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Design of a large scale rockfall protection bund for coastal transport corridor recovery following the 2016 Kaikōura earthquake

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ABSTRACT

On November 14th 2016, a Magnitude 7.8 (Mw) earthquake occurred in the South Island of New Zealand, centred approximately 60km south-west of the coastal town of Kaikōura. The complex sequence of ruptures resulted in significant damage to a number of major transport corridors, including the Inland Kaikōura Road, State Highway 1 (SH1) and the adjacent Main North railway Line (MNL), cutting off all land routes into Kaikōura.

The mountains of the Seaward Kaikōura Range rise abruptly from the adjacent coastline, with a limited coastal platform forming the transport corridor. Significant damage was incurred from large scale rockfall and rock avalanches triggered by the earthquake. Following the debris clearing and temporary protection works, a number of large scale geohazards remained above the transport corridor. These hazards, ranging from discrete rockfall to larger scale mass movements, required mitigation to provide adequate long term safety and resilience for the transport networks below. This paper details the selection, modelling, and design processes for a large rockfall protection bund, including the use of field information and analysis of the observed rockfall characteristics in order to define the required dimensions and energy capacity for the structure. Key aspects such as the planning of subsequent inspection, clearing and maintenance works are covered, as well as the application of Serviceability and Ultimate Limit States in the design approach.

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1 INTRODUCTION

Following initial assessment of the damaged areas of the coastline, discrete site areas were created, split into identified primary and secondary landslide areas. One of 9 primary work areas along the coast to the North of Kaikōura township is Site NRP7 (P7), located approximately 1 kilometre north of Ohau Point on the Kaikōura coast. A series of shallow soil and rock slides were triggered during the earthquake that left a large exposed rock face and debris slope above the road and rail carriageway. Landslide debris from upper catchment areas formed extensive debris fans which extended over the road and rail corridors at the toe. The potential for further release of material from the source areas was considered to pose an unacceptably high risk to road and rail users, such that engineered works were required to reduce this risk to acceptable levels.

Figure 1: Slip P7 after the Kaikōura earthquake

The crest of the uppermost source area is 250m above the road corridor below, and the length of corridor directly affected by the rockfall hazard is approximately 650 m. The overall area consists of discontinuous bluffs (particularly on the upper third of site) which are highly fractured/dilated and provide an ongoing source of rockfall. The highest degree of fracturing is along the ridge crest, and this forms the most active rockfall source area. More competent bluffs along the midslope area form a series of wedge shaped debris chutes, with large debris fans below forming a secondary source for rockfall. These debris cones are formed from the shallow slides above, and consist of fine rich debris with large boulders which have come to rest on the slope. This debris cone forms a rolling surface for falling boulders from the bluffs above, which allows smaller debris to come to rest on the slope face or at the toe of the debris cone.

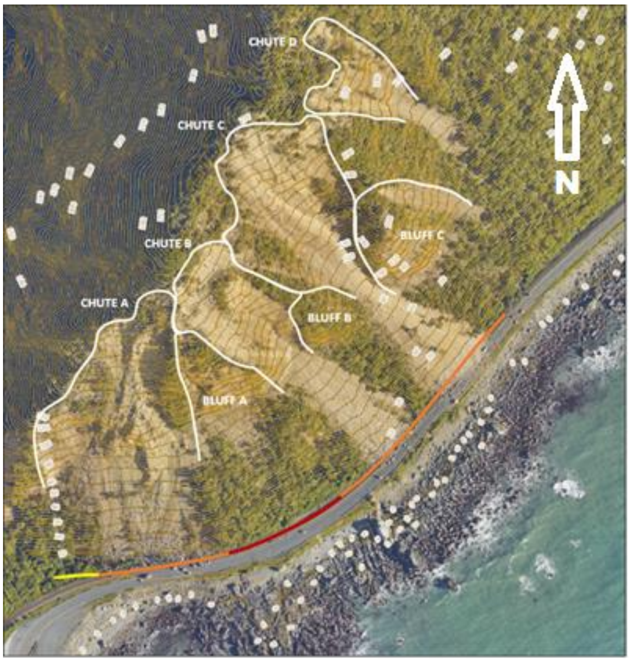

Figure 2: P7 zones and temporary protection. White = zone outlines, yellow = earth bund, orange = double stacked containers, red = single stacked containers

The slip was extensively treated by helicopter sluicing post-earthquake. This focussed on removal of unstable areas of rock from the upper source areas, and encouraging debris which came to rest on the lower debris fans to move to the slope toe where it could be removed by excavators. After sluicing was completed, teams of abseilers worked to target individual large boulders in the upper source areas and to clear out debris chutes, with a range of hand tools, airbags and non-explosive charges.

