Supplementary Technical Guidance on Design of Rigid Debris-resisting Barriers

GEO Report No. 270

J.S.H. Kwan

Geotechnical Engineering Office
Civil Engineering and Development Department
The Government of the Hong Kong
Special Administrative Region

Supplementary Technical Guidance on Design of Rigid Debris-resisting Barriers

GEO Report No. 270

J.S.H. Kwan

This report was originally produced in July 2012 as GEO Technical Note No. TN 2/2012

First published, August 2012

Prepared by:

Geotechnical Engineering Office,
Civil Engineering and Development Department,
Civil Engineering and Development Building,
101 Princess Margaret Road,
Homantin, Kowloon,
Hong Kong.

Table of Contents
Table of Contents
Table of Contents
Table of Contents

Preface

In keeping with our policy of releasing information which may be of general interest to the geotechnical profession and the public, we make available selected internal reports in a series of publications termed the GEO Report series. The GEO Reports can be downloaded from the website of the Civil Engineering and Development Department (http://www.cedd.gov.hk) on the Internet. Printed copies are also available for some GEO Reports. For printed copies, a charge is made to cover the cost of printing.

The Geotechnical Engineering Office also produces documents specifically for publication in print. These include guidance documents and results of comprehensive reviews. They can also be downloaded from the above website.

These publications and the printed GEO Reports may be obtained from the Government's Information Services Department. Information on how to purchase these documents is given on the second last page of this report.

Y.C. Chan
Head, Geotechnical Engineering Office
August 2012

Table of Contents Table of Contents Table of Contents Table of Contents

Foreword

This report presents supplementary guidelines on the design of rigid debris-resisting barriers with reference to a review of the state-of-the-art literature and international technical guidance documents. This report supplements and updates the guidance given in GEO Report No. 104.

The work was carried out by Dr J.S.H. Kwan initially under the supervision of Mr K.K.S. Ho and later under my supervision. Mr L.K.W. Shum took part in the project at the early stage. Dr M.H.C. Chan carried out dimensional analysis of the Hertz Equation. The worked example for the design of a rigid barrier following the supplementary guidelines was prepared and checked by Mr K.W.Y. Shum and Mr R.P.H. Law respectively. Ms C.S.K. Yau and Mr R.C.H. Koo assisted in preparing the report.

Draft versions of the report were reviewed by Professor O. Hungr of the University of British Columbia, Canada, who was engaged as an independent expert reviewer of the study. Many other colleagues in the GEO and practitioners in the geotechnical industry provided constructive comments on the formulation of the supplementary guidelines. All contributions are gratefully acknowledged.

Y.K. Shiu
Chief Geotechnical Engineer/Standards & Testing

Table of Contents
Table of Contents
Table of Contents
Table of Contents

Abstract

Rigid debris-resting barrier is one of the common types of mitigation measure to handle natural terrain hazards. GEO Report No. 104 "Review of Natural Terrain Landslide Debris-resisting Barrier Design" provides recommended guidance on the design of the rigid debris-resisting barriers. Some local practitioners also make reference to the design considerations and procedures presented in GEO Report No. 174 "Design Basis for Standardised Modules of Landslide Debris-resisting Barriers" for barrier design.

Since the publication of the above two reports, much experience has accrued in respect of the use of the above technical guidance and several overseas guidance documents have been published. A review to consolidate the state-of-the-art knowledge as presented in international journal papers and overseas design guidelines has therefore been carried out, and design guidance to supplement the current design guidelines is proposed.

This report documents the review and the recommended supplementary design guidance pertinent to (a) the design impact scenarios required, (b) estimation of boulder impact load, (c) calculation of debris dynamic impact pressure, and (d) determination of height of barriers.

Table of Contents
Table of Contents
Table of Contents
Table of Contents

Contents

List of Tables

Table of Contents
Table of Contents
Table of Contents

List of Figures

Table of Contents
Table of Contents
Table of Contents

1 Introduction

Rigid debris-resisting barrier is one of the commonly used defensive measures to mitigate natural terrain landslide hazards. Guidance and information pertinent to the design of rigid debris-resisting barriers are presented in Lo (2000) and Sun et al (2005).

Lo (2000) sets out the geotechnical parameters and considerations for the design of rigid debris-resisting barriers based on a review of the relevant literature and state of knowledge as of the late 1990s, and Sun et al (2005) proposed design of standardised debris-resisting barriers, and presents sample calculations of rigid barriers.

Since the publication of Lo (2000) and Sun et al (2005), local experience of use of the above technical guidance has built up over the years and several overseas guidance documents on the design of debris-resisting barriers have been published. These overseas guidance documents include:

(a) Austrian Standards (ASI, 2008; ASI, 2011):

(i) ONR 24801 (Draft) Protection Works for Torrent Control - Actions on Structures by Austrian Standard Institute, Austria.

(ii) ONR 24802 Protection Works for Torrent Control - Design of Structures by Austrian Standard Institute, Austria.

(b) Chinese Standard (MLR, 2006):

Specification of Geological Investigation for Debris Flow Stabilisation by the Ministry of Land and Resources, China.

(c) Japan Technical Standard (NILIM, 2007):

Manual of Technical Standard for Designing Sabo Facilities against Debris Flow and Driftwood by the National Institute for Land and Infrastructure Management, Japan.

(d) Taiwanese Technical Manual (SWCB, 2005):

Manual of Soil and Water Conservation by the Soil and Water Conservation Bureau, Taiwan.

This report provides a review of Lo (2000) and Sun et al (2005), and consolidates the state-of-the-art knowledge as presented in international papers and the above overseas design guidance documents. Recommended guidance to supplement the current local design practice pertinent to the design impact scenarios required, estimation of boulder impact load, calculation of debris dynamic impact pressure, and determination of height of barriers is also proposed.

Table of Contents Table of Contents Table of Contents Table of Contents

Professor Oldrich Hungr of the University of British Columbia, Canada was engaged as an independent expert reviewer to review draft versions of this report. Professor Hungr's comments and responses to his comments are documented in Appendix A.

2 Design Impact Scenarios

Debris flow is a two-phase material comprising a mixture of debris and water. If boulders (e.g. perched along streamcourses or debris flow path, or embedded within the failed ground mass) are mobilised, they would be engulfed to form part of the debris flow. The boulders usually travel near the top of debris flows due to vertical segregation of materials of different grain size that occurs during the debris transportation (Fabio, 2011). Very often, debris flow appears as an "elongated continuum" travelling along natural streamcourses, and rigid barriers may be subjected to pulses of impact in the form of surges. VanDine (1996) notes that the impact load of each surge would decrease once debris deposition begins. This could be due to the fact that debris at the front usually travels at a higher velocity, and that the debris of the frontal surge deposited behind the barrier could provide a cushioning effect on the trailing debris.

Lo (2000) describes the design impact scenarios that should be considered, i.e. the design should cater for the first frontal impact as well as the subsequent impacts. In addition, impact load arising from boulders within debris should also be considered, which is generally consistent with the international practices reviewed. The magnitude of the two dynamic loadings arising from boulder impact and debris impact respectively may be estimated separately based on pseudo-static approaches. This approach is also adopted by ASI (2011).

NILIM (2007) recommends that the design load should be taken as the superposition of the boulder and debris impact loads for the design of the upper portion of debris barrier. However, SWCB (2005) suggests that barrier stability should be checked by considering debris flow impact load and boulder impact load under separate design scenarios. For example, in the sample design calculation presented in SWCB (2005), the stability of a check dam is evaluated against (i) impact of debris flow without boulders, and (ii) impact of a single boulder hitting the top of the check dam without the need to consider the debris flow impact load. According to ASI (2008), boulder impact load should be considered in assessing structural damage of barrier. However, MLR (2006) does not specify how the above two respective impact loads should be assessed.

Some practitioners have opted to follow the sample calculations by Sun et al (2005) to carry out the rigid barrier design, in which the design is checked against an impact scenario whereby the design debris volume is doubled. Under this "doubled volume" scenario, a displacement of the barrier by a distance of up to 1.5 m is allowed. It should be noted that Sun et al (2005) are concerned with the design of standardised barriers which has been developed to cover different site settings. If the design debris volume has been estimated rigorously based on the guidelines given in Ng et al (2003), checking against this "doubled volume" scenario is normally not necessary.

Table of Contents Table of Contents Table of Contents Table of Contents

2.1 Recommended Design Impact Scenarios

The recommended design impact scenarios are formulated based on consideration of various design practices to supplement the guidelines stated in Sections 6.4 and 6.5 of Lo (2000). The recommendations are applicable to the design of rigid barriers resisting debris of open hillslope failure and channelised debris flow. Discussion of the appropriate factors of safety to be adopted in the design scenarios are given in Section 5. Details of the recommendations are elaborated as follows:

(a) Since the debris may hit the barrier in the form of pulses, multiple surges should be considered in checking barrier stability as illustrated in Figure 2.1. The impact load calculated for the first surge is recommended to be applied in all the subsequent surges for the sake of enhanced robustness in the design. In essence, the cases as shown in Figures 2.1(a) to 2.1(c) that are critical to barrier stability should be considered. "H_{Deb}" shown in the figures refers to the design debris thickness. Determination of the design debris thickness refers to Section 4.4.

(b) If the existence of boulders or large hard inclusions in the debris flow cannot be precluded or as judged necessary by the designer, boulder impact should be allowed for in the first debris flow surge. If deemed appropriate, boulder impact may also be considered in the subsequent surges. The term "$r_{boulder}$" in Figure 2.1 represents the radius of the largest entrainable boulder. In assessing the design boulder size, reference should be made to the presence of corestones within the ground mass. Very often, barriers are designed on a per metre run basis. The boulder impact load per metre run can be taken as the impact load of the largest entrainable boulder divided by the width of the barrier, or by the width of a single bay of the barrier where appropriate (e.g. distance between movement joints, if any). If there are abundant boulders perched along the streamcourse or potential debris flow path with diameters similar to that of the largest entrainable boulder and where simultaneous impact by several boulders cannot be ruled out, the boulder impact load per metre run should be taken as the impact load of the largest entrainable boulder divided by the boulder diameter. Nevertheless, suitable independent measures such as baffles or debris straining structures at the upstream of barrier can be provided to reduce the impact of boulders on the barriers.

(c) As a good practice, design consideration should also be given to debris flow and boulder hitting the very top of the barrier, and overtopping of the barrier by debris flows (see also Figures 2.2(a) and 2.2(b)). The shear stress (τ) which

Table of Contents Table of Contents Table of Contents Table of Contents

will give rise to the drag force as shown in Figure 2.2(b) should be determined by the following equation:

$$\tau = h\,\rho\,g\,\tan\phi_e \quad \text{.......................................} \quad (2.1)$$

where h = thickness of debris surge

ρ = density of debris surge

$\tan\phi_e$ = equivalent coefficient of friction at the interface of the overtopping debris surge and the deposited debris and should be taken as $\tan\phi + v^2/(h\,\xi)$ where v is velocity of overflow, ϕ and ξ are the apparent friction angle and turbulent coefficient adopted in the debris mobility analysis respectively (the second term, $v^2/(h\,\xi)$, can be dropped when frictional rheological model is used)

The length of active wedge behind the barrier may be adopted in calculating the drag force.

(d) In Figures 2.1 and 2.2, the zone between the bottom of the impacting debris and the base of the wall is assumed to be subject to static earth and water pressures with allowance for surcharge loadings. The corresponding lateral earth pressure coefficient may be assumed to be unity as the debris will essentially be akin to a thick slurry.

(e) Apart from debris impact, if the barrier could be subject to boulder/rock fall impact, the stability and the structural capacity of the barrier should also be checked against the design boulder rockfall event.

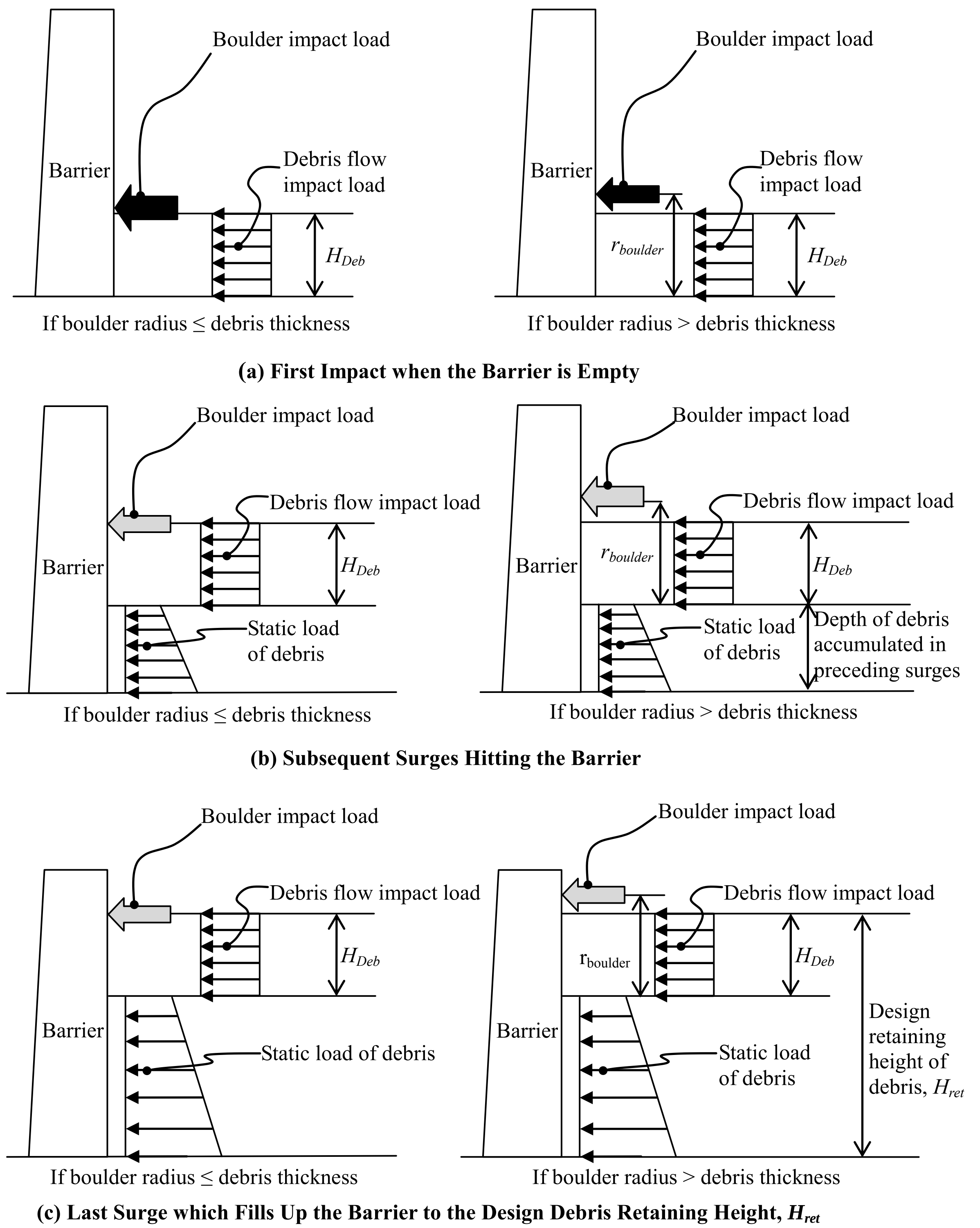
3 Boulder Impact Load

Most of the overseas practices including ASI (2011) and SWCB (2005) recommend the use of Hertz equation for estimating boulder impact load acting on a rigid barrier. Instead of adopting the Hertz equation, MLR (2006) recommends the flexural-stiffness equation for estimating boulder impact load on bridge piers and for bridge deck designs.

The Hertz equation in its original form is rather cumbersome. Experience has indicated that it can be prone to errors involving the mixing up inputs of different units[1]. It is noted that the systems of units quoted in different technical papers may not be consistent. A dimensional analysis has been carried out as part of the present study, which confirms that the use of SI units would produce results that are dimensionally correct (see Appendix B).

[1] It is noted that the boulder impact load calculated using the Hertz equation in Table 14 of Lo (2000), which is based on Zhang et al (1996), is not correct, possibly, due to mixing up the units of input parameters.

Table of Contents
Table of Contents
Table of Contents
Table of Contents



Notes:
(1) Design calculations for scenario (b) should be carried out for debris impacts at different levels corresponding to the depth of the debris accumulated in preceding surges.
(2) Debris retaining height, H_{ret}, shall be estimated using the retention capacity of the barrier. H_{ret} is not necessary to be the height of barrier wall. Normally, the height of the barrier wall could be greater than H_{ret} to provide free board and safety margin of debris retention volume.

Figure 2.1 Design Scenarios - Multiple Surges of Debris

Table of Contents
Table of Contents
Table of Contents
Table of Contents

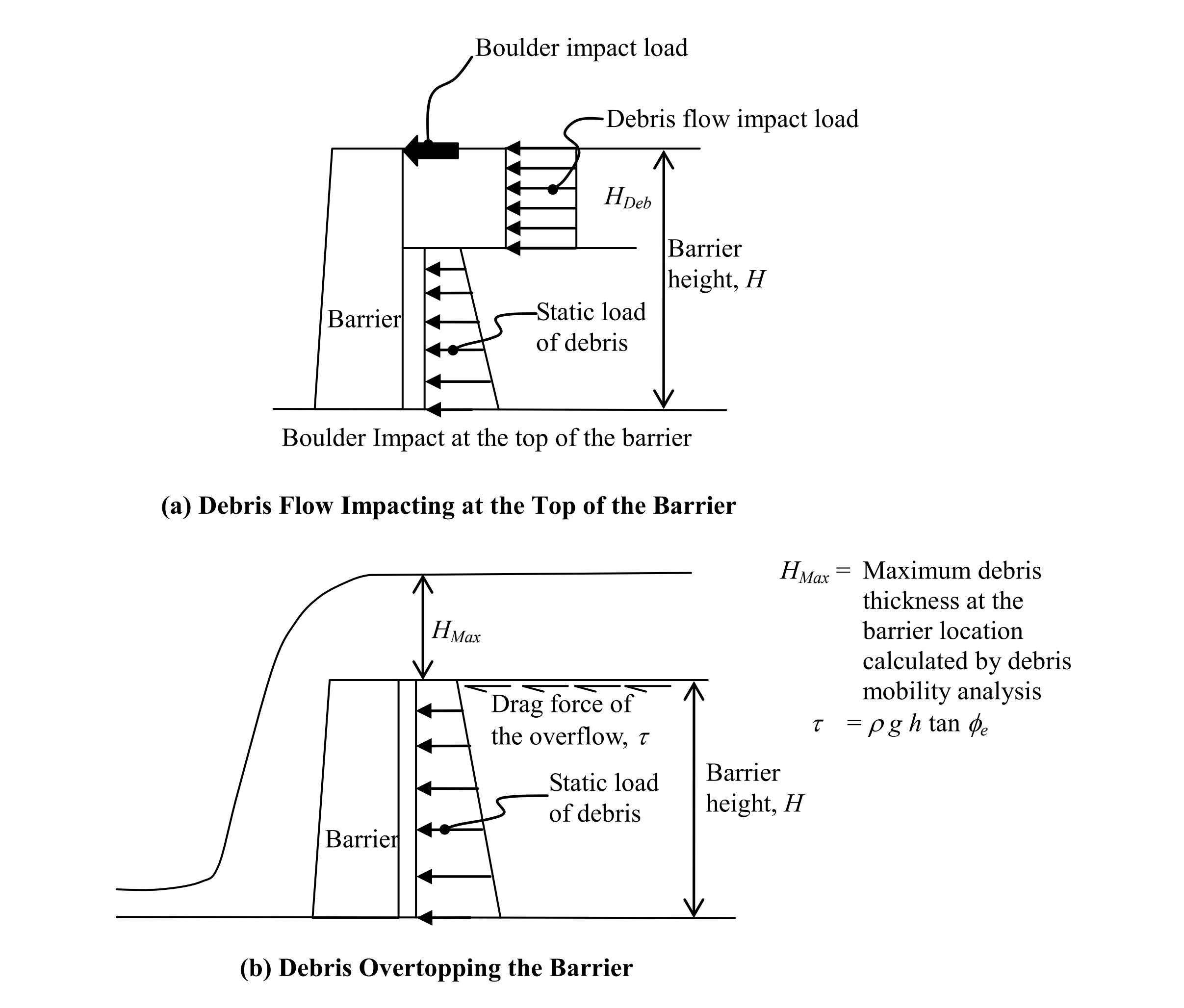


Figure 2.2 Design Scenarios - (a) Debris Impacting at the Top of the Barrier and (b) Debris Overtopping the Barrier

In practice, the Hertz equation may be simplified based on typical values of Poisson ratios and Young's moduli for boulder and rigid barrier respectively, assuming that the boulder is spherical in shape (see Appendix B). The simplified form of the Hertz equation, which may be used for preliminary design, is presented below:

$$F = K_c\, 4000\, v^{1.2}\, r^2 \quad \text{...} \quad (3.1)$$

where F = impact force of a single boulder (in kN)
K_c = load reduction factor
v = boulder impact velocity normal to the barrier (in m/s)
r = radius of boulder (in m)

Where necessary, designers can input parameters suitable for their specific design into the Hertz equation following the procedures given in Appendix B to establish a customised formula for their use.

