DESIGN OF A LANDSLIDE REMEDIATION SCHEME FOR A 42 LOT SUBDIVISION AT WINSTON HILLS, SYDNEY

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ABSTRACT

A 42 lot subdivision proposed for a known landslip area at Buckleys Road Winston Hills required the geotechnical design of earthworks to remediate the site. An initial remediation proposal using an anchored retaining wall was abandoned in favour of an earthworks solution involving the removal of the failed material, installation of a drainage blanket and replacement of engineered fill. In order to reduce earthworks costs a alternative earthworks solution was also developed where only the sloping portions of the site were treated and the flatter central area was untreated. Detailed earthworks design included treatments at the site boundary where the landslip extends outside the site boundary, a foundation inspection and testing regime and earth bunds to deflect potential future failures from the neighbouring property.

1 INTRODUCTION

Several large lots of undeveloped land are located to the south of Buckleys Road in the suburb of Winston Hills, in the North West of Sydney. This area contains a large landslip area that is clearly identifiable on aerial photographs and extends approximately 260 m across the slope and 130 m down the slope. From air photographs it is clear that the landslip failed prior to 1943, when debris flows were present over the southern portion of the site (Fig. 1). Air photographs from 1928 also show lobe like features similar to those present today and it is likely that the bulk of the slide pre-dates air photography.

The landslip extends across two lots and this paper is details the work undertaken to develop a remediation scheme to allow the subdivision of the western lot into 42 residential lots.

1.1 TOPOGRAPHY, GEOLOGY AND PROPOSED SUBDIVISION

The site comprises a trapezoidal shape with frontages to Buckleys Road along the northern site boundary and Rebecca Place along portion of the southern site boundary. The ground slopes towards Rebecca Place falling from RL 81 m at Buckleys road to RL 50 m near Rebecca Place.

Reference to the Sydney 1:100,000 geological series sheet indicates the site is located across the contact between the Bringelly Shale and the underlying Ashfield Shale, with the Minchinbury Sandstone forming the boundary between the two units.

No sandstone has been encountered in the investigations to date and it is possible that the Minchinbury Sandstone is very thin or absent in this location. The regional dip of the shales is approximately 1.3° towards the south west, parallel to the direction of sliding.

The proposed subdivision incorporates 42 residential lots with access provided by a subdivision road from Buckleys Road (Figure 1). Site stormwater and sewerage are directed via an easement through a public reserve to the existing services in Rebecca Place.

1.2 LANDSLIDE MORPHOLOGY

The landslip area can be broken down into three areas:

- Upper backscarp area
- Central gently sloping area
- Debris lobe area

The upper backscarp area is characterised by steep slopes (maximum average of 22° from horizontal) and rapidly increasing depth to bedrock. Test pits located at the crest of the backscarp generally encountered bedrock at depths of less than 1 m, overlain by residual clay soils. Test pits at the base of the backscarp encountered up to 7.0 m of colluvial material.

Figure 1: Air Photo dated 1943 with Cadastre (NSW Department of Lands).

The central area of the landslip comprises the greater area of the slide. The ground has a maximum slope of 8° below horizontal falling towards the south west. The ground surface in this area tends to be hummocky and air photo interpretation by Douglas Partners identified several lineaments (possibly internal slump planes) in the upper portion of the central area (Figure 2). The depth of colluvial material in the central area ranges from 7 m to less than 5 m.

