Finite Element Analysis for Rockfall and Debris Flow Mitigation Works

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for Rockfall and Debris Flow Mitigation Works

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Abstract

Landslide risks arising from boulder falls and debris flows are commonly mitigated using rigid and flexible barriers. Debris-barrier interaction is a complicated process so current design methods rely on the use of the pseudo-static force approach. In addition to physical testing, numerical simulations can be used to provide insight into the impact mechanism. This paper presents the applications of numerical models to simulate rigid and flexible barriers subjected to rockfall and debris flow impacts respectively. For rigid barriers, rock-filled gabions, recycled glass cullet, cellular glass aggregates and EVA foam were assessed for their performance as cushioning materials. From the results, empirical equations were established for predicting the boulder impact forces and penetration into the cushion layer. Amongst the materials considered in this study, rock-filled gabions appear to be the most promising for use in practice. For flexible barriers, finite-element models, calibrated using documented case histories, were developed to simulate the debris-barrier interaction. The models were used to investigate the barriers' behaviour under debris impacts from both force and energy perspectives. From the results, the hydrodynamic pressure coefficient was found to be lower than the current recommended value whilst only a small amount of debris energy was transferred to the barrier.

Keywords: finite-element method; debris-resisting barriers; boulder and debris flow impacts; cushioning materials; fluid-structure interaction

Introduction

Background

Large-scale landslide mass movements such as debris flows and rockfalls remain a threat to the mankind, and the challenges brought about by landslides are expected to increase due to more frequent extreme weather events (Ho et al. 2017). In many urban areas, installation of structural countermeasures along the predicted debris flow paths is an effective means of mitigating the landslide hazards (Kwan et al. 2016). Examples of these structures include reinforced concrete rigid barriers (Mancarella and Hungr 2010; Kwan et al. 2015) and steel flexible barriers (Roth et al. 2010; Volkwein et al. 2011). Landslide-prone countries around the world have been adopting these structures to combat against natural terrain landslide hazards.

Review of Current Design Practice

Debris-resisting barriers are designed to have adequate structural capacity to resist the impact loading of the debris flow. For debris flows, the impact loading is commonly assumed to be in the form of an areal pressure. If boulders along the flow path are mobilised, they may be engulfed to form part of the flow and may accumulate at the front (Jakob and Hungr 2005; Takahashi 2014). For this reason, debris-resisting barriers also need to be designed to resist the concentrated loadings of boulders (Hungr et al. 1984).

Rigid reinforced concrete barriers, due to their high bending stiffness and shear resistance, can resist debris-flow impacts without showing any notable deformation, though localised structural damage can still occur. At present, the pseudo-static force approach is commonly used for the design of rigid barriers in many countries (NILIM 2007; ASI 2011, 2013; SWCB 2005; Kwan 2012), though a displacement-based approach is also under active development

(Lam et al. 2018b; 2018c). In the force-based approach, the transient impact load acting on the barrier wall is assumed to be a sustained pseudo-static load. Debris impact pressure is typically estimated using the hydrodynamic equation (Lo 2000; Proske et al. 2011), while concentrated boulder impact forces can be estimated using the Hertz equation in its simplified form (Kwan 2012; Zhang et al. 1996). With the impact loads established, the design of the barrier can be carried out using the conventional approach in which the conditions of force and moment equilibrium are satisfied. In practice, in order to prevent localized structural damage, a cushion layer may also be installed on the barrier though the associated beneficial effect is often ignored in geotechnical design. Figure 1 shows a rigid barrier shielded by a gabion wall in Hong Kong.

Steel flexible barriers, on the other hand, work very differently from their rigid counterparts. When subjected to debris flows or boulder impacts, these barriers deflect and allow the debris or boulders to travel with the steel net for a certain distance. This effectively prolongs the impacting process and attenuates the impact load. Steel flexible barriers can absorb a significant amount of the impact energy via mobilization of the energy-dissipating devices installed along the wire ropes. Figure 2 shows a steel flexible barrier on a natural hillside in Hong Kong. Flexible barriers have been designed using a number of methods, which can be based on verified field performance (EOTA 2013), empirical experience (GEO 2014) and calculations (Brighenti et al. 2013; Kwan et al. 2014; Volkwein et al. 2015). At present, the calculation method still requires the use of equivalent pseudo-static forces. Numerical modelling is also required for the design of flexible barriers since these structures exhibit highly non-linear geometrical and material stress-strain behavior. For example, Volkwein (2004) developed a software known as FARO for modelling flexible barriers subjected to a rockfall impact, and Bugnion et al. (2012) extended the use of FARO for the simulation of

debris-flow impacts. Elsewhere, Chan et al. (2012) developed a nonlinear finite-element program known as NIDA-MNN for the structural analysis of steel flexible barriers. This program was used by Kwan et al. (2014) to back-analyse a case history of a shallow landslide impacting on a steel flexible barrier in Hong Kong. Sze et al. (2018) also adopted this program for the design of flexible debris-resisting barriers in practice. A common feature of these computer programs is that the analysis is decoupled, meaning that the evaluation of the impact loading and the structural assessment are carried out separately. These programs also require the input of a pseudo-static force, so that the dynamic interaction between the impacting debris and the barrier needs not be explicitly considered. Despite this convenience, the use of pseudo-static forces could lead to over-conservative design, since certain important phenomena such as the effect of strain rate and the transient nature of impact loading are ignored. This in turn could lead to the construction of unnecessarily large barriers which are both expensive and unsightly.

Numerical Method

LS-DYNA is a general-purpose finite-element program developed by Livermore Software Technology Corporation (LSTC 2017a). This program has an automated contact search algorithm, which is more robust than the non-automatic ones as there is no orientation requirement. LS-DYNA also contains a wide range of built-in constitutive material models. In geotechnical engineering, this program has been employed to solve dynamic problems such as the seismic performance of reinforced soil walls, soil behavior under blast loading, and spudcan-pile interaction (Lee et al. 2010; Tho et al. 2013; Xu and Zhang 2015). In recent years, this program has been successfully used to simulate the mobility of debris flows for the design of structural countermeasures. For examples, Kwan et al. (2015) and Koo et al. (2018) implemented the Voellmy model in LS-DYNA to back-analyse notable landslides in Hong

Kong, Ng et al. (2018a) also used LS-DYNA to study the dynamic interaction between a dry granular flow and multiple debris-resisting barriers. Compared to the conventional depth-averaged models in which the debris flow mass is discretized into a series of inter-connected slices (Hung 1995; Kwan and Sun 2006), the finite-element method can explicitly simulate the internal shearing and the deformation of the debris when impacting on a barrier. For the modelling of barrier structures, Ng et al. (2012) and Koo et al. (2017b) used LS-DYNA to analyse the performance of steel flexible barriers under the impacts of pseudo-static debris loadings and boulder falls, respectively. Preliminary findings by Cheung et al. (2018) also demonstrated that LS-DYNA is a useful tool for modelling the dynamic behavior of flexible barriers. More recently, Lam et al. (2018b) used LS-DYNA to simulate the rocking movement of a free-standing rigid barrier when subjected to a point impact load.

