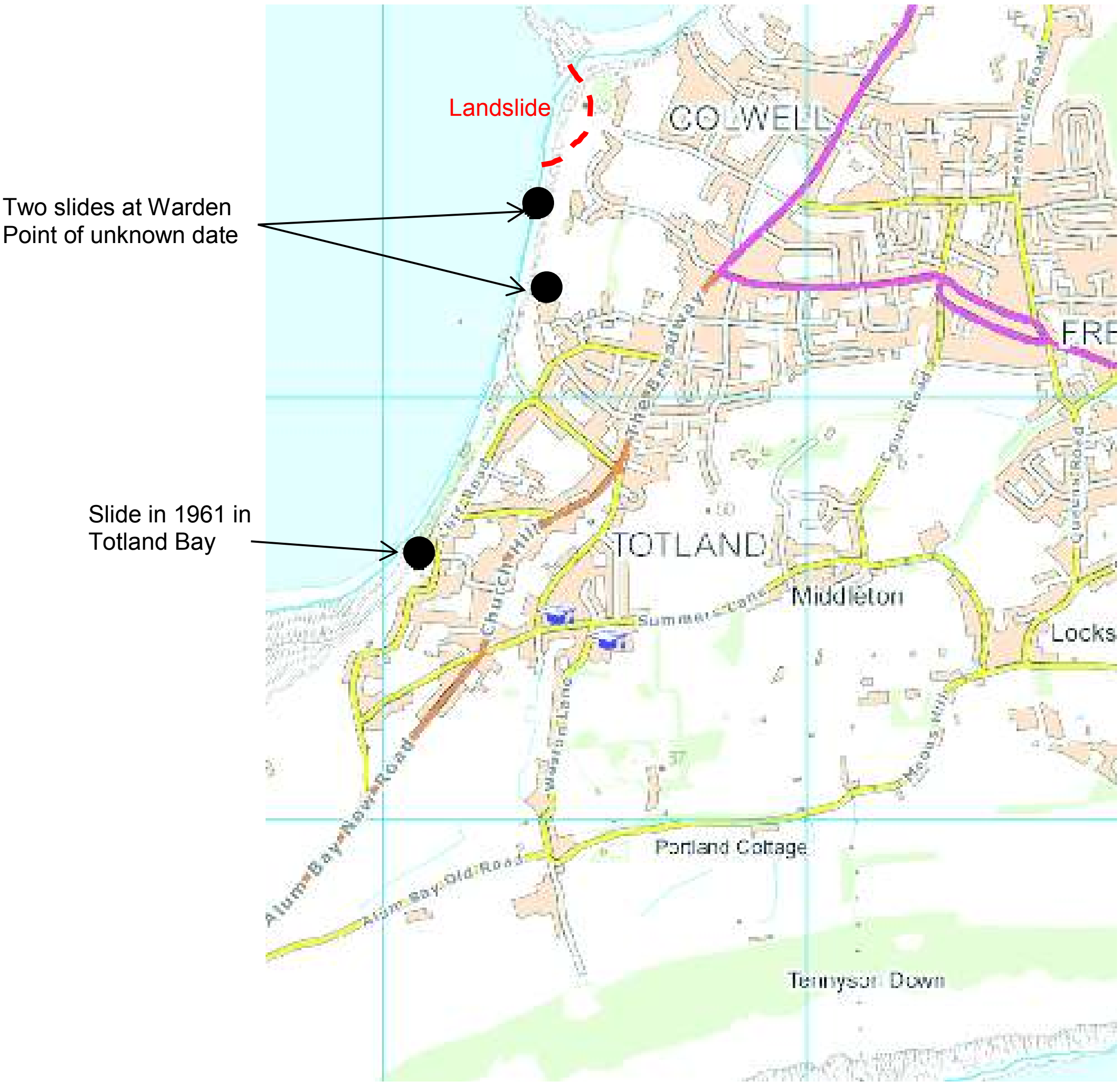
# 4 Coastal Instability

The shoreline between Totland and Colwell is comprised of 20-30m high cliffs formed of Oligocene and Eocene age material (refer to Section 2.6 for more information). The toe of the cliffs are protected from wave erosion by a 4m high seawall and promenade that runs the whole length of the frontage but cliff top recession is continuing owing to various forms of coastal instability along the majority of the frontage and scour of the cliff base behind the promenade as a result of wave overtopping. From the mapping exercises it was observed that failures occur along the whole section with some more recent than others.

## 4.1 Historical Failures between Totland and Colwell

There are few records of historical coastal instability within the area. The British Geological Survey (2013) has noted three past landslides (Figure 4.1). Two slides at Warden Point (unknown dates) and one in 1961 at South Totland Bay. The two slides at Warden Point most probably occurred before the first sea defences were installed in the early 20th Century (Isle of Wight Council, 2010), because the area was very susceptible to land sliding due to wave erosion at the base of the cliff (Isle of Wight History Centre, 2013). The slide in 1961 is described by Chandler (1991) 'an early seawall protecting a military installations at the top of the cliff was not maintained and became perforated by a combination of abrasion and corrosion, leading to the under-running of the wall.' The poor quality of the seawall led to the erosion of the toe of the slope and consequently a failure. As a consequence the seawalls were improved in 1966 (refer to Section 3.2). Further instability was noted by SCOPAC (2004) to continue, 'results in occasional extension of debris lobes across the esplanade e.g. winter of 2000/01' (SCOPAC, 2004). In addition recently there have been many smaller cases of slumping and water run-out onto the promenade (Isle of Wight Council, 2010) indicating instabilities and near surface ground movements are continuing.

Figure 4.1: Historic Failures within the area




Source: Map courtesy of BGS GeoIndex. http://mapapps2.bgs.ac.uk/geoindex/home.html?theme=hazards

## 4.2 Geomorphological Mapping

### 4.2.1 Methodology

A site walkover was undertaken of the landslide area on the 19th and 20th September 2013, and a further walkover of the rest of the frontage was undertaken on the 22nd and 23rd October 2013. During these walkovers transects of the cliff were completed to map the geomorphological and geological features including ridges, benches, mudslides, the slope angles and stratigraphy. Aerial photography dated 23rd June 2005 and 9th May 2008 from the Channel Coastal Observatory, and contour lines created from cliff laser scan profiles dated August 2013 were used as a basemap, at a scale of 1:500. The field data were incorporated into GIS and a digital map created. The geomorphological maps produced are presented in Appendix C.

### 4.2.2 Geomorphological Assessment of the landslide

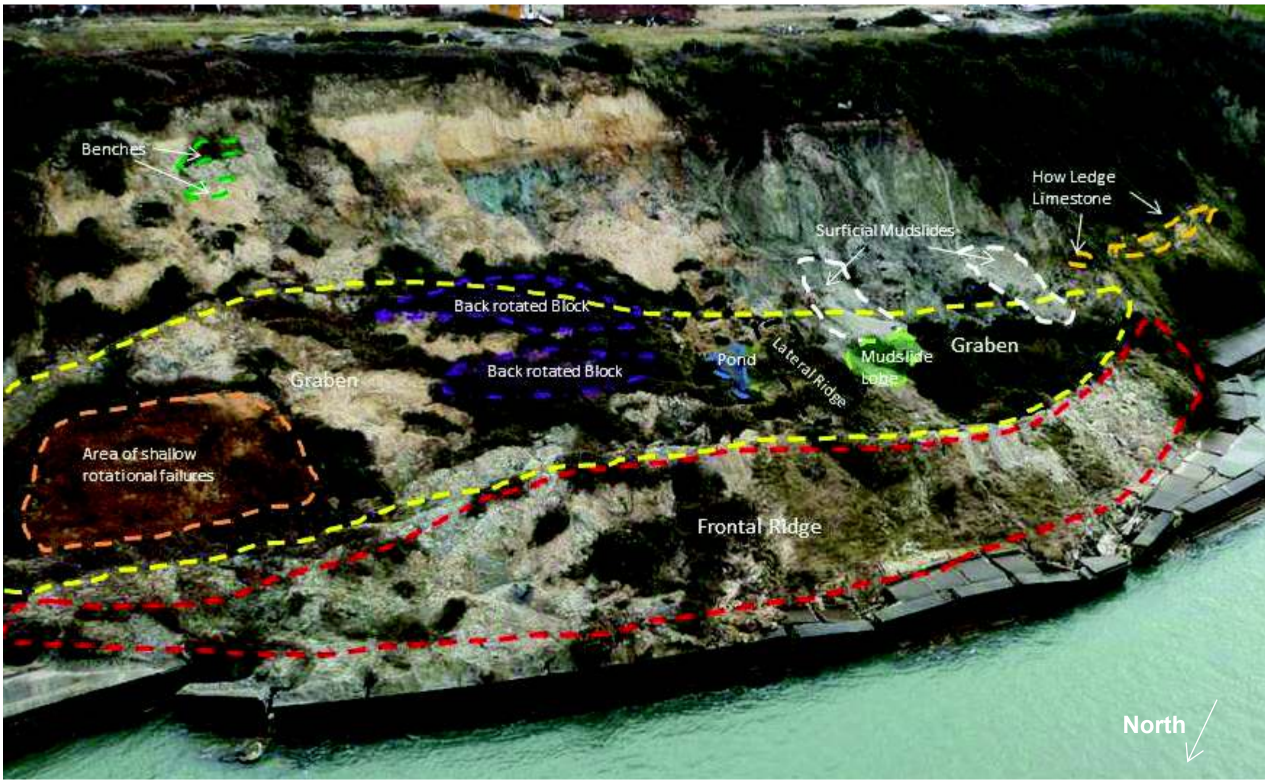
The geomorphological map of the landslide is presented in Appendix C as Figure referenced MM/328263/MNC/PCO/02/A. This highlights the main features observed on the landslide and provides the basis for the failure mechanism discussed in Section 4.2.4.

There is a distinct difference from one side of the landslide to the other; Section A is the northern most section whilst Section B is the southern portion of the slide (Figure 4.2). This difference is mainly due to the presence of numerous superficial mudslides in Section B. This is considered to be in part due to the geology; the lack of the Linstone Chine member in the southern section which is mostly a sand means there is less clay rich soil in the upper portion of the cliff; whilst it is in part due to the seepage points above and below the How Ledge Limestone. Section A also contains a few backtilted blocks, whereas Section B does not, indicating a rotational element to the slide in the upper portion in Section A.

The main features of the slide which characterise the slide are presented below and labelled on Figure 4.3

Figure 4.2: The distinction between Section A, containing greater amounts of sand within the strata and section B (red circle) with increased clay.

Figure 4.3: Some of the main features observed on the landslide. Image from the cliff top looking down over the landslide




### 4.2.2.1 Backscarp

The backscarp comprises a steep (>45 degree) bedrock scarp cliff. The apparent dip of the beds along the face can be seen and were measured at between sub-horizontal to 5 degrees towards the north. Localised sections in the upper Linstone Chine Member are subject to small slump (rotational failures) forming small benches on the scarp slope. Since the initial slide, the backscarp has regressed. In the 1 month time period between site walkover visits by MM, the fence posts at the top of the slide had moved and fallen down onto the backscarp slope indicating it is not at equilibrium and that regression is likely to be continuous. There was no vegetation covering due to the active nature of the backscarp, although it is considered likely that vegetation colonisation will begin next year and may help to stabilise the backscarp and upper slopes.

### 4.2.2.2 Debris / Talus accumulation zone at the foot of the backscarp.

The debris accumulation zone generally forms shallower angles than that the backscarp of between 22 and 32 degrees. It is not vegetated due to the continual supply of material fed from the regressing backscarp. In Section A this area is mostly comprised of sand and has few drainage channels. Conversely, in Section B the accumulation zone was mostly comprised of mudslide channels and lobes.

#### 4.2.2.3 The frontal ridge

The frontal ridge is a ridge (occasionally a rounded / truncated / flat topped ridge) which represents an accumulation of landslide material that built up behind the seawall. It is considered that this may be because the seawall prevented the landslide material from going out to sea and also due in part to a reduction in landslide velocity at this point. The seawall failed and was pushed forward by the landslide and this may have been a consequence of the additional stress of the frontal ridge on the seawall. This movement of the frontal ridge resulted in a zone of extension behind the frontal ridge creating the graben (Section 4.2.2.4 discusses the graben in more detail). The frontal ridge is comprised of a mixture of angular boulders and cobbles of the How Limestone and some calcareous sandstone, mixed with sand and clay. The ridge forms the front of the landslide and the centreline forms a crescent shape around the front of the landslide. In Section A the landslide material which forms the seaward side of the ridge covers the seawall up to 1.2m thick. This reduces in thickness towards the middle of the slide where it is only 0.3m thickness and is not present on top of the seawall and promenade within Section B. Slope angles vary from between 19 to 26 degrees.

#### 4.2.2.4 Graben area

The graben area represents an area of extension between the frontal ridge and the accumulation debris at the foot of the main backscarp. There is a pond within one section of the graben which is constrained due to two medial longitudinal ridges either side which are considered to be debris accumulation areas. The graben area is on average gently sloped seawards by approximately 10 degrees. In Section B the graben area is covered in mudslides and has ponded areas at the foot of the frontal ridge in some areas tension cracks are visible. In Section A the graben area is undulating and numerous tension cracks are present aligned perpendicular to the main movement direction, parallel to the frontal ridge crest alignment. In the northern flanks of Section A the graben is obscured due to superficial translational / rotational slides and a large mudslide occurring over the top of the graben area.

#### 4.2.2.5 Sub failures off the front of the frontal ridge

There are at least 2 main small landslides (it is likely more have since developed) which have occurred off the seaward side of the frontal ridge (Figure 4.4). These are considered to be due to over steepening of the slope and removal of toe support due to wave erosion because they have primarily occurred where the seawall is disintegrating the fastest.

Figure 4.4: Landslides of the frontal ridge due to removal of toe support by the waves. Photo taken looking SW from the frontal ridge of the landslide.


Source: Isle of Wight Council, 5th November 2013.

#### 4.2.2.6 Backtilted benches / rotated blocks

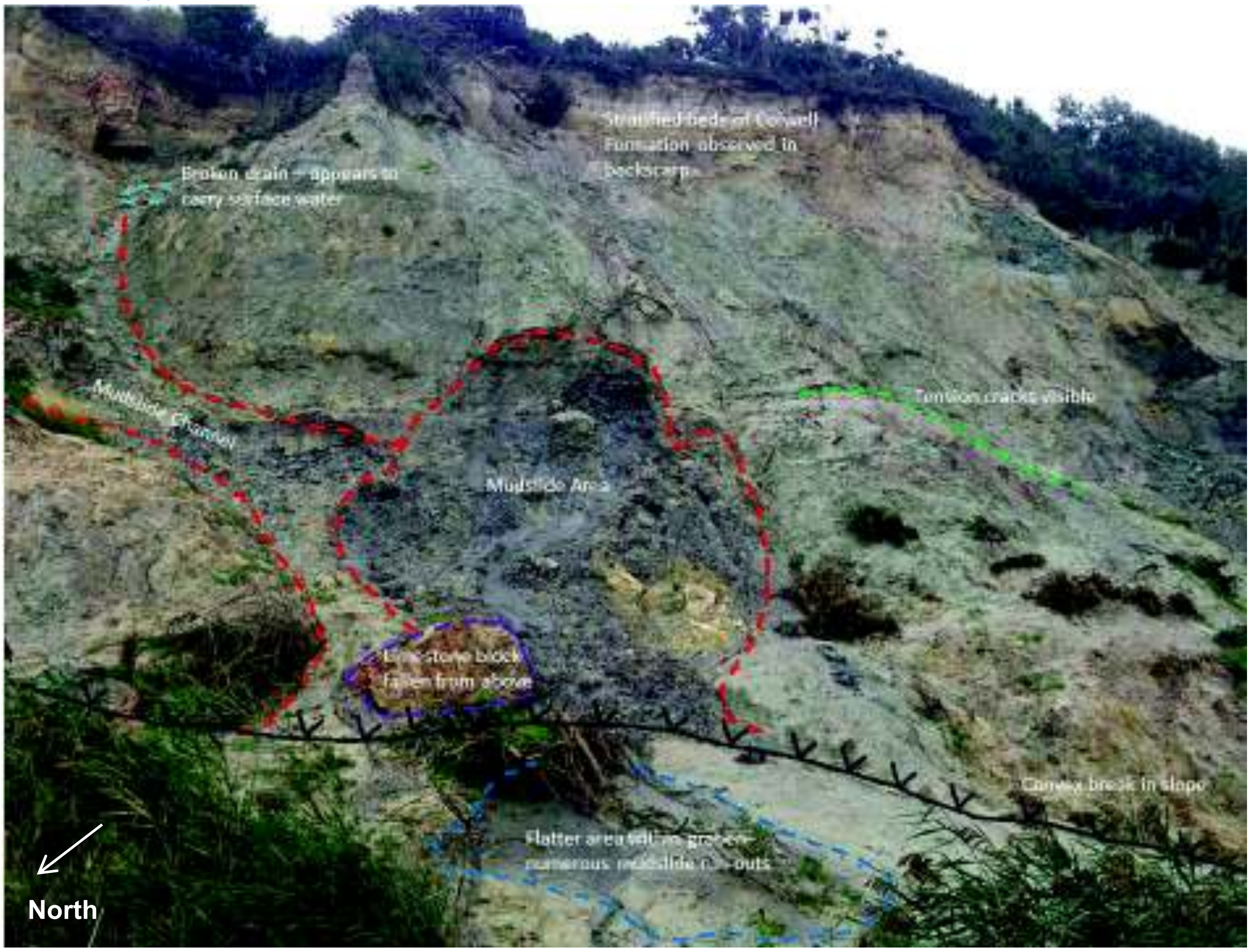
It is considered that some of the slopes were angled between 7 and 22 degrees towards the backscarp. These benches are approximately 5m wide by 30m long and were noted to be densely vegetated and the trees and shrubs were noted to be back tilted. This indicated that there is a rotational element at least to the upper portion of the landslide. They were only encountered in Section A.