1.1 Hazard description

The head areas of site P7 are underlain by highly fractured, slightly to moderately weathered greywacke rock. The current slope consists of fractured rock near the ridgeline, which extend down into 4 main chutes. These chutes are defined by 3 mid-slope bluffs which act to channel debris from the sources above. At chutes A-C the area downslope of these bluffs consists of accumulated debris from the shallow failure above. This debris generally has a high fines content, with numerous large intact boulders entrained in the debris which have been observed to act as a secondary rockfall source during rainfall events. The earthquake induced slide from chute D was approximately half the thickness of the rest of the chutes, and resulted in a significantly lower volume of material being transported downslope. As such, the debris in this chute does not extend as far down the slope as in other areas, with the lower talus/colluvium slope remaining heavily vegetated and relatively few instances of rockfall affecting the road at this area.

Discrete rockfall was assessed to be the dominant slope hazard to road and rail users in the selection and design of the protective structure. No evidence has been found to support a deep seated failure mechanism, and therefore the likelihood of global slope failure is considered very low. The vast majority of rockfall events are observed to travel down the main chutes, with few examples of rocks reaching the road in the areas below the bluffs. Minor mass movements have been observed on the lower debris slopes following heavy rain, however these are typically very shallow, and the extended runout of the debris cone allows for the deposition of this mobilised material as it travels downslope, resulting in relatively low volumes of material mobilised, typically at low speeds.

1.2 Failure characteristics

Block sizes at the headscarp and on the upper slope cover a wide range, with a large proportion of small fractured rocks, ranging up to isolated boulders >10m diameter. Initial site summary reports indicated the many of the larger falling rocks tend to break up as they travel downslope. In the upper source area, falling rocks generated very large bounce heights due to the uneven nature of the ground surface and the steep profile of the face. The fine rich nature of the debris cone below acts to significantly dampen the energy of these bouncing rocks along the lower portion of the slope, and extended runout of this debris cone meant that by the time rocks reached the toe of the slope their behaviour was dominated by the influence of this lower debris material, rather than that of the upslope source area.

From extensive observations during scaling and sluicing, analysis of the post-earthquake debris, and measurement and recording of debris pathways resulting from the blasting of large boulders, a number of key characteristics of the rockfall hazard at this site could be developed. These include specific observations related to the size and travel mode of the falling rocks:

- Smaller rocks were more sensitive to the apparent roughness of the debris slope, resulting in a wide range of bounce heights and angles when travelling down the debris slope, as randomly orientated boulders within the debris act as launch points for these smaller rocks
- Typically, the smaller rocks have significantly greater bounce heights and velocity when compared to larger boulders, but significantly lower kinetic energy. Impacts from smaller rocks (<0.5m diameter) are observed on the temporary container wall (discussed in temporary protection section, below), with impact heights ranging from between 0 to 5m above ground level at the slope toe.
- Larger rocks which developed significant bounce heights in the upper slope sections tended to bed into the comparatively fine debris material on first impact. Many of these large rocks came to rest on the debris slope, posing a secondary risk of reactivation following rainfall or seismic events.
- Large rocks which bounced on the debris slope lost a significant amount of energy with each bounce, due to the bedding in of these boulders into the finer debris material, and the effect of drag as the boulder

creates an impact scar in the debris. These boulders were observed to generally come to rest in the catch ditch at the slope toe, or impact the temporary protective barrier at low bounce heights

- Large rocks which slide on the debris surface and do not develop a rolling motion tended to generate low velocities and arrest in the catch ditch before impacting the protective barrier
- Large rocks which were able to develop rolling behaviour (particularly those with a tabular shape which were able to roll on edge) tended to develop very high downslope and rotational velocities. Large rocks with very high rotational velocities tended to roll through the catch ditch to the temporary protection with high impact energies.
- Photos from directly after the earthquake show this contrast in behaviour between rocks of different sizes. The vast majority of the landslide/rockfall debris was stopped by the raised rail formation upslope of the the road, with the rocks which passed the rail and reached the road being dominated by large individual boulders.