Table of Contents Table of Contents Table of Contents Table of Contents

Some researchers such as Hungr et al (1984) noted that since energy loss (e.g. due to plastic deformation) is neglected in the derivation of the Hertz equation, the equation consistently over-estimates the boulder impact load. For example, Zhang et al (1996) reviewed the boulder impact force estimated by Hertz equation based on case histories and noted that the equation over-estimated the impact force by one to two orders of magnitude. Hungr et al (1984) recommended applying a reduction factor (K_c) of 0.1 to the load predicted by the Hertz equation in order to obtain a more realistic boulder impact load. This value of reduction factor is also adopted by Lo (2000) and Sun et al (2005).

VanDine (1996) suggested following Hungr et al (1984)'s recommendation, and ASI (2008) has included a chart for estimating boulder impact load. The chart is the same as that shown in VanDine (1996). NILIM (2007) presents a slightly modified Hertz equation incorporating a load reduction factor as calculated based on the boulder impact velocity, together with the ratio of boulder mass to barrier mass. Japan practice (NILIM, 2007) suggests that the Young modulus of concrete should be reduced empirically by 90% for calculation of the boulder impact load in order to simulate the effect of local plastic deformation on the concrete surface. All in all, it can be shown that for typical design scenarios of boulder impact (i.e. boulder diameter = 1 to 2 m, boulder impact velocity = 8 to 15 m/s and barrier wall of dimensions 20 m long by 5 m high by 0.8 m thick), the NILIM's procedures would produce an equivalent load reduction factor (K_c) of less than 0.1.

The load reduction factor embedded in the Hertz equation as presented in the Taiwan practice (SWCB, 2005) ranges from 0.2 to 0.5. Although the upper bound of the range is larger as compared with the practice of other countries, it should be noted that the Taiwan practice recommends that boulder impact load and debris flow impact load are considered separately (i.e. not additive) in the stability calculation as mentioned in Section 2.

The value of K_c has been reviewed based on the case histories presented in Table 14 of Lo (2000). In three of the four case histories, the value of K_c is much lower than 0.1 (see Table 3.1). In the case of the bridge pier failure in Dongchuan on 30 June 1981 (i.e. the third case in Table 3.1), the calculated value of K_c is 0.15. However, as pointed out by Wu et al (1993), this bridge pier failure could have been contributed by the combined effects of boulder impact and debris flow. Therefore, the above K_c value could have been over-estimated.

3.1 Recommended Load Reduction Factor to Hertz Equation

Having reviewed the state of practice of various overseas countries and the available case histories, it is recommended that the value of K_c may be taken as 0.1, same as the one suggested in Section 5.4 of Lo (2000).

Table of Contents Table of Contents Table of Contents Table of Contents

Table 3.1 Boulder Impact Load Estimated Using Hertz Equation

Case Histories	Estimated Failure Capacity^{+}	Boulder impact load estimated using Hertz equation without load reduction factor $F = 4000\, v^{1.2}\, r^2$	Deduced value of K_c
On 7 September 1981, a 6 m diameter boulder entrained in a debris flow severed the centre pier of a bridge on the Chengdu-Kuming Railroad into three fragments (Zhang et al, 1996). $E_b = 49$ GPa, $E_B = 26$ GPa, $\mu_b = 0.2$ $\mu_b = 0.18$, $v_b = 8.9$ m/s, $I_B = 12.56$ m^4 $\rho_b = 2\,700$ kg/m^3, $\psi = 0.6$, $m_b = 414$ Mg $L_B = 22$ m, $r_b = 3.32$ m, $A_c = 0.5$ m^2	12 000 kN	$F = 4000\,(8.9)^{1.2}\,(3.32)^2$ $= 607\,581$kN	12000 / 607581 = 0.02
In June 1983, a 5.5 m boulder entrained in a debris flow buckled two 1 m diameter steel pipes in Dongchuan, Yunnan. These pipes were welded together side by side with a wall thickness of 1 cm (Zhang et al, 1996). $E_b = 42$ GPa, $E_B = 210$ GPa, $\mu_b = 0.2$ $\mu_B = 0.3$, $v_b = 12$ m/s, $A_c = 0.2$ m^2 $m_b = 230.6$ Mg, $\rho_b = 2\,700$ kg/m^3	28 600 kN	$F = 4000\,(12)^{1.2}\,(2.75)^2$ $= 596\,682$ kN	28600 / 596682 = 0.05
On 30 June 1981, a debris flow severed a concrete bridge pier in Dongchuan, Yunnan. The cross-sectional area of the bridge pier normal to the debris flow direction is 21 m^2. A 3 m diameter boulder was later found nearby the bridge pier and it was believed that this boulder probably contributed to the failure of the pier (Wu et al, 1993). $E_b = 42$ GPa*, $E_B = 26$ GPa*, $\mu_b = 0.2$* $\mu_B = 0.18$*, $v_b = 12.5$ m/s, $A_c = 0.25$ m^2 $\rho_b = 2\,700$ kg/m^3, $\rho_d = 2\,190$ kg/m^3, $m_b = 38.2$ Mg	28 770 kN	$F = 4000\,(12.5)^{1.2}\,(1.5)^2$ $= 186\,438$ kN	28770 / 186438 = 0.15
On 10 August 1968, a boulder estimated to be 2 m x 3 m x 4 m in size destroyed a reinforced concrete structure in a debris flow event in Dongchuan, Yunnan (Wu et al, 1993). $E_b = 42$ GPa*, $E_B = 26$ GPa*, $\mu_b = 0.2$* $\mu_B = 0.18$*, $v_b = 10.5$ m/s, $I_B = 0.16$ m^4 $\rho_b = 2\,700$ kg/m^3, $L_B = 16$ m, $m_b = 64.8$ Mg $A_c = 0.2$ m^{2*}	1 290 kN	Boulder radius = $((2x3x4)x3/(4x3.14))^{0.33}$ = 1.8 $F = 4000\,(10.5)^{1.2}\,(1.8)^2$ $= 217\,787$ kN	1290 / 217787 = 0.006

Notes: + provided by original authors   * assumed values adopted in this study

Table of Contents
Table of Contents
Table of Contents
Table of Contents

4 Debris Impact Load

4.1 Dynamic Pressure Coefficient

Two common models have been adopted in overseas practice for calculation of the dynamic impact load. The first model suggests that the thrust (F) acting on a vertical wall give rise by debris impact is estimated as $F = \beta \rho g h^2 / 2$; where β is empirical factor to account for dynamic effects of the flow impact, ρ is debris density, g is gravitational acceleration and h is debris thickness. This is a form of the hydrostatic pressure calculation. Armanini (1997) carried out small-scale tests to calibrate this model and he recommended $\beta = 7$ to 11. However, this model does not considered debris impact velocity which implies that the model is valid only for a limited range of velocities.

The second model is the widely adopted hydro-dynamic model. Guidelines published in Mainland China, Japan, Taiwan and Canada recommend the use of this model, in which the impact pressure is assumed to be proportional to the square of debris flow velocity. The equation is of the form: $p = \alpha \rho v^2$, where p is debris impact pressure, α is dynamic pressure coefficient, ρ is density of debris flow, and v is debris velocity normal to the barrier. Table 4.1 presents the recommendations adopted by the various international practices. The values of the dynamic pressure coefficient as recommended by various overseas countries using the hydro-dynamic approach range from 1.0 to 1.5. Proske et al (2011) reviewed the recent literature proposing this approach and noted that the recommended dynamic pressure coefficients for passable obstacles range from 1.0 to 2.5.

ASI (2008) follows a formula (i.e. $p = 4.5 \rho v^{0.8} (g h)^{0.6}$) similar to the hydro-dynamic model to calculate the debris impact pressure with consideration of debris impact velocity. The formula is derived empirically using the Froude number ($Fr = v / (g h)^{0.5}$) and hence debris thickness is involved in the calculation. It can be shown that for typical design debris flow events considered in local barrier design (i.e. debris velocity = 8 to 15 m/s and debris thickness = 0.5 to 2 m), the debris impact pressure given by ASI's formula is equivalent to those given by the hydro-dynamic model with dynamic pressure coefficients varying from 0.5 to 2.2 (see also Table 4.1 as reference). More recently, Proske et al (2011) have recommended a slightly different Froude number formula (i.e. $p = 5 \rho v^{0.8} (g h)^{0.6}$), of which the equivalent dynamic pressure coefficient varies from 0.5 to 2.5.

Lo (2000) suggested that the value of the dynamic pressure coefficient can be taken as 3.0 based on the field measurement data presented by Du et al (1987). According to Du et al (op cit), the larger dynamic pressure coefficient is adopted to cater for the heterogeneous nature of debris flow.

Hu et al (2011) reported a more recent and comprehensive instrumented field study in China. The debris flow of concern had a velocity of 5 to 12 m/s. Particle size distribution (PSD) tests of the solid content of the debris flow indicated that the particle size ranged from 0.001 to 100 mm. The measured debris flow impact pressures are presented in Figure 4.1 (see the solid circles in the figure). For completeness, the field data of China quoted by Lo (2000) are also shown in Figure 4.1. The dynamic pressure coefficient which fits the average impact pressure reported by Hu et al (2011) is 2. Nevertheless, there is a large scatter of the measured pressures and some of the local data are as high as four times that of the average value, which corresponds to a dynamic pressure coefficient exceeding 8.

Table of Contents
Table of Contents
Table of Contents
Table of Contents

Table 4.1 Estimation of Debris Impact Pressure

Design Practice	Debris Dynamic Pressure (p)	Design Approach and Factors of Safety	Approach in Dealing with Boulder Impact Load
GEO Report No. 104 (Lo, 2000)	$p = \gamma \rho v^2$; $\gamma = 3.0$	Partial Safety Factor Approach Partial safety factors on: tan ϕ' and c' = 1.2 Impact load = 1.0	Boulder impact load is calculated using the Hertz equation with a load reduction factor of 0.1. Design load is to be taken as the superimposition of the debris dynamic pressure and boulder impact load.
Austria (ASI, 2008)	$p = 4.5\rho v^{0.8}(g\,h)^{0.6}$ where h is the debris thickness Debris dynamic pressure given by the above formula is equivalent to those calculated using the hydro-dynamic approach with dynamic pressure coefficients varying from 0.5 to 2.2 for typical design scenario	Approach follows Eurocode 7 (Possible factors given in ASI, 2011) <u>Overall Stability</u> Partial safety factor on impact load = 1.1 Partial safety factor on tan ϕ' and c' = 1.2 Factor of Safety against overall stability = 1.0 <u>Sliding Stability</u> Partial safety factor on impact load = 1.2 Partial safety factor on tan ϕ' for calculation of shear resistance = 1.1 Partial safety factor on passive resistance = 1.3 Factor of Safety against sliding stability = 1.0	Load estimation chart of boulder impact as predicted by Hertz equation is included. Design load is to be taken as the superimposition of the debris dynamic pressure and boulder impact load.
British Columba, Canada (VanDine, 1996)	$p = \gamma \rho v^2$; $\gamma = 1.0$	Not mentioned Remarks: Debris thickness is to be factored up by 1.5 in the design	Boulder impact load is calculated using Hertz equation and reference is made to Hungr (1984) for details. Design scenario is not mentioned.
China (MLR, 2006)	$p = \gamma \rho v^2$ γ = 1.0 for circular structure γ = 1.33 for rectangular structure γ = 1.47 for square structure	Not mentioned	Boulder impact load is calculated using flexural-stiffness equation. Design scenario is not mentioned.
Japan (NILIM, 2007)	$p = \gamma \rho v^2$; $\gamma = 1.0$	Global Safety Factor Approach Factor of Safety: Sliding 1.2 (for wall height <15m) Sliding 1.5 (for wall height ≥ 15m)	Boulder impact load is calculated using the modified Hertz equation. The equivalent load reduction factor is less than 0.1 (see Section 3). Design scenario is not mentioned.
Taiwan (SWCB, 2005)	$p = \gamma \rho v^2$; $\gamma = 1.0$	Global Safety Factor Approach Factor of Safety: Sliding 1.25 (for wall height ≤ 10m) Sliding 1.5 (for wall height > 10m)	Boulder impact load is calculated using Hertz equation with a load reduction factor of 0.2 to 0.5. Debris dynamic pressure and the boulder impact load are considered under separate design scenarios.

Table of Contents Table of Contents Table of Contents Table of Contents

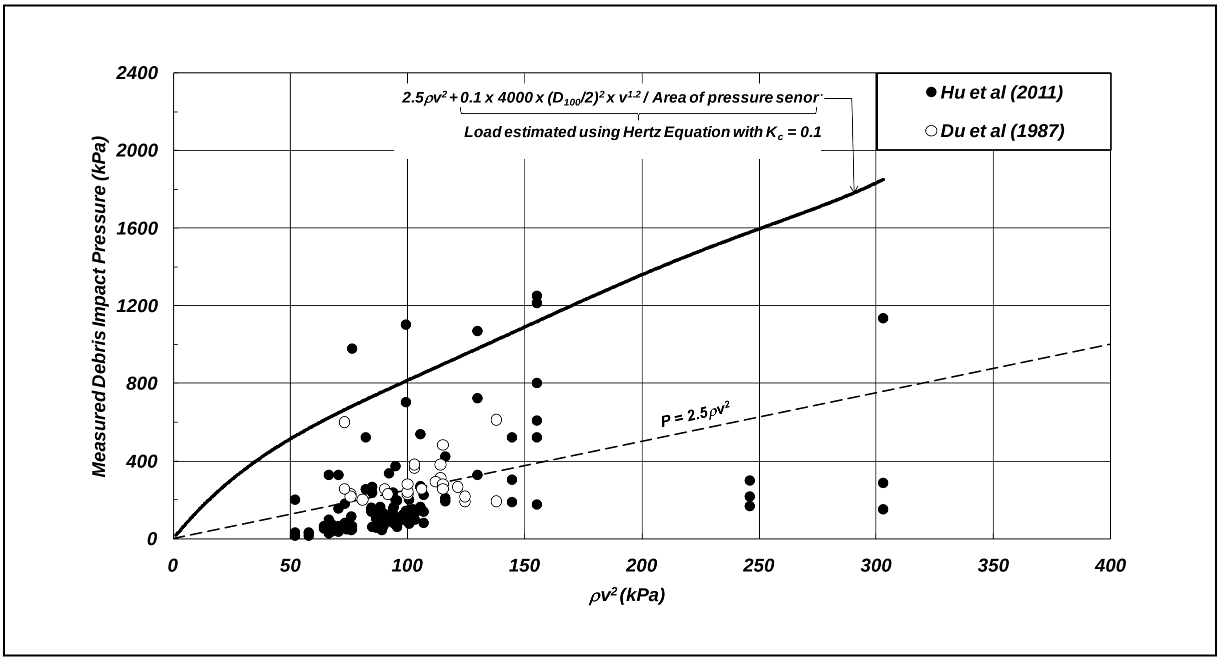


Figure 4.1 Measured Impact Pressure

Hu et al (op cit) opined that the extreme impact pressures were potentially caused by the impacts of boulders larger than 300 mm in size as observed on site. An attempt is made in the present study to estimate the impact pressure based on the superposition of the measured average debris flow impact pressure, which is notionally taken as 2.5 ρ v^2, and the impact pressure of hard intrusions as derived using the Hertz equation with a K_c value of 0.1 as recommended in Section 3 (see Figure 4.1). The maximum diameter of hard inclusions determined from PSD tests (i.e. 100 mm) is considered in the estimate. The predicted envelope almost brackets all the data. Given that the size of boulder hitting the sensor could potentially be larger than 300 mm (c.f. 100 mm as adopted in the present calculation), some conservatism could have been built into the above.

4.2 Recommended Dynamic Pressure Coefficient

All in all, having reviewed the data as well as the international design practice, it is suggested that the dynamic pressure coefficient in the hydro-dynamic equation can be taken as 2.5 for estimating the dynamic debris impact pressure in combination with boulder impact load as appropriate, i.e. Equation (8) stated in Lo (2000) should read $p = 2.5\ \rho_d\ v_d^2 \sin\beta$ where p is impact pressure, ρ_d is debris density, v_d is velocity of debris at impact and β is angle between the impact force and the debris motion direction.

4.3 Method to Establish the Debris Impact Load

Using the hydro-dynamic model, the force (F) acting on a vertical rigid barrier induced by debris impact is calculated as follows:

$$F = \alpha\, \rho\, v^2 \sin\beta\; h\, w \quad \text{...} \quad (4.1)$$

where
α = dynamic pressure coefficient
ρ = density of debris flow (in kg/m^3)
v = debris velocity at impact (in m/s)
h = debris thickness (in m)
w = debris width (in m)
β = angle between impact face of barrier and debris motion direction

The thickness (h) and velocity (v) of the moving debris are usually estimated from debris mobility models. The debris width (w) should be established based on the geometry of runout path or by means of three-dimensional debris mobility models, otherwise it should be established based on the geometry of the debris runout path. The rheological parameters applicable for design purpose are recommended by Lo (2000) and GEO Technical Guidance Note (TGN) No. 29 (GEO, 2011).

Debris reaching the barrier at different times may have different combinations of h, v and w. Many local practitioners established the impact force (F) based on the maximum velocity and the maximum debris thickness, calculated by debris mobility model (at present, two-dimensional debris mobility model is normally adopted). This debris impact load established could be overly conservative, as the maximum debris thickness and the maximum debris velocity may not occur at the same instance. Alternatively, the debris impact load can be calculated as:

(i) the maximum impact load calculated based on the combination of h, w and v at different times; or

(ii) the impact load calculated based on the debris frontal velocity, and the average debris width and debris thickness at the barrier location.

Loadings defined in terms of (i) and (ii) above have been calculated based on design scenarios and geometry of runout paths of ten Landslip Prevention and Mitigation projects. Those design scenarios cover both open hillside failures and channelised debris flows. Figure 4.2 compares the above loadings. The comparisons reveal that item (i) gives a greater maximum impact force in most cases. However, item (ii) gives a greater impact force when the barrier is located well within debris deposition zone where the debris decelerates and debris surge becomes thicker. In view of this, the larger of the loads as obtained by (i) and (ii) should be taken as the design impact load.

4.4 Recommendations on the Calculation of Debris Impact Load

(a) The thickness (h) and velocity (v) of the moving debris should be estimated from debris mobility models with an algorithm agreed by the GEO. The debris width (w) should be established based on the geometry of runout path or by means of three-dimensional debris mobility models, otherwise it should be pre-determined based on the

Table of Contents
Table of Contents
Table of Contents
Table of Contents

geometry of the debris runout path. Realistic topographical profiles, including the geometry of the drainage lines, should be incorporated in the mobility analysis. The rheological parameters should follow those recommended in GEO Report No. 104 (Lo, 2000) and GEO TGN No. 29 (GEO, 2011).

