Monitoring and Remediation of a Slow Moving Landslide

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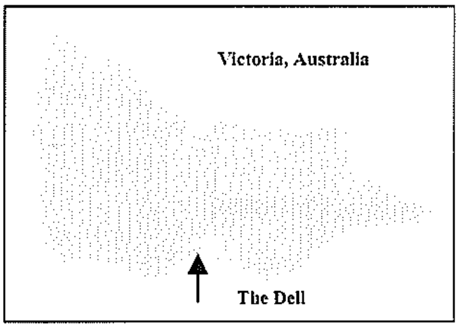
Summary: An area in the township of Clifton Springs called The Dell is a prominent coastal amphitheatre feature of the Bellarine Peninsula in southern Victoria. The local community and visitors to the area have used The Dell as a safe access point to Corio Bay and for its recreational facilities.

The Dell comprises a relatively flat coastal area surrounded by steep slopes. The groundwater springs, after which the township of Clifton Springs is named, emerge at the foreshore. The mineral springs have been known and utilised dating back to the early 1800's.

A long and continuous tension crack near the crest of the amphitheatre developed during the latter months of 2001. Visually observed acceleration of slope movements prompted the authorities to close the area.

The slow moving landslide has since been subject to geotechnical investigation and monitoring of ground movements, surface movements and pore water pressures. Remedial measures have been put in place and these have been successful in slowing the landslide.



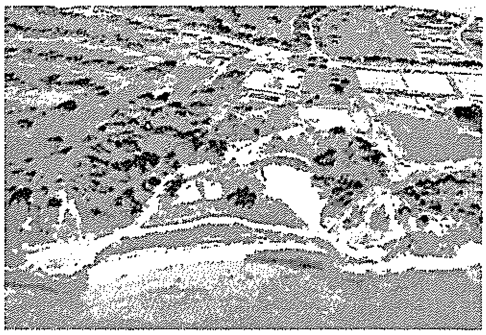


Figure 1. Location Plan and Aerial Photograph

INTRODUCTION

The Dell (Figure 1) is located on the foreshore of Clifton Springs on Bellarine Peninsula, Victoria. The springs have been known for a long time dating back to the early 1800's. Development of the springs commenced in the 1870's with a hotel and resort being constructed by 1880. The hotel was destroyed by fire in the mid 1920's and the wells and pipework associated with the springs have since fallen into disrepair. Since this time, the Dell has provided an easy access point to the bay and foreshore and has been only recently redeveloped by the City of Greater Geelong through the installation of children's playground and barbeque facilities. The site is about 150m long and 100m in width. The lowest levels on the site are at sea level (RL0m) and the crest of the steep slopes is at a level of around RL 30m.

Closure of the The Dell occurred at the end of July 2002 following the development of a large landslide. The first telltale signs of the landslide were noticed in October 2001 with the observation of a tension crack on the upper slopes of the natural amphitheatre of The Dell. Subsequent monitoring has indicated that movement has continued to occur over a large majority of the site. The movements were of great concern to the governing authorities and the local community due to safety issues and the loss of public amenity, as well as possible effects on infrastructure (pressure sewer, swimming pool and golf club complex) near the slope crest.

GEOLOGY

Clifton Springs is located on the northern coast of the Bellarine Peninsula, which has been formed by an uplifted block between the Bellarine fault to the east and an east-dipping fault to the west. The northern coast of the peninsula is structurally controlled by the Curlewis Monocline. The Bellarine peninsula consists mainly of Tertiary age volcanic rocks and sediments, which overlie Cretaceous age rocks of the Otway Group.

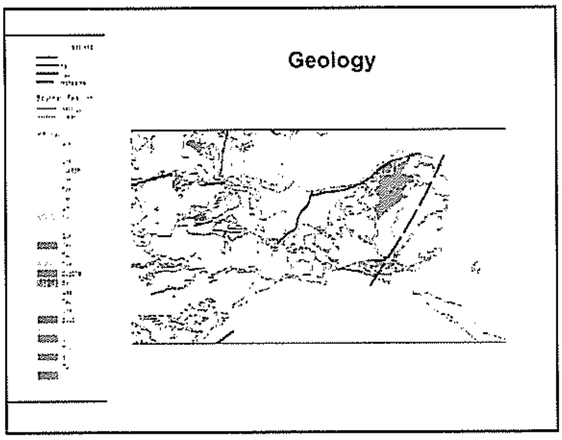


Figure 2: Site Geology.