#### 4.2.2.7 Superficial mudslides

There were numerous superficial mudslides encountered on the landslide (Figure 4.5). These were generally located within Section B, however, there were two mudslides noted in Section A. One mudslide delineates the northernmost flank of the landslide and probably initiated due to a small channel /stream running along the flanks here. The other mudslide in Section A covers the middle portion of the slide body and shows signs that it will continue to grow as there were a lot of tension cracks opening up around the head scarp area. The mudslides in Section B originated both above and below the How Ledge Limestone (this strata was not observed / was obscured by debris in Section A). Mudslide backscarps in Section B appear to be related to seepage points beneath / above this prominent limestone band within the clayey layers. The mudslides were active and quite deep. A supply of surface water exacerbating the mudslides was also noted from an old broken drain which forms small channel in the backscarp between the two sections of the landslide. Several mudslide lobes were noted within the graben area of Section B. They were identified due to differing colours and thicknesses on the smooth mud-filled ponding area and are considered to be related to different rain events since the slide has occurred. They are not considered to be deep mudslides, but are thought to be shallow features that have occurred following the main slide.

As is presented in Appendix D in the sketch cross sections / ground models, the landslide failure is a complex slide with both rotational and a translational elements. It is believed that because the seawall remained vertical and that there was no failure plane / ridge disruption observed in front of the seawall, that the lower portion of the slide is translational. The basal failure plane is considered to have developed along a clay layer which was identified by Card Geotechnics (2004) borehole. This translational slide led to the development of a graben feature at the base of a sub-rotational failure. It is believed it is this mechanism which has enabled the frontal ridge to move seaward by over 20m, yet allowing the seawall to remain vertical.

Figure 4.5: Recent Mudslides observed on the 22nd of October following a period of heavy rain. Image looking up at the Back scarp on the southern side of the landslide, within Section B.



#### 4.2.3 Geomorphological Assessment along the rest of the frontage

No large active deep seated landslides were observed along this section of the coast. However to the north of the main landslide a relict deep seated failure is considered to be present. From site visits it was also observed that the rest of the frontage between Totland and Colwell is susceptible to instabilities with relict landslides and more recent movements observed and mapped (Appendix C). However these were not on as large a scale as the landslide on 26th December 2012, but may indicate there is propensity for movement.

The backscarps of these failures originated both above and below the How Ledge limestone band that can be traced through the whole section. The How Ledge Limestone band is shown to rise gently in the cliff section from the north to the south and forms a prominent strong band, which appears more resistant to weathering and erosion than the clay rich layers both above and below the Totland and Colwell Beds respectively. The backscarps of mudslides tend to originate along the seepage/spring line along the Venus Beds creating surficial slides, similar to those in Section B of the main landslip.

The drains along this section are comprised of a geotextile around a flint gravel infill. These have generally been ineffective on this slope as they don't seem to extend to the problem area of the clayey Venus Beds. They are often disrupted or destroyed by localised mud flows. Slope angles on the slopes along the frontage were measured between 20-30 degrees. The cliff top behind the slopes has areas of pooling and depressions (see Appendix C). These may be local drainage channels or areas of tension crack development and therefore be precursors to a large scale failure Therefore the hazard of mudslides occurring along the rest of the frontage and onto the promenade is high and does actively occur; recent instabilities and failures are shown in Figure 4.6

Figure 4.6: The recent surficial instabilities within the section of frontage to the south of the landslide.

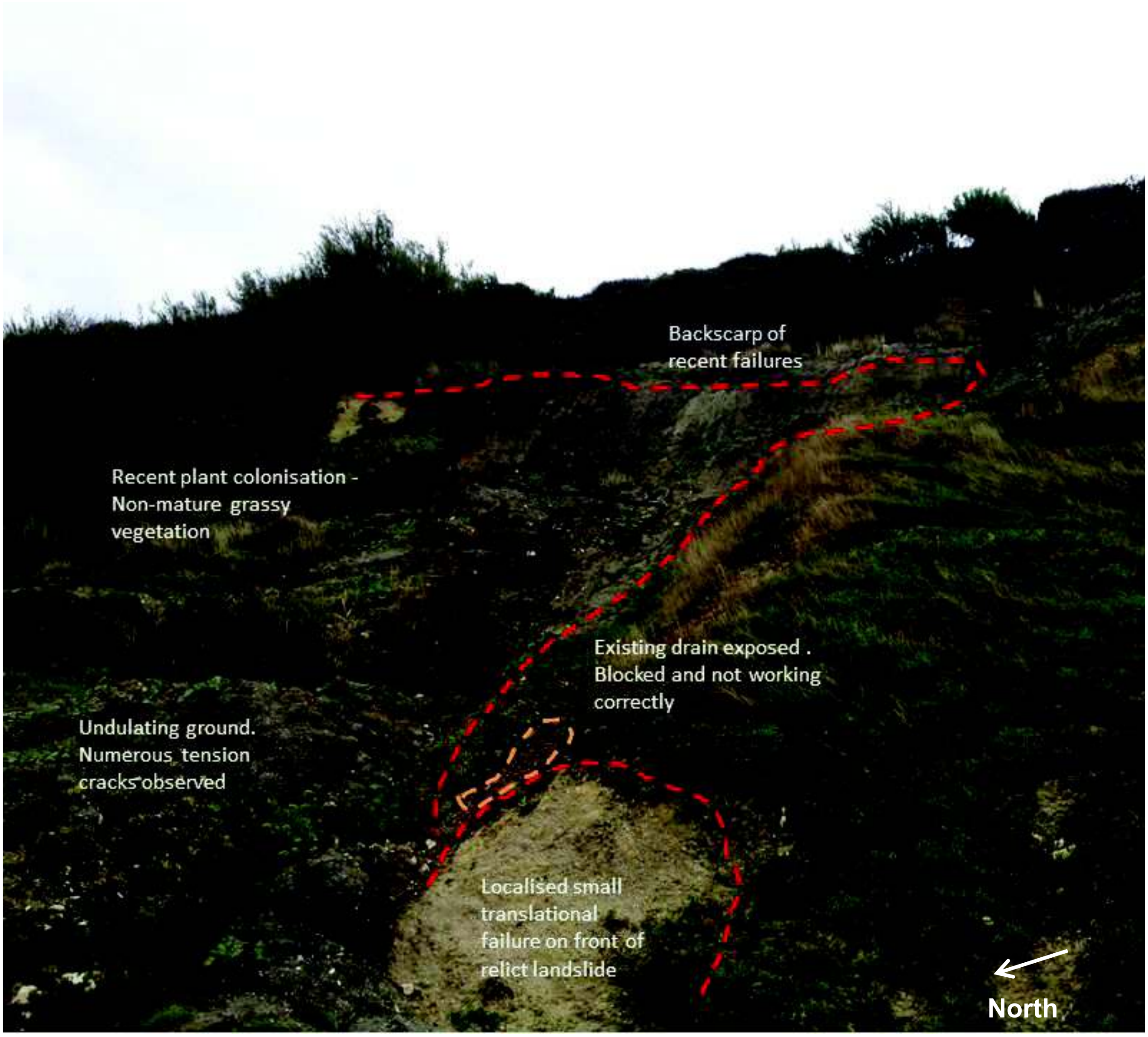


Figure 4.7: Toe of the landslide (seen in figure 4.6) overtopping the concrete promenade


### 4.2.4 Mechanisms of Instability

In formulating any remedial measures for coastal instability it is imperative to develop an understanding of the failure mechanisms Appendix D shows cross sections through Sections A and B which have been constructed based on the geomorphological mapping observations, ground investigation borehole data from Card Geotechnics (2003) reports and topographical data created from LiDAR images and laser scans of the cliff by Channel Coastal Observatory (2013).

With reference to Appendix D the postulated failure mechanism for the complex failure in Section A is summarised as follows:

- The main slip plane appears to be within the clay layers beneath the piles of the seawall. Identified in the 1966 Borehole 1. In the geotechnical investigation by Card Geotechnics (2004) it was noted that seepage was observed in the boreholes at 27.5m below cliff height, which, with the high rainfall in late 2012 may have increased the pore water and reduced the shear strength along these lower clay layers leading to the slide occurring. This has enabled the seawall to move forward. The piles have not been pulled from the seawall therefore indicating the seawall has retained its vertical structure with the failure plane passing below or close to the base of the piles.
- Backtilted blocks indicate a rotational element in the upper portion of the slide. It is believed the multiple rotational blocks have slipped as a result of the removal of material in front, reducing support and leading to a domino effect of multiple slips up the cliff face.
- The ridge at the base of the slide is caused by material building up at the toe, behind the seawall before it failed, as the energy for movement was reduced.
- A graben has developed behind the ridge and as a result of translational movement along the failure plane. This graben has continued to widen as the toe moves seawards due to the translational movement. Tension cracks are observed within the ridge indicating that this is still active.
- Continual erosion at the cliff base by marine processes will tend to steepen the overall cliff profile, increasing the probability of a further major slope failure. There is expected to be a critical threshold where continued recession of the cliff toe cannot pass without the whole cliff failing in response.

Section B has slightly different surface features than Section A, these are discussed below:

- The main failure is a complex failure with both rotational and translational elements. There were no multiple rotational blocks observed. The failure plane is considered to be along the same clay layer as in Section A. It was observed that the geology of this section contained more clay than Section A and there were seepage lines observed in the backscarp
- Owing to the increased amount of clay in the Colwell and Totland Members, it is considered that spring lines within this section formed at the horizons between the Venus beds and Neritina beds. These led to the development of the head scarp of shallow surficial mudslides. These mudslides were observed to have further developed in the month during the site visits.

The main cause of this failure is considered to be the increased groundwater related to the wet later part of 2012 along key stratigraphic horizons. An increase of pore pressure along the failure plane reduces the shear strength enough to be overcome by the shear force. In addition to ground water, the pooling of surface water and spring lines has led to the shallower surficial mudslides following the main landslide event.

# 5 Future Recession Model

## 5.1 Introduction

The overall cliff recession model is based on the magnitude/frequency of large events with the intervening erosion of the cliff considered too. It is required to provide an accurate and reliable estimate for the cost benefit analysis in Section 6. This section provides an assessment of existing recession rates provided by public and private document rates and then combine this with our findings to develop a cliff recession model in both the recent landslide area and the rest of the frontage.

## 5.2 Historical Rates from publically available documents

From an analysis of historical maps Card Geotechnics (2003) noted that the cliff top has receded approximately 10-14m during the period between 1862 and 1898, a further 5m from sometime between 1908-1971, and 4-5m between 1988-2003. The total maximum regression over the last 140 years is stated to be approximately 20-24m (Card Geotechnics, 2003), which equates to 0.17m/year; this is an average and does not reflect the mechanisms of failure from the recent Totland landslide where 20m was lost in one event. It should be noted the retreat rate of 0.17m/year is assuming that the toe of the cliff has been protected from coastal erosion since the late 19th century.

The SCOPAC 2004 report suggests the rate of retreat from different sources that may have been expected in this area if there was no seawall, ranging from 0.1-0.3m/year (May, 1966) to 0.56m year (Lewis and Duvivier, 1962), with an average retreat rate of 0.5m/year being assumed. Also this type of assessment does not take into consideration the mechanisms of coastal instability. The loss of land may be via one major landslide followed by a period of quiescence and relative stability, or conversely a small on-going loss every year due to smaller failures. The identification of coastal instability processes is therefore extremely important in assessing the average of such rates of cliff recession.

Sea levels are projected to rise by 0.75m in the next 100 years. This would tend to exacerbate the problems of coastal erosion and instability over time, due to a change in the equilibrium of the cliff and coastal processes, and will reduce the effectiveness of the wall in protecting against erosion of the toe of the cliff.

## 5.3 Landslide Area

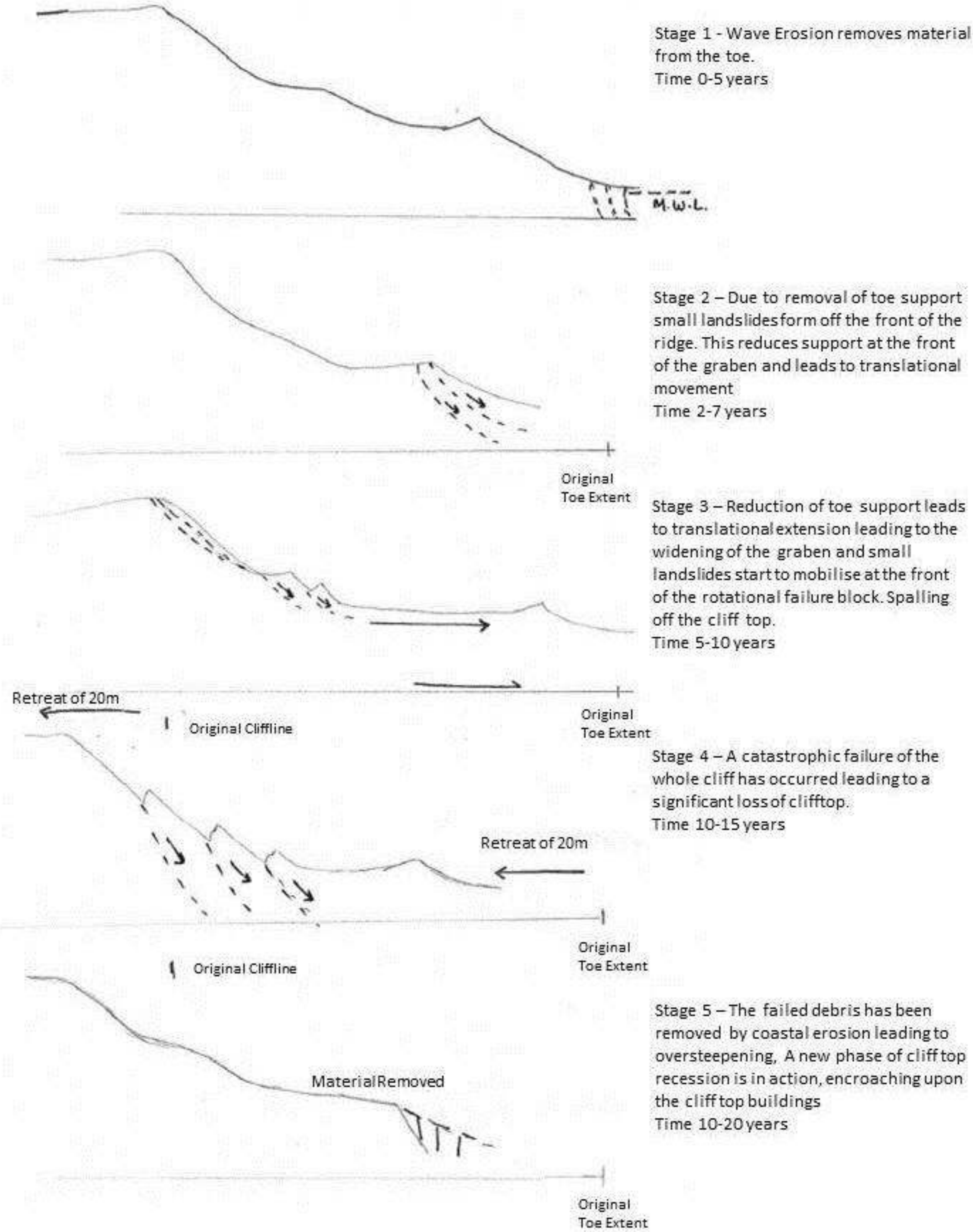
The area of the landslide is no longer stable. Since its failure, the equilibrium has become unbalanced (steeper scarp slope and toe has moved 20m seaward) so movement will continue to occur until a new equilibrium is reached. This equilibrium is constantly changing due to the erosion of the toe since the removal of the seawall, changes in ground and surface water regime, and strength conditions of the ground materials

The future recession of the cliff is understood by assuming a ‘Do Nothing Scenario’. This assumes that no management of the cliff is undertaken and the mode of recession will be as a result of natural retreat. The key aim of considering the ‘Do nothing’ scenario is to understand the potential changes to the coastline from the current and future coastal processes. This is linked to an understanding of historical erosion and sea level trends, in addition to other examples in similar coastal geomorphological settings and applying these to the protection afforded by the current condition of structures.

Under a Do Nothing scenario it is predicted that a large scale failure, like the one in late 2012, will occur on average every 10 years retreating the cliff line landward by ~20m in each event. However between major events there will be smaller movements and erosion in the landslide complex that will lead to this eventual large scale failure. A postulated model of failure (Figure 5.1) has been suggested based on four distinct stages. The ongoing erosion of the toe, caused by wave scour, will lead to destabilisation primarily near the toe, causing smaller mudslides and slips of the front of the slide. The removal of material will reduce toe support to the failed mass behind leading to the release of the translational extension failure. The removal of this large amount of material and reduced toe support will lead to a large scale rotational failure from the landward cliff, as the landslide tries to maintain its equilibrium, resulting in the landward retreat of the cliff to obtain a more stable less steep profile. This process will continue in a cyclic nature unless the toe is protected because the cliff will be in a constant state of disequilibrium.

