REAL TIME MONITORING OF A ROAD EMBANKMENT SUBJECT TO COASTAL EROSION AND SLOPE INSTABILITY

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ABSTRACT

In this paper, monitoring the stability of a road embankment at Lawrence Hargrave Drive along the Illawarra coast is discussed. The embankment supports a coastal road and had been subject to an ongoing process of coastal erosion at the embankment toe and instability due to elevated piezometric pressures after heavy or sustained rainfall. In this paper, the stability issues and potential failure mechanisms are discussed. In order to allow a long term solution to be developed, an interim real time, web based monitoring system was installed. An outline of the investigations and geotechnical conditions at the site is presented. The paper discusses the monitoring system, including the instrumentation used (in-place inclinometers, piezometers, rain gauge, wave height buoy and web cam, the development of monitoring plans, the integration of instrumentation to develop a Trigger Action Response Plan (TARP), the development of trigger levels based on antecedent monitoring data, and the successful operation of the system.

1 INTRODUCTION

Lawrence Hargrave Drive is a coastal road along the Illawarra coast north of Wollongong NSW. North of the Seacliff Bridge viaduct, the road is constructed on an embankment rising from the shore below. The site location is shown in Figure 1 and an oblique view of the site is presented in Figure 2. The embankment was originally constructed over old landslide debris and colluvium, which now forms the base of the slope. The slope has been subject to episodic instability, the formation of a slip zone due to elevated piezometric pressures within and behind the slope, and coastal erosion at the toe. The slip is located to the north of the Seacliff Bridge and south of a major drainage line. The slip is a cuspate slip with the back scarp located on the slope batter east of Lawrence Hargrave Drive and the toe located at the base of the slope, approximately at sea level. The toe has been over-steepened as a result of coastal erosion forces. With the development of further localised instability in the slope, this slip is referred to as the “main slip”. Following heavy rain and high seas in July 2011, the main slip back scarp regressed towards the pedestrian footpath, and significant material was eroded from the toe of the slope by wave action. Surface monitoring results obtained from Roads and Maritime indicated a significant downward movement had occurred in the lower slope and flanks of the slipped mass, leading to more active management of the slope and acceleration of remedial option development. Heavy rain and wave action caused further movement of the slope in March 2012

By this time a 2.5m back scarp had formed at the rear of the main slip and slope movements had become more severe. There was a concern that the instability would regress further back into the embankment and endanger first the footpath and rest area immediately behind the slope and ultimately the road itself. Geotechnical investigations and analyses were carried out and monitoring and slope management programs were put in place as an interim measure to allow the development of permanent stabilisation measures.

Lawrence Hargrave drive is an iconic route on the Australian tourist map, and any remediation measures would need to be sensitive to this aspect in addition to the environmental, urban design and technical challenges at the site.

