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Case study of remediation of a landslide south of Sydney, NSW, Australia

Remédiation d'un site d'éboulements au Sud de Sydney, NGS, Australie.

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ABSTRACT: This paper presents a case study of the remediation of a historical landslide beneath a portion of coastal road south of Sydney, NSW. The site is located about 50 kilometres south of Sydney CBD, Australia. The landslide has been known since the 1950s and has been monitored in various forms since then. A maintenance approach was adopted whereby the road area over the landslide was regularly reinstated. Recently, reinstatement was necessary every 3 years, and it was decided in late 2017 that a more sustainable, long term solution to the remediation was required. The background and design approach for the remediation is presented, including the options for remediation that were explored, and the results of three-dimensional numerical seepage modelling that was used to assess and optimise the trench drain spacing. Field performance after several years of monitoring post remediation is also generally discussed. Thus far it has been found that the remedial measures have controlled movement as predicted, and the client is no longer having to regularly repair the road.

RÉSUMÉ : Cette étude de cas présentes un résumé des travaux de remédiations d'un site d'éboulements situé au pieds d'une portion d'une des routes côtières au Sud de la région métropolitaine de Sydney, Nouvelle Gales du Sud, Australie. Le site est situé à environ 50 kilomètres au sud de Sydney. Le site à initialement été reconnu en 1950 et a depuis été mis sous surveillance géotechnique. Typiquement, une approche de maintenance progressive a mené a des réparations périodiques du site. Plus récemment, les réparations ont été complétées sur une base périodique de moins de 3 ans, ce qui a mené à la décision qu'une solution durable, à long terme était nécessaire. Cette étude présente le contexte général du site, l'approche de conception des travaux de rectifications, la description des diverse options proposées, le résultat d'analyses par élément finis, modélisation des infiltrations des eaux souterraines nécessaire a l'optimisation d'un système de drainage du sol. Les résultats du programme de surveillance qui avait débuté depuis la fin des travaux de remédiations sont inclus dans l'étude. A date de la rédaction de cette étude, les mouvements associés à l'éboulement ont été grandement réduit par les travaux de remédiation ce qui a permis d'éliminer le besoin d'entretien périodique sur le site.

KEYWORDS: Landslide, remediation, numerical analysis, drains, monitoring.

1 INTRODUCTION

The location of the landslide is about 50 km south of Sydney CBD and 20 km north of Wollongong, NSW, Australia. At this location, the portion of coastal road is an important road link for both residents and tourists. The alternate route if the road is blocked would require a 45minute detour totaling 40 kilometres.

The landslide discussed in this paper has been known since the 1950s to local authorities and has been repaired and monitored in various forms since then. The landslide will be referred to as The Slide in this paper.

To maintain road serviceability, the client had adopted a maintenance approach, whereby the road area over the landslide was regularly reinstated. Recently, reinstatement was necessary every 3 years, and it was decided in late 2017 that a more sustainable, long term, lower maintenance solution to the remediation was required.

2 SITE HISTORY

2.1 *History of Landsliding*

The Slide is located beneath and downslope from Lawrence Hargrave Drive, on the footslopes of the Illawarra coastal escarpment. The Illawarra escarpment is known for the presence of many ancient landslides and colluvial soils. Whilst many landslides remain dormant, some landslides in the area remain active. Landslides in the area are generally extremely to very slow moving, creep style landslides, and are rarely fast moving. The landslides in this area are sensitive to movement from increased pore water pressures (PWPs) and other factors including human induced disturbance and poor-quality filling practices.

The Slide has been known since at least the 1950s as road maintenance crews regularly needed to maintain and repair/relevel the road (Walker 1985). The Slide has been monitored in various forms since then.

Wells or inclinometers that were previously installed between 1984 to 1989 sheared within 6 months of installation. Instruments installed onsite between 1989 and 2017 were often lost or damaged quickly after installation through ground movement/shearing. Inclinometers and standpipe piezometers installed in early 2017 showed ground movement of between 7mm and 15mm within a few months of installation at depths of about 5.5m and 3m, respectively.