The process described above puts the Grade II listed gun emplacements on top of the cliff at risk within the next 5-10 years, and if this process is not managed the existing houses which are about 70m from the cliff top will also be at risk within the next 50 years.

Figure 5.1: Postulated mode of retreat of the cliff of the landslide section if the toe remains unprotected



### 5.4 Rest of the section

A key concern for the Isle of Wight Council is to be able to determine whether a failure of similar magnitude and effect as the large landslip at Totland and the resultant processes associated with this failure (Section 5.3) may occur along the rest of the frontage. A relict large deep seated landslide north of the existing landslip area was encountered during the mapping. However, south of the landslide area there was no evidence that a large deep seated landslide had occurred, and only superficial mudslides were encountered.

There are a number of differences and similarities between the Totland landslip and the rest of the frontage to the south of the Totland landslip area, these are discussed below.

- The existing seawall is currently intact along the rest of the frontage, but not in the landslip area.
- The existing seawall along the frontage is not properly protecting the lower cliff face from scour. (>2m waves were observed overtopping the seawall) (Figure 5.3)
- The landslide area is on a headland and is subject to more wave action erosion than the rest of the frontage.
- The seawall construction drawings and text indicates that the seawall piles are deeper in the area of the landslide than the rest of the cliff frontage. Indicating that the anchorage reduces further south along the cliff section.
- The height of the cliff at the landslide area is 30m elevation (prior to failure) relative to an average of 20m elevation for the rest of the cliff face.
- The stratigraphy across the landslide area changes very slightly due to the regional dip of the beds. From our analysis of the duke
- Mode of failure this results in the main slip plane for the landslip being approximately at -4mOD elevation. Based on the stratigraphy within the borehole logs and out site observations the borehole 1 used for the lower landslide stratigraphy is from the 1966 investigation for the defence works. The borehole did not extend much further than the base of the piles. Therefore there may be other clay layers upon which the landslide has slipped upon. Further investigations would be required to confirm the depth of the slip plane(s).
- Using the marker beds and assuming similar distance between the marker beds and the clay layer main slip plane further south along the coast, indicates that the level of the clay layer in which the failure occurred may be reduced to 2mOD elevation. Therefore, if there is a failure within the same clay layer at the southern end of the cliff frontage, it may be assumed to be found within the seawall.
- The landslide area shows evidence that there were drainage measures put in place on the slopes. These are flint gravel trenches in geotextile wrapping. The same method for controlling drainage has been installed in the rest of the frontage. However, this is now considered ineffective as superficial mudslides often disrupt / destroy them and they do not extend to the source area of the Venus and Neritina beds, higher in the cliff face and so do not act drain the correct layers.
- Slope angles within the landslide vary considerably depending on the area being assessed. There are some sub-vertical cliff sections and some flat areas relating to the extensional graben. To the north of the landslide area slope angles are generally between 28 and 35 degrees, generally becoming shallower near the base of the cliff where not benched. The areas south of the landslide

area vary between approximately 34 and 27 degrees which may indicate that the area is susceptible to failure as the cliff face is oversteepened.

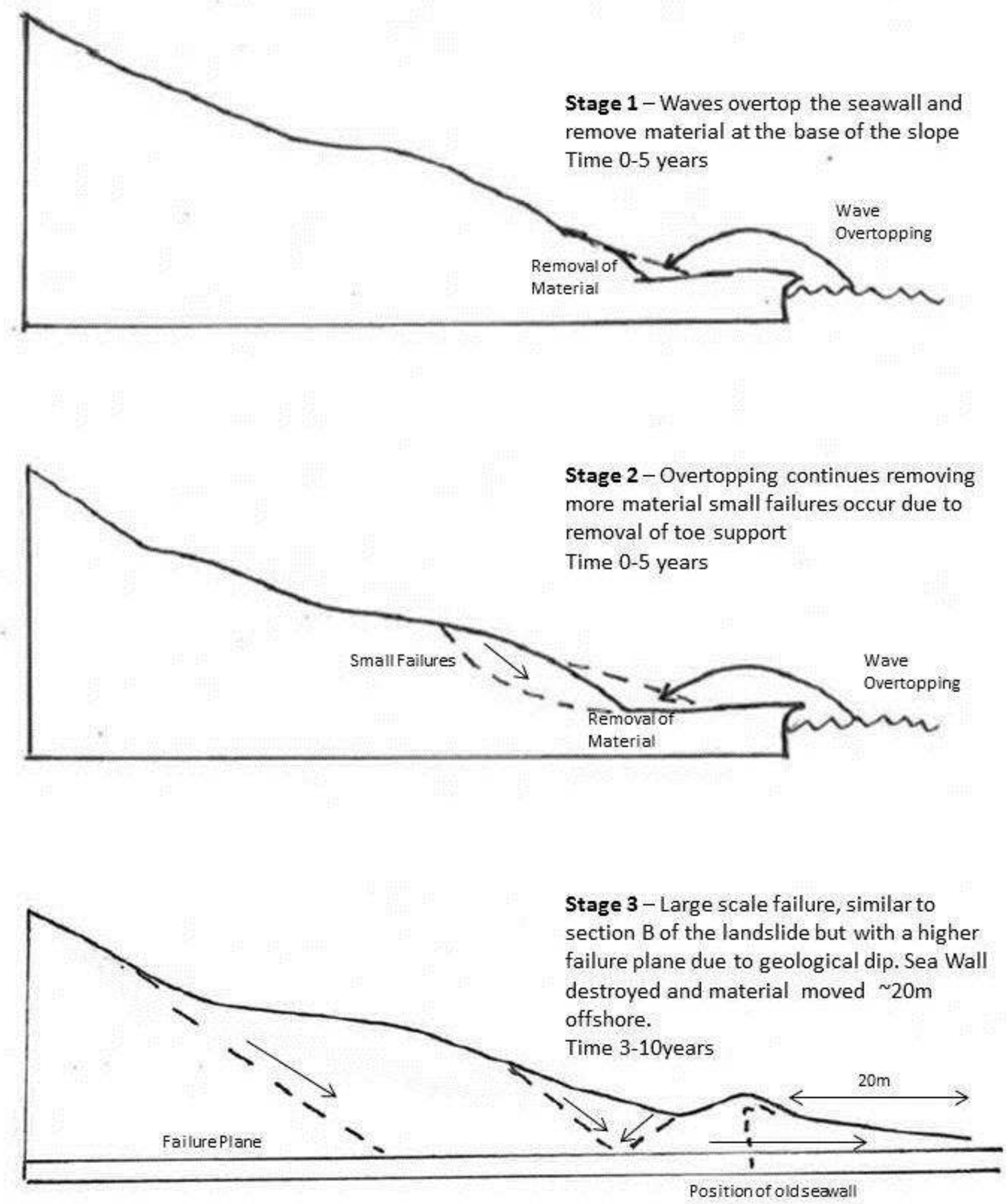
- Pooling water was observed in the cliff face in the walkover of the area south of the main landslide, and numerous seepage points associated with localised shallow mudslides.
- Gentle inflections and ponding water areas were also noted on the cliff top near the gun batteries, and also further south along the cliff top within a garden area. It is considered that these may be pre-cursors to tension cracks, which may be pre-cursors to a major slide.
- There are areas where the mudslides have built up at the toe (toe bulge) and in places this has split out over the concrete walkway behind the seawall (see Figure 4.8).
- Wave scour was observed causing further local instability behind the seawall due to overtopping at both the landslide area and the area to the south of the landslide (Figure 5.3). It is considered that this is a probable forcing mechanism that could trigger large scale failures.

Owing to the scour of cliff due to wave action overtopping the seawall, the mechanism of failure shown in Figure 5.4 can be driven. This will eventually lead to a large scale failure, similar in scale to the recent Totland landslide which caused the seawall to move forward 20m, if no intervention is put in place. Following this the mode of cliff retreat would follow the same model as the landslide (Figure 5.1). Objective assessment considers this could occur anywhere along the frontage between the recent landslide and Totland Pier under current conditions.

Figure 5.2: The run out of water in the section south of the landslide observed on 22nd October 2013. This photo is taken looking south towards Totland Pier.

Figure 5.3: Waves overtopping the seawall on 22nd October 2013. Photo taken along the frontage south of the Landslide, from the cliff looking NW.

Figure 5.4: Model of mode of potential failure and cliff retreat for the rest of the frontage under the current status.(no mitigation)




Following this the landslide will behave and retreat in the same way as the previously failed section (Figure 5.1)

# 6 Options

## 6.1 Scheme Options

### 6.1.1 Introduction

In Section 2.6 the mechanisms of cliff instability have been identified as:

- Increased pore pressure due to increased Ground water, reducing the shear strength along the basal failure plane;
- Oversteepened backscarp subject to continual regression;
- Section 5.6 notes that protection of the toe is limited due to the movement of the seawall caused by the landslide.  Erosion by waves and overtopping are able to erode the base of the landslide and remove material to sea.
- Overtopping water (during storms) may also cause erosion of the lower cliff face along sections within Totland Bay where the seawall has not currently failed.

To achieve a long term managed slope it will be necessary to address the mechanisms identified above. The failed section of the landslide needs protecting to reduce the removal of material by wave action, to prevent further movement and localised slips. For the rest of the frontage, between the landslide and Totland Pier, the main instability mechanism that needs preventing is wave overtopping causing erosion at the base of the cliff.

The possible measures which may be implemented to address the problems of erosion and slope instability will be considered in the following sections. Schemes will be suggested for the failed seawall and landslide area, and the whole frontage including Totland Bay.

### 6.1.2 Coast Protection

There are a variety of forms of coast protection measures which may be adopted. The decision as to the form of construction depends on a number of factors such as performance, cost and appearance.  The potential options and suitability are listed in Table 6.1.

Table 6.1: Potential Options and Suitability

| Option | Suitability for Landslide section | Suitability for remainder of the frontage (between the recent landslide and Totland Pier) |
|---|---|---|
| Concrete Seawall | Y | N (Economically Unviable) |
| Steel sheet Piles | Y | N (Technically Unviable) |
| Concrete Stepped Revetment | Y | Y |
| Concrete Revetment | Y | Y |
| Rock Revetment | Y | Y |
| Flood Wall | N (Technically Unviable) | Y |
| Gabions | N (Technically Unviable) | N (Technically Unviable) |
| Masonry Seawall | N (Technically Unviable) | N (Economically Unviable) |
| Timber Wall | N (Technically Unviable) | N (Technically Unviable) |
| Offshore Berm | N (Technically Unviable) | N (Economically Unviable) |
| Groynes | N (Technically Unviable) | N (Technically Unviable – groynes already present) |

#### 6.1.2.1 Landslide

For the landslide area, the main aim of the coast protection is to prevent the removal of material from direct wave action at the base, which would destabilise the material resulting in further landslide. Also wave overtopping should be reduced to prevent erosion of material which could destabilise the cliff at higher levels. Any coastal protection structure may need to be flexible to accommodate some movement as a result of the recent landslide activity without suffering significant damage.

Concrete structures would be expensive and require extensive ground works. They are also not flexible, so wouldn't allow for small movements associated with the instabilities in the cliff and therefore may break down and become less effective with only small movements. Steel sheet piles would also be expensive; they would have to be deep to stabilise the failure but also extend up and be filled in behind to stabilise the rest of the cliff.

Therefore, a rock revetment is deemed to be the most suitable option. The revetment structure would prevent wave action removing material from the landslide toe and the rough surface and slope within the design will dissipate wave energy reducing overtopping and preventing significant erosion of the cliff / landslide at higher levels. The rocks themselves will provide a super incumbent weight at the toe; toe weight is important in reducing further movement. Rock protection would not disturb the failure plane, therefore reducing the risk of further failure. Finally, this option is more aesthetically suitable for the shoreline which already has rock groynes and is also a cheaper option compared to non-porous concrete defences.

#### 6.1.2.2 Rest of the frontage

Along the rest of the frontage there is already a sea wall which does help protect the base of the cliff from erosion. However this is regularly overtopped and a large amount of scour and pooling of water on the cliff behind has been observed. Therefore one of the main aims of the coastal protection is to reduce the amount of overtopping. Two options are suitable:

1. Rock revetment, for similar reasons to the landslide area, the rock would provide extra toe weight which may prevent the existing sea wall failing. This is of particular importance because the piles in this section are shorter than in the landslide section, so could be more likely to be damaged under a smaller failure. Also would reduce overtopping on to the cliff face.
2. Upstand Wall, either on top of the current sea wall, or on the cliff side of the promenade. This will prevent the overtopping waves from causing scour at the base of the cliff. However, this option is less aesthetically pleasing for this relatively undeveloped area.

### 6.1.3 Slope Stabilisation

Coast protection methods only protect the toe of the landslide, so there may be further movements within the cliff face as it aims to achieve equilibrium, still resulting in cliff top retreat and producing ongoing pressure on the protected toe potentially driving subsequent movement. Therefore, slope stabilisation is a further method that will help stabilise the cliff. Various methods may be adopted to improve the stability of an unstable slope. In this particular situation the presence of excess ground and surface water within the cliff face has led to changes in pore water pressure, which reduces the shear strength and as a result causes primarily shallow failures but also the larger failure that occurred in late 2012. There a variety of options that could be undertaken. Their suitability for a scheme at Totland is assessed in Table 6.2

Table 6.2: Potential options and suitability

| Option | Suitability for Landslide Section | Suitability for the remainder of the frontage from the recent landslide to Totland Pier |
|---|---:|---:|
| Dig out the failure and regrade the slope | Y | N (Economically Unviable) |
| Piled ground improvements | N (Economically Unviable) | N (Economically Unviable) |
| Drainage | Y | Y |

Therefore, the most appropriate solution is considered to be drainage works. Shallow drainage can be obtained by establishing a shallow open channel filled with gravel to wick water from the shallow surface layer and allow it to drain from the slope while preventing the fine grained soils being washed out and reducing seepage erosion. The drain will consist of a geotextile filter fabric laid beneath a protective granular layer, which will filter the water off the cliff face. Deeper drainage systems could be installed e.g. counterfort drains, adits and ejection wells. These reduce pore water pressures and lower ground water levels helping to maintain the shear strength of the rock and reduce the risk of failure. With a solution based on drainage measures of this nature it is also necessary to monitor and maintain the drains to ensure their continued performance.

In addition to drainage slope profiling could also be viable. Following this the slopes could be left to degrade naturally to a stable long term profile. The advantages of this approach are that construction costs would be reduced and the final slope would appear more natural and therefore more sustainable. However there are also possible disadvantages associated with this approach. The cliffline will be required to move landwards to achieve a stable profile. It is calculated that for a 30m high cliff the cliffline, assuming a 25˚ slope to be stable would have to be 70-75m from the toe; this would lose the protected gun emplacements and could be perceived to be moving the cliff line risk towards the housing. In adopting this approach further analysis would be required to determine the stable slope and ground profiles assuming the toe is stable and protected. Monitoring and maintenance works would still be required to address any issues arising after intense storm events to ensure the stable profile is maintained.

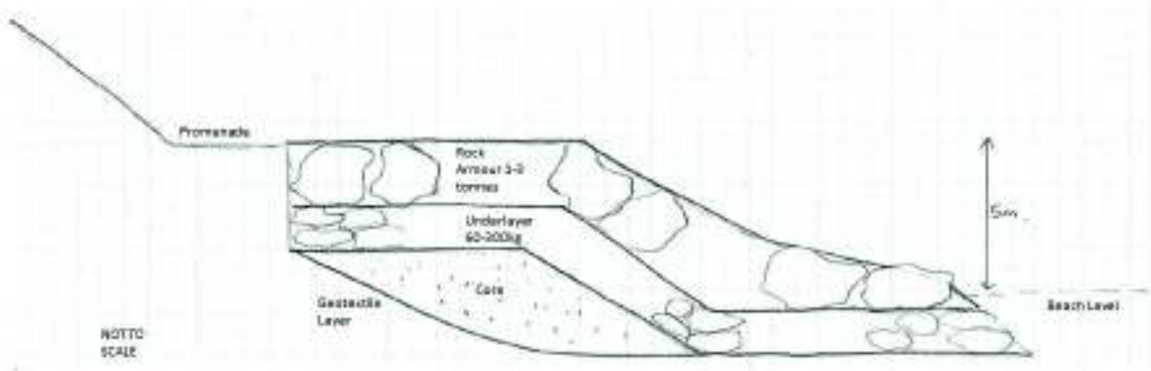
### 6.1.4 Preferred Options

#### 6.1.4.1 Coast Protection - Landslide

From the high level option assessment a rock revetment is considered to be the preferred technical solution to protect the toe of the landslide because it gives a better hydraulic performance than a vertical or stepped mass concrete seawall at a more economical cost. The permeable and flexible nature of this type of structure dissipates wave energy rather than reflecting it, thereby reducing the effects of the new structure on the surrounding foreshore and allows for further small movement associated with the instabilities. It will also assist to reduce overtopping and minimise the spray reaching the cliff slope. The weight of the armoured toe also assists in stabilising the toe of the cliff to provide additional support to the cliff slope. However, further analysis of the failed surface below is required to ensure that a piled base is not required.