(b) Debris reaching the barrier location at different times may have different combinations of h, v and w (if w is also determined using debris mobility analyses). The design impact load should be the larger of (i) the maximum impact load calculated based on the combinations of h, w and v at different times, and (ii) the impact load calculated based on the debris frontal velocity, and the average debris width and debris thickness at the barrier location.

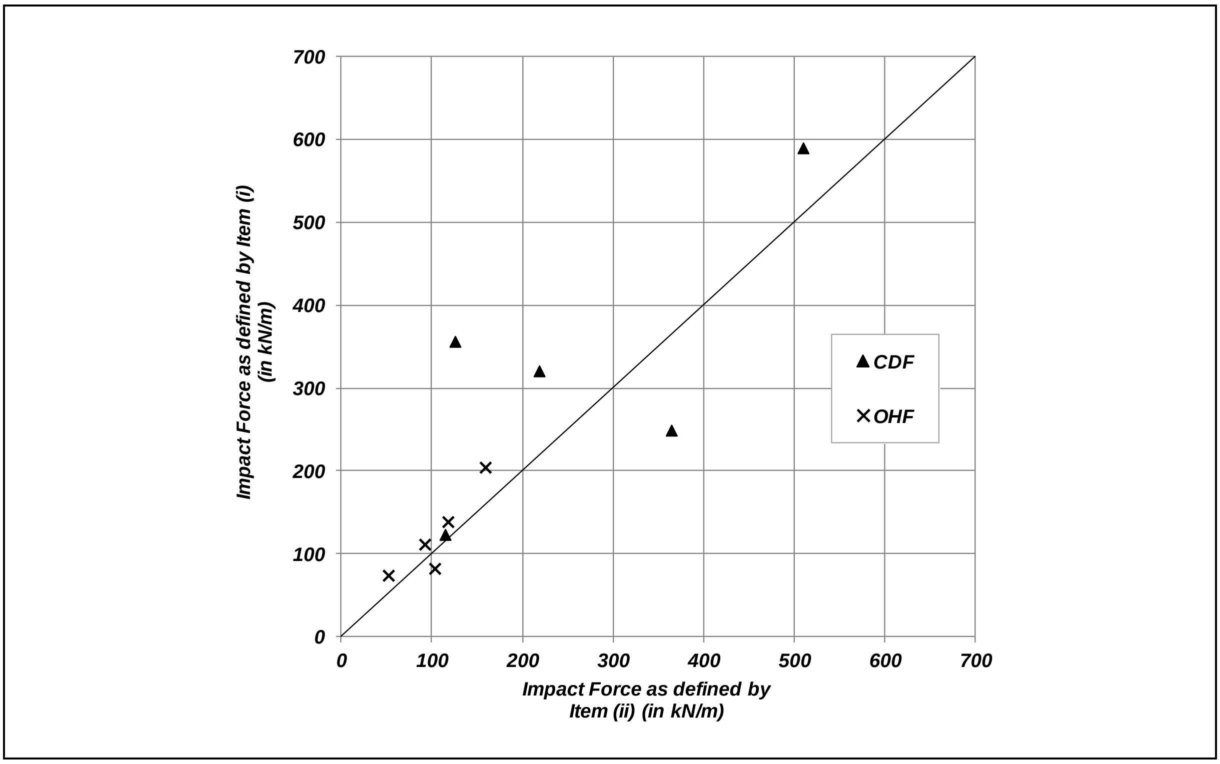


Figure 4.2 Comparison of Impact Forces as Defined by Item (i) and Item (ii)

5 Safety Margin

Consideration has been given to the required safety margin taking into account the recommended approach in estimating debris impact and boulder impact in the design. The design factors of safety adopted by different practices are summarised in Table 4.1. It is noted that with the currently used partial factors based on Geoguide 1 (GEO, 1993) (i.e. 1.0 for impact load and 1.2 for tan ϕ' and c'), the overall safety margin of the local design is

Table of Contents Table of Contents Table of Contents Table of Contents

broadly comparable with that of other approaches in international practices. Detailed recommendations on factors of safety are given in Section 5.1 to supplement Lo (2000).

5.1 Recommended Factors of Safety for Checking of Geotechnical Stability

(a) The partial safety factors based on Geoguide 1 (GEO, 1993) should be adopted in establishing the stabilisation and destabilisation forces at the ultimate limit state for design scenarios illustrated in Figure 2.1. A partial safety factor of unity may be applied on the debris and boulder impact load.

(b) Having considered the nature of the surge of impacting debris and the uncertainty involved, the self weight of the impacting debris should be considered as a live load in the design, and the limit state method recommended in Geoguide 1 (GEO, 1993) should be followed (i.e. self weight of impacting debris of height H_{Deb} shown in Figures 2.1 and 2.2(a), which has a beneficial effect on geotechnical stability, should be ignored; whilst the self weight of the impacting debris behind the barrier, which acts as a surcharge to produce a lateral earth pressure destabilising the barrier, should be multiplied by a partial load factor of 1.5).

(c) The partial safety factor to be applied on tan ϕ_e in Equation 2.1 and on the drag force induced by overtopping can be taken as unity.

(d) For terminal barriers that are designed for retaining debris of a given design event, the check against debris impact at the top of the barrier and overtopping is considered a good practice to cater for the uncertainty in the design debris volume. In this regard, a reduced partial factor of safety of 1.1 for tan ϕ' and c' can be adopted for checking the design scenarios shown in Figures 2.2(a) and 2.2(b) for terminal barriers. However, for design of intermediate barriers, no reduction in the values of the partial factors of safety should be allowed.

6 Height of Barrier

The height of a terminal barrier should be designed to provide adequate retention capacity and to avoid debris spilling over the barrier. Lo (2000) notes that barriers should be high enough to account for the possibility of debris run-up. VanDine (1996) highlights the requirement to check the barrier height against potential debris run-up as per the suggestion by Hungr et al (1984). However, such a design provision is not required by other codes or

Table of Contents
Table of Contents
Table of Contents
Table of Contents

practices. It is noted that the debris resisting structure considered by Hungr et al (1984) is a debris check dam with a debris wedge formed at the back.

6.1 Estimation of Debris Run-up Height against Sloping Ground

Mancarella & Hungr (2010) show that dynamic analysis could provide a reasonable estimate of the run-up height of granular flow against steep slopes (from about 35° to 90° against the runout path) as verified by flume tests (see also Figure 6.1). According to Mancarella and Hungr (op cit), direct application of the leading edge model to estimate debris run-up against steep barrier faces may be non-conservative. Results of appropriate debris dynamic analysis seem to be more robust.

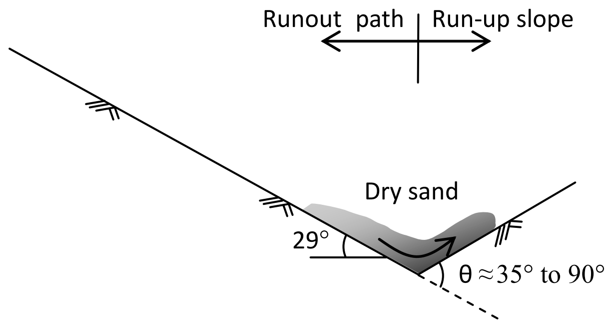
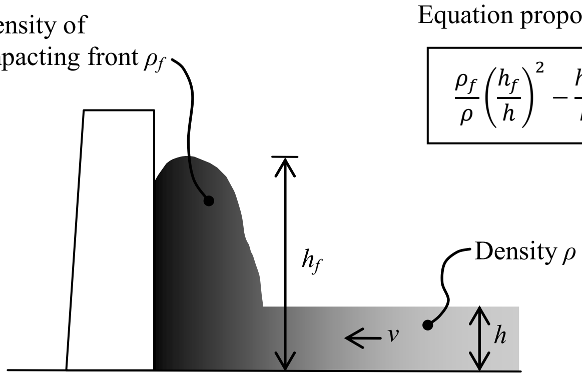


Figure 6.1 Set Up of Mancarella & Hungr (2010)'s Flume Test

6.2 Estimation of Debris Run-up Height against Vertical Wall

Debris run-up height against a vertical rigid barrier can be calculated using energy principle, i.e. $h_f = h + v^2/2g$; where h_f is run-up height, v is debris velocity, h is debris thickness and g is gravitational acceleration. This equation will give a conservative estimate of debris run-up height, as it assumes no energy loss during the run-up process.

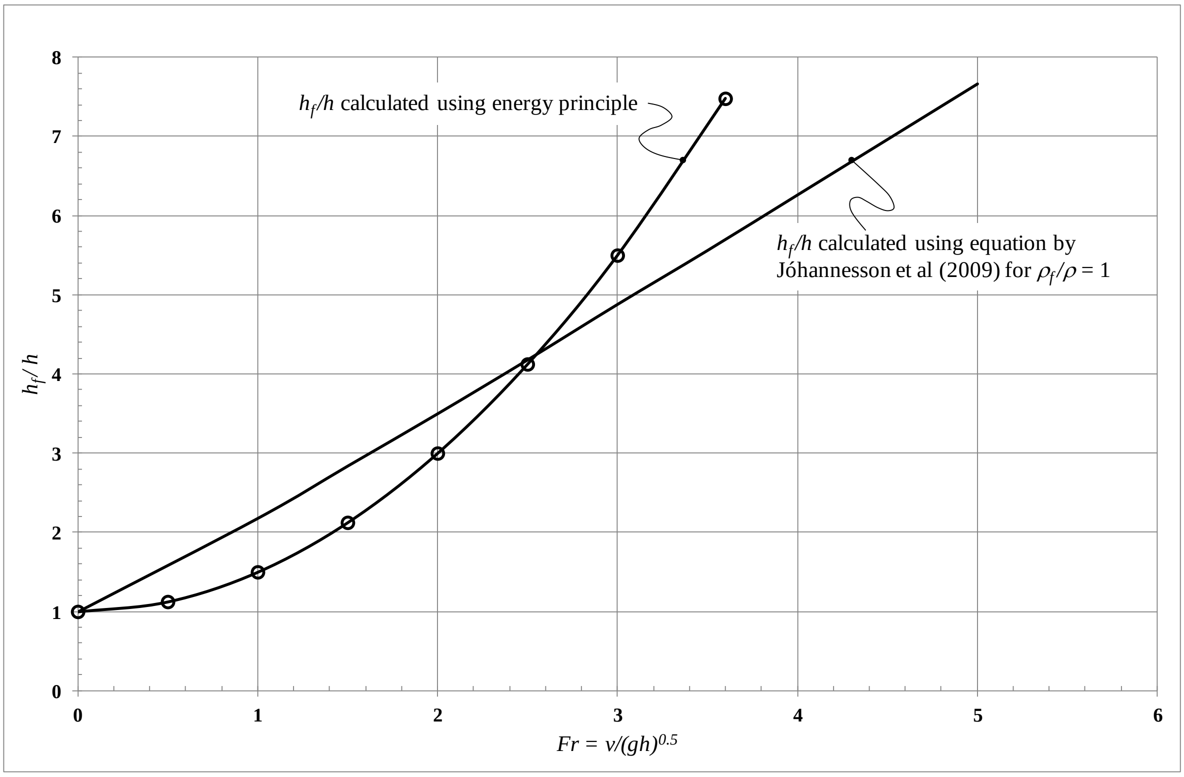
More recently, Jóhannesson et al (2009) have developed an analytical solution for the calculation of debris run-up height against a vertical barrier on a horizontal channel bed (see Figure 6.2(a)). The relationship between h_f/h and Froude number (Fr) given by the analytical solution is illustrated in Figure 6.2(b). The equation is derived based on consideration of conservation of mass and momentum for a shock that occurs during the impact of a shallow flow layer on vertical rigid barrier (see also Hákonardóttir, 2004), and is considered suitable for debris run-up height calculation. However, it is noted that this equation will give a run-up height greater than $h + v^2/2g$ when Froude number is less than 2.5 (see Figure 6.2(b)). In such a circumstance, the debris run-up height given by the energy principle may be more appropriate for design purpose. When Fr exceeds 2.5, run-up height can be assessed using the analytical solution. The assessment of run-up height for flow on an inclined bed using Jóhannesson's equation may not be applicable, in which case the use of energy principle should be considered.



Equation proposed by Jóhannesson et al (2009):

$$\frac{\rho_f}{\rho}\left(\frac{h_f}{h}\right)^2 - \frac{h_f}{h} - 1 + \left(\frac{\rho_f h_f}{\rho h}\right)^{-1} - 2\frac{v^2}{gh} = 0$$

(a) Calculation of Debris Run-up Height Proposed by Jóhannesson et al (2009)



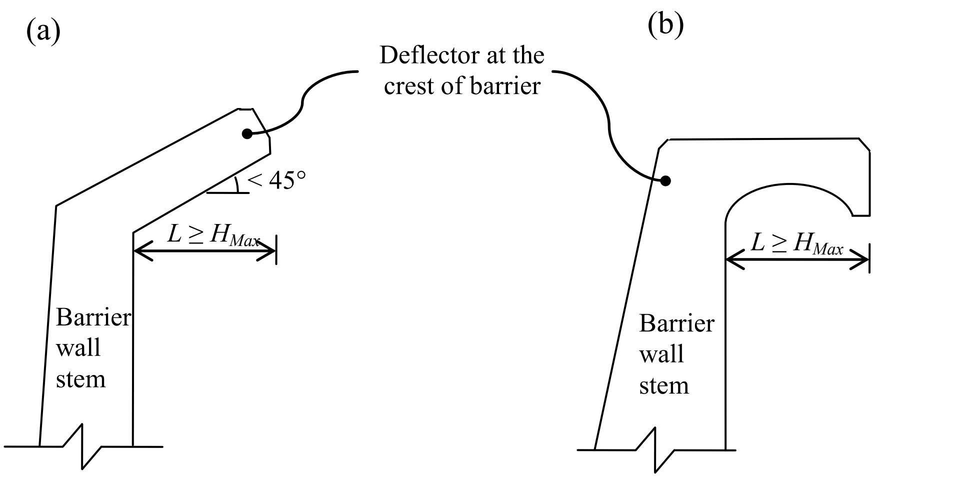
(b) Relationship between h_f/h and Froude number (Fr) of Debris Flow Calculated using Equation by Jóhannesson et al (2009) and Energy Principle.

Figure 6.2 Calculation of Debris Run-up Against a Vertical Barrier

Table of Contents
Table of Contents
Table of Contents
Table of Contents

6.3 Measures to Prevent Over-spilling of Debris Run-up

Another possible approach to prevent debris over-spilling is to provide a suitable deflector detail at the top of the wall stem, which deflects debris splash back towards the retention basin. The deflector at wall crest and any accompanying structural elements should be designed to withstand the impact of the landslide debris and boulders. Figure 6.3(a) shows an example of such a deflector. Deflectors can be constructed in other form as long as it serves the purpose, Figure 6.3(b) illustrates another example of deflector.



H_{Max} = Maximum debris thickness at the barrier location calculated by debris mobility analysis

Figure 6.3 Schematic Designs of Deflector at the Barrier Wall Top

6.4 Recommendations on Determining Barrier Height

In summary, the following recommendations are made to supplement the run-up height calculation requirement stated in Section 7 of Lo (2000):

(a) The height of a terminal barrier should be designed to provide adequate retention capacity. Barrier height should also be checked against debis run-up height, unless measures to deflect debris splash back towards the retention basin is provided.

(b) For assessments of debris run-up height on barriers with a sloping back, reference should be made to the latest research findings reported by Mancarella & Hungr (2010). Appropriate debris mobility analysis may be applied for estimating the debris run-up height.

Table of Contents Table of Contents Table of Contents Table of Contents

(c) Analytical solution by Jóhannesson et al (2009) may be adopted for calculation of debris run-up height against vertical wall on horizontal bed. The calculated height solution may exceed the value given by the energy principle. In this circumstance, the design debris run-up height may be assumed as $h + v^2/2g$. If the retention zone the vertical barrier is inclined, the assessment of the debris run-up height may follow the energy principle.

7 Conclusions and Recommendations

A review to consolidate the state-of-the-art knowledge as presented in international journal papers, and design guidelines/practices in Austria, Mainland China, Japan, Taiwan and Canada is presented in this report. The following recommendations are put forward to supplement the current design guidance:

(i) Design Debris Impact Scenarios - design loading on barriers shall be established based on a multi-surge scenario that fills up the barrier to the design retention height in sequence. Dynamic impact load of debris and boulder impact load should be included in the design calculations as appropriate. In addition, design check against loading induced by overtopping of debris is recommended.

(ii) Design Boulder Impact Load - the Hertz Equation with a load reduction factor of 0.1 can be adopted for determining the boulder impact load; the erroneous units of the input parameters for the equation, as referenced by Lo (2000), are rectified.

(iii) Design Dynamic Impact Load of Debris - the hydro-dynamic equation can be used to determine the impact pressure of debris. The pressure coefficient can be taken as 2.5 (c.f. 3.0 recommended by Lo, 2000). Procedures to establish the debris impact load based on the results of debris mobility analysis are proposed.

(iv) Design Height of Barriers - the height of barriers should be established based on the design retention capacity, and should be checked against the debris run-up height. Alternatively, a deflector structure may be provided at the crest of barrier to prevent debris splash from spilling over the barrier.

Table of Contents
Table of Contents
Table of Contents
Table of Contents

8 References

Armanini, A. (1997). On the dynamic impact of debris flows. *Recent Developments on Debris Flows.* Lecture notes in Earth Sciences 64. Springer Verlag, Berlin, pp 208-226.

ASI (2008). *ONR 24801 Protection Works for Torrent Control - Actions on Structures (Draft).* Austrian Standard Institute, Austria, 25 p.

ASI (2011). *ONR 24802 Protection Works for Torrent Control - Design of Structures.* Austrian Standard Institute, Austria, 96 p.

Du, R., Kang, Z., Chen, X. and Zhu, P. (1987). *A Comprehensive Investigation and Control Planning for Debris Flow in the Xiajiang River Basin of Yunnan Province*. Science Press, 287 p (in Chinese).

Fabio, V.D.B. (2011). *Introduction to the Physics of Landslides, Lecture Notes on the Dynamics of Mass Wasting*. Springer, 408 p.

GEO (1993). *Guide to Retaining Wall Design (Geoguide 1) (2nd Edition).* Geotechnical Engineering Office, Hong Kong, 258 p.

GEO (2011). *Guidelines on the Assessment of Debris Mobility for Channelised Debris Flows, GEO Technical Guidance Note No. 29 (TGN 29)*. Geotechnical Engineering Office, Hong Kong, 6 p.

Hákonardóttir, K.M. (2004). *The Interaction between Snow Avalanches and Dams.* Doctor of Philosophy Dissertation. School of Mathematics, University of Bristol, 142 p.

Hu, K., Wei, F. & Li., Y. (2011). Real-time Measurement and Preliminary Analysis of Debris-flow Impact Force at Jiangjia Ravine, China. *Earth Surface Processes and Landforms,* doi: 10.1002/esp. 2155.

Hungr, O., Morgan, G.C. & Kellerhals, R. (1984). Quantitative Analysis of Debris Torrent Hazards for Design of Remedial Measures. *Canadian Geotechnical Journal*, vol. 21, pp 663-677.

Jóhannesson, T., Gauer, P., Issler, P. & Lied, K. (2009). *The Design of Avalanche Protection Dams - Recent Practical and Theoretical Developments*, Directorate-General for Research, Environment Directorate, European Commission, EUR2339, 205 p.

Lo, D.O.K. (2000). *Review of Natural Terrain Landslide Debris-resisting Barrier Design (GEO Report No. 104)*. Geotechnical Engineering Office, Hong Kong, 91 p.

Mancarella, D. & Hungr, O. (2010). Analysis of Run-up of Granular Avalanches against Steep, Adverse Slopes and Protective Barriers. *Canadian Geotechnical Journal*, vol. 47, pp 827-841.

