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CASTLEHAVEN COAST PROTECTION & SLOPE STABILISATION WORKS, NITON, ISLE OF WIGHT , UK.

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ABSTRACT: A major landslide at Castlehaven on the Isle of Wight occurred as a reactivation of pre-existing landslides as a consequence of both coastal erosion and high groundwater levels caused by the very wet and stormy winter of 1994/95. The Castlehaven landslide lies within the Undercliff landslide complex and a strong relationship between landslide movement, groundwater level and the 4 month antecedent effective rainfall (4AER) has been developed for the area. In the site area movement occurs when the 4AER exceeds approximately 380 mm. Castlehaven also lies within an environmentally sensitive area with both national and international designations. The selection of remedial measures was influenced by the need for environmental protection but also needed to be robust to cope with predicted future changes in climate including increased storminess and winter total and effective rainfall, rising groundwater levels and sea level changes. A scheme including siphon wells, and electro-pneumatic wells, both used in UK for the first time, was designed and constructed to

1. INTRODUCTION

Castlehaven, situated on the south coast of the Isle of Wight, is the site of the recently completed construction of a coast protection and landslide stabilisation scheme which won the 2005 Fleming Award of the British Geotechnical Association. This paper will describe the details of the scheme and how the impacts of predicted climate change have influenced the design and successful implementation of the works.

The site is located at the southern most point of the Island at Reeth Bay, south of the village of Niton (Fig. 1) and lies within the area known as the “Undercliff”. This is a complex



Figure 1. Aerial Photograph of Site

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landslide system which fringes the whole south coast of the Island and is some 12 km in length and up to 0.6 km in width and contains the town of Ventnor and other areas of development including St. Lawrence, Bonchurch and southern Niton. The area is considered to be the largest urbanised landslide in western Europe.

Landsliding within the Undercliff is an ongoing post glacial phenomenon and has caused significant distress to the development and infrastructure throughout the area over recent centuries. The historic record of landsliding in the area has been well documented (Ibsen 2000 a-b, Hutchinson 1991, Ibsen & Brunsten 1996) with landslides of different sizes being a frequent event caused by the combination of marine erosion at the toe of the sea cliffs combined with recurrent periods of high rainfall, high ground water levels and susceptible geology. Several “great” landslide events occurred in the Undercliff in the very wet and stormy winter of 1994/95 including at Castlehaven where the reactivation of the pre-existing postglacial landslides has affected an area up to 400 m inland from the coast. If left unchecked this landslide had the potential to cause direct and consequential losses estimated in £20 million at 2004 prices.

To address the problem of landsliding and to reduce the risk to public safety, property and infrastructure a coast protection and landslide stabilisation scheme was designed. However the choice of the design and the implementation of the scheme was further complicated by the fact that the area is highly designated and protected for the environmental quality of the area and consequently the final scheme not only had to address the technical issues of preventing erosion and improving the stability of the landslides, including provision for predicted climate change, but had to incorporate design principles and methodologies that did not compromise the environmental quality of the area. The details of the environmental issues and the process undertaken to develop a scheme which met these criteria and the lengthy process required to achieve approval through the consultation and planning process has been the subject of a separate paper (Clark et al. 2002).

2. GEOLOGICAL BACKGROUND TO SCHEME

The regional solid geology of the area dips towards the south at approximately 2 degrees however locally the dip is towards the southeast due to the influence of the shallow St. Lawrence syncline (Hutchinson & Bromhead 2002). The strata consist of an interbedded sequence of high and low permeability Cretaceous rocks including the Upper Greensand which forms the inland cliff of the landslide backscarp some 500 m inland from the coastal cliffs. This is underlain by the Gault Clay, which is notorious locally for being susceptible to instability and is known as the “Blue Slipper”. This in turn is underlain by the Carstone and the Sandrock of the Lower Greensand. The coastal cliffs at Reeth Bay are formed by intact near vertical Sandrock which has been subjected to marine erosion and on the basis of historic map analysis and local history the cliff line has retreated into the pre-existing landslide complex by up to 40 m since records of 1862.

This undermining of the toe of the landslides and steepening of the slope, combined with high rainfall and groundwater levels are the triggers for the reactivation of landslide movement within the Castlehaven area and the Undercliff in general.

3. GROUND MODEL

In order to address the options for remediation of the instability a ground model was developed on the basis of geomorphological mapping and borehole investigations in 1996 and 2002

to assess the ground conditions, groundwater behaviour and aquifer characteristics of the area. The scope of these investigations is summarised in Table 1.