Objectives

To advance our understanding of the performance of flexible and rigid barriers resisting debris flows and boulder impacts respectively, the finite-element program LS-DYNA was used to carry out two numerical investigations at the Geotechnical Engineering Office in Hong Kong.

In the first investigation, the performance of different cushioning materials when subjected to successive boulder impacts was investigated. To this end, a finite-element model was established to simulate a spherical boulder impacting on the deformable cushion layer. The model was calibrated using the results of large-scale experiments conducted using a pendulum impact test facility. With the calibrated model, a parametric study was then carried out by varying the size and the velocity of the impacting boulder. Based on the results, empirical equations were established for use in design.

In the second investigation, finite-element models were built to simulate the interaction between debris flows and steel flexible barriers. In contrast to the conventional decoupled analyses where the debris flow and the behavior of the barrier are considered separately, the present investigation considers the dynamic interaction between the impacting debris flow and the impact-resisting barrier. The simulation results were compared against field measurement results reported in the literature. After calibration, the finite-element models were then used to obtain useful information which cannot be easily obtained by field measurements, such as the debris impact pressure and the energy-dissipation mechanism.

Details of the two numerical investigations are presented in the following sections.

Boulder Impacts on Rigid Barriers Shielded by Cushioning Materials

Physical Tests and Results

A pendulum impact test facility was established to simulate boulder impacts on a rigid barrier. Figure 3 shows the test facility and its key dimensions. As can be seen, the facility comprises a reinforced concrete wall representing a rigid barrier, a concrete sphere representing a boulder, and a steel frame for suspending the sphere. The concrete wall was structurally connected to a shallow footing measuring 11 m long and 5 m wide. The footing was founded at 1.5 m below the ground surface and had a shear key to provide additional resistance against sliding. The wall was sufficiently large to represent a rigid barrier for the purpose of the present study. A 1-m thick cushion layer was constructed on the impact face of the wall. The cushion layer sat on the ground surface and was anchored to the wall around its perimeter by tie rods. The test facility was instrumented by eight strain-gauge load cells on the wall, a single-axis accelerometer at the back of the sphere, a high-speed camera and a video recorder. Figure 4 shows the arrangement of the load cells. Four types of cushioning materials, namely,

rock-filled gabions, recycled glass cullet, cellular glass aggregates, and ethylene-vinyl acetate (EVA) foam were tested. Figure 5 shows the four materials.

To conduct a test, a boulder with a diameter of 1160 mm was lifted up to the required elevation (1.0 m or 3.5 m above the at-rest position which is 2.5 m above ground) by a mobile crane and then released so that it swung down and collided with the cushion layer. After the first impact, the above procedure was repeated so that the boulder hit the cushion layer again until the difference in the measured force at the centre of the wall was less than 20% between successive impacts. For each cushioning material, the tests were conducted at impact velocities (v) of 4.5 m/s and 8.4 m/s. The corresponding impact energy levels are 20 kJ and 70 kJ respectively.

The results of the impact tests have been reported in Ng et al. (2016, 2018b) and Lam et al. (2018a), so that only the results used for the calibration of the numerical models are discussed here. Figure 6 shows the peak boulder impact forces measured during the first impact for the four cushioning materials. The impact forces were calculated by multiplying the measured peak (positive) acceleration of the boulder by the mass of the boulder (2,000 kg). Ideally, the effectiveness of the cushioning materials should be assessed by comparing impact forces with and without the cushioning materials. However, since conducting an impact test on a bare wall would damage the test facility, the performance of the cushioning materials was evaluated by referring to the theoretical impact force between an impacting boulder and concrete. Currently, there are different methods available for the estimation of boulder impact forces acting on concrete and masonry structures (e.g. Schellenberg and Vogel 2009; Mavrouli et al. 2017; Yu et al. 2018). The simplified Hertz equation reported in Kwan (2012) is used in Hong Kong for estimating the peak impact force between a boulder and concrete for

the design of rigid barriers. This equation is dimensionally correct and is derived using typical elastic moduli of boulder (50 GPa) and concrete (25 GPa):

$$(1) \quad F = 4000K_c v^{1.2} r^2$$

where F = boulder impact force (kN)

r = boulder radius (m)

v = boulder impact velocity normal to the barrier (m/s)

K_c = empirical load-reduction factor

According to Hung et al. (1984), the elastic contact forces should be divided by a factor of about 10 to account for contact crushing. Kwan (2012) reviewed the published guidelines and case history data of boulder impacts on structures, and concluded that the value of the load-reduction factor (K_c) in Equation 1 can be conservatively assumed as 0.1 for design purposes though the actual value can vary to some extent. If the value of K_c is taken as 0.1, then Equation 1 effectively becomes $F = 400v^{1.2}r^2$ and the impact forces would be 818 and 1,730 kN for the 20- and 70-kJ impact scenarios respectively if there is no cushion. By comparing these values with the results shown in Figure 6, it can be seen that all the cushioning materials are effective in reducing the peak impact force, which was transmitted to the wall through the cushion layer. If the value of K_c is taken as 0.05, the estimated impact forces for the case of no cushion will be reduced by half to 409 and 865 kN for 20- and 70-kJ impacts, respectively. Still, the effectiveness of the cushioning materials in reducing impact forces is evident. Figure 6 also shows that the performance of EVA foam, cellular glass aggregates, and recycled glass cullet are rather similar, and that rock-filled gabions give slightly higher impact forces for the two energy levels considered. Since the physical tests were conducted only for the case of using a 1160-mm boulder and at velocities of up to

8.4 m/s, the performance of the four materials was evaluated further by numerical modelling for other impact conditions, discussed as follows.

Numerical Investigation and Results

A numerical investigation was carried out to provide insights into the performance of the cushioning materials subjected to impacts by boulders of other sizes and velocities. To achieve this objective, a three-dimensional finite-element model was established using the program LS-DYNA and then calibrated using the available physical test results. A parametric study was then carried out using different boulder sizes and impact velocities. From the results, empirical equations for calculating boulder impact forces acting on cushioning materials were derived.

Setup of the Finite-element Model

The finite-element model consists of a spherical boulder, a concrete wall, and a cushion layer. The boulder was constructed using shell elements following the previous work of Koo et al. (2017b), and both the concrete wall and the cushion layer were constructed using solid elements. The wire rope connecting the boulder was modelled using beam elements. In terms of boundary conditions, the base of wall was fixed in all six degrees of freedom to simulate the rigid foundation support, and no vertical downward movement was permitted for the base of the cushioning layer. In addition, the finite-element nodes around the perimeter of the cushion layer and the wall were merged together to simulate the anchoring of the cushion layer to the wall by tie rods as shown in Figure 5. Figure 7 shows the finite-element model at the start of the simulation. The simulation was initiated by applying Earth's gravity to the model and then releasing the boulder to let it swing down and collide with the cushion layer.

Contact Handling

The automatic surface-to-surface contact function available in LS-DYNA was used to deal with the contacts between different materials. In LS-DYNA, any two parts in contact are defined as a master and a slave. In every time step, checks were carried out on the slave nodes' penetration into the master segment, and similarly the master nodes' penetration into the slave segments. A penalty method was then used to calculate the compression forces between the materials in contact. This method consists of placing normal interface springs between all penetrating nodes and the contact surfaces. The stiffness of the springs are calculated using the Young's moduli and the Poisson's ratios of the materials in contact. Further details of the contact algorithm used in LS-DYNA can be found in LSTC (2017a).