Up until 2018, cracking of the road pavement occurred relatively quickly after repair work and reinstatement of the road was necessary every 3 years. On the landslide headscarp area eastwards of the road tension crack, asphalt to about 800mm thick was measured, whereas to the west of the tension crack only relatively thin asphalt/pavement conditions have been encountered.

A review of Wollongong City Councils Spyglass Aerial Imagery also showed that a house structure was present in the 1948-51 aerial photo, however based on review of other photos (Flentje, circa 1970s/early 1980s) the house has not been present onsite for some decades. There is anecdotal evidence that the house was removed due to landslide issues and presence of poor-quality fill (possibly coarse coal washery reject fill).

2.2 *General Geology and Topography*

A schematic geological section of the Illawarra Escarpment geology through The Slide is shown in Figure 1, (Walker 1985). The upper parts of the site along Lawrence Hargrave Drive are about 78m AHD (above sea level) with the lower parts being about 60m AHD (above sea level).

The upper parts of the site are underlain by Wombarra Claystone and an unnamed relatively thin Siltstone unit, which in turn overlie Coal Cliff Sandstone. The lower parts of the site are bounded by a wave cut cliffline about 60m high, exposing Coal Cliff Sandstone, the Bulli Coal Seam and the upper parts of the Illawarra Coal Measures.

The ground model formed for the site explains the site conditions in more detail, below.

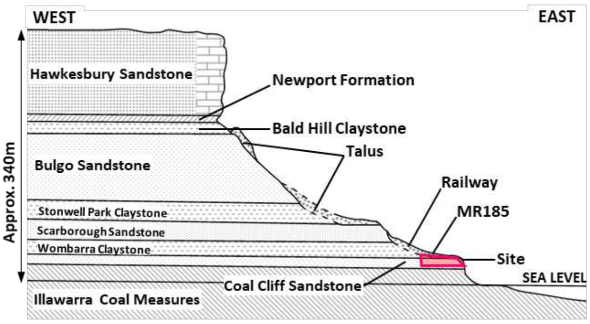

Figure 1. Schematic Geological Section for the Scarborough-Clifton area with position of Lawrence Hargrave Drive and Site (after Walker 1985).

2.3 *Site Boundaries and Land Ownership*

A further complicating factor for this site is that not all land encompassed by The Slide is publicly owned. The privately owned lots within The Slide are defined below in Figure 2 as "Inaccessible Lot". Two privately owned, but presently undeveloped, 'house lots' are located downslope (below the road). It was important that any construction work for a remedial solution did not extend within these privately owned lots. This had implications for lateral spacing and assessment of effectiveness of dewatering measures.

3 GROUND MODELLING

3.1 *Site Mapping, Aerial Photos and Surface Expression of Landslide*

To update the desktop review, site mapping was carried out in early 2018 to update the relevant site features to The Slide. During mapping work, the extent of the main headscarp tension crack was marked along with a number of tension cracks, water seepages and other features downslope. Following markup of site features on aerial photography a drone was used to capture and better view site conditions at the base of the slope (inaccessible on foot) and give perspective to the site and The Slide, particularly in the near cliff and cliff areas downslope.

Figure 2. Site Features, with assessed extent of The Slide shown.

3.2 *Assessed Subsurface Conditions*

The subsurface units were assessed from historical and recent site investigation and mapping data, which are summarised in Table 1 and shown in Figure 4.

