The Effects of Upstream Flexible Barrier on the Debris Flow Entrainment
and Impact Dynamics on a Terminal Barrier

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Abstract: The destructive nature of debris flows is mainly caused by flow bulking from entrainment of an erodible channel bed. To arrest these flows, multiple flexible barriers are commonly installed along the predicted flow path. Despite the importance of an erodible bed, its effects are generally ignored when designing barriers. In this study, three unique experiments were carried out in a 28 m-long flume to investigate the impact of a debris flow on both single and dual flexible barriers installed in a channel with a 6 m-long erodible soil bed. Initial debris volumes of 2.5 m$^3$ and 6 m$^3$ were modelled. For the test setting adopted, a small upstream flexible barrier before the erodible bed separates the flow into several surges via overflow. The smaller surges reduce bed entrainment by 70% and impact force on the terminal barrier by 94% compared to the case without an upstream flexible barrier. However, debris overflowing the deformed flexible upstream barrier induces a centrifugal force that results in a dynamic pressure coefficient that is up to 2.2 times higher than those recommended in guidelines. This suggests that although compact upstream flexible barriers can be effective for controlling bed entrainment, they should be carefully designed to withstand higher impact forces.

Keywords: landslide; debris flow; flexible barrier; entrainment; flume
1. Introduction

Debris flows may increase in scale by entraining bed material along the flow path (Hung et al. 2005; Iverson and Ouyang 2015). Entrainment occurs when a debris flow shears a soil bed (Medina et al. 2008; Iverson 2012; Iverson and Ouyang 2015) and generates excess pore pressures, which reduces the shear resistance of the bed (Hung et al. 2005; Iverson et al. 2011). Over recent decades, flexible barriers have been increasingly used to mitigate debris flow hazards (Wendeler et al. 2008). One of the design functions of installing multiple flexible barriers in a channel is to reduce bed entrainment. However, there is a dearth of literature on the use of flexible barriers for controlling channel bed entrainment. Instead, the emphasis of the existing work in the literature on impact dynamics is placed on non-erodible beds (Ng et al. 2016; Song et al. 2017). To improve mitigation measures for controlling entrainment, the effects of flow-bed-barrier interaction need to be elucidated.

Some research work has been carried out to reveal the interactions between multiple barriers installed in a channel (Kwan et al. 2015; Ng et al. 2019). Ng et al. (2020) proposed and evaluated an analytical framework for the design of multiple rigid barriers on non-erodible beds. They concluded that multiple barriers enable the progressive reduction in debris flow volume and dissipation of energy during successive sequences of impacts, overflow, and landing. The reduced velocities and volume after interacting with each barrier results in lower impact forces exerted on downstream barriers. However, the aforementioned work focused on the impact dynamics of rigid barriers installed in channels with non-erodible beds. The deformation of a flexible barrier attenuates the impact force and alter overflow dynamics (Ng et al. 2020), which can affect the entrainment process between barriers.

Shen et al. (2019) carried out numerical simulations using a depth-averaged model to investigate the influence of a series of successive rigid check dams on reducing entrainment. Their results showed that a series of dams can minimise flow bulking by creating a cascading effect. However, check dams are fundamentally different compared to flexible barriers. Rigid check dams are designed to impound static material behind them and the filled dams are not cleaned. Instead, a series of filled check dams serve to stabilise the terrain to minimise further erosion. In contrast, flexible barriers are designed to take both dynamic and static impact loading, and they need to be cleaned after each event. Also, flexible barriers deform during impact and are expected to reduce in height after impact, thereby altering
overflow dynamics. With the emergence of flexible barriers for debris flow mitigation, the fundamental mechanisms of interaction between a debris flow and flexible barriers on erodible beds is an important topic that requires further investigation.

The main scientific challenges that have limited progression towards a rational basis for leveraging flexible barriers to control channel bed entrainment includes: (i) the poor temporal predictability of natural debris flows, which has hindered our understanding of the fundamental flow-bed-barrier mechanisms of interactions; and (ii) the non-linear dynamic scaling of debris flows caused by the interplay between the solid and fluid phases (Iverson 2015). The nonlinear scaling necessitates the largest testing facilities feasible. In this study, a series of unique flume tests were carried out in a 28 m-long flume to model up to 6 m$^3$ of initial debris material impacting single and dual flexible barriers installed in a channel with an erodible bed.

2. Flume experiments

2.1. Model setup

As aforementioned, the largest test facilities feasible are required to address the scaling issues pertaining to the timescale for pore pressure diffusion (Iverson 2015). The scale of the flume experiments in this study are among the largest of its kind in the world. Figure 1a shows a cross-section of the 28 m-long flume (Ng et al. 2019) that was used to carry out experiments in this study. The channel has a rectangular cross-section with a width of 2 m and depth of 1 m. The side walls are transparent on one side of the channel to enable the flow and impact kinematics to be observed. A storage container, that is 5 m in length, is inclined at 30° at the upper end of the channel. The container can store up to 10 m$^3$ of debris material. A double gate system is used to retain the debris material inside the storage container. The gate is secured and released by a mechanical arm that is controlled by an electric motor. The main channel is 15 m long and is inclined at 20°. An erodible bed with a thickness of 120 mm was prepared along the last 6 m of the inclined section. The erodible bed is prepared on top of a 5 mm thick metallic mesh to increase the channel bed roughness to hold the erodible bed in place. A rigid platform with an inclined length of 2 m was used to allow the debris flow to smoothly transition from the fixed bed to the erodible one. The transition consists of two parts. The first
part forms an angle of $7^\circ$ with respect to the inclined channel for an inclined distance of 1 m. The second part is 120 mm in thickness and runs parallel to the inclined channel. At the end of the inclined section is a horizontal 4.4 m runout section.

