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Technical note

Project	**Runswick Bay Slope Stability**	**Date**	31st May 2013
Subject	Review of Previous Work	**Ref**	
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1 Introduction

This report collates the current understanding of the coastal erosion and slope stability issues affecting Runswick Bay. The following reports have been reviewed:

- Runswick Bay Coastal Defence Strategy Rapid Risk Assessment High-Point Rendel (1998)
- Runswick Bay Coastal Defence Strategy Study HR Wallingford (2001)
- Runswick Bay coastal Defence Strategy Study Cauldron Cliff to Kettleness Point. High-Point Rendel (2002)
- Scottish Border to Flamborough Head Shoreline Management Plan SMP2 Royal Haskoning (2007)
- Cell 1 Monitoring Programme Wave Data Analysis Report 1: 2010 - 2011 Halcrow (2011)
- Ongoing Analysis and Interpretation of Coastal Monitoring Mouchel (2012)
- Cell 1 Regional Coastal Monitoring Programme Analytical Reports, (from 2008 to present). Scarborough Council. Halcrow (various)

The reports discuss the coastal processes and slope instability affecting Runswick Bay and document monitoring of beach levels, ground movement and groundwater levels.

1.1 Area of Interest

Runswick Bay is formed between the bedrock headlands of Caldron Cliff to the north and Kettle Ness to the south and comprises a deeply indented sandy bay approximately 2 km in length that is cut in softer

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glacial sediments. The margins of the bay are backed by steep cliffs of Jurassic shale and sandstone while its centre is backed by less-steep slopes of superficial glacial sediments that are deeply incised by streams. Both the glacial sediments and the bedrock are prone to instability and thick sequences of landslide debris have been commonly encountered.

The village of Runswick Bay is developed between the valleys of the Runswick and Nettledale Becks in the north-western part of the bay. Most of the eastern part of the village is founded on weathered shale and associated landslide debris. Properties further west and the access road (Runswick Bank) and car parks are founded on glacial sediments that have been affected by landsliding to a depth of many metres. The village is fronted by four separate sea defences, of varying age and construction, which stretch from Runswick Beck north of Caldron Cliff south to Nettledale Beck.

1.2 Timeline of Previous Instability and Management/Monitoring

Runswick Bay has a long history of slope instability, the first recorded slope failures occurred in 1682 when the whole village, located further north than at present, collapsed towards the shore. Successive landslips of varying severity occurred in 1873, 1953 and, in 1958 when the old road was closed twice in one week due to landslides. This road was abandoned in 1961 with the construction in 1961 and 1963 of a new access road on its present alignment further to the west. Around the same time a sea wall extension and new car park were constructed at the base of this road. Landslips and rock falls were experienced immediately north of the village during the 1970’s, including a landslip at Rose Cottage in 1975, resulting in the loss of various assets.

A mass concrete sea-wall constructed in 1970 provided coastal protection to the southern edge of the village, access road and car park areas. Since its construction, the sea-wall was subjected to a combination of marine and land based erosional mechanisms causing the wall to move in a seaward direction with backwards rotational tilting. Sea-wall deterioration and failure has been caused by earth pressure loading from slope failures behind the wall, beach erosion exposing the toe of the wall and wall toe failure of the fractured and folded shale bedrock (Mouchel 2012).

1.2.1 Recent Instability and Management

In 1998 the sea defences were at risk of collapse. High-Point Rendel was commissioned to inspect the defences and design a new seawall. They carried out a Rapid Risk Assessment in 1998 due to the rapidly deteriorating condition of the sea wall and concerns about an area of landsliding that extended a distance of 300m inland from the rear of the sea wall at the time. There were concerns about the southern area of the village, car park and the only public highway to the village.

Following the Raid Risk Assessment a slope stabilisation and coastal defence scheme was built.

1.2.2 Coastal Management

A Strategy Study was produced by High-Point Rendel in 2002. The strategy study aimed to provide a plan for the prioritisation of future capital schemes, monitoring and maintenance programmes. The coastal defence strategy and associated action plan were developed to ensure that once implemented the existing defences at Runswick Bay remain sustainable over a 50 year period. The action plan was to be subject to review at 5 year intervals throughout the life of the strategy.

The Shoreline Management Plan was reviewed and an SMP2 for Runswick Bay was produced in 2007 by Royal Haskoning.

1.2.3 Monitoring

Following the construction of the scheme two monitoring regimes were set up, Mouchel for monitored any ground movement and Halcrow carried out coastal monitoring

1.2.3.1 Mouchel

In October 2008, Mouchel were instructed by Scarborough Borough Council (SBC) to provide services relating to an Analysis and Interpretation of Coastal Monitoring Data from sites (Runswick Bay, Whitby, Scalby Ness, Scarborough North and South Bay, Knipe Point, Killerby, Filey Town & Brigg and Filey Flat Cliffs) along the North Yorkshire coastline. Mouchel were required to review, analyse and interpret existing data for all the sites mentioned above. The data covered previous plans, monitoring records, strategies, ground investigations, borehole records, groundwater information, laboratory test data and geomorphological mapping.

Site specific monitoring regimes have been planned to take place at intervals of one, two, three and six months beginning in July 2009. The report reviewed for this work described and detailed the findings of the Seventh Full Suite monitoring event undertaken, in late May and early June 2012, as part of the monitoring regime recommended in a previous report of March 2009.

1.2.3.2 Halcrow

The Cell 1 Regional Coastal Monitoring Programme covers approximately 300km of the northeast England coastline, from the Scottish Border (just south of St. Abb's Head) to Flamborough Head in East Yorkshire. This coastline is often referred to as 'Coastal Sediment Cell 1' in England and Wales. The work commenced with a three-year monitoring programme in September 2008 that was managed by Scarborough Borough Council on behalf of the North East Coastal Group.

The main elements of the Cell 1 Regional Coastal Monitoring Programme involve:

- beach profile surveys
- topographic surveys
- cliff top recession surveys
- real-time wave data collection
- bathymetric and sea bed characterisation surveys
- aerial photography
- walk-over surveys

Runswick Bay has a six monthly topographic survey to assess the fluctuation in beach level.

2 Coastal Processes

The replacement of the sea wall following the Rapid Risk assessment was based on the wave climate assessment carried out by HR Wallingford in 2011. The report looked at joint probability, inshore wave climate, beach behaviour and overtopping on the Runswick frontage.

More recently as part of the North East Coastal Monitoring programme a review was carried out on hydrocyanics by Halcrow (2011). A wave buoys were operational between 20/05/2010 and 04/11/2011.

2.1 Tides

There is a tide gauge at Whitby that is operated continuously by the National Tide and Sea Level Facility (NTSLF) on behalf of the Environment Agency as part of the main UK tide gauge network. Information on this tide gauge installation is available on the NTSLF website: http://www.ntslf.org/tgi/portinfo?port=Whitby including the site history reproduced below. The Chart datum at Whitby is 3m below Ordnance Datum (http://www.ntslf.org/tides/datum).

Table 2-1 Predicted tide levels at Whitby

Tidal State	Level (m Chart Datum)	Level (m Ordnance Datum)
HAT	6.21	3.21
LAT	0.22	-2.78
MHWS	5.59	2.59
MHWN	4.50	1.50
MLWN	2.25	-0.75
MLWS	0.99	-2.01
Highest predicted 2013	6.03	3.03
Lowest predicted 2013	0.41	-2.59
Highest predicted 2014	6.17	3.17
Lowest predicted 2014	0.32	-2.68

Note: Based on data from http://www.ntslf.org/tgi/portinfo?port=Whitby

Data is available on the internet in real time (http://www.ntslf.org/data/realtime?port=Whitby) and quality controlled data can be downloaded from the British Oceanographic Data Centre (BODC) website (Halcrow 2011)

2.2 Waves

The design of the replacement sea wall was based on work carried out by HR Wallingford on behalf of High-Point Rendel. The information collated by Wallingford was used to assess wave conditions at the toe and the current overtopping performances of the existing coastal defences and newly constructed rock armour structure situated long the village frontage.

The analysis identified that the waves from the north north east exhibit the most severe offshore wave conditions for return periods of 1, 10 and 50 years. Inshore wave conditions corresponding to offshore extremes were subsequently derived for wave refraction points located along to -18m OD bathymetric contour. The inshore wave directions for off shore waves from the north and north north east were established at 11 degrees north and 29 degrees north respectively.

2.2.1 Wave Monitoring

More recent work by Halcrow involved the analysis of the wave data collected from 2010 to 2011 as part of the North East monitoring programme.

There are two wave buoys at Whitby, one inside the outer breakwaters (WV#1) and one outside the harbour walls (WV#2). Because the second Whitby wave buoy is in a more exposed, open coast position, it is likely to be analogous to the wave climate experienced in Runswick Bay.

The wave rose for the buoy outside of the outer Whitby breakwater is shown in Figure 2-1

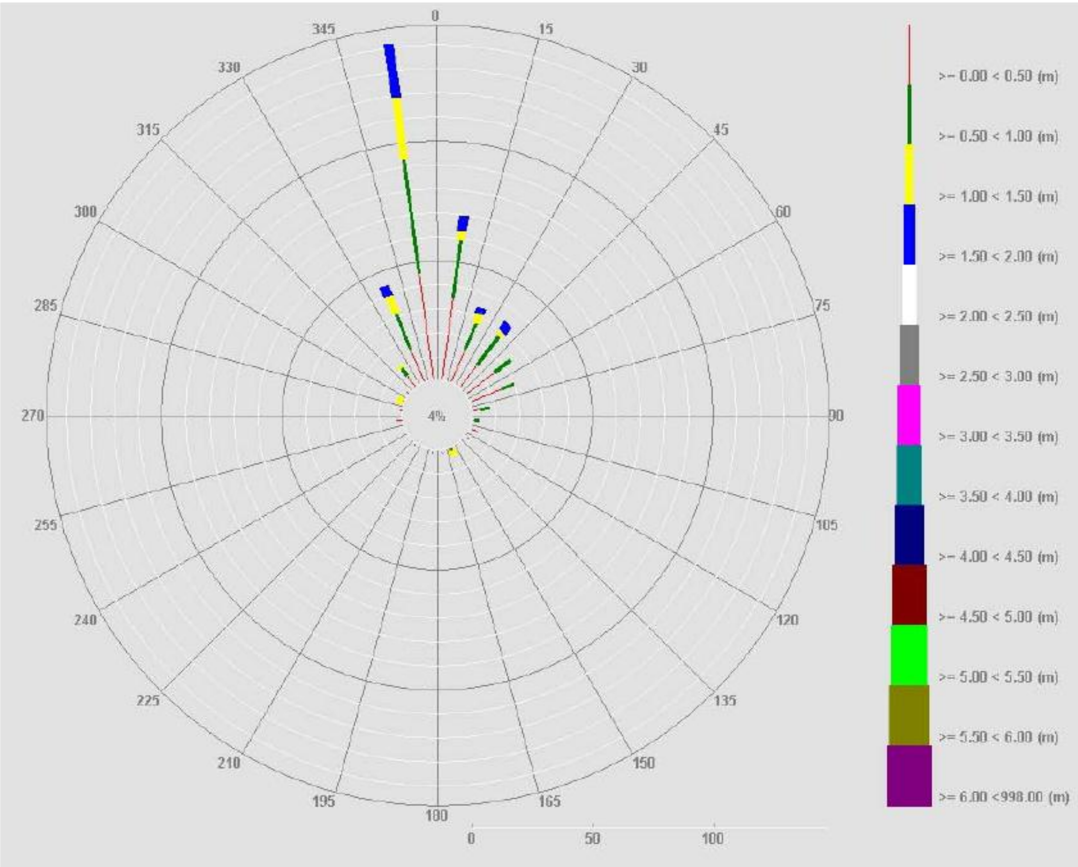


Figure 2-1 Wave Rose for Offshore Wave Height at Whitby East Pier (WV#2)

2.2.2 Overview

Both the Rendel-commissioned HR Wallingford work and the more recent data collection show that the predominant wave direction is north north east. The data collection took place for around 18 months, so they should not be taken as necessarily representative of the wave climate at Runswick Bay.