Table 1. Summary of assessed subsurface units

Unit	Description
Fill*	Sands, Gravels, Clays and Coalwash. Consistency of loose, soft to very stiff. Moist to wet. Thickness:0.45m to 5.20m
Colluvium	Underlying Coalwash at the toe of The Slide, Gravelly Clay. Estimated firm to stiff.
Residual Clay	Clay medium to high plasticity. Consistency stiff to very stiff, M>W_p.
Scarborough Sandstone	Observed in the rock cut above Lawrence Hargrave Drive. Fine to medium grained sandstone, brown, with shell and pebbly marker bands.
Wombarra Claystone	Claystone with Siltstone layers, brown in colour. Estimated very low to low strength, extremely to moderately weathered.
Siltstone	Encountered at the base of the Wombarra Claystone in borehole P3 (2017). Logged as Siltstone pale grey, estimated low strength.
Coal Cliff Sandstone	Sandstone, pale grey, medium strength. Slightly weathered.
Bulli Coal Seam	Not encountered in boreholes drilled onsite, but observed in cliff face exposure downhill. Top of unit estimated at 10m below cliff hinge point.

Note: Asphalt has not been included in this table but is present to thicknesses of up to 800mm on downslope side of the road tension crack.

The interpreted subsurface profile is discussed in Section 3.4.

3.3 Groundwater

Groundwater monitoring was carried out between 1984 to 2017. The monitoring data collected between 1984 to 1989 only involved 'point in time' monitoring (rather than continuous and regular water level logging).

The most recent 2017 groundwater data was captured using data loggers recording water levels at better than daily intervals at borehole P1, P2 and P3 in conjunction with inclinometer monitoring at borehole BH1i and BH2i at several points in time. Both water level and rainfall data were plotted vs time and the results of monitoring for Piezometer P2 are shown in Figure 3.

It was noted that within the first two weeks of March 2017 the site experienced a significant rainfall event and the total monthly rainfall in March 2017 of 369mm was the highest ever recorded monthly rainfall for Bellambi AWS. Upon review of the available data, trends indicated that The Slide reactivates when peak groundwater levels reached in BH1i and BH2i, which are listed below.

- 0.6m bgl in borehole 1P (P1),
- 2.50m bgl in borehole 2P (P2), and
- 1.60m bgl in borehole 3P (P3).

The inclinometer data indicated significant movements at The Slide around mid-March 2017, which was likely due to the higher groundwater levels in this period.

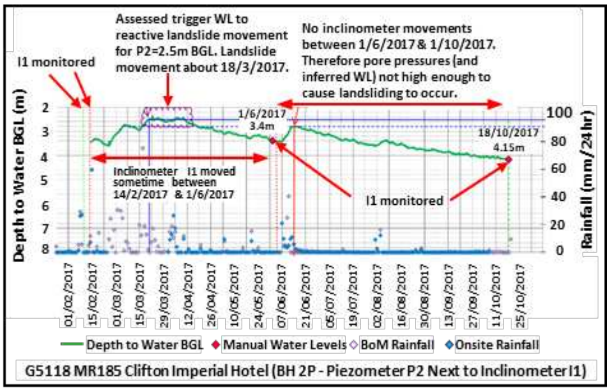

Figure 3. Groundwater monitoring and inferred groundwater trigger level at Piezometer P2 to reactivate The Slide movement.

3.4 *Interpreted Subsurface Profile*

Two scaled cross sections nominated as Section 2 and Section 3 were developed using survey data, site mapping and available subsurface data including the depth of slide plane assessed from shears recorded within inclinometers and assessed groundwater levels during reactivation in mid March 2017. The results for Section 2 are shown in Figure 4, along with the inferred slide plane.

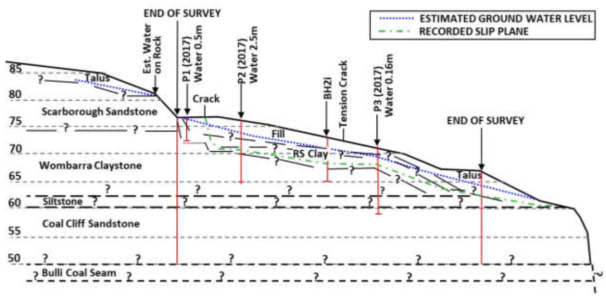

Figure 4. Interpreted subsurface Conditions at Section 2 (through the Head of The Slide) showing an inferred slide plane and interpreted groundwater levels at time of reactivation of The Slide in mid-March 2017.