Figure 2a shows the upstream flexible barrier, which is installed orthogonally to the channel bed just 4.3 m downstream from the gate. The barrier is 0.6 m in height and 2 m in width. The primary ring of the barrier net consists of 100 mm diameter rings made using 4 windings of high yield strength steel wires that are 2 mm in diameter. The primary ring net is suspended by the top and bottom cables. The ring net is laterally anchored to the side of the flume using two smaller cables. The top and bottom load bearing steel cables with 12 mm diameter are equipped with brake elements to replicate the load-displacement behaviour of prototype barriers (Ng et al. 2016). The brake elements simplify the loading response of a prototype barrier using a bi-linear curve (Ng et al. 2020). In addition to the primary ring net, a secondary mesh with 25 mm square openings made of stainless-steel wire with a diameter of 1 mm are connected to the main cables to retain the debris material.

The terminal flexible barrier is installed at the end of the runout section (Fig. 2b). The terminal barrier is 1.5 m in height and 4 m in width. The barrier is supported by three load bearing steel cables with 16 mm diameter, which are connected to a steel frame. Braking devices are installed on each cable. The primary ring net consists of 200 mm diameter rings made using 4 windings of steel wires that are 2 mm in diameter. The secondary mesh has 25 mm square openings made of steel wires that are 1 mm in diameter.

### 2.2. Instrumentation

A laser sensor, U1, and an ultrasonic sensor, U2, (model: Keyence IL600/IL1000 and Banner TUB30X) were mounted above the centreline of the flume bed to measure the flow depth at inclined distances of 3.4 m and 12.5 m from the gate, respectively (Fig. 1a). The resolution of the sensors is $\pm 1$ mm. An unmanned aerial vehicle with an on-board video camera (model: UAV DJI Phantom 3) was used to capture aerial videography of each test. By knowing the distance between reference lines marked on the flume bed and the time between frames, the frontal velocity could be calculated. High-speed cameras (model: Mikotron EoSens 4CXP) were also used to capture the kinematics of the
debris flow, at 300 fps and with a resolution of 2336x1712 px, over the erodible bed and the impact dynamics on the flexible barriers. Basal instrumentation cells were installed along the flume bed to measure the normal and shear stresses, and changes in pore pressures at the base of the flow. Details of the instrumentation cell is discussed in Ng et al. (2019). Figure 1b shows the cross-section of the instrumentation Cell 1 located directly underneath depth sensor U1. Cell 1 is instrumented to simultaneously measure the normal and shear stresses at the base of the flow using a triaxial load cell (model: sensor ME K3D160). The triaxial load cell is rigidly connected to a force plate roughened by epoxy and sand. Cell 2 is located underneath depth sensor U2, which is instrumented to measure the normal stress and pore pressures at the base of the erodible bed. Henceforth in the text, the location of 3.4 m and 12.5 m from the gate is represented by L1 and L2 respectively. L1 is located prior to the upstream flexible barrier and L2 is located in the erodible bed section (Fig. 1a).

The cables of the flexible barriers were equipped with tension load cells to measure the cable forces. For both top and bottom cables of the upstream flexible barrier, a thru-hole load cell is used (model: Omega LSHD-50k). The load cell is fixed to the wall of the flume and measures the horizontal component of the cable force ($T_H$). For the terminal flexible barrier, the top and middle cables are also connected to a thru-hole load cell (model: TML TCLK-50 kNA), which is fixed to the frame of the terminal flexible barrier to measure the horizontal component of the cable force ($T_H$). As for the bottom cable of the terminal barrier, an in-line load cell is used (model: TML TCLP-100kNB). The in-line load cell deflects in the cable direction and therefore measures the cable force ($T$). Figure 2c shows the top view of a deflected cable of a flexible barrier. The impact force ($F$) is orthogonal to the barrier. The measurements of the cable loads ($T$ and $T_H$) and deflection angles ($\psi$) can be used to deduce the impact force ($F$) (Ng et al. 2016).

For the top and bottom cables of the upstream barrier and for the top and middle cables of the terminal barrier that use thru-hole load cells to measure $T_H$, the impact force $F$ can be calculated as follows:

$$F = 2T_H\tan\psi$$

while, for the bottom cable of the terminal barrier that uses in-line load cells that measure $T$, the impact force $F$ can be calculated as follows:
A data logger (model: NI cDAQ-9137) with a sampling rate of 2 kHz was used to record the measurements of Cell 1, Cell 2, U1, U2 and of the cable forces.

\[ F = 2T \sin \psi \]

2.3. Entrainment depth measurement

A unique method of erosion columns (Fig. 3), similar to that proposed by Berger et al. (2011), was used to differentiate deposition from entrainment. This allowed the entrainment depth to be measured. The columns were installed in the erodible bed prior to its construction. A total of 5 rows of erosion columns were installed at increments of 1 m along the centreline of the channel. Each erosion column consists of 21 nuts. A single nut is 5 mm in height and has an inner diameter of 6 mm. To align the erosion column each nut is threaded through a bolt with a diameter of 4 mm. Thus, the accuracy of this approach is equivalent to the thickness of a nut (± 5 mm). The bolt is initially fixed to a plate attached to the base of the channel (Fig. 3a). The soil bed is then prepared around the erosion columns (Fig. 3b). Afterwards, the bolts are removed, and the nuts are free to be entrained with the erodible bed (Fig. 3c). After each test, the difference between the initial number and the remaining number of nuts are counted to determine the entrainment depth (Fig. 3d). The deposition depth is also measured at the end of each test. The deposition depths are distinguished as the height of deposited material above the level of the remaining nuts.