2.3 Storms

As part of the Halcrow 2011 tidal review, a storm analysis of the Whitby data set was undertaken using a wave height threshold of 4m and a storm separation threshold of 120 hours. The period of data examined ran from 20/05/2010 to 25/10/2011 and the results are presented in Table 2-2 below. The storms mostly arrive from the North to East-Northeast (5 to 66 degrees). The storm with the largest wave height (5.1m Hmo) at peak was on 25th September 2010, whist the storm with greatest wave energy at peak was on 20th June 2010.

Table 2-2 Storm Analysis Results for Whitby

General Storm Information							At Peak			
Start Time	**End Time**	**Duration (Hours)**	**Peak of Storm**	**Mean Dir(°)**	**No of Events (30 min dataset)**	**Mean Dir Vector (°)**	**Hs (m)**	**Tp (s)**	**Dir (°)**	**Energy@ Peak (KJ/m/s)**
19/06/2010 08:30	20/06/2010 07:30	23	20/06/2010	26	38	64.52	4.89	11.36	28	6085.33
29/08/2010 16:30	30/08/2010 02:30	10	29/08/2010 17:30	5	7	85.09	4.39	8.00	6	2429.65
17/09/2010 09:00	17/09/2010 11:00	2	17/09/2010 11:00	28	2	67.50	4.39	11.30	22	4853.93
24/09/2010 05:30	**26/09/2010 03:30**	**46**	**25/09/2010 17:00**	**23**	**67**	**66.86**	**5.06**	**10.24**	**28**	**5298.61**
09/11/2010 03:30	09/11/2010 19:00	16	09/11/2010 05:30	66	19	24.89	4.72	9.25	68	3755.58
29/11/2010 20:00	02/12/2010 01:00	53	29/11/2010 22:00	60	19	31.32	4.72	9.93	56	4327.96
23/07/2011 15:30	24/07/2011 11:00	20	24/07/2011 03:00	29	29	61.86	4.24	10.77	22	4114.25

2.4 Extremes

Hr Wallingford produced estimates of extreme water levels for return periods of 1, 10, and 50 years together with the wave height and direction distributions were correlated to provide estimates of extreme wave heights and water levels for given joint return periods of 1,10 and 50 years.

Table 2-3 Joint probability of waves over the -18mOD at Staithes (HR Wallingford 2001)

Table 2 Joint probability of waves over the -18mOD at Staithes (Reference 1)

Joint return period (years)	Water level return period (years)	Water level (m OD)	Wave height return period (years)	Wave height (m) for each direction sector: N (11°N)		N/NE(29°N)		NE	
				H_s	T_p	H_s	T_p	H_s	T_p
1	0.025	2.75	1.0	6.9	12.1	5.1	10.1	4.3	9.1
	0.05	2.85	0.5	6.3	11.6	4.5	9.5	3.7	8.4
	0.1	2.95	0.25	5.8	11.1	3.9	8.8	3.3	7.9
	0.25	3.1	0.1	**4.9**	**10.2**	**3.0**	**7.7**	2.3	6.6
	0.5	3.2	0.05	**4.2**	**9.5**	**2.3**	**6.8**	1.9	6.0
	1.0	3.3	0.025	**3.8**	**9.0**	**1.6**	**5.7**	1.3	5.0
10	0.05	2.85	10.0	9.0	13.9	7.3	12.1	6.1	10.8
	0.1	2.95	5.0	8.1	13.2	6.7	11.6	5.6	10.3
	0.5	3.2	1.0	6.9	12.1	5.1	10.1	4.3	9.1
	1.0	3.3	0.5	**6.2**	**11.5**	**4.5**	**9.5**	3.7	8.4
	5.0	3.52	0.1	**4.9**	**10.2**	**3.0**	**7.7**	2.3	6.6
	10	3.61	0.05	**4.2**	**9.5**	**2.3**	**6.8**	1.9	6.0
50	0.12	3.0	50	10.5	15.0	8.8	13.3	7.4	11.9
	0.25	3.1	25	9.9	14.5	8.3	12.9	7.0	11.6
	0.6	3.25	10	9.0	13.9	7.3	12.1	6.1	10.8
	1.0	3.3	6.0	8.5	13.5	6.9	11.8	5.8	10.5
	6.0	3.55	1.0	6.9	12.1	5.1	10.1	4.3	9.1
	10	3.61	0.6	**6.2**	**11.5**	**4.6**	**9.6**	3.9	8.6
	25	3.77	0.25	**5.9**	**11.2**	**3.9**	**8.8**	3.2	7.8
	50	3.85	0.12	**5.1**	**10.4**	**3.3**	**8.1**	2.6	7.0
100	0.15	3.05	100	11.1	15.4	9.4	13.7	8.0	12.4
	0.3	3.2	50	10.5	15.0	8.8	13.3	7.4	11.9
	0.6	3.25	25	9.9	14.5	8.3	12.9	6.9	11.5
	1.5	3.3	10	9.0	13.9	7.3	12.1	6.1	10.8
	10	3.61	1.5	7.2	12.4	5.6	10.6	4.7	9.5
	25	3.77	0.6	**6.2**	**11.5**	**4.6**	**9.6**	3.9	8.6
	50	3.85	0.3	**6.0**	**11.3**	**4.0**	**8.9**	3.3	7.9
	100	3.99	0.15	**5.2**	**10.5**	**3.5**	**8.4**	2.7	7.2

The water levels shown in Table 2-3 were the basis of the overtopping analysis for Rendel's Rapid Risk Assessment.

2.5 Sea Level Rise

2.5.1 Sea Level Rise Projection used in the Strategy Study

The basis of the slope stability assessment was the HR Wallingford study for High-Point Rendel. They considered that future extreme water levels were estimated by increasing present day extreme predictions by an amount equal to the predicted future rise in sea level. MAFF guidelines recommend an allowance for future mean sea level rise of 4mm/yr for the Whitby area. For the purpose of this strategy a life of 50 years have been assumed, which corresponds to a sea level rise of 200mm.

The High-Point Rendel strategy used MAFF sea level rise guidance of 4mm/yr over the next 50 years. This should be compared to the most recent guidance and the sea level rise for 100 years. The change is sea level is likely to be accompanied by secondary effects, such as increased storminess.

2.5.2 Most up-do Date Sea Level Rise Scenario

The UKCIP09 projections have been reviewed to assess the most recent sea level rise projections. The data was extracted from theUKCIP09 website for Runswick Bay and the medium emissions scenario. The predictions are based on a spread of probabilities for reactions of sea level to climate change. The central estimate is considered to be the most likely.

The recent projections for the period between 2002 and 2052 are shown below for comparison with the High-Point Rendel figures.

Table 2-4 Comparison between Sea Level Rise Predictions

	MAFF 2001	UKCIP09 Medium Emissions		
Scenario	**High-Point Rendel**	**Low Estimate (5%ile)**	**Central Estimate (50%ile)**	**High Estimate (95%ile)**
Increase in Sea Level from 2002 to 2052	0.2m	0.08m	0.18m	0.28m
Increase in Sea level from 2002 to 2100	0.4m*	0.19m	0.42m	0.66m

* this is an estimate based on an assumption of a linear relationship between the High-Point Rendel 50 and 100 year projection

The High-Point Rendel and UKCIP09 ‘central estimate’ emissions scenario predictions are very similar to 2052. The High-Point Rendel Strategy did not take account of a 100 year time span so the sea level rise is likely to be double the design parameters of the rock armour by 2100.

3 Beach Characteristics and Evolution

3.1 Sediment Transport

HR Wallingford stated that littoral drift is the prevalent sediment transport method in the study area. Movement of sand and shingle along the Runswick Bay frontage takes place primarily as a result of wave action. Waves breaking at an angle to the beach contours generate a current parallel to the shoreline. Where this current reaches the Kettleness headland at the southern boundary of the study area it is deflected seawards. In this manner sediments that are swept along the shoreline can also be diverted offshore by the deflected current. The coast to the east of Runswick Bay, beyond Kettle Ness headland shows little in the way of beach deposits, suggesting that the wave action along the coast outside the embayment is too high to allow sediments to settle (High-Point Rendel 2002).

In terms of general distribution of sediment within Runswick Bay, beach movement is dominated by the process of offshore/onshore transport, while littoral transport serves to redistribute material within the bay. The beach tends to show seasonal variation in its profile. Typically in winter beach levels are lower than in summer. The sediments are drawn down by larger waves to produce a flatter beach profile. In the summer material lying on the nearshore seabed s transported landward by more constructive wave action resulting in the formation of a steeper beach profile.

More recent work by Halcrow (2013) has shown that the wave cut platform has a huge influence on coastal behaviour. There is a thin veneer of beach sediment which moves over the wave cut platform in response to waves and currents. The fluctuations in the position and volume of the veneer of sediment have an influence on the beach within Runswick Bay. The topographic surveys carried out for the monitoring programme show that erosion of the beach has been noted over the summer of 2011 and 2012, which is not expected. There were a few areas of accretion around the outcropping rocks. High-Point Rendel (2002) considered that significant landslide events such as a 70,000 tonne slip that occurred in Kettleness Village in 1999, may affect the sediment transport pattern for a few months until the landslide debris has been removed by coastal processes.

3.2 Future Sediment Transport and Budget

Much of the foreshore of Runswick Bay has remained relatively stable over the last 100 years. This is probably due to a combination of onshore sand transport and some influx of material as a result of landsliding and cliff erosion. The areas showing greatest sediment fluctuation are to the north and south of the lifeboat station and in the vicinity of the sailing club near Hob Hole. It is in these areas, particularly by the lifeboat station where consideration must be given to ensuring that high beach levels are maintained. If beach levels are kept high the toe of both the concrete revetment and composite masonry/concrete walls fronting the village will be protected from long term scour (High-Point Rendel 2002)

The coastal monitoring programme (Halcrow 2013) does not considerer future trends in the beach or cliffs. However, a key consideration will be the volume of beach sediment which is mobile within the bay. The amount of sediment available from the parts of the bay which have a No Active Intervention policy needs to outweigh the amount of sediment moved offshore during storm events and other erosive events.

4 Slope Instability

4.1 Introduction

Much of the work carried out on slope stability such as the mapping, slope modelling and the description of the geomorphological units was carried out by High-Point Rendel as part of their Rapid Risk Assessment (1998). The rapid risk assessment was commissioned because the sea defence at Runswick Bay was at risk of failing. The defence was constructed between 1969 and 1971 (defence code 240/6510). Within a few years of completion of the sea wall in the 1970's significant structural cracking and displacement of the wall had become evident. Much of the upper beach sands have been eroded exposing the very weak shale rock platform at an approximate level of 0.6m OD.

At the time of the Rapid Risk Assessment there had been acceleration in ground movement during spring 1998. The sea wall, having suffered two major dislocations, was articulated and experiencing a complex east north-easterly movement coupled with counter-clockwise rotation in plan. In the vertical plane the toe of the wall has risen and is displaced forward relative to the crest line, causing backward leaning of about 15^{o}, forward movement is directly evident by shearing and more than 4m displacement at the north end along the slipway. Greater movement in the central and southern segments of the wall is compressing and rotating the northern detached element. Along the length of the wall the present crest level rises by more than 1m towards the south. The indicative toe failure mechanism is by backwards rotation as the toe and clay yields in the folded fault debris and rise up towards the present beach level.

The loss of beach level in front of the wall was exposing the highly fractured and folded fault debris/wall failure shear zones. The toe of the wall is at about the rock surface level such that there is no residual passive resistance available other than horizontal sliding friction and the backward component of the wall self weight.

It is considered that total wall failure was imminent. It was considered that the effects of storm scour on the exposed rock and wave impact loading coupled with progressive landslide displacements, could have initiate the total collapse of the wall within a matter of months, at some stage during winter 1998/1999.