Table of Contents
Table of Contents
Table of Contents
Table of Contents

MLR (2006). *Specification of Geological Investigation for Debris Flow Stabilization.* DZ/T 0220-2006, National Land Resources Department, China, 32 p (in Chinese).

Ng, K.C., Parry, S., King, J.P., Franks, C.A.M. & Shaw, R. (2003). *Guidelines on Natural Terrain Hazard Studies (GEO Report No. 138)*. Geotechnical Engineering Office, Hong Kong, 138 p.

NILIM (2007). *Manual of Technical Standard for Designing Sabo Facilities against Debris Flow and Driftwood*. Technical Note of NILIM No. 365, Natural Institute for Land and Infrastructure Management, Ministry of Land, Infrastructure and Transport, Japan, pp 73 (in Japanese).

Proske, D., Suda, J. & Hübl, J. (2011). Debris flow impact estimation for breakers. *Georisk*, vol. 5, no. 2, June 2011, pp 143-155.

Sun, H.W., Lam, T.T.M and Tsui, H.M. (2005). *Design Basis for Standardised Modules of Landslide Debris-resisting Barriers (GEO Report No. 174)*. Geotechnical Engineering Office, Hong Kong, 161 p.

SWCB (2005). *Manual of Soil and Water Conservation*. Soil and Water Conservation Bureau, Taiwan, 692 p (in Chinese).

VanDine, D.F. (1996). *Debris Flow Control Structures for Forest Engineering*. Ministry of Forests, British Columbia, Canada, 68 p.

Wu, J., Tian, L., Kang, Z., Zhang, Y. & Liu, J. (1993). *Debris Flow and Its Comprehensive Control*. Science Press, 332 p (in Chinese).

Zhang, S., Hungr, O. & Slaymaker, O. (1996). The calculation of impact force of boulders in debris flow. *Debris Flow Observation and Research*, edited by R. Du, Science Press, pp 67-72 (in Chinese).

Table of Contents

Table of Contents

Table of Contents

Table of Contents

Appendix A

Responses to Comments by Professor O. Hungr

Table of Contents
Table of Contents
Table of Contents
Table of Contents

Contents

Table of Contents
Table of Contents
Table of Contents

A.1 Comments on the First Draft of the Report by Professor O. Hungr and responses by GEO

Comments	Responses to Comments
Professor O. Hungr (Independent Expert Reviewer): **General** The purpose of this report, as defined by a Draft Brief prepared by AECOM, Hong Kong on January 12, 2012, is to undertake an independent technical review of the above-mentioned GEO's draft discussion note on the supplementary design guidelines for debris-resisting rigid barriers.	Noted.
Before undertaking this review, we have studied a number of relevant documents, as listed in the References. In general, we find the Note and its preceding reports to be on a very high technical level, combining an open-minded review of a wide spectrum of publications and standards from various countries in Europe and Asia with a practically-oriented and critical analysis. Debris flow behaviour is complex and contains many variables, often random. As a result, design procedures for debris flow protection structures are exceedingly difficult to standardize. It must be recognized that no standard will probably ever replace the need for the application of experienced judgment, tailored to observed specific conditions of each application. This is correctly reflected in the tone of the reviewed GEO documents, which provide an appropriate and honest acknowledgment of existing uncertainties.	Noted.
Specific comments **General comment, need for classification:** The term "debris flow" is frequently used in Hong Kong and	It is agreed that proper classification of flow-like landslides are important for the design of debris-resisting barriers. Simple

Table of Contents
Table of Contents
Table of Contents
Table of Contents

Comments	Responses to Comments
elsewhere to describe a diverse range of phenomena. If techniques of hazard characterization and design of remedial measures are to be to some extent standardised, they must recognise this phenomenological diversity. The existing GEO debris barrier design reports (104 and 174) as well as older documents (e.g. Wong and Ho, 1996) recognise differences between types of debris events, but this effort should be further developed and implemented more systematically in all steps of the design process, from estimating debris mobility, to developing design configuration and calculating structural loads. Unfortunately, existing classifications of flow-like landslides are poorly developed and not uniformly accepted (e.g. Hungr et al., 2001 and 2005). Attempts to classify debris phenomena for design purposes have been made in other countries (e.g. Aulitzki, 1973, Suda et al., 2009, Zhang, 1993). But more work is needed to bring this aspect of the scientific/engineering practice to a level that is required to standardize the methodology. It is recommended that a formal classification be developed and incorporated systematically into the suggested design framework. It is not our intent here to attempt to give the final word on this subject. However, some suggestions for an approach are sketched out in the following paragraphs.	classifications of landslide types are given in GEO Report No. 138 (Ng et al, 2003) for practical engineering purposes. We will review the classifications further when the JTC-1 proposed classification is finalised. The guidelines given in the Note are intended to cater for different types of landslide events in Hong Kong in a robust manner. Reference: Ng, K.C., Parry, S., King, J.P., Franks, C.A.M. & Shaw, R. (2003). Guidelines for Natural Terrain Hazard Studies (GEO Report No. 138). Geotechnical Engineering Office, Hong Kong, 138 p.
Flow-like landslides occurring in Hong Kong should be divided into at least three categories. Each category should be described by a fairly comprehensive definition (possibly accompanied by examples). Approaches to hazard characterization and design of remedial measures should then be defined separately for each type. Briefly, the three types could be defined as follows:	Please see response above.

Comments	Responses to Comments
<u>Debris avalanches</u> ("hillslope debris flows") are extremely rapid (i.e. faster than 5 m/s) flow-like movements of fully or partially-saturated soil which has detached by sliding from a steep slope. The movement usually contains large proportion of vegetation, as well as boulders and soil in various degrees of saturation and/or liquefaction. The flow has not become significantly channelised (although many debris avalanches starting on upper slopes do enter channels and proceed as debris flows). Debris avalanches often occur as single surges. They may, or may not entrain significant quantities of additional material from the path. Debris avalanches have several special characteristics, relevant to hazard mitigation. They occur on open slopes, thus their width and precise path and impact location cannot be easily determined beforehand. They are often relatively shallow, but may possess certain internal rigidity due to incomplete mixing and the presence of root systems, so that fluid-like behaviour cannot be automatically assumed. The flow velocity and depth can be determined by a dynamic analysis of the propagation of an estimated landslide volume. Solid particles (boulders and tree trunks) must be assumed to move at a velocity equal to the mean velocity of the flow front. Debris avalanches can have high density (possibly greater than 2,000 kg/m^3). <u>Debris flows</u> (channelised) are very rapid to extremely rapid surging flows of saturated debris in steep, established channels	Ditto.

Table of Contents
Table of Contents
Table of Contents
Table of Contents

Comments	Responses to Comments
(often zero or first order drainage channels). They may begin with a relatively small instability on upper slopes, but build up larger volume by entrainment and liquefaction of material from the channelized path. They also increase their water content by entraining surface water. Debris flows follow established paths, so their impact location can be predicted. They occur in multiple surges. As shown by an example in Figure 1, a debris "event" in a channel may consist of a number of surges of momentarily increased discharge. Many of these may be merely translatory waves of water, magnified ("bulked") by large quantities of sediment. One or two of the largest surges, however, qualify as debris flow surges and, in that case, the entire event should be called a debris flow. We suggest that the classification be based on the ratio of the peak discharge of the largest surge and the peak discharge of an extreme hydrologic flood. If the ratio is more than 3 (and often ranging up to several tens), the surges qualify as debris flows. The peak discharge of debris flow surges is determined by surge-building processes and is thus unrelated to the hydrologic flood discharge. Lesser surges are debris flood, or simply water flood surges. The significance of debris flow surging towards hazard assessment is obvious. The surges are usually several metres deep, move with extremely rapid speed and are often fronted by accumulations of boulders or wood debris. Design of barriers to resist debris flows should be based on the largest expected discharge. This discharge can be determined from experienced judgment or by the use of empirical relationships. Alternatively, dynamic flow	Ditto.

Table of Contents
Table of Contents
Table of Contents
Table of Contents

Comments	Responses to Comments
analysis can be used, if the source and volume of the initial slide and the entrainment yield rate can be estimated. For predictions based on dynamic analysis, it is necessary to decide (subjectively) how many major surges will occur in a given event. A conservative analysis would obtain from the assumption that only one surge will be dominant and will transport most of the material. The scenario to be considered may need to include one where the main surge arrives first, as well as one where the main surge encounters a basin partially, or fully filled by earlier activity. Each design should be based on an explicitly defined set of surge flow scenario. The boulder velocity needs to be considered equal to the mean velocity of the largest surge, except in case of extremely large particles. Debris flow surges can have high density (approx. 2,000 kg/m^3). <u>Debris floods</u> are debris events which may transport large quantities of sediment, but in which the peak discharge never exceeds 3 times the peak discharge of an extreme water flood. Thus, debris flood design discharges can be estimated by appropriate "bulking" of a water hydrograph, estimated by hydrological techniques. The resulting discharges and associated impacts are usually much lower than debris flow discharges (an exception being in cases where the water discharge itself is unusually large, as a consequence of an accidental dam break or similar). Large clasts are transported by debris floods by means	Ditto.

Table of Contents
Table of Contents
Table of Contents
Table of Contents

Comments	Responses to Comments
of rolling and the impact velocity of such particles is usually much less than the peak mean velocity of the surge (Prochaska et al., 2008). Figure 2 shows an example of a debris flood which transported large boulders, but without exerting high impact forces on structures. Debris flood surges often travel to channel segments with low slope angle (<10°). The density of debris flood surges in motion is a strong function of solids concentration and may range from 1,000 to about 1,600 kg/m^3. Assigning a correct and conservative density value for design requires judgment. One example of potential problem is in the suggestion by Suda et al. (2009) to use a density of 2,100 kg/m^3 for "granular debris flows", while the same quantity for "muddy debris flows" is 1,150 kg/m^3. There must be intermediate phenomena between these two extremes. Debris flow surges usually begin to drop their load of coarse particles and reduce their discharge on slopes less than about 10°. Once this occurs, the flow may be modified to become a debris flood (also called "afterflow"). This may affect debris flow surges that pass through a deposition area. One possible beneficial outcome of this process is that debris passing through an open accumulation basin by design can be relatively easily diverted or channelised.	Ditto.
It is recommended that similar considerations be systematically incorporated into the design procedures.	Please see response above.

Table of Contents
Table of Contents
Table of Contents
Table of Contents

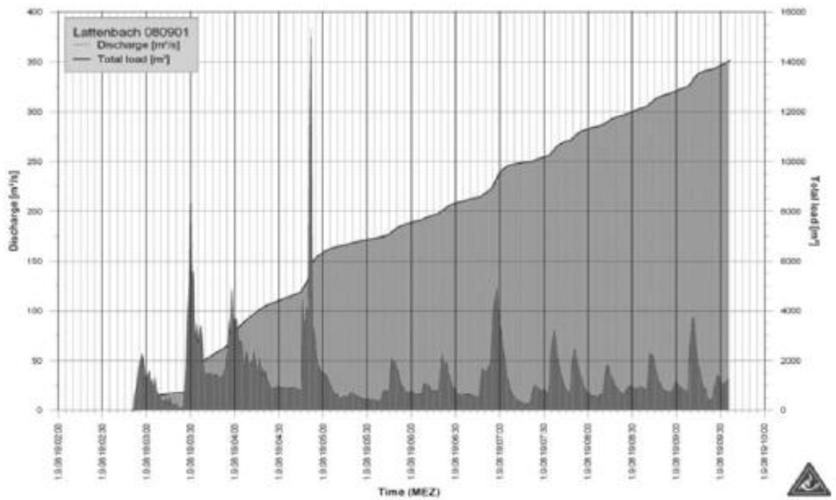
Comments	Responses to Comments
 Figure 1 Discharge hydrograph of a debris flow event (Hübl et al, 2009)	

Table of Contents
Table of Contents
Table of Contents
Table of Contents

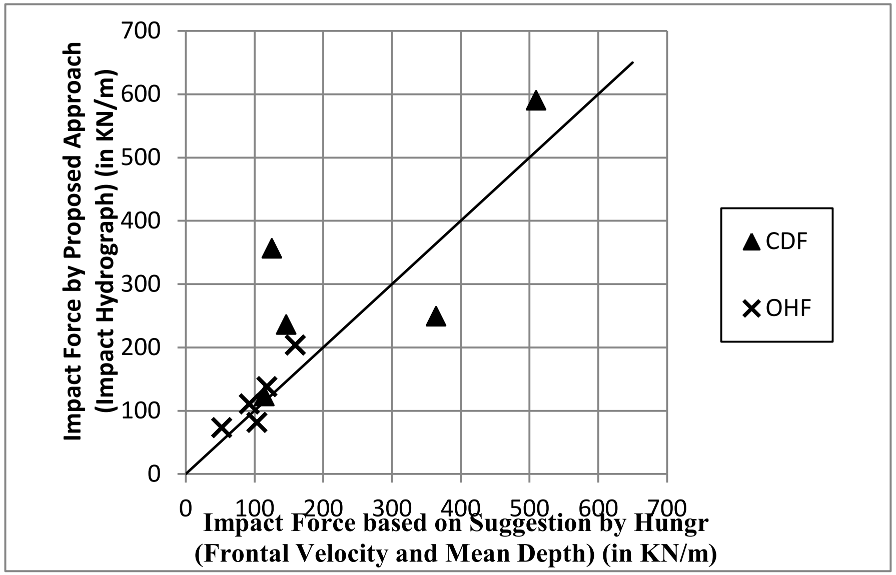
Comments	Responses to Comments
 Figure 2 A large boulder transported by a debris flood in Brazil, adjacent to a weak structure showing moderate impact forces. The channel slope at this location is approximately 5^{o} and the flow depth indicated by mud lines was between 2 and 3 metres. The flow velocity was probably not much greater than 5 m/s (estimated from the height of splash marks). **Par. 4, Surges:** Individual surges in debris flows are randomly spaced and the maximum impact may not be delivered by the first surge. For	Thank you for providing the discharge record of a site-specific debris flow event in Austria (Hübl et al, 2009) showing the discharge of

Table of Contents
Table of Contents
Table of Contents
Table of Contents

Comments	Responses to Comments
example, the record of Figure 1 indicates that, by far the largest discharges (and hence impacts) have been delivered by the second and fourth surge of this particular event. Similar scenarios are known from Canada as well. Since we do not know which part of an event will produce the highest loads, we should apply the design load at various levels of the structure. For high walls, this requirement may be too conservative and should be modified based on a specific estimated basin-filling scenario.	several distinct surges. In the design of a natural terrain landslide debris-resisting barrier in Hong Kong, the design debris volume is determined using the design event approach, and the debris is assumed to detach from the source area in one go and reach the proposed barrier in a single pulse. This approach as given in this Note is considered suitably robust given the many uncertainties involved and should be adequate to deal with debris events resulting from multiple detachments giving rise to distinct surges. The impact scenarios given in this Note require consideration of design load at various levels of a barrier structure. The recommendation is considered reasonably robust.
Par. 5 and 6, Superposition of thrust and point loads: The reviewed existing international standards are often rather unclear, regarding to whether the boulder point loads and the fluid thrust should be superimposed. The reason for this is that some of the recommended thrust loads are quite conservative and already contain an allowance for magnification due to granular impacts. The consequences of the loading should also be considered. For example, a large structure may be completely overturned by a major thrust load, but may suffer only relatively localised damage due to point loads. Also, large boulders often move at speeds that are less than the mean velocity of the flow (Prochaska et al., 2008).	 Noted. Given the limited literature available and the fact that boulders usually travel at the frontal end of debris flow as suggested by many field observations (Pudasaini & Hutter, 2007), the assumption of large boulders moving at a speed lower than the debris flow frontal velocity cannot be substantiated. To account for the uncertainties, it is considered prudent to adopt a more robust approach with boulder point loads and fluid thrust superimposed (i.e. the superposition of debris dynamic impact pressure and boulder impact load as proposed). Reference: Pudasaini, S. P. & K. Hutter (2007). *Dynamics of Rapid Flows of Dense Granular Avalanches*, Springer, New York, 602 p.
Par. 7, Flow depth and velocity: The selection of an appropriate combination of flow depth and velocity requires careful consideration. Where these parameters	 Analyses have been carried out to compare your suggested approach (i.e. use of the frontal velocity and the mean depth of the entire

Table of Contents
Table of Contents
Table of Contents
Table of Contents

<table>
<tr><th>Comments</th><th>Responses to Comments</th></tr>
<tr><td>are produced by an analytical unsteady flow model, it must be recognised that the shape of the flow front may not be correctly represented. Most existing models are based on the assumption of a homogeneous rheology and do not account for discharge magnification due to the building of a frontal boulder accumulation (e.g. Hungr, 2000). In particular, models based on the frictional rheology produce relatively high frontal speeds, but combined with low frontal depth (tapered fronts). Rate-dependent rheologies, such as the Voellmy model, produce deeper fronts. But, in fact, no existing dynamic model allows for the characteristic bulbous shape of a frontal boulder accumulation, a phenomenon that depends on certain random factors (cf. Hungr, 2000). We would recommend for the interim, that the velocity needed for impact calculations should be taken from the front of the model run, but the depth should be equal to the mean depth of the entire surge flow at the point of impact.</td><td>surge) and the approach given in the Note (i.e. using a combination of debris depth and velocity that corresponds to the maximum calculated debris impact load). The calculations are based on the runout profiles and design scenarios of ten LPMit projects. The results are shown below.



The results show that the maximum impact force calculated based on our suggested approach is greater in most of the cases. Nevertheless, it is observed that the impact force estimated by your suggested approach (i.e. use of frontal velocity and mean depth of the entire surge) is greater when the barrier is located well within debris deposition zone where the debris decelerates and debris surge becomes thicker.</td></tr>
</table>

Table of Contents
Table of Contents
Table of Contents
Table of Contents

Comments	Responses to Comments
	In view of the above, it is suggested that the greater value given by the two approaches should be adopted as the design debris impact load (see Section 4.2 of this Technical Note).
If the design of debris control structures is to be based on dynamic analysis, it becomes strongly dependent on calibration of the models. In particular, the correct selection of the appropriate rheology is important. There probably is not sufficient rational basis at present to recommend, for example, the frictional model or the Voellmy model for a given situation. Dynamic modeling has now been used in Hong Kong for more than 10 years. However, the only systematic collection of back-analyses is the report by Ayotte and Hungr (1998). We recommend that a new compilation of back-analyses of real cases be assembled, to reflect the extensive dynamic modeling experience that has accumulated in Hong Kong in recent years. The collection should be organized on the basis of the classification system mentioned earlier in this report. Maximum use should be made of detailed field observations, such as eyewitness reports and videos.	Systematic back-analyses of about 60 cases of channelised debris flows with runout distance exceeding 200 m in Hong Kong have been carried out. Results are discussed by Wong (2009). Further analyses have also been conducted to back analyse four massive, long-runout debris flow events occurred in 2008 (Kwan et al, 2011). The guidelines on the application of the rheology parameters for dynamic mobility analysis have been given in GEO Technical Guidance Note 29 - Guidelines on the Assessment of Debris Mobility for Channelised Debris Flows (GEO, 2011). We have recently undertaken a systematic back-analysis of the mobility of open hillside failures. Results will be compiled and reported in due course. We will consider organising the cases based on a more refined classification system. References: GEO (2011). *Guidelines on the Assessment of Debris Mobility for Channelised Debris Flows*, Technical Guidance Note 29. Geotechnical Engineering Office, Hong Kong, 6 p. Kean, J.S.H., Hui, T.H.H. & Ho, K.K.S. (2011). *Modelling the Motion of Mobile Debris Flows in Hong Kong*. *Proceedings of the Second World Landslide Forum*, Rome. Wong, H.N. (2009). Rising to the Challenges of Natural Terrain Landslides. *Proceedings of the HKIE Geotechnical Division*

Table of Contents
Table of Contents
Table of Contents
Table of Contents

Comments	Responses to Comments
	Annual Seminar on Natural Hillsides: Study and Risk Management Measures, Hong Kong Institution of Engineers, pp 15-53.
Par. 8: Impact scenarios: See comments under Par. 4 above.	Please refer to responses to comments on paragraph 4.
Figs. 1 and 2, Loading diagrams: The loading diagrams are considered appropriate.	Noted.
Par. 9, Partial safety factors: In order to reduce the conservativeness of the design, we agree with reducing the safety factors used with boulder loads to 1.0 (see Par. 31 below), according to the estimated consequences of global versus localized structural damage (see comments on Par. 5 and 6 above).	Noted. Your view has been incorporated into this Technical Note.
Par. 10-16: Impact scenarios: Agreed.	Noted.
Par. 17: Mistakes in formulas: Yes, there are frequent typos and mistakes in published formulas, so that all such formulas must be independently checked before being put to use or reproduced in GEO documents. For example, the Hertz Equation as quoted by VanDine (1996) places the symbol "g" where the original equation has the number 9 (Timoshenko and Goodier, 1969). Fortunately, this causes little problem in SI units. (Even some papers by Hungr have mistakes in formulas!).	Noted. We have carried out a dimensional analysis to verify the units that are used in the Hertz Equation.