2.4. Test program

Three experiments were carried out in the 28 m-long long flume. In each test an erodible bed was prepared at the same location along the inclined section. The first two tests were conducted using initial volumes of 2.5 m\(^3\) and 6 m\(^3\). In both of these tests, only a terminal flexible barrier was installed at the end of the runout zone. In the third experiment, an initial volume of 6 m\(^3\) was used and an upstream flexible barrier was installed before the erodible bed at an inclined distance of 4.3 m from the gate (Fig. 1a). This position of the upstream flexible barrier should be installed to avoid overflow from directly landing on the erodible bed. Otherwise, scour may occur and undermine the stability.
and serviceability of the flexible barrier foundation. In the field, the channel bed may be protected from scour using geotextiles (e.g. Feiger and Wendeler 2019). A terminal barrier was also installed at the end of the runout zone for this third test. Table 1 gives a summary of the test program.

2.5. Modelling procedure

The debris flow material consists of a sand-gravel-clay mixture (Ng et al. 2019). The mixture comprises of 36% gravel (2 mm - 20 mm), 61% sand (0.075 mm - 2 mm), and 3% fines (<0.075 mm) by mass. A solid concentration, $C_s$, of 70% was selected for the tests. The initial density of the debris mixture was 2155 kg/m$^3$. For the erodible bed, a sand-gravel-clay mixture was also used. However, the composition differed slightly from the debris material. The bed material consists of sand (0.075 mm - 2 mm, 63%), fine gravel (2 mm - 5 mm, 33%) and fines (< 0.075 mm, 4%).

The initial water content and void ratio of the erodible bed play an important role in the entrainment dynamics (Iverson et al. 2011). Excess pore pressures in the bed material can develop only if the water content is high enough to create continuous water networks to transmit pore pressures (Iverson et al. 2011). The target gravimetric water content was selected to be 15% because it avoided pre-mature bed failure, which would occur for high degree of saturation.

Before each experiment, the bed material was mixed to achieve a gravimetric water content of 15%. The mixture was then placed on the 6 m-long erodible bed section. After bed preparation, soil samples were obtained from the bed to measure the gravimetric water content, which was approximately 15%, and electrical capacitance sensors (model: Decagon EC-5) were used to measure the volumetric water content, which was about 25%. From these water content measurements, the void ratio was calculated to be about 0.6. Afterwards, the gate was secured, and the storage tank was prepared with the debris mixture. The gates were then released to initiate dam-break.

3. Interpretation of test results

3.1. Observed flow and impact kinematics
Figure 4 shows the typical kinematics for upstream flexible barrier for test V6-B2 where an upstream barrier is installed before the erodible bed and a terminal barrier is installed at the end of the runout zone. The interaction between the flow and the upstream flexible barrier is viewed through the transparent sidewall. The flume inclination of 20° is indicated in the Fig. 4a as a reference. The time when the flow front impacts the upstream barrier is taken as $t = 0$ s (Fig. 4a). At $t = 0.12$ s (Fig. 4b), the top and bottom cables are deformed while the subsequent flow material is accumulated. The retained material, called a dead zone herein, forms a curved ramp for incoming flow to override. The curvature of the ramp is likely due to the deformation of the barrier. In contrast, a similar curvature is not reported for the dead zone forming behind a rigid barrier (Ng et al. 2019). The flow on top of the dead zone runs-up and resembles ($t = 0.35$ s) the vertical jet mechanism reported by Choi et al. (2015) for viscous flows or fast flows (Faug 2021). In contrast, a pile-up mechanism is typically observed for slow moving flows (Wendeler et al. 2019). The jet eventually reaches its maximum height (Fig. 4c) and overflows the barrier at $t = 0.95$ s (Fig. 4d). This overflowing material forms the first surge that reaches the erodible bed after the upstream barrier. At the same time incoming flow is observed to follow a curvilinear trajectory as it overrides the dead zone. As the supply of debris material from the storage container diminishes, the launch angle of the overflow from the barrier crest changes from an upward angle to a downward angle towards channel bed (Fig. 4e) which forms the second surge that flows downstream. Finally, the debris material comes to rest behind the upstream barrier at $t = 4.30$ s to form a final horizontal deposit (Fig. 4f).

The overflow was observed to form two distinct surges separated by about 1 s in duration. The first surge lands at an inclined distance of 4 m downstream from the upstream barrier. The second surge lands directly on top of the debris material on the flume bed from the first surge and is observed to decelerate. Figure 5 shows the impact kinematics on the terminal barrier for a volume of 6 m$^3$ impacting a single terminal barrier (test V6-B1) compared to a dual barrier system (test V6-B2). The volume and impact velocity of the flow that the terminal barrier in test V6-B1 arrests is larger and higher, respectively, than that for the terminal barrier in test V6-B2. Therefore, a distinct run-up mechanism is observed on the terminal barrier for test V6-B1 compared to a pileup mechanism for the terminal barrier in test V6-B2. Details on the measured impact forces are discussed later.

3.2. Flow depth and frontal velocity
Figure 6 shows the evolution of the measured flow depths at inclined distances of 3.4 m (L1) and 12.5 m (L2) from the gate measured using U1 and U2, respectively. The flow depth measurements for the test V2.5-B1, test V6-B1 and test V6-B2 are compared. The $t = 0$ s in the abscissa shows the time that the flow front reaches the respective locations at L1 and L2. At L1 (Fig. 6a), the effects of scale on the flow depths are evident. More specifically, the maximum measured flow depth for the test with a smaller initial volume of 2.5 m$^3$ (test V2.5-B1) is about two times less than that for the larger volume of 6 m$^3$ (tests V6-B1 and V6-B2). Larger flow depths generate higher normal bed stress and have larger timescales for pore pressure diffusion (Iverson 2015). Furthermore, the flow depth profiles for tests V6-B1 and V6-B2 are initially similar in magnitude until impact occurs on the upstream barrier in test V6-B2. In test V6-B2, the presence of an upstream flexible barrier causes the flow depth to increase after $t = 2$ s as debris is arrested and accumulates behind the upstream barrier. At L2 (Fig. 6b), the measured flow depths for all three tests have noticeably decreased compared to those measured at L1 as the debris mass spreads along the channel (Ng et al. 2013). Moreover, a noticeable difference between the flow depth profiles due to scale is observed between the test conducted using the smaller initial volume of 2.5 m$^3$ (test V2.5-B1) and the larger initial volume of 6 m$^3$ (tests V6-B1 and V6-B2). The flow depth measurements for $t > 3$ s indicate the final change in the bed height, which decreased due to entrainment but increased due to deposition of debris material.