High-Point Rendel (1998) considered that failure was likely to be rapid and regressive as toe weight was lost due to marine erosion and the 300m long zone of unstable tills gather forward momentum, sliding over the fractured mudstones where high groundwater pressures exist. The probable absence of the sandstone cliff immediately behind the existing back scarp will permit large, first time failures of intact glacial till. By increasing the head mass of the landslide the destabilizing forces will also increase and movement continue. Left unchecked, it was conceivable that the overall length of an active landslide mechanism could be of the order of 500m. The initial stages of ground movement that would immediately follow collapse of the sea wall would almost certainly extend throughout the length of the convex ground surface profile that exists below 30m OD. Accordingly vehicular access to and services within the village would be lost at a very early stage.

4.2 Stratigraphy

Mouchel undertook a review of site geology and stated that the published geological map of the area 1:50,000 British Geological Survey (BGS) Sheet 34 Solid and Drift Guisborough indicate the site is underlain by superficial deposits of glacial sediments. These comprise stiff silty sandy clays (tills), sands and gravels and laminated stiff silty clays. The solid succession of the area is indicated as Middle Jurassic sandstones (Saltwick Formation) and ironstones (Dogger Formation) (rocks of the high cliff headland north of the village) which lie unconformably on Lower Jurassic shales (Whitby Mudstone Formation). The shales are exposed as a wave cut platform, dipping at 2 degrees to the south at the front of the cliffs along the north of the bay. The map indicates a north-south trending fault passing beneath the village and across the upper beach area to the south, with down throw to the west (Mouchel 2012).

Stage/Sub-stage	Lithostratigraphy (Formation)	Lithostratigraphy (Member)	Notes
Quaternary – Holocene (c. 10ka BP)		Landslide debris	Widespread in northern part of Runswick Bay
Quaternary – Pleistocene (c. 20ka BP)		Tills, sands and gravels and laminated clays	Infills former valleys in centre of Runswick Bay and Port Mulgrave. Caps surrounding land and cliff tops.
Middle Jurassic – Aalenian (c. 182Ma BP)	Dogger Fm		Exposed in cliffs from Port Mulgrave to Sandsend Ness
Lower Jurassic – Toaracian (c. 182Ma BP)	Blea Wyke Sandstone Fm	Yellow Sandstone Mbr	Exposed in cliffs from Port Mulgrave to Sandsend Ness
		Grey Sandstone Mbr	Exposed in cliffs from Port Mulgrave to Sandsend Ness
	Whitby Mudstone Fm	Fox Cliff Siltstone Mbr	Exposed in cliffs from Port Mulgrave to Sandsend Ness
		Peak Mudstone Mbr	Exposed in cliffs from Port Mulgrave to Sandsend Ness
		Alum Shale Mbr	Exposed in cliffs from Port Mulgrave to Sandsend Ness
		Mulgrave Shale Mbr	Exposed in cliffs and foreshore at Port Mulgrave.
		Grey Shale Mbr	Crops out on cliffs and lower foreshore at Port Mulgrave and foreshore of Thorndale Shaft
Upper Pliensbachian	Cleveland Ironstone Fm	Kettleness Mbr	Crops out on foreshore in southern part of Runswick Bay and Kettle Ness
		Penny Nab Mbr	Crops out on foreshore in southern part of Runswick Bay and Kettle Ness

4.3 Preliminary Slope Stability Modelling from the Rapid Risk Assessment

As part of the Rapid Risk Assessment for Runswick Bay (High-Point Rendel 1998) a preliminary slope stability analysis was carried out on a surveyed section of the landslide to the west of the failed sea wall Figure 4-1. In addition to an assessment of the current global stability of the slope section, the effect on global stability of a number of proposed engineered schemes was assessed.

The analysis was carried out using the Oasys Geo SLOPE software package Version 4.7. A simplified geotechnical model considered to be best representative of the current field conditions was formed using data obtained from both recent and historical ground investigations (Figure 4-2). Janbu's method with variably inclined forces was the method of solution adopted during the analysis.

The findings of the analysis are summarised in Table 4-1.The data and output files are presented within Appendix D of the Rapid Risk Assessment Report.

The analysis revealed an increase in the global factor of safety as a consequence of the implementation of each phase of the construction works. The preliminary analysis indicates that in the long term an increase of 40% in the global factor of safety could be achieved if the proposed scheme is successfully implemented.

Table 4-1 Results from the Slope Stability Modelling

Run	Design Option	Factor of Safety
Run001	Current Global Stability	0.974
Run002	Reprofiling of Slope Short Term Drainage Achieving 2m Drawdown In Phreatic Surface	0.951
Run003	Reprofiling of Slope Short Term Drainage Achieving 2m Drawdown In Phreatic Surface Addition of Toe Rock Armour	0.980
Run004	Reprofiling of Slope Short Term Drainage Achieving 2m Drawdown In Phreatic Surface Addition of Toe Rock Armour Addition of Portal Pile Frames	1.192
Run005	Reprofiling of Slope Long Term Drainage Achieving 4m Drawdown In Phreatic Surface Addition of Toe Rock Armour Addition of Portal Pile Frames	1.364

The landslide study area represents a small proportion (approx. 10%) of the overall landslide complex at Runswick Bay. Therefore the potential for landslides in the central part of the bay reactivating landslides at the village are unknown. It is believed that ground movements associated with the landside complex to the south of the study area currently influence the stress regime and strain compatibility to the south of the study area currently influence the stress regime and strain compatibility manifested in the landslide study area.

The limitations of the two-dimensional stability analysis are such that the presence and effect of high shear stresses and strains induced by neighbouring ground movements are ignored.

During the Rapid Risk Assessment it was "proposed that during the detailed design the effect of drag down shear forces and associated strains on the long-term stability of the study area will be analysed. In addition a means of restraining shear effects and ensuring strain compatibility will be outlined". However, the detailed design was not provided to Halcrow as part of this review.

