Rock mass fall – rock avalanche design loads for railway track protection structures

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ABSTRACT
On November 25th, 2012 a 53,000 m^3 rock avalanche buried the Canadian National Railway (CN) main track at Mile 109.4, Ashcroft Subdivision, near Boston Bar, causing a 4 day service disruption. The rock avalanche resulted in the collapse of a 21 m long concrete rock shed for track protection. This paper describes the analyses carried out to derive impact loads for design of a new track protection structure including; i) description and characterization of the November 25th, 2012 event; ii) dynamic runout back-analysis; iii) forward-analysis to derive rock avalanche impact loads; and iv) interpretation of loads for structural design of a new composite barrier wall/rock shed structure. A modified version of the pseudo-three-dimensional runout analysis software DAN-W is used, allowing output of normal and shear stresses at the base of a sliding frictional mass. The software allows computation of centrifugal acceleration for each sliding mass element. Results show that peak normal and shear stresses are sensitive to sharp terrain breaks (i.e. changes of the terrain slope), due to centrifugal forces generated by the moving mass. Peak stresses occur in the frontal part of the rock avalanche. Stress magnitudes are sensitive to angle (or radius) of terrain break and incoming velocity. These findings were used to optimize the design of the new track protection structure to the loading from a rock avalanche hazard.

RÉSUMÉ
Le 25 novembre 2012, un éboulement avec un volume de 53,000 m^3 a recouvert la voie de la compagnie des chemins de fer nationaux du Canada (CN) au Mile 109.4 de la subdivision de Ashcroft, près de Boston Bar; il engendra une interruption de quatre jours le long de cet important tronçon de la ligne est-ouest, en Columbie Britannique. L' éboulement a détruit une gallerie de protection de 21 m. Cet article présente les analyses qui ont permis de calculer les forces d'impact pour le design d'une nouvelle gallerie de protection. Les analyses comprennent : i) caractérisation de l'éboulement du 25 novembre 2012; ii) analyse dynamique de propagation; iii) estimation des forces d'impact; et iv) design d'une nouvelle structure de protection formée d'un mur de soutènement combiné à une gallerie de protection. Nous avons utilisé une version modifiée du programme DAN-W, qui permet de calculer les contraintes normales et cisaillantes à la base d'une masse en movement. Ce programme calcule l'accélération centrifuge des éléments de la masse. Les résultats montrent que les contraintes normales et cisaillantes dépendent des changements abruptes de pente, en raison des forces centrifuges engendrées par la masse en mouvement. La contrainte maximale se situe à l'avant de l'éboulement. L'ampleur de la contrainte est fonction de l'angle (ou rayon) des changements de pentes et de la vitesse acquise. Ces résultats ont permis d'optimiser le design de la nouvelle structure de protection en fonction des forces engendrées par un éboulement potential.

1 INTRODUCTION

On November 25th, 2012 a 53,000 m^3 rock avalanche buried the Canadian National Railway (CN) track at Mile 109.4, Ashcroft Subdivision, near Boston Bar, causing a four day service disruption on British Columbia's main east-west railway connection. The rock avalanche resulted in the collapse of a 21 m long concrete rock shed for track protection. Subsequently, Sturzenegger et al. (2014) characterized and analyzed the geohazards on the slope above the railway track. Keegan et al. (2014) describe the geohazard assessment carried out at the site as well as the design and construction of a new 80 m long track protection structure. The permanent composite barrier wall/rock shed structure consists of a tied-back, gravel-filled barrier wall designed to absorb a large portion of the impact loads during a future rock landslide event, and a rock shed allowing the rock landslide to safely travel over the railway track. The following sections describe the analyses carried out to derive impact loads for design of a new track protection structure.

Figure 1. Canadian National Railway (CN) track at Mile 109.4 Ashcroft Subdivision near Boston Bar, BC above Fraser River. The previous rock shed is visible in the before photo. Red dotted line indicates section for back-analysis.