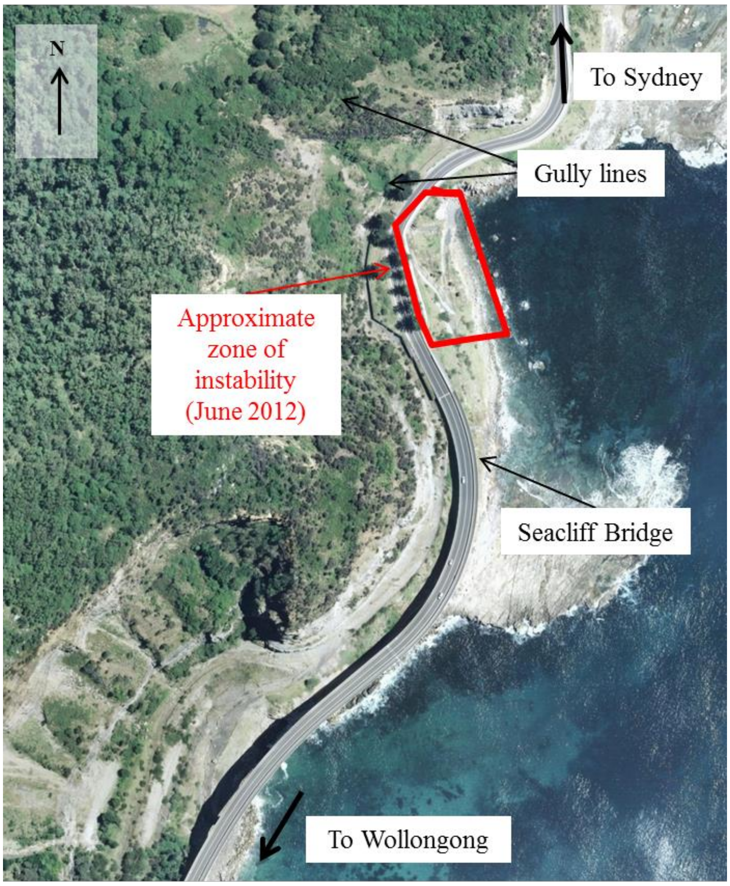

Figure 1: Site location

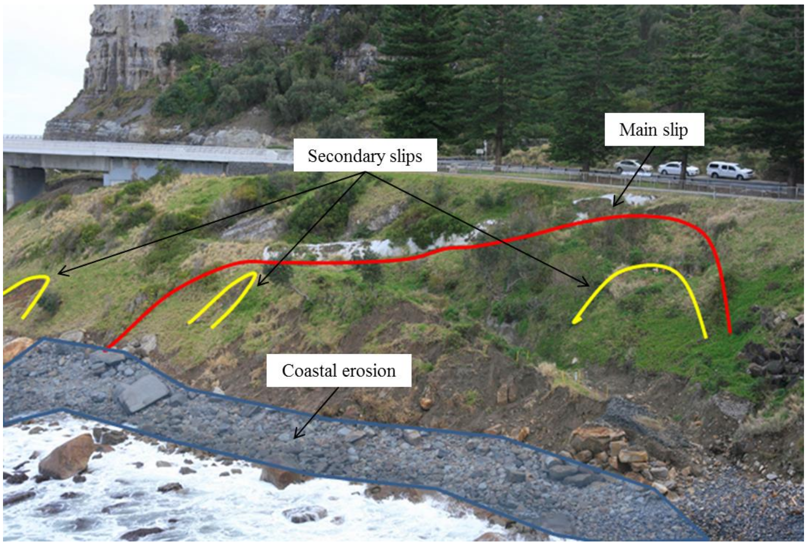

Figure 2: View of the main slip, showing secondary slip features and coastal erosion

2 GEOTECHNICAL CONDITIONS

Geotechnical investigations were carried out to assess subsurface conditions for stability analyses. A description of the ground conditions is presented in Table 1 and a cross section showing the inferred geotechnical conditions for analysis is shown in Figure 3.

Table 1: Summary of General Ground Conditions

Unit	Thickness of unit (m)	RL at base of unit (m AHD)	Description
Colluvium	0.5 to 21.0	-0.35 to -1.25	Highly variable but predominantly composed of Clayey Gravelly Sand and Clayey Sandy Gravel with numerous cobbles and boulders of sandstone (approx. 15%-20% by volume). Subordinate pockets of clay dominated the fine fractions. Generally moist, becoming wet with depth. Generally medium dense.
Gravelly Sandy clay (possible crushed zone)	0.05 to 0.4	-0.47 to -1.45	Generally Sandy Gravelly Clay of high plasticity, light grey. Generally firm to stiff, estimated moisture content greater than plastic limit. This is possibly a mix of crushed bedrock and colluvium.
Bedrock (Wombarra Claystone)	Base not seen	Base not seen	Generally interbedded fresh medium to high strength sandstone and siltstone, sub-horizontally bedded (possibly dipping northward at less than 5°).

Underlying the colluvium is a crushed zone interpreted as possible crushed / reworked bedrock and colluvium, which could have a lower shear strength than the overlying colluvium. Underlying the crushed zone is the Wombarra Claystone bedrock, which forms a gently northward dipping surface that is likely to represent a relict wave cut platform. The depth to the top of the bedrock below the surface is about 21.5 m at the embankment crest and about 1 m just beyond the toe of the main slip.

Two possible slope failure mechanisms were identified below the road:

- A deep seated failure, which could potentially result in tension cracks and ground displacement behind the crest of the slope and would be more likely to impact on the road.
- A shallow slip, which was less likely to have a direct impact on the road.

Back analyses were undertaken using the two failure modes described above. Both failure planes pass through the existing back scarp. The shallow failure plane remains within the colluvium. The deep seated failure plane passes through the previously identified crushed zone at the base of the slope. The ground model and geometry of the failure mechanisms is presented in Figure 3 below.

Monitoring, investigations and analyses combined to confirm that the shallow failure mode was more likely to occur. This was an important outcome as it suggested that other than local instability at the 2.5m high back scarp, there was a relatively stable zone behind the main slip. This zone could be managed by monitoring in the short term whilst stabilisation measures were developed. Stabilisation measures were likely to include substantial marine protection measures.

The analyses also indicated that slope stability could be improved by lowering the piezometric pressures in the slope. This provided an option to include depressurisation wells as an option for interim stability management of the slope. However, lower piezometric levels alone did not offer sufficient improvement in stability to meet the target design safety margins. Hence, drainage measures in isolation were not considered to be a feasible long term remediation option.