The proposed structure comprises a rubble core protected along the crest and the seaward slope with rock underlayer and armour. The size of the rock is calculated using an assumed 1:3 slope and the maximum wave height modelled by HR Wallingford (Isle of Wight, 2010).  A wide crest is provided to give additional protection to the cliff from erosion by overtopping water during heavy seas. The toe of the revetment should be designed to prevent undermining by erosion of the foreshore material, such as through the use of a falling toe (depending on ground conditions). Once the rock revetment is installed the footpath could be reinstated; this would need to account for some potential future movement and settlement of the landslide material (Figure 6.1).

Figure 6.1: Outline Design of the Rock Revetment

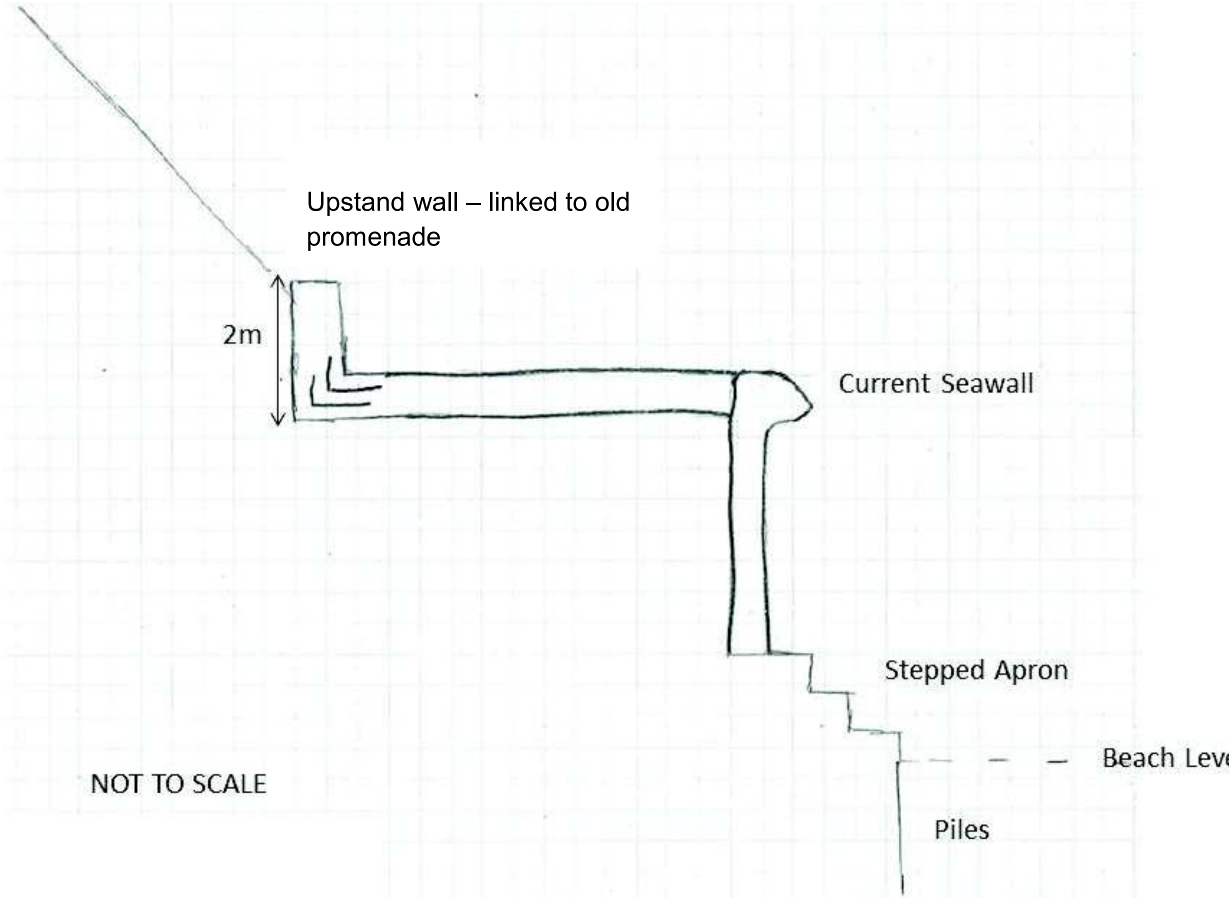



### 6.1.4.2 Coast Protection – Rest of the Frontage

Two options have been suggested for the rest of the frontage to illustrate the different costs of schemes that could be installed. Firstly similarly to the landslide section, the rock revetment would be extended along the base of the present seawall along the rest of the section. The specification would be exactly the same and would provide an aesthetic appeal by linking the whole frontage under one scheme. The extra weight added at the toe under this scheme would be a further benefit as the steel sheet piles in this section of the wall are shallower than the failed section (Figure 6.1).

The second option is to install a upstand wall at the rear of the promenade, which could be adapted for future climate change (Figure 6.2). Similar to the rock revetment this wall will reduce the amount of overtopping and the spray reaching the cliff slope reducing the erosion of material at the base of the cliff. However this wall will not provide any extra weight to stabilise the toe.

Figure 6.2: Outline Design of the upstand wall




The options are summarised in Table 6.3.

Table 6.3: Summary of the options.

| Option | Option Details | Description |
|---|---|---|
| **Option A- Do Nothing** | Defences are left as existing. Failure of the seawall will extend due to loss of fill material. Failed seawall sections will permit erosion of the cliff toe behind leading to further cliff failures. This will result on a loss of clifftop assets. | |
| **Option B- Rock Revetment along the base of the landslide and reinstatement of the footpath. Cliff drainage included** | A rock revetment is laid in front of the new toe of the landslide and the promenade re-instated. Shallow drainage is installed on the cliff face. | The rock revetment will protect the toe of the landslide from erosion, helping stabilise the landslide. However, retreat of the cliff top will still occur as the cliff adjusts to a new equilibrium. Drainage will help reduce the pooling of water leading to changes in pore water pressure and a resultant reduction in shear strength. |
| **Option C1- Rock Revetment along the base of the landslide and reinstatement of the footpath. Rock revetment along the rest of the frontage from the recent landslide to Totland Pier. Cliff drainage included** | A rock revetment is laid in front of the new toe of the landslide and along the rest of the section to Totland Pier. The promenade re-instated along the landslide. Shallow drainage is installed on the cliff face. | The rock revetment will protect the toe of the landslide from erosion, helping stabilise the landslide. The revetment will also help reduce wave overtopping of the whole length therefore reducing scour at the base of the landslide. The rock will also provide some toe weight to help stabilise the cliff. However there may still be retreat of the cliff top as the cliff adjusts to a new equilibrium. Drainage will help reduce the pooling of water leading to changes in pore water pressure and a resultant reduction in shear strength. |
| **Option C2- Rock revetment only along the base of the landslide and reinstatement of the footpath. An upstand wall to limit overtopping to be installed at the rear of the promenade between the recent landslide and Totland Pier. Cliff drainage included** | A new upstand wall will be installed on top of the existing seawall along the unfailed seawall section and then a rock revetment will be installed at the base of the landslide. Shallow drainage is installed on the cliff face. | The upstand wall will prevent the wave overtopping of the seawall and prevent the scour at the base of the cliff. The rock revetment will protect the toe of the landslide from erosion, helping stabilise the landslide. However there may still be retreat of the cliff top as the cliff adjusts to a new equilibrium. Drainage will help reduce the pooling of water leading to changes in pore water pressure and a resultant reduction in shear strength. |

## 6.2 Outline Costs

The outline costs of the options are stated in Table 6.4. The costs of options are based on the cost of the installation of similar schemes because no design development has been undertaken; only conceptual ideas have been developed at this stage. As a result the costs should be treated as guidance as prices may fluctuate due to changes in the price of materials, commercial factors, detailed design, timings of work etc. These costs have then been used as a general guide to look at possible funding potential form the Environment Agency and other funding requirements.

The costs are calculated as present values (PV). This is a method of discounting which is used in the economic calculations to compare costs and benefits that occur at different points in the appraisal peried i.e. over the 100 years. This discounting is undertaken because it is assumed that coasts and benefits are worth less than those in the short term because it is assumed that economic growth will mean the future generations are richer, such that, £1 in today's prices, will be worth much less in 100 years' time. Present value rates are usually calculated in year 0, and are intended to reflect the total value of all future cost benefits in today's prices (Environment Agency, 2010b).

Optimism bias has also been incorporated into the values. This is a percentage of the whole cost (60% in this study as a result of increased uncertainty surrounding the estimates) added to the whole cost as part a risk based contingency approach. "It covers the systematic tendency for appraisers to be overoptimistic about key project parameters, including project costs, works durations and benefits delivery" (Environment Agency 2010b).

Table 6.4: Table of the Options costs (PV) (rounded to nearest £1000)

| Option | PAR and detailed design costs | Construction Cost (based on approximate price per linear metre of structure) | Maintenance Cost (Total scheme) | Lifetime Cost (PV) |
|---|---|---|---|---|
| Option B- Rock revetment along the base of the landslide and reinstatement of the footpath. Cliff drainage included | £126,000 | £812, 000 | £5,000 | £1,510,000 |
| Option C1- Rock revetment from the Totland Pier to and including the landslide. Reinstatement of footpath through the landslide. Cliff drainage included | £126,000 | £2,500,000 | £23,000 | £4,240,000 |
| Option C2- New upstand wall installed on top of the current seawall and rock revetment only placed at the base of the landslide. Cliff drainage included. | £126,000 | £1,185,000 | £16,000 | £2,125,000 |

### 6.2.1 Options benefits (Damages avoided)

£5,100,000 present value (PV) benefits from residential properties could be associated with implementing a scheme along the whole frontage from coastal erosion over 100 years. £485,000 PV benefits from properties are associated with a scheme limited to protecting the current landslide frontage only.

The erosion damages have been calculated using guidance from the Multi Coloured Manual (MCM; Middlesex University, 2010). Rate of retreat has been calculated using historical erosion rates alongside residual life of the defences, which were taken as zero due to the possibility of failure from a landslide at any time (Appendix E). Values of residential properties were obtained from www.zoopla.co.uk. The discount rates used for the benefit cost analysis are taken from the Treasury Green Book (March 2003), to allow for future uncertainties to be taken into account. The discount rate from year 0-30 is 3.5%, year 30-75 is 3% and years 75-100 is 2.5%. The present value of the Do Nothing option is used as a baseline for calculation of the damages avoided.

As all the options have the same timing and phasing for construction of defences, because it is believed the cliff could fail at any time, i.e. within year 1 which will result in the failure of the seawall (more information in Appendix E), the PV damages avoided values are the same for each option. Table 6.5 summarises the PV Damages and Benefits. These also assume that schemes are designed and maintained to provide a 100 year design life.

In addition to the above benefits, there are a number of non-tangible benefits which although have not been valued within the economic assessment, are important considerations when assessing the likely benefit of a scheme. The benefits to the local community of having an access path linking the two towns, increases the space for activities such as walking and running, but also providing an important route for tourists, which is likely to have a key positive impact to the local community and economy. Also the intangible benefits of protecting the heritage structures of the gun battery at Fort Warden on the top of the cliffs should be considered.

Further information on the calculations can be found in Appendix E.

Table 6.5: Summary of the Present Value (PV) damages and benefits (60% Optimism Bias)

| Option | Damage (PVd) | Damage Avoided | Benefits (PVb) | Total PV damages | Total PV Benefits |
|---|---|---|---|---|---|
| Option A- Do Nothing | £5,100,000 | | | £5,100,00 | |
| Option B- Rock revetment along the base of the landslide and reinstatement of the footpath. Cliff drainage included | £4,615,000 | £485,000 | £485,000 | £4,615,000 | £485,000 |
| Option C1- Rock revetment from the Totland Pier to and including the landslide. Reinstatement of footpath through the landslide. Cliff drainage included | | £5,100,000 | £5,100,000 | | £5,100,000 |
| Option C2- New upstand wall installed on top of the current seawall and rock revetment only placed at the base of the landslide. Cliff drainage included. | | £5,100,000 | £5,100,000 | | £5,100,000 |

### 6.2.2 Benefit Cost Assessment

The benefit cost assessment of each scheme is presented in Table 6.6. From the results in Table 6.5 the option with the best cost benefit ratio and therefore the most economically viable is Option C2.

Furthermore this cost benefit ratio highlights that greater benefits are to be gained from a scheme along the whole section rather than just the landslide, due to the greater amount of assets that are going to be protected in the longer term although at additional overall cost.

Table 6.6: Benefit-cost assessment (PV costs include 60% Optimism Bias)

| Heading Left | PV Costs | PV Benefits | Av. Benefit/Cost Ratio |
|---|---|---|---|
| Option B- Rock revetment along the base of the landslide and reinstatement of the footpath. Cliff drainage included | £1,510,000 | £485,000 | 0.3 |
| Option C1- Rock revetment from the Totland Pier to and including the landslide. Reinstatement of footpath through the landslide. Cliff drainage included | £4,235,000 | £5,100,000 | 1.2 |
| Option C2- New upstand wall installed on top of the current seawall and rock revetment only placed at the base of the landslide. Cliff drainage included. | £2,125,000 | £5,100,000 | 2.4 |

### 6.2.3 Partnership Funding

The partnership funding calculator is used to understand the amount of funding that the Environment Agency may provide. However, this is merely an indication and the Environment Agency assesses each scheme on an individual basis to determine the amount of funding it will grant. The partnership funding scores, calculated using FCRM Partnership Funding Calculator for Flood and Coastal Erosion Risk Management Grant in Aid (FCRM GiA) version 7 (April 2013), are shown in Table 6.7.

Table 6.7: The amount of partnership funding that each scheme may be eligible for.

| Option | PV Total Whole Life Costs[1] | EA Partnership Funding Score | Contribution Required |
|---|---|---|---|
| Option B- Rock revetment along the base of the landslide and reinstatement of the footpath. Cliff drainage included | £1,510,000 | 8% | £1,375,000 |
| Option C1- Rock revetment from the Totland Pier to and including the landslide. Reinstatement of footpath through the landslide. Cliff drainage included | £4,235,000 | 17% | £3,500,000 |
| Option C2- New upstand wall installed on top of the current seawall and rock revetment only placed at the base of the landslide. Cliff drainage included. | £2,125,000, | 34% | £1,400,000 |

The scheme likely to receive the largest grant would be option C2 due to the high partnership funding value.  However a contribution of £1,400,000 would still be needed to fund the scheme. In contrast Option B has the lowest Partnership Funding score due to the reduced number of assets being protected; but also there are fewer contributions required due to the lower cost of the scheme as a result of the smaller shoreline extent being protected.

[1] Whole life costs is the total costs associated with a project for its full design and potential residual life span, taking proper account of all aspects of design, construction, maintenance and external impacts (Environment Agency, 2010b).

The partnership funding scores were also tested for sensitivity based on an increase in the cost of a scheme, and an extension in the time before houses are lost. The results are shown in Table 6.8. The patterns are very similar, with Option C2 gaining the most funding. However if the price of a scheme increases, the amount of funding is reduced. This is linked to a reduction in the cost benefit ratios.

Table 6.8: Sensitivity testing of the partnership funding scores

| | 25% increase in whole life cost of scheme | | 50% of houses that may have been lost in years 0-20 are now lost in years 21-50 | |
|---|---|---|---|---|
| Option | EA Partnership Funding Score | Contribution required | EA Partnership Funding Score | Contribution required |
| Option B- Rock revetment along the base of the landslide and reinstatement of the footpath. Cliff drainage included | 7% | £1,750,000 | 8% | £1,385,000 |
| Option C1- Rock revetment from the Totland Pier to and including the landslide. Reinstatement of footpath through the landslide. Cliff drainage included | 13% | £4,545,000 | 16% | £3,515,000 |
| Option C2- New upstand wall installed on top of the current seawall and rock revetment only placed at the base of the landslide. Cliff drainage included. | 27% | £1,920,000 | 33% | £1,415.000 |

### 6.2.4 Sensitivity Testing

The previous results were calculated with an Optimism Bias of 60%, which accounts for levels of risk within the concept design assumptions and calculations. As a sensitivity test the Optimism Bias was reduced to 30%, reducing the amount of project contingency which is more appropriate at PAR stage. The results are summarised in Table 6.9.

Table 6.9: Cost of option calculated with a 30% Optimism Bias

| Option | PV costs | PV benefits | Av. Benefit/Cost Ratio | PF Score | Contributions Required |
|---|---|---|---|---|---|
| Option B- Rock revetment along the base of the landslide and reinstatement of the footpath. Cliff drainage included | 1,225,000 | 485,000 | 0.4 | 10% | £1,105,000 |
| Option C1- Rock revetment from the Totland Pier to and including the landslide. Reinstatement of footpath through the landslide. Cliff drainage included | 3,440,000 | 5,100,000 | 1.5 | 18% | £2,790,000 |
| Option C2- New upstand wall installed on top of the current seawall and rock revetment only placed at the base of the landslide. Cliff drainage included. | 1,725,000 | 5,100,000 | 3 | 36% | £1,085,000 |

## 6.3 Summary

Table 6.10 summarises the costs. Option B is shown from the work we have done to be the least expensive to install, however the related benefits of this scheme are >10x less than the other schemes which cover the whole of the frontage because there are fewer benefits being protected by this scheme; consequently the cost benefit ratio is the lowest.