Table of Contents
Table of Contents
Table of Contents
Table of Contents

Comments	Responses to Comments
Par. 18 and 20, Factoring the Hertz equation: High precision of the Hertz calculation is not required, since we acknowledge that the result is more than an order of magnitude too high for practical use.	Agreed.
Par. 19, Reduction of the Hertz impact load to 10%: Agreed.	Noted.
Par. 21, Use of the Hertz Equation in Taiwan: This approach from the Taiwan code does not seem to be advantageous.	Agreed. We have therefore proposed the superposition strategy.
Par. 22 and 23, Reduction of the Hertz impact load to 10%: Agreed.	Noted. It is also noted that Austrian practice recommends assessing structural damage of barrier based on boulder impact (see Section 2 of this Technical Note).
Par. 24, Point load safety factor: See comment under Par. 9.	Please refer to responses to comments on paragraph 9.
Par. 25 to 30: Dynamic thrust equations: The international practice recognises two distinct approaches to the determination of the dynamic thrust of a debris flow on structures: 1) Static approach (widely used in Austria, since the 1970's and also recommended by the research group of A. Armanini, e.g. 1997, based on small-scale testing): $F = \beta \rho g \frac{h^2}{2}$ ....(1)	Thank you for the useful discussion of the various approaches to the determination of the dynamic thrust of a debris flow on structures. We have incorporated the relevant points into the revised Note (see Section 4.1 of this Technical Note).

Table of Contents
Table of Contents
Table of Contents
Table of Contents

Comments	Responses to Comments
Here, F is the thrust force on a wall perpendicular to a flow with density ρ and depth h, and g is the gravity acceleration. In short, this is a form of the familiar hydrostatic pressure equation, magnified by a factor β to account for the dynamic effects of the flow impact. β varies widely in various sources. The Armanini group recommends β = 7 to 11. Equation 1 does not include velocity and this fact implies that the equation is valid only for a limited range of velocities.	Agreed. This is one of the drawbacks of the static approach.
The much more wide-spread "dynamic" approach is based on a simple form of the momentum equation, as used in hydraulics for calculating thrust on pipe bends and similar structures. $F = \alpha \rho v^2 h$ .... (2)	
This equation includes the velocity, v, and a multiplication factor α which varies usually between 1 and 3 (although some higher values have also appeared).	Noted.
There are several reasons for using a value of α different from 1 (as would be appropriate for a prismatic flow of an ideal fluid). Firstly, α could be less than 1.0, if it is the drag coefficient applying to an object that can be bypassed by the flow. Some measurements reported in the literature have been obtained on observation posts, so they may be too low if applied to an impact on a wall (e.g. Hu et al., 2011).	Agreed.
A value greater than 1.0 may result from a flow with internal shear strength, which may refuse to pass through a 90° direction change	Noted.

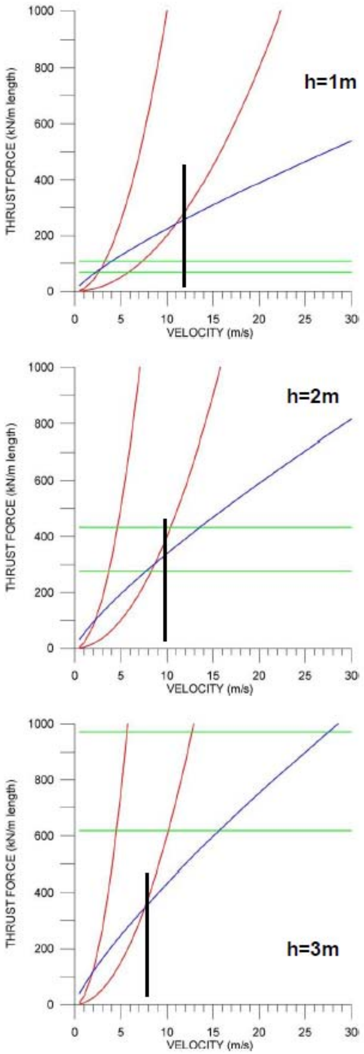
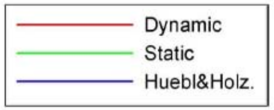
Table of Contents
Table of Contents
Table of Contents
Table of Contents

Comments	Responses to Comments
in the form of a jet, but instead "backs up", producing a backward-propagating bore. The added thrust from this scenario was studied by the Armanini group (e.g. 1997) and also recently by Sun and Law (2011).	
The second reason for increasing the dynamic thrust is the sum of small impacts from solid particles carried in the flow. When measured by small plate sensors, individual particle impacts can exceed the simple fluid thrust by more than an order of magnitude, even if the sensors are mounted on a narrow post (e.g. Hu et al., 2011).	Agreed.
The relationship between the two approaches is shown in Figure 3 for three values of flow depth, 1, 2 and 3m. The "static" thrust force is seen to be less conservative only at flow velocities of 5 to 8 m/s maximum. At velocities exceeding 10 to 15 m/s, the dynamic force exceeds the static estimate dramatically, even if the "correction factor", α, remains at 1.0. At velocities in excess of 20 m/sec, the dynamic force greatly exceeds all practically measured published values, producing pressures in excess of 800 kPa (80 tonnes per m^2). This is of concern, because certain open slope debris avalanches and debris flows may be quite rapid. For example, the Tsing Shan debris flow had an estimated maximum velocity of 10 - 15 m/s (King, 1996). The 2008 Yu Ting debris flow is seen on video moving for several hundred metres at an average speed of 10 m/s, with local velocity probably greater than this value. Natural terrain debris flows recorded by video in Seoul, Korea in the summer of 2011 are seen to be moving at more than 20 m/sec (see for example	Noted.

Table of Contents
Table of Contents
Table of Contents
Table of Contents

Comments	Responses to Comments
http://www.youtube.com/watch?v=L0ctCSZIHto&feature=youtube).	
Hübl et al. (2009) point out that most field measurements of debris flow thrust forces have been carried out at relatively low velocities, with Froude numbers generally less than 2.0. Only small-scale laboratory experiments, such as those carried out by Hübl and Holtzinger (2003) have been made at higher Froude numbers. The blue lines in Figure 3 show a pressure equation derived from these laboratory experiments by means of multi-variate regression. The line lies between the static and dynamic values for depths of 1 and 2 m. It appears that the Austrian Norms are about to adopt the Hübl–Holtzinger equation, but their final decision apparently has not yet been made.	Noted.
(In a recent personal message, Dr. J. Hübl informed us that the draft Austrian Norm ONR 24801 may be adopted within a few months. For now, however, it is still under review.)	Thank you for your information.

Table of Contents
Table of Contents
Table of Contents
Table of Contents

Comments	Responses to Comments
  Figure 3 Ranges of impact thrust forces estimated by various formulas: Dynamic: $F = \alpha \rho v^2 h$ (α = 1 to 5) Static: $F = \beta \rho g \frac{h^2}{2}$ (β = 7 to 11) Hübl and Holzinger (2003): $F = 4.5 \rho v^{0.8} (gh)^{0.6}$ In all cases, the debris density, ρ = 2Mg/m^3. h is the debris flow depth and F is the thrust force in kN per meter of barrier width. α and β are empirical factors. The black vertical line indicates Froude Number = 2.0	Noted

Comments	Responses to Comments
It is difficult to make a decision between these conflicting approaches, without measurements of impact in some major and extremely rapid debris flow events. In view of the above discussion, we conclude that the use of Equation [2], with an α of 2.5 and separate treatment of point impacts is reasonable for velocities of up to about 10 m/s. However, the pressure should perhaps be capped by some value (say, 500 kPa) for debris flow or avalanche impacts with greater speeds. The use of reduced Factors of Safety could also be considered to reduce the conservativeness of this approach.	It is noted that according to the design guidance in this Note, when the velocity of the debris is greater than 10 m/s, the pressure on the structure will be greater than 500 kPa (estimated based on equation $F = \alpha \rho v^2$ with α = 2.5 and debris density, ρ = 2 Mg/m^3). The above approach without capping the design dynamic impact pressure is considered more robust and appropriate at this stage given that available measurement results are limited.
Par. 31 and 32, Point load safety factor: See comments under Par. 9 above.	Please refer to responses to comments on paragraph 9.
Par. 33 and 34 Barrier run-up: Please note that recent experiments by Mancarella and Hungr (2010) indicate that the direct application of the Takahashi and Yoshida ("leading edge") model to run-up against steep barrier faces may be non-conservative. The DAN-W analysis results seem to be more robust.	The information has been incorporated in the revised Note (see Section 6.1 of this Technical Note).
When analysing retention of debris behind barriers or other obstructions that are not high enough and become over-run at high speed, trial DAN-W runs should be made with the "Trajectory" option turned on. Any dynamic models that force the flow to remain attached to the ground where it should launch into an air trajectory, may introduce errors in the analysis of subsequent flow and filling of the basin.	Agreed. We are preparing design guideline of multiple barriers in which debris mobility analysis to simulate projectile flight would be involved. Your comments will be incorporated in the corresponding design guideline accordingly.

Table of Contents
Table of Contents
Table of Contents
Table of Contents

Comments	Responses to Comments
Par. 35, Hydraulic jump behind a barrier: The equation given here is a hydraulic jump calculation, intended to ensure that debris “backed up” in the reservoir does not build up to such a depth as to overtop the barrier (Johánnesen et al., 2011). This is an overtopping scenario different from simple runup by the leading edge. Mancarella and Hungr (2010) have shown that Release 10 of DAN-W can simulate both overtopping scenaria in an approximate, but conservative fashion, when compared to laboratory tests.	Noted. The Note has been revised to suggest the use of an appropriate dynamic mobility model (see Section 6.1 of the Technical Note).
Annex A, Comments: We have nothing to add to most of the comments presented in Annex A, most of which are helpful and should be addressed. Two of the comments are considered especially important:	Noted.
CGE/I Comment 2(ii): The requirement to provide positive erosion protection on the downstream side of channel-crossing structures is very important. This protection should extend to the wings of barriers and check dams. Downstream undermining is one of the most common failure mechanisms for these structures.	Agreed. The comments will be followed up under a separate project on consolidating good practice in the detailing of rigid barriers.
Fugro comment: The concerns expressed by Fugro regarding the high velocities obtained from dynamic analyses are justified. Please refer to discussions under Par. 7 and Par. 25 to 30 above.	Noted. Please refer to responses above.
Comment on Equation: Equation B-4: $\tau = h\, \rho_d\, g \tan \phi$	We have carried out calculations to find out the equivalent friction

Table of Contents
Table of Contents
Table of Contents
Table of Contents

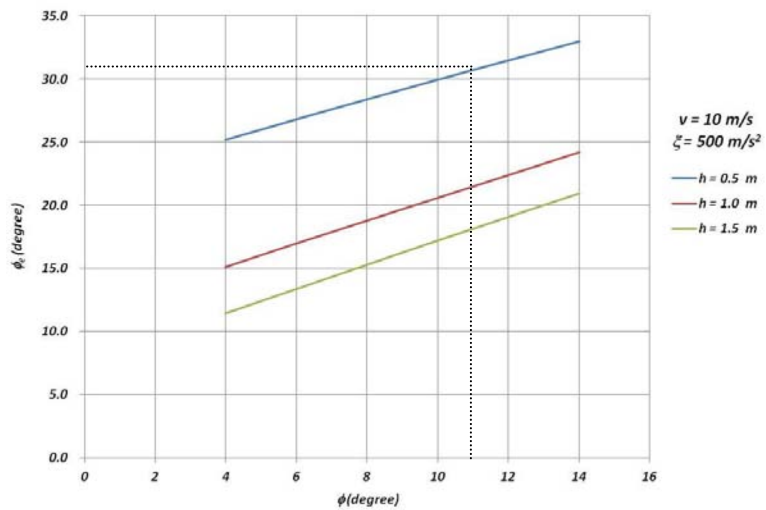
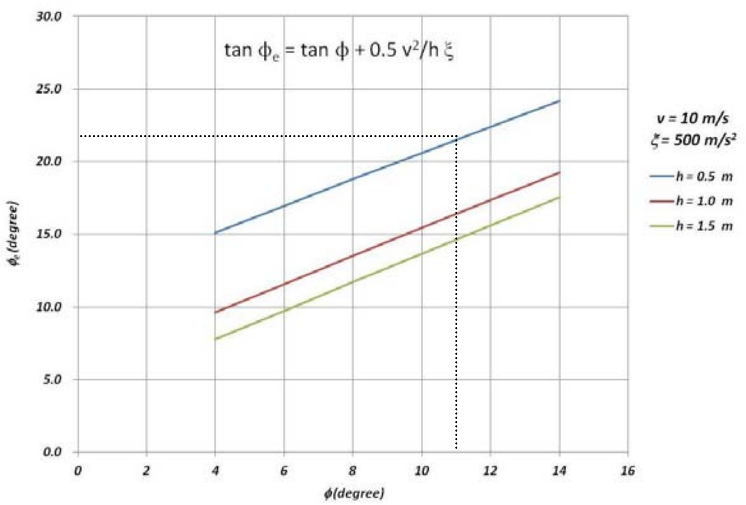
Comments	Responses to Comments
It should be made clear that the term ϕ in this equation should be the "friction slope" (i.e. the arc tangent of the ratio of resisting shear stress to total normal stress on the base of the flow). In some rheological models, such as the Voellmy model, this will be a function of velocity. DAN-W can output the shear stress directly in a file called STRENGTH.DAT at set time intervals.	angle (ϕ_e) which takes into account both the Columb friction (i.e. tan ϕ) and the Voellmy friction (i.e. v^2 / h ξ; v is the debris velocity, h is the debris height and ξ is the Voellmy coefficient) i.e. $\tan \phi_e = \tan \phi + v^2 / h\ \xi$ For a typical range of design flow depth between 0.5 m and 1.5 m, and v and ξ are taken as 10 m/s and 500 m/s^2 respectively, and it can be shown that ϕ_e could exceed 30^o (typically, the Veollmy parameters for debris mobility analysis of channelized debris flow would be taken as 11^o-500 m/s^2 by local practitioners, see also Figure below).  According to WSL (2009), the drag force induced by the overflow

Table of Contents
Table of Contents
Table of Contents
Table of Contents

<table>
<tr><th>Comments</th><th>Responses to Comments</th></tr>
<tr><td></td><td>

would be less than one-tenth of the normal force (i.e. the over-burden). It follows that the equivalent friction angle is tan-1(0.1) = 6^o.

We are of the opinion that Voellmy model estimates an ‘quivalent friction’ experienced by debris. The second terms takes into account the effects of turbulence within the debris flow as a kind of ‘friction’. Such a ‘friction’ would not be fully transmitted to the runout path in an action-and-reaction manner. It is suggested to estimate the drag force by superposition of the entire frictional component and notionally, half of the Veollmy component i.e. $\tan \phi_e = \tan \phi + 0.5\ v^2 / h\ \xi$. With this, the equivalent friction angle would be capped at around 20^o (see figure below and the revised equation).



</td></tr>
</table>

Table of Contents
Table of Contents
Table of Contents
Table of Contents

Comments	Responses to Comments
	Referece: WSL (2009). *Full-Scale Testing and Dimensioning of Flexible Debris Flow Barriers*. Swiss Federal Institute for Forest, Snow and Landscape Research (WSL), 22 p.

Table of Contents
Table of Contents
Table of Contents
Table of Contents

A.2 Comments on the Second Draft of the Report by Professor O. Hungr and Responses by GEO

Comments	Responses to Comments	Final Reply by Professor Hungr
In general, the Note answers the comments I made regarding the first draft, except:		
In Paragraph 12, I would again consider the full value of the friction slope corresponding to turbulent resistance from the Voellmy Model. There is no justification for reducing it by 50%.	Noted. Full value of the friction slope corresponding to turbulent resistance from the Voellmy Model will be considered in calculating the drag force. Equation 2.1 will be revised.	After some reflection, I agree that the barrier height can be limited to the velocity head when the hydraulic bore equation is used. My concern is that, when dealing with inclined channels, the debris volume may fill the available storage space and overtop the barrier, even if it is high enough to contain the initial run-up. The designer must ensure in this case that the available storage is sufficient.
In Paragraph 38, I think that the debris accumulation depth predicted by the hydraulic jump equation (Johanessesen) could lead to overtopping of the barrier and should be taken as the design value, even if it is greater than the velocity head. In case if inclined beds, DAN-W (if correctly used) can simulate the depth of debris accumulation behind the barrier, as shown by Mancarella and Hungr, 2010.	In theory, the velocity head would correspond to the upper bound of the runup height. Experimental data reported by Mancarella & Hungr (2010) also show that the measured runup heights are lower than the velocity heads. The rationale to use the hydraulic jump value where it is greater than the velocity head may not be strong. Our intention is to recommend the value be capped by the velocity head in such circumstance, as per the draft. We would be grateful for your more detailed elaboration of your suggestion.	

Table of Contents
Table of Contents
Table of Contents
Table of Contents

Appendix B

Dimensional Analysis and Simplified Form of Hertz Equation

Table of Contents

Table of Contents

Table of Contents

Table of Contents

Contents

Table of Contents
Table of Contents
Table of Contents

B.1 Hertz Equation

The Hertz Equation, for estimation of boulder impact load (F), presented by Zhang et al (1996) is shown below:

$$F = K_c\, n\, \alpha^{1.5} \quad \text{(B.1)}$$

where K_c is load reduction factor (dimensionless), and n and α can be calculated by the following equations.

$$n = 4\, r_b^{0.5} / 3\pi\, (k_b + k_B) \quad \text{(B.2)}$$

$$\alpha = (5\, m_b\, v_b^{2} / 4\, n)^{0.4} \quad \text{(B.3)}$$

r_b in Equation B.2 is radius of boulder, m_b is mass of boulder and v_b is velocity of boulder normal to barrier wall. k_b and k_B are coefficients related to the Young's moduli and Poisson ratios of boulder and barrier wall as shown in Equations B.4 and B.5 below:

$$k_b = (1 - \mu_b^{2}) / (\pi\, E_b) \quad \text{(B.4)}$$

$$k_B = (1 - \mu_B^{2}) / (\pi\, E_B) \quad \text{(B.5)}$$

where E_b = elastic modulus of boulder
E_B = elastic modulus of barrier
μ_b = Poisson ratio of boulder
μ_B = Poisson ratio of barrier

B.2 Dimensional Analysis

If SI units are applied to the input parameters, units of the input parameters involved in the above Equations are as follows ([**X**] denotes the unit of variable **X**):

$[r_b]$ = Metre (m)
$[m_b]$ = Kilogram (kg)
$[v_b]$ = Metre per second (m/s)
$[E_b]$ = Newton per unit area (N/m^2)
$[E_B]$ = Newton per unit area (N/m^2)
μ_b, μ_B and K_c are dimensionless

The units of n, k_b, k_B and α can be determined as follows:

$[k_b]$ = $[(E_b)^{-1}] = \text{m}^2 / \text{N}$
$[k_B]$ = $[(E_B)^{-1}] = \text{m}^2 / \text{N}$
$[n]$ = $[(r_b)^{0.5} / (k_b + k_B)] = (\text{m})^{0.5} / (\text{m}^2/\text{N}) = \text{N/m}^{1.5}$
$[\alpha]$ = $[(m_b)\, (v_b)^2 / n]^{0.4}$
= $[\text{kg}\, (\text{m/s})^2 / (\text{N/m}^{1.5})]^{0.4}$ = m (taking note that 'kg' can be converted to 'Ns^2/m')

Table of Contents Table of Contents Table of Contents Table of Contents

Substitute the above into Equation B.1 to obtain the unit of F,

$$[F] = [(n)\ (\alpha^{1.5})]$$
$$= (\text{N/m}^{1.5})\ (\text{m})^{1.5} = \text{N}$$

Therefore, it can be shown from the above analysis that the use of SI units is dimensionally correct.