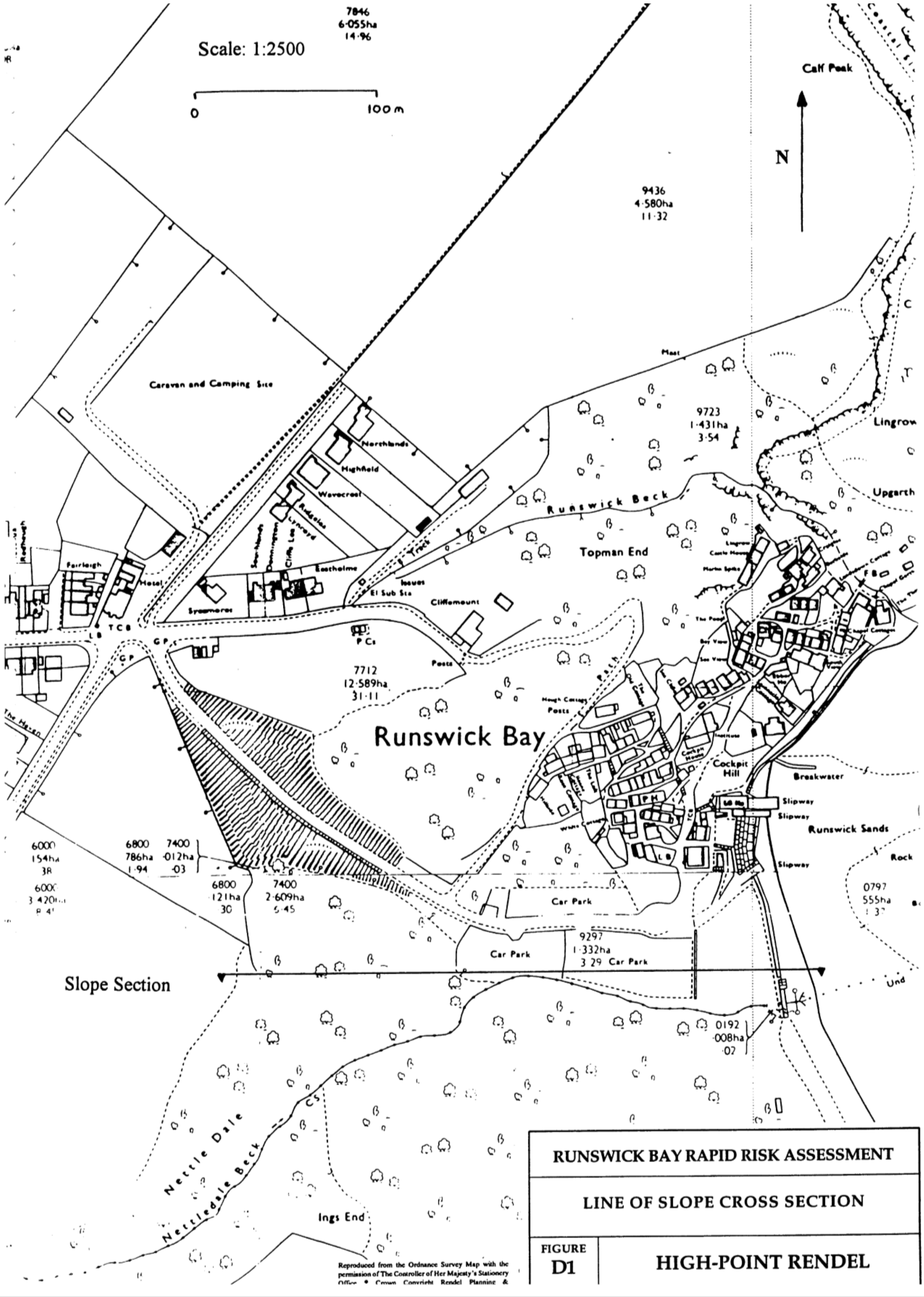


Figure 4-1 Map showing the Line of the Cross Section

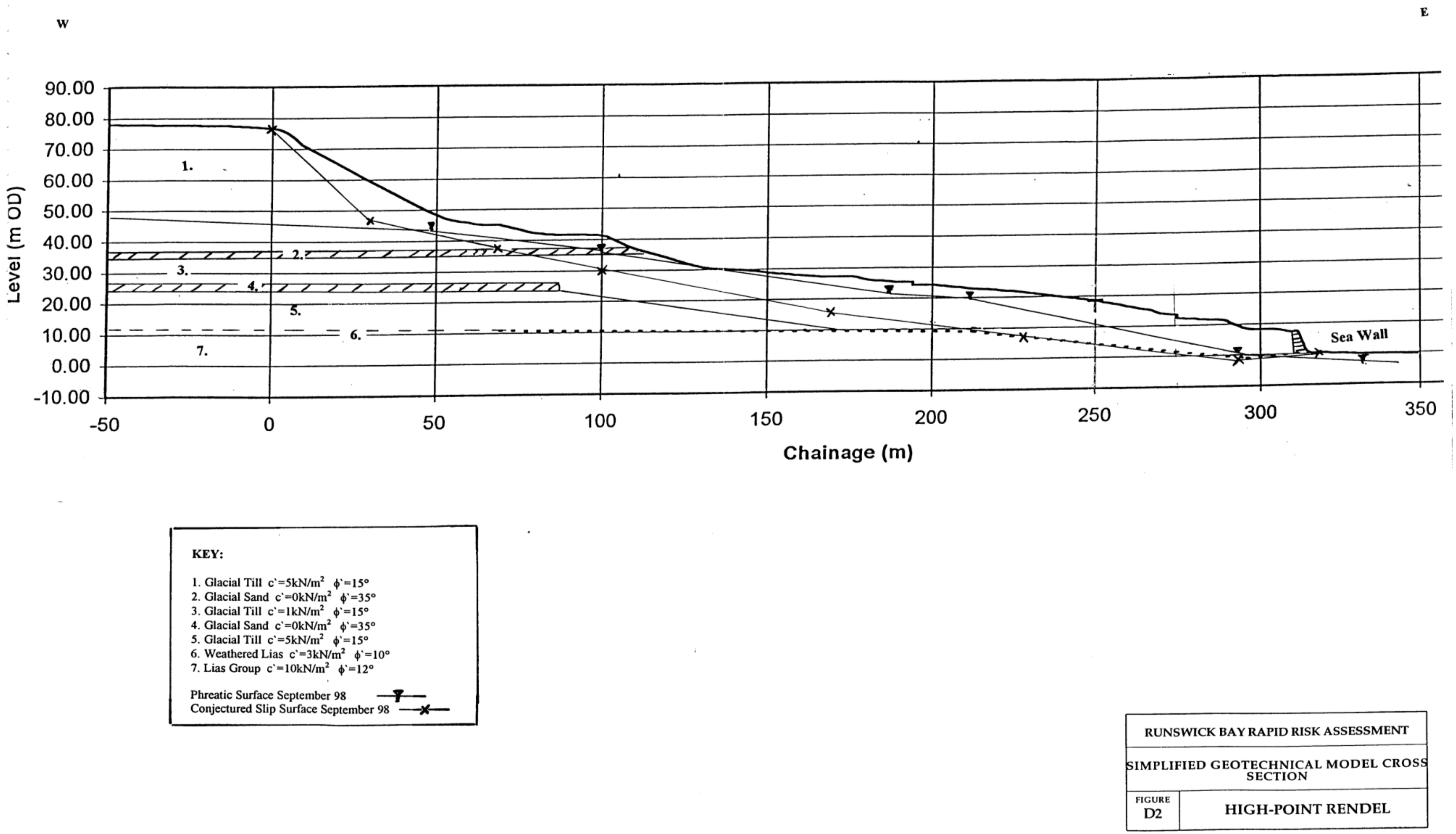
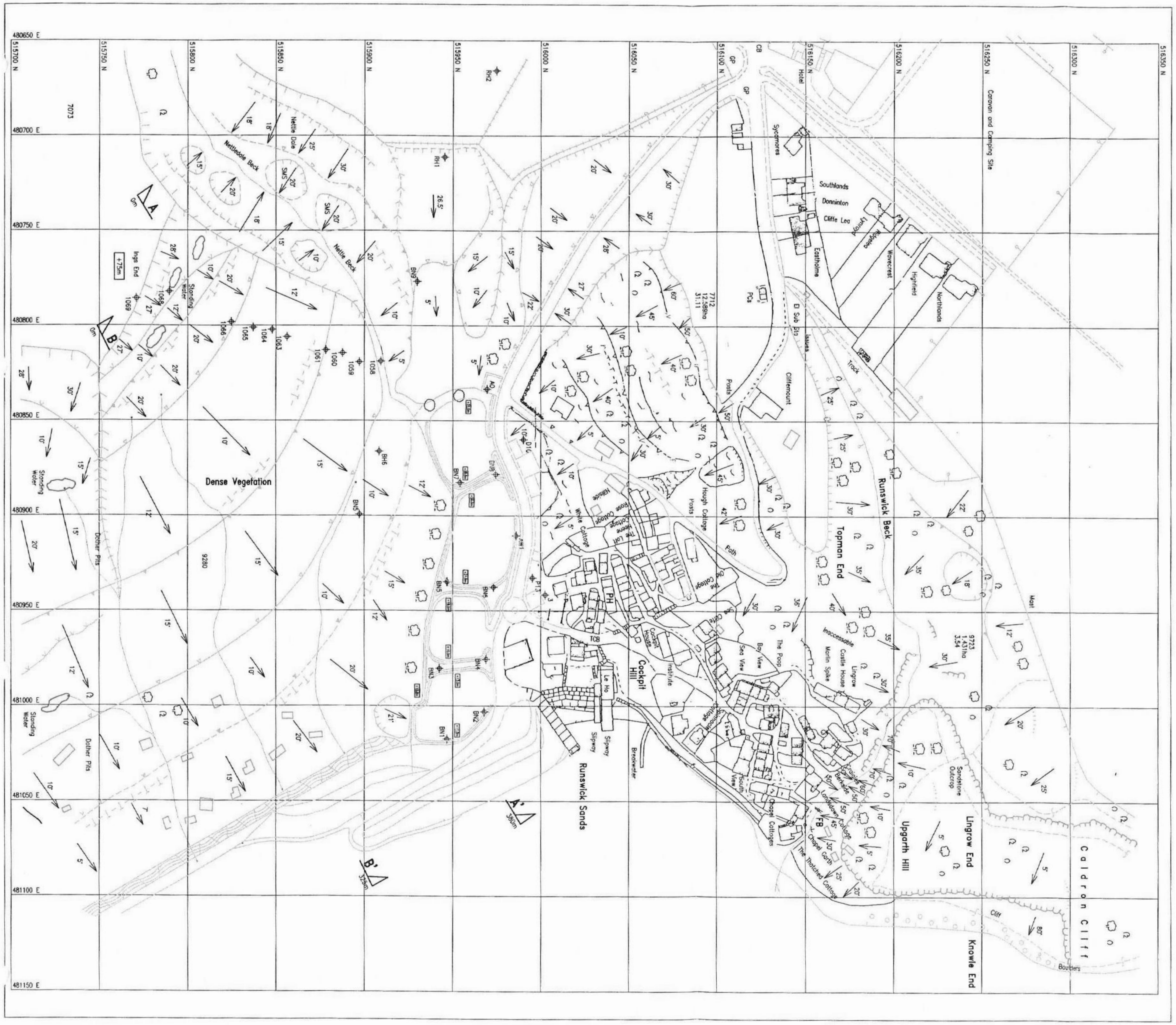


Figure 4-2 Cross Section of the Slope Stability model used by High-Point Rendel (1998)

4.4 Geomorphological Mapping

The geomophological mapping was carried out as part of the strategy study in 2002. The HPR risk assessment was based on an understanding of understanding of the local geomorphology and a ground model was developed to describe slope failure mechanisms. Routine topographic surveys and the reading of inclinometer access tubes installed in the Dother Pitts area provided quantifiable data that was used to determine the depths and magnitude of ground movement in order to verify the geometry of the failure mechanisms observed in the field.

Three slope complexes were recognised as being susceptible to future landsliding. Each pose a potential threat to private properties, infrastructure, and the recently completed coast protection and slope stabilisation works. These slopes located below Topman End and Upgath Hill to the north of the village and in the Dother Pits region to the south of the Nettledale Beck were mapped in detail. The gemorphological map is reproduced below. It is noted that the geomorphological mapping is limited to identification of slope units only, with no attempt made to interpret the landforms. Consequently, it may be beneficial to re-map some, or all, of the study area to gain a better understanding of contemporary slope processes and how they have responded to the stabilisation works of the 2002.



Notes:

1. All dimensions are in millimetres.
2. All levels are in metres related to Ordnance Datum (Newlyn).
3. All chainages are in metres.

Key

AW1 MONITORING STATIONS
TENSION CRACK
CONCAVE BREAK IN SLOPE
CONVEX BREAK IN SLOPE
SUB VERTICAL HEADSCARP
15° SLOPE ANGLE
SHALLOW SLOPE FAILURE
SUB VERTICAL SANDSTONE CLIFFS
STANDING WATER

0 10 20 40 60 80 100m
ORIGINAL SCALE 1:1000

Revision	Date	Description	Drawn	Checked	Approved
0	06/09/02	Final Issue	MEB	DH	SG

Client

SCARBOROUGH BOROUGH COUNCIL

Project

RUNSWICK BAY COASTAL DEFENCE STRATEGY STUDY

Drawing Title

GEOMORPHOLOGICAL MAP

High-Point Rendel

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Format	Scale	File Name
A1	1:1000	n://1321-P-003.dwg

Drawing Number	Revision Number
1321/P/003	0

The following geomorphological units were identified:

4.4.1 Topman End

The till slopes below Topman End are over steep as a result of historic coastal erosion and are heavily vegetated. The slopes are characterised by an extensive pattern of small scarps and traverse tension cracking behind small superficial failures (Max depth 1.5m) that to some extent have been stabilised by the dense vegetation. Generally slope angles vary between 30˚ and 40˚. However, mid way down the slope the profile shallows to between 5˚ and 10˚over a distance of 10-15m. Where the slope angle exceeds 35˚ there are a numerous superficial failures similar in character to the Rose Cottage Slip, which occurred in 1977. These failures tend to be caused by excessive water entrainment and generally leave behind triangular scars bounded by steep sides and disrupted vegetation. The observed scars are all lightly vegetated, indicating that the failures had occurred within the last 5-10 years.

4.4.2 Upgath Hill

The cliffs below Upgath Hill are formed in weathered Upper Lias shales capped by sandstone beds of the Saltwick Formation. The sandstone beds are mantled by a thin layer of till, which is inclined south at 10˚. The intact cliffs stand at angles between 50˚ and 70˚. Where past failures have occurred the slopes are partially formed in talus debris and stand at between 20˚ and 30˚. The shallower slopes are lightly vegetated with assorted species of grasses and shrubs. The toe of the east facing slopes is protected by a reinforced concrete sea wall. The toe of the southern facing slopes is continually undercut by stream flow in Runswick Beck. Over the years Runswick Beck has cut down through the weathered shale forming an incised valley with sides that are characteristically oversteep.

4.4.3 Ings End

The most significant ground movements with regard to safeguarding access to the village are taking place in an area to the south of Nettledale Beck below Ings End in an area known locally as Dother Pits. The geometry of the landslide failure and rates of slope movement have been determined following detailed geomorphological mapping and interpretation of inclinometer and topographic survey data collected over the previous two years. Mapping reviled the presence of a series of sub vertical head scarps, up to 2.5m in height, below the cliff top over a length of 500m between incised valleys of Nettledale Beck and Limekiln Beck. These headscarps overlook a complex series of undulating slopes formed in till, which can be divided in to three zones by prominent boundary ridges, as shown on the detailed geomorphological map. All three zones are characterised by dense vegetation, irregular springs, streams and areas of surface water ponding. The shallow slopes are believed to have formed following the denudation of a sequence of partially rotated landslide blocks. Retrogressive failure of the slopes has occurred in a succession along a series of curved deep seated basal shear planes. Subsequent failures have been triggered by the stabilising effect of an initial failure caused by undercutting of the leading block by progressive coastal erosion. Slope angles vary between 15˚ and 20˚with the crest of the individual landslide blocks being well defined by breaks of slope inclined at between 5˚ and 10˚. The slopes are continually regressing at the sub vertical headscarp by processes similar to that described below Topman End. There processes provide a source of debris material causing adverse lading to the lower landslide blocks. Furthermore shallow mudslides that become active during periods of prolonged heavy rainfall can lead to increased loading of the lower landslide blocks leading to increased instability and rates of movement.

Inclinometer access tubes, installed during a supplementary ground investigation carried out in December 1998, have been monitored on a quarterly basis since instillation. The inclinometer tubes were installed with principal axis aligned parallel to the grade of slope to detect sub surface ground movement. Two distinct planes of movements were detected, corresponding to an interface between successive landslide blocks at (16-20m below ground level) and a lower basal shear surface at the interface between glacial till and rock head (30-33m below ground level). Ground movements were inferred from the recorded data for each of the shear planes identified. Rates of displacement approximating to 5mm/wk, orientated at a vector angle of 030˚N. These rates of displacement correspond well with data obtained from successive surveys carried out on a number of monitoring stations positioned on the slope face. A total of 13 survey markers were established on the slopes below Ings Eng prior to the coast protection and slope stabilisation emergency works being commissioned in March 1999. In the spring of 1999 following a characteristically wet winter, average ground surface movements were measured at 9mm per week at an average orientation of 46˚.

Monitoring was carried out throughout the construction phase of the emergency works contract and following completion of the project. Average ground surface movements were reduced to 4mm/wk during the summer months while construction was begin carried out with a slight variation in vector movement of 1˚ clockwise. Post project rates of displacement recorded following prolonged periods of heavy rain in the winter of 200 had further reduced to 3mm/wk. with a change in vector movement of 5˚ clockwise.

Although the reduction in rate of displacement is evidence that the permanent works comprising piling, drainage and earthworks undertaken on the slopes to the north and the toe of the slopes below Ings End is having a positive effect on increasing slope stability. Of greater significance is the change in angle of the average orientation of slope movement by 6˚ to the east.