1.3 Temporary Protective Measures

After initial sluicing and scaling had been completed, the nature of the rockfall hazard meant temporary protection measures were required to ensure the safety of the site staff travelling past the slope toe. Across the majority of site, this involved constructing a double stacked wall of shipping containers, ballasted with slip debris.Through the period these shipping containers were in place, they were subjected to numerous impacts, particularly as a result of the large rock blasts upslope which mobilised numerous large boulders (Figure 3). Through analysis of video footage of these blasts, and the impact locations on the containers, it was possible to develop a greater understanding of the behaviour of oversized falling rocks at this site.

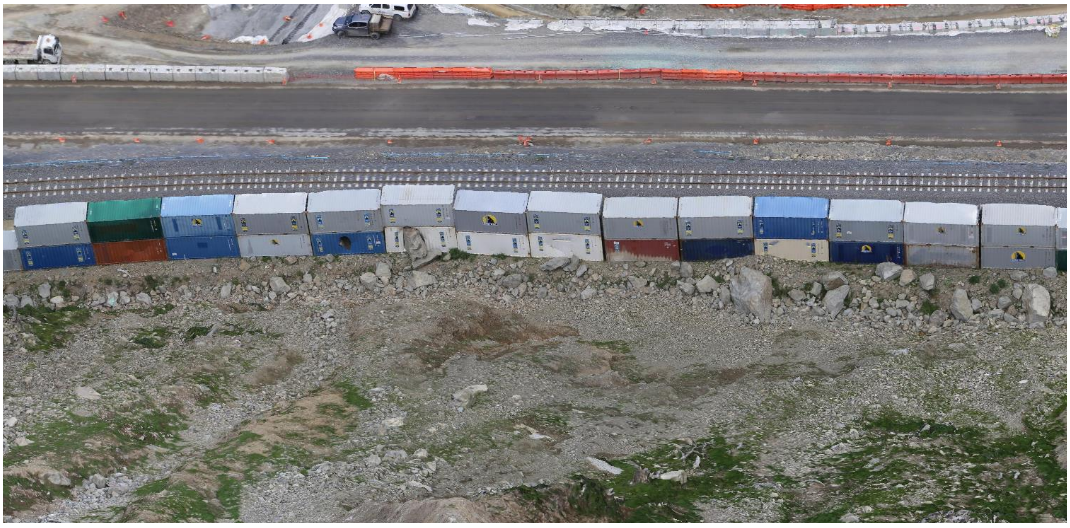

Figure 3: Boulder impacts at container wall following rock blasting

2 REMEDIAL OPTIONS ASSESSMENT

For remedial options assessment a number of key features had to be considered relating both to the effectiveness of the solution, and its applicability to the specific nature of this site. Key considerations included: Construction within a small footprint, maximising available space for a catch ditch on the upslope side; maintenance costs over the lifespan of the structure; design life in a coastal environment; material cost and procurement times; effectiveness at dealing with large/over design-level impacts; ease of repair in the event of an impact; avoiding on-slope work (i.e. on-slope anchors) which may be affected by surface movement of the debris material; ease of access for heavy clearing equipment following future rockfall/mass movements; and low deflection to prevent encroachment into the rail corridor.

2.1 High Energy Rockfall Barrier (Fence)

Rockfall fences could be used at the site, but were not favoured for a number of reasons: Design impact energies required very high energy fence structures; no footprint advantage due to the need for a deflection zone on the downslope side and extensive upslope anchors; higher routine maintenance requirements due to the requirement to reinstate fence energy capacity by regular removal of trapped rock; more prone to corrosion (exposed steel components); visual intrusion (steel structure versus natural rockfill/revegetated bunds); less resilience than GTM bunds to over 'design-level' impacts, multiple impacts or other mass failure modes; significantly higher long term maintenance costs than GTM bunds, due to the requirement to replace sacrificial components such as braking rings, which deform to absorb the energy from large impacts.