The Slide is a relatively shallow landslide varying in thickness from 3m to 6m. The base of The Slide is generally sliding close to the residual clay/weathered rock interface over most of the landslide. Towards the toe, the slide plane also passed through remnant colluvium materials.

Contributing to activation of The Slide is the presence of fill in the vicinity of the backscarp and midpoint. The fill is assessed from review to be placed during construction of the road and the platform for the former house downhill. The presence of several metres of additional fill loading of about 50kPa to 60kPa coupled with high PWPs from rainfall events likely cause The Slide to regularly reactivate and creep downhill.

3.5 *Initial Geotechnical Design Parameters*

Some geotechnical parameters were presented in a report by Walker (1985) for The Slide based on laboratory test results, limited investigation data, and were developed for peak shear strength failure analysis.

Building on the Walker (1985) report, a decision was made to adopt new site-specific parameters for sub-surface units at The Slide using investigation data from 1989, 2016 and 2017.

3.6 *Back Analysis and Slide Plane Geotechnical Design Parameters*

To back analyse the properties along the zone of the landslide, back analysis was carried out using a defined failure plane. An inferred failure unit approximately 200mm thick could be modelled relatively accurately as several known points could be included in the model including the location of tension/surface cracking and deeper inclinometer movements down the landslide body, recorded between 1988 and 2017. The toe of the landslide also daylights at the crest of a 60m high cliff where drone photography showed a relatively thin mantle of soil overlying near vertical cliff line areas with outcropping weathered rock.

Residual soil strength parameters were adopted for the inferred failure unit at The Slide. The residual soil strength parameters were assessed using back-analysis in the software package SLOPE/W and the laboratory test results in BH9 from Walker (1985). From the desktop review and site inspection, adopted material parameters for the stability and seepage analyses are summarised in Table 2.

Table 2. Summary of material properties for subsurface units

Unit	Unit Weight (kN/m^3)	Angle of Friction (Degrees)	Cohesion (kPa)
Fill*	17	30	0
Colluvium	18	30	0
Residual Clay	17	27	5
Slide Plane*	16	15^o to 18^o	0
Scarborough Sandstone	23	32	100
Wombarra Claystone	22	30	10
Siltstone	23	30	30
Coalcliff Sandstone	24	40	600

* angle of friction for Sections 2 and 3 was 15^o and 18^o, respectively

To achieve a Factor of Safety (FoS) of ~1.0 for the assessed groundwater level at The Slide, a relatively low friction angle of 15 degrees (Section 2) to 18 degrees (Section 3) was required.

3.7 *Initial Dewatering Assessment and Effect on Stability*

Initial two-dimensional analysis of the effect of dewatering on stability was carried out assuming two-dimensional dewatering was effective to a depth of both 3m and 4m below ground level (bgl).

Table 3. FoS results from sensitivity analysis from dewatering of The Slide

Section	Groundwater Level (bgl)	FoS
2	3m	1.23
	4m	1.38
3	3m	1.40
	4m	1.40

3.8 *Review of Treatment Options*

At this stage, various treatment options were assessed for The Slide and the cost/benefit, safety and constructability of each option were considered and are outlined in the SMEC concept options report by Beeston and Morrison (2018). Table 4 below presents the options (listed in order from cheapest to most expensive). The landslide is a very slow-moving slide and rapid failure involving sudden movement and loss of life to road users was considered extremely unlikely. Options 2 to 6 were considered either too costly and/or difficult to construct. The client was interested in using a relatively low-cost treatment solution first, monitor performance and explore other options if necessary in future years. The client indicated that Option 1 treatment was preferred.