4.5 Future Landslide Risk (High-Point Rendel 2002)

A judgement has been made on the frequency of different types of landslide failure occurring at each of the three slopes under consideration for the next 50 years over 10 year intervals. These judgements are based upon the assumption that a 'do nothing' scenario is adopted and the evidence provided by the geomorphological mapping, slope monitoring data and the frequency and size of historic failure events. The potential risk to assets, infrastructure and the recently completed emergency works is proportional to the probability of occurrence. However, the consequence of slope failure is dependant only upon the level of risk apportioned to occurrence, which in turn determines the extent of the preferred management strategy.

Table 4-2 The Risk Apportioned to the Probability of Occurrence in any given Year

Cumulative Probability of Occurrence	Apportioned Risk	Risk Category
0.01-0.10	Very Low	5
0.10-0.30	Low	4
0.30-0.60	Moderate	3
0.60-0.8	High	2
0.8-1.00	Very High	1

The results of the risk assessment carried out on the slopes below Topman End, Upgath Hill and Ings Head are considered separately.

4.5.1 Topman End

There is a high probability that further superficial failures will occur on the oversteepened slopes below Topman End as a result of excessive water entrainment over the next 50 years. All such failures, irrespective of size, will result in undercutting of the overlying slopes. This in turn will cause down slope creep movement leading to development of progressive collapse mechanisms, resulting in further cliff top recession.

The footpath linking Cliffmount Road with the lower village is showing signs of cracking. This cracking, which is caused by creep movement, is generally aligned cross slope (parallel to the alignment of the footpath). Individual cracks vary in length between 3m and 15m. In the past two years maintenance work has been periodically undertaken along the full length of the footpath to repair the cracks. The fact that the footpath continues to require maintenance is evidence that creep movements are ongoing and further cliff top recession with occur.

The risk of further cliff top recession affecting the hotel garden and neighbouring public car park over the next 10 years is low. Thereafter as the cliff top recession continues there is a moderate to high risk to these areas of land. Cliff top recession rates have remained constant at 2.5m per century despite the construction of coastal defences in the 1930’s and 60’s. Allowing for slope deterioration, wetter winters and drier summers it is unlikely that cliff top assets will be at risk from cliff recession during the design life of the strategy.

The risk posed by superficial slope failures to properties located on the north western fringe of the village below Topman End is considered high. It is likely that any such failures will occur following periods of prolonged heavy rainfall. The timing and size of any future failure is difficult to predict. It is reasonable to assume that the condition of slopes is progressively deteriorating, with a chance of failures expected to increase with time. Any future failures may be equivalent in size to the Rose cottage Slip (approx 600 tonnes).

The risk of a deep-seated failure occurring in the slopes below Topman End is considered to be very low. The presence of rockhead at shallow depth beneath till coupled with the recent engineering works preventing lateral unloading and under cutting of the toe eliminates the risk of deep seated failure mechanism being initiated.

4.5.2 Upgath Hill

As a consequence of the continued undercutting of the toe of the slopes by the stream flow in Runswick Beck there is a high probability that cliff failures of the type that occurred in February 1977 will continue to occur. Failures generally involve the collapse of sections of the oversteepened cliff following accelerated undermining caused by high velocity stream flows after prolonged periods of heavy rainfall. It therefore follows that properties located on the southern banks of Runswick Beckare at risk from such collapse mechanisms. The timing and size of any future failure is difficult to predict. It is ensilaged that failures similar in magnitude to the February 1997 event (approximately 200 tonnes) would be expected to occur once very ten years throughout the design life of the strategy.

4.5.3 Ings End

There is a high probability that regressive failure of the coastal slopes in the central and southern zones of the landslide complex below Ings Head will continue. Cliff top recession is likely to remain constant at between 2 and 3m per century. There are few assets at risk along the coastal frontage apart from a small number of isolated beach huts requiring extensive repair. It therefore follows that the risk from continued landsliding to property is minimal and that a 'do nothing' policy should be adopted.

The risk of progressive failure of the slopes in the northern zone of the landslide complex below Ings Head has been assessed in detail. Stabilisation works carried out on the adjacent coastal slopes as part of the coast protection and slope stabilisation emergency works has had a positive effect on the rate and mechanism of slope movement. Following completion of the emergency works, average rates of surface ground movement measured in the spring, following a characteristically wet winters have been reduced by 6mm/wk in 2 years. More significantly it is evident from post construction monitoring that average vector orientation of slope movement has been altered by 6° clockwise. This is due to the stabilising effect of both the piling works and earthworks (toe loading) constructed at the toe of the Ings End Slope.

It is envisaged that following prolonged periods of heavy rainfall the slopes will continue to fail. However, the probability and risk to village infrastructure of a deep seated failure occurring in the future is considered low due to the stabilising effect of piling and earthworks. It has previously been determined by calculation that if equilibrium is to be maintained the piles must contribute a minimum of 20% of their allowable capacity in terms of pile resistance. The contribution of the load transfer piles once fully mobilised will be to increase the global factor of safety against deep seated failure by 15%. Estimates of current loading from pile inclinometer plots indicate that the piles on average are currently loaded to within 56% of ultimate capacity and that the factor of safety against deep seated failure due to the action of piling has been 7%.

Any future failures are most likely to be shallow and caused by excessive water entrainment. It is likely that if such failures are allowed to occur over the design life of the strategy, adverse loading of the partially reloaded lower landslide blocks will occur resulting in further loading of the piles. This mechanism may be controlled by undertaking relatively inexpensive slope betterment works.

5 Management

5.1 Slope Stability

5.1.1 Proposed Options for the Slope Stabilisation Works

When designing the slope stabilisation measures in the Rapid Risk Assessment, High-Point Rendel stated that the overall landslide complex involves an area of approximately 500 by 300m, with an average thickness in the order of 15m, i.e. a 4,000,000 tonne mass of material. The northern end of the landslide which would affect the proposed new coast protection measures involves approximately 10% of this mass. Geotechnical limitations on the extent of reprofiling and the time lapse for deep drainage measures to become effective means that structural strengthening of the landslide will be necessary. Because of the thickness of the slipped mass, typically between 10-20m, conventional shear keys or anchors are not appropriate. It is considered that bored pile portal frame sheer keys will allow fixity in the intact rock to be achieved and to provide sufficient resistance to reduce the rate and magnitude of landslide movements to tolerable levels. Spacing of the portal frames will be critical and preliminary design demonstrates that 3nr 1m diameter poles at 4m c/c with a 4m deep pile cap could be the appropriate configuration for a portal frame. Portal frame would be 10m apart except towards Nettledale Beck where the spacing would be reduced to 5m in order to resist the forces arising from differential movement between the treated/untreated interface of the landslide. Upslope interceptor drains would improve overall stability of the reprofiled slopes some time after their instillation.

A computational assessment of the effects of the computational designs on global slope stability was undertaken; the results are shown in Table 4-1.

5.1.2 Preferred Stabilisation Option

High-Point Rendel (1998) considered that in order to provide the essential flexibility and non reflective properties of the sea wall, it is considered that rock armour is the only option. Stepped block work walls and voided reinforced concrete walls were considered but their satisfactory performance over a 50 year design life cannot be guaranteed. The cliff toe erosion south of the existing sea wall has been carefully considered and the conclusion is that a transition zone of at least 70m in length will be required t in order to reduce the risk of outflanking of the proposed 'hold the line' sea defences.

As a consequence of the high probability that movement will continue within the overall landslide complex to the south of the treated zone, it would be prudent if further ground investigation and extended survey monitoring were carried out during the process of detailed design. In addition to providing a more comprehensive model of ground conditions and slope mechanisms of the site, further investigation would enable a better understanding of the effect of prolonged movements on the treated zone to be outlined.

5.2 Coastal Management

5.2.1 Strategy Study

The strategy study carried out a number of technical and economic analyses in order to decide upon a preferred scheme for slope stabilisation. The High-Point Rendel Strategy Study provided a programme of future capital works, together with details of the management, monitoring and maintenance needs required to successfully implement the strategy plan over the next 50 years. The slope stabilisation scheme was completed in 2001 and is discussed in Section 4 of this report.

The coastal parts of the strategy study were based upon the first round of Shoreline Management Plans (SMPs). The proposed management of Runswick Bay did not change between the first and second round of SMP's so the strategy study is still relevant.

The strategy study considered that because of the nature of ground instability periodic reviews of the scheme and ground model would need to be carried out.

5.2.2 Shoreline Management Plan (Royal Haskoning 2007)

The SMP considered that the priority for management of this area (Management Unit 21) was to maintain the natural appearance and future evolution of the coast in support of the objectives of the National Park and Heritage Coast. Within this and in line with these objectives is to sustain the local community at Runswick Bay.

For the economic appraisal within the SMP2 the costs were based on strategy including for future works. Damages assume strategy values included during medium term.

Description of damage and benefits under preferred SMP2 plan:

- Loss of potentially two properties south of Runswick Bay Village in the long term, 2055 to 2105.
- Loss sailing club frontage in the longer term 2055 to 2105. This needs to be assessed at a local scale.
- Potential longer term loss of properties, beyond the period of the SMP2, at Kettleness.
- Retain the main village area.
- Need to relocate the Cleveland Way but no substantial physical barriers to allow this to happen

The SMP concluded that the inhabited part of Runswick Bay should be 'Hold the Line' for 20, 50 and 100 years.

5.2.3 Longer Term Management

The original response to the failure of the seawall and the associated ground stabilisation was based on an appraisal period of 50 years. The appraisal period for a coastal strategy would now be 100 years. The SMP2 (Royal Haskoning 2007) stated that Runswick Bay village should be 'Hold the Line' for the next 100 years. The rest of the Bay would be no active intervention for the next 100 years.

The ground stabilisation and coastal defence work carried out by High-Point Rendel should be reviewed in light of a 100 year appraisal time and a 'Hold the Line' policy.

6 Recent Monitoring of Slope Stability and the Coast

6.1 Slope Monitoring (Mouchel)

Following the identification of the failure mechanisms affecting the old sea-wall and car parks the late 1990's, remedial works were instigated and completed in 2001. The reduction in the rate of displacement of the land-slipping is evidence that the permanent works which comprised of drainage, piling and earthworks, undertaken on the slopes to the north of and at the toe of the slopes below Ings End, have had a positive effect upon slope stability.

Mouchel (2012) considered that the greater significance has been the reorientation of the vector angle of slope movement in a clockwise direction from northeast, in a more easterly direction. It is envisaged that following prolonged periods of heavy rainfall, the slopes will probably continue to fail. However, the probability and risk to village infrastructure of deep seated failures occurring in the future is considered low, as a result of the stabilising effects of the piling and earthworks.

6.1.1 Geomorphological Units

The description of the three units does not change between the High-Point Rendel and Mouchel work. As a result it is considered that Mouchel based their work on the ground model presented by HPR. There did not appear to be any update to the geomorphological map or modelling.

6.1.2 Rainfall Data

Under the Framework agreement, rainfall data records were made available to Mouchel by SBC and the Environment Agency as part of the Framework Agreement. Data supplied is referenced to stations throughout the region in particular at Loftus, Fylingdales, Whitby School, Scarborough, Mulgrave Castle, Ruswarp and Knipe Point. Within Mouchel Report "Analysis and Interpretation of Coastal Monitoring Data" 721228/001/GR/01/02/FINAL, reference was made to 'periods of heavy and / or prolonged rainfall' in terms of considering such an event with respect to their possible effects upon slope stability.

The definition of heavy and / or prolonged rainfall has been developed through the analysis of rainfall data records made available by the EA and SBC. Unfortunately it was not possible to determine how much rainfall would trigger a landslip event. Instead a quantity of rainfall was determined that would be likely to produce a significant rise in groundwater levels that might trigger a landslip. A definition of heavy / prolonged rainfall events was investigated in terms of determining statistically derived values of daily rainfall each month for the period 1995/8 to 2008/9. To this end the 75th percentile was calculated as a determining threshold value. A rainfall value, for a specific day, at the 75th percentile would be equal to or greater than 75 percent of the daily rainfall values recorded on that day of the year during all years that measurements have been recorded (Mouchel 2012).