Material Models

The spherical boulder was modelled as a rigid material, and the reinforced concrete wall was modelled using the Continuous Surface Cap Model (MAT_CSCM) which was originally developed by the Federal Highway Administration in the USA for simulating concrete roadside barriers (Murray 2007). This model includes isotropic constitutive equations, yield and hardening surfaces, damage formulations to simulate softening and modulus reduction, and formulation to simulate rate effects. There are 37 input parameters, but for ease of use this model includes initialization routines that provide the user with default input parameters for concrete with compressive strengths ranging between 20 and 58 MPa. For the present study, a compressive strength of 30 MPa was used to generate the input parameters. The reinforcing steel in the concrete was not simulated in the finite-element model.

The Low Density Foam Model (MAT_LOW_DENSITY_FOAM) was used to simulate the EVA foam, which showed negligible permanent deformation in the physical impact tests

(Croop and Lobo 2009, LSTC 2017b). The compressive behavior is governed by a compressive stress vs. volumetric strain curve, while the shear and tensile behavior is elastic and controlled by the Young's modulus. Poisson's ratio is assumed to be zero in this model (Pan et al. 2005). The amount of hysteresis during unloading is controlled by two additional input parameters. Preliminary runs showed that compared to other input parameters, the compressive stress vs. volumetric strain curve governs the overall behavior of the EVA foam when subjected to a boulder impact. This finding is not surprising since the impacting boulder mainly induces a compressive force on the cushion layer. Therefore, during calibration of the input parameters, the stress-strain curve was manually adjusted, and typical or default values were used for other parameters.

The Crushable Foam Model (MAT_CRUSHABLE_FOAM) was used to model rock-filled gabions, recycled glass cullet and cellular glass aggregates since there was a significant amount of non-recoverable deformation after the physical impact tests. This material model requires the input of three basic material parameters (density, Young's modulus, Poisson's ratio) and a compressive stress vs. volumetric strain curve. Unloading is assumed to be elastic and is based on the Young's modulus and Poisson's ratio. As for the case of EVA foam, preliminary analyses showed that the compressive stress vs. volumetric strain curve controls the behavior of these three cushioning materials under a boulder impact. The stress-strain curves of these three materials were manually adjusted during calibration, and typical or default values were used for the other input parameters.

Table 1 summarizes the basic input parameters used in the material models for the cushioning materials, and Figure 8 shows the calibrated compressive stress vs. volumetric strain curves. Note that the compressive stress of the EVA foam is in kilopascal (kPa), while those of the

other three materials are in megapascal (MPa).

Calibration Results

During the calibration process, the compressive stress vs. volumetric curves for the four cushioning materials were manually adjusted to achieve, as far as practicable, an overall match between the measured and simulated boulder accelerations and transmitted loads on the wall for two successive boulder impacts on the cushioning materials. Figures 9 to 12 show the measured and simulated boulder acceleration-time histories for the four cushioning materials during the first impact. Figures 13 to 16 show the simulated and measured transmitted loads to the concrete wall for the first impact. Since some of the instrumentation data for the test conducted on the rock-filled gabions at 70 kJ were not successfully recorded during the experiment, the result of an additional test conducted at 40 kJ was used for the calibration (Figure 13b). Taken together, these results show that the finite-element simulations are generally capable of reproducing the physical impact test results. However, it is also acknowledged that the degree of agreement between the measured and simulated wall loads can be less than satisfactory at places, and this was probably due to the non-ideal contact conditions (e.g. minor gaps) between the cushion layer and the barrier wall in the physical tests. For the case of rock-filled gabions, the discrete nature of the rock fragments inside the gabions also resulted in local force concentrations, and this may explain the larger differences between the simulated and measured wall loads.

Parametric Study Results

After calibration, the numerical models were used to investigate the performance of the cushioning materials when impacted by a boulder. Three different boulder diameters (0.8, 1.2 and 1.6 m) and a range of impact velocities (4.5, 8.4, 12.0, 14.0 and 18.0 m/s) are considered.

Figure 17 presents the simulated peak impact force (F) by plotting F/r^2 against impact velocity (v) for the four cushioning materials. The measured impact forces from the physical impact tests are also shown for comparison. The simplified Hertz equations with empirical coefficients of 100, 150 and 400 are also shown. It can be seen that, for the impact scenarios considered, the equation $F = 100v^{1.2}r^2$ forms an upper bound of all the data points for both rock-filled gabions and recycled glass cullet. This equation may therefore be used to provide a conservative estimate of the peak boulder impact force when these cushioning materials are used. However, for cellular glass aggregates the data points can only be bounded the equation $F = 150v^{1.2}r^2$ and this is due to the more crushable nature of this material thus resulting in higher boulder impact loads for high-energy impacts. It should be noted that the findings presented herein are only valid for a 1-m thick cushion layer. If a thicker layer is used, the degree of load reduction should improve though this effect is beyond the scope of this investigation. Lastly, it can be seen that only a few data points are shown for the EVA foam, and this is because numerical instability occurred in some of the simulations after full compression of the material. Therefore, the EVA foam may be more suitable for resisting low-energy boulder impacts, though it should be emphasized that this finding is valid for foams with a material density of about 120 kg/m^3 . EVA foams with higher material densities are also commercially available.

The maximum penetration of the boulder into the cushion layer has been investigated for rock-filled gabions, cellular glass aggregates and recycled glass cullet. If the cushion layer is idealized as a linear spring and the kinetic energy of the impacting boulder is assumed to be fully absorbed by the cushion layer, then it can be shown that the penetration of the boulder t is proportional to $\phi^{0.8}$ where ϕ is the diameter of the boulder. The derivation of this relationship is given in the Appendix. Figure 18 plots the measured and simulated boulder

penetrations for the three cushioning materials. After the physical tests, the amount of boulder penetration into the cushion layer was estimated from high-speed videos and checked by examining the deformed profile of the cushion layer. Figure 18 shows that, as expected, the overall trend becomes nonlinear as the penetration approaches the cushion thickness of 1000 mm. Empirical equations having the following form have also been added to the plots:

$$(2) \quad t = 1000 \left[1 - \exp \left(-\frac{\phi v^{0.8}}{c} \right) \right]$$

where t = boulder penetration into cushion layer (mm)

ϕ = boulder diameter (m)

v = boulder impact velocity normal to the barrier (m/s)

c = empirical coefficient (dimensionless).

From the results, it would appear that the empirical coefficient, c , may conservatively take a value of 9, 6 and 3 for rock-filled gabions, recycled glass cullet and cellular glass aggregates, respectively, though other suitable values may also be used. By way of example, for a 1-m diameter boulder hitting a 1-m thick cushion layer at a velocity of 10 m/s, the penetration of the boulder will be 504, 651 and 878 mm for rock-filled gabions, recycled glass cullet and cellular glass aggregates respectively if these coefficient values are used. The estimated boulder penetration may be used in conjunction with the recommendations given in the Swiss guideline on rockfall protection galleries (ASTRA 2008) to calculate the degree of load spread through the compressed cushion layer and the impact pressure acting on the barrier wall in a conventional pseudo-static force calculation.