The site is underlain by the Werribee formation which is a regional aquifer and the source of the springs. Several other aquifers have been identified beneath the site including what have been called the mid level sands and a basalt layer underlying these.

BACKGROUND TO THE CURRENT SITUATION

Small scale erosion has been noted at The Dell over many years. Cracking and a step in the site access road was noted soon after road sealing in April 2000 and after heavy rainfall. A long and continuous tension crack in the upper slopes adjacent to the swimming pool and extending down to the access road was first noted in November 2001. The ongoing development of the tension crack was then only visually monitored until April 2002. A risk management strategy and Survey monitoring of the site were implemented in early May 2002.

Visual monitoring of the tension crack noted widening and initial survey results indicated that the foreshore retaining wall , some 150m from the tension crack, was rising. A decision was taken at this time to close and fence the site and emergency response protocols were implemented.

GEOTECHNICAL INVESTIGATION & INSTALLATION OF INSTRUMENTATION

Geotechnical investigation, comprising 48 boreholes and laboratory testing was conducted. The monitoring schemes which have been installed at the site, include:

- Piezometers within aquifers and the soils interpreted to contain landslide shear planes,
- Inclinometers to monitor changes in borehole orientation with time. These instruments allow the assessment of shear plane levels within the soil mass;
- Extensometers connected to a mobile phone modem to allow remote, real time monitoring of movements at the tension crack; and
- Precise line and level surveying to monitor horizontal and vertical movements of specific points across the site.

GEOLOGICAL MODEL

The geotechnical investigations at the site highlighted its geological and hydrogeological complexity, and it became apparent that some structural geological control (faulting, folding,) has impacted on the formation of The Dell.

Geological modelling, using the results of the investigation and three dimensional mapping by Dalhaus Environmental Geology, yielded the following conclusions :

- The Dell is a geomorphological feature formed by mass wasting. Historical landslides have redistributed the geological materials at the site;
- The axis of the Curlewis Monocline is probably coincident with the southern boundary of the amphitheatre;
- Parts of the current landslide are probably reactivations of historic landslides;
- The Werribee Formation forms an extensive regional aquifer under the site;

- The mid-level sands probably form a perched aquifer of limited extent in the main body of the landslide.

MECHANISMS OF FAILURE

The location of failure surfaces and the direction of movement within the soil profiles can be assessed from inclinometer profiles. In addition, the survey results allow the horizontal and vertical surface movements to be assessed in various parts of the site. Good agreement was obtained between the results from different types of monitoring and the combination of this monitoring data allows the failure mechanisms acting at the various parts of the site to be better understood. Figure 3 presents a summary of the relevant monitoring data at the site. Movement directions from the survey, in particular, were generally in agreement with inclinometer results.

The survey and inclinometer results indicate that the steep area of the site to the south of the access road is moving in a NW direction. The area to the north of the access road is moving in a NNW direction. The eastern slopes appear to be moving in a WNW direction; and

Ground levels are dropping along the access road, on the southern steep slopes and on the eastern slopes. Points at the base of the steep southern slope and on the foreshore retaining wall are rising. The combination of the monitoring results and visual observations suggest that there are 3 main failure mechanisms operating at the site (Figure 4).