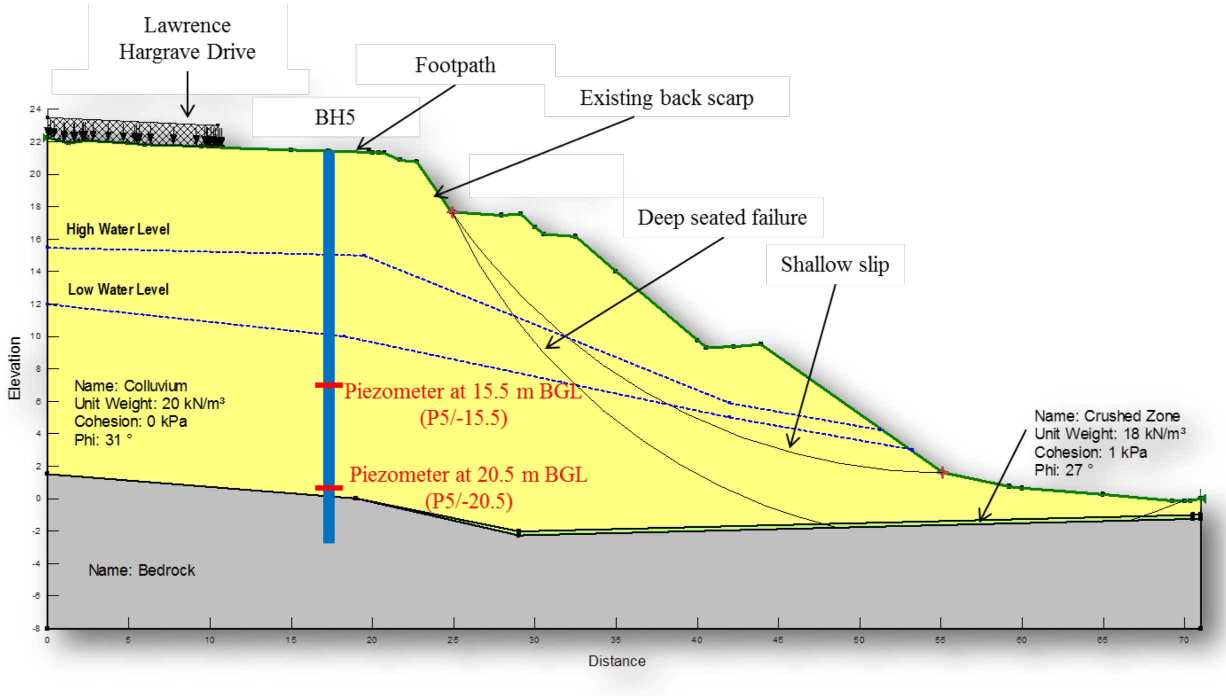

Figure 3: Ground model of main slips

The stability of the embankment is influenced by multiple factors including geotechnical, marine and environmental conditions. Therefore, it was necessary to develop a monitoring plan for this embankment to integrate these various aspects.

3 INTEGRATED MONITORING PROGRAM

A monitoring program was developed to provide information so that Roads and Maritime could manage the road asset on the embankment behind the main slip in such a way as to mitigate the landslip risks to road users and pedestrians. The monitoring program would be required as an interim measure as the development of permanent stabilisation measures were potentially time consuming due to the need for physical coastal modelling. The monitoring program was designed to manage the road asset behind the main slip. Local instability of cuts, fills and scarps on the slope were not captured as part of the program due to the dynamic nature of the slope.

The monitoring program was designed as an integrated system that would allow data from various monitoring instruments to be recorded by data loggers and transmitted to a web-based portal. This data could then be accessed by personnel involved in the project. This integrated, real-time monitoring program allowed the identification of relationships between the various parameters recorded by the monitoring devices.

3.1 INSTRUMENTATION

The borehole locations and the instrumentation in these boreholes are presented in Figure 4.