Mouchel stated that in the event that the 75th percentile of daily rainfall values (a period of heavy / prolonged rainfall) are exceeded, it was recommended to carry out monitoring one week after the end of the rainfall event and at monthly intervals thereafter for three months. Further to the heavy rainfall experienced in December 2009, these recommendations were followed by SBC who instructed Mouchel to undertake additional monitoring at selected locations along the coast in order to comply with monitoring recommendations.

6.1.3 Inclinometer and Piezometer Readings from the Ongoing Monitoring Regime

The ongoing monitoring regime was initialised in July 2009. The Mouchel monitoring regime consists of existing inclinometers (A001, A002, A003 and A004) located along the edge of the main access road leading down into Runswick village (See Figure 6-1). Groundwater was measured in the inclinometer tubes with a dip meter.

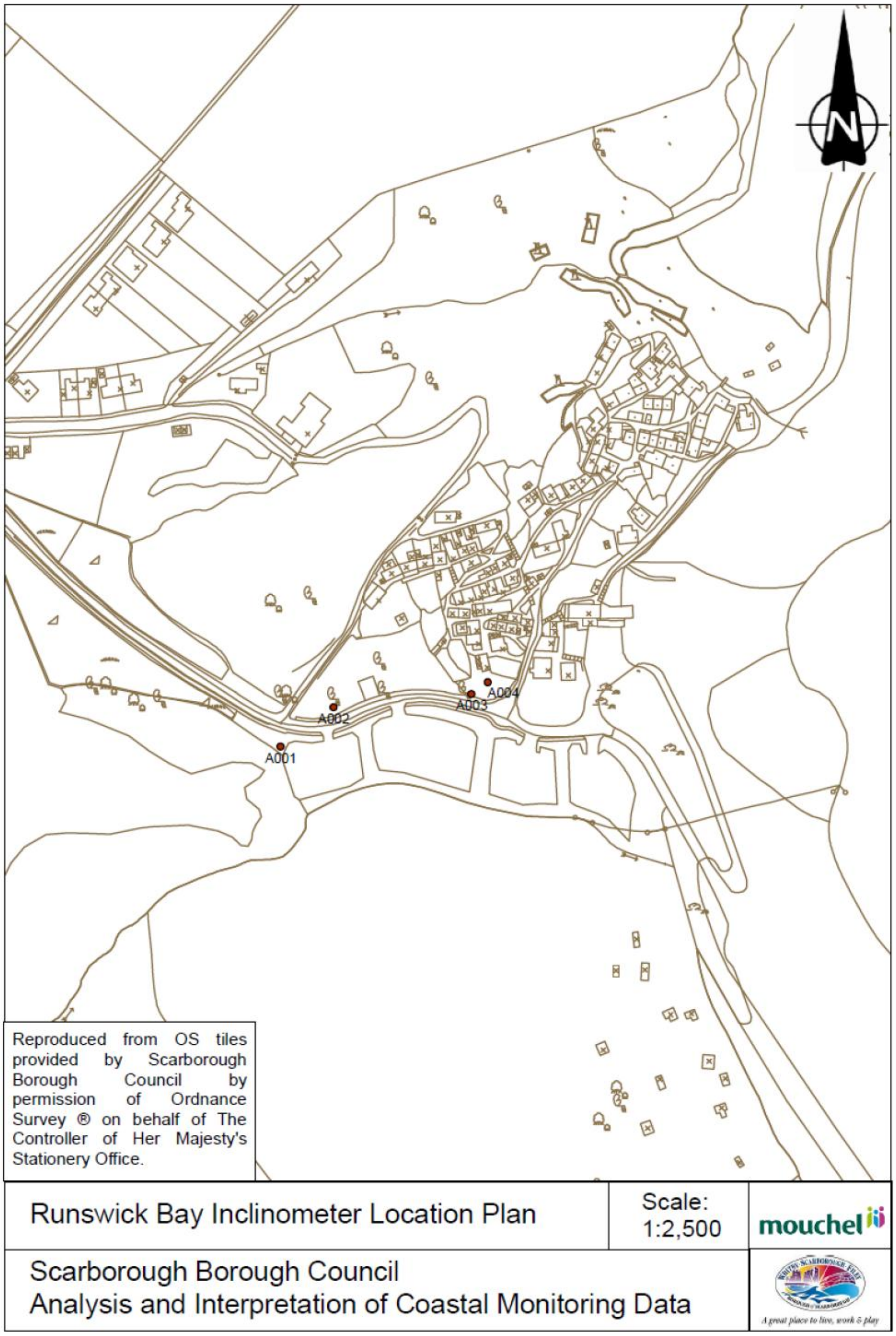


Figure 6-1 Location Plan for Inclinometers at Runswick Bay

6.1.3.1 Inclinometer Readings

Monitoring of inclinometers has been undertaken in accordance with the procedures detailed in Section 1.2 of the Mouchel 2012 Report. Incremental readings of June 2012 indicate that no ground movements have occurred within inclinometers installed in boreholes A001, A003 and A004. Data recorded in June 2011 from A002 would indicate that 10 mm ground movement has occurred between 17.0 metres and ground level.

However, the readings are most likely to be due to an erroneous reading rather than actual movements of the ground. Data recorded in December 2010 seemed to indicate ground movements in A001 although this was due to a discrepancy in the readings recorded from 22.0 to 20.0 metres depth. The erroneous readings were attributed to dirt within the tube tracks and subsequent readings have confirmed this by indicating no further development of suspected movements. Inclinometer readings are presented in Appendix A of this report.

6.1.3.2 Groundwater Readings

Groundwater levels at this site have been recorded from 16th June 2009 up to the present by Mouchel. A comparison of the groundwater readings of December 2011 and June 2012 shows decreases of 30 mm and 3290 mm in A001 and, A002 and increases of 50 mm and 90 mm in A003 and A004 respectively over this period. Groundwater readings are presented in Appendix B.

6.1.4 Findings from the Runswick Bay Monitoring 2012

Inclinometer instrumentation installed within selected piles of a portal frame shear key system was constructed as part of remedial works to restrict ground movements within the Runswick Bay area. Inclinometers were installed in piles in order to indicate shear stresses within them caused by ground movements. In the High-Point Rendel Strategy Study reference has been made to the determination of the piles' response to loading between successive inclinometer readings. It has not been stated how this was to be done or how it was to be achieved. To date, Mouchel have been made aware by the Client that this information is not available and therefore no further comment can be made relating to this. Hence, initial and successive inclinometer readings are only related to any general ground movements indicated by instrument readings.

The results from monitoring the inclinometers have so far shown that no ground movements have taken place within the vicinity of these instruments. Movements previously interpreted from data recorded within the inclinometers have now been attributed to erroneous readings. As the data bank from the inclinometers has increased, more information has been available to analyse and refine the on-going interpretation. The inclinometer graphs have, in the majority, plotted an identical path of inclination and indicate a steady state with no ground movements apparent.

A comparison of current groundwater levels with those of December 2011 do not indicate any definite relationship with increased rainfall experienced in the region of Runswick Bay up to June 2012.

6.2 Coastal monitoring by Halcrow

6.2.1 Bi-annual Beach Monitoring Programme

A six monthly topography survey of Runswick bay is carried out on behalf of Halcrow in order to understand beach behaviour.

The most recent Full Measures Beach Monitoring Report shows that during 2012 Runswick Bay showed signs of widespread erosion, which is not expected during the summer months. Accretion was noted close to the rock outcrops in the north of the bay. The other area of accretion was in the south of the bay, where the mouth of the stream could have lead to deposition of sediment.

The erosion of the shore was also noted in the previous full measures report, which reviewed the data from 2011. It was considered that there may be a lag between material being deposited on the beach from the eroding cliffs and the fines being washed offshore. In the centre of the bay there is a large bar, which persisted but experienced loss of sediment over the summer of 2011 and 2012. There are also areas of erosion close to the shore along the defended sections of the bay (Halcrow 2013).

The overall difference in topography between autumn 2008 to autumn 2012 was also reviewed. The difference in topography should show the longer term trends in beach evolution. The difference in topography shows that the upper and lower extents of the survey have been accreting. The observed erosion is centred on specific areas, rather than being scattered throughout the bay. The patch of erosion in the north of the bay appears to be related to the veneer of sediment over the rocks which outcrop on the foreshore here. Overall the bay appears to be stable but the erosion in the mid-bay could be a precursor to erosion at the back of the bay (Halcrow 2013).

6.2.2 Cliff Activity Status (Halcrow 2012)

The cliff activity survey assesses the cliffs on the Scarborough Bay coast for how active the cliff face is. The coast is divided into the management units used in the SMP. Runswick Bay Village is called Mu7A. Each unit is give one of five activity classes (see Table 6-1).

Table 6-1 Cliff Activity Classification

Activity Class	Description
Dormant	Protected cliff line or landslide complex with no visible evidence of landslide activity.
Inactive	Relict cliffs or landslides with vegetated slopes and localised erosion of the toe or failure of the headscarp.
Locally Active	Retreating cliff line with localised small landslides or areas of erosion.
Partly Active	Retreating cliff line with very common smaller-scale landslides or areas of intense erosion.
Totally	Retreating cliff line almost entirely affected by large-

Active	scale landsliding or intense erosion.

Unit MU7/1 includes Runswick Bay village itself and the adjacent slopes. The slope is well vegetated and is defended at the toe by a sea wall and rock armour. The outflanking of these defences to the north (within Mu6C) indicates the nature of the erosion which may be occurring here if it were not for the protective influence of these structures. There is some minor evidence of very localised, small scale erosion on the engineered slopes to the south of the village, but this does not appear to be significant.

Unit MU7/2 is a narrow unit located to the south of Runswick Bay village. The slopes are relatively shallow and well vegetated. The only activity evident is at the unit toe, where the cliffs are subject to marine erosion in the absence of protection measures.

Both units are classified as Inactive and have remained stable since the previous Cliff Activity Status walkover in 2009.

6.2.3 Asset Condition Survey (Halcrow 2012)

In recent years new coastal defences have been constructed in Runswick Bay, associated with the building of a new pumping station (adjacent to the lifeboat station) and associated with the remediation of the landslip that damaged the defence near the end of the road. The other defences fronting the properties at the north of the bay are of variable age and condition.

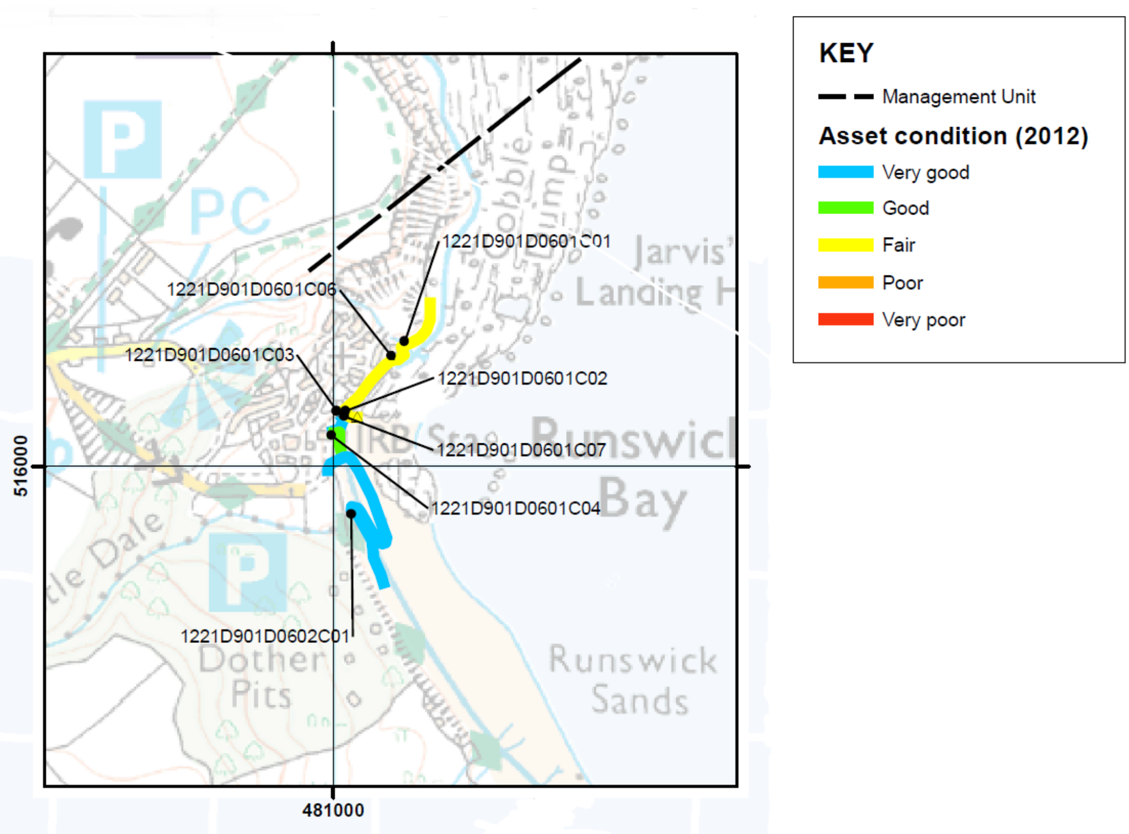


Figure 6-2 Map Showing the Condition of the Defences in Runswick Bay

Figure 6-2 shows the position and condition of the coastal defences in Runswick Bay. A description of the condition of each of the defences during the 2012 asset condition survey is provided in Table 6-2 .