Discussion

The overall performance and other practical considerations (e.g. availability, cost, durability, health and safety) have been considered for the four materials included in this study, and some

of the key advantages and disadvantages of these materials are summarized here. Although the performance of the cellular glass aggregates is good for low-energy impacts, they have been found to give higher boulder impact forces and penetrations than the other cushioning materials for high-energy impacts. EVA foam has been found useful for resisting low-energy impacts only though the performance may improve for foams with a higher material density. Recycled glass cullet, despite its good overall performance, may have health and safety concerns if used in practice, and the long-term durability of the bags containing the glass cullet also becomes an issue. On the other hand, rock-filled gabions have been found to give low boulder impact forces and penetrations. Rock-filled gabions are also widely available and durable, have no health and safety concern, in general, and have already been used for the construction of retaining walls around the world. Therefore, all in all, it would appear that rock-filled gabions are the most promising among the four cushioning materials tested in this study.

Modelling of Debris Flow Impacting on Flexible Barriers

A new numerical technique was implemented in the program LS-DYNA to perform coupled analyses of debris-barrier interaction so as to improve our understanding of this topic. In the context of this paper, a coupled analysis refers to the modelling of debris flow and its dynamic interaction with a steel flexible barrier in the same numerical space and time scale. Two finite-element models were built and verified against instrumented field case histories involving debris flow impacting on flexible barriers: the first case (Case 1) is the Illgraben field test reported by Wendeler et al. (2008) and the second case (Case 2) is the Veltheim field test reported by WSL (2011).

Development of Modelling Method for Coupled Analysis

Modelling of Debris Flow

In the finite-element model, a debris flow was modelled as a continuum using solid elements. To simulate the large deformation of the debris during the flow process, the arbitrary Lagrangian-Eulerian (ALE) formulation was used (Kwan et al. 2015; Koo et al. 2018). ALE is a finite-element formulation in which the computational system is not *a priori* fixed in space or attached to a material. The computational mesh inside the domains can move arbitrarily to optimize the shapes of elements, while the interfaces of the domains can move along with materials to precisely track the interfaces of a multi-material system. In order to capture the position of the debris at each time step, an “ALE container” made up of regular tetrahedral solid elements is built in to cover the possible debris transportation zone.

The internal strength of the debris was described using the elasto-plastic model with an internal friction angle ϕ and a Drucker-Prager yield criterion (Crosta et al. 2003). Since pore water pressure is not explicitly considered, ϕ is a total stress term and was obtained through back-analyses of field tests. This total stress approach significantly reduces the number of modelling parameters required, and is the state-of-the-art engineering practice in post-failure landslide motion simulations (Soga et al., 2018; Koo et al., 2018).

Modelling of Flexible Barrier

A flexible barrier is a metallic netting structure supported by wire rope cables spanning across steel posts, which are usually hinged at their base. All cables and posts are anchored to the ground. Depending on the design, energy-dissipating devices can be installed on the cables to limit the peak loads transferred to other barrier components through large elasto-plastic deformation, thereby absorbing some of the impact energy. For the modelling of steel flexible barriers, it is crucial that the adopted simulation methodology can handle the geometrical and

material nonlinearities of the individual structural components.

In the finite-element model, all structural components were modelled using beam elements. All the elements were modelled as elastic materials except that the steel net and the energy-dissipating devices were assigned with inelastic material models, which plasticity may develop under large deformation demand. In particular, to correctly capture the deformation behavior of a flexible barrier, it is important to explicitly model the deformability of the energy-dissipating devices installed on the cables. To do this, idealized force-elongation curves, such as the one shown in Figure 19, were derived using physical tensile tests and assigned to the beam elements representing the cables equipped with energy-dissipating devices. The stiffness of the cables would then follow the assigned curves until the devices were fully mobilized. Other structural components were modelled as an elastic steel. The interaction between these elements was simulated using either a beam-to-beam contact algorithm or shared nodes. More details about the individual model descriptions are provided in the subsequent sections.

Contact Modelling

The contact between the ALE material and the ground surface (modelled using rigid shell elements which movements were fully constrained) and the flexible barrier was handled using the penalty coupling method (Aquelet et al. 2006; Olovsson and Souli 2000). This method searches for the intersections between the ALE material and the shell elements, and then tracks the independent motions of the contacting elements over a small time step. Any penetration of the ALE material into the shell elements induces a normal interface reaction force, which is distributed evenly to both elements. The magnitude of the force is proportional to the penetration and is calculated using an interface spring stiffness, which is governed by

the Young's moduli of the ALE material and the shell elements. During the simulated debris flow process, the interface shear resistance between the flow and the ground surface was handled using Coulomb's friction law. A basal friction angle ϕ_b of 12° was adopted in the models for the two field cases and this resulted in comparable simulated and measured debris flow velocities and thicknesses. More details are given in the following sections. It is worth noting that this value is within the typical range of ϕ_b (8° to 15°) reported by Koo et al. (2018) for mobile landslides including debris flows.

In the finite-element model, a virtual impervious membrane was applied to the surface of the nets. The membrane was modelled as "null" shell elements acting as an invisible elastic layer so that the debris could not penetrate through as shown in Figure 20. The membrane itself had no stiffness nor strength and was only connected to the nets. Therefore, when the debris hit the membrane, it will transmit all the forces to the nets, which will then deform and mobilize other barrier parts.

Verification of Finite-element Model

Case 1 – The Illgraben Field Test (Wendeler et al. 2008)

Wendeler et al. (2008) described several field experiments involving actual debris flows impacting on steel flexible barriers installed in the Illgraben catchment of Switzerland. The field experiments were fully instrumented. Since the debris flow characteristics and the barrier responses are known for the event that took place on 18 May 2006, this event was chosen for the verification of the finite-element model. In this case, a large amount ($15,000 \text{ m}^3$) of slow-moving debris flow with a frontal velocity of about 2.9 m/s filled up and overtopped a flexible barrier spanning across the debris flow channel. Details of this field experiment can be found in Wendeler et al. (2006, 2008) and are not repeated here.

Descriptions of the Numerical Model

The model was initialized under Earth's gravity. The topography of the debris flow channel was modelled as a slightly inclined trapezoidal track. To simulate a large volume of debris mass running down in a continuous manner, a new technique was used to introduce debris mass into the numerical flume, described as follows. Instead of specifying a finite volume of debris mass, a slice of the debris source (2.5 m high covering the measured debris flow height) was defined at the back end boundary of the flume. By assigning a constant inward movement of the debris source at a constant rate, the continuous in-flow of the debris mass with a velocity comparable with the field measurement was simulated. The flexible barrier, replicating the actual setup, was placed at 10 m from the debris source. Figure 21 shows the finite-element model. Table 2 summarizes the key input parameters for the models of debris material and structural elements of the flexible barrier.