Table 4. Summary of landslide treatment options for The Slide

Option	Description	FoS
1	Construct surface water interception measures and trench drains to dewater to 3m bgl.	1.23
2	Construct surface water interception measures and an underbored drainage system that dewaters the underlying rock and drains outwards from the cliffline. The exit point of drain likely to be located about 10m down from the top of the near vertical cliff area.	> 1.5*
3	Construct a combination of Option 1 and Option 2.	>1.5
4	Excavate a block of soil mass around the head of the failure, and backfill with bulk coarse slag fill and perhaps reinforce with strengthening geotextile.	>1.5
5	Drill vertical dewatering drains uphill of the tension crack/slide area down to drain into the underlying coal seam, located vertically about 20m to 30m below the site.	>1.5
6	Construct a piled road 'bridge' solution to span the head of the landslide and prevent road movement due to landslide movement.	>1.5

* FoS > 1.5: considered possible for the given option.

The client's technical direction requires a long term FoS of at least 1.25 and short term FoS of at least 1.20 for the category class of road on the existing slope. As this site is categorised with a consequence of C4 (client's rating), the assessed FoS of 1.23 (very close to 1.25) for the 3m deep dewatered slope was assessed as a tolerable risk by the client.

It was decided to target control of the landslide using trench drains and surface water control measures so that groundwater would not rise higher than 3m bgl in future.

It was also important to note that the primary task of the dewatering was to provide lower risk of landslide reactivation for road users. Two as yet undeveloped private lots are also located downslope spaced alternately with publicly owned lots. The brief for work excluded a requirement to improve the private lots, in terms of land sliding and dewatering.

3.9 *Seepage Analysis*

A seepage analysis to assess the zone of influence by installing trench drains in The Slide was carried out using SEEP/W 2D and PLAXIS 3D software packages.

The analysis considered varying drain spacings, trench depths, soil permeability and ground water head pressures. The adopted permeability for the soil and rock units was selected based on the proposed values in the available literature.

An initial seepage analysis was carried out for the ground section presented in Figure 4 to calibrate the permeability of the soil units by matching groundwater level in the model with the observed groundwater level in The Slide. The trench drains were modelled based on the proposed arrangement. The adopted permeability (k) for the soil and rock units were assumed for modelling purposes to be homogenous and isotropic (equal in the x, y and z axis). The adopted permeabilities of the units for the seepage analysis are presented in Table 5.

Key outcomes of seepage analyses are summarised in the sections below.

Table 5. Summary of adopted material permeability (k)

Unit	K (m/sec)
Fill	0.5×10^{-8}
Colluvium	0.5×10^{-8}
Residual Clay	0.5×10^{-9}
Drain Material	0.5×10^{-2}
Scarborough Sandstone	1×10^{-9}
Wombarra Claystone	1×10^{-10}
Siltstone	0.5×10^{-9}
Coalcliff Sandstone	1×10^{-9}

A 2D seepage analysis using SEEP/W was performed 'across the slope' to assess effectiveness of trench drain spacing. Extracts from the performed analysis from SEEP/W are presented in Figure 5. We note that the 5m trench spacing results are not presented as this showed significant dewatering effect. This was to be expected given the trenches themselves were 5m wide. Figure 5 therefore shows only the effect of the drains to the groundwater table (blue dashed line) at 10m and 15m 'trench wall to trench wall' intervals.

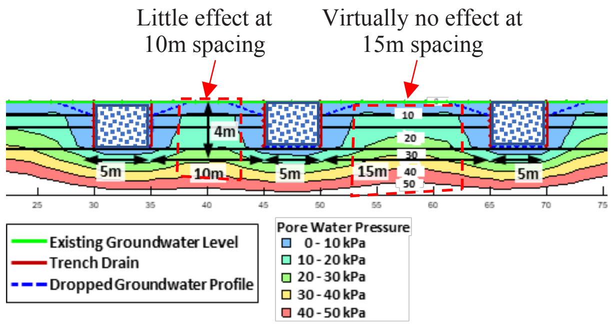

Figure 5: Impact of trench drain spacing on dewatering (output from SEEP/W 2D).