Scope of Ground Investigations		Table 1.	
Activity	1966 No.	2002 No.	
Geomorphological Mapping	yes	yes	
Boreholes			
Rotary Core (100 mm)	5	7	
Rotary Core (300 mm)		2	
Rotary Open Hole (100 mm)		4	
Trial Pits			
Shallow	11	7	
Deep	5		
In-site Testing			
Natural Gamma	6		
Neutron – Neutron	6		
Gamma - Gamma	6		
Caliper	6		
Absolute/Differential Temperature	6		
Absolute/Differential Conductivity	6		
Flow	6		
Permeability Tests			
Pumped Well Step & Constant Rate Discharge		2	
Borehole Variable Falling Head		8	
Trial Pit Variable Rising Head		2	
Instrumentation			
Standpipe Piezometer	4		
Vibrating Wire Piezometer		3	
Inclinometers	2	2	

The landslide debris in the upper tier varied in thickness across the site increasing inland and consisted of an upper zone of Upper Greensand debris up to 10.5 m in thickness overlying a zone of Gault Clay debris up to 25 m in thickness. The overall geological section of the site area based on geomorphological mapping and the borehole interpretation is shown in Figure 2.

The landslides can be divided into an upper tier or inland area of multi-rotational failures separated from a seaward or lower tier of compound landslides by exposure of a Gault clay scarp. The upper tier landslides was determined from the degree of disturbance of the Gault Clay within the exploratory boreholes which separated the slipped “Gault Clay Debris” from the intact Gault and from inclinometer data. This model confirmed the preliminary landslide model developed from the geomorphological mapping undertaken soon after the major landslide of 1994/95 (Moore et al 1995).

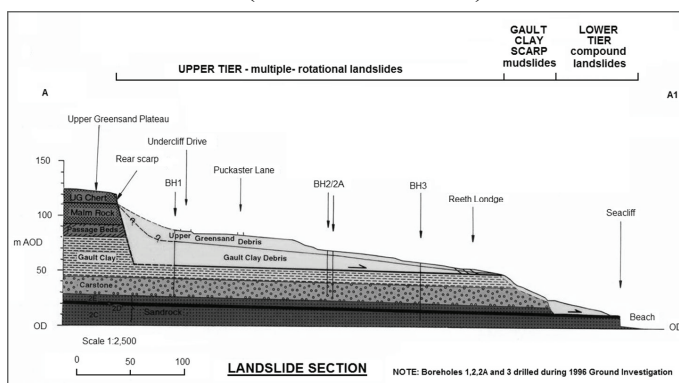


Figure 2. Geological Section

The landslide was also characterised by the widespread occurrence of springs, seepages and sinks, discontinuous surface and groundwater flow from north to south with a hydraulic gradient of around 12% towards the sea and by low to moderate permeabilities of the landslide materials. Local zones of high flow were also suspected along a fossil valley axis cutting through the

escarpment and behind back tilted blocks.

4. SCHEME DEVELOPMENT AND DESIGN

4.1 Overview

In view of the overall thickness of the landslide debris within the model coupled with the severe environmental constraints imposed on construction in the area by the environmental consultants and the planning conditions it was evident that improvement in the stability of the upper tier landslide could not be achieved by major construction and the installation of physical strengthening of the ground with techniques such as piles, deep counterforts, retaining, etc.

Accordingly a design concept was developed which consisted of two elements, firstly the prevention of the erosion by the sea and landward retreat of the Lower Greensand cliff by construction of a rock armour toe and apron and secondly improving the stability of the upper tier landslide by drainage techniques. The design concept requirement was a fifty year design life as determined by Department for Environment, Food and Rural Affairs (DEFRA), the government funding department.

4.2 Allowing for Climate change

Since the initial study phase it was obvious that predicted climate change (e.g. IPCC 2002, Hulme et al. 2002, Hulme & Jenkins 1998, Bray et al. 1997) had the potential to alter coastal processes and slope stability at Castlehaven and consequently the design of the scheme had to be robust and to allow for this scenario. The approach adopted at this site included the identification of all the factors influencing the stability of the area followed by a determination of the extent to which climate change could influence them. The key climate variables considered to influence the stability of the site and in turn to be influenced by climate change are:

- Sea level;
- Nearshore waves;
- Effective rainfall;
- Infiltration and additional leakage from pipes, soakaways and septic tanks and increased run off from roads and houses.

The study highlighted the complexity of the site and the complex inter-relation of the key triggering factors. In particular the effect of climate change on the groundwater and hydrogeology of the slopes was anticipated to be complex. Therefore the design of the foreshore and of the slope stabilisation works was based on two different approaches.

- The foreshore works were designed on the basis of the most recent predictions (at the time) in sea level rise for the future and wave modelling.
- The design of the slope stabilisation had to be flexible and adaptable to a relatively wide range of scenarios.