Figure 7 shows a comparison of the measured frontal flow velocities ($v_f$) along the channel for the three tests. The frontal flow velocities are measured using the video recordings from the top view of the flume during the flow propagation. For reference, theoretical velocity profiles calculated for a rigid block is shown. Based on the conservation of energy, the velocity of a rigid block can be calculated as:

$$v_b(x) = \sqrt{2g(x \sin (\theta) + h_0 - x \cos (\theta)\bar{\mu}_{bot})}$$

where $g$ is the acceleration due to gravity, $x$ is the distance along the flume from the gate, $\theta$ is the slope angle, $h_0 = 1.13$ m is the initial height of the center of mass of the flow and $\bar{\mu}_{bot}$ is the apparent friction coefficient at the base of the block. A frictionless block corresponds to $\bar{\mu}_{bot} = 0$, which as expected, overestimates the flow velocities. The measured debris flow front in each test accelerates over the fixed bed section and decelerates over the erodible bed because of a higher basal friction. Generally, the velocity increase is governed by the conversion of potential energy
into kinetic energy (Sassa 1988). Thus, the frontal velocities in tests V6-B1 and V6-B2 are higher than in test V2.5-B1, because of the larger initial volume, which results in a higher potential energy when the debris mass is released. In reality, friction is dissipating energy at the base of the flow to decelerate the flow. Thus, a frictional block that considers energy dissipation (third term on the right hand side of eq. (3)), would provide a more realistic comparison with the measured flow velocities. If an apparent basal friction coefficient of $\mu_{fb,bot} = 0.16$ is assumed, then the calculated and measured velocities for test V6-B1 over the fixed bed section show reasonable agreement. An explanation for the choice of this value of $\mu_{fb,bot}$ will be discussed later when examining the role of basal stresses on the entrainment dynamics.

3.3. Measured entrainment

Figure 8 shows the measured entrainment depths using the erosion columns placed at the centreline of the channel for the three tests. The entrainment magnitudes range from 5 mm to 105 mm (maximum depth that can be measured using erosion columns). For each experiment, the maximum entrained depth is observed near the start of the bed and the entrained depth generally decreases along the flow direction of the erodible bed. A similar entrainment pattern was observed in experimental results reported by de Haas and van Woerkom (2016), which adopted a similar test configuration to study entrainment. One possible explanation for the maximum entrainment occurring near the start of the erodible bed is because the thickness of the erodible bed decreases during entrainment, while the height of the rigid platform remains unchanged, which causes the flow to dig into the erodible bed.

The entrained depths over the bed length in test V6-B2 are lower than the entrained depths of test V2.5-B1 and test V6-B1. The total entrainment volume can be calculated by integrating the measured entrainment depths along the length of the erodible bed. As expected, the debris flow with an initial volume of 6 m$^3$ entrained a much higher volume (test V6-B1, measured entrained volume 0.9 m$^3$) compared to the test with an initial volume of 2.5 m$^3$ (test V2.5-B1, 0.5 m$^3$). In test V6-B2, the upstream flexible barrier retained a part of the initial volume (approximately 1.2 m$^3$) and the measured entrainment volume decreased to 0.3 m$^3$. By partially arresting the debris flow volume, flow bulking is reduced. Furthermore, deposition on the erodible bed is also observed for each test. The maximum deposition heights of 50 mm, 90 mm, and 130 mm were measured for test V2.5-B1, test V6-B1, and test V6-B2,
respectively. The higher deposition measured on the erodible bed in test V6-B2 indicates that the influence of the upstream flexible barrier has altered the flow and impact dynamics.

3.4. Flow basal stresses

Figures 9a and 9b show the time-histories of the basal normal stress ($\sigma_{f,\text{bot}}$) and shear stress ($\tau_{f,\text{bot}}$) measured by Cell 1 for tests V6-B1 and V6-B2, respectively. For test V2.5-B1, the instrumentation in Cell 1 malfunctioned. Therefore, no basal stress measurements are shown for this test. The flow depths measured by U1, which is mounted directly above Cell 1 are also shown for reference. In test V6-B1, the measured peak normal stress is 9.5 kPa, which occurs at about the same time as the peak flow depth, which is 0.4 m. The normal stress, $\sigma_{f,\text{bot}}$, can be compared to estimated theoretical value by assuming static equilibrium perpendicular to the channel bed and neglecting bed-normal accelerations (McArdell et al. 2007; Iverson et al. 2010):

$$\sigma_{f,\text{bot}} \ (\text{calc}) = \rho g h f \cos \theta$$

where $h_f$ is the flow depth. The bulk density of the flow can be taken as $\rho = \rho_s C_s + \rho_w (1 - C_s) = 2155 \, \text{kg/m}^3$ where density of the solid grain, $\rho_s$, is 2650 kg/m$^3$, density of water, $\rho_w$, is 1000 kg/m$^3$, and solid concentration, $C_s$, is 0.7. Based on eq. (4), the calculated normal stress is $\sigma_{f,\text{bot}} \ (\text{calc}) \approx 20 h_f$ kPa. It is worthwhile to note that in test V6-B1 the normal stress generally follows the same trend as the flow depth. Any differences in trend are likely due to not considering flow acceleration in the bed-normal direction, which may induce normal stresses that are higher than those under static conditions. Similarly, Iverson et al. (2010) reported higher measured normal stress on a smooth bed compared to calculated ones using eq. (4).