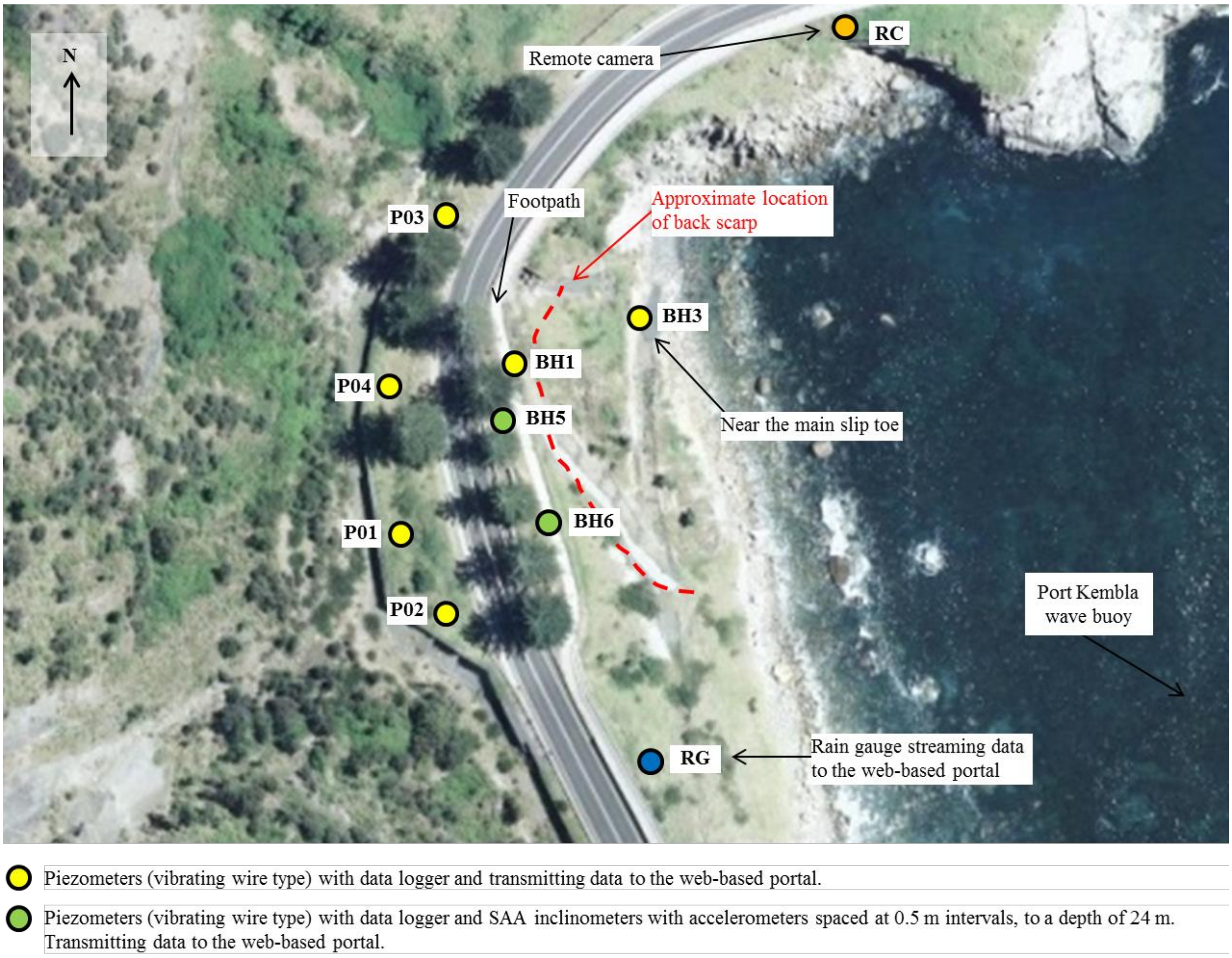

Figure 4:, Borehole locations and Instrumentation plan.

Boreholes BH5 and BH6, which contained the main geotechnical instrumentation (vibrating wire piezometers and SAA inclinometers), were located behind the slip failure. This was done for three reasons:

- It was likely that due to the dynamic nature of the slip zone, instrumentation located across the slip would be rendered inoperative in a short space of time
- The aim of the monitoring program was to manage risk to road users. Locating the instrumentation immediately behind the slip would allow any regression of the slip into the marginally stable area behind the slip zone to be detected by the instrumentation
- The ground behaviour in the slip zone was reasonably well understood. Further detailed monitoring in this area would provide little further useful information in relation to either slip behaviour or road user risk mitigation.

3.2 REMOTE REAL-TIME DATA COLLECTION SYSTEM

A web-based online portal with data base application, Vista Data Vision (VDV), was adopted to collect and manage monitoring data from the field instruments. The portal provided real-time updates of the incoming data and setting of pre-programmed triggers. It allowed automatic transmission of trigger exceedances via SMS and email to selected recipients.

3.3 MONITORING PLAN

A monitoring plan focussed on mitigating road user risk was developed for the instrumentation. The essential components of the monitoring plan are presented in Table 2 below. The table provided the basic monitoring requirements for each instrument so that all personnel involved in the project could quickly identify their responsibilities and understand the capabilities of the instrumentation. Also, if any amendment to instrumentation was required, the purpose of the existing instrumentation would be understood and considered.

Table 2: Monitoring Plan

Instrument	Accuracy	Frequency	Responsibility	Purpose
Rain Gauge	0.1 mm/hr	Recorded in mm/half hour intervals Transmitted to web portal every 15 minutes	AECOM	Provide rainfall intensity and duration data in the GD4 area
Piezometers	0.01 mm piezometric pressure	Transmitted to web portal every 15 minutes	AECOM	Provide piezometric pressure data at particular points within the main slip in GD4
Inclinometers (SAA type)	1 mm/m	Transmitted to web portal every 15 minutes	AECOM	Provide ground movement data west of the main slip
Inclinometers (Manual)	0.5 mm/m	Monthly	AECOM	Provide ground movement data in the slope below the piled roadway to the north of the slip
Bureau of Meteorology (BoM) rainfall predictions	BoM standard	Daily	AECOM	Provide predictions of rainfall and duration in slip area
Wave Buoys (Sydney and/or Port Kembla)	MHL standard	Daily	AECOM	Provide significant wave height information
Remote Camera		Transmitted to web portal every 15 minutes	AECOM	Provide visual image of slope