In terms of the foreshore works a sea level rise of 6mm/year (Environment Agency 1999) was allowed with an overall sea level rise of 0.3 m in 50 years.

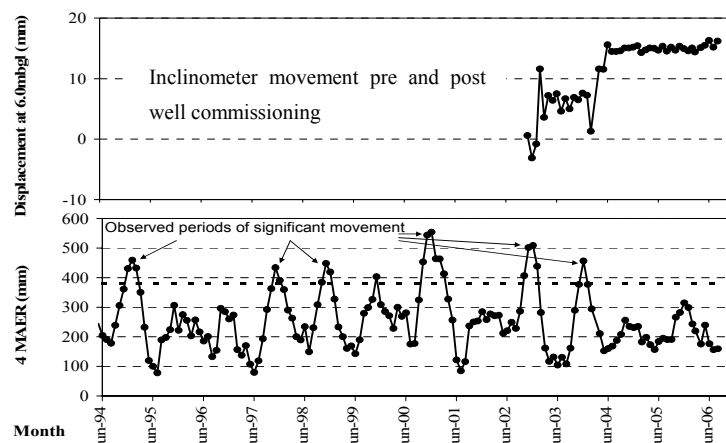


Figure 3. 4 Month AER and Ground Movement.

The provision for increased storminess and near shore waves was allowed and the rock armour stability was based on physical modelling and 1 in 100 year return period wave conditions. The slope stability design philosophy included the ability to cope with increased rainfall and groundwater levels raised to existing ground levels (i.e. total saturation).

The close relationship between rainfall, groundwater levels and landslide movement within the Undercliff, particularly to the 4-month antecedent effective rainfall (4AER) level has been recognised for some time. However the threshold levels at which significant movement is initiated varies at different locations within the Undercliff due to varying sensitivity to movement based on local ground conditions and physiographic features (Hutchinson 1991, Moore et al 1995).

At Castlehaven since the major movements in the spring of 1994 and the winter of 1994/95 there have been subsequent significant movements during the winters of: 1997/98, 1999/2000, 2000/01 and 2002/03. No significant movement was observed during the summer periods or dry winters. There is a continuous rainfall record for the area since 1992 based on a Ventnor weather station supplemented in 2004 by an automatic station at Castlehaven set up as part of the works. Analysis of this data shows a good correlation between the onset of movement and a 4AER in excess of c. 380 mm (Fig. 3).

At the site groundwater and the hydrogeology of the landslide is influenced by rainfall and infiltration, recharge at the rear of the landslide from the main Upper Greensand aquifer perched on the underlying impermeable Gault and confined aquifers within the Sandrock. The area is also influenced by artificial recharge from leaking sewers, septic tanks, gutters, road runoff, etc., (Fig. 4).

Groundwater levels have been monitored in standpipe piezometers since 1996 and in both standpipe and vibrating wire piezometers continuously since 2002. In December 2002 movement was again recorded by two inclinometers installed during the 2002 ground investigations and by surface movement and datum point monitoring when the 4AER exceeded c.380 mm which was equated to a triggering groundwater level at the site.

The considerable limitations placed on the scheme for environmental reasons, including the elimination of ground improvement construction options meant that the only feasible option was to lower groundwater. Thus the main criteria for the improvement in the stability of the landslide was to lower and maintain groundwater locally within the upper tier of the landslide to a level comparable to that experienced during previous summer levels when no significant movement occurs. A further constraint also precluded lowering of the groundwater below equivalent summer levels because of the potential of damage or stress caused to the local flora. This approach maintains partial activity of the spring line that occurs at the Gault Clay scarp and causes localized slope movement which is important for unstable slope habitat creation.

Thus the remedial measures available could not completely stabilise the landslide and the design approach was to achieve a target improvement on the current factor of safety based on a required minimum groundwater drawdown to achieve a minimum "Factor of Improvement" in excess of 1.10. Accordingly a groundwater control system was selected and designed that would eliminate the peak groundwater levels during periods of prolonged wet weather that trigger landslide movement. Furthermore the system had to be flexible and robust to be able to cope with the anticipated increase in "climatic events" that could occur as a result of a changing climate and the consequential seasonal increase in groundwater levels.

The ground model indicates landslide debris thicknesses varying up to a maximum in excess of 35 m at the rear of the landslide. The groundwater levels similarly varied throughout the site with maximum recorded winter levels of approximately 6m and 1m below ground level at the rear and the front of the upper tier respectively. Slope stability analyses indicated that the groundwater drawdown required to achieve the required factor of improvement was between 11.4 m and 7.7 m at the rear and front of the landslide respectively. The analyses were conducted on a number of cross sections for typical slope and basal shear geometries us-

ing laboratory determined residual shear values, worst case ground water conditions with an assumed existing factor of safety marginally above unity.