B.3 Simplified Form of Hertz Equation

In practice, the Hertz equation may be simplified based on typical values of Poisson ratios and Young's moduli for boulder and rigid barrier, assuming that the boulder is spherical in shape as shown in Equation B.6:

$$m_b = 4 / 3\ \pi\ \rho\ r^3 \quad \text{(B.6)}$$

where ρ is density of boulder in kg/m^3

The typical values of Poisson ratios and Young's moduli assumed for boulder and rigid barrier are as follows:

E_b = 50×10^9 N/m^2 (i.e. 50 GPa)
E_B = 25×10^9 N/m^2 (i.e. 25 GPa)
μ_b = 0.2
μ_B = 0.2
ρ = 2650 kg/m^3

Based on the above assumptions for typical reinforced concrete barriers, the Hertz equation may be simplified as follow:

$$F = 4000\ K_c\ v_b^{1.2}\ r_b^2 \text{ (in kN)} \quad \text{(B.7)}$$

B.4 Reference

Zhang, S., Hungr, O. & Slaymaker, O. (1996). The calculation of impact force of boulders in debris flow. *Debris Flow Observation and Research*, edited by R. Du, Science Press, pp 67-72 (in Chinese).

Table of Contents
Table of Contents
Table of Contents
Table of Contents

Appendix C

Worked Example for Rigid Debris-resisting Barrier Design

Table of Contents
Table of Contents
Table of Contents
Table of Contents

Contents

List of Figures

Table of Contents
Table of Contents
Table of Contents

C.1 Introduction

A worked example of design of a rigid debris-resisting barrier is presented to illustrate the use of the recommended design guidance proposed in this Technical Note. The design parameters such as shear strength parameters of soil, debris density, debris bulking factor, angle of debris deposition, boulder diameter, elastic modulus, rheological parameters, etc. are selected arbitrarily and should not be regarded as generalised parameters for design of mitigation measures.

C.2 Design Event

The rigid barrier is designed to retain debris of a channelised debris flow with an active volume of 630 m^3 (source volume of 530 m^3 and entrainment volume of 100 m^3). The barrier will also be designed for impact by boulders. The volume of the boulders is assumed to be 1 m^3 each.

The source area is located at an elevation of about 240 mPD. The inclination of the runout profile ranges from 30° to 50°. The rigid barrier is located at about 170 mPD where the inclination reduces to a range between 10° and 20°. The plan distance between the source area and the barrier is about 150 m. The runout trail is 10 m wide. Figure C1 shows the profile of the runout trail.

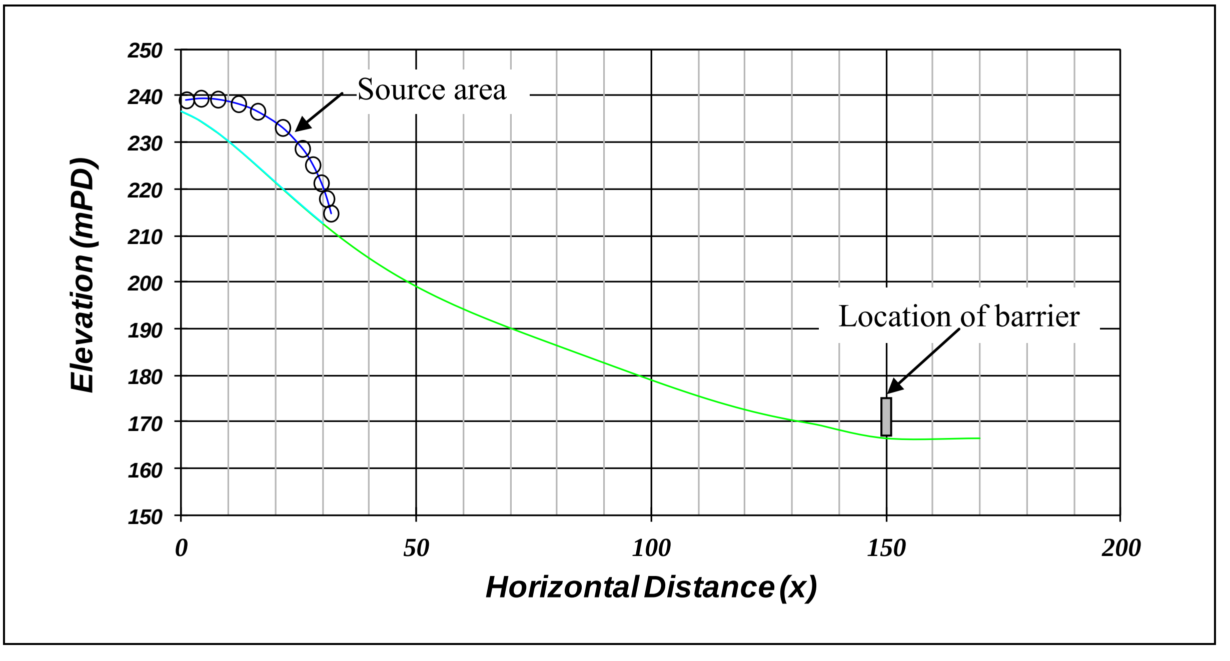


Figure C1 Runout Trail of the Design Event

Table of Contents
Table of Contents
Table of Contents
Table of Contents

C.3 Design of Rigid Barrier

C.3.1 Debris Mobility Analysis and Determination of Debris Impact Load

Debris mobility analysis is undertaken using computer program 2d-DMM (version 1.2). Voellmy rheological model of parameters 11^{o} - 500 m/s^{2} is adopted. Velocity and thickness hydrographs at the barrier location (x = 150 m) obtained from the analysis are shown in Figures C2 and C3 respectively. According to the hydrographs, debris reaches the barrier at 15 s after the onset of the landslide. The frontal velocity is about 6.8 m/s. Debris deaccelerates quickly and is brought to a complete stop at time ≈ 22 seconds. Not the entire debris 'chain' passes the barrier location. When debris stops, the thickness of the debris at the barrier location is about 2 m.

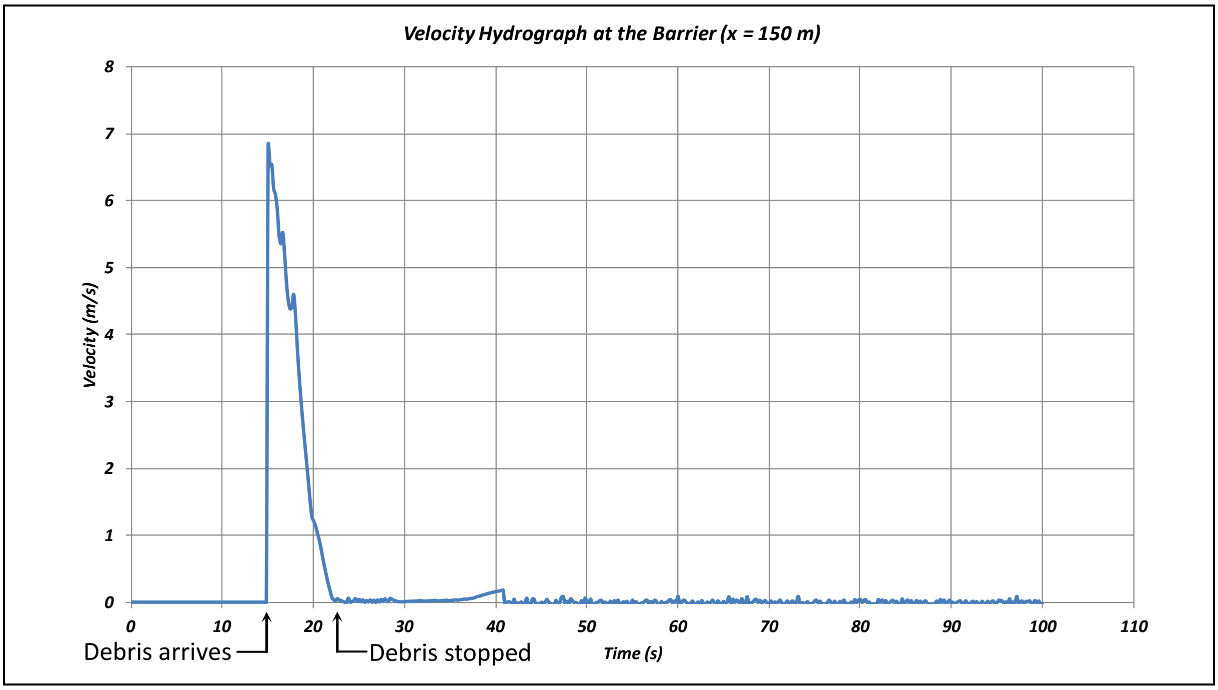


Figure C2 Velocity Hydrograph at x = 150 m

Table of Contents
Table of Contents
Table of Contents
Table of Contents

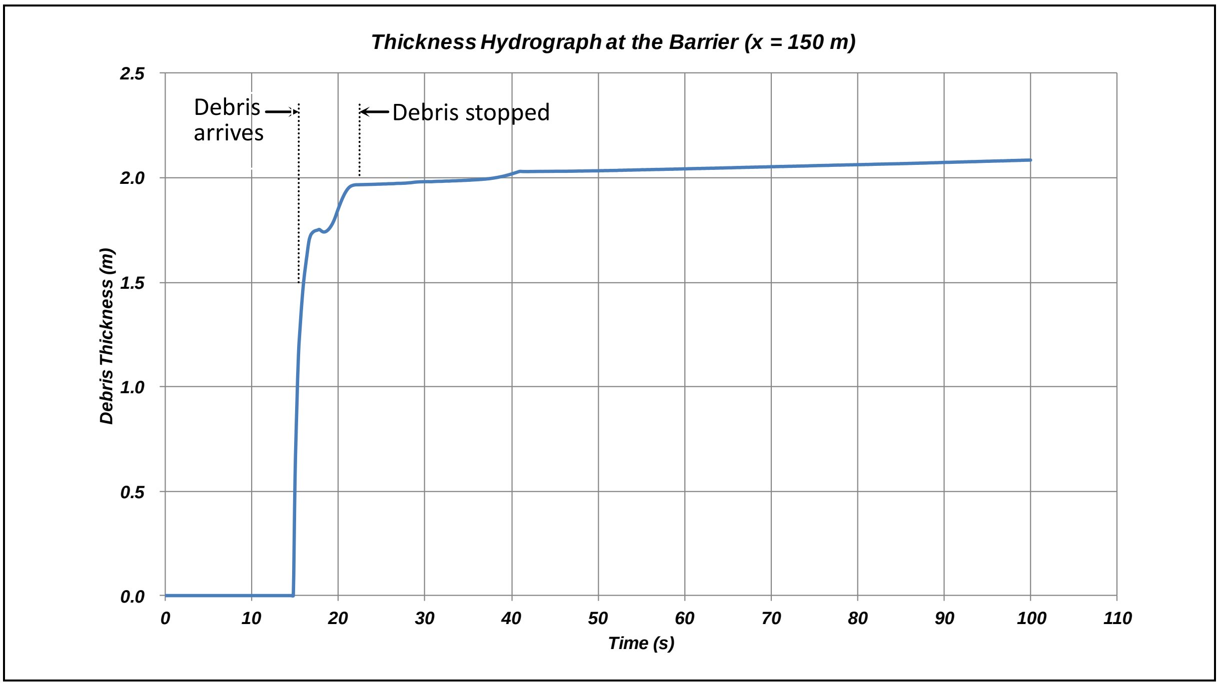


Figure C3 Thickness Hydrograph at *x* = 150 m

Debris impact force should be calculated using Equation (4.1). With combining the debris velocity and the debris thickness given by the hydrographs above, a time series of debris impact force can be established. Figure C4 shows the results, an assumed debris density of 1800 kg/m^3 is adopted. The maximum impact load obtained is 236 kN/m and the corresponding debris velocity and thickness are 6.1 m/s and 1.4 m respectively.

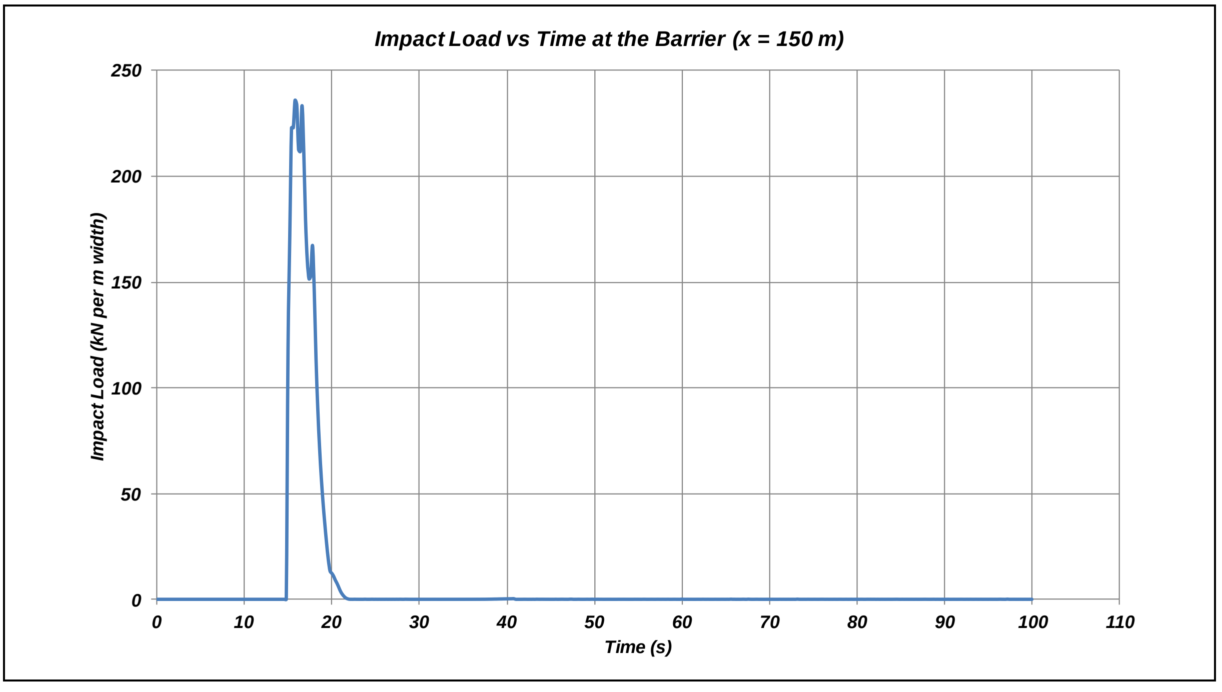


Figure C4 Impact Load Hydrograph at *x* = 150 m

Table of Contents
Table of Contents
Table of Contents
Table of Contents

The impact load calculated based on debris frontal velocity and the average debris thickness at the barrier location is also calculated as per Section 4.4(b)(ii) of this Technical Note. With the frontal velocity of 6.8 m/s (see Figure C2) and the average debris thickness of 1.6 m (see Figure C3), the calculated debris impact load is 344 kN/m.

Since the impact load calculated using the average debris thickness and the frontal velocity gives a greater debris impact force (i.e. 344 kN/m) comparing with that calculated based on combinations of *h* and *v* at different times (i.e. 236 kN/m), the design impact load is taken as 344 kN/m following recommendation given in Section 4.4(b). The design debris velocity and design debris thickness are 6.8 m/s and 1.6 m respectively.

C.3.2 Dimensioning the Rigid Barrier and Determination of Debris Impact Load

Figure C5 shows the geometry and dimensions of the barrier. The barrier is an inverted T-shaped reinforced concrete wall with a deflector equipped at the wall crest. The overall wall height is about 6 m and the width of the barrier is 11.5 m.

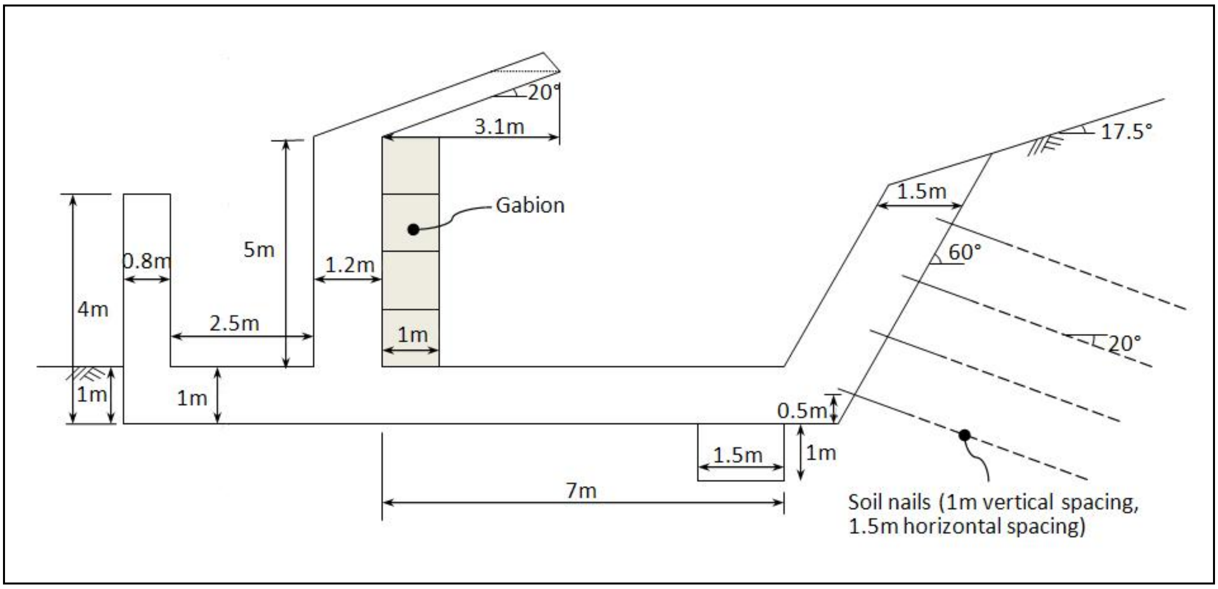


Figure C5 Dimensions of the Rigid Barrier

The wall is tied-back by four rows of soil nails. The design soil nail stabilisation forces are 110 kN/m for the top three rows and 120 kN/m for the row at the bottom. The forces are calculated following guidelines given in Geoguide 7 "Guide to Soil Nail Design and Construction". As this worked example focuses on the barrier design, detailed calculations of the soil nail stabilisation forces are not presented herein.

Table of Contents
Table of Contents
Table of Contents
Table of Contents

C.3.3 Checking of Retention Volume and Barrier Height

The procedures of checking the retention volume and barrier height are given as follows:

(a) The retention volume of the barrier is checked following the guidance provided in Section 7 of GEO Report 104 (Lo, 2000). The storage angle and bulking factor are taken as 2/3 of the channel bed angle and 8% respectively.