2.2 Un-Reinforced Soil Bunds

Un-reinforced earth bunds were discounted due to the additional cross-sectional footprint required to provide the design height. Unreinforced bunds would need 1V to 1.5H slopes for long term stability, and therefore could not be fitted into the available space across the majority of the slope. There is also the difficulty in quantifying the energy capacity of an unreinforced bund, and the risk of rollover with the shallow face angle.

2.3 Upslope Catch Benches

Extensive upslope catch benches were not favoured due to difficulty of access for construction and for operational maintenance/clearance. The upslope bench constructed during initial debris clearing was left in place across Chutes B and C, and is considered as part of the remedial design for these areas, but modelling has shown this strategy is not effective enough to operate as a standalone solution to the rockfall risk posed at this site.

2.4 Reinforced Soil Bunds (Green Terramesh-GTM)

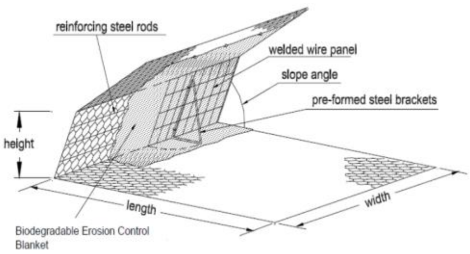

Figure 4: GTM unit diagram

A GTM reinforced soil bund was selected as the preferred type of mitigation option due to the smaller footprint, low maintenance, visual aesthetic, and greater resilience to both repeated SLS level impacts and over-design/extreme event compared to the other options. Green Terramesh units are an integral soil reinforcing unit consist of Galfan PVC coated double twisted wire mesh forming the face, reinforcing tail, and top lid. These units are laid back to back to form a bund with 70 degree front and back faces. Additional geogrid reinforcement is normally included for taller structures.

This structure type is used elsewhere on the project, allowing for easier procurement, and has been used extensively around Christchurch following the 2010-11 Canterbury Earthquakes. Construction uses standard earthworks techniques and can utilise spoil material from landslide debris on-site as backfill with limited processing, offering significant cost and time savings.

3 DESIGN BOULDER ASSESSMENT

Determining the required bund dimensions and energy capacity took a two part approach. In order to ensure that the rockfall barrier was appropriately dimensioned to intercept falling rocks at this site, the requirements for bund height and position were based on rockfall modelling calibrated to the observations of rocks falling during scaling and blasting on site. This covered a broad distribution of rock sizes from small to moderately large, which were observed to have a wide range of bounce heights and trajectories, and as such define the required dimensions (namely crest height) for the bund in order to effectively reduce the likelihood of rocks bouncing into the rail and road corridors. The rock size distribution for this assessment was intended to give a proportional representation of the rocks which are observed in the source areas. As such, this distribution is dominated by the smaller blocks which form a significant portion of the debris cone and source rock.

Boulder sizes for determining the required impact energy capacity of the bund have been modelled from field mapping and observations of the characteristic rockfall at this site as part of the November 2016 earthquake and during blasting throughout 2017. Observations indicated that large rocks, rather than bouncing down the slope and impacting the catch ditch at the slope toe, attained a high rotational velocity as they travel down the slope, making them more likely to roll across the catch ditch and onto the road. Using a rock size distribution based on the rocks observed in the source areas to determine the impact energy for the barrier design skews heavily towards the smaller (<0.5 m) boulders. Carrying out the impact energy analysis using 'design boulder sizes' based on those rocks observed in the road corridor after the November 2016 earthquake was considered to be more appropriately conservative.

From aerial imagery and site observations, 32 significant boulders were identified and measured, with volumes ranging from 0.4 m^3 to 28m^3. Impact energies at the location of the proposed barriers have been modelled based on these measured boulders, with 4 discrete boulder sizes chosen as representative: the mean of the whole data set; a screened mean of the data set (the upper and lower 10% screened out to reduce the effect of outliers); the screened mean of data set + 1 standard deviation; and the max observed boulder size.