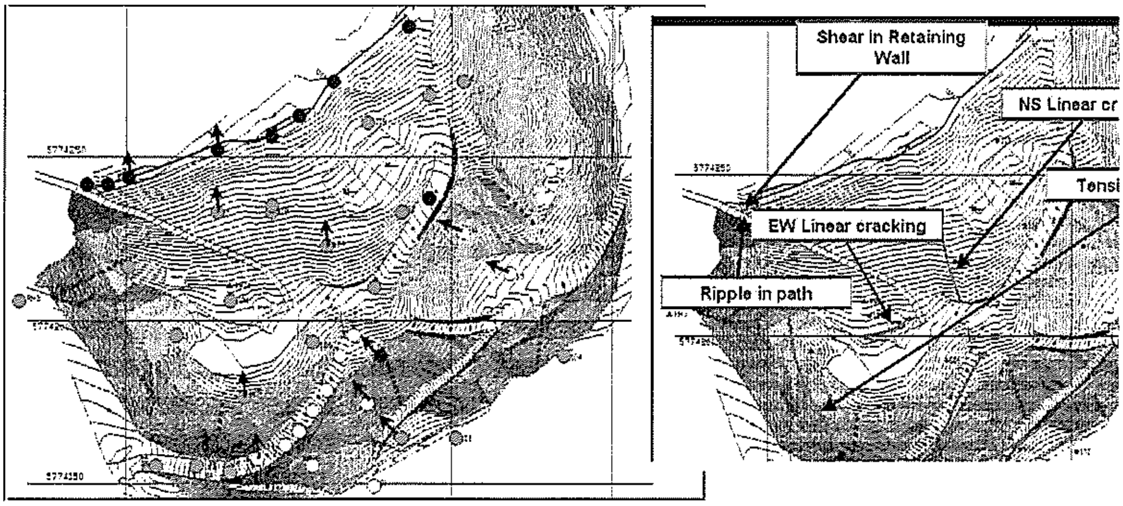


Figure 3. Monitoring and Site Observations

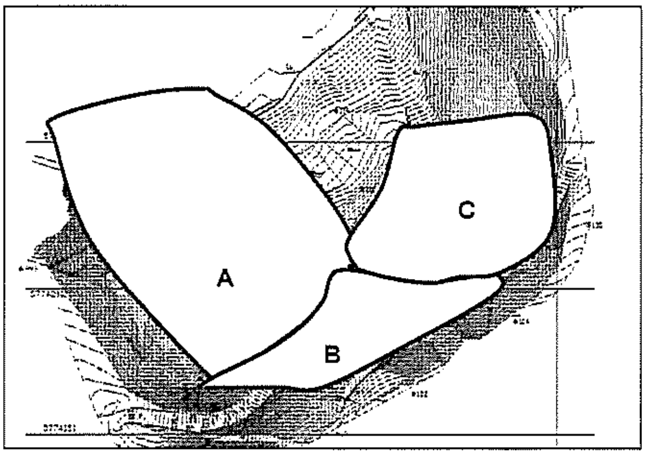


Figure 4. Postulated 3 Block Mechanism

ANALYSES

Each of the postulated landslide blocks (A to C) were analysed and the strength parameters mobilised along the failure surfaces (defined by the inclinometers) were back calculated. These strength parameters were compared to values obtained from laboratory testing results to ensure that sensible parameters were adopted. A cross section showing the inferred geology and the analysed block A is shown in Figure 5.

Block B has been interpreted as a first time slide in a natural slope which has been modified by engineering work and average shear strengths on it's failure plane were taken to be intermediate between the peak strength and residual strength.

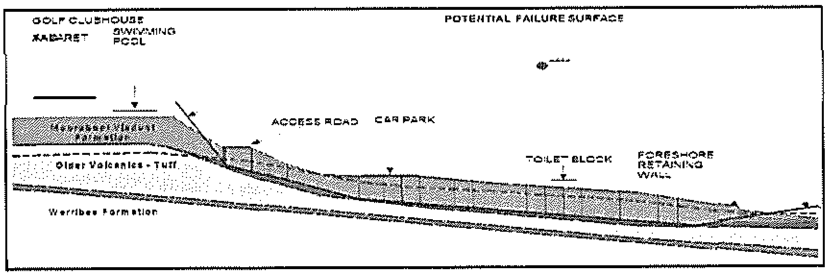


Figure 5. Cross Section

However, for inferred reactivation slides (Blocks A and C), shear strengths mobilised during failure were considered to approach residual values. In addition, an attempt was made to assess the stability of Block C before the first-time failure in this block, by restoring the slope to its estimated original configuration. Three dimensional mapping was utilised to assist in this process. This allowed a check to be made on the intermediate strength utilised for the analysis of Block B. The back calculation process resulted in the following strengths along the failure surfaces in the 3 blocks postulated at the site.