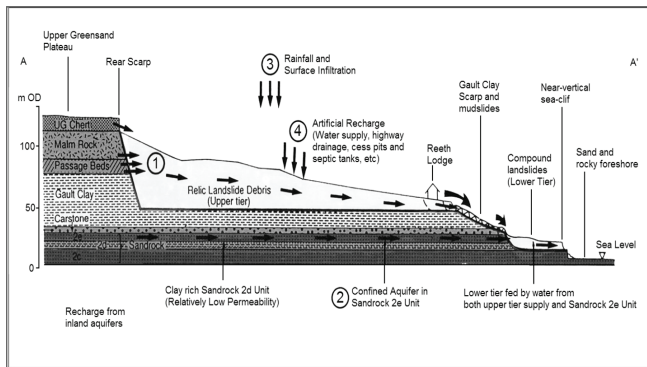


Figure 4. Sources of Groundwater

was adopted. The details of the siphon drain concept are described in Bomont et al. 2005 and the main principles of the system are described below.

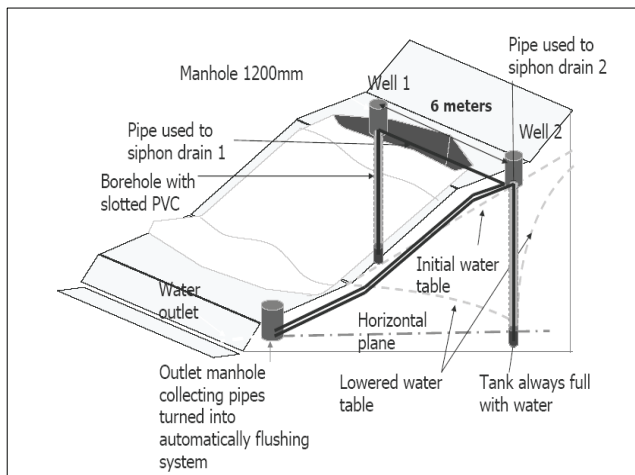


Figure 5. Section of Siphon Drain Principle

the well. The flow will continue until the water level in the well falls to the level of the outlet, provided that the flow rate in the siphon is sufficient to keep the siphon primed. As the water rises towards the top of the siphon the pressure falls, and may approach a perfect vacuum. In the upstream section the low pressure causes small bubbles to appear. These bubbles tend to coalesce into larger ones further downstream. Two forces act on the bubbles, firstly buoyancy and secondly hydraulic force due to the flow in the pipe. If buoyancy becomes the major force, the bubbles will collect at the summit of the pipe and combine into a single large bubble, which in time would break the siphon flow. This is avoided by using a system that automatically flushes out bubbles by turbulent flow.

The flushing system consists of a number of PVC pipes at the downstream end of the siphon pipe which acts as a hydraulic accumulator. When there is no flow in the siphon pipe the water level will be nearly the same as at the upstream end in the drainage well. When the water in the PVC accumulator reaches a certain level, the stored water is quickly emptied by a simple flushing system requiring no energy. The sudden lowering of the level of water in the accumulator causes flow in the siphon pipe. This rapid flow flushes out any air bubbles within the siphon (Fig. 6). It is important that the pipe work in the accumulator is sized to achieve a

To achieve the required level of drawdown a series of lines of deep pumped wells was considered the most appropriate approach and a number of pump options were considered including electrical submersible, electrical pneumatic, electro-pneumatic and gravity fed siphon wells. Based on the thickness of the landslide debris that had to be penetrated by the wells, the level of drawdown required and a cost-benefit analysis of the whole life costing of each option, a combination of deep electro-pneumatic and gravity fed siphon wells

4.3 Siphon drain principle

Small diameter siphon drains are placed in vertical drilled drainage wells. These drains are generally spaced at between 3 m to 6 m centres and are sufficiently deep to provide the required drawdown. The wells are dewatered using the siphon principle and are able to drawdown under the influence of gravity up to approximately 8 mbgl. Siphon tubes are introduced into a permanently water filled reservoir at the base of the well with an outlet down stream at an outlet manhole, situated down slope (Fig. 5).

If the water level rises in the well, the siphon will flow and abstract water out of

sufficient flow rate and duration to remove the air bubbles from the siphon tube. The flow continues until the siphon is clear of air bubbles when the flushing system stops the flow. The water level will then rise in the well and flushing system up to the predetermined level in the accumulator. The flow is then primed and the flushing cycle starts again.

One of the reasons for the selection of the system, which had never been used in the UK before although at various sites in Europe (Gress 1996), was its simplicity of operation and cost since the siphon does not require any source of power to operate. In addition the siphon system is extremely flexible and is able to respond to changing conditions. The system as designed was capable of pumping at least twice the required rate to allow for increased inflow due to higher predicted rainfall intensities. Furthermore in the event of major climatic and significant ground water changes, the system if required could be retrofitted with additional tubing to increase the discharge output of the system.