In contrast Option C2 has the highest cost benefit ratio for the whole of the frontage, due to the option costs which protect more assets. Compared to option B it protects significantly more assets with only a relatively small cost increase. Therefore it may prove more economically viable (£65,000 (30% optimism bias) or £120,000 (60% optimism bias) with partnership funding) to protect 10x the amount of assets along the coastline.

However these prices, although calculated as accurately as possible, can only be used as a guide as they are based on MM experience of previous project costs and are based on concept designs; if the options are to be taken forward a preliminary design and subsequent refinement of the calculations will be required.

The Partnership Funding scores calculated at this stage are for guidance and subject to revision following further development of the options. The Environment Agency prioritises projects and funding based on the cost and delivered benefits. Furthermore, in this case the risk to the defences is from a landslide which will then cause failure and an erosion risk, rather than the typical coastal cliff erosion risk, funding may be reduced. However, in Environment Agency reports they have stated “coastal retreat is often due to a combination of different processes…Thus many of the major protection schemes which have received grant in aid funding nationally over the last 20 years have included components of slope strengthening and drainage in addition to toe protection against waves.” (Environment Agency, 2010). Also “several grants in aided schemes have involved work to stabilise cliffs behind established seawalls. This has acknowledged the fact that unless these works are undertaken; risks may be posed to the coastal defence structure from landsliding occurring behind the defence itself” (Environment Agency, 2010). Examples of these schemes include Lyme Regis Cost Protection Scheme, Phase 2 where the old seawall was threatened by landsliding and continual cliff recession processes, despite the toe of the cliff system being protected from wave action.

Recently the Environment Agency’s funds have become increased. Historically about £250-275 million has been available nationally for flood and coastal risk erosion management. These allocations will be increasing to £370 million by 2015/16 and being protected in real terms until 2020/21. Hence the Partnership Funding score is a guide and further clarification would have to be sought form the Environment Agency if these schemes were to be taken further.

Table 6.10: Summary of the Schemes

| Option | Total Lifetime Cost (PV) | | Total Benefits (PV) | | Total Damages (PV) | | Av. Benefit/ Cost ratio | | PF score | | Further Contributions Required | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | 30% Optimism Bias | 60% Optimism Bias | 30% Optimism Bias | 60% Optimism Bias | 30% Optimism Bias | 60% Optimism Bias | 30% Optimism Bias | 60% Optimism Bias | 30% Optimism Bias | 60% Optimism Bias | 30% Optimism Bias | 60% Optimism Bias |
| Option B- Rock revetment along the base of the landslide and reinstatement of the footpath. Cliff drainage included | £1,225,000 | £1,510,000 | £485,000 | £485,000 | £4,615,000 | £4,615,000 | 0.4 | 0.3 | 10% | 8% | £1,105,000 | £1,375,000 |
| Option C1- Rock revetment from the Totland Pier to and including the landslide. Reinstatement of footpath through the landslide. Cliff drainage included | £3,440,000 | £4,235,000 | £5,100,000 | £5,100,000 | - | - | 1.5 | 1.2 | 18% | 17% | £2,790,000 | £3,49,000 |
| Option C2- New upstand wall installed on top of the current seawall and rock revetment only placed at the base of the landslide. Cliff drainage included. | £1,725,000 | £2,125,000 | £5,100,000 | £5,100,000 | - | - | 3 | 2.4 | 36% | 34% | £1,085,000 | £1,395,000 |

# 7 Recommendations for further work

The next stage would be to complete a FRM-7 from the Environment Agency, in order to obtain funding for development of a Project Appraisal Report (PAR). Additional information will be required and further aspects need to be considered as part of the PAR/ detailed design.

## 7.2 Further Design of the Options

To further develop the design of the rock revetment it is proposed to augment the existing design information by obtaining site specific information on the coastal conditions in the area of Totland Bay including the characteristics of the tides and currents, wave heights and wind speeds. This will be used in developing the wave heights for the use in the detailed design of the toe protection works. In addition, boreholes drilled along the whole stretch of the frontage may be required.

The boreholes drilled by Card Geotechnics Limited used for much of the ground model work are >70m from the cliff top and only one borehole was within the Fort Warden site boundary. The 1966 boreholes are old and not descriptive or deep enough to allow sufficient understanding of the ground conditions below the sea wall. Site specific boreholes would help provide an understanding of the discrete changes in geology, the geotechnical parameters of key strata along the stretch of the frontage and potentially highlight any areas of increased groundwater seepage or unconformities which could increase the risk of failure and hence be accommodated within the designs. Locating the slip plane (s) for the recent Totland landslide would also help with refining the failure mechanisms and provide detailed designs with more accurate information. Piezometers could also be installed to understand groundwater movements over time and assist with the planning of the drainage schemes.

In the design of the slope stabilisation works it will be necessary to obtain additional survey data for the coastal slopes to augment that which is already available. Should significant time have elapsed between the original survey and the commencement of detailed design, a new survey of the coastal slope would be required as inevitably the processes of instability will have modified the ground profile. It is also recommended that further geological mapping of the slopes be undertaken at the same time as the survey to take advantage of any additional exposures to enhance the ground model. Ideally the construction works should be taken in the summer months, say between May and September when rainfall should be at a minimum and hence so should ground water levels, reducing the propensity for failure.

## 7.3 Planning and Permissions

To undertake the next stage of development, early contact with the planning authority is recommended to ascertain their particular requirements for a scheme. Other aspects such as any restriction regarding road access to the site or any other matter which may influence the design should be identified at an early stage such that the detailed design may be developed within the framework of constraints. The land owners of the cliff tops and English Heritage should be contacted to check whether access can be gained from the land.

Also it should be ascertained whether any specific permissions or licenses are required with respect to the foreshore works.

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# Appendices

# Appendix A. Coastal defence condition assessment

## A.1 Intoduction

### A.1.1 Background

Mott MacDonald has been appointed to undertake an assessment of the landslip that occurred during December 2012 to March 2013 at Warden Point along the cliffs between Totland Bay and Colwell Bay on the north-west tip of the Isle of Wight. As part of the landslip assessment the condition of the remaining coastal defences has been assessed, along with an investigation into the causes of the seawall failure at the landslip.

A site visit to the landslip area was carried out on the 20th September 2013; during this visit, visual assessments of the defence condition and seawall failure were made. These assessments are contained within this report and are supplemented with additional site information obtained by Mott MacDonald and provided by the Isle of Wight Council.

### A.1.2 Previous Studies

Defence condition surveys have been previously carried out for this area as part of the Isle of Wight Shoreline Management Plan 2 (Isle of Wight Council, 2010). The West Wight Coastal Defence Strategy Study is currently in development.

The SMP2 referenced the section of Totland Bay and Colwell Bay as section IW45. Along this section all defences were graded as either 2 "Good" or 3 "Fair", with the exception of a section of seawall and groynes south of Totland Pier.

Table 8.1 below summarises the condition assessment and details for the defences at the location of the landslip (IW45/005):

Table 8.1: SMP Defence Condition and Residual Life at location of 2013 Landslip (Section IW45/005)

| Structure | Description | Condition Grade | Residual Life |
|---|---|---|---|
| Seawall | Seawall constructed in 1960's. Steel sheet pile toe with stepped concrete apron. Concrete wall with overhung coping at +3m ODN. | 2 "Good" | 15-25 Years |
| Seawall Toe (Rock) | Rock toe protection constructed in 1993 to exposed section of wall at the headland. | 2 "Good" | 15-25 Years |
| Rock Groynes | Shore perpendicular rock groyne constructed in 1993 | 2 "Good" | 15-25 Years |
| Timber Groynes | Shore perpendicular timber groynes (possibly constructed 1976) | 3 "Fair" | 8-12 Years |

Source: Isle of Wight Shoreline Management Plan 2 – Appendix C2: Defence Appraisal

## A.2 Coastal Defence Condition Assessment

### A.2.1 Methodology

The defence condition assessment has been carried out using the Environment Agency's Condition Assessment Manual (Environment Agency, 2006). The manual provides a consistent methodology for grading the condition of a wide variety of coastal and fluvial structures. The condition of each structure is graded against a 5 point score ranging from 1 "Very Good" to 5 "Very Poor". Table 8.2 below is reproduced from the Defence Condition Manual and provides descriptions of the various grades.

Table 8.2: Defence Condition Grade Descriptions

| Grade | Rating | Description |
|---|---|---|
| 1 | Very Good | Cosmetic defects that will have no effect on performance |
| 2 | Good | Minor defects that will not reduce the overall performance of the asset |
| 3 | Fair | Defects that could reduce the performance of the asset |
| 4 | Poor | Defects that would significantly reduce the performance of the asset |
| 5 | Very Poor | Severe defects resulting in complete performance failure |

Source: Environment Agency Condition Assessment Manual (2006) Ref:

Alongside these grades, observations are also made of the type and construction of structure and key features or defects.

It should be noted that whilst the Condition Assessment Manual provides a consistent methodology for undertaking assessments they rely heavily upon the subjective interpretation of visual features of the structures.

### A.2.2 Overview of Defence section

The coastal defences from Totland Bay to Colwell Bay consist of a variety of seawall sections with rock and timber groynes along some sections. Parts of the seawall have additional rock armour along the toe to provide increased protection against scour along the most exposed sections of the seawall.

The foreshore is a mix of sand and shingle. Waves can propagate directly from the west on to the frontage, while currents run parallel to the shoreline approximately north-south.

### A.2.3 Defence Condition Tables

The following tables show the results of the defence condition assessment. The tables in this section apply to the sections assessed during the site visit (in some instances sections have been grouped due to the similarity in construction, cross reference is made to the SMP defined defence elements. The defence assessment has been ordered running from Totland Pier to the Slipway in Colwell Bay. At the time of the survey (20th September 2013) tide levels did not permit access to the foreshore or clear visibility of insitu toe levels.

Table 8.3: Defence Section 1 – Totland Bay

| | | | |
|---|---|---|---|
| **Asset Location** | | | |
| **Section ID:** | Totland Bay | **Location:** | North of Totland Pier up to the Cliff Failure |
| | | **Survey Date:** | 20/09/2013 |
| | | | |
| **SMP Unit:** | | **SMP Defence Section:** | IW45/003 |
| **SMP Policy:** | Short Term | Medium Term | Long Term |
| | Hold the line – Maintain / Replace | Hold the line – Maintain / Replace | Hold the line – Maintain / Replace |
| **Coastal Defence Condition** | | | |
| **Defence Type:** | Concrete Seawall (SW), Rock Toe Protection (RT), Rock Groynes (RG) | | |
| **Coastal Defence Length:** | 370m | **Ownership:** | Isle of Wight Council |
| **Foreshore Type:** | Sand and Shingle Beach | **Assets Protected:** | Public Footpath, Cliff Hinterland with residential properties setback on top of cliff. |
| **Exposure:** | Moderate | **Year Built:** | SW – Late 1960's<br>RG / RT – 1993 |
| **Condition Grade:** | SW – Good, with some areas of Fair due to damage to crest.<br>RT – Good<br>RG – Good | **Threshold Grade:** | N/A |
| **Residual life min:**<br>**(No Maintenance)** | SW – 10<br>RG – 10 | **Residual life max:**<br>**(No Maintenance)** | SW – 20<br>RG – 20 |
| **Description of the defences and the foreshore** | | | |

**Asset Location**

This section of defence consists of a concrete seawall fronted by sand/shingle beach or rocky foreshore. There are two rock groynes perpendicular to the shoreline between Totland Pier and the location of the landslip.

The seawall is constructed from reinforced concrete, and has a slight recurve profile, at the base of the curve a stepped toe leads down to sheet piling. The seawall has been cast in sections approximately 5m in length and is approximately 2.5m in height. Access steps to the foreshore are located at two points along the seawall. There is some variation in the seawall profile, crest level and apron size. Rock toe protection exists along the apron toe to prevent scour.

Seawall condition is Fair to Good; some areas of the crest of the recurve wall are showing signs of significant cracks (See Figure 8.1) as well as previous repairs, the remainder of the wall is showing typical wear consistent with the exposed marine environment. At the time of the site visit of there was no sign of movement along the seawall along the crest-line of the wall (See Figure 8.2 and Figure 8.3). Joints in the promenade seem to be relatively recent and are generally well bonded. Seawall face joints could not be visually determined at the time of the visit although viewing from crest level many were missing or in poor condition.

Behind the promenade areas of the cliffs are showing signs of landslip, with debris material present immediately adjacent and spilling on to the promenade. (See Figure 8.4)

Groyne condition was difficult to assess at the time of the survey due to tide levels but there are no significant signs of missing rock such that performance of the groyne is affected. Aerial imagery confirms that groynes do not show signs of significant rock displacements. Groyne performance in maintain sediment may be limited by available sediments and highly variable with season.

Figure 8.1: Defence Section 1 - Cracks along seawall crest.

Figure 8.2: Defence Section 1 South – No current signs of seawall movement

Figure 8.3: Defence Section 1 North – No current signs of seawall movement

Figure 8.4: Defence Section 1 - Minor landslips at Totland Bay, spilling on to Promenade.

Table 8.4: Defence Section 2 – Warden Point Section 1 – Landslide Failure

| **Asset Location** | | | |
|---|---|---|---|
| **Section ID:** | Warden Point 1 | **Location:** | Landslide Failure |
| | | **Survey Date:** | 20/09/2013 |
| | | | |
| **SMP Unit:** | | **SMP Defence Section:** | IW45/005 |
| **SMP Policy:** | Short Term | Medium Term | Long Term |
| | Hold the line – Maintain / Replace | Hold the line – Maintain / Replace | Hold the line – Maintain / Replace |
| **Coastal Defence Condition** | | | |
| **Defence Type:** | Concrete Seawall (SW), Rock Toe Protection (RT) | | |
| **Coastal Defence Length:** | 120m | **Ownership:** | Isle of Wight Council |
| **Foreshore Type:** | Large cobbles and boulders along with some stone debris on the foreshore at toe of structure.<br>Sand and Shingle forshore in some places where beach material exists | **Assets Protected:** | Public Footpath, Cliff Hinterland with residential properties setback on top of cliff. |
| **Exposure:** | Moderate | **Year Built:** | SW – Late 1960's<br>RT – 1993 |
| **Condition Grade:** | Failed | **Threshold Grade:** | N/A |
| **Residual life min:** | 0 | **Residual life max:** | 0 |

| **Description of the defences and the foreshore** |
|---|
| This section of defence consists of a concrete seawall fronted by sand/shingle or rocky foreshore. The landslide along this section has caused failure of the seawall displacing it seawards up to 20m. Rock toe protection remains in front of the sheet piles having also been displace seawards.<br><br>The seawall is constructed from reinforced concrete, and has a slight recurve profile. At the base of the curve a stepped toe leads down to sheet piling. The seawall has been cast in sections approximately 5m in length and is approximately 2.5m in height. Rock toe protection exists along the apron toe to prevent scour.<br><br>Whilst the seawall remains in place along the toe of the landslide it still provides limited protection to the cliff toe. Significant gaps between the seawall units are present, due to the displaced alignment being longer than the original. In these gaps erosion of the material behind the seawall will occur, particularly during storms. |

Table 8.5: Defence Section 3 – Warden Point Section 2 – Non-failure Section

| | | | |
|---|---|---|---|
| **Asset Location** | | | |
| **Section ID:** | Warden Point 2 | **Location:** | Immediately north of seawall / cliff failure. |
| | | **Survey Date:** | 20/09/2013 |
| | | | |
| **SMP Unit:** | | **SMP Defence Section:** | IW45/005 |
| **SMP Policy:** | Short Term | Medium Term | Long Term |
| | Hold the line – Maintain / Replace | Hold the line – Maintain / Replace | Hold the line – Maintain / Replace |
| **Coastal Defence Condition** | | | |
| **Defence Type:** | Concrete Seawall (SW) | | |
| **Coastal Defence Length:** | 125m | **Ownership:** | Isle of Wight Council |
| **Foreshore Type:** | Large cobbles and boulders along with some stone debris on the foreshore at toe of structure. | **Assets Protected:** | Public Footpath, Cliff Hinterland with residential properties setback on top of cliff. |
| **Exposure:** | Moderate | **Year Built:** | SW – Late 1960's<br>RT – 1993 |
| **Condition Grade:** | SW – Good<br>RT – Good | **Threshold Grade:** | N/A |
| **Residual life min:**<br>**(No Maintenance)** | SW – 10 | **Residual life max:**<br>**(No Maintenance)** | SW – 20 |
| **Description of the defences and the foreshore** | | | |

**Asset Location**

This section of defence consists of a concrete seawall fronted by sand/shingle beach or rocky foreshore.

The seawall is of the same construction as the failed section, constructed from reinforced concrete; with slight recurve profile, at the base of the curve a stepped toe leads down to sheet piling. The seawall has been cast in sections approximately 5m in length and is approximately 2.5m in height. Rock toe protection exists along the apron toe to prevent scour.

Seawall condition is generally Good, showing typical wear consistent with the exposed marine environment with little evidence of significant defects that would reduce the performance of the structure in its role as erosion protection.