Calibration of the Internal Friction Angle of Debris Flow Material

The strength of the debris flow material was calibrated using the measured cable forces and the barrier deformations as reported by Wendeler et al. (2008). During the calibration process, the internal friction angle (ϕ) of the debris flow material was adjusted so that the simulated cable forces and barrier deformations match the field measurements. This parameter was chosen because it dictates the shear strength and energy dissipation of the debris flow as well as the flow-structure interaction (Ng et al., 2018a; Aaron and Hungr, 2016; Mancarella and Hungr, 2010). However, this parameter cannot be readily determined in the field and has to be back-analyzed (Koo et al., 2018). Table 3 compares the simulated and measured peak cable forces for ϕ ranging from 5° to 45° . As can be seen, the tensile forces in the cables increase with the value of the internal friction angle. When ϕ increases, the debris flow material

becomes more solid-like and induces higher cable forces during the impact. In all cases analyzed, the axial force of the top cable was over-estimated by the numerical simulations. Since the debris was slow-moving, the upper cable force was probably controlled by the static earth pressure of the deposited debris. The over-estimation was probably caused by the use of an impermeable membrane in the numerical model, as this setup resulted in higher static loading acting on the net when the barrier was filled up. During the field tests, water in the debris seeped away during the deposition process and thus the actual static earth pressure was less than the predicted value. For the design of debris-resisting flexible barriers, however, the most critical component is the bottom cable as it is always subjected to the greatest tensile force during impact. As can be seen from Table 3, when ϕ_i was taken as 15° , the tensile force in the bottom cable was over-predicted by about 7% (266 kN simulated vs. 248 kN measured) and this difference was the smallest among all simulations. Table 3 also summarizes the deformation of the barrier in terms of the maximum bulging of the net in the flow direction. It can be seen that the use of an internal friction angle of 15° also resulted in a good match between the simulated and measured net bulging (2.53 m simulated vs. 2.50 m measured).

The interaction between the debris flow and the flexible barrier was also found to be controlled by the internal friction angle. Figure 22a shows the instant of the debris overtopping the flexible barrier for the four cases analyzed. When ϕ_i was taken as 5° , the debris became very mobile and liquid-like when overtopping the barrier. At the other extreme, when ϕ_i was set at 45° , the debris became solid-like and continued to push the barrier upon the impact, thus prolonging the interaction process without resulting in any overflow. The moments before and during debris overflowing the barrier in the real event (TSUS, 2017) are shown in Figure 22b for comparison. It can be observed that the simulation with $\phi_i = 15^\circ$ appears to replicate the actual event reasonably well. Based on the above comparisons, an

internal friction angle of 15° was considered appropriate for use in the finite-element model to simulate the Illgraben field test. It is also worth noting that the debris flow in the field test was described as watery in Wendeler et al. (2008). In fact, Koo et al. (2018) also reported the use of $\phi_1 = 15^\circ$ to model watery debris flows based on back-analyses of mobile landslides.

Analysis of Debris-Barrier Interaction

Based on the above comparison, an internal friction angle of 15° was adopted in the finite-element model simulating the Illgraben field test. Figure 23 shows the debris-barrier interaction process at different times and the velocity profile of the debris flow. It can be seen that the debris flow impacted on the barrier at around 1.8 s. The debris front was then arrested by the deformable net and became deposited behind the bulged barrier. As shown by the velocity vectors at 2.4 s, the deposited debris mass was static although the debris flow kept arriving. The deposited mass thus became a dead zone shielding the barrier from further debris impact. As a result, the remaining debris flow was deflected upwards, overrode the deposited material and then struck the upper portion of the barrier at an attenuated velocity. This process continued until the barrier net became fully deformed and the space behind the net became completely filled with the debris. At 3.7 s, the remaining debris overtopped the barrier at a much reduced velocity. This velocity attenuation behavior is similar to the field and laboratory findings reported by Koo et al. (2017a). Because of the huge amount of debris flow in this case, the dead zone behind the barrier kept enlarging as more debris was arrested (see the instant at 6.0 s).

Case 2 – The Veltheim Field Test (WSL, 2011)

Bugnion and Wendeler (2010) reported several large-scale instrumented field tests to investigate the performance of steel flexible barriers in resisting shallow landslides. The test

conducted on the barrier with a pressure rating of 150 kPa on 25 November 2010 was selected for the present study. It involved four consecutive releases of debris along a 30° and 41-m long channel on a slope. The debris was mixed to a water content of 15% and consisted of a mixture of clay- to cobble-sized particles. The test results have been reported in WSL (2011) so only the relevant information is described in this paper. In the present study, only the first release was modelled in which 50 m³ of debris was released and 28 m³ was arrested by the barrier. The remaining debris was either deposited along the channel or leaked through the barrier net.

Descriptions of the Numerical Model

For the numerical back-analysis of this case, the modelling technique described earlier for Case 1 was employed, except that the ‘dam-break’ modelling technique was used to simulate the release of a finite volume of debris. Here, the debris mass was allowed to run down the channel under Earth’s gravity after the gate was opened. The finite-element model shown in Figure 24 was built to replicate the actual site settings and barrier configurations. To improve the efficiency of the simulation, several simplifications were made to the flexible barrier. For example, the shackles connecting the wire rope cables and the barrier net were not explicitly modelled. To simulate the free sliding movement between these components, their contact was simulated using the “Contact_Guided_Cable” function in LS-DYNA. This function defines a sliding contact which guides the beam elements through a list of nodes. The connection between the cables and the steel posts was also handled in the same manner. It is worth noting that this contact handling method is similar to the moving node mechanism available in the finite-element program NIDA-MNN developed by Chan et al. (2012) and used by Kwan et al. (2014) for simulating the behavior of a flexible barrier subjected to a shallow landslide. As regards the properties of the debris flow material, an internal friction

angle of 15° was assumed. Table 4 summarizes the key input parameters for the models of debris material and structural elements of the flexible barrier.

Analysis of Debris-Barrier Interaction

Figure 25 shows the simulated and measured arrival times of the debris front as well as the debris flow thickness at different chainages along the flume. It can be seen that the actual mobility of the debris was satisfactorily simulated in the finite-element model. Regarding the barrier responses, Table 5 summarizes the simulated and field measurement results in terms of peak cable forces and barrier deformations. It can be seen that a reasonably good match between the simulated and measured results was obtained. Similar to Case 1, the simulated and measured peak forces in the bottom cable were similar (79 kN simulated vs. 71 kN measured), but the finite-element model slightly over-estimated the force in the top cable (63 kN simulated vs. 39 kN measured). The barrier in the finite-element model also deformed more than the actual barrier in the field (max. bulging of net: 2.3 m simulated vs. 1.8 m measured). This can again be attributed to the assumption that the debris was not allowed to penetrate through the model barrier thus exerting a higher static earth pressure on the net. Nevertheless, the simulated debris deposition profile was still comparable with that observed in the field as shown in Figure 26.

Debris Impact Pressure and Energy Dissipation Assessment

Numerical modelling of coupled debris-barrier interaction enables the determination of a spectrum of physical parameters including those critical for design in practice, e.g. the impact pressure and energy. These parameters cannot be readily measured in the field. In the following sections, the calibrated finite-element model built for Case 2 (Veltheim field test) is used to study these physical parameters.