The diameter of the siphon tubes can be varied from 10 mm and 20 mm achieving flows of around 2.9 m³/day up to 24 m³/day per well respectively and sufficient space was allowed for such a retrofit of multiple tubes within the design of the ducting of the drain system.

4.4 Electro-pneumatic drain principle

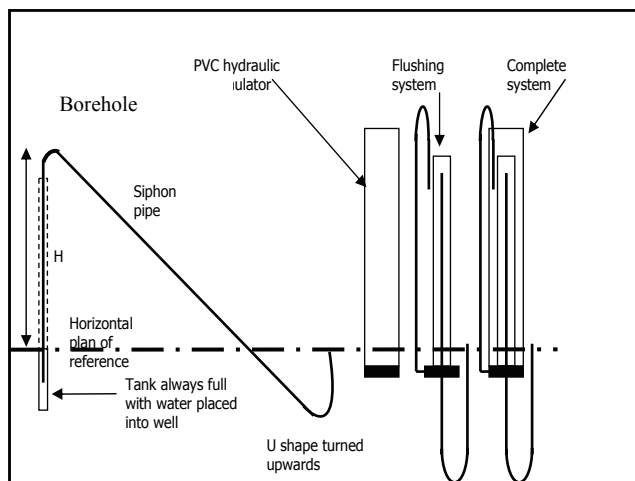


Figure 6. Siphon Flushing System

The electro-pneumatic drain has been developed to stabilise landslides by intercepting groundwater at greater depths or lowering groundwater to lower levels than that capable using siphon drain techniques. The electro-pneumatic drain design principle is similar to that for the siphon drains with a network of man-holes and ducting for electrical cabling and pipework for water discharge. The wells are equipped with slotted PVC well casing of 110 mm internal diameter and centralisers and fine gravel filter to ensure its central location and filtering of incoming water.

The system is driven by an air compressor and the air reservoir tank that can be located up to 3 km away from the control panel and electrical supply.

control panel and electrical supply.

When the water fills the well, it enters the pump by a non-return ball valve and fills it. The low level sensor is reached, the water level continues to rise, and when it finally reaches the high level sensor, an electrical connection is made by water conductivity. These two sensors are linked to a relay, which opens a solenoid and compressed air passes through the inlet air tube to the electro-pneumatic drain, filling it with air and pushing water out of the through the outlet tube. A non return ball valve prevents any water coming back into the pumping chamber. An electrical cable, which is connected to the water level detector, is linked through the duct to a control panel. The control panel contains a relay and solenoid that controls the operation of the compressed air-supply pumps. The solenoid switch controls the compressed air supply allowing it to pass from the air compressor and its air tank to the compressed air inlet tube to the pump.

5. SCHEME DETAILS

5.1 Slope Works

An extensive series of instrumented in situ permeability and pumping tests were carried out to define the aquifer characteristics and as anticipated within a complicated heterogeneous landslide mass the hydrogeological conditions were anisotropic and varied throughout the site. In addition field trials to confirm the practical operation and performance of the siphon well system were carried out. The results confirmed that the required abstraction rates and groundwater lowering could be achieved. Estimates of permeability were calculated from a number of in-situ tests and from back analysis of pump test data. Results of these tests are summarised in Table 2. They indicate a range of permeabilities for the Upper Greensand (UGD) and Gault Clay (GCD) landslide debris of between 1.29×10^{-6} m/s and 3.95×10^{-6} m/s to 9.65×10^{-8} m/s respectively. The back analysed value of 4.43×10^{-6} m/s compared very well with the value estimated from previous studies of GDC of 4.9×10^{-6} m/s. The design requirements were specified in a schedule of minimum drawdown or trigger levels to be achieved in each siphon and electro-pneumatic wells and these were included in the performance specification.

Summary of Permeability Calculations.

Table 2.