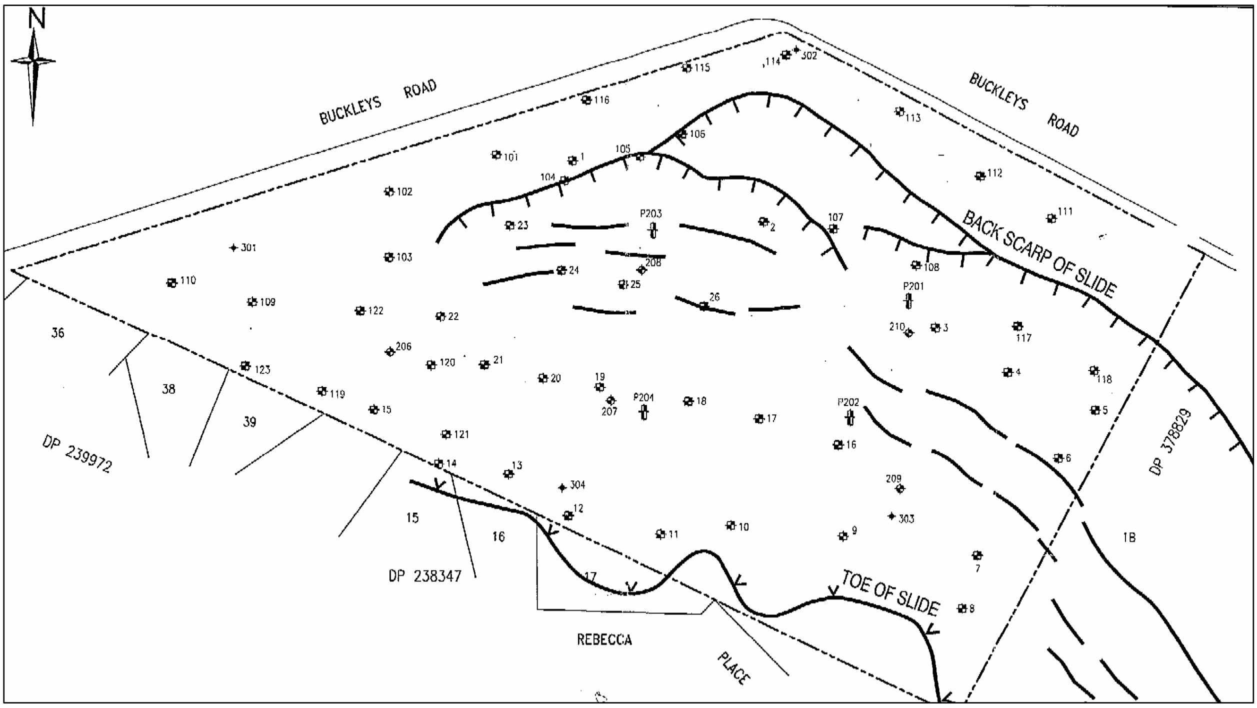

Figure 2: Air photo Interpretation (Douglas Partners Pty Ltd).

The toe of the slide is characterised by distinctive lobes of colluvial material that form an irregular and steeply sloping topographical feature. Below the lobes shale bedrock is with 1m of surface and outcrops in a cut adjacent to Rebecca Place.

1.3 PREVIOUS WORK

The site has been the subject of several geotechnical investigations by Golders Associates in 1985-1986, Douglas Partners in 1995 and 2002 and by SMEC in 2005. The work generally involved test pit excavation, air photograph interpretation, limited drilling and piezometer installation and stability analyses.

An application to subdivide the site was refused by Parramatta Council apparently on environmental and geotechnical grounds. An appeal to the Land and Environment Court was upheld and the conditions of consent in the judgement require a high level of geotechnical certification in the development works.

In the Land and Environment Court case the following remedial measures we put forward to show that the site could be developed:

- Installation of subsurface drainage lines at a minimum of 3 m depth below ground level
- the construction of several piled and anchored retaining walls to support the subdivision roads
- piling of all structures to sound bedrock
- restrictions on unlined detention basins to control water infiltration into the site soils

While these measures will improve the stability of the site, the subdivision designer was concerned that they did not address the stability of the steeper portion of the site and there were concerns regarding the cost and constructability of the retaining structure.

The author was subsequently tasked to develop an alternative remediation scheme.

2 SCHEME 1

In order to address the stability of the site a remedial scheme was developed involving the removal and replacement of the landslip material. This scheme was termed Scheme 1 and comprised:

- excavation of the of all disturbed material
- Installation of a drainage blanket and drainage system
- replacement of the excavated material as an engineered fill
- monitoring and maintenance of the installed drainage measures.