Table 1. Back-calculated Shear Strengths.

Block	**Average Effective Shear Strength (degrees) on failure surface**
A	11 (residual)
B	18 (intermediate)
C, C (pre-failure)	12 (residual), 18 (intermediate)

The second stage of the analysis considered the potential increase in factor of safety for each block following the completion of various types of remedial works. .Table 2 shows the types of remedial works considered and some of these were adopted in the analyses.

Table 2: Summary of Potential Remedial Options

Blocks	**Remedial Measure**	**General Description**	**Advantages**	**Disadvantages**
A,B & C	Ground Water depressurisation (Passive or active Systems)	Reduce groundwater pressure and hence porewater pressures around failure surface lowered through installation of vertical wells that tie into horizontal drains or through installation of pumped vertical wells.	Passive system – low maintenance costs. Minimal disruption to site as all structures are below ground level. In long term could be part of spring reactivation. Limits water pressures in ground during heavy rainfall events. Unobtrusive works.	Passive System - Only relatively small reductions in pressure possible. May not satisfy public that site is safe. Active System - Require ongoing maintenance and monitoring.
A,B & C	Ballast at Toe of Slope	Ballast placed at toe of slopes.	Requires no maintenance. Robust technique as effects are easily evaluated.	Major disruption to site – landscaping required. Source of fill needs to be found. Requires access by heavy plant.
B	Anchors or soil nails	Row of anchors or soil nails in upper slope	Stabilising effect on both large failure and smaller failures in upper slopes.	There may be no suitable anchor points,

Blocks	Remedial Measure	General Description	Advantages	Disadvantages
B	Gabion Wall	Retaining wall at toe of slope	Minimal land take required.	Only minor effects on large failure.
B	Reshaping of slope crest	Earthworks to reduce steepness of upper slope	Requires only low maintenance. Robust technique.	Possible disruption to site and adjoining facilities.

Table 3 is a summary of the stability analyses results for the preferred remedial works after a target factor of safety of 1.3 was adopted for long term stability of The Dell.

Table 3. Summary of Stability Analyses Factors of Safety

Analyses / Block	A	B	C
Prior to Remedial Works (assessed)	1.0	1.0	1.0
Following Groundwater Depressurisation	1.1	1.06	1.01
Long Term Remedial Works Types Adopted in the Analyses	Filling at toe	Gabion wall at toe, reshaping at crest	Filling at toe
Following Long Term Remedial Works	1.3	1.3	1.3

REMEDIAL WORKS IMPLIMENTED

Emergency Remedial Works

In response to visual observations of acceleration of movements at the site in July 2002, the following emergency remedial works were installed:

Drainage remediation works to prevent direct discharge of runoff into the Dell amphitheatre; Emergency pumping of groundwater from two of the site investigation bores; Partial sealing of the tension crack; and Emptying of the swimming pool, which was noted to have cracked.

Short Term Remedial Works

Additional remedial works were proposed for the site in June 2003, involving groundwater depressurisation. These works were the first stage of site remediation, with further remedial works planned for a later date. This work included the installation of 8 active extraction wells. Two of these wells were installed at the crest of the site, adjacent to the swimming pool, within the deeper Werribee Formation sands. Four wells were installed in the central car park, within the mid-level sands. The remaining two wells were installed within existing geotechnical investigation bores, screened within the basalt. With this system in place, the 3 main aquifers identified at the site were targeted for de-watering.

INSTRUMENTATION RESULTS FOLLOWING REMEDIAL WORKS

Piezometer Results

There has been a positive response to the groundwater depressurisation as observed via the piezometers at the site. The critical piezometers are those installed within the tuffaceous soils, in which inclinometer results have indicated that the basal shear planes of the landslides exist. These piezometers should therefore be measuring the porewater pressures at or near the shear planes.

The results indicate that a cone of depression (of groundwater pressures) has been established at the site, centred upon the car park, where the groundwater depressurisation of the mid level sand and the basalt has been ongoing for a number of months.

Typical plots of piezometric level against time for the vibrating wire piezometers is presented in Figure 6, which indicates that the porewater pressures within the tuff are still reducing and that the full beneficial effect of groundwater depressurisation may still be developing. The graph on the left indicates how some of the prezometers responded to emergency remedial works before active pumping began.