4 BUND DESIGN

4.1 Rockfall modelling

In order to determine the required dimensions and location for the protective structure, rockfall modelling was carried out using 11 cross sections of the site. Determining the rockfall pathways for modelling used adjusted water drop analysis along with site notes and observations on observed rockfall pathways, in order to develop 2.5D or "quasi-3D" modelling (Chen 2013). This involves determining the horizontal rockfall trajectory on the X-Y plane by identifying preferential pathways rocks are observed to travel down, and then translating this into a 2-D profile for use in traditional 2D modelling. Modelling was carried out separately for the "boulder distribution" and the "design boulder" cases, with the slope surface materials and roughness calibrated to accurately reflect the different behaviour observed for these two boulder sizes. Rockfall modelling determined the required height for the bund in order to stop a minimum of 95% of falling rocks from passing. This varied across site, with the midslope bench across chutes B and C acting to significantly reduce the number of rocks reaching the bund with large bounce heights. The area beneath Bluff A also had a limited likelihood of rocks passing the bund, due to the topography and presence of thick vegetation in this area. The results of the modelling indicated a required minimum bund height of 5m under Chute A, 3m under Chutes B and C, and that a 1m deep catch ditch was sufficient protection under Chute D.

4.2 Foundation Requirements

The GTM bund is founded 0.4 m below ground surface at the downslope toe of the bund, and constructed to heights of 5.4m and 3.6m at chute A and chutes B/C, respectively (total height from footing), giving an above ground height of 5 m and 3.2 m. The existing material at the location of the bund footing consists of granular earthquake debris. This material has previously been compacted and prepared during the installation of the temporary ballasted shipping container protection. Minor subgrade preparation and proof rolling was required to ensure no soft spots are present along the bund footprint. Due to the semi-flexible nature of the GTM bund, this structure is not as sensitive to minor settlement in the subgrade as more rigid rockfall protection structures such as rigid walls and barriers; and does not require the same foundation formation as other similarly high energy capacity structures, such as anchored fences.

4.3 Design Details

The GTM reinforced bund was selected for: its proven effectiveness as a protective structure through independent full scale impact trials; the availability of a simplified design approach, calibrated through numerical analysis and verified back analysis of actual boulder impacts; its high level of redundancy and resilience; constructability, and the ability to reuse onsite material for fill. Key points of the design include:

- The design bund height includes an appropriate design freeboard to reduce the likelihood of large rocks impacting near the crest and passing over the bund;
- Adequate thickness at the predicted rock impact heights ensuring bund stability requirements are met
- Appropriate embedment depth at the downslope face to ensure adequate sliding resistance
- Front and rear faces angled at 70 degrees (2.75V:1H) to: minimise the required footprint for the structure whilst achieving the required design height; maximise the upslope catchment space for storage for rockfall; and to allow earthworks equipment behind the bund to clear future rockfall material. The 70 degree upslope face is important to prevent boulders with high rotational velocity from overtopping the bund (Figure 5)

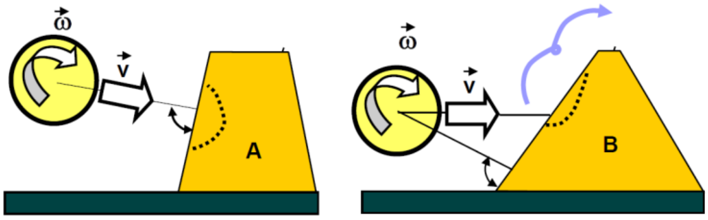

Figure 5 70 degree face of the GTM prevents rocks with high rotational energy from rolling over the bund

- The bund is to have a 1.5m wide crest along its entire length, to ensure the appropriate energy capacity is met (detailed below), and to allow easier and safer access to the upper portion of the bund with compaction equipment during construction
- The modular construction style of the GTM bund allows local repair of individual sections of the bund to be carried out in instances where large isolated rockfall causes damage to the structure. The ductile nature of the GTM bund, and the way it mobilises its resistance to large rockfall impacts by internal plastic deformation means that the bund is capable of withstanding multiple ULS level events, or isolated events in excess of its design limit without catastrophic failure. Following these events, the individual damaged areas can be patched up, or removed and replaced as necessary
- Free draining spoil material from on-site can be used as the general embankment fill, subject to rework/grading to ensure suitability. The outer portion of the GTM units are formed with clean rock fill. This is both for aesthetic properties, as well as reducing the maintenance requirements of the bund by reducing the likelihood of weeds growing on the bund face.
- The 5.4m high bund sections require geogrid reinforcement in the lower GTM unit layers to ensure the internal stability of the bund during design earthquake events
- The bund alignment has been designed to allow ease of access for machinery in order to clear the storage area at the upslope face. This access is facilitated by space at the southern end of Chute A, and at the northern end of Chute C to allow earthworks equipment ease of access to the area upslope of the bund to remove individual rocks resulting from discrete rockfall events
- An intermediate access point is facilitated by the construction of a non-reinforced section of the bund between Chutes A and B,