3.4 RELATIONSHIPS BETWEEN MONITORED PARAMETERS

3.4.1. GENERAL

Using the data obtained from the instrumentation in the monitoring plan, it was possible to develop relationships between the various monitored parameters. Data from various sets of instruments were plotted to the same time scale as in the example in Figure 4. The figure was plotted to assess the interaction between rainfall, piezometric pressure and wave height with inclinometer movement. The data set presented in the figure is from borehole BH5, located immediately behind the main slip. The following data are plotted from this borehole in Figure 4:

- Daily rainfall
- 30 day cumulative rainfall.
- Maximum, significant wave height and tide obtained from MHL.
- Piezometric pressure (15.5m (Piezometer P5/-15.5) and 20.5m (Piezometer P5/-20.5) below ground level (BGL)).
- SAA inclinometer movements at rock interface (20m below ground level), 10m below ground level and surface.
- Rate of pore pressure rise (rates of rise greater than +/-3m/day not shown in order to filter out anomalous responses due to the effects of surface water).

In the sections that follow, the relationship between piezometric pressure and inclinometer movement is discussed in some detail and in more general terms for other data. The storm event in March 2012 presented in Figure 4 provides much of the data for establishing relationships between the various monitored parameters and referred to from here on as ‘the storm event’.

3.4.2. RELATIONSHIP BETWEEN PIEZOMETRIC PRESSURE AND RAINFALL

Following rainfall, pore pressures rise rapidly at the piezometer at P5/-15.5 with very little lag, suggesting the recharge is close to the slope. The effect of surface water ingress can be clearly seen with sharp spikes in piezometric pressure associated with rainfall. It would not be practical to accommodate these spikes in a response plan. However, after relatively simple upgrades and repairs to surface drainage, it can be seen from the figure that these sharp spikes were eliminated after February 2012. It illustrates graphically the importance of surface water management in slope risk management. After this time the rate of pore pressure rise associated with rainfall is approximately 1.4 m/day.

Lower piezometric pressures and flatter and smoother recharge curves lagging approximately three days behind rainfall were recorded at the piezometer at 20.5m BGL suggesting a longer flow path for groundwater at this level. The rate of piezometric pressure rise after rainfall at 20.5m BGL is approximately 0.4m/day, a significantly lower rate of rise than at P5/-15.5, and less critical in terms of response planning.

Towards the end of the monitoring period, piezometric pressures do not rise after rainfall due to the effects of the depressurisation wells.

3.4.3. RELATIONSHIP BETWEEN PIEZOMETRIC PRESSURE AND INCLINOMETER MOVEMENTS

It can be seen from Figure 5 that there are a number of distinct patterns of movement in the inclinometer at the three levels plotted in the figure.

At the rock interface (-20.5 m) there is distinct inclinometer movement of approximately 1 mm at the time of the storm event. From review of the full height time displacement curves for this inclinometer, this movement was interpreted as shear movement within the crushed zone at the base of the colluvium as a result of elevated piezometric pressures. It can be inferred from the data that once piezometric pressures at P5/-15.5 exceed approximately 14 m, there was a possibility of deep seated failure occurring. There was no further significant movement in this zone during the monitoring period.

At the slope mid-level (-10 m), piezometric pressures during the storm event once again result in mobilisation of the slope with movements in the order of 2 mm. Whilst there is some oscillating movement in the inclinometer after the storm event, the slope remains stable.

At the surface, the inclinometer records indicate that movement can occur at less severe rainfall events such as that in February 2012. However, the greatest sustained movement occurs during the storm event when piezometric pressures are in excess of approximately 14 m. These movements cease once piezometric pressures drop.

Further surface movement occurs during sustained rain in April. This movement was considered to be the result of saturation of the ground surface and subsequent movement towards the back scarp of the main slip as a result of lack of restraint at the free face of the scarp.

Surface movement occurred once again in June/July but with very little rainfall and without any associated rise in piezometric pressure. This was attributed to further ground movement within the main slip and subsequent movement of the ground behind.