The earthworks were designed to be carried out in eight strips running down the axis of the site. This was to limit the length of temporary slope below Buckleys Road.

While Scheme 1 appears to be a conventional excavate and replace process, there were several features that needed further geotechnical input.

2.1 FOUNDATION DESIGN

After the completion of the remedial works the consent conditions require that proposed lots be classified in accordance with AS2870-Residentail Slabs and Footings. As conditions vary across the site differing foundations systems are appropriate in different areas.

2.1.1 Upper Portion of Site

Along the Buckleys Road frontage the ground is sloping and rock is generally at a shallow depth. Thus the lots uphill of the subdivision road are anticipated to be Class P (problem site).

In this area the proposed residences may be supported by shallow foundations bearing on undisturbed bedrock. While the foundations may be piled, there is potential for lateral loading of the piles due to creep movements of the soil. Therefore it is recommended that the structures in this area be founded on blade walls.

The blade walls may extend into the area occupied by the drainage blanket and in these cases great care will be required during construction to ensure the drainage blanket is not disturbed. Where the drainage blanket is suspected to be in the footprint of the blade wall the following precautions shall be followed:

- Careful excavation to expose the geotextile on the top of the drainage blanket. It is important that the geotextile is not snagged and pulled.
- Cutting the geotextile with a knife.
- Removal of the drainage material

As the excavation will need to be entered, support such as trench boxes will be required.

2.1.2 Lower and Central Portion of Site

Within the lower portion of the site up to 7 m of fill may be placed, although the thickness of fill will decrease towards the south. Deep fill generally results in Class P, however, Clause 2.4.6 of AS2879 allows reclassification based on the natural site classification. Laboratory testing of the shrink swell potential of the soils has not been carried out, but it is anticipated that the soils are moderately to highly reactive. Thus lots in the lower portion of the site are anticipated to be Class H.

The fill is also anticipated to undergo creep settlement in the long term. Creep settlement is thought to be due to a gradual reduction in void space in fill materials under self-weight and constant moisture content. This settlement is time dependent, reducing with time. Past experience indicates that if the fill is well compacted the long term (post construction) compression within the fill (internal settlement) can be controlled to within 0.2% of the height of the fill.

Depending on the depth of the fill, creep settlements of up to 14 mm per log cycle are anticipated.

After construction it is intended to produce a work as executed plan showing the depth of fill under each lot together with the potential creep settlements.

2.2 TREATMENT AT THE EASTERN BOUNDARY

As the landslip extended over the eastern boundary it was not possible to extend the remedial works over the entire slide. The following treatment was proposed for the areas adjacent to the eastern boundary:

- The edge of the excavation shall be battered at not steeper than 1H:1V, with the crest of the batter at the site boundary.
- A section of batter not more than 10m long is the excavated and inspected by an experienced geotechnical engineer or engineering geologist to confirm the stability of the batter. Longer batters may be adopted after this first assessment is carried out.
- Slot drains extending from the excavated area to the site boundary and excavated normal to the boundary are required to provide drainage of this portion of the site. These slot drains shall have a minimum width of 500mm.
- After backfilling, and completion of the remediation area, a trench is to be excavated along the entire eastern site boundary and extending to undisturbed bedrock. Shoring boxes or excavation support will be required where personnel will enter the trench. It should be noted that a double layer of plastic is required on the trench wall (see below) and this may impact on any trench support measures.
- A subsoil drain is to be installed in the base of the boundary trench and surrounded by drainage blanket material. Clear hydraulic connections shall also be made to each of the slot drains at 10m centres.
- Prior to backfilling a double layer of builders plastic (or similar) shall be placed on the site boundary side of the trench to provide a slip zone in the event of future soil movements.

A sketch through this treatment is shown in Figure 3.

This treatment will leave a wedge of disturbed material along the eastern boundary. However, the remaining colluvial material will be well clear of any buildings and is anticipated to have negligible effect on the stability of the site.