- This section of the bund has been positioned downslope of a rock bluff which has reduced rockfall activity compared to the main chutes, as noted from site observations; and below a heavily vegetated section of the slope, further reducing the likelihood of impact by large scale rockfall
- This section of the bund is to have 1V:1.5H angled front and back faces to preclude the use of any internal reinforcement, so that if temporary access is required behind the structure, this bund section can be temporarily deconstructed by earthworks equipment, and reinstated after works have finished
- The unreinforced bund is 3m tall, with a crest width of 1.5m. This geometry has been included in the rockfall modelling, and shown to effectively reduce the risk from rockfall in this area

4.4 Capacity Assessment/Checks

4.4.1 Bund Stability

Calculations of internal and external stability of the bund were carried out by the supplier (Geofabrics NZ Ltd) using their in-house software "Macstars", and verified and accepted by NCTIR. These calculations used input parameters derived from the material available on-site for foundation and bund fill materials. These properties were consistent for both of the fill types (foundation and bund fill), and represent processed slip debris. The specific material properties used for these calculations are presented in

Table 1. A maximum particle size of up to 200mm diameter was used for this fill material, in order to include the appropriate level of installation damage and particle interaction when determining working geogrid strengths.

The results of these calculations show the bund to be stable in static conditions, with a yield threshold reached when subjected to peak horizontal earthquake foundation acceleration of greater than 0.63g, well in excess of the ULS design acceleration for this structure. Under the prescribed loading from SLS or ULS events, the bund is expected to show negligible levels of deformation or displacement, as design PGA levels are lower than the yield threshold for the bund. Above the design ULS earthquake acceleration, critical failure pathways identified in internal stability analysis identify shallow internal slip planes near the base of the bund, indicating very slight deformation of the lowest GTM unit face could be expected, which would not be expected to compromise the stability or overall effectiveness of the bund. The Bridge Manual requirements for static and seismic stability are therefore met for the MSE bunds.

4.4.2 Impact Resistance

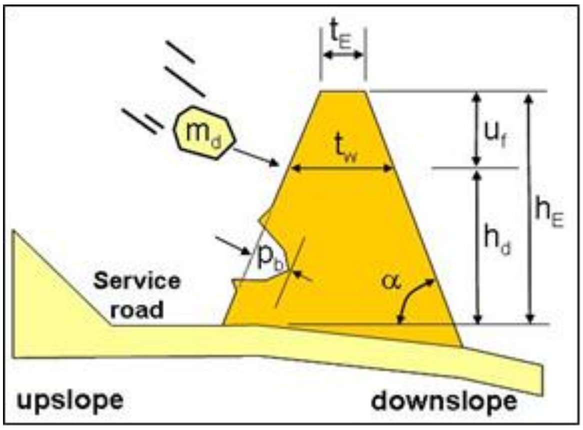

Figure 6: Essential parameters for sizing of the GTM reinforced soil bund

The design of the reinforced soil bund is primarily based on the penetration depth of an impacting rock, based on kinetic energy and size. This penetration depth largely determines the required thickness of the bund. The relationship between penetration depth and boulder sizes with their kinetic energy has been derived in a simplified chart developed by Calvetti & Di Prisco (2007). The soil mass combined with reinforcement elements exhibits vastly enhanced strength deformation properties and provides greater ability to resist forces and absorb kinetic energy. This is due primarily to the increased ductility of the reinforced soil mass. As each individual GTM unit and section is connected to the adjacent bund, large sections of the bund are utilised to develop resistance to impact in terms of larger scale bund failure modes such as sliding or toppling. The primary mode of deformation from rockfall impact is the internal deformation of the bund, which presents as a displacement on the upslope side of the bund from the penetration of the boulder, and displacement outwards on the downslope face.