Debris Impact Pressure

With the fluid-structure interaction module in LS-DYNA, useful information about the barrier interacting with the ALE material can be obtained. For examples, the time-histories of the contact force and area between the debris and the barrier during the impact process were recorded in the model. Figure 27a plots the deduced average contact pressure (contact force divided by the contact area) at the deforming debris-barrier interface at different times, and Figure 27b shows the pressure distribution on the net. It can be seen that the maximum impact pressure was about 126 kPa and it occurred at 7.2 s. The pressure was concentrated over the bulged area on which the debris impacted. A few seconds after the initial impact, the contact pressure reduced and remained steady at about 20 kPa. This was due to the static earth pressure exerted by the deposited debris on the barrier.

The deduced contact pressure from the finite-element model can also be used to improve the current design practice. Kwan and Cheung (2012) suggests the use of a hydrodynamic pressure coefficient (α) of 2.0 to calculate the impact pressure of moving debris resisted by a flexible barrier. The impact pressure is assumed to be uniformly distributed across the width and depth of the debris mass. Using the same hydrodynamic equation and the peak average impact pressure of 126 kPa obtained from the simulation (Figure 27a), the value of α is about 0.58 if the field measured debris density of 2,085 kg/m³ and a flow velocity of 10.2 m/s are used in the back-calculations. It is also worth noting that this back-calculated α value lies well within the range (0.4 to 0.8) reported by Bugnion et al. (2012), who analyzed the α value based on field measurements taken. This finding suggests that the design parameter recommended by Kwan and Cheung (2012) may be on the conservative side and there is room for optimizing the design α value for flexible debris-resisting barriers.

Dissipation of Debris Flow Energy

In the LS-DYNA model, a set of energy data relevant to the debris-barrier interaction was obtained. This provided an opportunity to study the impact mechanism from the energy perspective. The interpretation of the energy data obtained from the LS-DYNA model built for the Veltheim field test is discussed as follows. These energy data include: (i) the kinetic energy (KE) due to the debris mass in motion; (ii) the potential energy (PE) due to an elevation change of the debris mass; (iii) the internal energy (IE) due to plastic deformation of the debris mass; (iv) the frictional energy (FE) due to work done against friction between the debris flow and the ground; and (v) the strain energy of the flexible barrier (SE). KE, IE and SE are direct outputs from the finite-element model whereas PE and FE are calculated using the outputs.

In the following, the energy balance of the debris flow from the moment when the debris was released to the moment when the debris front arrived at the barrier location was analyzed. Before the release of the debris, $KE = 0$ kJ and $PE \approx 12,200$ kJ. Note that $PE = W_{\text{debris}} \times H$, where W_{debris} is the weight of the debris and is approximately 580 kN, which is calculated based on a debris volume of 28 m^3 , and H is the height difference of the debris mass to the datum taken as the final debris deposition level and is about 21 m. As the debris flowed down the flume, KE increased and PE reduced. KE and PE were about 4,200 kJ and 3,500 kJ respectively when the debris front just reached the barrier location. This means that the debris had lost about 4,500 kJ after this runout process. Some of this dissipated energy was converted to the internal energy of the debris, IE, which was about 2,400 kJ and also the frictional energy, FE, which was about 2,100 kJ. Note that $FE = W_{\text{debris}} \times \cos \beta \times \tan \phi_b \times L$, where W_{debris} is the weight of the debris (580 kN), β is the slope angle (30°), ϕ_b is the basal

friction angle (12°), and L is the average travel length of the debris mass (≈ 20 m), which was taken as half of the flume length because most of the debris trail remained on the flume even when the debris front firstly arrived at the barrier location. SE remained zero during the runout process because the flexible barrier was not impacted by the debris. As expected, when the debris front reached the barrier location, the total energy was conserved since $KE + PE + IE + FE$ equals the initial energy content of the system which was about 12,200 kJ.

The energy transfer from the debris to the barrier during the impact process was also analyzed and the findings are discussed as follows. In the context of this study, the “impact process” starts from the time when the debris front first arrived at the barrier location to the time when the debris was completely brought to rest. Table 6 summarizes the energy data at these two moments. The data of interest include: (i) the change of debris energy ($KE + PE$) as a result of the impact, and (ii) the strain energy developed in the flexible barrier, i.e. the gain in SE. SE was calculated as the total strain energy of all structural components of the barrier, including the net, cables, energy-dissipating devices and posts. As can be seen in Table 6, the change in the debris energy was about 7,700 kJ whilst the gain in SE over the same period was only about 450 kJ. This means that, in this particular case, only 6% of the debris energy was transferred to the flexible barrier. The rest was dissipated internally within the debris mass and via frictional loss. This observation may be used to optimize the design of flexible debris-resisting barriers using the energy approach in the future (e.g. Kwan & Cheung, 2012). However, it must be emphasized that the findings of the present study are only valid for the particular impact scenario and the barrier type considered. In addition, energy loss through heat and sound was not considered in the finite-element model. Further research, either by numerical modelling or physical testing, will be required to investigate the possible range of energy absorption by different types of flexible barrier under different impact scenarios.

Concluding Remarks

This paper presents the applications of three-dimensional finite-element models for simulating rigid and flexible debris-resisting barriers. In particular, boulders impacting on rigid barriers shielded by different types of cushioning materials and debris flows impacting on steel flexible barriers have been discussed. In each study, the finite-element model was verified and calibrated against actual instrumented field tests. The findings of these studies have enhanced our understanding of the dynamic interaction between barriers and landslides, and can be used to optimize guidelines for the design of debris-resisting barriers in the future. The findings of these two studies are briefly summarized as follows.

In the first study, the potential use of cushioning materials to reduce boulder impact loads on rigid barriers was investigated through large-scale physical experiments and finite-element simulations. Four different types of materials were examined, including rock-filled gabions, recycled glass cullet, cellular glass aggregates, and ethylene-vinyl acetate (EVA) foam. It was found that the use of these materials could provide a means to increase the impact duration and thus reduce the peak boulder impact load on a rigid barrier. A parametric study was carried out using the calibrated finite-element model and empirical correlations were proposed for the prediction of the peak impact load and boulder penetration into the cushion layer. The performance of the cushioning materials could also be evaluated from the energy perspective but this is outside the scope of the present study.

In the second study, coupled finite-element simulations were carried out to simulate the complete process of a debris flow impacting on a steel flexible barrier. By comparing the simulated and measured cable forces and barrier deformations, the internal shear strength

parameter (ϕ) of the debris materials was obtained. The model enabled a detailed investigation of the dynamic impact mechanism. It also provided useful information such as the debris impact pressure and energy transfer during the process of debris-barrier interaction. Since this information cannot be readily measured in field experiments, the finite-element technique opens up a new research path for further investigation of debris-barrier interaction, both from force and energy perspectives. For example, it was deduced from the Illgraben field test model that the hydrodynamic pressure coefficient (α), which is used commonly for predicting dynamic impact pressure of debris flows, was only about 0.58, compared to the current recommended design value of 2.0. Moreover, an analysis of the energy data from the finite-element model developed for the Veltheim field test also revealed that the flexible barrier only absorbed a small amount of the energy from the debris flow, and the remaining energy was lost via internal dissipation and work done through friction.

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Table Captions

Table 1. Summary of key input parameters used in material models for the study of cushioning materials

Table 2. Summary of key input parameters used in material models for Case 1 – Illgraben field test.