More interestingly, the maximum flow depths for tests V6-B1 and V6-B2 did not differ significantly. However, it is observed that in test V6-B2 (Fig. 9b) the measured normal stress reaches a peak value of 18.9 kPa, which is almost two times that measured in test V6-B1. The higher normal stress measured in test V6-B2 was caused by the presence of the upstream flexible barrier. After the flow impacts the upstream barrier, debris deposits accumulate on Cell 1 (Fig. 4d). The deposits form a curved ramp due to the deformation of the barrier to enable...
subsequent debris material to overflow the upstream barrier following a curvilinear trajectory. The flow on top of the ramp-like deposit is curvilinear, of which the centrifugal component transmitted to the channel bed and the barrier. This component can be calculated as proposed by Hungr (1995) as follows:

\[
\sigma_c = \frac{\rho h L v^2}{R}
\]

where \( h \) is the depth of the flow layer over the retained material (\( \sim 0.2 \) m), \( v \) is the flow velocity of the layer travelling towards the flexible barrier (\( \sim 5 \) m/s), and \( R \) is the radius of curvature (\( \sim 0.9 \) m), which is approximated from the high-speed camera images (Fig. 4). The centrifugal stress from eq. (5) is estimated as 12 kPa. This centrifugal component can be added to the calculated static basal stress \( \sigma_{\text{bot (calc)}} \approx 8 \) kPa from eq. (4) to obtain a similar value as the maximum measured normal stress in test V6-B2.

In test V6-B1, the peak shear stress occurs at \( t = 0.11 \) s, which is earlier than the peak normal stress at \( t = 0.59 \) s. A similar result is also observed in test V6-B2. The delay between the measured shear and normal stresses can be explained by an unsaturated flow front characterized by low pore pressures. Similar observations were reported by Iverson et al. (2010).

McArdell et al. (2007) and Berger et al. (2011) measured the flow basal shear, normal stress and pore pressure of natural debris flows in the field (Illgraben channel, Switzerland) and McArdell et al. (2007) showed the shear stress at the base of the flow using a Mohr-Coulomb relationship. Iverson et al. (2010) also measured the basal normal stress and pore pressures and modelled the flow basal shear stress, assuming a frictional flow and also adopted a Mohr-Coulomb relationship. Estimates of the Savage and Friction numbers (Iverson 1997) for the flows modelled in this study (refer to online Supplementary material, Table S1) suggest that collisional and viscous stresses are less important than the frictional stresses. The Savage number varies between \( 6 \times 10^{-5} \) and \( 4 \times 10^{-3} \), which suggests frictional stresses to be dominant over collisional ones, as defined by the threshold of 0.1 reported by Savage and Hutter (1989). Furthermore, we find the Friction number of the flows in this study to vary between \( 8 \times 10^4 \) and \( 1 \times 10^6 \), which is higher.
than the threshold for friction stresses to be dominant over viscous ones (1400 as defined by Iverson and LaHusen 1993). Assuming frictional behaviour is an appropriate idealisation, the basal shear stress can be expressed as follows:

\[
\tau_{\text{bot}} = \sigma'_{\text{f,bot}} \mu' = (\sigma_{\text{f,bot}} - p_{\text{b,bot}}) \mu' = \sigma_{\text{f,bot}} (1 - \lambda_{\text{f,bot}}) \mu'
\]

where \( \sigma'_{\text{f,bot}} \) is the basal effective stress normal to the bed, \( p_{\text{b,bot}} \) is the flow basal pore pressure, \( \mu' \) is the effective friction coefficient of the debris material. \( \lambda_{\text{f,bot}} \) is the flow basal pore pressure ratio, which indicates the degree of flow liquefaction: a value of 0 characterises a dry flow, while a value of 1 characterises a liquefied flow. The flow basal pore pressure ratio has a significant influence on the debris flow mobility (Iverson et al. 2010) and bed entrainment (Iverson 2012). The basal pore pressure ratio can be calculated using eq. (6), and measurements of the normal and shear stresses and the effective friction coefficient of the soil, which was measured as \( \mu' = 0.89 \) using direct shear test in the laboratory.

Figure 10 shows the calculated flow basal pore pressure ratio for tests V6-B1 and V6-B2 at L1. The flow basal pore pressure ratio gradually increases from 0.2 to about 0.9. McArdell et al. (2007) reported a flow basal pore pressure ratio of 0.8. Moreover, Iverson et al. (2010) reported a flow basal pore pressure ratio of 0.6 for sand-gravel flow mixtures and 1.0 for sand-gravel-mud mixtures. For both flow mixtures, the flow front was reported to be unsaturated with low excess pore pressures. The influence of the flow basal pore pressure ratio on flow mobility can be observed by considering the frictional block model discussed in Fig. 7. The average value of the basal pore pressure ratio, \( \bar{\lambda}_{\text{b,bot}} \), is 0.82. Consequently, an average value of the apparent friction coefficient at the base of the flow can be defined as \( \bar{\mu}_{\text{b,bot}} = (1 - \bar{\lambda}_{\text{b,bot}}) \mu' = 0.16 \). This value was used previously to describe the behaviour of the frictional block model using eq. (3). The high measured flow velocities (Fig. 7) likely resulted from a high degree of flow fluidisation, which significantly decreased the basal apparent friction. This analysis shows that perhaps an idealised frictional rheology may provide a first order approximation of debris flow mobility on non-erodible beds, but may present difficulties in capturing flow mobility on erodible beds.