However, the variable subsurface conditions along the boundary will mean that the founding of structures on shallow footings in this area may result in unacceptable differential settlements. The installation of piles close to the boundary is considered too risky as they may damage the drainage blanket. Therefore a “No build Zone” was imposed along the eastern boundary and incorporated into the planning controls for the proposed development.

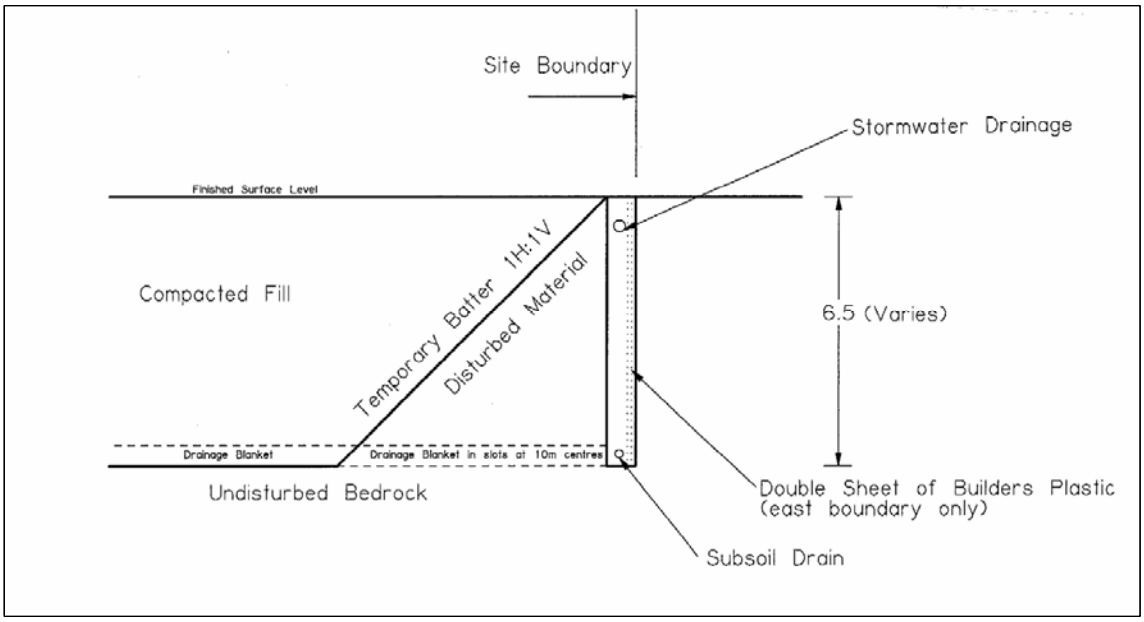

Figure 3: Treatment at Site Boundaries.

2.3 TREATMENT OF SOUTHERN SITE BOUNDARY

Disturbed material extends also extends over the southern site boundary. The following measures were devised to treat the southern boundary:

- The excavation to remove disturbed material shall be battered at 1H:1V with the crest of the batter extending to the site boundary.
- The trunk drainage line for the drainage blanket over the lower portion of the site is to be located close to the toe of the batter.
- Slot drains extending from the excavated area to the site boundary and excavated normal to the boundary are required to provide drainage of this portion of the site. These slot drains shall have a minimum width of 500 mm.

This treatment shall leave a wedge of disturbed material along the southern boundary. This material will be well clear of any buildings and is anticipated to have negligible effect on the stability of the site.

However the variable subsurface conditions along the boundary will mean that the founding of structures on shallow footings may result in unacceptable differential settlements. The installation of piles close to the boundary is considered too risky as they may damage the drainage blanket. Therefore a “No build Zone” is required along the eastern boundary and shall be incorporated into the planning controls for the proposed development.

2.4 POTENTIAL DEBRIS FROM THE SITE TO THE EAST

The sloping area on the site to the east does not have an acceptable factor of safety and thus it is reasonable to assume that a failure may occur on this slope. Should such a failure occur debris will flow down slope and potentially cross the eastern site boundary. To prevent the debris impacting the proposed development a bund is required along the site boundary to deflect such debris flows.