	Test No / Depth (mbgl)	Calc. Method	Calc. perme- ability (m/s)	Av. perme- ability (m/s)
BH 102	1 / 4.9 (UGD)	BS593 Hvorslev	1.29×10^{-6}	1.42×10^{-6} (UGD)
	2 / 8 (UGD)		1.55×10^{-6}	
	3 / 11.7 (GCD)		3.95×10^{-6}	
	4 / 18.5 (GCD)		9.65×10^{-8}	2.02×10^{-6} (GCD)
BH P2	1 / 5.4 (UGD)	BS5930 Hvorslev	2.95×10^{-6}	2.60×10^{-6} (UGD)
	2 / 8.5 (UGD)		2.30×10^{-7}	
	3 / 12.5 (GCD)		1.20×10^{-7}	1.25×10^{-7} (GCD)
	4 / 18 (GUD)		1.30×10^{-7}	
BH 101	Full depth	Back analy- sis: pump test data & Dupuit- Forcheimer equation	10m spacing	3.79×10^{-6}
			20m spacing	4.43×10^{-6}
TP3	Rising head Pit at 4.5 m (gravely clay)	BRE 365 Method CIRIA 113		1.58×10^{-6}
	Rising head Pit at 3.4 m (sandy clay)			1.13×10^{-6}
TP5	Rising head Pit at 3.4 m (sandy clay)	BRE 365 Method CIRIA 113		5.67×10^{-6}
				4.12×10^{-7}

The designed drainage system was constructed between September 2003 and October 2004 and comprised 151 deep drainage wells installed to depths up to 25 mbgl. The wells are equipped with slotted uPVC well casing of 110 mm internal diameter and centralisers and fine gravel filter to ensure its central location and filtering of incoming water. A total of 116 siphon wells were supplemented with 35 electro-pneumatic wells. The wells were connected by a system of trench drains, ducts and manholes, predominantly sited within the public highway for ease of access for construction and subsequent maintenance. They were arranged in three overlapping lines across the landslide with the electro-pneumatic wells (Line 1) sited at the rear of the landslide where the required drawdown was greater than the capability of a siphon system (Fig. 7).

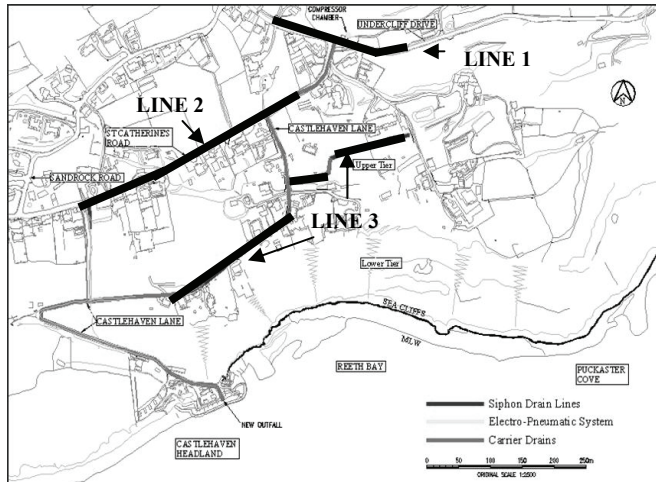


Figure 7. Layout of Wells and Drains

frastructure £1.4 million.

5.2 Foreshore Works

The foreshore coast protection works which cost £2.0 million consisted of a rock armour buttress revetment with a 3.5 m wide buried toe placed in front of the erodible 8 m high Sandrock cliff which formed the foreshore and the toe of the landslide. The rock armour, which consisted of 45,000 t of Carboniferous limestone, was imported by barge from Calais in northern France.

The primary armour ranged in size from 3 to 6 t placed on 5 to 500 kg sized core stone. The revetment had a 1 in 1.5 front slope with a 3.5 m crest at an upper level of 5m above ordnance datum.

6. SCHEME PERFORMANCE AND POST CONSTRUCTION MONITORING

It was recognised that the performance of the drainage system would require regular monitoring and maintenance during its design life to ensure that the design objectives and assumptions were met and the system dealt effectively in the long term with the consequences of climate change and continued to comply with the performance specification.

A comprehensive system of instrumentation was installed as part of the construction works and comprised:

- Pressure transducers and dataloggers installed in observation wells between rows of siphon drain and electro-pneumatic pumping wells to measure water levels (9 no.);

Each siphon well (Lines 2 & 3) was installed from the base of a manhole between 1.5 m and 3.5 m deep which connected the trench drains and improved the theoretical drawdown to approximately 12 m. The electro-pneumatic pumps were required to draw down to a minimum of 15 mbgl.

The total drainage installation quantities included a total installed well depth of 3180 m, 4800 m of twin wall plastic pipe, 1700 m of drainage trench, 206 manholes, 7500 t and 18,000 t of gravel trench backfill and spoil disposal respectively. Of a total construction cost for the project of £4.5 million, the wells, siphons and underground compressor chamber was £1.1 million and the drainage and in-

- Vibrating wire piezometers installed in selected siphon and electro pneumatic wells and linked to multi-level dataloggers to monitor water level drawdown in the wells (42 no.);
- Vibrating wire piezometers installed and sealed at discrete levels within boreholes and linked to individual dataloggers to measure pore water pressures within the landslide (6 no.);
- Borehole inclinometers installed at various locations within the landslide to monitor lateral subsurface ground movements (4 no.);
- Survey of a network of ground markers by a global positioning system;
- An open channel flow logger to measure the discharged water flows within drainage pipe works (2 no.);
- An automatic weather station to monitor local meteorological input parameters during the contract and Defects Correction Periods.