Analysis in SEEP/W indicated that in slope cross-section 3m deep trench drains with spacing of approximately 5m laterally 'wall to wall' can dewater the soils between the trench drains to levels close to the toe/floor of the drain. For trench drains spaced at 10m to 15m intervals horizontally, the dewatering effect would only partially dewater the slope. The results of this 2D analysis indicated that potentially the drains would not dewater across the private lots and their effectiveness will reduce significantly. Stopping the drainage analysis here would have resulted in the clients selected option not being adopted due to lack of dewatering The Slide.

To advance the seepage analysis and provide a higher order assessment for the client, the ground model was input into a three-dimensional model to explore whether the trench drains could be potentially effective for dewatering the inaccessible private lots further down slope.

To assess the potential draw-down by the 3m deep trench drains for dewatering downslope, a three-dimensional (3D) numerical analysis was carried out in PLAXIS 3D with the installation of trench drains at 5m, 10m and 15m spacing. With review of the construction staging and initial outputs from the PLAXIS 3D model, it was decided to adjust the trench drain setout from slot trenches to a terraced trench for construction purposes. The adjustment to the trench shape, allowed for 4 trenches to be installed across the accessible lots. The layout and results of the seepage analysis for terraced trenches at nominal 10m and 20m spacing (centre to centre) of the PLAXIS 3D analysis are presented in Figure 6 and Figure 7, respectively.

In addition to the 2D analysis, the results of the PLAXIS 3D model indicate that as the trench drains continue down slope, the effect of dewatering between the 10m and up to 20m spaced trench drains gradually draws the groundwater level to the toe of the drains. The effect of the dewatering occurs progressively further downslope for a wider drain spacing, and that a spacing of ~10m is assessed to have relatively rapid dewatering capacity.

Assessments with trench drains at 4m depths were carried out to understand the effect of deepening the trenches. The rate of dewatering the groundwater profile utilizing the deeper trench drains assessed to have some beneficial effect to the 10m and 20m spaced drains in the PLAXIS 3D assessment. However, the full effectiveness of the 4m deep trench drains does not occur until downslope similar to the 3m deep trench drains.

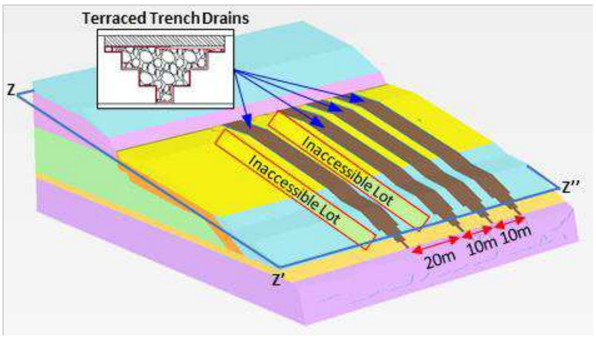

Figure 6: Arrangement of proposed terraced trench drain in PLAXIS 3D.

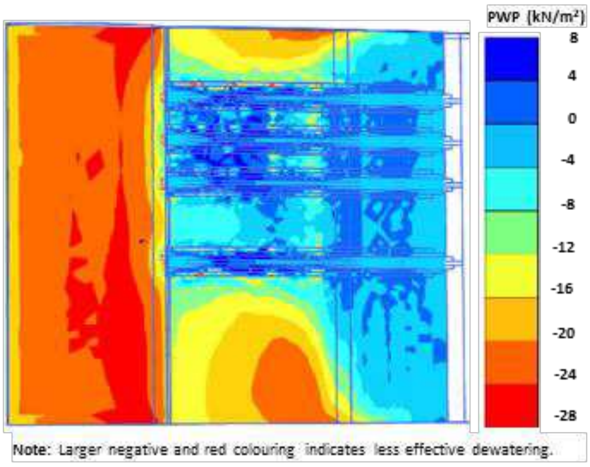

Figure 7: Impact of terraced trench drain spacing on PWP in PLAXIS 3D, in a Z-Z'-Z'' inclined plane. The darker shades of blue indicate lower to zero pressure indicating dewatering, where the orange to reds indicate excess PWP.