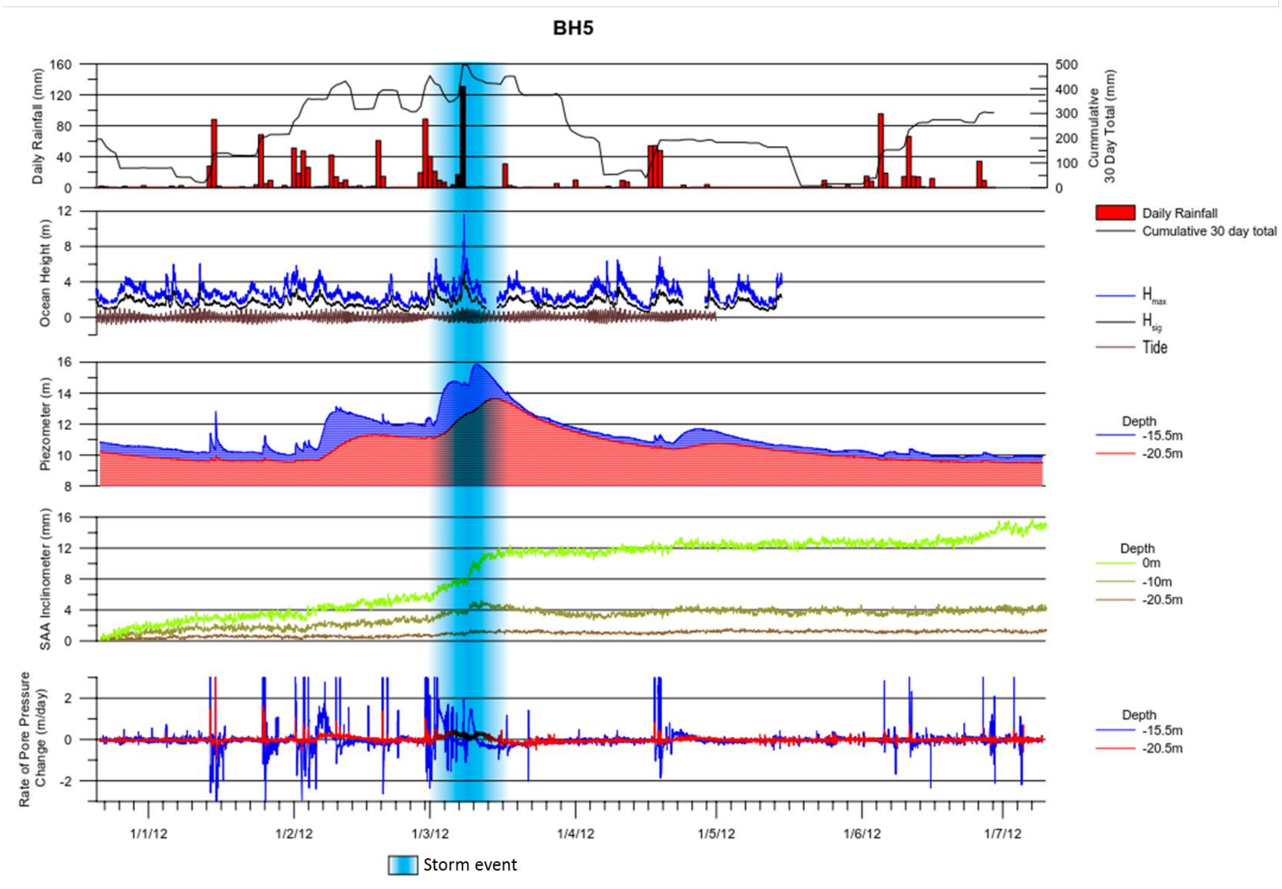

Figure 5: Temporal relationship between monitoring data at Borehole BH5.

Piezometric pressure peaks in P5/-20.5 generally lag approximately three days behind those in P5/-15.5 and the curve is substantially smoother. It is considered that this was due to recharge from further behind the slope or slow percolation through the slope.

Pressure at Piezometer P5/-20.5 did not appear to be a good predictor of the initiation of slope movement, but triggers were set for this piezometer as it was considered that there could be some contribution to ground movements with elevated pressures, and piezometric pressures here did track the piezometer above albeit with a lag. This lag could result in elevated piezometric pressures at depth some time after a rainfall event.

Piezometric rates of rise to establish trigger levels were modified from the values in Figure 5 to account for the following:

- Implementation of surface water control measures. It can be seen in Figure 5 above that after February 2012 the sharp spikes immediately following rainfall events have been eliminated.
- Installation of groundwater depressurisation wells. The impact of these wells can be seen in the muted response to rainfall in the piezometers after June 2012.

3.4.4. OFFSHORE WAVE HEIGHT AND EMBANKMENT TOE EROSION

Toe erosion due to wave action has been a contributor to slope instability and movement at the site. Hence, the potential for toe erosion due to wave action was monitored. However, no direct correlation between significant wave height and extent of erosion was established. Slope movements and erosion did occur during the storm event and towards the end of May 2012, in association with high tides and a significant wave height of approximately 5.5 m. However, no movements associated with slope instability or erosion were recorded during a sea storm with 8 m significant wave height. It is likely that the direction of the swell, which is not monitored at the buoy, is an important factor in toe erosion at this site.