2 NOVEMBER 25TH, 2012 ROCK AVALANCHE

Figure 1 shows photos of the slope before and after the November 25th event. The landslide was likely triggered by a combination of freeze-thaw cycles and heavy precipitation. The failure mechanism was discontinuity-controlled; by sliding along joint set J3 (dipping at 44˚ out of slope), while the steeply dipping joint set J4 acted as the back release. Shearing occurred through rock bridges between J3 and J4. Using the landslide classification after Hungr et al. (2001), the event is characterized as rock avalanche; i.e. release of a dry, fragmented rock mass (jointed but relatively intact at source), running out in flow-like motion at extremely rapid velocity (> 5 m/s). The landslide released an in-situ volume of 53,000 m^3, leaving a 70 m wide crest at EL. 295 m. At track level (EL. 186 m) the landslide deposit was 80 m wide, spreading to a total width of 140 m at Fraser River bank (EL. 113 m). Horizontal runout distance was approx. 190 m from crest to river bank, with some material travelling further into the river.

Photos and field observations from before the event confirm the rock mass was relatively intact. After the event, field observations revealed relatively high proportions of cobbles, gravel and sand particles in the landslide deposits. Consequently, rock fragmentation must have occurred during the event. Initially, the mass may have moved as a relatively coherent, sliding block, but the mass likely fragmented after a few meters of movement. After that, ‘flow-like’ rather than ‘block-sliding’ behavior is inferred.

Deposition in travel direction was mainly controlled by topography, namely the presence of the horizontal railway track and the Fraser River further downslope. In absence of bathymetric survey, the river bed is inferred to be flatter than the rock slope above. Lateral spreading above the tracks was limited by a rock outcrop above the southern end of the previous rock shed (Note; south is right in Figure 1). Below the tracks, three distinct bedrock outcrops somewhat divided the flow and increased lateral spreading. Generally, spreading increased with travel distance.

Water does not appear to have played a significant role in either the initial sliding or rheology during runout. Talus deposits along the landslide path and in the gully, above the previous rock shed, may have been (partially) saturated but no evidence is at hand. The ballast under the railway tracks is inferred as non-saturated. For the purpose of this analysis, all other surfaces in the landslide path are interpreted as bedrock outcrops. Hence, no pore-pressure or liquefaction effects are considered. No evidence of air-cushioning at the base of the flow is available.

Consequently, a purely frictional, dry rheology is used for modeling.

3 MODIFIED DAN-W SOFTWARE FOR RUNOUT ANALYSIS

A modified version of the pseudo-three-dimensional runout analysis software DAN-W is used for the runout analysis. The rock mass fall is modeled as a homogeneous continuum of “apparent fluids” using a Lagrangian solution for the shallow flow equation (Hungr, 1995). A variety of material rheologies can be defined in

DAN-W by specifying parameters along the path and within the slide mass.

The software models rock mass falls as a series of blocks moving downhill under gravity, while resistance is acting at the base of the blocks, and pressure terms are acting between the blocks.

For the purposes of this project, the model was modified to allow output of normal and shear stresses acting at the base of the moving landslide mass.

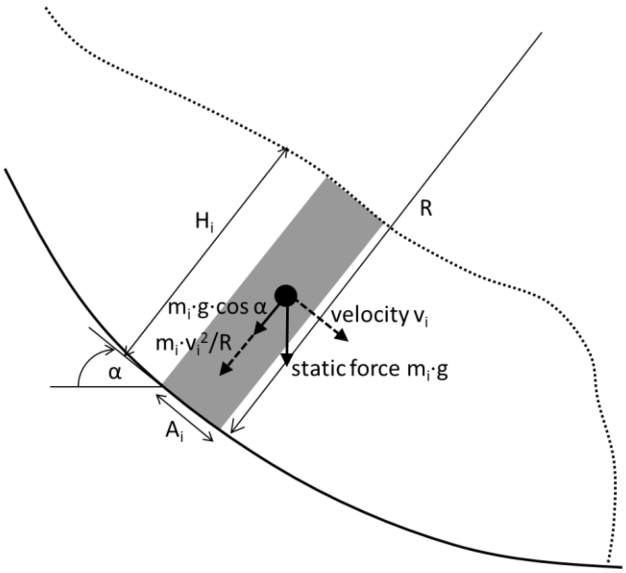

Figure 2. Static and dynamic normal forces acting at base of a sliding element.