As the main objective of the dewatering exercise is to reduce the likelihood of instability of the road rather than stability of the private lots, it was considered appropriate to construct the shallower 3m deep trench drains. The client indicated that to excavate 4m deep trench drains that additional resources and construction staging challenges could hinder progress (trench drain construction cannot impact on the private lots, meaning trench earthworks including collapse and/or benching must not be allowed to extend into private lots), and that the 3m deep drain appear to provide sufficient dewatering at the road.

4 ADOPTED SOLUTION

4.1 *Trench Drain Remedial Measures*

4.1.1 *Subsurface Drainage*

Based on the outcomes of the seepage analysis, the trench drain layout in Figure 6 was proposed for installation at The Slide for subsurface drainage measures comprising:

- Install a network of spaced trench drains in available lots up through the headscarp of the slope under the road to intercept water before entering and saturating The Slide.
- Excavate the trench drains to depth of 3.0m whilst terracing the excavation to allow placement of geofabric, aggregate and pipe. Narrow single width trench slots were not recommended due to likely consistency of fill/clay/ colluvium material onsite, and potential implication from groundwater that can result in instability of the trench walls and make it impractical to install more than one trench drain per lot.
- Trench drains to be terraced, geofabric lined, and filled with clean free draining durable angular aggregate. Three 100mm diameter subsoil pipes to be installed towards the base of the trench for redundancy/maintenance purposes. Trench drains to have a minimum fall of 2% to 3%.
- As a guide deepen the trench drains to a minimum 2.5m depth or top of weathered rock where possible.
- Cap trench drains with at least 1m of low permeability clayey material (compacted to 95% std), dressed with 0.3m of topsoil (compacted to 90% std) seeded to encourage vegetation to regenerate.
- A minimum 0.5m to 1m thick drainage rock should be kept in the toe outlet area of the drains.
- Inspection pits which include flushing points should be installed at the head of the trench drains to allow maintenance of the pipes by the client.

4.1.2 *Surface Drainage*

In addition to subsurface drainage, surface drainage measures were recommended to control surface water away from The Slide and aid in preventing infiltration through existing open tension cracks down the slope. The surface drainage measures included construction of:

- A rolled asphalt kerb on the down slope side of the road to act as a barrier and to direct water to two batter chutes.
- Two new batter chutes down the slope below the toe of The Slide with appropriate dissipation structures to mitigate potential erosion.
- A surface drain adjacent to the road Northbound fog-line directing surface water to the lead stormwater drainage pit for the dual cell culvert.

4.1.3 *Other Remedial Measures*

From the sensitivity analysis, it is evident that fluctuations in the groundwater table have potential to reduce the FoS. Based on analysis, the FoS is likely to be as low as 1.23 (23% improvement). Monitoring is recommended in the form of inclinometers and standpipe piezometers with data loggers. Standpipes positioned equidistance between trench drains and installation of a standpipe within the center of a trench drain to assess the groundwater fluctuation within the drains was recommended.

5 PERFORMANCE

Since installation of Option 1 in 2018, ongoing inspections have been completed by the client at The Slide. It is understood that monitoring has involved visual inspection of the pavement, no new inclinometers or piezometers have been installed down slope in the public lots (no factual data from the existing instrumentation has been provided).

Information from the client in mid-2021 has indicated that since the installation of Option 1 in 2018, that no further deformation or cracking has appeared in the road pavement at the crest of The Slide. This duration of 3 years without any evidence of deformation in the pavement is showing less maintenance requirements than the regular historical 3-yearly asphalt patch required previously. Without evidence of cracking in the pavement, the performance of the trench drains is considered to likely be mitigating excessive PWPs that would have typically reactivated the landslide.