At the time of the site visit there was one short length of seawall that potentially showed signs of movement along the crest-line of the wall (See Figure 8.6). A check of adjacent joints on the promenade was undertaken, Figure 8.7 shows images of the joint either side of the potential movement (locations illustrated in Figure 8.6):

- The Left image shows a well bonded joint, indicating no movement.
- The Right image shows that the joint material has become debonded and is a further indicator of potential wall movement.

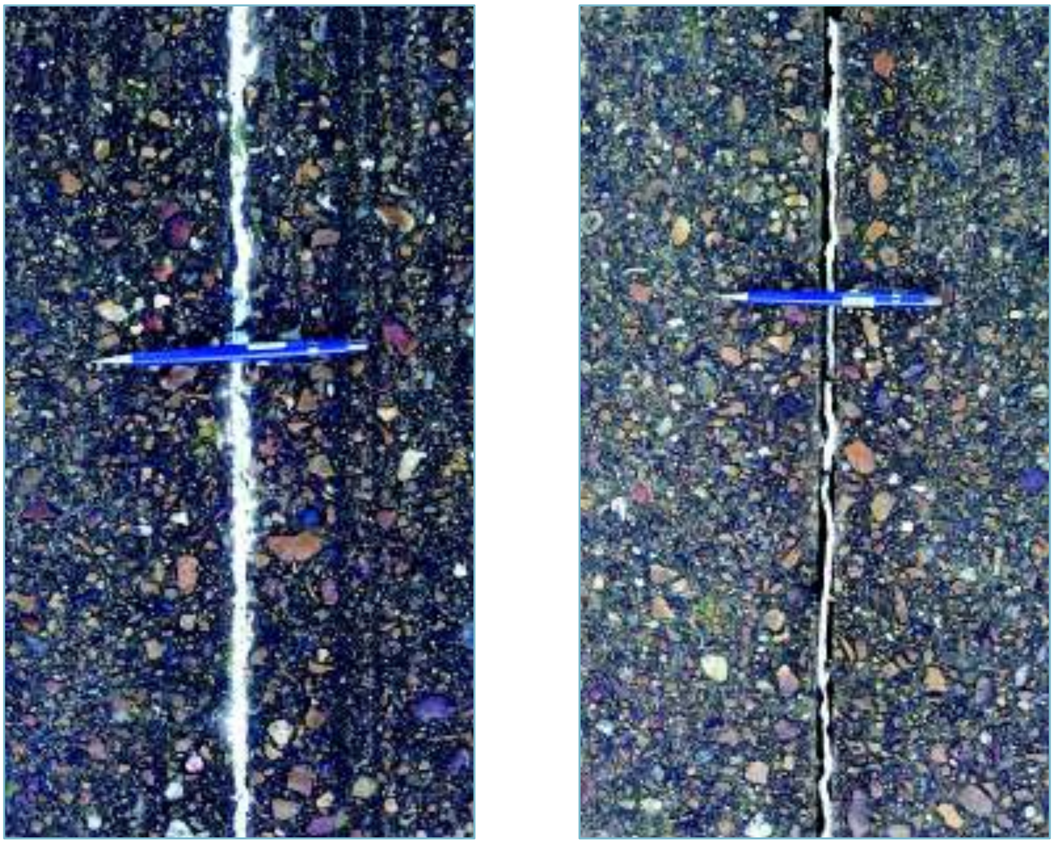
The movement may be a result of historic movement or indeed variation in the original construction alignment. I**t is recommended that an indelible line is marked perpendicular across the promenade and seawall joints in this area and photographed such that small movements can be detected and monitored.** The condition of seawall face joints could not be visually determined at the time of the visit.

Behind the promenade the cliffs are not currently showing signs of landslip, except for areas of debris material caused by the recent adjacent slip.

Figure 8.5: Defence Section 3 – Seawall adjacent landslide

Figure 8.6: Defence Section 3 – Potential seawall movement

Figure 8.7: Defence Section 3 – Potential joint movement.



Left: Joint along section without movement, Right: First joint in area of potential movement

Table 8.6: Defence Section 4 – Colwell Bay Section 1

| **Asset Location** | | | |
|---|---|---|---|
| **Section ID:** | Colwell Bay 1 | **Location:** | Warden Point to Colwell Slipway |
| | | **Survey Date:** | 20/09/2013 |
| | | | |
| **SMP Unit:** | | **SMP Defence Section:** | IW45/005 |
| **SMP Policy:** | Short Term | Medium Term | Long Term |
| | Hold the line – Maintain / Replace | Hold the line – Maintain / Replace | Hold the line – Maintain / Replace |

| **Asset Location** | | | |
|---|---|---|---|
| **Coastal Defence Condition** | | | |
| **Defence Type:** | Concrete Seawall (SW), Rock Toe Protection (RT), Timber Groynes (TG) | | |
| **Coastal Defence Length:** | 245m | **Ownership:** | Isle of Wight Council |
| **Foreshore Type:** | Predominantly Sand with some fractions of shingle. | **Assets Protected:** | Public Footpath, Cliff Hinterland with residential properties setback on top of cliff. |
| **Exposure:** | Moderate-Sheltered, due to headland, change in coastal orientation ad offshore rock reef. | **Year Built:** | SW – Late 1960's<br>RT – 1993<br>TG - |
| **Condition Grade:** | SW – Good<br>TG – Fair | **Threshold Grade:** | N/A |
| **Residual life min:**<br>**(No Maintenance)** | SW – 10<br>TG – 5 | **Residual life max:**<br>**(No Maintenance)** | SW – 20<br>TG – 15 |
| **Description of the defences and the foreshore** | | | |

Immediately behind the first groyne on Warden Point a short section (26m) of vertical concrete seawall exists. The vertical wall transitions to a section of the original block masonry wall, 80m in length (See Figure 8.8). The masonry wall then transitions to a concrete recurve seawall with a crest level approximately 400mm below the masonry crest level, this wall continues to the slipway within Colwell Bay (See Figure 8.9 and Figure 8.10).

The concrete seawall condition is generally Good, showing typical wear consistent with the exposed marine environment with little evidence of significant defects that would reduce the performance of the structure in its role as erosion protection. The masonry section of wall is also in Good condition, likely as a result of its orientation and the location behind a timber groyne resulting in reduced wave exposure. However, there are sections of the masonry wall (at the northern end) which would benefit from repointing (Figure 8.8).

In front of the seawall beach material is retained by the use of four timber groynes, up to the slipway in Colwell Bay. Timber groyne condition is Fair overall. Groyne planks are supported by two steel "I" Beams piles driven closely together, all piles remain straight and upright (See Figure 8.11). There are a few missing planks potentially affecting the ability of the groynes to beach material (See Figure 8.12 and Figure 8.13). Due to tides during the condition survey the lower plank levels for the offshore sections of groyne were not visible.

Figure 8.8: Defence Section 4 – Masonry Seawall north of Warden Point

Figure 8.9: Defence Section 4 – Colwell Bay Seawall

Figure 8.10: Defence Section 4 – Colwell Bay Seawall to Slipway

Figure 8.11: Defence Section 4 – Timber groynes straight and example “I” piles

Figure 8.12: Defence Section 4 - Timber groyne with missing planks towards offshore end.

Figure 8.13: Defence Section 4 – Missing planks at inshore end of 2nd groyne.

## A.3 Defence Failure Assessment

This section reviews the available information to understand the causes of the seawall failure along the 2013 landslip section.

### A.3.1 1960s Seawall Remedial Works

During the late 1960's a remedial works scheme was undertaken along the Totland to Colwell Bay section of cliffs known as Warden Point. Documents of the scheme have been provided by the Isle of Wight Council and include:

- Report of County Surveyor – Coast Protection Works Scheme, Warden Point, Totland. (Dated 30th August 1966)
- Plan of Works – assumed to be part of the above report (No reference available)
- Cross Section of Proposed Seawall Works – assumed to be part of above report (Drawing ref not legible)

The Report of the County Surveyor (1966) provides a description of the original seawall lists the visual defects and outlines a proposal for remedial works. The details relevant to the failed section (Part II) are provided below:

*"The existing seawall is approximately 13ft high* [4m] *above the beach level and comprises cyclopean stone blocks with a vertical face (with one step) 9'6"* [2.9m] *in height, surmounting a sloped stone apron, the toe of which is protected by horizontal timber planking held by timber piles at 10ft* [3m] *centres. A further concrete apron has been removed by wave action. Behind the wall there is a 12ft* [3.7m] *wide concrete surfaced promenade.*

*<u>Defects.</u> The face of the wall needs extensive pointing; much of the stone apron has perished; there are a number of cavities at the base of the wall which is being undermined in places; the surface of the promenade is badly cracked.*

*<u>Proposals.</u> Drive steel sheet piling up to 13ft* [4m] *in length, the top of the piles to be three feet* [0.9m] *above beach level and driven through three feet [0.9m] of beach material and four feet* [1.2m] *of compacted sand and gravel and three feet* [0.9m] *into the firm clay base; provide a reinforced concrete protective toe and stepped apron all as in Part I; fill all cavities with concrete; point all wall joints; make good promenade."*

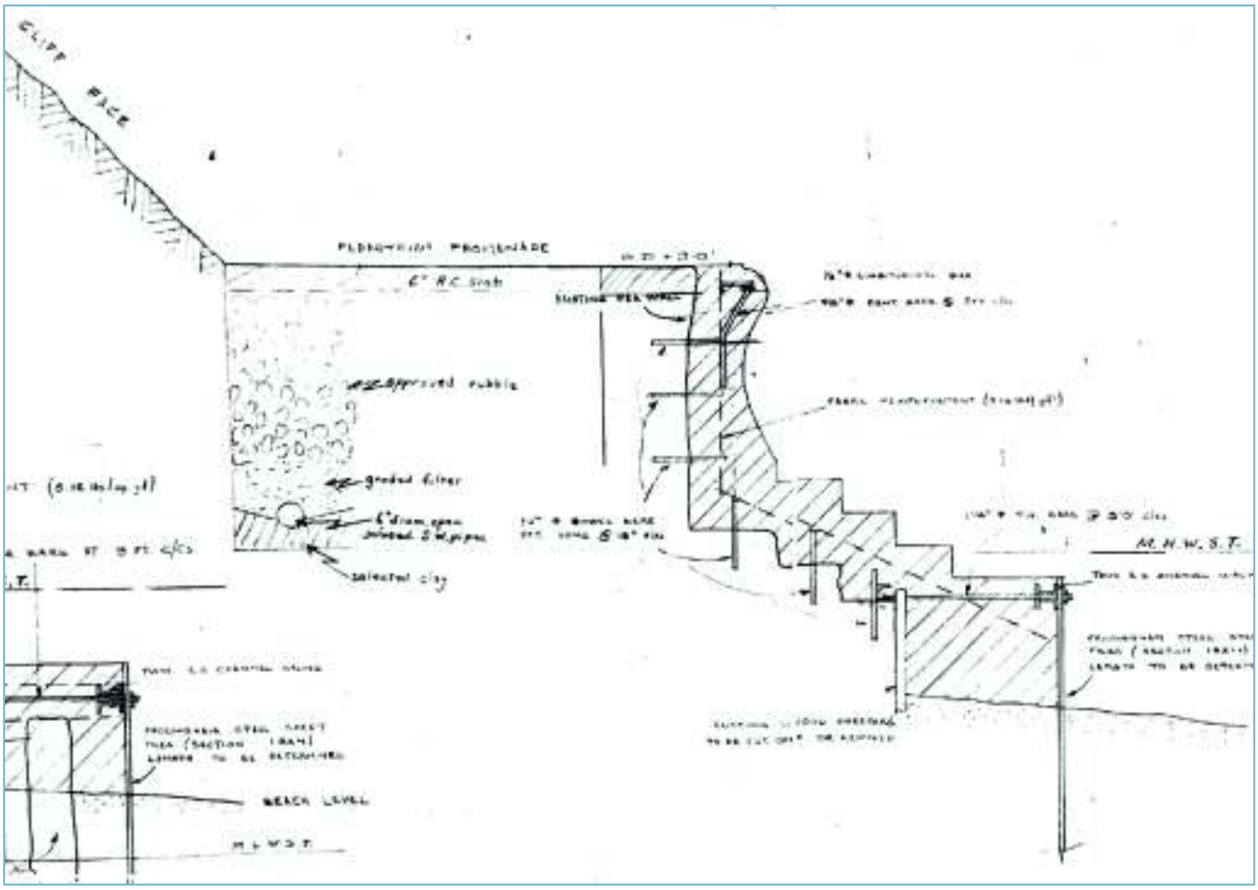
The proposal for this section makes reference to the details of Part 1, which is the seawall section further south towards Totland. The details of the proposal for Part I is:

*<u>Proposals.</u> Drive 10ft* [3m] *long steel sheet piles, the top of the piles to be three feet above beach level and driven through 2'6" [0.8m] of beach material and 4'6"* [1.4m] *into the firm clay base; tie with twin steel channel walings and tie-bars; construct reinforced concrete toe, stepped apron and wall face encasement complete with bullnose profile, the whole keyed to the existing wall by dowel bars; fill all cavities in existing work* sic [wall] *with concrete; make good to promenade.*

It is noted that the driven depth of the sheet steel piles was to be varied between different Parts of the seawall. There is no information available to confirm that the installed lengths were as proposed.

The cross section of the seawall construction illustrates the described proposal in the Report of the County Surveyor (1966), refer to Figure 8.14. It shows the proposed installation of the sheet piling, stepped concrete apron and concrete encasement tied to the existing wall with dowel bars.

Figure 8.14: Proposed seawall cross section from 1966 remedial works




### A.3.2 1993 Remedial Works

It is written in the Defence Appraisal of the Shoreline Management Plan 2 that additional remedial works were undertaken along the seawall section in 1993 when the rock groynes to the south of the landslip area were constructed. In the area of the landslip, rock was placed along the toe of the seawall. It is assumed that this was placed in order to prevent scour of the beach in front of the wall, this may be an indication that beach levels in 1993 were reduced from those present in 1966.

### A.3.3 Failure observations

The site visit to the landslip (conducted 20th September 2013) was able to view the cross section of the seawall at the failed section (Figure 8.15) and confirms that the general arrangement shown in Figure 8.14 was installed as proposed.

Figure 8.15: Cross section of failed seawall (20th September 2013)

Additional information was also gained during the site visit on the seawall construction:

- The seawall encasement and stepped apron were cast in approximately 5m sections, minimum thickness approximately 350mm.
- Reinforcement in the concrete was not continuous across any joints. (i.e. all joints allowed movement)

The following figures show views of the seawall taken the day prior to the defence condition survey (19th September 2013) when tidal levels were low enough to view the toe of the seawall along the failure area; commentary on the figures identifies aspects of the potential failure mechanisms.

Figure 8.16: Failed seawall – viewed from Totland Bay looking north (19th September 2013).

Figure 8.16 shows the southern extent of the landslide viewed from Totland Bay looking north. The image shows the seawall has been pushed horizontally seaward. The seawall does not show significant signs of forward rotation indicating that the toe of the concrete structure has moved relatively freely without restraint by the steel sheet pile. In this location the seawall has been displaced seawards by approximately 10m. Given the extent of displacement there is no significant build-up of material in front of the structure toe, this indicates the seawall has predominantly moved across the surface, pushing only a shallow layer of material.

Figure 8.17: Failed seawall central section at low water – view looking towards Totland Bay (11th February 2013)

Figure 8.17 was taken by the Isle of Wight Council during low water in February. At this section the CCO analysis (Refer to Appendix A) shows the wall has displaced seawards by 17m. The image shows that the central sections of seawall have remained intact, but failure of the piles and apron connection has occurred. Along the section it can be seen that as the piles have rotated they have caused the stepped apron concrete to be cracked horizontally allowing the tie bars to pull free from the concrete. It is not clear if the walings were fitted, as proposed within the original design (shown in Figure 8.14), or if only the hooked tie bars were installed.

Figure 8.18: Failed seawall – viewed from Colwell Bay extent of landslip looking south (19th September 2013)

Figure 8.18 shows the failed seawall viewed from the Colwell Bay extent of the landslide, looking south towards Totland. The analysis by CCO (Refer to Appendix A) illustrates that this section has been displaced seawards by approximately 7m. The sheet piles are visible in front of the concrete apron, though connecting bolts have sheared and the sheet piles have become detached from the concrete apron and are bent forwards. Foreshore material has been displaced forwards ahead of the piles (indicated by the upwelling of sand through the rock material), this indicates that pile failure occurred at a lower level than the southern section shown in Figure 8.16. The back rotation of the end section of displaced wall is also an indicator that as piles were pushed forwards a void was formed beneath the seawall. Other sections may have failed under the same mechanism however the void behind could have been filled with landslip material as it moved seawards preventing back rotation.

**Key Observations**

Observations made during the site visit and from assessment of the photographs are summarised to provide evidence for the consideration of the failure mechanism:

- The seawall encasement was cast in 5m sections; these sections are not connected and therefore act independently.
- The masonry block wall behind the encasement is unlikely to provide any significant resistance due to its block construction and potentially poor condition of mortar between blocks.
- The seawall encasement did not fail or crack during the movement. It is unclear if the upper encasement was fixed to the toe as some units show signs of separation (at the wall and stepped apron interface).
- Sheet steel piles are evident along the displaced seawall toe. Connections between the steel piles and the toe apron have in most instances failed. Failure has been through pull out of the tie bars, assisted by horizontal cracking of the stepped apron concrete, or by shearing of the bolt connections.
- The sheet steel piles have failed through a combination of rotational and flexural failure. Once failed it is possible that the sheet steel piles provided a slip surface for the wall to slide along.
- There is evidence of material upwelling in front of the seawall as it has displaced seawards, although this is not significant. It should also be noted that the site visit was conducted approximately 6 months after the landslide failure and therefore upwelled material may have been settled or eroded.