4 TRIGGER ACTION RESPONSE PLAN

4.1 GENERAL

The purpose of the Trigger Action Response Plan (TARP) was to provide planned, graduated responses to changing slope conditions at the site in order to mitigate the landslip risks to road users/pedestrians. Clear and concise responses would be required at each trigger level to ensure that the correct actions were taken by responders, particularly those not directly involved in the formulation of the TARP. The points listed below were considered when developing trigger levels:

- The trigger levels need to reflect the urgency or criticality of the actions required.
- Realistic responses must be possible on receiving automated SMS / email alerts.
- These responses must be possible within a reasonable time frame.
- SMS / email trigger alerts must not occur so frequently that they are regarded as nuisance occurrences or important events are dismissed due to repeated false alarms.

Monitoring frequencies need to be consistent with trigger response times, particularly for electronic monitoring devices. If time is of the essence, the monitoring frequency must be reduced. The time taken to implement responses needs to be considered as well. The logistics of implementing a response may require frequent sampling to ensure the response can be implemented timeously.

4.2 TRIGGER STATUS

Trigger status or trigger levels for this project were set to provide escalating responses when each trigger level was exceeded. The general concept of the status for each trigger level is provided in Table 3.

Table 3: General concept of the status for each trigger

Trigger Status	Condition	Response
Green	Normal operation.	Monitor in accordance with monitoring plan.
Blue	Trends in data indicate that amber trigger may be exceeded.	Consider whether pre-emptive action is required, and implement where necessary.
Amber	Data indicates that unless action is taken within a predetermined time frame slope movement may take place.	Decide on and implement actions.
Red	Slope is potentially unstable.	Implement predetermined actions. Actions are prescriptive.
Grey	Loss of communication with web portal. Loss of data. Malfunctioning of instrumentation.	Response is targeted at reinstating instrumentation and action related to the slope is not necessarily required.

4.3 DEVELOPMENT OF TRIGGERS

4.3.1. GENERAL

A set of trigger levels were developed based on back analysis of the stability of the slope and review of monitoring data over the timeline of previous slope instability events. This review involved the assessment of the relationships between monitoring data such as piezometric pressure, inclinometer movement, antecedent rainfall (both daily and cumulative) and wave height as well changes in the slip area from remote camera images.

The slope movements were dependent on piezometric pressures, which in turn were dependent on rainfall intensity and duration, and antecedent piezometric pressures. Hence, triggers for rainfall could be varied depending on the piezometric pressures prevailing in the slope at the time. However, introducing interdependent triggers would have brought a level of complexity to the TARP, which could impact on reliability and risk.

Also, the slope was located in a very dynamic environment. Changes such as significant toe erosion leading to a reduction in slope stability, could change the piezometric level at which the slope is mobilised, rendering existing trigger values unconservative and requiring adjustment. Alternatively, pre-emptive action such as installing dewatering pumps to stabilise the slope could allow some relaxation in trigger levels. Hence, it was very likely that the trigger levels would be changed during the course of the monitoring program. Therefore, independent trigger levels were adopted for each monitoring device in this TARP.

4.3.2. SETTING PIEZOMETRIC TRIGGER LEVELS

Development of trigger levels for the piezometer at P5/-15.5 at borehole BH5 are discussed in some detail as an example of how trigger levels were set. Trigger levels for other instruments were developed in a similar fashion.

The critical piezometric level, the level which initiated approximately 5 mm of surface movement in the inclinometer during the storm event was set at 14.7 m BGL. From this level, there was a further rise in piezometric pressure of approximately 1.4 m. This represents one day's rate of pressure rise. The higher pressures did not lead to failures within the area of the footpath and Norfolk pines, although visual inspection indicated that significant further movement of the main slip had occurred. Survey carried out after the storm indicated ground movements of up to 1.5 m occurred in the main slip since a previous survey a week before the storm event. Some of these movements may have been associated with heavy rain and erosion due to high wave heights after the storm. This behaviour indicated that failure occurred as relatively shallow circular slip failure on the slope face, followed by ongoing creep movement identified by inclinometer data as shear at the rock/colluvium interface and stress relief in the upper section of the main slip. In view of this embankment behaviour behind the main slip when critical piezometric pressures were exceeded, it was considered that these pressures had been selected conservatively.

With the above in mind, red triggers were set to allow six hours of response time to execute management measures, such as closing the footpath, before the critical pressure was reached.

An amber trigger was set to allow one day for action before a red trigger was reached. Action could for instance include activating pumps in depressurisation wells which had recently been installed or assessing what further pre-emptive action could be taken to avert a red trigger.

A blue trigger was set to allow one day before an amber trigger was reached. Whilst no specified action was required on reaching a blue trigger, it allowed review of the data to identify trends which could result in further instability and consider possible pre-emptive actions.

To set the above trigger levels, the following process was followed:

Determine the piezometric pressure at which slope movement is initiated (critical pressure)

- The six hour rise is subtracted from the critical pressure to obtain the red trigger.
- The 24 hour rise is subtracted from the red trigger to obtain the amber trigger.
- The 24 hour rise is subtracted from the amber trigger to obtain the blue trigger.