To comparatively assess the level of rainfall experienced at The Slide, a review against the AEP was completed. Daily rainfall levels indicated that ~100mm recorded onsite in March was less rainfall then a 63.5% AEP event (ie. roughly ~100mm/24 hours would occur more than once per year). Whilst daily rainfall (short intensity) is a contributor, longer term rainfall and catchment wetting is considered important. We have therefore compared significant monthly rainfall events in the area between late 2018 to mid-2021 for comparison against events prior to installation of the trench drains for the nearby BOM Bellambi AWS, site number: 068228. In the period between 2014 to 2017 that preceded trench drain installation work, five significant monthly events were recorded:

- August 2014: rainfall 279mm, >95th percentile 272.9mm,
- March 2014: rainfall 290mm, >95th percentile 285.7mm,
- August 2015: rainfall 245mm, >90th percentile 213.9mm,
- June 2016: rainfall 436mm, highest ever recorded June rainfall, and
- March 2017: rainfall 368.7mm, the highest ever recorded March rainfall

After trench drain installation, significant rainfall events which are considered likely to have reactivated the landslide against the historical data are assessed to have been experienced onsite in:

- March 2019: rainfall 187mm, median rainfall 107mm, 90th percentile 235.7mm,
- February 2020: rainfall 399mm, the highest ever recorded in February, and
- July 2020: rainfall 207mm, >95th percentile 207.1mm

These large events between 2018 and 2021, and *in particular the February 2020 event*, historically would have normally required the asphalt to be repaired. As no cracking of the pavement has thus far occurred, the system is assessed to have been 'adequately tested' through similar rainfall. The initial performance is showing improvement (which is generally as predicted).

The visual observations that no cracking or deformation has occurred in the asphalt pavement at the crest indicates positive performance so far. It is noted that this has not been based on monitoring from inclinometers as the client was satisfied with performance of the road surface, no inclinometer monitoring has been carried out.

Due to the location and proximity of The Slide, coastal erosion of cliff or toe areas may in future still mobilise The Slide. Our recommendation to the client has been to continue visual monitoring, and that inclinometer/pore pressure monitoring in The Slide should be carried out to assess the performance.

6 CONCLUSIONS

With the development of a subsurface ground model and a back analysis for The Slide, numerous concept design solutions with varying risk profiles were developed and communicated to the client (Table 4). After review and further consultation, the client selected Option 1 as their preferred option.

After fine tuning the ground model and understanding construction limitations for the client, the likely draw down effects of trench drains were modeled in a 2D plane with software package SEEP/W. The 2D analysis provided an indicator of dewatering from varying trench drain spacing and likely drawdown profile between aggregate trenches at the site. Effective in a single plane, the 2D SEEP/W modelling didn't provide an opportunity to demonstrate the 3D radial effects of dewatering with trench drains down slope, and it was unclear if the 10m to 20m spacing (over the private lots) would have any effect on lowing the PWPs in the mid portion of The Slide.

Adopting the developed ground model and results from SEEP/W into finite element software PLAXIS 3D, the effects of dewatering downslope could be assessed and visually communicated to the client for Option 1 in a 3D space. The 3D numerical modeling indicated that the 5m, 10m and 20m laterally spaced drains offer dewatering between aggregate trenches and provide an assessment of downslope dewatering capability across the private lots at The Slide. The 3D modelling provided outputs for comparing varying trench profiles and depths for assessment of dewatering potential to optimize the solution.

The implementation of Option 1 to date is understood to have slowed the rate of movement of The Slide. No further cracking or displacement has been observed in the 3 year period since installation of Option 1. Ongoing visual monitoring is being carried out by the client to review performance. At this stage the performance has been achieving the desired outcome and the client has decided no new inclinometer or piezometer monitoring is yet required.

The implementation of Option 1 at a relatively low construction and maintenance cost has mitigated the requirement for 3 yearly remediation work for The Slide for the client.

7 ACKNOWLEDGEMENTS

We are grateful to our client for their discussion and comments of the historical behavior of The Slide at the site south of Sydney, NSW. The authors also acknowledge the support of their respective organisations for the preparation of this paper.

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