### 3.5. Entrainment rate
Figure 11 shows a comparison of the back-calculated entrainment rates at 12.5 m downstream from the gate (L2) for each test. The entrainment rate is the volume transfer per unit area at the flow-bed interface. The back-calculation method is described in this section. The total entrainment depth, $z_b$, can be calculated by integrating the entrainment rate with time:

$$|z_b|_{(calc)} = \int_0^{t_{DF}} E \, dt$$

where $t_{DF}$ is the debris flow duration and $E$ is the entrainment rate. The entrainment rate can be expressed as follows (Iverson 2012, refer to online Supplementary material, Fig. S1):

$$E(t) = 2gh(t)\cos \mu \frac{\lambda_{b,top}(t) - \lambda_{b,bot}(t)}{v_f}$$

where $\lambda_{b,top} = p_{b,top}/\sigma_{f,bot}$ is the pore pressure ratio at the top of the erodible bed, assuming $\sigma_{f,bot} = \sigma_{b,top}$. Equation (8) is used to calculate the entrainment depth in eq. (7) using measurements obtained at 12.5 m downstream from the gate (L2), notably the flow depth, the frontal flow velocity and the flow basal pore pressure ratio. It is worthwhile to note that, the flow basal pore pressure changes as the flow travels from L1 to L2. However, for simplicity, the value of $\lambda_{b,top}$ is taken at L1 and is assumed to also be applicable at L2. To account for the different flow duration at L2 ($t_{L2} \approx 3 \text{ s}$) compared to the flow duration at L1 ($t_{L1} \approx 2 \text{ s}$), we stretch the time axis of $\lambda_{f,bot}$ (Fig. 10), by a factor of 1.5, so that the maximum time becomes 3 s instead of 2 s. By doing so, the measurement of $\lambda_{f,bot}$ can be used in eq. (8) for the calculation at L2.

Obtaining $\lambda_{b,top}(t)$ would require multiple pore pressure and normal stress sensors placed at different depths in the erodible bed (e.g. McCoy et al. 2011). Given the difficulty of predicting or measuring the evolution of the bed pore pressure, Medina et al. (2008) and Shen et al. (2020) adopt a constant value of $\lambda_{b,top}$, which is back-calculated – through a trial-error approach – to make the computed entrainment converge to the observed entrainment measurements. Similarly, this approach was adopted by considering a constant value in time of $\lambda_{b,top}$. The magnitude of $\lambda_{b,top}$ for the three tests is unknown a priori but is back-calculated using eq. (8) to match the calculated entrainment...
depth (eq. (7)) with the measured entrainment depth at L2. Figure 11 shows the entrainment rates calculated by using eq. (8) with $\lambda_{b,\text{top}} = 0.84$ for test V2.5-B1, $\lambda_{b,\text{top}} = 0.87$ for test V6-B1 and $\lambda_{b,\text{top}} = 0.74$ for test V6-B2 at 12.5 m downstream from the gate (L2). Indeed, assuming these particular values of the bed pore pressure ratio allows to match the theoretical entrainment depths (eq. (7) and the area under the entrainment rate curve for test V6-B1) with the measured entrainment depth at L2.

The difference in pore pressure generated at the base of the flow and the erodible bed governs entrainment (Iverson 2012). The volumetric water content of the erodible bed in this study was 25%. As aforementioned, bed pore pressures can only develop if the water content is high enough to create continuous water networks that are capable of transmitting pore pressures (Iverson et al. 2011). For example, in the experiments of Iverson et al. (2011), positive bed pore pressures close to liquefaction ($\lambda_{b,\text{top}} \approx 1$) were reported for volumetric water contents higher than 22%. The rapid loading applied on the erodible bed by the debris flow may have generated undrained conditions, which rapidly increase pore pressures in the bed. The undrained loading causes the bed pore pressures to be proportional to the normal stress (compression), implying that it may be reasonable to assume a constant value of $\lambda_{b,\text{top}}$. Furthermore, the shear stress transmitted by the flow to the erodible bed may cause shear deformation (Iverson et al. 2011), which may cause contractive behaviour of the loose bed (Iverson 2012) and generate excess pore pressures. Notwithstanding, it may still be reasonable to assume that the generation of pore pressures via shearing should be more significant compared to generation of pore pressure by compression of voids of the granular bed material, given the low excess pore pressure measured (approximately 1.4 kPa) at the base of the bed (Cell 2) compared to a theoretical normal stress of 7 kPa (eq. (4)). Pore pressures were likely only increasing locally at the sheared interface between the entrained and the stationary bed.

The calculated entrainment rate in test V6-B1 reaches a maximum value of 0.13 m/s. In contrast, the entrainment rate reaches a maximum value of 0.04 m/s in tests V2.5-B1 and V6-B2. The calculated entrainment rates are consistent with the fact that in test V6-B1 led to the highest entrainment depth. Entrainment of the bed occurred within 0.4 s for test V6-B2, 1 s for test V2.5-B1 and 1.4 s for test V6-B1 upon arrival of the flow front. The entrainment rate then becomes 0 or negative, which indicates deposition. The high positive entrainment rates up to $t = 1$ s imply that entrainment is dominant at the flow front, especially when the difference between $\lambda_{b,\text{top}}$ and $\lambda_{f,\text{bot}}$ is the highest.
In fact, unsaturated flow front, characterized by low values of $\lambda_{\text{f,bot}}$, transmits higher basal shear stress and can entrain more bed material compared to the liquefied flow body. Higher entrainment at the flow front was also reported by Berger et al. (2011) in field measurements and Iverson et al. (2011) in their flume experiments. The rate of entrainment depends on the driving stresses caused by the flow and the resisting strength of the bed material. The driving stresses may be collisional or frictional in nature depending on the particle sizes involved (Song and Choi 2021). The resisting stress depends on the shear strength of the soil material involved and its water content (Song and Choi 2021). Also, the driving stresses may vary spatially and temporally within a debris flow. As such, the entrainment rate is case dependent and is expected to differ from event to event. Furthermore, deposition also occurs concurrently with entrainment. The entrainment depths in nature are often masked by the deposition so the elevation of steep creeks and riverbeds may even be higher than before a debris flow event.