The conclusion of the failure assessment is that failure was caused by the pile design being exceeded. Piles were principally constructed to prevent toe erosion and undermining of the seawall rather than to stabilise the cliff or hold the seawall in the event of a landslide.

### A.3.4 Implications

It should be noted that if the wall further south within Totland Bay was installed according to the Report of the County Surveyor (1966) then the sheet piles have less embedment depth and would therefore also be at risk of similar failure if the horizontal imposed loads of the cliffs behind increase.

## A.4 Defence Options

### A.4.1 Options to Address Seawall Failure

In the area of the seawall failure a short-term solution is required to address the potential for material to be lost between the gaps in the seawall.

# Appendix B. Seawall Movement



| Profile | Length (m) |
|---|---|
| 4 | 7.10 |
| 5 | 11.55 |
| 6 | 14.71 |
| 7 | 16.71 |
| 8 | 17.39 |
| 9 | 14.48 |
| 10 | 10.92 |
| 11 | 6.00 |

## TOTLAND BAY

CHANGE IN SEAWALL/CLIFF POSITION

**Legend**

- Sea Wall Position 2013
- Sea Wall Position 2008

N

Top Map

0 50 100 m

Bottom Map

0 70 140 m

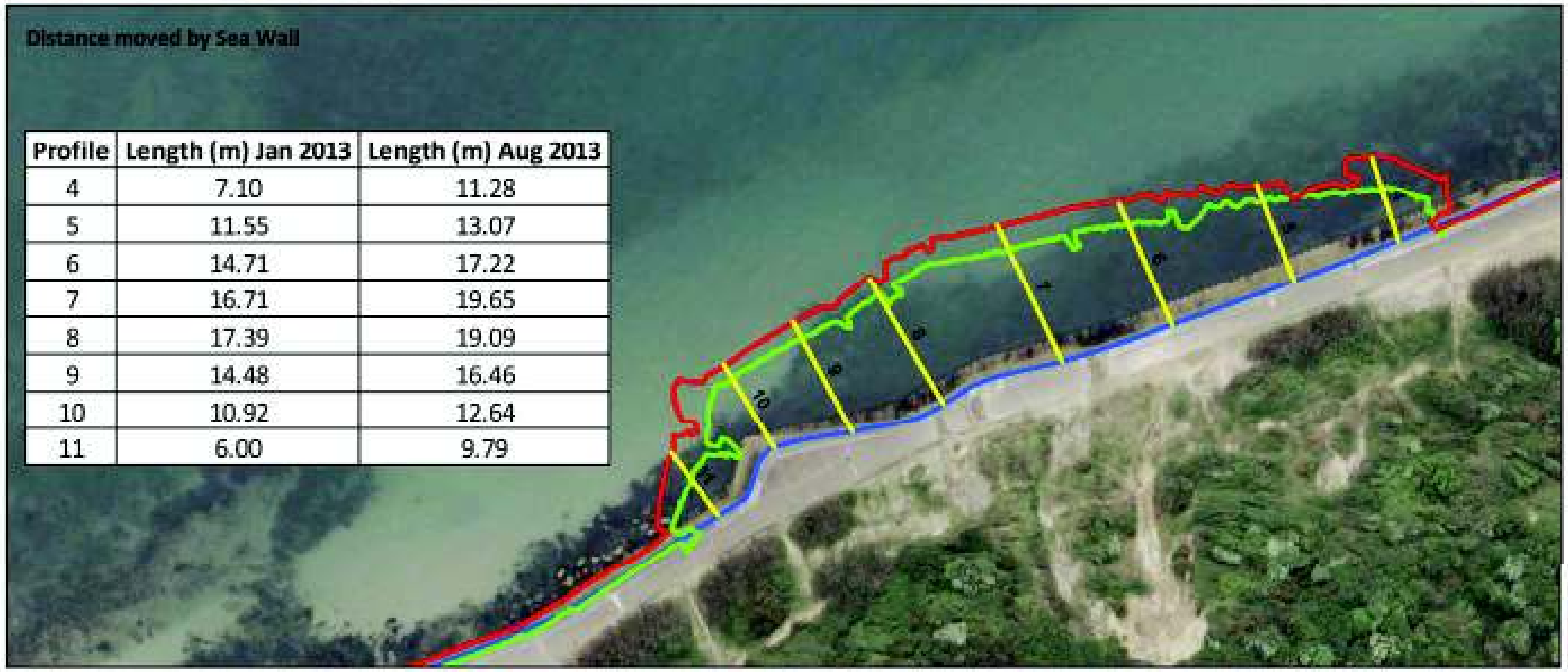
Aerial Photography: 2008

Channel Coastal Observatory

Map created: 2013

# TOTLAND BAY

CHANGE IN SEAWALL/CLIFF POSITION


| Profile | Length (m) Jan 2013 | Length (m) Aug 2013 |
|---|---|---|
| 4 | 7.10 | 11.28 |
| 5 | 11.55 | 13.07 |
| 6 | 14.71 | 17.22 |
| 7 | 16.71 | 19.65 |
| 8 | 17.39 | 19.09 |
| 9 | 14.48 | 16.46 |
| 10 | 10.92 | 12.64 |
| 11 | 6.00 | 9.79 |

## Legend

- Sea Wall Position Aug 2013
- Sea Wall Position Jan 2013
- Sea Wall Position 2008

N

Top Map

0 60 120 m

Bottom Map

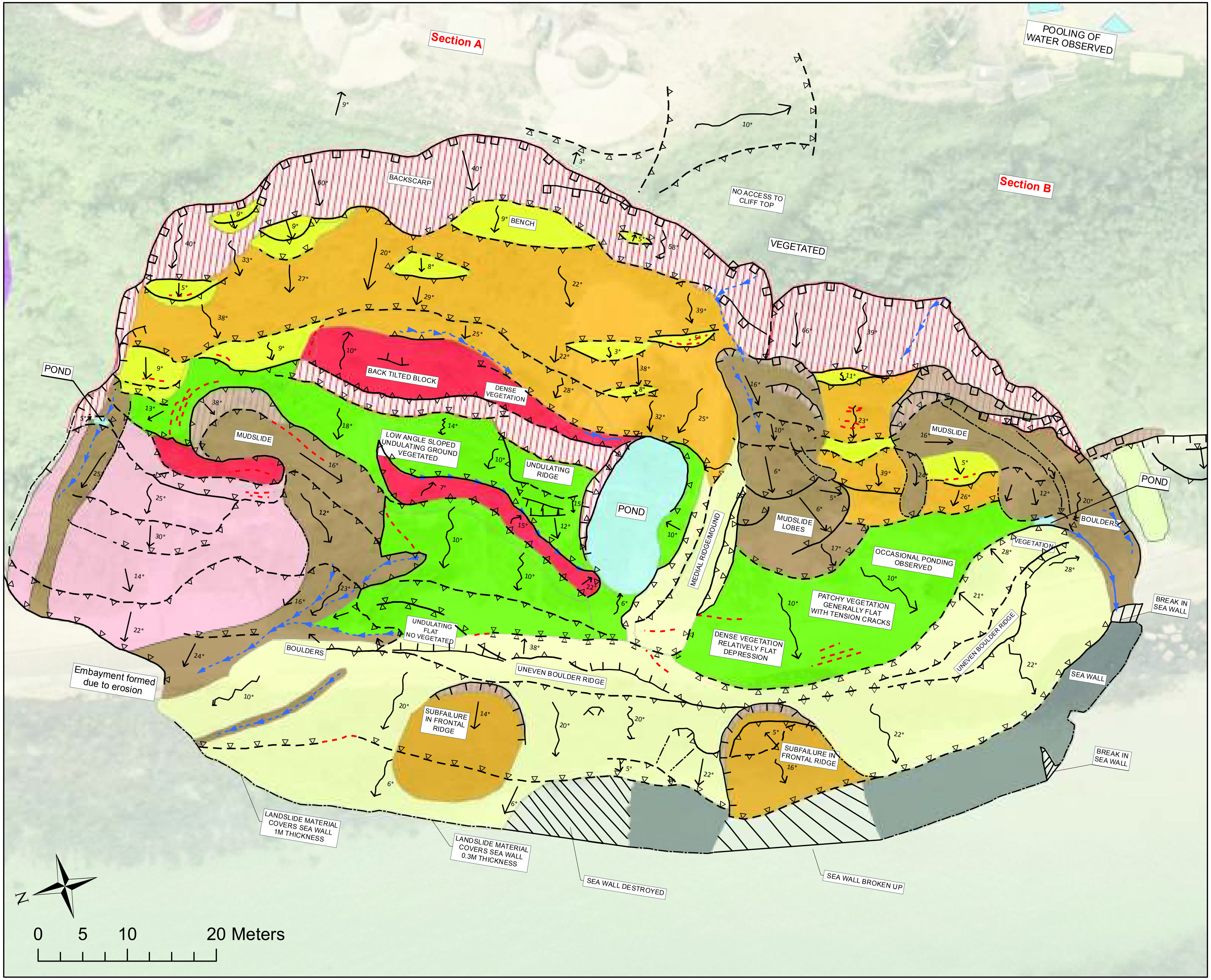
0 20 40 m

Aerial Photography: 2008

Channel Coastal Observatory

Map created: 2013

# Appendix C. Geomorphological Maps




## Gemorphological Lines

- Boundary Line
- Cliff Line
- Scarp Line
- Tension cracks
- Convex Break of slope
- Convex Change in Slope
- Concave Break of Slope
- Concave Change in Slope
- stream
- dry stream
- boundary
- Crest
- Crest Undulating
- Gully/ linear depression
- How Ledge Limestone - resistant band. Forms a prominent buff scarp. Seen in section B of landslide only and south of the landslide intermittently towards Totland. I tincrease in elevation by approximatly ~7m from the landslide southwards to the end of the study area near Totland Bay.
- Backscarp - Typically steeper than 40 degrees
- Backscarp of Superficial Mudslides- TYpically steeper than 37 degrees corresponding to clayey strata (Venus and Neritina Beds) above the HOw LEdge Limestone and has number of seepage points along the length. More visible in Section B.
- Backtilted block - A displaced block that has moved and rotated downslope so that it tilts back into the slope. Gullys often form in the low points behind. These are generally vegetated and have failed en mass as a block therefore aside from the backtitled trees the vegetation is relatively undisturbed
- Bench/Plateau- a small block that has been moved down the cliff generally translational movment. Oftne soil and grassy vegetation starting to colonise
- Rear Debris Scree accumulation zone- Material accumulated at the base of the backscarp. No vegetation can take hold due to continual unravelling of material downslope. Predominantly sand in section A, whilst thisis clay/silt forming mudslides in section B.
- Mudslide - Surficial mudslides which have occured folowing the main failure, initaited by groundwater seepage erosion.
- Surficia llandslides on the seaward side of the frontal ridge as a result of toe weight removal by wave erosion
- Ridge- accumulation of material at the base of the slide due to reduction in energy as the slide moved seawards. Debris comprised of large boulders of shelly limestone, blue grey clay and sandstonebouelders.
- Extensional area / graben relatively low relief - depression developing as the result of a translational failure. Numberof tension cracks, uneven surface variably vegetated. Ponding in this area common
- Area of shallow sliding - area of shallow surficial movmeent occured following the main slide.
- Pond
- Channel - large channel of water
- Sea wall visible- segments are seperated and erosion between segments is causing the pavemetn above to become unsupported and tilt.
- Surficial Mudslides- failures the rest of the frontage
- Deep Seated LandslideArea - relict, dense vegetation and benched

| Rev | Date | Description | Drawn | Ch'k'd | App'd |
|---|---|---|---|---|---|
| B | 28/11/13 | First Issue | LW | LCW | PJP |
| A | 09/10/2013 | For Review | BR | LW | |

Mott MacDonald

Mott MacDonald House
8-10 Sydenham Road
Croydon, Surrey, CR0 2EE
T +44 (0)20 8774 2000
F +44 (0)20 8681 5706
W www.mottmac.com

Client

Isle of Wight Council
County Hall
High Street
Newport
Isle of Wight
PO30 1UD

Title

# Totland Bay Phase 1
# Geomorphological Mapping

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## Gemorphological Lines

- Boundary Line
- Cliff Line
- Scarp Line
- Tension cracks
- Convex Break of slope
- Convex Change in Slope
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Mott MacDonald

Mott MacDonald House
8-10 Sydenham Road
Croydon, Surrey, CR0 2EE
T +44 (0)20 8774 2000
F +44 (0)20 8681 5706
W www.mottmac.com

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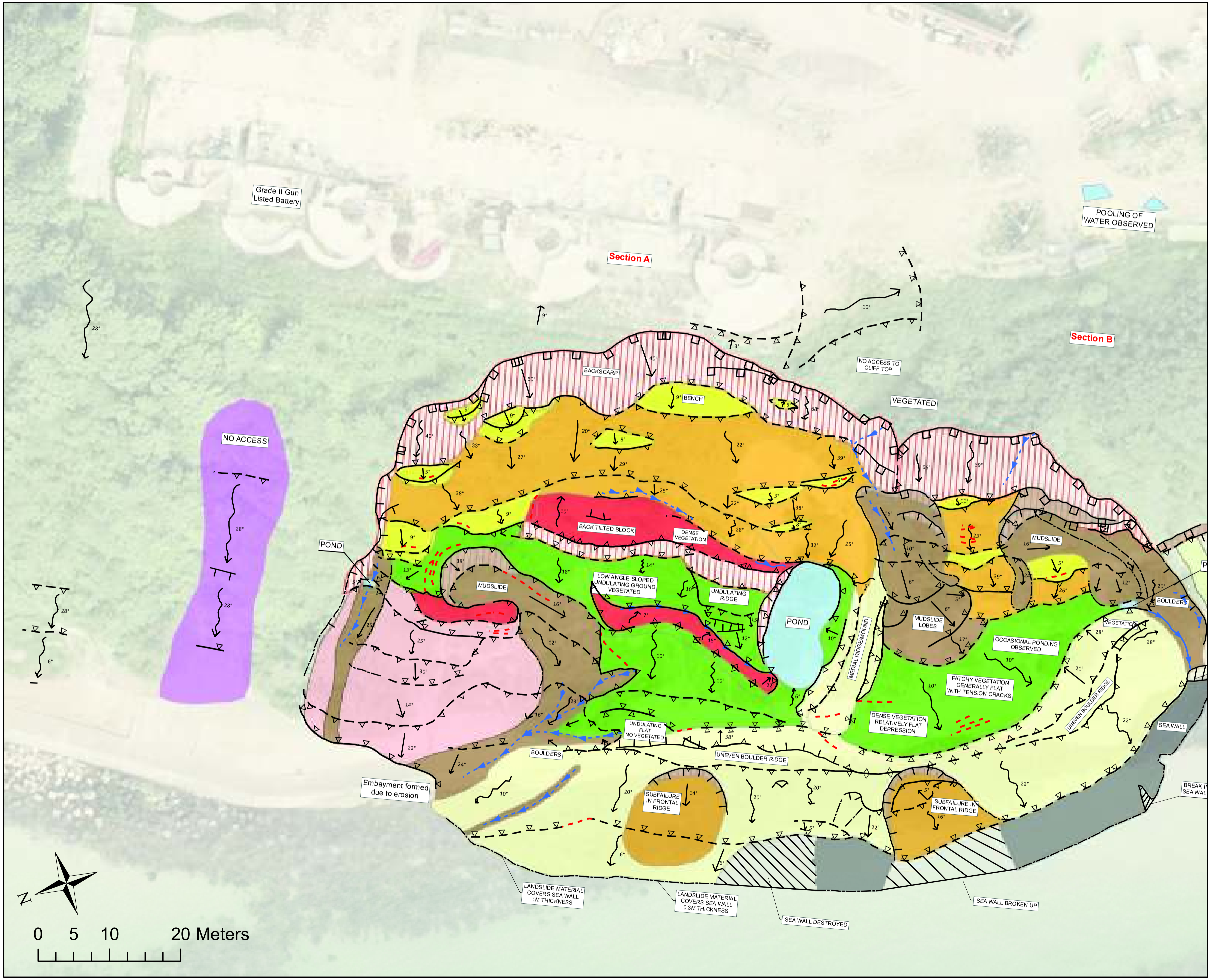