The pumping system was commissioned in July 2004 and immediately there was a response within the pumping wells and in the observation wells situated between the lines of pumping wells. Typical results of the pumping and the drawdown of groundwater levels are shown on Figure 8.

The total discharge to the sea outfall has been monitored as part of the Environment Agency's conditions for discharge consent to the sea outfall.

The rate of discharge to the outfall with time for all pumps and drains, except the western limit of Line 2, is presented in Figure 9 and compared with the daily rainfall over the same period.

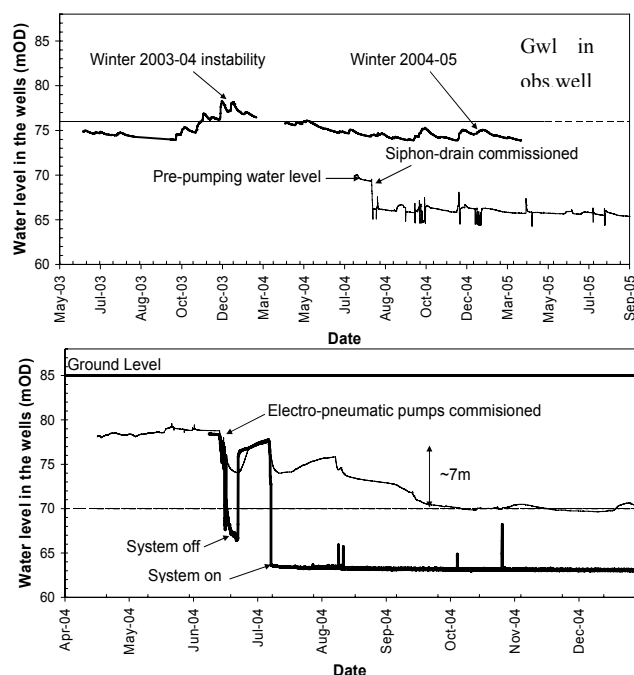


Figure 8. Groundwater Response to Pumping

It is evident from these records that there is a rapid response to rainfall and that the monitored flows of up to 350 m³/day do not exceed predicted long term base flow rates of approximately 441 m³/day or maximum allowable rates of 500 m³/day prescribed by the Environment Agency.

During the construction period the data from the instrumentation was reported initially two weekly reducing to monthly. Up to 1 year after construction the contractor and the specialist supplier (TP Geo) carried out the two monthly performance review which compared the water levels in each well with the minimum drawdown levels (Trigger Levels) given in the per-

formance specification. Following the maintenance period the scheme responsibility transferred to the Coastal Authority (Isle of Wight Council - IOWC) and on site training was given to IOWC staff on all aspects of the scheme. In addition a detailed procedure for scheme performance, monitoring and maintenance was prepared. The performance review procedure included two water levels the “Alert” and “Action” level at 1m and 2m above the Trigger level respectively in each pumping well. If monitoring identified Action or Alert levels were operating in either single or groups of wells, specified checking procedures, data interrogation and as necessary maintenance responses were identified. Of particular importance was monitoring of the possibility of siltation, iron or calcium carbonate encrustation or bio-fouling which could impede

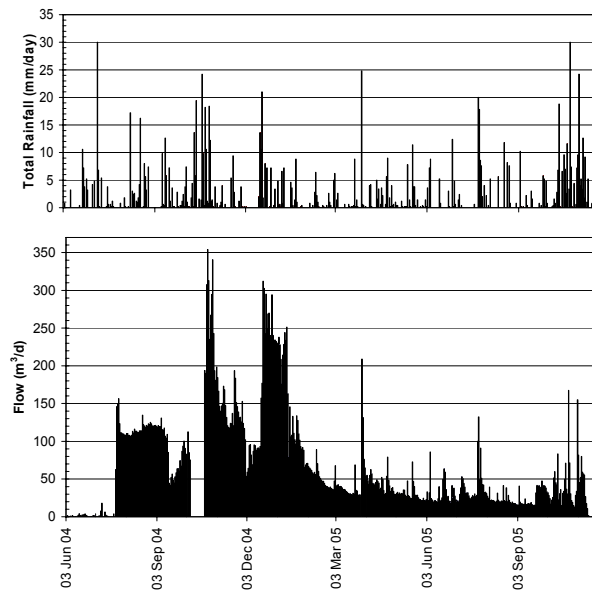


Figure 9. Rainfall and Well System Out-

the long term performance of the wells. Flushing of the system during the regular maintenance which is scheduled every 6 months, should deal with any siltation problems and additional purging with low concentration solutions of mild biodegradable organic acid are able to reduce bio-fouling.