Lateral loads on structures near the boundary have been calculated based on the following extract from *Debris Flow Control Structures for Forest Engineering.*

> *“The momentum equation, which incorporates the entire estimated peak surge of the debris flow travelling at a uniform velocity, is used to calculate dynamic thrust. The momentum equation is:*
>
> $$F = \rho \times A \times v^2 \times \sin\beta$$
>
> *Where* F *= dynamic thrust*
> ρ *= density of debris*
> A *= cross sectional area of flow*
> v *= velocity of flow*
> β *= the angle between flow direction and face of structure”*

For a debris flow 2 m deep travelling at 1 m/sec, with a density of 15 kN/m^3 and impacting the structure at an angle of 5 degrees:

Dynamic thrust (parallel to ground surface) (F) = 2.6 kN per m run of wall

The lateral loadings may be calculated using the following equation:

Un-factored lateral load = hydrostatic load + dynamic thrust.

2.5 DRAINAGE BLANKET DESIGN

The drainage blanket is to comprise the following:

- Sound, durable, free draining material with a maximum grainsize of 40 mm and less than 5% passing the 2.36 mm sieve.
- Be a minimum 300 mm thick.
- Be encapsulated by geotextile fabric to prevent fines from entering into the gravel and clogging the blanket.

The drainage blanket shall be drained by a pipe network placed in a herringbone or similar configuration consistent with the proposed excavation plan. The pipes shall comprise slotted and un-slotted (as required) uPVC pipes to AS1477. The pipes shall drain by gravity via drainage pits to a pit located close to the southern boundary of the site. From this pit it appears feasible to discharge into Council's stormwater system.

Flushing points will be required at the upstream end of each pipe and access for flushing and cleaning will be required at each of the intermediate pits.

The number, size and capacity of the pipes shall be determined in accordance with the requirements of the DRAINS user manual. An infiltration rate of 6mm/hr over the extent of the drainage blanket may be assumed for the pipe design.

A minimum of three drainage pipes (in addition to the pipe along the eastern boundary) are required to connect the upper and lower drainage blankets. As some clogging is inevitable, it would be prudent to design the pipe drainage such that flow from the entire portion of drainage blanket can be accommodated in any one pipe.

2.6 SUPERVISION AND MONITORING

The long term performance of the drainage blanket is critical in ensuring an acceptable factor of safety against failure.

A system of piezometers is required to allow assessment of the performance of the drainage blanket away from the piped drainage. These piezometers shall comprise a uPVC standpipe with a 1000 mm long slotted section extending upwards from the base of the drainage blanket. The top of the piezometer shall be protected by an iron gatic cover. Monitoring shall be by a data logger capable of collecting up to six months data. Nine such piezometers are proposed, three each for the upper, middle, and lower portions of the site. The piezometers in the central portion of the site shall be located between the trench drains connecting the upper and lower drainage blankets.

It is recommended that the operation of the drains be assessed on an annual basis with flushing of the pipes as required. The piezometers are to be initially monitored on a monthly basis, reducing to annual readings after 6 months. Additional readings shall also be taken when the total rainfall in a calendar month exceeds 300 mm as measured at the nearest Bureau of Metrology rain gauge.

The recording of water levels above the top of the drainage blanket would indicate the drainage blanket is ineffective and geotechnical assistance should be sought ASAP.

2.7 SCHEME 1 SUMMARY

Scheme 1 was reviewed and verified by Council's appointed geotechnical engineer. However, during pricing of the works it became evident that the cost of the earthworks was high and the client questioned if the proposed scope of works was necessary.

3 SCHEME 2

Investigations carried out for Scheme 1, showed that the landslide debris generally comprised hard, low plasticity clay with some areas having a high proportion of weathered shale clasts. This material was difficult to dig with a 20 t excavator, and several previous test pits excavated by backhoe appear to have refused on shale clast rich clay, which was incorrectly logged as undisturbed shale.