Table 6-2 A Description of the Condition of the Defences in Runswick Bay for the Halcrow Asset Condition Assessment

Asset Reference	Defence Type	Description
1221D901D0602C01	Rock Armour	Runswick Bay village rock armour defences are in very good condition, with the rocks tightly packed with good coverage and no evidence of significant deformation. The associated slipway towards the south from the end of the road and boat park is also in good condition. Beach levels appeared relatively high at the time of the inspection, so the toe was not visible. There is ongoing erosion of the undefended cliff at the southern end of the defence and some of the locally sourced smaller rock used at the tie in has been scattered, see below right, however this is not a cause for concern at present.
1221D901D0602C05	Mixed	The Sailing Club, located in the bay some 600m south of the village, where the beach is wider has been constructed on timber struts and features a mix of coastal defences below left and right. The informally placed relic tank trap blocks at the south end show extensive cracking. The timber defences across the front of the main building are showing signs of rot and will need replacement in future. Runswick Bay sailing club timber defences at south end.
1221D901D0602C05	Mixed	This defence is at the northern end of defences at Runswick Bay sailing club. On the northern end of the rock armour, the slipway adjacent to the RNLI building remains in good overall condition. However, the timber strips to support the small boats are rotting in many places and will need replacing and joints between slabs need resealing. The seawall around the pumping station to the north of the RNLI building is in very good condition, below right, although the standards for all of the handrails are showing corrosion and need cleaning and repainting.

1221D901D0601C01	Sea Wall	The sea wall defences to the north of the new pumping station show a variety of defects ranging from minor to more significant issues. The most northern coastal sea wall is suffering from surface cracking and erosion. Erosion of the underlying rocky foreshore continues to cause undercutting of the sea wall. Further investigation is required to determine the rate of undercutting. Further defects include washed out sealant joints, flap valves on weep holes which have seized shut, wash out of the joints under the capping beam, vertical cracks through the wall, missing joints and filler in the seawall face and promenade surface with vegetation growth, and outflanking at tie in to eroding cliff at northern end.
1221D901D0601C06	Sea Wall	Moving south, the protruding section of wall protecting the individual property is in fair condition. There are signs of repairs to the large vertical cracks in the wall and toe apron. However there are cracks in the top of the concrete bagwork part of the wall. Repaired vertical cracks to toe of wall, and horizontal cracks are holding.
1221D901D0601C03	Sea Wall	The main length of wall below the properties is in variable condition. Although there are signs of repair work there are significant cracks in the wall and undercutting of the toe in several locations, see photos below. The beach level had recovered here since the 2009 inspection with a small accumulation of coarse grey shale sand from the eroding landslip to the north of the village.
1221D901D0601C02	Concrete Breakwater or Groyne	The concrete breakwater or groyne to the north of the pumping station and lifeboat slip was noted to be in need of repair during the 2009 survey, with large horizontal and vertical cracks on both sides propagating through the defence. The condition at the time of the 2012 inspections seems to be similar or worse. It is recommended that forthcoming strategy study should consider the need for this asset in terms of sheltering to the RNLI Slip and pumping station seawall, and the retention of sand and gravel beaches in the area.

7 Discussion

The cliff stability analysis could not draw a clear link between rainfall and ground instability. A statistical analysis was carried out to find the amount of rain which would cause an increase in groundwater which could lead to a landslide, but this is not clear cut. The link will be even less clear in potential future scenarios including climate change and related increased rainfall.

The High-Point Rendel slope stability modelling is based on information gained from the rapid risk assessment. The ground model is based on historical BGS boreholes, a new ground investigation comprising of six boreholes and three trial pits and geomorphological mapping.

The High-Point Rendel Strategy study states that it had been determined by calculation that if equilibrium is to be maintained the piles must contribute a minimum of 20% of their allowable capacity in terms of pile resistance. The contribution of the load transfer piles once fully mobilised will be to increase the global factor of safety against deep seated failure by 15%. Estimates of current loading from pile inclinometer plots indicate that the piles on average are currently loaded to within 56% of ultimate capacity and that the factor of safety against deep seated failure due to the action of piling has been 7%.

Following the construction of the slope stabilisation works Mouchel have been reading the inclinometers. The 2012 Mouchel Report stated that initial and successive inclinometer readings are only related to any general ground movements indicated by instrument readings. The Mouchel Study reported that in the High-Point Rendel Strategy Study reference has been made to the determination of the piles' response to loading between successive inclinometer readings. It has not been stated how this was to be done or how it was to be achieved. To date, Mouchel Ltd have been made aware by the Client that this information is not available and therefore no further comment can be made relating to this.

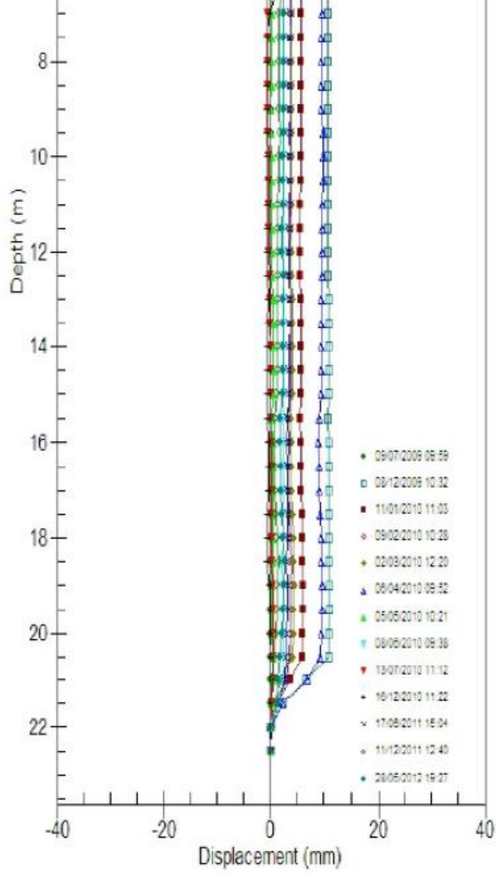
8 Conclusions

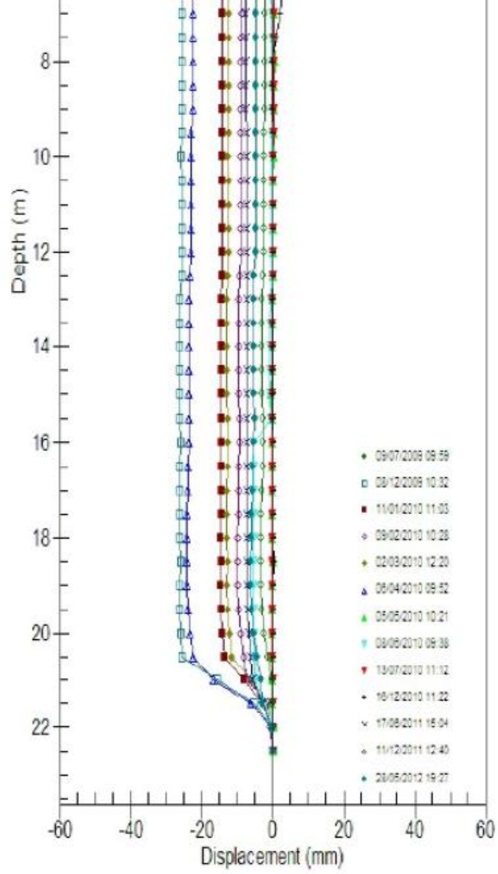
The ground model was created as part of the rapid risk assessment and does not appear to have been tested since. The inclinometers show that the slope has apparently been stabilised since the scheme so the scheme is has been working. Because there has not been any ground movement since the monitoring instrumentation was installed the link between rainfall and failure cannot be reviewed in light of new data.

The review is a good opportunity to test the ground model presented by Rendel by geomorphological mapping and seeing what changes have happened to the surface of the slope since 2002.

The scheme should be reviewed in light of a 100 year appraisal period. The estimates for sea level rise and climate change should be updated with the most recent estimates and a 100 year time span to assess the likely future pressures on the scheme.

Appendix A Inclinometer Plots

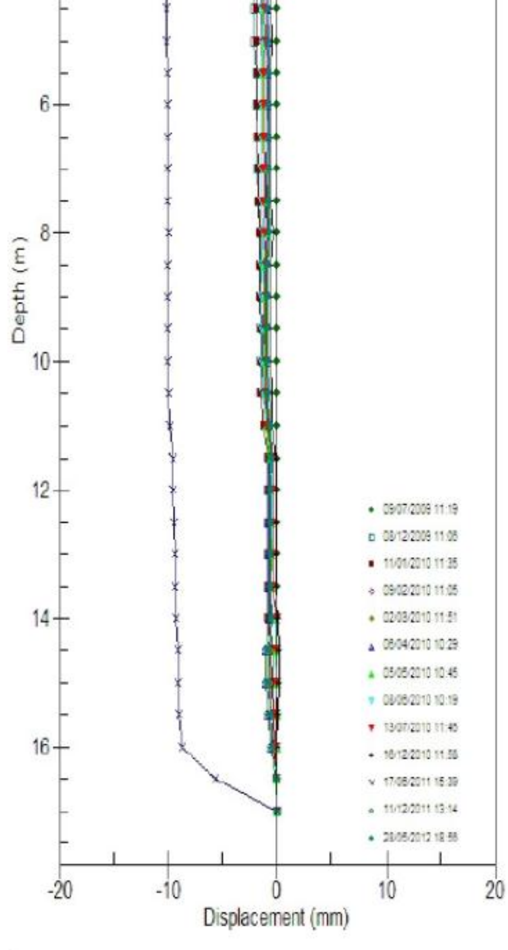


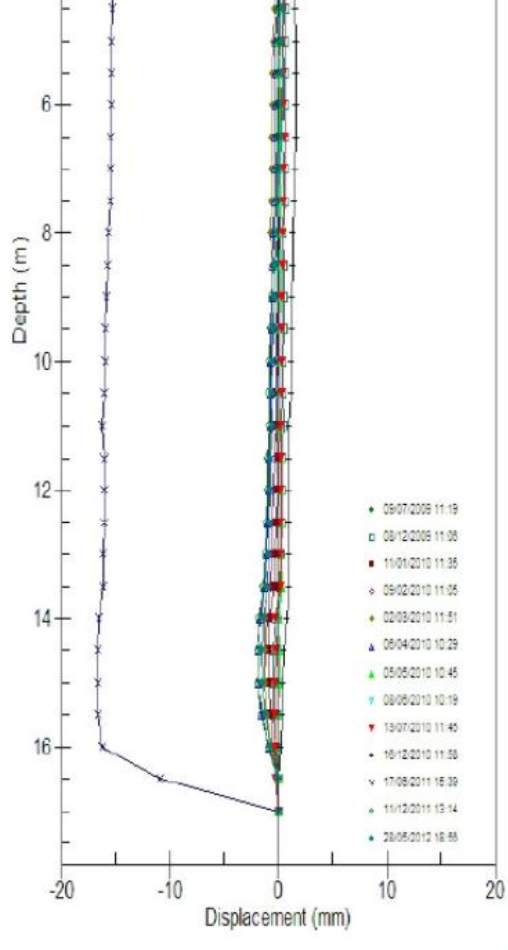


PROJECT: 1022971 Ongoing Analysis of Coastal Monitoring Data
SITE: Runswick Bay
INSTALLATION: A001
COMPANY: Mouchel Ltd
CLIENT: Scarborough Borough Council