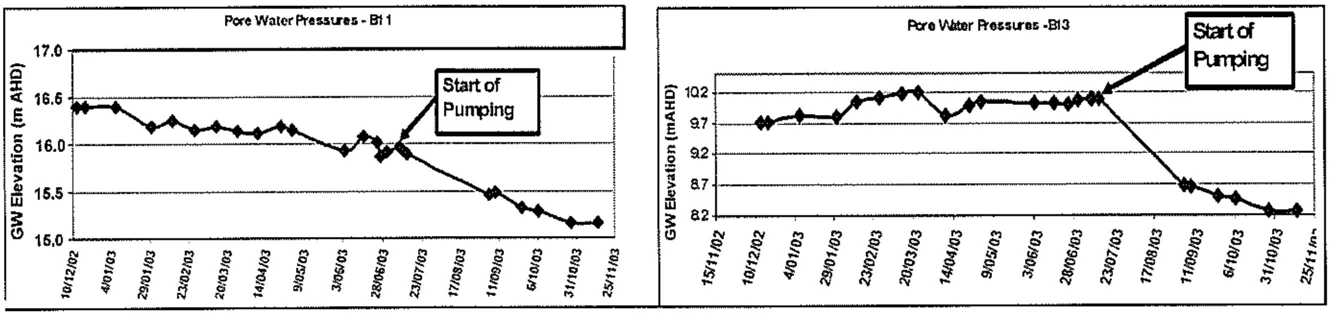


Figure 6. Typical Piezometer Results

Extensometer Results

Before groundwater depressurisation, extensometer No 6, which is located in one of the more active parts of the site, indicated a general increasing trend of movement. However, during groundwater depressurisation the extensometer has stabilised and has shown relatively steady readings. This is highlighted by the extension versus time plot for this extensometer, presented in Figure 7.

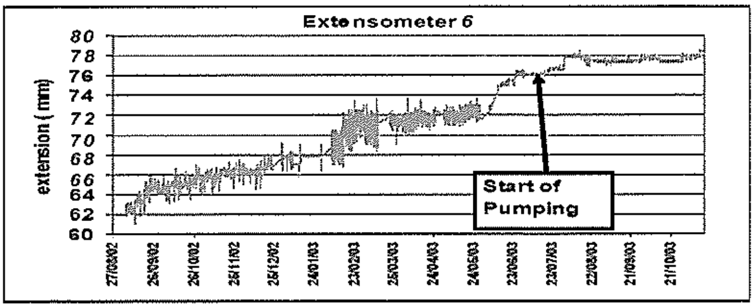


Figure 7. Extensometer Results

Other extensometers around the site have also shown a stabilisation or decrease in movement since commencement of groundwater depressurisation. A decrease in reading would represent a closing of the tension crack. The implications for this decrease are not resolved at present, although it has been hypothesised that they may be a result of some surficial creep due to horizontal relaxation above the tension cracks.

Inclinometer Results

Inclinometer results suggest that there has been some reduction in the rate of slope movement in some of the inclinometers at the site since the commencement of groundwater depressurisation. This is highlighted on the plots of rates of movement presented in Figure 8. Inclinometer results from other bores are inconclusive at this time and additional monitoring data is required to assess trends. However, it should be noted that even though there has been significant rainfall over winter 2003 (96mm in July), there has been no increase in rates of movement in sections of the site which had been accelerating coincident with high rainfall.