The mass of an element m_i in DAN-W is given by flow depth H_i, area of element A_i material density ρ. Eq. 1 is modified from Hungr (1995, Eq. 10) and Figure 2 illustrates that the normal force N below a sliding element is made up by the normal component of the static force and the dynamic force due to centrifugal acceleration a_c.

$$N = m_i \, \mathrm{g} \cos\alpha + m_i a_c \qquad [1]$$

The centrifugal acceleration a_c is defined as squared velocity of the moving element v_i divided by the vertical curvature radius of the path R.

$$a_c = \frac{v_i^2}{R} \qquad [2]$$

The normal stress σ_N at the base of an element is:

$$\sigma_N = \frac{N}{A_i} \qquad [3]$$

The shear stress τ below the element is the product of effective normal stress $\sigma_N(1 - r_u)$ and frictional resistance $\tan\varphi$.

$$\tau = \sigma_N(1 - r_u)\tan\varphi \qquad [4]$$

Note; r_u is the ratio of pore pressure to normal stress. Since the analysis in this paper is for dry material, r_u is equal zero.

4 DYNAMIC BACK-ANALYSIS

A section for back-analysis was cut just to the north (i.e. left in Figure 1) of the pre-existing rock shed, between two bedrock outcrops below the tracks and close to a point where the track was visible below the debris. Small path irregularities in the section cut from LiDAR data were smoothed to avoid energy losses and increase model stability.

The relatively uniform angle of repose (approx. 40° to 45°) of the ultimate deposit was likely caused by debris raveling onto an initial deposit of the main rock avalanche. Hence, the calibration objective was not to match the 'even deposit' (at angle of repose) on the tracks, but to achieve an initial deposit on the tracks.

Back analysis was carried out to find a best-fit dynamic friction angle. Parameters used include; a nominal 1 m path width, 50 normal slices, frictional rheology, 20.0 kN/m^3 unit weight, 35° internal friction angle, no pore pressures, and no erosion. Launch into trajectory was not allowed.

A best fit dynamic friction angle of 35° resulted in significant deposition on the track, but smaller than the ultimate deposit.

5 FORWARD-ANALYSIS OF ROCK AVALANCHE IMPACT LOADS

Forward analysis was carried out using a dynamic friction angle of 35° along a section oriented on the fall line towards the new proposed rock shed. Path width was assumed to be similar to the November 25th, 2012 event. Fracturing of the initially 'intact' rock mass was modeled by restricting slice deformation for the first 10 m of movement, then allowing slices to deform and to runout in a flow-like manner.

Runout scenarios for three different failure surfaces (FL-0, FL-1, FL-2) were analyzed, of which FL-1 is presented here. The failure surface retrogresses 52 m uphill from the main scarp of the November 25th, 2012 landslide. The transition from the bedrock slope onto the horizontal backfill surface (EL. 196 m) was approximated by a spline function with gradually changing curvature (min. radius 9.4 m).

DAN-W simulation of scenario FL-1 (Figure 3) resulted in 510 kPa peak normal stress, with 360 kPa peak shear stress, at 4 m flow height and 21 m/s. Numerical simulations of impacts on the proposed track protection structure show that normal and shear stresses peak in the concave transition from rock slope to horizontal backfill surface (i.e. 178 m horizontal distance in Figure 3). Peak stresses occur when the frontal part of the rock avalanche passes through this transition.