mouchel

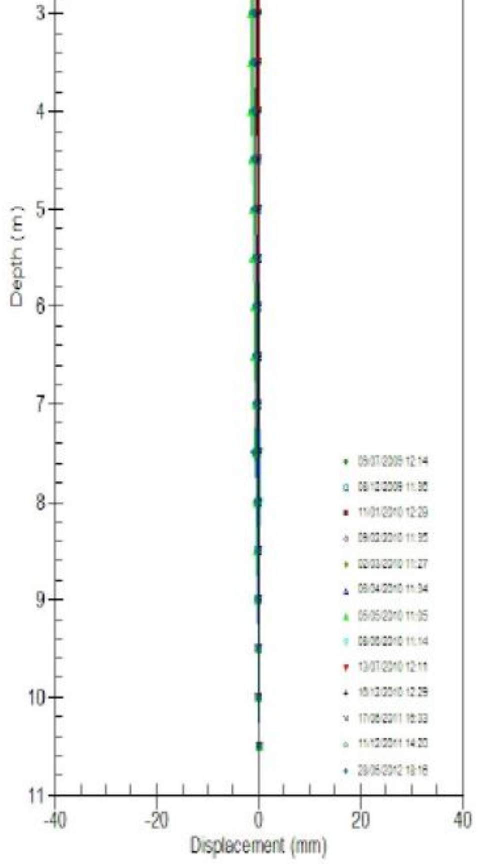
NOTE: A0 Direction = East

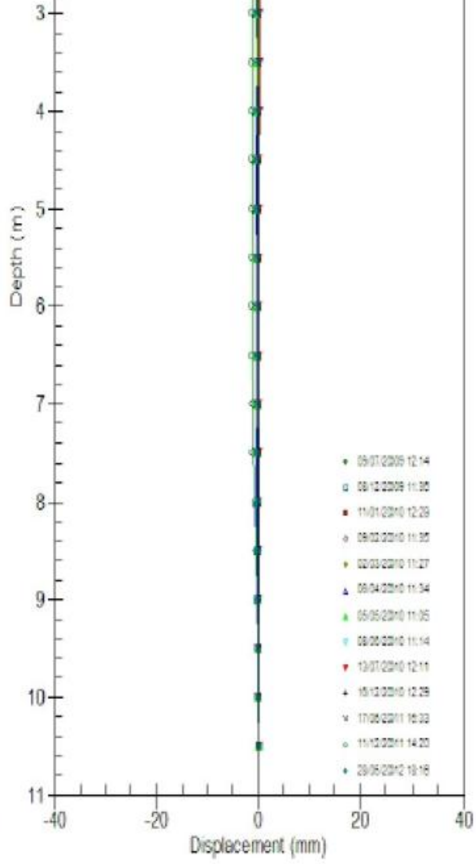




PROJECT: 1022971 Ongoing Analysis of Coastal Monitoring Data
SITE: Runswick Bay
INSTALLATION: A002
COMPANY: Mouchel Ltd
CLIENT: Scarborough Borough Council

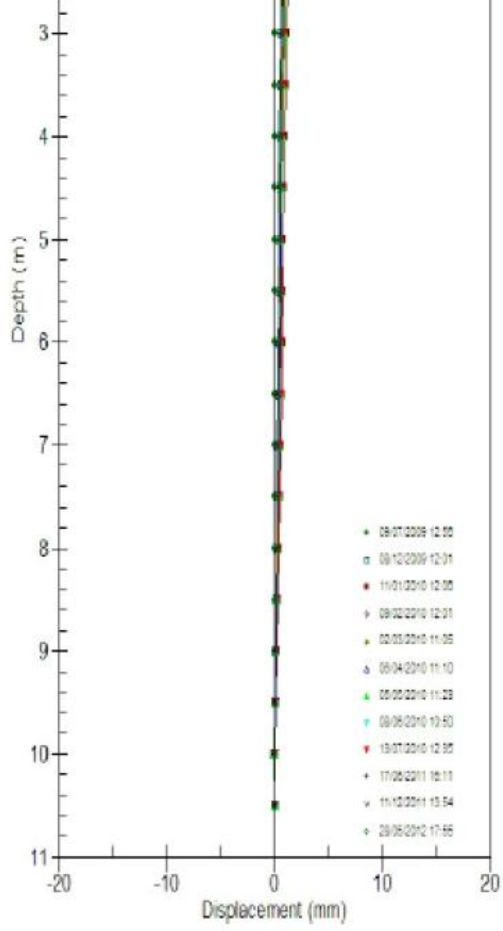
NOTE: A0 Direction = East

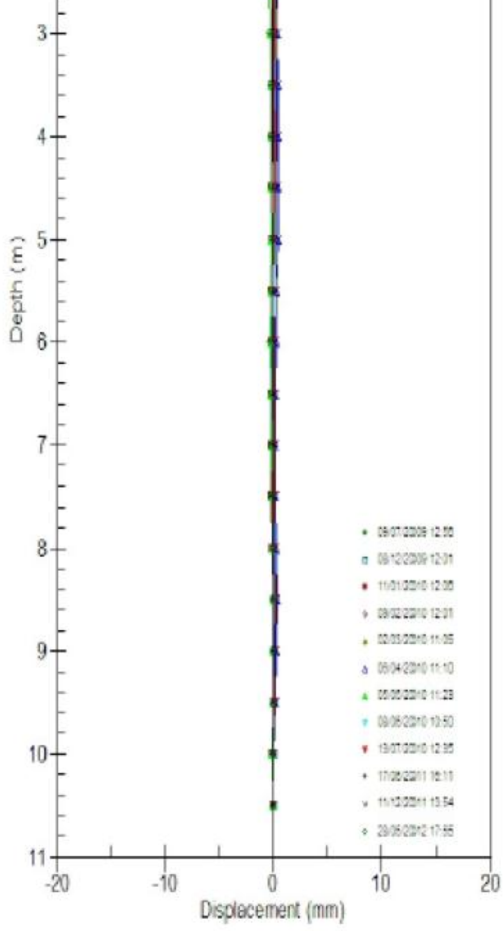




PROJECT: 1022971 Ongoing Analysis of Coastal Monitoring Data
SITE: Runswick Bay
INSTALLATION: A003
COMPANY: Mouchel Ltd
CLIENT: Scarborough Borough Council

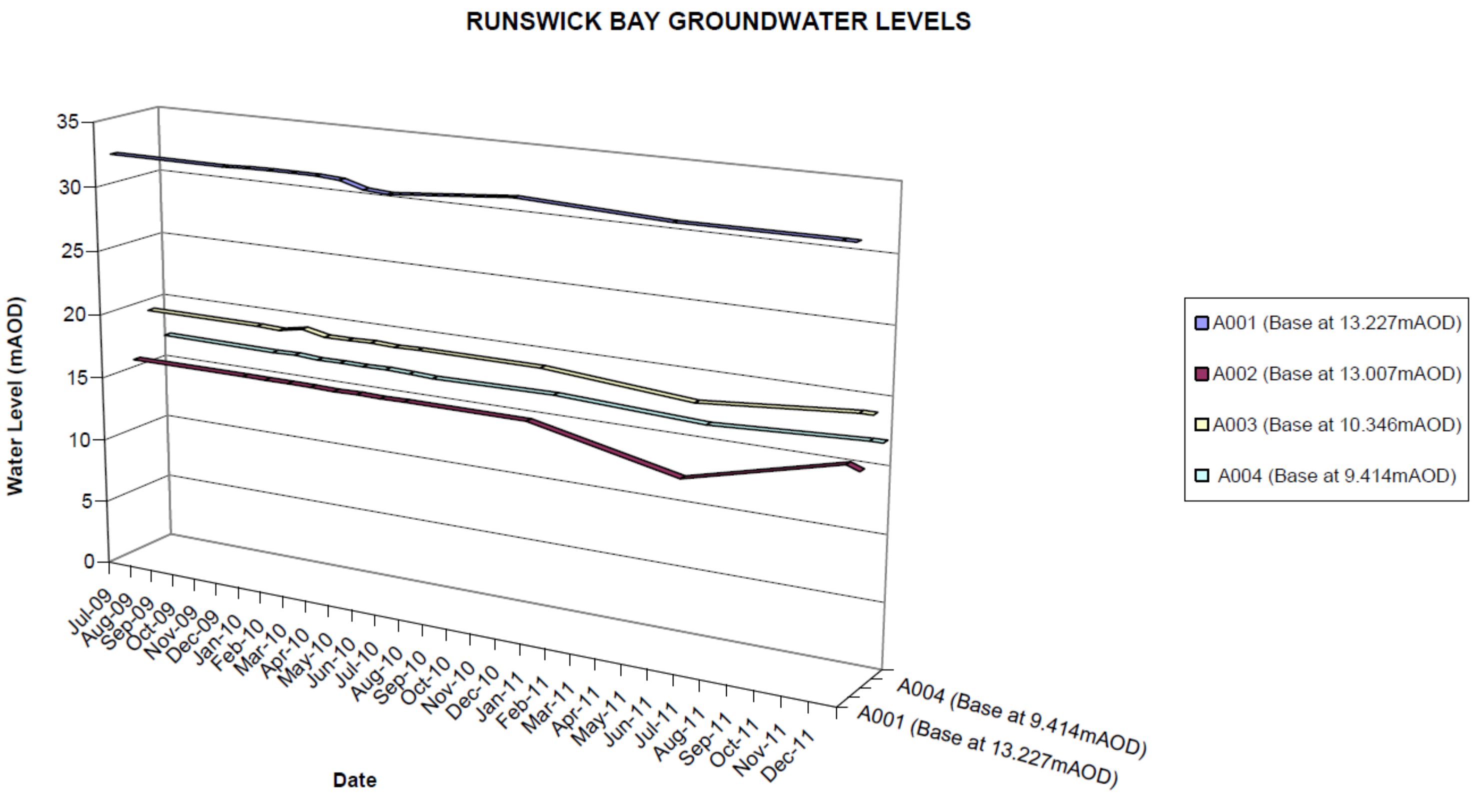
NOTE: A0 Direction = South East





PROJECT: 1022971 Ongoing Analysis of Coastal Monitoring Data
SITE: Runswick Bay
INSTALLATION: A004
COMPANY: Mouchel Ltd
CLIENT: Scarborough Borough Council

NOTE: A0 Direction = South East


RUNSWICK BAY GROUNDWATER LEVELS
Water Level (mAOD)
35
30
25
20
15
10
5
0
Jul-09
Aug-09
Sep-09
Oct-09
Nov-09
Dec-09
Jan-10
Feb-10
Mar-10
Apr-10
May-10
Jun-10
Jul-10
Aug-10
Sep-10
Oct-10
Nov-10
Dec-10
Jan-11
Feb-11
Mar-11
Apr-11
May-11
Jun-11
Jul-11
Aug-11
Sep-11
Oct-11
Nov-11
Dec-11
Date
A004 (Base at 9.414mAOD)
A001 (Base at 13.227mAOD)
A001 (Base at 13.227mAOD)
A002 (Base at 13.007mAOD)
A003 (Base at 10.346mAOD)
A004 (Base at 9.414mAOD)

9 References

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