The higher entrainment in test V6-B1, compared to test V2.5-B1, is mainly influenced by the larger flow depths recorded in test V6-B1. The larger flow depths generate higher shear stress at the flow bed interface. It can be concluded that the initial flow volume has a significant influence on the flow depth and therefore on the entrainment magnitude. As expected, the entrainment process is scale-dependent. In test V6-B2, despite a higher value of the flow depth compared to test V2.5-B1, the entrainment depth is the lowest compared to the other two tests. In test V6-B2, the bed pore pressure ratio, $\lambda_{\text{b,top}}$, is consistently lower than in test V6-B1, implying that only the first part of the flow, up to $t = 0.4$ s, entrains. In fact, the upstream flexible barrier changes the flow behaviour downstream, especially by reducing entrainment. Based on the two distinct overflow surges observed in test V6-B2, only the first overflow surge generated significant shear on the bed capable of increasing bed pore pressures and entraining bed material. In contrast, the second surge only flowed over the deposited material from the first surge and decelerated as it sheared the slower-moving debris material.

3.6. Impact dynamics

Figure 12 shows the measured cable force time histories for the top, middle and bottom cables of the terminal barrier ($T$ for the bottom cable and $T_{\text{H}}$ for the middle and top cables) for the three tests. The $t = 0$ s indicates the time the flow front reaches the terminal barrier. The y-axis shows the cable force in kN while the x-axis shows the elapsed time.
time in seconds. In general, for all three tests the peak cable force is reached within \( t = 1 \) s and gradually the loading attenuates to a residual static state. In comparison of the 3 cable force time histories, the bottom cable attains a higher peak and final force. Test V6-B1 shows the highest cable forces on the terminal barrier, while the magnitude of the cable forces in tests V2.5-B1 and V6-B2 are similar and lower than those measured in test V6-B1. A summary of the measured deflection angles of the cables are shown in Table 2. The deflection angle is measured at the end of the test. On the terminal barrier, the deflection angle is higher for the bottom and middle cables since more loading is induced on them. The deflection angles on the terminal barrier are also much higher in test V6-B1 compared to test V6-B2, because the impact load was larger, and the cables deflected more.

The measured peak cable forces (\( T_{1i} \) and \( T \)) (Fig. 12) and the measured cable deflection angles (\( \psi \)) (Table 2) are used to calculate the impact force (\( F \)) on each cable based on eq. (1) and eq. (2). Consequently, the total impact force (\( F_T \)) on the barrier is derived as the summation of the top, middle and bottom cable impact force (\( F \)). The total peak impact forces, \( F_T \), for the terminal barrier in tests V6-B1 and V6-B2 are 55 kN and 3 kN, respectively (Table 2). For comparison, the total peak impact force on the upstream barrier in test V6-B2 (113 kN) is also shown.

The total peak impact force is used by engineers for the design of flexible barriers to resist the debris flow impact. This peak impact force is generally estimated using the hydrodynamic equation (Kwan and Cheung 2012):

\[
F_T = \alpha \rho \bar{v}^2 h w
\]

where \( \alpha \) is a dynamic pressure coefficient, \( \bar{v} \) and \( h \) are the flow velocity and depth before impact (Table 2) and \( w \) is the width of the flow before impact (2 m). Wendeler et al. (2019) reports a value of \( \alpha \) equal to 1.0 for debris flows impacting a flexible barrier in the field, but a range of values from 1.0 to 5.0 can be found in literature (Poudyal et al. 2019). The dynamic pressure coefficient \( \alpha \) can be back-calculated as 2.2 for the upstream barrier in test V6-B2 and 1.2 and 0.2 for the terminal barrier in tests V6-B1 and V6-B2 respectively. The dynamic pressure coefficient is not calculated for test V2.5-B1 because the load cell for the middle cable malfunctioned during test.
The value of $\alpha$ is higher for the upstream barrier (2.2) in test V6-B2 compared to that of the terminal barrier (1.2) for test with only the terminal barrier (test V6-B1). The high value of $\alpha$ at the upstream barrier may have been caused by the centrifugal component of the force (Hungr 1995) from the curved layer flowing over the dead zone (Fig. 4d) and drag from overflow (Wendeler et al. 2019). Furthermore, the small barrier to flow depth ratio (1.5) may have limited energy dissipation internally in the flow, with higher force transferred to the upstream barrier.

More importantly, the value of $\alpha$ on the terminal barrier decreases significantly when the upstream flexible barrier is used (test V6-B2 compared to test V6-B1). A rational explanation for this reduction can be found by analysing the impact process on the terminal flexible barrier (Fig. 5). In test V6-B1, a higher volume (5.7 m$^3$) impacted the barrier with a higher flow velocity (6.1 m/s). A run-up impact mechanism therefore developed (Fig. 5a), which mobilised all three cables in the barrier. In contrast, in test V6-B2, the volume impacting the terminal barrier was much smaller and limited to the second surge (2.6 m$^3$), which was flowing over the deposit from the first surge. Furthermore, the velocity impacting the terminal barrier in test V6-B2 had decreased (4 m/s) due to the upstream barrier. The impact on the terminal barrier was therefore a pileup mechanism (Fig. 5b).