Allowable penetration depths for SLS and ULS states are related to minimal patch up work, and full repair of damaged sections, respectively. Table 2 is based on the Calvetti & Di Prisco (2007) graph summarizing estimated boulder size and penetration depth on the upslope side of the bund, with corresponding energy at impact. From here, the energy capacity for SLS and ULS conditions can be determined.

Table 1: Material properties for stability calculations

Material type	Max particle size	Friction angle	Cohesion	Bulk unit weight	Poisson's ratio
Slip debris	200mm	36°	0 kPa	20 kN/m³	0.3

Table 2: GTM bund energy capacity based on upslope penetration depths

		Minimum patch up work*					Moderate repair to possible reconstruction of impacted section						
Boulder diameter (m)	Boulder Vol (m^3)	Energy (kJ) at 0.3m penetration	Energy (kJ) at 0.4m penetration	Energy (kJ) at 0.5m penetration	Energy (kJ) at 0.6m penetration	Energy (kJ) at 0.7m penetration	Energy (kJ) at 0.8m penetration	Energy (kJ) at 0.9m penetration	Energy (kJ) at 1.0m penetration	Energy (kJ) at 1.1m penetration	Energy (kJ) at 1.2m penetration	Energy (kJ) at 1.3m penetration	Energy (kJ) at 1.35m penetration
0.42	0.039	4.0	40.0	-	-	-	-	-	-	-	-	-	-
0.72	0.195	-	60.0	180.0	350.0	-	-	-	-	-	-	-	-
0.90	0.382	-	80.0	200.0	400.0	700.00	-	-	-	-	-	-	-
1.56	1.988	-	-	500.0	900.0	1600.00	2000.00	3000.00	3500.00	-	-	-	-
1.96	3.942	-	-	650.0	1300.0	2000.00	3000.00	4000.00	5000.00	6500.00	8000.0	9000.0	10000.0

Serviceability Limit State

For SLS, the embankment upslope penetration should be <20% of the embankment thickness at the impact height, and not greater than 0.7m. Rockfall modelling for SLS events uses the "rock size distribution" in the source area. The modelling results for this SLS case show that for the sections under Chute A where the bund is 5.4m tall, the 90^{th,} 95th and 99th percentile rock impact energies are less than 100kJ, 200kJ, and 500kJ, respectively, and the 95th percentile rock impact height is at 3m or less. At this impact height the bund width is 3m, giving a maximum allowable penetration of 0.6m. Based on the information presented in Table 2, this equates to a maximum impact energy of 350-1300kJ depending on the impacting rock size.

For the sections under Chutes B and C where the bund is to be 3.2m tall, the 90^{th,} 95th and 99th percentile rock impact energies are less than 100kJ, 200kJ, and 600kJ, respectively, and the 95th percentile rock impact height is at 2m or less. At this impact height the bund width is 2.5m, giving a maximum allowable penetration of 0.5m, equating to an impact energy of 180-650kJ depending on the impacting rock size. Predicted and allowable penetration depths for SLS cases are summarised in

Table 3.

Ultimate Limit State

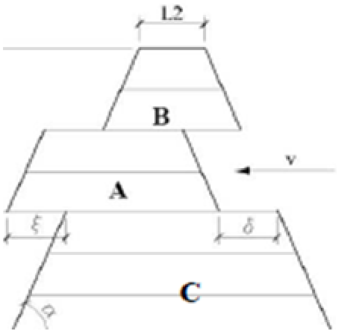

Figure 7: Simplified cross section of the embankment geometry after rock impact (Ronco et al., 2009)

The Ultimate Limit State of the bund is determined by evaluating the static stability of the structure after impact deformation. The projection of Block A centre of mass has to be inside the front support of Block C or the projection of the Block B centre of mass has to stay in equilibrium with Block A (Figure 7).