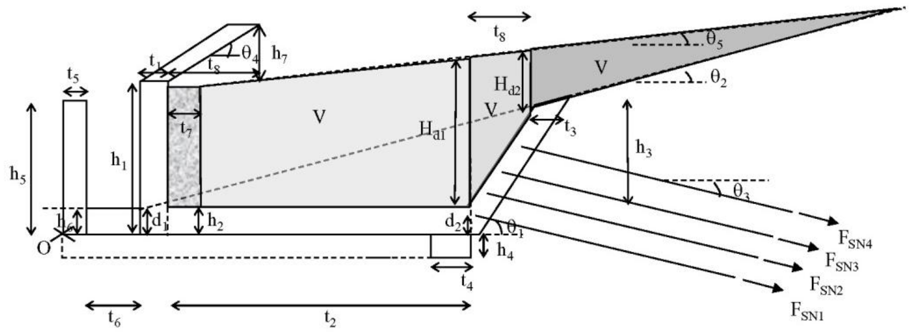
(b) Calculations show that retention volume of the barrier is 683 m^3, when debris is accumulated to 3.9 m high (see calculations on the following page). This is greater than the bulked design debris volume 680 m^3 (= 630×1.08%). The barrier is 5 m high, hence adequate retention volume is provided. A deflector is provided at the crest of the barrier wall, checking of debris run-up height against the design wall height is not required (Section 6.4(a) of this Technical Note).

(c) The design retaining height of debris, H_{ret} (see Figure 2.1(c) of this Technical Note), is 3.9 m, say 4 m.

Table of Contents
Table of Contents
Table of Contents
Table of Contents

Geotechnical Engineering Office
Job title: Worked Example for Design of Rigid Debris-resisting Barriers

Checking of Retention Volume behind the Concrete Barrier



N.T.S. (illustration only)

Design Event:
630m^3 debris volume with boulder fall (boulder volume < 1m^3)
Width of Barrier, L = 11.5 m

Soil Properties				Barrier Geometry		
concrete unit wt., γ_{conc} =	24	kN/m^3		wall stem 1 thickness, t_1 =	1.2	m
soil unit wt., γ_s =	18	kN/m^3		wall stem 1 height, h_1 =	4.9	m
soil saturated unit wt., γ_{sat} =	18	kN/m^3		wall base 1 thickness, h_2 =	1	m
water unit wt., γ_{wat} =	9.81	kN/m^3		wall base 1 width, t_2 =	7	m
				wall stem 2 thickness, t_3 =	1.5	m
ϕ' of back soil =	35	deg.		wall stem 2 height, h_3 =	3.16	m
c' of back soil =	0	kPa		key width, t_4 =	1.5	m
partial safety factor =	1.2			key depth, h_4 =	1	m
design ϕ' of back soil =	30.3	deg.		wall stem 3 thickness, t_5 =	0.8	m
design c' of back soil =	0.0	kPa		wall stem 3 height, h_5 =	4	m
K_a of back soil =	0.32	(Figure 18, Geoguide 1)		wall base 2 thickness, h_6 =	1	m
K_0 of back soil =	1.00			wall base 2 width, t_6 =	2.5	m
K_p of back soil =	5.1	(Figure 19, Geoguide 1)		embedded depth, d_1 =	1	m
				gabion block width, t_7 =	1	m
wall back friction angle, $\delta = 2\phi'/3$	20.18	deg.		coping length, t_8 =	3.1	m
		(Tables 14, Geoguide 1)		coping height, h_7 =	1.1	m
ϕ' of base soil =	34	deg.		soil nails above wall toe, d_2 =	0.5	m
c' of base soil =	5	kPa		θ_1 =	60	deg.
				θ_2 =	17.5	deg.
				θ_3 =	20	deg.
				θ_4 =	20	deg.

Checking of Retention Volume behind the Concrete Barrier

Geometry of accumulated debris mass

Storage angle , $\theta_5 = 2/3 \times \theta_2$ =	11.67	deg.	(Section 7, Geo report 104)
Back height of Debris V1, Hd_1 =	5.14	m	
Back height of Debris V2, Hd_2 =	2.35	m	
length of debris V2, t_8 =	1.82	m	
Retained volume of debris mass V1, =	27.12	m^3/m	
Retained volume of debris mass V2, =	6.84	m^3/m	
Retained volume of debris mass V3, =	25.48	m^3/m	

Total volume capacity of barrier V, = Length of barrier x (V1 + V2 + V3)
= 683.5 m^3
Volume of debris, V_d 630.0 m^3
Bulking factor of the debris, B = 8 % (Section 7, Geo report 104)
Designed debris volume, $V_{dd} = V_d(1+B)$ = 680.4 m^3
< capacity of the barrier, V OK

Table of Contents
Table of Contents
Table of Contents
Table of Contents

C.4 Stability Check

Since H_{ret} is 4 m high and the design debris thickness is 1.6 m, the retention zone behind the barrier will be filled up by 3 ($\because$ 4/1.6 = 2.5) debris surges.

C.4.1 Impact of the First Debris Surge

Details of checking barrier stability are given below:

(a) The debris impact load is 344 kN/m, and the corresponding design debris velocity and debris thickness are 6.8 m/s and 1.6 m respectively.

(b) A row of boulders is assumed to impact on the barrier. Each of the boulders has a volume of 1 m^3. The radius of each boulder is 0.62 m, calculated with an assumption that the boulders are spherical in shape. The boulder impact load is calculated based on the Hertz equation with $K_c = 0.1$ (see also Section 3.1 of this Technical Note). Gabion behind the concrete wall will be provided and the corresponding elastic modulus of gabion is also considered in the calculation. The velocity of the boulders is assumed to be the same as debris (i.e. 6.8 m/s). Since the radius of the boulder is smaller than the depth of debris, the boulders travel on top of the debris front (see also Figure 2.1 of this Technical Note).

(c) Design checks against sliding, overturning and bearing failure of the barrier have been conducted with the following assumptions:

- Lateral pressure coefficient of static debris is 1.0 (Section 2.1(b) of this Technical Note).

- Water uplift at wall base has been included in the design.

- Contribution of self weight of impacting debris is not considered for estimation of the base friction and restoring moment (see also Section 5.1(b) of this Technical Note).

- Surcharge effect due to the weight of impacting debris is considered for calculating lateral earth pressure and a load factor of 1.5 is applied (see also Section 5.1(b) of this Technical Note).

- Partial factors are applied to soil parameters in accordance with Geoguide 1 (see also Section 5.1(a) of this Technical Note).

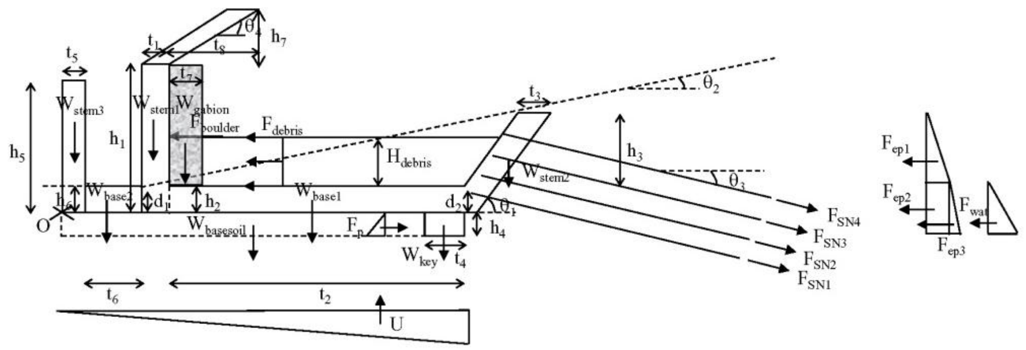
(d) Calculations of the stability check are presented in the following pages.

Table of Contents
Table of Contents
Table of Contents
Table of Contents

Geotechnical Engineering Office
Job title: Worked Example for Design of Rigid Debris-resisting Barriers

Stability Checking of Final Barrier (Overflow is not permitted)

Scenario 1 - Refer to Figure 2.1a of this Technical Note.
First impact, when the barrier is empty.



N.T.S. (illustration only)

Design Event:
630m^3 debris volume with boulder fall (boulder volume < 1m^3)

Debris Force Determination

impact depth, H_{debris} =	1.6	m	Report No. 104, Figure 21 or DMM model
wall height, h_1 =	6.0	m	
static soil depth, H_{sta} =	0	m	
debris impact velocity, v_d =	6.8	ms^{-1}	Report No. 104, Figure 21 or DMM model
density of debris, r_d =	1800	kg/m^3	
density of gabion, r_g =	1800	kg/m^3	
angle between velocity vector and surface of the barrier, β =	90	deg.	
debris impact force, F_{debris} =	**336**	kN/m	Equation 3 of Section 4 in this TN

Boulder Force Determination

density of boulder r_b =	2600	kg/m^3	
volume of boulder, r_b =	1.00	m^3	
mass of boulder, m_b =	2.60	Mg	
velocity of boulder, v_b =	6.83	ms^{-1}	
radius of boulder, r_b =	0.62	m	
Poisson's ratio of barrier,u_B =	0.18		
elastic modulus of barrier, E_B =	25	GPa	
Poisson' ratio of boulder, u_b =	0.2		
elastic modulus of boulder, E_b =	42	GPa	
Poisson's ratio of gabion blocks,u_I	0.18		
elastic modulus of gabion blocks,	0.3	GPa	
n =	3.23E+08		
Width of Barrier, L =	11.5	m	
Boulder Impact Force, $F_{boulder}$ =	**261**	kN/m	Equation B1 in Appendix B of this TN with K_c = 0.1

Soil Properties			Barrier Geometry		
concrete unit wt., γ_{conc} =	24	kN/m^3	wall stem 1 thickness, t_1 =	1.2	m
soil unit wt., γ_s =	18	kN/m^3	wall stem 1 height, h_1 =	6.0	m
soil saturated unit wt., γ_{sat} =	18	kN/m^3	wall base 1 thickness, h_2 =	1	m
water unit wt., γ_{wat} =	9.81	kN/m^3	wall base 1 width, t_2 =	7	m
ϕ' of back soil =	35	degree	wall stem 2 thickness, t_3 =	1.5	m
c' of back soil =	0	kPa	wall stem 2 height, h_3 =	3.16	m
partial safety factor =	1.2		key width, t_4 =	1.5	m
design ϕ' of back soil =	30.3	deg.	key depth, h_4 =	1	m
design c' of back soil =	0.0	kPa	wall stem 3 thickness, t_5 =	0.8	m
K_a of back soil =	0.32	(Figure 18, Geoguide 1)	wall stem 3 height, h_5 =	4	m
K_0 of back soil =	1.00		wall base 2 thickness, h_6 =	1	m

Table of Contents
Table of Contents
Table of Contents
Table of Contents

Geotechnical Engineering Office
Job title: Worked Example for Design of Rigid Debris-resisting Barriers

Stability Checking of Final Barrier (Overflow is not permitted)

K_p of back soil =	5.1	(Figure 20, Geoguide 1)	wall base 2 width, t_6 =	2.5	m
wall back friction angle, δ	20.18	deg.	embedded depth, d_1 =	1	m
ϕ' of base soil =	34	deg.	gabion block width, t_7 =	1	m
c' of base soil =	5	kPa	coping length, t_8 =	3.1	m
design ϕ' of base soil =	29.3	deg.	coping height, h_7 =	1.1283	m
design c' of base soil =	4.2	kPa	soil nails above wall toe, d_2 =	0.5	m
K_a of base soil =	0.32	(Figure 18, Geoguide 1)	θ_1 =	60	deg.
K_0 of base soil =	0.51		θ_2 =	17.5	deg.
K_p of base soil =	5.1	(Figure 19, Geoguide 1)	θ_3 =	20	deg.
wall base friction angle, δ_b	29.3	deg.	θ_4 =	20	deg.
wall base cohesion, c_b	4.2	kPa	soil nail vertical spacing =	1	m
			soil nail horizontal spacing =	1.5	m
			F_{SN1} =	110	kN/m
			F_{SN2} =	110	kN/m
			F_{SN3} =	110	kN/m
			F_{SN4} =	120	kN/m
			F_{SN5} =	0	kN/m
			pressure height (at the back slab	5.16	m
			water height (at the back slab)=	5.16	m

A. Check Factor of Safety against Sliding Failure

Force from Debris and Water

Type									=	Force (kN)	F_{h1} (kN)	F_{v1} (kN)
F_{debris} =									=	336	336	0
$F_{boulder}$ =									=	261	261	0
F_{ep1} =	0.5	x	0.32	x	18.00	x	0.00 ^2		=	0.00	0.00	0
F_{ep2} =	0.32	x	18.00	x	0.00	x	5.16		=	0.00	0.00	0
F_{ep3} =	0.50	x	0.32	x	8.19	x	5.16 ^2		=	34.90	34.90	0
F_{wat} =	0.50	x	9.81	x	5.16 ^2				=	130.64	130.64	0
F_p =	0.25	x	5.1	x	8.19	x	1.00 ^2		=	-10.4423	-10.44	0
											752	0.00

Force from Soil Nail

Type	=	Force (kN)	F_{nh} (kN)	F_{nv} (kN)
F_{SN1} =	=	110	-103.37	37.62
F_{SN2} =	=	110	-103.37	37.62
F_{SN3} =	=	110	-103.37	37.62
F_{SN4} =	=	120	-112.76	41.04
F_{SN5} =	=	0	0.00	0.00
			-423	153.91

Self Weight and Uplift Pressure

Type								=	Force (kN)	F_{h2} (kN)	F_{v2} (kN)
W_{stem1} =	1.2	x	6.0	x	24			=	172.80	0	172.80
W_{stem2} =	1.5	x	3.16	x	24			=	113.79	0	113.79
W_{stem3} =	0.8	x	4.0	x	24			=	76.80	0	76.80
W_{gabion} =	1	x	5.0	x	18			=	90.00	0	90.00
W_{coping} =	1.2	x	1.1	x	24			=	32.50	0	32.50
W_{base1} =	1	x	7	x	24			=	168.00	0	168.00
W_{base2} =	1	x	2.5	x	24			=	60.00	0	60.00
$W_{basesoil}$ =	1	x	10	x	18			=	180.00	0	180.00
W_{key} =	1.5	x	1	x	24			=	36.00	0	36.00
U =	0.5	x	5.16	x	9.81	x	11.5	=	-291.11	0	-291.11
										0	638.78

Total Sliding Force (F_{th}), F_{h1} + F_{h2} =	752	kN
Total Vertical Force (F_{tv}), F_{v1} +F_{nv} + F_{v2} =	792.68	kN

Table of Contents Table of Contents Table of Contents Table of Contents

Geotechnical Engineering Office
Job title: Worked Example for Design of Rigid Debris-resisting Barriers

Stability Checking of Final Barrier (Overflow is not permitted)

Total Sliding Resistance, F_{tv} x tan δ_b + c_b x wall base - F 916.34 kN

F.O.S against Sliding Failure = 1.22 **OK**

B. Check Factor of Safety against Overturning Failure

Overturning Moment about pt. O

Type	Force (kN)		Lever Arm (m)		Moment (kNm)
F_{debris} =	336	x	1.8	=	604.57
$F_{boulder}$ =	261	x	2.6	=	679.75
F_{ep1} =	0.00	x	4.16	=	0.00
F_{ep2} =	0.00	x	1.58	=	0.00
F_{ep3} =	34.90	x	0.72	=	25.14
F_{wat} =	130.64	x	0.72	=	94.10
U =	291.11	x	7.67	=	2231.85
			M_o	=	3635.40

Restoring Moment about pt. O

Type	Force (kN)		Lever Arm (m)		Moment (kNm)
W_{stem1} =	172.80	x	3.9	=	673.92
W_{stem2} =	113.79	x	12.70	=	1445.27
W_{stem3} =	76.80	x	0.40	=	30.72
W_{gabion} =	90.00	x	5.00	=	450.00
W_{coping} =	32.50	x	6.05	=	196.60
W_{base1} =	168.00	x	8	=	1344.00
W_{base2} =	60.00	x	2.05	=	123.00
W_{key} =	36.00	x	10.75	=	387.00
F_{SN1h} =	103.37	x	0.5	=	51.68
F_{SN2h} =	103.37	x	1.5	=	155.05
F_{SN3h} =	103.37	x	2.5	=	258.42
F_{SN4h} =	112.76	x	3.5	=	394.67
F_{SN5h} =	0.00	x	4.5	=	0.00
F_{SN1v} =	37.62	x	11.79	=	443.52
F_{SN2v} =	37.62	x	12.37	=	465.24
F_{SN3v} =	37.62	x	12.94	=	486.96
F_{SN4v} =	41.04	x	13.52		554.92
F_{SN5v} =	0.00	x	14.10		0.00
			M_r	=	7460.96

.O.S. against Overturning Failure = 2.05 **OK**

C. Check Factor of Safety against Bearing Failure

Vertical Force (N), F_{tv} - $W_{basesoil}$ =	612.68	kN					
Eccentricity e_b, B/2 - (M_r - M_o)/N =	0.71	m	<	B/6 =	2.32	m	**OK**
P = N/(B-2e_b) =	49.06	kPa	<	q_{all} =	284.48 (Appendix A, Geoguide 1)	kPa	**OK**

F.O.S. against Bearing Failure = 5.80 **OK**

Table of Contents Table of Contents Table of Contents Table of Contents

C.4.2 Impact of the Second Debris Surge

A second surge of debris riding on top of the stopped debris of the first surge is considered in this design scenario (see also Figure 2.1(b) of this Technical Note). The debris impact load, design debris velocity and debris thickness same as the first surge are adopted in the design calculations as per the recommendation stated in Section 2.1(a) of this Technical Note. The same boulder impact load is also included. The self weight of the impacting debris is neglected in the calculation of the stabilization forces against sliding and overturning. A partial load factor of 1.5 is applied to the lateral earth pressure induced by the self weight (see also Section 5.1(b)).

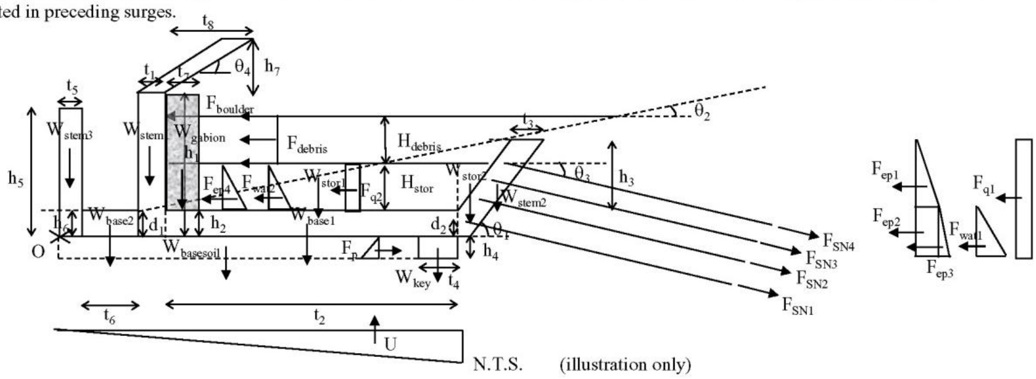
The design calculations are presented in the following pages.

Table of Contents
Table of Contents
Table of Contents
Table of Contents

Geotechnical Engineering Office
Job title: Worked Example for Design of Rigid Debris-resisting Barriers

Stability Checking of Final Barrier (Overflow is not permitted)

Scenario 2 - Refer to Figure 2.1a of this Technical Note.
Subsequent surges hitting the barrier. Design calculations should be carried out for debris impacts at different levels corresponding to the depth of the debris accumulated in preceding surges.