Table 3. Effect of internal friction angle of debris material on the simulated peak cable forces and barrier deformations for Case 1 – Illgraben field test.

Table 4. Summary of key input parameters used in material models for Case 2 – Veltheim field test.

Table 5. Comparison of measured and simulated peak cable forces, barrier deformations and thickness of deposited debris for Case 2 – Veltheim field test.

Table 6. Summary of debris energy and strain energy of flexible barrier before and after impact for Case 2 – Veltheim field test.

Figure Captions

Figure 1. Reinforced concrete rigid barrier with a cushion layer.

Figure 2. Steel flexible barrier.

Figure 3. Large-scale pendulum impact test facility: (a) photograph; (b) schematic drawing with key dimensions.

Figure 4. Load-cell layout on the barrier wall (from Ng et al. 2016).

Figure 5. Cushioning materials: (a) rock-filled gabions; (b) EVA foam; (c) bulk bags and gabion baskets containing recycled glass cullet or cellular glass aggregates; (d) recycled glass cullet; (e) a cellular glass aggregate.

Figure 6. Summary of measured peak boulder impact forces.

Figure 7. Finite-element model used for simulating boulder impact on a cushion layer.

Figure 8. Calibrated compressive stress vs. volumetric strain curves.

Figure 9. Measured and simulated acceleration-time histories of a boulder impacting on rock-filled gabions.

Figure 10. Measured and simulated acceleration-time histories of a boulder impacting on cellular glass aggregates.

Figure 11. Measured and simulated acceleration-time histories of a boulder impacting on recycled glass cullet.

Figure 12. Measured and simulated acceleration-time histories of a boulder impacting on EVA foam.

Figure 13. Measured and simulated transmitted loads to the wall shielded by rock-filled gabions: a) 20 kJ; b) 40 kJ.

Figure 14. Measured and simulated transmitted loads to the wall shielded by cellular glass aggregates: a) 20 kJ; b) 70 kJ.

Figure 15. Measured and simulated transmitted loads to the wall shielded by recycled glass cullet, a) 20 kJ; b) 70 kJ.

Figure 16. Measured and simulated transmitted loads to the wall shielded by EVA foam: a) 20 kJ; 70 kJ.

Figure 17. Measured and simulated peak impact forces: (a) rock-filled gabions; (b) cellular glass aggregates; (c) recycled glass cullet; (d) EVA foam.

Figure 18. Measured and simulated boulder penetration into the cushion layer: (a) rock-filled gabions; (b) cellular glass aggregates; (c) recycled glass cullet.

Figure 19. Idealized force-elongation curve of an energy-dissipating device.

Figure 20. “Null” shell membrane covering up the ring net for retention of the debris and transmission of forces to the ring net.

Figure 21. Finite-element model for the Illgraben field test: (a) the flume; (b) the flexible barrier.

Figure 22. Comparison of the debris-barrier interaction at the instant of debris overflow: (a) finite-element models using different internal friction angles ϕ_i ; (b) field observations (from TSUS, 2017).

Figure 23. Simulated process of debris-barrier interaction with velocity vectors for $\phi_i = 15^\circ$.

Figure 24. Finite-element model for the Veltheim field test: (a) the flume; (b) the flexible barrier.

Figure 25. Measured and simulated arrival times of debris front and debris thickness for the Veltheim field test.

Figure 26. Deformed barrier shape: (a) finite-element model; (b) field test (from WSL 2011).

Figure 27. Contact pressure between debris and barrier net: (a) time history of the deduced average contact pressure; (b) pressure contour and interaction at the instant of peak contact pressure at 7.2 s.

Table 1. Summary of key input parameters used in material models for the study of cushioning materials.

Cushioning Material	Material model	Compressive stress vs. volumetric strain curve	Density (kg/m ³)	Young's modulus (MPa)	Poisson's ratio	Other parameters
Rock-filled gabions	MAT_CRUSHABLE_FOAM	Figure 8	1560	50	0.3	Damping coefficient = 0.1 (default)
Recycled glass cullet	MAT_CRUSHABLE_FOAM		1500	30	0.3	Damping coefficient = 0.1 (default)
Cellular glass aggregates	MAT_CRUSHABLE_FOAM		110	10	0.3	Damping coefficient = 0.1 (default)
EVA foam	MAT_LOW_DENSITY_FOAM		120	2	0	Damping coefficient = 0.05 (efault) Hysteretic unloading factor (HU) = 1.0 Shape factor for unloading (SHAPE) = 1.0

Table 2. Summary of key input parameters used in material models for Case 1 – Illgraben field test.

Materials	Material model	Input parameters	Remarks
Debris	MAT_SOIL_AND_FOAM	Density = 1,620 kg/m ³	- based on field measurement
		Shear modulus = 500 kPa	- assumed values as these parameters are not critical to debris flow behaviour
		Bulk modulus = 1,000 kPa	- sufficiently low values for better numerical efficiency
		Internal friction angle = 15°	- calibrated parameter
		Basal friction angle = 12°	
Ring net	MAT_HYSTERETIC_BEAM	Density = 7,800 kg/m ³	- based on actual material properties
		Young's modulus = 200 GPa	
		Poisson's ratio = 0.3	
Cable	MAT_ELASTIC	Density = 7,800 kg/m ³	
		Young's modulus = 100 GPa	
		Poisson's ratio = 0.3	
Energy-dissipating device	MAT_HYSTERETIC_BEAM	Density = 7,800 kg/m ³	
		Young's modulus = 100 GPa	
		Poisson's ratio = 0.3	
		Force-elongation curve: $x_1 = 0.65 \text{ m}, y_1 = 112 \text{ kN}$ $x_2 = 0.90 \text{ m}, y_2 = 144 \text{ kN}$ $x_3 = 1.10 \text{ m}, y_3 = 192 \text{ kN}$	- See Figure 19

Table 3. Effect of internal friction angle of debris material on the simulated peak cable forces and barrier deformations for Case 1 – Illgraben field test.

Items	Field measurement	Finite-element model			
		$\phi_1 = 5^\circ$	$\phi_1 = 15^\circ$	$\phi_1 = 25^\circ$	$\phi_1 = 45^\circ$
Peak bottom cable force (kN)	248	277 (+12%)	266 (+7%)	304 (+23%)	561 (+126%)
Peak top cable force (kN)	150	186 (+24%)	201 (+34%)	232 (+55%)	384 (+156%)
Maximum bulging of net (m)	2.50	2.42 (-3.2%)	2.53 (+1.2%)	3.07 (+22.8%)	4.7 (+88.0%)
Note: 1. For the case of $\phi_1 = 45^\circ$, the values shown were peak cable forces attained prior to numerical instability which took place when the debris filled up the barrier. 2. Percentage difference between the results of field measurements and finite-element model is shown in bracket.					

Table 4. Summary of key input parameters used in material models for Case 2 – Veltheim field test.