As the central third of the site is relatively flat the main areas of instability are confined to the steeply sloping backscarp area below Buckleys Road and the sloping area at the toe of the debris. To limit the volume of earthworks required for the remediation an alternative proposal (Scheme 2) was devised based on undertaking remediation only in the steeper parts of the site.

Scheme 2 comprises the excavation and re-compaction of landslip material only from the steeply sloping upper and lower portions of the site. The central area of the site would remain un-remediated and buttressed by the block of remediated soil immediately down slope.

Material from the two sloping areas would be excavated, a drainage blanket installed and the soil replaced and compacted to form an engineered fill. Trenches shall be excavated in this area to allow drainage between the two remediated areas.

3.1 STABILITY ANALYSES

As Scheme 2 involves retaining a portion of the landslip, the stability of the proposed remedial measures needed to be fully assessed. Six cross sections (Sections A to F) developed at intervals of 40 m to 60 m across the site. Section A is located to the west of the failure, while sections B, C, D, E and F are located within the landslip. The analyses were undertaken using SLOPE/W, a program included in GeoStudio 2004, developed by GeoSlope.

Conservative groundwater conditions are used with total saturation of any landslide debris. In the remediated areas it is assumed that the drainage blanket is in operation and the groundwater level is located at the top of undisturbed bedrock (weathered shale).

The soil parameters used in the analyses are provided in Table 1:

Table 1: Soil Strength Parameters.

Material Unit	Weight (kN/m^3)	Cohesion (c') (kPa)	Friction Angle (ϕ')(°)
Residual Clay	20	0	28
Disturbed Clay	20	0	16
Disturbed Clay – possible slip zone	20	0	12
Undisturbed Weathered Shale	22	20	26
Conditioned and compacted Clay	20	0	28

A typical section through the site is shown in Figure 4:

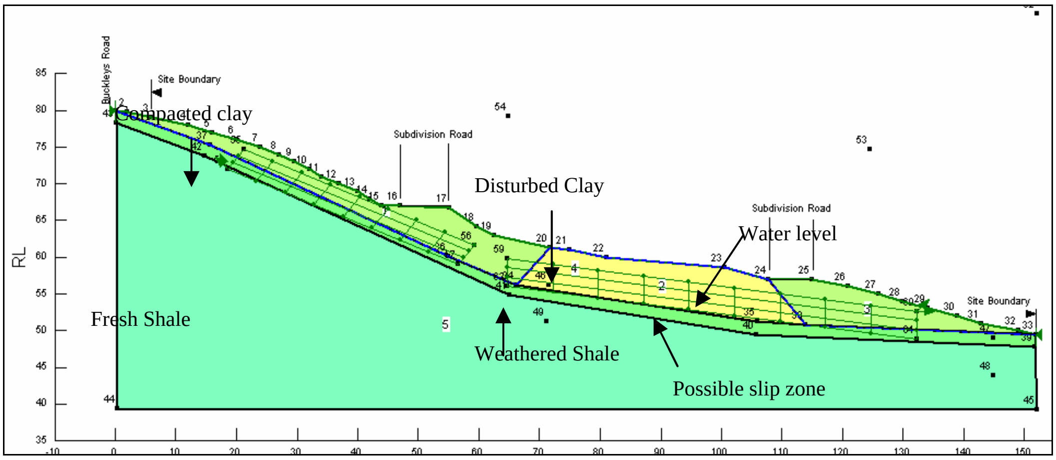

Figure 4: Typical Section Scheme 2.

Three stability analyses were undertaken for each section. A circular slip analysis of the upper and lower slopes, and a non circular (block sliding) analysis of the central portion of the site. The results of the analysis are provided in Table 2.

Table 2: Slope Stability Analysis Results.

Section	Circular Slip – Upper part of Slope	Circular Slip – Lower part of Slope	Non Circular Slip
Section A	1.2	Not undertaken	2.0
Section B	1.7	1.4	1.6
Section C	1.5	1.5	1.5
Section D	0.6	1.4	0.6
Section E	1.4	1.4	1.5
Section F	1.6	1.3	1.5

It is notable that several results give a factor of safety of less than the industry accepted value of 1.5. The two low factor of safety results for Section D are located outside the site boundaries and indicate that the site is susceptible to failures in the neighbouring lot (as discussed in Section 2.4).