4. Conclusions

A series of unique physical experiments were conducted in a 28 m-long flume to study the impact of a debris flow on both single and dual flexible barriers installed in a channel with a 6 m-long erodible soil bed. Two different initial volumes of 2.5 m$^3$ and 6 m$^3$ were investigated. Based on the experimental results, the following conclusions for the settings adopted in this study may be drawn as follows:

a) A compact upstream flexible barrier (1.5$h$) with a design capacity of only 20% of the initial debris volume installed before an erodible bed reduces the peak discharge by creating several smaller overflowing surges. The formation of surges when interacting with the upstream barrier decreases entrainment by 70% and the impact force on the terminal barrier by 94% compared to the case without an upstream barrier. The design dynamic pressure coefficient $\alpha$ for the terminal barrier decreases from 1.2 to 0.2 in presence of the additional
upstream barrier. This suggests that installing a compact upstream flexible barrier can lead to design optimisation for the terminal barrier.

b) Although the upstream barrier was successful in reducing entrainment and the impact force induced on the terminal barrier, the deformation of the upstream barrier upon impact formed a curved ramp-like deposit for subsequent flow to induce centrifugal force on the barrier. This barrier configuration unexpectedly led to a back-calculated dynamic pressure coefficient of 2.2, which is much higher than the recommended 1.5 in design guidelines. This finding suggests that the maximum deformation and orientation of a flexible barrier with respect to the channel requires further consideration as the overflow trajectory differs compared to rigid barriers.

c) Experimental results show that debris flow-bed-barrier interaction is a complex problem driven by the scale of the event. This is supported by the differences in flow depths, frontal velocities, entrainment depths and rates, and impact forces between the 2.5 m$^3$ and 6 m$^3$ tests. Furthermore, we show that not only the scale is important in determining the entrainment volume and impact forces, but the upstream barrier can reduce entrainment and the impact force on the terminal barrier. Evidently, debris flows in nature are much larger in scale and occur in successive surges. Further investigation of the spacing between barriers, barrier deformation, barrier height and slope are required to move towards a rational basis to leverage multiple flexible barriers to reduce the scale of and mitigate a debris flow event.
Author statements

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Competing interests

The authors declare there are no competing interests.

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**Data availability**

All data are available upon reasonable request.
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Table captions

**Table 1.** Test program.

**Table 2.** Measured and calculated parameters for the tests with an initial volume of 6 m$^3$ impacting the upstream and terminal barriers.
Figure captions

**Fig. 1.** (a) Cross-section of the 28 m-long flume and instrumentation layout (U refers to ultrasonic sensors; E refers to the erosion column; Cell 1 measures simultaneously normal and shear stress; and Cell 2 measures normal stress and pore pressure; L1 and L2 refer to the locations 3.4 m and 12.5 m from the gate). (b) Cross-section of Cell 1 (triaxial cell).

**Fig. 2.** (a) Upstream flexible barrier (looking downstream in the channel). (b) Terminal flexible barrier (looking upstream in the channel). (c) Top view of a deflected flexible barrier cable and calculation of the impact force ($F$) from the measurements of the cable force ($T$), horizontal force ($T_H$) and deflection angle of the cable ($\psi$).

**Fig. 3.** Side schematic showing working principle of an erosion column: (a) initial placement of the erosion column; (b) erosion column installed inside erodible bed with bolt still in place; (c) erosion column inside erodible bed with bolt removed; (d) side schematic of the erosion column after each test, with erodible bed material and nuts entrained and debris material deposited on top.

**Fig. 4.** Impact kinematics for upstream flexible barrier (Test V6-B2): (a) $t = 0$ s; (b) $t = 0.12$ s; (c) $t = 0.35$ s; (d) $t = 0.95$ s; (e) $t = 1.50$ s; (f) $t = 4.30$ s.

**Fig. 5.** Observed impact mechanism from front of terminal barrier: (a) run-up impact mechanism for test V6-B1; and (b) pile-up impact mechanism for test V6-B2.

**Fig. 6.** Flow depth measurements: (a) 3.4 m downstream from the gate (L1); (b) 12.5 m downstream from the gate (L2).

**Fig. 7.** Comparison of frontal flow velocity (velocities for Test V6-B2 not shown in the middle because of the presence of the upstream barrier). The erodible bed section (from 9 m to 15 m) is indicated.
Fig. 8. Measured entrainment depths along the centreline of the erodible bed.

Fig. 9. Basal stress and flow depth measurements 3.4 m downstream from the gate (L1): (a) test V6-B1; and (b) test V6-B2.

Fig. 10. Calculated flow basal pore pressure ratio for test V6-B1 and test V6-B2, 3.4 m downstream from the gate (L1).

Fig. 11. Comparison of back-calculated entrainment rate 12.5 m downstream from the gate (L2).

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| Test ID | Released flow volume (m³) | Barrier configuration          |
|---------|---------------------------|--------------------------------|
| V2.5-B1 | 2.5                       | Single flexible barrier (terminal) |
| V6-B1   | 6                         |                                |
| V6-B2   | 6                         | Dual flexible barriers (upstream + terminal) |

### Table 2. Measured and calculated parameters for the tests with an initial volume of 6 m³ impacting the upstream and terminal barriers.

| Test ID           | Deflection angle bottom cable | Deflection angle middle cable | Deflection angle top cable | Total peak impact force, $F_T$ (kN) | Flow velocity (m/s) | Flow depth (m) | Back-calculated dynamic pressure coefficient $\alpha$ |
|-------------------|-------------------------------|-------------------------------|---------------------------|--------------------------------------|---------------------|---------------|-----------------------------------------------|
| V6-B1 (terminal barrier) | 20°                           | 17°                          | 4°                        | 55                                   | 6.1                 | 0.3           | 1.2                                           |
| V6-B2 (terminal barrier) | 8°                            | 3°                           | 2°                        | 3                                    | 4*                  | 0.2*          | 0.2                                           |
| V6-B2 (upstream barrier) | 15°                           | /                            | 15°                       | 113                                  | 5.5                 | 0.4           | 2.2                                           |

* denotes second surge
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