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# Appendix D. Ground Models

Figure D.1: The position of the cross sections used for the ground models

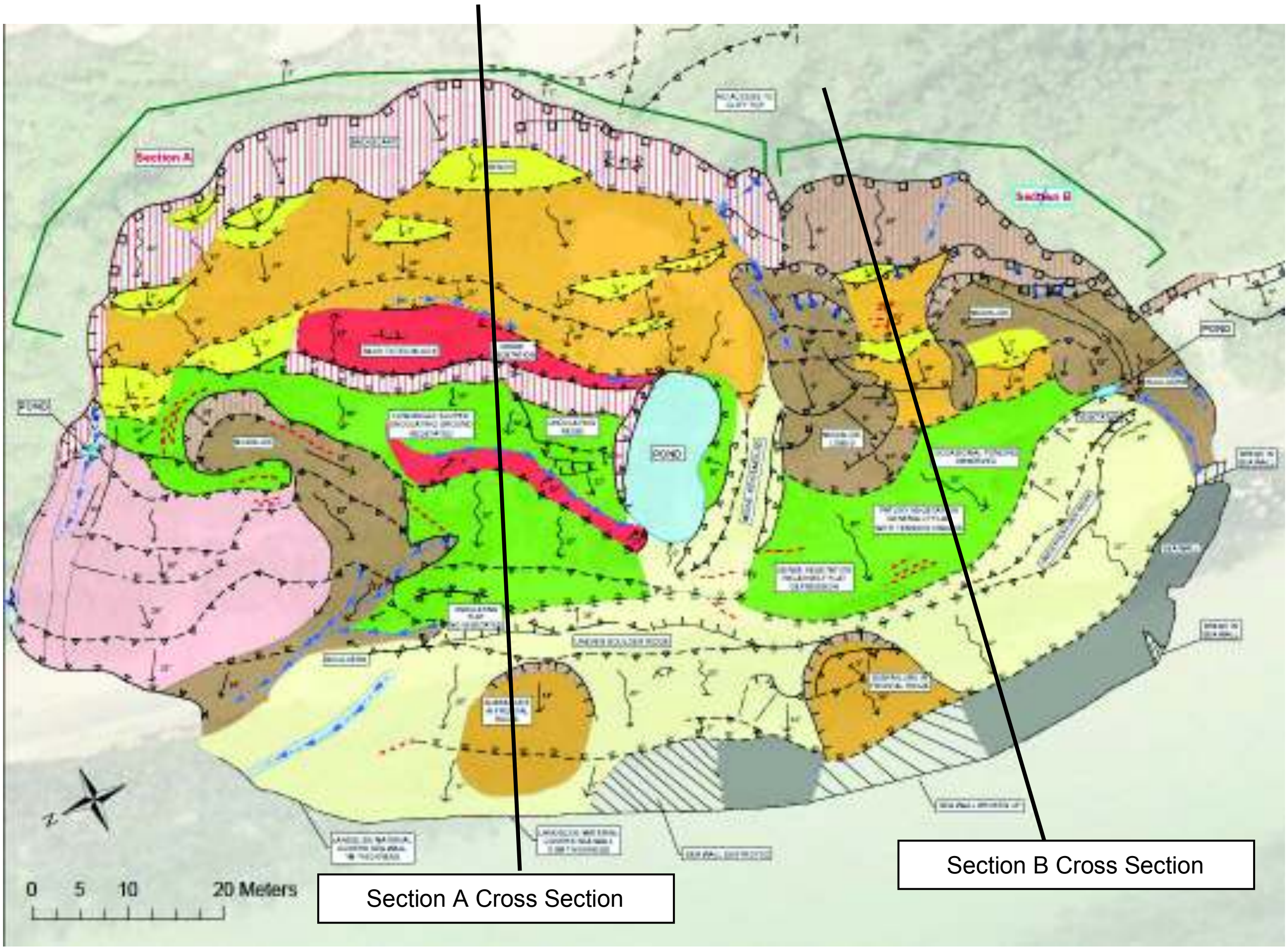


Figure D.2: Ground Model of Section A

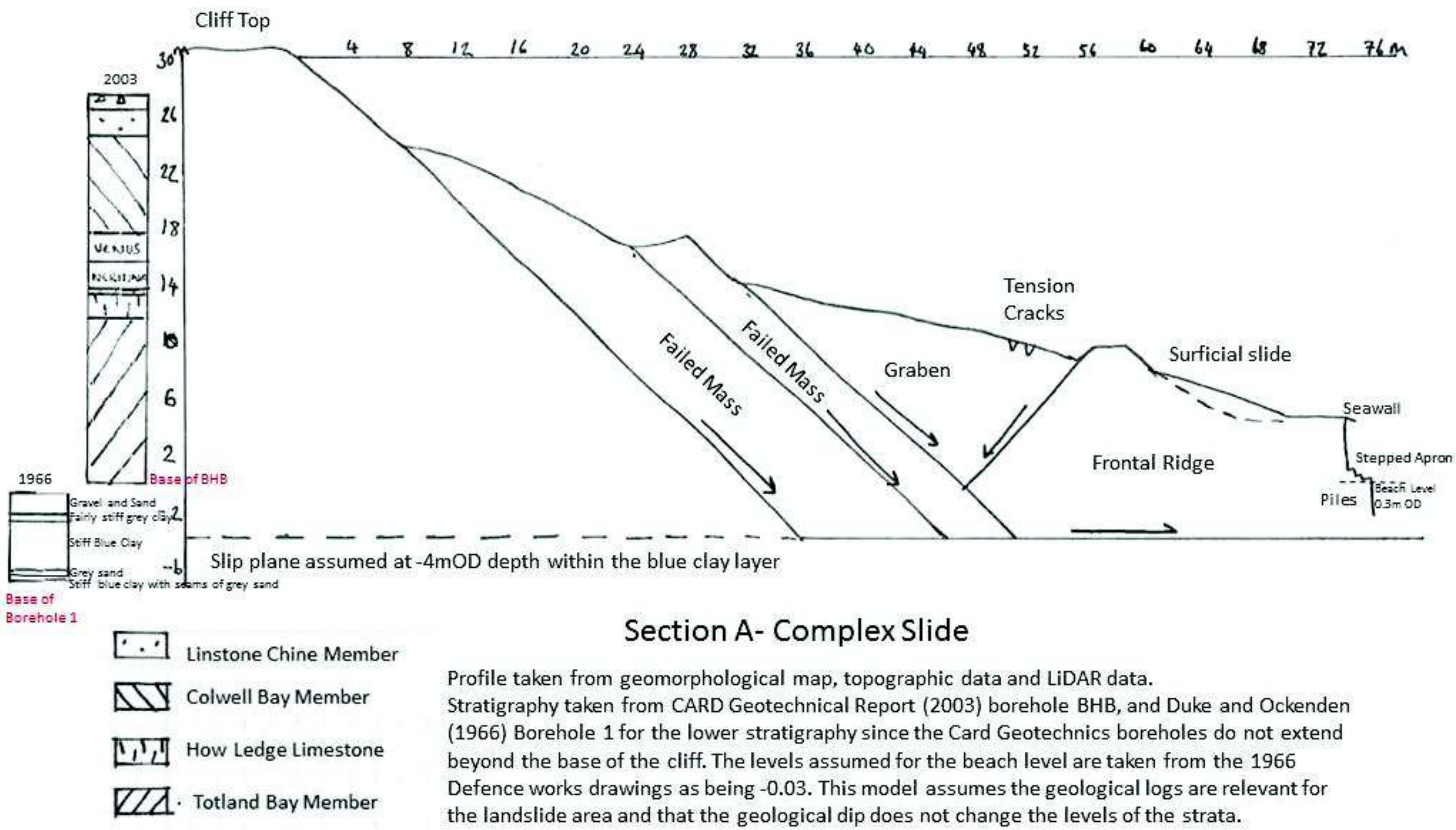
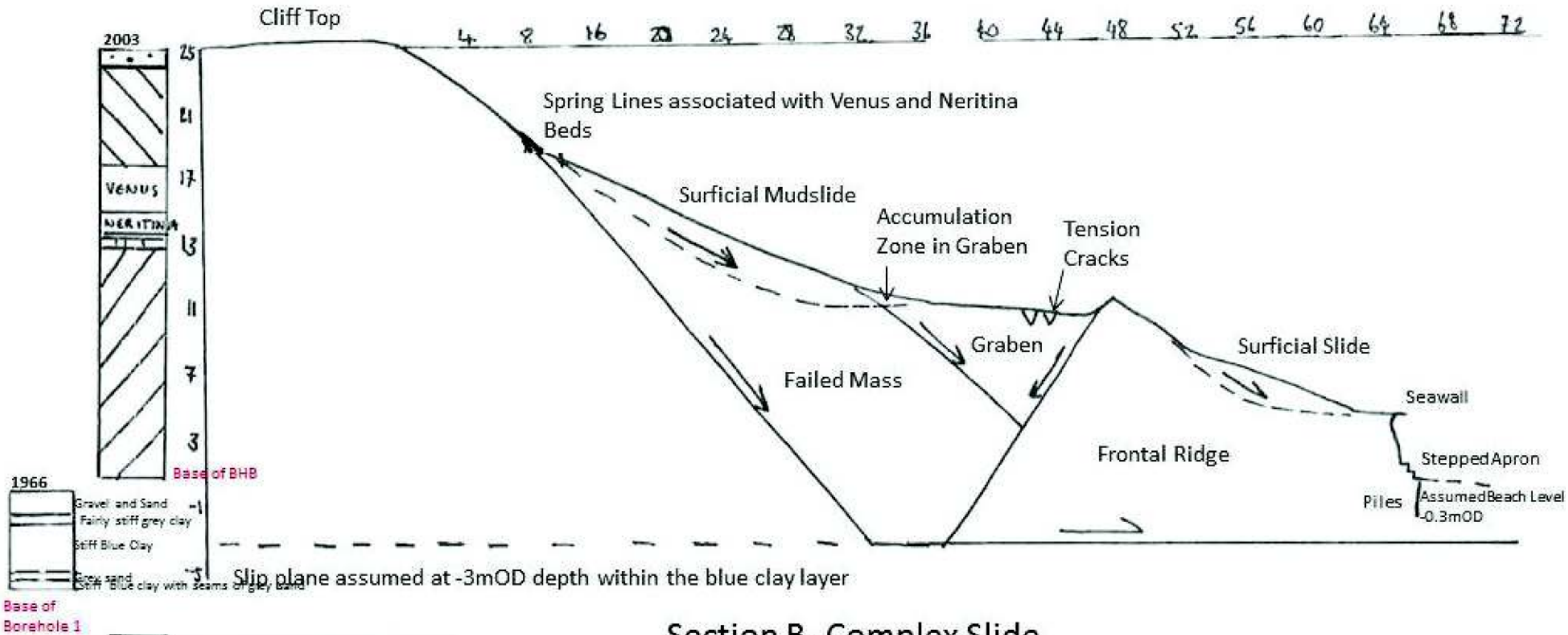


Figure D.3: Ground Model of Section B




Linstone Chine Member

Colwell Bay Member

How Ledge Limestone

Totland Bay Member

## Section B- Complex Slide

Profile taken from geomorphological map, topographic data and LiDAR data.
Stratigraphy taken from CARD Geotechnical Report (2003) borehole BHB, and Duke and Ockenden (1966) Borehole 1 for the lower stratigraphy since the Card Geotechnics boreholes do not extend beyond the base of the cliff. The levels assumed for the beach level are taken from the 1966 Defence works drawings as being -0.03. This model assumes the geological logs are relevant for the landslide area and that the geological dip does not change the levels of the strata.

# Appendix E. Erosion Line Assessment

The existing coastal defences have been assessed in terms of their risk of coastal erosion under a 'Do nothing' scenario i.e. assuming that no maintenance takes place. The key aim of considering the 'Do nothing' scenario is to understand the potential changes to the coastline from the current and future coastal processes. This is linked to an understanding of historical erosion and sea level trends, in addition to other examples in similar coastal geomorphological settings and applying these to the protection afforded by the current condition of structures.

The robustness and accuracy of 'Do nothing' scenarios are influenced by many factors, which include, but are not limited to;

- The availability of historical trend analysis for the specific frontages under consideration;
- The availability of cliff and general recession scenarios that could be similar to those viable at Totland;
- The accuracy of condition survey results for existing structures;
- Knowledge of the nature and distribution of materials itself and how these may react to on-going failure and erosion in the future;
- Records of the location and value of assets that would be impacted by erosion over time;
- Future events and conditions that are the forcing mechanisms for coastal erosion i.e. water levels, climate change, storm events etc.;

This assessment has assumed that the defences will fail within the first year because there is a chance of a landslide occurring behind the defence at any time, which will cause the wall to fail; therefore the whole frontage has a residual life of zero.

Once the defences have failed, further erosion of the coast occurs. Erosion rates have taken into account the likely geomorphological behaviour of the land behind including different geologies and geomorphologies (limestone cliffs, made ground or industrial fill) and different exposures to wave conditions.

Coastal erosion/retreat has been projected into the future based on recession scenarios of the coastline. From this assessment we have identified when various assets in terms of property, buildings, roads, utilities and environmental features are impacted.

Specific rates for shoreline erosion have been identified using the Historical Trend Analysis Rule (HTAR). The HTAR is a model relating the rate of shoreline retreat to the rate of sea level rise (Bray et al, 1992). In order to effectively calculate the potential extent of future erosion, the coastline was divided into several sections based on current residual life of the defences and historical rates of retreat.

Future shoreline retreat rates have been estimated using the HTAR equation below:

$$\mathbf{R_2= (R_1/S_1).\ S_2}$$

Where: $S_1$= *historical sea-level rise rate (m/yr.)*

$S_2$= *future sea level rise rate (m/yr.)*

*$R_1$= historical retreat rate (m/yr.)*

*$R_2$= future retreat rate (m/yr.)*

The HTAR is a commonly used approach for the assessment of shoreline retreat over defined periods. This approach however is relatively simplistic as the HTAR assumes that sea level rise is the dominant cause of coastal recession and other factors such as the wave climate remain constant. In reality it is likely that wave heights and potential energies will increase with climate change. However this is a valid assumption to make during periods of rapid relative sea level rise (Bray et al, 1992). Results also indicate that the shoreline erosion occurs every year whereas in reality there may be a time lag where the coastal profile is adjusting to new wave conditions.

The HTAR also ignores the influence of longshore drift (and therefore potential accretion) but it has been argued that the HTAR is calibrated to local coastal processes and environments due to the use of location specific historical erosion rates as inputs (Bray et al, 1992). Other authors (e.g. Leatherman, 1988) also suggest that HTAR analysis may actually cause an underestimation of future shoreline erosion due to lack of consideration of longshore drift i.e. longshore drift would cause sediment to be moved down drift and away from the frontage, thereby exposing the frontage further.

## E.1 Calculation Inputs

### E.1.1 Historical sea level rise (S1)

For the whole study area the rate of historical sea level rise (S1) has been taken to be 2.26mm/year based on mean sea level data from 1997-2012 at recorded at Bournemouth from the Permanent Service for Mean Sea Level website.

### E.1.2 Future sea level rise (S2)

Future sea level rise rates ($S_2$) are based on the UKCP09 data (95th percentile) outputs as recommended in the Environment Agency guidance (EA, 2011). Retreat rates have been calculated under a high, medium and low emissions scenario. Estimated sea level rise is summarised in table 3.1 below.

Table 8.7: Projected sea level rise at Totland over the next 100 years (medium emission scenario)

| Year | Cumulative sea level rise (medium scenario) (m) |
|---|---|
| 2015 | 0.119 |
| 2040 | 0.267 |
| 2065 | 0.441 |
| 2090 | 0.642 |
| 2115 | 0.787 |

Source: UKCP09 User Interface

### E.1.3 Historical Retreat Rate ($R_1$)

The historical retreat rate ($R_1$) which has been used to calculate the future retreat rate ($R_2$) was 0.5m/year. This is an average of all the suggested rates of retreat for the Totland area, using figures obtained in the SCOPAC (2004) report. These figures have been used because the frontage has been protected by some form of wall since the early 20th century so there is a general lack of information regarding local historical erosion rates. Hence the average has been taken of all the different sources to make assumptions about realistic erosion rates.

### E.1.4 Key assumptions in the erosion assessment

- It is assumed that the first 5 years of erosion after defence failure are the greatest as the cliff tries to regain equilibrium.
- It is assumed that within the whole section the defences will fail within the next year because there is a possibility of the landslide occurring at any time.
- It is assumed that the scheme will start in 2015 due to further designs being required and funding being obtained, so the first year for calculations will be from 2015.

## E.2 Calculation Outputs

### E.2.1 Future Retreat Rates

Table E.8 Shows a summary of cumulative retreat for each unit individually.

Table E.8: Cumulative Shoreline retreat (m) over the next 100 years

| Year | Cumulative Shoreline Retreat |
|---|---:|
| 2015 | 0 |
| 2020 | 13.24 |
| 2025 | 19.41 |
| 2030 | 25.81 |
| 2035 | 32.65 |
| 2040 | 39.71 |
| 2045 | 46.76 |
| 2050 | 54.26 |
| 2055 | 61.99 |
| 2060 | 69.63 |
| 2065 | 78.09 |
| 2070 | 86.47 |
| 2075 | 95.07 |
| 2080 | 104.12 |

| Year | Cumulative Shoreline Retreat |
|---|---|
| 2085 | 113.16 |
| 2090 | 122.43 |
| 2095 | 132.13 |
| 2100 | 142.06 |
| 2105 | 152.16 |
| 2110 | 162.51 |
| 2115 | 173.10 |

### E.2.2 Geographic information Systems Mapping (GIS) mapping

The 5 yearly retreat rates were buffered in GIS to create a map of potential erosion extent over the next 100 years

These lines were then laid over aerial photography and the amount of properties lost each 5 year interval were counted and recorded. The year in which the property is lost was then altered depending upon the erosion of access roads or services.


landwarderosion
Year
5
10
15
20
25
30
35
40
45
50
55
60
65
70
75
80
85
90
95
100
0
25
50
100 Meters