The resulting raw trigger levels for a sample piezometer are presented in Table 4.

Table 4: Raw piezometric trigger levels.

Borehole	Piezometer	Critical Piezo level (mAHD)	Maximum Rate of rise (m/day)	Six hour rise	Red Trigger	24 hour rise	Amber Trigger	Blue Trigger
BH5	P5/-15.5	14.7	1.4	0.35	14..4	1.4	13.0	11.6

Table 5 below presents the piezometric trigger levels and response plan for the P5/-15.5 in BH5. Similar response plans were developed for the other instrumentation on site.

Table 5: Piezometric Pressure Trigger Levels at P5/-15.5.

Peizometric Triggers (mRL)			Maximum Response time from trigger notification
Grey	Loss of communication with web portal Loss of data Malfunctioning of instrumentation	Identify reason for malfunction and reinstate device	12 hours during green and blue status 2 hours during amber and red status
Green	N/A	Monitor in accordance with monitoring plan	N/A
Blue	10.8	Review data (AECOM) and meet with Roads and Maritime to agree if action is required. Review to include following data: Rain gauge information. BoM rainfall predictions. Wave height data. Tide data. camera images Evidence of erosion at the toe of the main slip. Piezometric pressures in BH5, BH6, and BH1. Rate of rise of piezometric pressures in BH5, BH6, and BH1.	24 hours
Amber	12.75	Inspect footpath, slope and road between bridge abutment at southern end of GD4 and BH4 (if safe to do so). Activate depressurization pumps and check discharge to confirm operation Review data (AECOM) and meet with Roads and Maritime to agree further action. Review to include following data: Rain gauge information. BoM rainfall predictions. Wave height data. Tide data. Camera images Site observations. Evidence of erosion at the toe of the main slip. Piezometric pressures in BH5, BH6, and BH1. Rate of rise of piezometric pressures in BH5, BH6, and BH1. Decide whether to run depressurisation pumps.	12 hours 12 hours 24 hours
Red	14.2	Inspect footpath, slope and road between bridge abutment at southern end of GD4 and BH4 (if safe to do so) Close footpath Activate pumps and check discharge to confirm operation Review data (AECOM) and meet with Roads and Maritime to agree further action. Review to include following data: Rain gauge information. BoM rainfall predictions. Wave height data. Tide data. Camera images. Evidence of erosion at the toe of the main slip. Piezometric pressures in BH5, BH6, and BH1. Rate of rise of piezometric pressures in BH5, BH6, and BH1.	6 hours 6 hours 6 hours 24 hours

4.4 PIEZOMETRIC TRIGGER REVIEW AGAINST HISTORICAL DATA

The piezometric data was assessed against the historical monitoring data to check the trigger levels under which the slope would have been managed had the TARP been in place from the installation of the piezometers. The assessment indicated that trigger levels would have been exceeded over the time frames listed below and the slope managed in accordance with those trigger levels in that time:

- Blue – approximately three and a half months
- Amber – approximately 30 days
- Red – approximately 10 days

Had these triggers been in place, the footpath would have been closed as a precautionary measure for a period of 10 days during the storm event. This would have been a reasonable management measure as there would have been very little pedestrian traffic in this area during that time. The need to manage the slope under an amber trigger for a period of 30 days would be undesirable, but it is likely that the depressurisation pumps would have been activated to avert this need. Hence, the risk to road users could be reasonably managed with the trigger levels developed for the site.

During the subsequent operation of the monitoring plan and TARP, the footpath was closed for one weekend due to high piezometric pressures and recorded inclinometer movements.

5 CONCLUSION

The road embankment at Lawrence Hargrave Drive, immediately north of Seacliff Bridge, was subject to instability due to elevated piezometric pressures and erosion due to wave action at the toe. Interim management measures which would not impact on the tourist value of this iconic route were required to ensure safety of the public whilst long term stabilisation solutions could be developed.

A comprehensive monitoring program was developed to provide an interim management tool to ensure public safety. The monitoring plan included detailed real-time monitoring of several parameters, which contributed to instability issues. These included rainfall measurements, piezometric pressures, slope movement, wave climate and toe erosion due to wave action. From the data collected as part of the monitoring program, it was possible to establish relationships between these parameters, leading to development of a comprehensive TARP, which allowed efficient interim management of the slope whilst long-term solutions were developed.

Management of the slope under the TARP has been a success and a permanent solution consisting of a rock revetment at the base of the slope, surface drainage measures and slope protection have now been installed. These measures are illustrated in Figure 6 below.

Figure 6: Slope showing rock revetment at the base of the slope, surface drainage measures and slope protection.

6 ACKNOWLEDGEMENTS

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7 REFERENCES

Roads and Maritime Services NSW. *Guide to Slope Risk Analysis (Version 4)*, 2014.