Factors of safety of 1.4 and 1.3 were considered adequate as they were based on worst possible groundwater conditions that could not be realistically achieved given the presence of the drainage blankets.

3.2 PASSIVE WEDGE ANALYSIS

It was also considered possible that the passive resistance of the mass of the soil forming the central portion of the site may be destabilised by lateral loading from the fill placed as part of the remediation works on the sloping backscarp area. To check this theory the capacity of the passive wedge was checked.

Two potential failure mechanisms were checked for the passive wedge. Mechanism 1 compared the active pressure from the up slope reconditioned fill to the passive pressures from the landslide debris. Full saturation of the landslip debris was assumed and the triangular pressure distribution was determined. The passive resistance calculated using this method provided a factor of safety of 1.7 against failure.

Mechanism 2 comprised the lateral sliding of a fully saturated block of landslide debris due to loading from the active pressures developed in the reconditioned fill. To simplify the problem it was assumed that there was no lateral support from the material placed at the toe of the slide and thus the entire lateral load would need to be resisted by friction developed along the base of a block of landslide debris ‘x’ m wide.

The sliding resistance is dependent on the groundwater pressures along the slide plane and rather than assume a water pressure, the program SEEP/W was used to determine the likely pore pressures developed under the landslide debris. It was assumed that the majority of the groundwater would be flowing horizontally from fractures in bedrock. This was modelled by assigning the nodes at the side and bottom slide boundaries water pressures corresponding to the full hydrostatic pressure. Nodes in the drainage blankets were assigned zero water pressure to model drainage, while nodes in the landslide material were allowed to reach equilibrium under steady state conditions. This analysis was considered conservative as a rainfall event would be anticipated to produce a transient period of higher water pressures and it would be unlikely that equilibrium could be reached.

The output from the SEEP/W modelling indicated that the influence of the drainage blankets extends under the landslide debris and negligible pore pressures build up under the landslide debris.

Reference to the site plans shows that over 20 m width of material is present downslope from the toe of the reconditioned fill. The lateral capacity of a 20 m wide block of landslide debris provides a factor of safety against sliding of 1.6.

The sensitivity of Mechanism 2 was checked by assuming 10 kPa and 15 kPa excess pore pressures on the slide plane. For these conditions the factor of safety falls to 1.3 and 1.2 respectively.

It is considered that the soil under the central portion of the site will has sufficient capacity to resist lateral loads imposed by the fill placed as part of the remediation of the backscarp area.

3.3 CONSTRAINTS ON DEVELOPMENT

In addition to the constraints mentioned above, the following restrictions also apply to the proposed development:

- Houses or other structures over the drainage blanket shall not be piled.
- Houses or other structures over uncompacted landslide debris shall be piled.

- After completion of the remedial works fill is not to be imported to the site. Finished surface levels are not to be altered by more than 600 mm without assessment and approval by a geotechnical consultant.
- Excavations deeper than 1.2 m or retaining walls higher than 0.6 m shall not be constructed without assessment and approval by a geotechnical consultant.
- Leaking services are to be repaired as soon as they are detected.

These constraints are to be incorporated into the planning documents for the subdivision so that potential purchasers will be aware of the geotechnical constraints prior to purchase.

4 CONCLUSIONS

Remediation of a landslide developed on shale bedrock at Winston Hills initially involved a conventional excavate and replace solution with the installation of a drainage blanket. Due to the morphology of this slide it is feasible to reduce the volume of earthworks and remediate the steeply sloping upper and lower portions of the site, leaving the central area buttressed.

As the landslide extends across the site boundaries, solutions were also required to allow treatment up to the site boundaries without disturbing the adjacent sites.

5 REFERENCE

Van Dine D.F. (1996) Debris Flow Control Structures for Forest Engineering. Working Paper 22/1996. British Columbia Ministry of Forests.