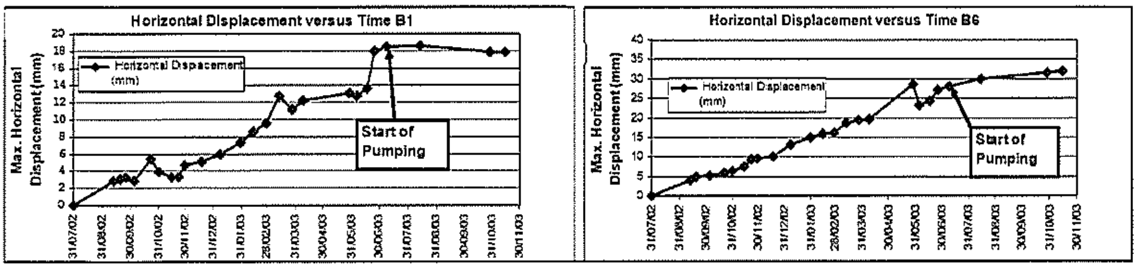


Figure 8: Inclinometer rates of Movement

Survey Results

The surveying before groundwater depressurisation indicated that horizontal surface movement downslope towards the coast may have been up to around 24 mm since August 2002 (average of 2.7 mm/month) and was accelerating. It is difficult to compare the survey results during groundwater depressurisation with those before, as a revised, more comprehensive, survey regime was set up at the site concurrently with the groundwater depressurisation. However, the following general observations can be made:

- Vertical movements have been generally less than 5mm in the 3 months since depressurisation started;
- The foreshore retaining wall had risen by up to 13mm but levels are steady post-depressurisation:
- The horizontal movements appear to have stabilised, although there are still some vertical movements. Continued monitoring will be required to establish any patterns of movement; and
- The eastern slope crest has shown a 3mm drop, whereas the base of these slopes has risen by 6mm.

Overall, the survey results indicate that groundwater depressurisation has arrested the acceleration of movement but the site is still moving, albeit relatively slowly. More survey data sets will be required to establish movement patterns and to distinguish survey error limitations from "real" movements.

CURRENT SITUATION

The Blocks A, B and C models were re-analysed using the piezometric surfaces measured in the shear plane materials after groundwater depressurisation. The re-analysis results indicate that there has been an increase in the stability of Blocks A and B of around 5 to 10% due to groundwater depressurisation operations. There has been no significant increase in the stability of Block C at this time, which is not unexpected, as this block is relatively distant from the groundwater depressurisation works and the soils present in this area are of low permeability. Although the groundwater depressurisation has arrested the acceleration of the landslide and has gone some way to stabilising the site in the long term, the target factor of safety has not been achieved. Additional remedial works are therefore required for each of the 3 landslides blocks present at the site.

FURTHER REMEDIAL WORKS PLANNED

The increase in factors of safety calculated to be achieved by the groundwater depressurisation works is insufficient to meet the target factor of safety. Further remedial works, including earthworks and soil nailing, are required to increase the stability of the site to an acceptable level.

CONCLUSIONS

Monitoring, visual observations and geological modelling suggest that there are 3 main failure mechanisms acting at The Dell site: A translational block extending from the access road to the foreshore, probably the reactivation of an historic landslide (Block A); A rotational block encapsulating the steep slope to the south of the site, probably a first time failure (Block B); and a rotational block in the eastern portion of the site, probably another reactivated historic landslide (Block C).

The reactivated landslides (A and C) are unlikely to fail rapidly, as the shear strengths mobilised in these slides is probably at or near a residual value; The first time slide (Block B) may fail more rapidly in a brittle manner.

Initial post-groundwater depressurisation monitoring has indicated that:

- Piezometric levels within the tuff have been reduced in the central part of the site;
- Some inclinometers (generally within Block A) have shown decreased rates of movement. Reductions in the rates of movements of other inclinometers are not proven, although there has been no increase in rates of movement in the wet, winter months of 2003; and
- Tension crack extensionmeter readings support the inclinometer readings in showing that the acceleration of the landslide has been arrested and small steady movements are being recorded;

Theoretically, the reductions in the piezometric surfaces should have increased the stability of the site as a whole and calculations suggest that an increase in the factors of safety of between 5 and 10% is appropriate.

A target factor of safety of 1.3 has been adopted as a requirement in the remediation of the site.

The long term remedial works outlined meet the target factor of safety in conjunction with the groundwater depressurisation already active at the site. It may be possible to reduce pump usage to during spells of wet weather or when trigger water levels within bores are reached. This is likely to involve the installation of remote reading prezometers connected to the pumps.

Additional remedial works are required to address the local stability of the access road, including soil nails in slopes below the road and reconstruction of the road using a reinforced pavement.

Additional security, safety, management protocols and risk issues also need to be addressed prior to final site re-opening.