Materials	Material model	Input parameters	Remarks
Debris	MAT_SOIL_AND_FOAM	Density = 2,085 kg/m ³	- based on field measurement
		Shear modulus = 500 kPa	- assumed values as these parameters are not critical to debris flow behaviour
		Bulk modulus = 1,000 kPa	- sufficiently low values for better numerical efficiency
		Internal friction angle = 15°	- calibrated parameter
		Basal friction angle = 12°	
Steel net	MAT_HYSTERETIC_BEAM	Density = 7,800 kg/m ³	- based on actual material properties
		Young's modulus = 200 GPa	
		Poisson's ratio = 0.3	
Cable	MAT_ELASTIC	Density = 7,800 kg/m ³	
		Young's modulus = 125 GPa	
		Poisson's ratio = 0.3	
Energy-dissipating device	MAT_HYSTERETIC_BEAM	Density = 7,800 kg/m ³	
		Young's modulus = 125 GPa	
		Poisson's ratio = 0.3	
		Force-elongation curve: $x_1 = 1.00 \text{ m}, y_1 = 130 \text{ kN}$ $x_2 = 1.15 \text{ m}, y_2 = 220 \text{ kN}$	- See Figure 19

Table 5. Comparison of measured and simulated peak cable forces, barrier deformations and thickness of deposited debris for Case 2 – Veltheim field test.

Force / deformation	Field measurement	Finite-element model
Peak bottom cable force	71 kN	79 kN
Peak top cable force	39 kN	63 kN
Peak upslope cable force	42 kN	46 kN
Maximum bulging of net	1.8 m	2.3 m
Maximum sagging of top rope	0.4 m	0.9 m
Maximum thickness of debris deposited behind barrier	1.5 m	1.6 m

Table 6. Summary of debris energy and strain energy of flexible barrier before and after impact for Case 2 – Veltheim field test.

Energy (kJ) ⁽¹⁾		Before impact - Debris front arriving at the barrier	After impact - Debris completely deposited behind the barrier	Change due to the impact ⁽²⁾
Debris energy = KE + PE	Kinetic energy (KE) of debris	4,200	0	-4,200
	Potential energy (PE) of debris	3,500	0	-3,500
Strain energy of flexible barrier (SE)		0	450	+450
Note: (1) Energy data shown are rounded off values. (2) Negative values indicate a loss of energy whereas positive values indicate a energy gain.				

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Figure 1. Reinforced concrete rigid barrier with a cushion layer.

Figure 2. Steel flexible barrier.

(a)

(b)

Figure 3. Large-scale pendulum impact test facility: (a) photograph; (b) schematic drawing with key dimensions.

297x420mm (300 x 300 DPI)

Figure 4. Load-cell layout on the barrier wall (from Ng et al. 2016).

Figure 5. Cushioning materials: (a) rock-filled gabions; (b) EVA foam; (c) bulk bags and gabion baskets containing recycled glass cullet or cellular glass aggregates; (d) recycled glass cullet; (e) a cellular glass aggregate.

297x420mm (300 x 300 DPI)

Figure 6. Summary of measured peak boulder impact forces.

87x56mm (300 x 300 DPI)

Time = 0 s

Figure 7. Finite-element model used for simulating boulder impact on a cushion layer.

192x160mm (300 x 300 DPI)

Figure 8. Calibrated compressive stress vs. volumetric strain curves.

118x83mm (300 x 300 DPI)

Figure 9. Measured and simulated acceleration-time histories of a boulder impacting on rock-filled gabions.

118x83mm (300 x 300 DPI)

Figure 10. Measured and simulated acceleration-time histories of a boulder impacting on cellular glass aggregates.

118x83mm (300 x 300 DPI)

Figure 11. Measured and simulated acceleration-time histories of a boulder impacting on recycled glass cullet.

118x83mm (300 x 300 DPI)

Figure 12. Measured and simulated acceleration-time histories of a boulder impacting on EVA foam.

118x83mm (300 x 300 DPI)

Figure 13. Measured and simulated transmitted loads to the wall shielded by rock-filled gabions: a) 20 kJ; b) 40 kJ.

297x420mm (300 x 300 DPI)

Figure 14. Measured and simulated transmitted loads to the wall shielded by cellular glass aggregates: a) 20 kJ; b) 70 kJ.

297x420mm (300 x 300 DPI)

Figure 15. Measured and simulated transmitted loads to the wall shielded by recycled glass cullet: a) 20 kJ; b) 70 kJ.

297x420mm (300 x 300 DPI)

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297x420mm (300 x 300 DPI)

Figure 18. Measured and simulated boulder penetration into the cushion layer: (a) rock-filled gabions; (b) cellular glass aggregates; (c) recycled glass cullet.

297x420mm (300 x 300 DPI)

Figure 19. Idealized force-elongation curve of an energy-dissipating device.

99x61mm (300 x 300 DPI)

Figure 20. "Null" shell membrane covering up the ring net for retention of the debris and transmission of forces to the ring net.

270x140mm (300 x 300 DPI)

Figure 21. Finite-element model for the Illgraben field test: (a) the flume; (b) the flexible barrier.

297x420mm (300 x 300 DPI)

Figure 22. Comparison of the debris-barrier interaction at the instant of debris overflow: (a) finite-element models using different internal friction angles ϕ_i ; (b) field observations (from TSUS, 2017).

297x420mm (300 x 300 DPI)

Figure 23. Simulated process of debris-barrier interaction with velocity vectors for $\phi_i = 15^\circ$.

229x301mm (300 x 300 DPI)

(a)

Height = 3.5 m
Length = 15 m (5 m per net panel)
Boundary conditions:
- ends of cables and posts are pin-ended and restrained from displacements
- ends of posts are allowed to rotate about X-axis only

(b)

Figure 24. Finite-element model for the Veltheim field test: (a) the flume; (b) the flexible barrier.

297x420mm (300 x 300 DPI)

Figure 25. Measured and simulated arrival times of debris front and debris thickness for the Veltheim field test.

125x85mm (300 x 300 DPI)

(a)

(b)

Figure 26. Deformed barrier shape: (a) finite-element model; (b) field test (from WSL 2011).

297x420mm (300 x 300 DPI)

Figure 27. Contact pressure between debris and barrier net: (a) time history of the deduced average contact pressure; (b) pressure contour and interaction at the instant of peak contact pressure at 7.2 s.

297x420mm (300 x 300 DPI)

Appendix

If the cushion layer is idealized as a linear elastic spring and if the kinetic energy of the impacting boulder is assumed to be fully absorbed by the cushion layer, then the following equation can be written:

$$(A1) \quad \frac{1}{2} Ft = \frac{1}{2} mv^2$$

where F is the impact force (spring force), t is the boulder penetration into the cushion layer (spring compression), m is the mass of the boulder and v is the impact velocity of the boulder. Since it has been shown in Kwan (2012) that F is proportional to $v^{1.2}r^2$ (Eq. 1), substituting this relationship into Eq. (A1) and expressing m in terms of the boulder diameter ϕ thus gives:

$$(A2) \quad t \propto \phi^{0.8}$$

Eq. (A2) shows that t is proportional to $\phi^{0.8}$, though it is appreciated that the actual relationship is also influenced by factors such as the thickness of the cushion layer, the elasto-plastic load-deformation behaviour of the cushioning materials, and energy loss in other forms during the impact process.

References

Kwan, J.S.H. 2012. Supplementary technical guidance on design of rigid debris-resisting barriers, GEO Report No. 270, Geotechnical Engineering Office, Hong Kong SAR Governemnt, 91 p.