With regular monitoring and maintenance it is anticipated that the siphon and electro-pneumatic well system will effectively improve the stability of the landslide throughout its design life whilst at the same time minimising the impact on sensitive environmental assets yet being able to respond to any changes in the climate that increase seasonal rainfall, groundwater levels and wave climate.

7. CONCLUSION

The major landslide movement at Castlehaven was caused by a combination of coastal erosion and high precipitation and groundwater levels. A close relationship between a 4 month antecedent effective rainfall of c. 380mm, groundwater levels exceeding trigger levels and the onset of movement was established for the site area. Because of the high level of environmental protection at the site and an embargo on conventional ground strengthening techniques a design that reduced groundwater levels to equivalent summer levels, when movement did not occur, was designed and constructed using a system of siphon and electro-pneumatic wells. The design was robust and included for the effects of climate change. It allowed for ground water at ground surface and was capable of delivering twice the design flow rate in

the event of predicted higher rainfall intensities. The siphon system is effective in use and environmentally sustainable; it is gravity fed and needs no power to operate. The system could also be retrofitted with additional capacity to further respond to climate change. The design approach aimed at improving the stability of the area to protect property and infrastructure but at the same time the environmental assets of the area were safeguarded by retaining appropriate levels of water within the slopes to maintain the important habitats.

8. ACKNOWLEDGEMENTS

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9. REFERENCES

- Bomont, S., Fort, D.S., and Holliday, J.K (2005). Two applications for deep drainage using siphon and electro pneumatic drains. Slope works for Castlehaven Coast Protection Scheme, Isle of Wight (UK) and slope stabilisation for the Railways Agency, France. In, Proceedings of the International Conference on Landslide Risk Management. 18th Annual Vancouver Geotechnical Society Symposium.
- Clark A.R., Storm C.V, Fort D.S & McInnes R.G. 2002. The planning and development of a coast protection scheme in an environmentally sensitive area at Castlehaven, Isle of Wight. Procurement International Conference on Instability, Planning and Management. pub. London:Thomas Telford. pp. 509-518
- Environment Agency, 1999. Overtopping of Seawalls, Design and Assessment Manual, Technical Report W178
- Gress, J.C.1996. Dewatering a landslip through siphoning drain – Ten years experiences. Proc.7th International Symposium on Landslides, Trondheim.Pub.
- Hulme, M., Jenkins, G.J., Lu, X., Turnpenny, J.R., Mitchell, T.D., Jones, R.G., Lowe, J., Murphy, J.M., Hassell, D., Boorman, P., McDonnald, R., and Hill, S. 2002. Climate change scenarios for the United Kingdom: the UKCIP02 scientific report, Tyndall Centre for Climate Change Research, School of Environmental Sciences, University of East Anglia, Norwich, UK.
- Hulme M. & Jenkins, G.J., 1998. Climate changes scenarios for the UK: Scientific report. UKCIP Technical Report No. 1. Climate Research Unit, Norwich.
- Hutchinson, J.N. 1991. Theme lecture: The landslides forming the South Wight Undercliff. International conference on slope stability engineering developments and applications, R.J. Chandler Ed., 157-168, London: Thomas Telford.
- Hutchinson, J.N., & Bromhead, E.N., 2002. Isle of Wight landslides. In R.G. McInnes & J. Jakeways (ed.), Instability planning and management, 3-70. London: Thomas Telford.
- Ibsen M. & Brunnsden D. 1996. The nature, use and problems of historical archives for the temporal occurrence of landslides, with specific reference to the south coast of Britain, Ventnor, Isle of Wight. *Geomorphology* 15. pp 241-258
- Ibsen, M-L. 2000a. The impacts of climate change and instability – some results from an EU LIFE project. In R.G. McInnes & J. Jakeways (ed.). Instability planning and management, 609-616. London: Thomas Telford.
- Ibsen, M-L. 2000b. The impacts of climate change. In: R.G. McInnes & D. Tomallin (ed). European Commission LIFE project, LIFE-97 ENV/UK/000510. Vol. 1. Chapter 6.IPCC, 2002. Climate change 2001: Summary for policy makers. The Intergovernmental Panel on Climate Change third assessment report.
- IPCC,2002. Climate change 2001: Summary for policy makers. The Intergovernmental Panel on Climate Change third assessment report. Available at <http://www.ipcc.ch/index.htm>.
- Moore, R., Lee, E.M., & Clark, A.R. 1995. The Under cliff of the Isle of Wight, a review of ground behaviour. Rendel Geotechnics. Pub. Cross.