Rockfall modelling for ULS events considers the design boulder sizes mentioned in Section 3. Rockfall observations indicate that these large boulders are rolling or bouncing at less than 0.5m high when they reach the bund location, so the impact height is considered to be the centre of mass of the rock, or half its diameter, plus the bounce height. The energy capacity of the bund is evaluated using the energy plots from the "Screened mean + 1 standard deviation" boulder size, with an additional capacity check for the "Max observed" boulder size to assess the behaviour of the bund when subjected to impact from the largest boulders observed from the 2016 earthquake. The average diameter for the "Screened mean + 1 standard deviation" boulder size equates to 2m, so the predicted impact height for the design boulder will be 1.5m. At this impact height, the bund width on the 5m high section is 4m; and the bund width on the 3.2m high section is 2.7m; giving a maximum allowable penetration of 2m under Chute A, and 1.35m under Chutes B and C. For the max observed boulder size, the average diameter was 3.9m, giving a design impact height of 2.45m. The bund width under Chute A at this height is 3.4m, giving a maximum allowable penetration depth of 1.7m; and the bund width under Chutes B and C at this height is 2m, giving a maximum allowable penetration depth of 1m.

Rockfall modelling results for the sections under Chute A show the 90th and 95th percentile impact energies for the "Screened mean + 1 standard deviation" boulder size to be 2700kJ and 4000kJ, respectively, giving a predicted penetration of between 0.9-1.0m. The 90th impact energy for the max observed boulder size is 9750kJ, giving an approximate penetration depth of 1.35m.

Rockfall modelling results for the sections under Chutes B and C show the 90th and 95th percentile impact energies for the "Screened mean + 1 standard deviation" boulder size to be 500kJ and 1400kJ, respectively, giving a predicted penetration of between 0.6-0.7m. The 90th impact energy for the max observed boulder size is 2000kJ, giving an approximate penetration depth of 0.8m.

Predicted and allowable penetration depths for ULS cases are summarised in

Table 4.

Figure 8: Partially completed bund beneath Chute B before removal of temporary protection

Table 3: SLS boulder impact penetration

Bund Section	Boulder size	Impact energy (percentile)	Impact energy (kJ)	Predicted penetration depth (m)	Allowable penetration for minimal patch-up work (m)
Chute A	SLS Distribution	90th	100	0.4-0.5	0.6-0.7
		95th	200	0.4-0.5	
		99th	500	0.5-0.6	
Chute B/C	SLS Distribution	90th	100	0.4-0.5	0.5-0.7
		95th	200	0.4-0.5	
		99th	600	0.5-0.6	

Table 4: ULS boulder impact penetration

Bund Section	Boulder size	Impact energy (percentile)	Impact energy (kJ)	Predicted penetration depth (m)	Allowable penetration for minimal patch-up work (m)
Chute A	Screened mean + 1 standard	90th	2700	0.9	2.0
		95th	4000	1	
	Max observed	90th	9750	1.35	1.7
Chute B/C	Screened mean + 1 standard	90th	500	0.6	1.35
		95th	1400	0.7	
	Max observed	90th	2000	0.8	1.0

5 CONCLUSION

For the protection of the downslope infrastructure and to provide safety to people using the transport corridor passive protection in the form of a large Green Terramesh Bund was determined to be the most robust and effective solution beneath this area of significant rockfall hazard. This structure provided a number of critical benefits over other available protection measures, which allowed it to be constructed within the limited space available between the slope toe and the downslope rail and road network. Crucially these included the ease and speed of construction, the limited requirement for site preparation and anchorage, and the robust nature of the structure in its response to large rockfall impacts, and the relatively low level of deflection experienced as a result of large impacts. Key aspects in the modelling and design of this protective structure include:

- A good understanding of the range of hazards exhibited at the site
- Calibration of rockfall modelling with observations from rockfall events
- Calculation of the required dimensions of the bund in order to ensure the appropriate crest height to prevent rocks passing the bund
- An understanding of the rockfall behaviour at site, including the specific characteristics of large and small rocks, and the calculation of potential impact energies to ensure the appropriate bund thickness to resist large impacts
- Consideration during the design phase for safe areas to inspect the area behind the bund for future monitoring, and to allow access for equipment to safely clear future accumulated debris

When appropriately designed and dimensioned, this type of structure can form a resilient and robust protective barrier for significant rockfall events, with a large energy capacity, limited footprint and deflection, and significantly reduced whole of life costs compared to other hard engineering structures such as high energy rockfall barriers and debris fences.

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