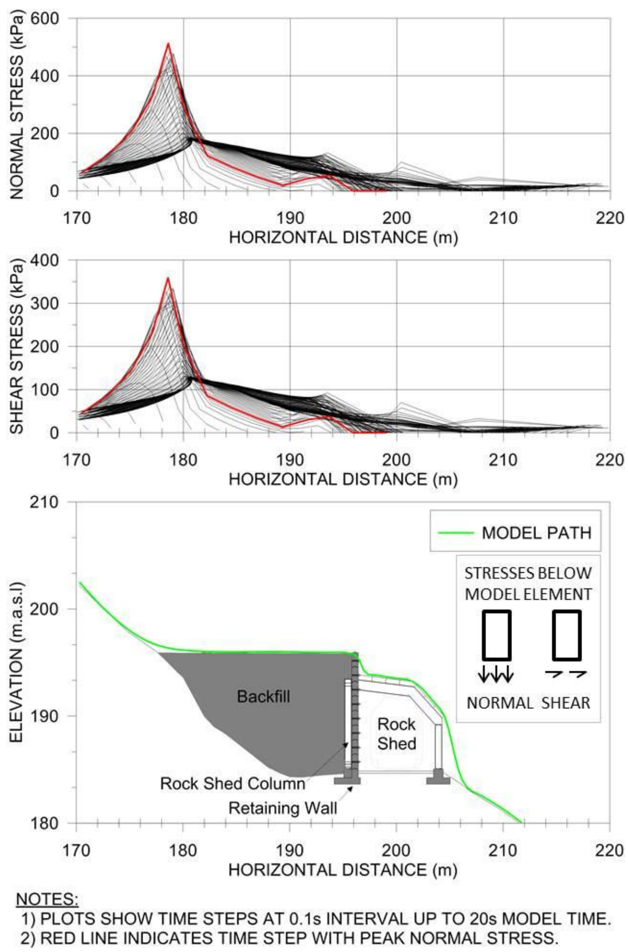

Figure 3. Top two figures are DAN-W results for normal stress and shear stress. Bottom figure shows the proposed retaining wall and rock shed.

6 INTERPRETATION OF LOADS FOR STRUCTURAL DESIGN OF ROCK SHED

The new track protection structure (Figure 3) consists of two main components; a backfilled retaining wall absorbing a large portion of the impact loads during the rock avalanche event, and a rock shed allowing the rock avalanche to safely travel over the railway track. Both components are structurally independent. The retaining wall is made from pre-cast concrete panels, tied back with anchors to bedrock and backfilled with debris from the previous event. The rock shed is made from a steel frame covered with pre-cast concrete panels on the top. The upslope vertical steel column is recessed into the retaining wall and the column top is tied back into bedrock. The structures are modular, facilitating construction under railway traffic.

The rock avalanche impact loads on the structures were established based on the DAN-W results. Various simulations were carried out to verify the adequacy of the design loads used for the preliminary design and to refine the detailed design of the structures. In preliminary structure design of the retaining wall, an equivalent 200 kPa surcharge (normal stress) was assumed to be constant over the top of the backfill behind the retaining wall, in addition to the backfill earth pressure at rest. A drag force (shear stress) of 200 kPa at the top of the backfill was also used for the wall analysis based on the DAN-W analysis; this drag force was linearly reduced to zero at the bottom of the wall. The unit weight of the backfill material was assumed as 20 kN/m^3.

In the detailed design the rock avalanche load interpretation was refined; the normal stress (acting perpendicular to horizontal backfill surface) and shear stress (drag force acting parallel to horizontal backfill surface) were integrated along the horizontal distance for a unit width. This approach accounted for the variation of stresses along the horizontal distance and the location of the stresses in relation to the wall.

7 CONCLUSION

1. This paper demonstrates a rational approach for definition of realistic rock avalanche design loads on a composite barrier wall/rock shed structure. DAN-W software was used for dynamic simulation of normal and shear stresses on top of the backfilled barrier wall.

2. For the problem analyzed in this paper, peak stresses occur when the frontal part of the rock avalanche passes through the transition from the sloped rock face to the horizontal backfill surface.

3. Numerical integration of DAN-W stress outputs along the backfill surface was carried out in order to account for stress variation for design of the retaining wall.

4. Simulation results can be verified with simple hand calculations using the equations provided in this paper.

The design approach summarized in this paper was used for design of the composite barrier wall/rock shed structure for Canadian National Railway main track at Mile 109.4. Construction was completed in spring 2014.

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REFERENCES

Hungr, O. 1995. A model for the runout analysis of rapid flow slides, debris flows and avalanches. *Canadian Geotechnical Journal*, 32: 610-623.

Hungr, O., Evans, S. G., Bovis, M. J., and Hutchinson, J. N., 2001. A review of the classification of landslides of the flow type. *Environmental & Engineering Geoscience*, Vol. VII, No. 3, Aug. 2001, pp.221-238.

Keegan, T., Edwards, T., Sturzenegger, M., Busslinger, M., Willoughby, B., and Wen, A. 2014. Construction of a composite barrier wall/rock shed structure at mile 109.43 of CNR's Ashcroft Subdivision. *Geohazards 6*, Kingston, Canada, June 15-18.

Sturzenegger, M., Keegan, T., Wen, A., Willms, D., Stead, D. and Edwards, T. 2014. LiDAR and discrete fracture network modeling for rockslide characterization and analysis*, IAEG XII Congress*, Torino, Italy, September 15-19.