N.T.S. (illustration only)

Design Event:
630m^3 debris volume with boulder fall (boulder volume < 1m^3)

Debris Force Determination

impact depth, H_{debris} =	1.6	m	Report No. 104, Figure 21 or DMM model
wall height, h_1 =	6.0	m	
static soil depth after 1st surge, H_{stor} =	1.60	m	
debris impact velocity, v_d =	6.8	ms^{-1}	Report No. 104, Figure 21 or DMM model
density of debris, r_d =	1800	kg/m^3	
density of gabion, r_g =	1800	kg/m^3	
angle between velocity vector and surface of the barrier, β =	90	deg.	
debris impact force, F_{debris} =	**336**	kN/m	Equation 3 of Section 4 in this TN

Boulder Force Determination

density of boulder r_b =	2600	kg/m^3	
volume of boulder, r_b =	1.00	m^3	
mass of boulder, m_b =	2.60	Mg	
velocity of boulder, v_b =	6.8	ms^{-1}	
radius of boulder, r_b =	0.62	m	
Poisson's ratio of barrier, u_B =	0.18		
elastic modulus of barrier, E_B =	25	GPa	
Poisson' ratio of boulder, u_b =	0.2		
elastic modulus of boulder, E_b =	42	GPa	
Poisson's ratio of gabion blocks, u_B	0.18		
elastic modulus of gabion blocks, E	0.3	GPa	
n =	3.23E+08		
Width of Barrier, L =	11.5	m	
Boulder Impact Force, $F_{boulder}$ =	**261**	kN/m	Equation B1 in Appendix B of this TN with K_c = 0.1

Soil Properties				Barrier Geometry		
concrete unit wt., γ_{conc} =	24	kN/m^3		wall stem 1 thickness, t_1 =	1.2	m
soil unit wt., γ_s =	18	kN/m^3		wall stem 1 height, h_1 =	6	m
soil saturated unit wt., γ_{sat} =	18	kN/m^3		wall base thickness, h_2 =	1	m
water unit wt., γ_{wat} =	9.81	kN/m^3		wall base width, t_2 =	7	m
ϕ' of back soil =	35	deg.		wall stem 2 thickness, t_3 =	1.5	m
c' of back soil =	0	kPa		wall stem 2 height, h_3 =	3.16	m
partial safety factor =	1.2			key width, t_4 =	1.5	m
design ϕ' of back soil =	30.3	deg.		key depth, h_4 =	1	m
design c' of back soil =	0.0	kPa		wall stem 3 thickness, t_5 =	0.8	m
K_a of back soil =	0.32	(Figure 18, Geoguide 1)		wall stem 3 height, h_5 =	4	m
K_0 of back soil =	1			wall base 2 thickness, h_6 =	1	m
K_p of back soil =	5.1	(Figure 20, Geoguide 1)		wall base 2 width, t_6 =	2.5	m
wall back friction angle, δ	20.18	deg.		embedded depth, d_1 =	1	m
ϕ' of base soil =	34	deg.		gabion block width, t_7 =	1	m

Table of Contents
Table of Contents
Table of Contents
Table of Contents

Geotechnical Engineering Office
Job title: Worked Example for Design of Rigid Debris-resisting Barriers

Stability Checking of Final Barrier (Overflow is not permitted)

c' of base soil =	5	kPa	coping length, t_8 =	3.1	m
design ϕ' of base soil =	29.3	deg.	coping height, h_7 =	1.1	m
design c' of base soil =	4.2	kPa	soil nails above wall toe, d_2 =	0.5	m
K_a of base soil =	0.32	(Figure 18, Geoguide 1)	θ_1 =	60	deg.
K_0 of base soil =	0.51		θ_2 =	17.5	deg.
K_p of base soil =	5.1	(Figure 19, Geoguide 1)	θ_3 =	20	deg.
wall base friction angle, δ_b	29.3	deg.	θ_4 =	20	deg.
wall base cohesion, c_b	4.2	kPa	soil nail vertical spacing =	1	m
			soil nail horizontal spacing =	1.5	m
			F_{SN1} =	110	kN/m
			F_{SN2} =	110	kN/m
			F_{SN3} =	110	kN/m
			F_{SN4} =	120	kN/m
			F_{SN5} =	0	kN/m
			pressure height (at the back slab)	5.16	m
			water height (at the back slab)=	5.16	m
			pressure height (at the stem)=	1.60	m
			water height (at the stem)=	1.60	m

A. Check Factor of Safety against Sliding Failure

Force from Debris and Water

Type								=	Force (kN)	F_{h1} (kN)	F_{v1} (kN)
F_{debris} =								=	336	336	0
$F_{boulder}$ =								=	261	261	0
F_{ep1} =	0.5	x	0.32	x	18.00	x	0.00 ^2	=	0.00	0.00	0
F_{ep2} =	0.32	x	18.00	x	0.00	x	5.16	=	0.00	0.00	0
F_{ep3} =	0.50	x	0.32	x	8.19	x	5.16 ^2	=	34.90	34.90	0
F_{wat1} =	0.50	x	9.81	x	5.16 ^2			=	130.64	130.64	0
F_{q1} =	0.04	x	27	x	0.32	x	5.16	=	1.75	1.75	0
F_p =	0.25	x	5.1	x	8.19	x	1.00 ^2	=	-10.4423	-10.44	0
F_{ep4} =	0.50	x	1.00	x	8.19	x	1.60 ^2	=	10.48	10.48	0
F_{wat2} =	0.50	x	9.81	x	1.60 ^2			=	12.56	12.56	0
F_{q2} =	1.60	x	27	x	1.00	x	1.60	=	69.12	69.12	0
										846	0.00

Note: F_{q2} is surcharge effect due to the weight of impacting debris for calculating lateral earth pressure and a load factor of 1.5 is applied (see also Section 5.1(b) of this TN)

Force from Soil Nail

Type	=	Force (kN)	F_{nh} (kN)	F_{nv} (kN)
F_{SN1} =	=	110	-103.37	37.62
F_{SN2} =	=	110	-103.37	37.62
F_{SN3} =	=	110	-103.37	37.62
F_{SN4} =	=	120	-112.76	41.04
F_{SN5} =	=	0	0.00	0.00
			-423	153.91

Self Weight and Uplift Pressure

Type								=	Force (kN)	F_{h2} (kN)	F_{v2} (kN)
W_{stem1} =	1.2	x	6.0	x	24			=	172.80	0	172.80
W_{stem2} =	1.5	x	3.16	x	24			=	113.79	0	113.79
W_{stem3} =	0.8	x	4.0	x	24			=	76.80	0	76.80
W_{gabion} =	1	x	5.0	x	18			=	90.00	0	90.00
W_{coping} =	1.2	x	1.1	x	24			=	32.50	0	32.50
W_{base1} =	1	x	7	x	24			=	168.00	0	168.00
W_{base2} =	1	x	2.5	x	24			=	60.00	0	60.00
W_{stor1}=	6	x	1.6	x	18			=	172.80	0	172.80
W_{stor2} =	0.5	x	0.92	x	1.6	x	18	=	13.30	0	13.30
$W_{basesoil}$ =	1	x	10	x	18			=	180.00	0	180.00
W_{key} =	1.5	x	1	x	24			=	36.00	0	36.00
U =	0.5	x	5.16	x	9.81	x	11.5	=	-291.11	0	-291.11
										0	824.88

Table of Contents
Table of Contents
Table of Contents
Table of Contents

Geotechnical Engineering Office
Job title: Worked Example for Design of Rigid Debris-resisting Barriers

Stability Checking of Final Barrier (Overflow is not permitted)

Total Sliding Force (F_{th}), $F_{h1} + F_{h2}$ =	846	kN
Total Vertical Force (F_{tv}), $F_{v1} + F_{nv} + F_{v2}$ =	978.79	kN
Total Sliding Resistance, F_{tv} x tan δ_b + c_b x wall base - F_{nl}	1020.94	kN
F.O.S against Sliding Failure =	1.21	OK

B. Check Factor of Safety against Overturning Failure

Overturning Moment about pt. O

Type	Force (kN)		Lever Arm (m)		Moment (kNm)
F_{debris} =	336	x	3.40	=	1141.97
$F_{boulder}$ =	261	x	4.20	=	1097.43
F_{ep1} =	0.00	x	4.16	=	0.00
F_{ep2} =	0.00	x	1.58	=	0.00
F_{ep3} =	34.90	x	0.72	=	25.14
F_{wat1} =	130.64	x	0.72	=	94.10
F_{q1} =	1.75	x	1.58	=	2.76
F_{ep4} =	10.48	x	1.53	=	16.07
F_{wat2} =	12.56	x	1.53	=	19.25
F_{q2} =	69.12	x	1.80	=	124.42
U =	291.11	x	7.67	=	2231.85
			M_o	=	4752.98

Restoring Moment about pt. O

Type	Force (kN)		Lever Arm (m)		Moment (kNm)
W_{stem1} =	172.80	x	3.9	=	673.92
W_{stem2} =	113.79	x	12.70	=	1445.27
W_{stem3} =	76.80	x	0.40	=	30.72
W_{gabion} =	90.00	x	5.00	=	450.00
W_{coping} =	32.50	x	6.05	=	196.60
W_{base1} =	168.00	x	8	=	1344.00
W_{base2} =	60.00	x	2.05	=	123.00
W_{stor1} =	172.80	x	8.5	=	1468.80
W_{stor2} =	13.30	x	11.81	=	157.07
W_{key} =	36.00	x	10.75	=	387.00
F_{SN1h} =	103.37	x	0.5	=	51.68
F_{SN2h} =	103.37	x	1.5	=	155.05
F_{SN3h} =	103.37	x	2.5	=	258.42
F_{SN4h} =	112.76	x	3.5	=	394.67
F_{SN5h} =	0.00	x	4.5	=	0.00
F_{SN1v} =	37.62	x	11.79	=	443.52
F_{SN2v} =	37.62	x	12.37	=	465.24
F_{SN3v} =	37.62	x	12.94	=	486.96
F_{SN4v} =	41.04	x	13.52	=	554.92
F_{SN5v} =	0.00	x	14.10	=	0.00
			M_r	=	9086.83

F.O.S. against Overturning Failure = 1.91 **OK**

C. Check Factor of Safety against Bearing Failure

Vertical Force (N), F_{tv} - $W_{basesoil}$ =	798.79	kN					
Eccentricity e_b, B/2 - (M_r - M_o)/N =	1.53	m	<	B/6 =	2.32	m	OK
P = N/(B-2e_b) =	73.61	kPa	<	q_{all} =	246.62	kPa	OK
					(Appendix A, Geoguide 1)		

F.O.S. against Bearing Failure = 3.35 **OK**

Table of Contents
Table of Contents
Table of Contents
Table of Contents

C.4.3 Impact of the Third Debris Surge (The Final Surge)

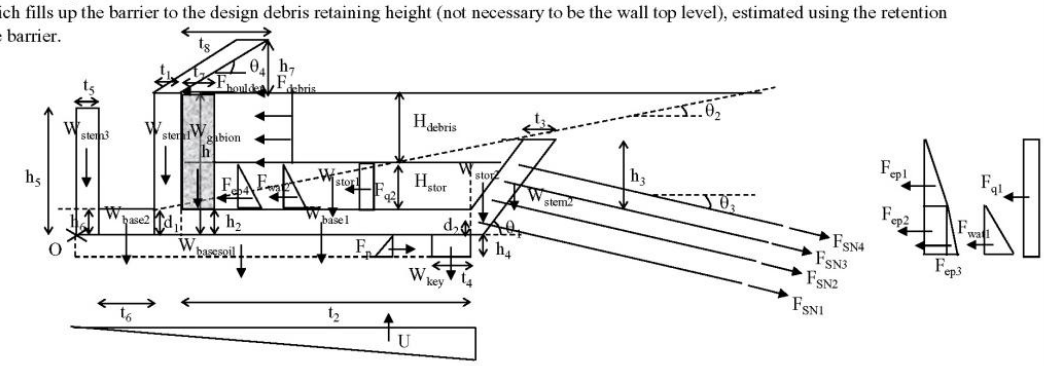
A surge of debris with the top level reaching the design retaining height of debris, H_{ret}, is considered in this design scenario (see also Figure 2.1(c) of this Technical Note). Debris impact load and boulder impact load same as the previous scenarios are applied.

The design calculations are presented in the following pages.

Geotechnical Engineering Office
Job title: Worked Example for Design of Rigid Debris-resisting Barriers

Stability Checking of Final Barrier (Overflow is not permitted)

Scenario 3 - Refer to Figure 2.1c of this Technical Note.
Last surge which fills up the barrier to the design debris retaining height (not necessary to be the wall top level), estimated using the retention capacity of the barrier.



N.T.S. (illustration only)

Design Event:
630m^3 debris volume with boulder fall (boulder volume < 1m^3)

Debris Force Determination

impact depth, H_{debris} =	1.6	m	Report No. 104, Figure 21 or DMM model
wall height, h_1 =	6.0	m	
static soil depth before last surge, H_{sto}	2.4	m	(H_{ret} - H_{debris})
debris impact velocity, v_d =	6.8	ms^{-1}	Report No. 104, Figure 21 or DMM model
density of debris, r_d =	1800	kg/m^3	
density of gabion, r_g =	1800	kg/m^3	
angle between velocity vector and surface of the barrier, β =	90	deg.	
debris impact force, F_{debris} =	**336**	kN/m	Equation 3 of Section 4 in this TN

Boulder Force Determination

density of boulder r_b =	2600	kg/m^3	
volume of boulder, r_b =	1.00	m^3	
mass of boulder, m_b =	2.60	Mg	
velocity of boulder, v_b =	6.8	ms^{-1}	
radius of boulder, r_b =	0.62	m	
Poisson's ratio of barrier,u_B =	0.18		
elastic modulus of barrier, E_B =	25	GPa	
Poisson' ratio of boulder, u_b =	0.2		
elastic modulus of boulder, E_b =	42	GPa	
Poisson's ratio of gabion blocks,u_B	0.18		
elastic modulus of gabion blocks, E	0.3	GPa	
n =	3.23E+08		
Width of Barrier, L =	11.5	m	
Boulder Impact Force, $F_{boulder}$ =	**261**	kN/m	Equation B1 in Appendix B of this TN with K_c = 0.1

Soil Properties

concrete unit wt., γ_{conc} =	24	kN/m^3	
soil unit wt., γ_s =	18	kN/m^3	
soil saturated unit wt., γ_{sat} =	18	kN/m^3	
water unit wt., γ_{wat} =	9.81	kN/m^3	
ϕ' of back soil =	35	deg.	
c' of back soil =	0	kPa	
partial safety factor =	1.2		
design ϕ' of back soil =	30.3	deg.	
design c' of back soil =	0.0	kPa	
K_a of back soil =	0.32		(Figure 18, Geoguide 1)
K_0 of back soil =	1		
K_p of back soil =	5.1		(Figure 20, Geoguide 1)
wall back friction angle, δ	20.2	deg.	
ϕ' of base soil =	34	deg.	
c' of base soil =	5	kPa	

Barrier Geometry

wall stem 1 thickness, t_1 =	1.2	m
wall stem 1 height, h_1 =	6	m
wall base thickness, h_2 =	1	m
wall base width, t_2 =	7	m
wall stem 2 thickness, t_3 =	1.5	m
wall stem 2 height, h_3 =	3.16	m
key width, t_4 =	1.5	m
key depth, h_4 =	1	m
wall stem 3 thickness, t_5 =	0.8	m
wall stem 3 height, h_5 =	4	m
wall base 2 thickness, h_6 =	1	m
wall base 2 width, t_6 =	2.5	m
embedded depth, d_1 =	1	m
gabion block width, t_7 =	1	m
coping length, t_8 =	3.1	m

Table of Contents Table of Contents Table of Contents Table of Contents

Geotechnical Engineering Office
Job title: Worked Example for Design of Rigid Debris-resisting Barriers

Stability Checking of Final Barrier (Overflow is not permitted)

design ϕ' of base soil =	29.3	deg.	coping height, h_7 =	1.1	m
design c' of base soil =	4.2	kPa	soil nails above wall toe, d_2 =	0.5	m
K_a of base soil =	0.32	(Figure 18, Geoguide 1)	θ_1 =	60	deg.
K_0 of base soil =	0.51		θ_2 =	17.5	deg.
K_p of base soil =	5.1	(Figure 19, Geoguide 1)	θ_3 =	20	deg.
wall base friction angle, δ_b	29.3	deg.	θ_4 =	20	deg.
wall base cohesion, c_b	4.2	kPa	soil nail vertical spacing =	1	m
			soil nail horizontal spacing =	1.5	m
			F_{SN1} =	110	kN/m
			F_{SN2} =	110	kN/m
			F_{SN3} =	110	kN/m
			F_{SN4} =	120	kN/m
			F_{SN5} =	0	kN/m
			pressure height (at the back slab)	5.16	m
			water height (at the back slab)=	5.16	m
			pressure height (at the stem)=	2.40	m
			water height (at the stem)=	2.40	m

A. Check Factor of Safety against Sliding Failure

Force from Debris and Water

Type									Force (kN)	F_{h1} (kN)	F_{v1} (kN)
F_{debris} =								=	336	336	0
$F_{boulder}$ =								=	261	261	0
F_{ep1} =	0.5	x	0.32	x	18.00	x	0.00 ^2	=	0.00	0.00	0
F_{ep2} =	0.32	x	18.00	x	0.00	x	5.16	=	0.00	0.00	0
F_{ep3} =	0.50	x	0.32	x	8.19	x	5.16 ^2	=	34.90	34.90	0
F_{wat1} =	0.50	x	9.81	x	5.16 ^2			=	130.64	130.64	0
F_{q1} =	1.84	x	27	x	0.32	x	5.16	=	82.01	82.01	0
F_p =	0.25	x	5.1	x	8.19	x	1.00 ^2	=	-10.4423	-10.44	0
F_{ep4} =	0.50	x	1.00	x	8.19	x	2.40 ^2	=	23.59	23.59	0
F_{wat2} =	0.50	x	9.81	x	2.40 ^2			=	28.25	28.25	0
F_{q2} =	1.60	x	27	x	1.00	x	2.40	=	103.68	103.68	0
										990	0.00

Force from Soil Nail

Type		Force (kN)	F_{nh} (kN)	F_{nv} (kN)
F_{SN1} =	=	110	-103.37	37.62
F_{SN2} =	=	110	-103.37	37.62
F_{SN3} =	=	110	-103.37	37.62
F_{SN4} =	=	120	-112.76	41.04
F_{SN5} =	=	0	0.00	0.00
			-423	153.91

Self Weight and Uplift Pressure

Type									Force (kN)	F_{h2} (kN)	F_{v2} (kN)
W_{stem1} =	1.2	x	6.0	x	24			=	172.80	0	172.80
W_{stem2} =	1.5	x	3.16	x	24			=	113.79	0	113.79
W_{stem3} =	0.8	x	4.0	x	24			=	76.80	0	76.80
W_{gabion} =	1	x	5.0	x	18			=	90.00	0	90.00
W_{coping} =	1.2	x	1.1	x	24			=	32.50	0	32.50
W_{base1} =	1	x	7	x	24			=	168.00	0	168.00
W_{base2} =	1	x	2.5	x	24			=	60.00	0	60.00
W_{stor1}=	6	x	2.4	x	18			=	259.20	0	259.20
W_{stor2} =	0.5	x	1.39	x	2.4	x	18	=	29.93	0	29.93
$W_{basesoil}$ =	1	x	10	x	18			=	180.00	0	180.00
W_{key} =	1.5	x	1	x	24			=	36.00	0	36.00
U =	0.5	x	5.16	x	9.81	x	11.5	=	-291.11	0	-291.11
										0	927.91

Total Sliding Force (F_{th}), F_{h1} + F_{h2} =	990	kN
Total Vertical Force (F_{tv}), F_{v1} + F_{nv} + F_{v2} =	1081.81	kN

Table of Contents Table of Contents Table of Contents Table of Contents

Geotechnical Engineering Office
Job title: Worked Example for Design of Rigid Debris-resisting Barriers

Stability Checking of Final Barrier (Overflow is not permitted)

Total Sliding Resistance, F_{tv} x tan δ_b + c_b x wall base - F_{nl} 1078.86 kN

F.O.S against Sliding Failure = 1.09 **OK**

B. Check Factor of Safety against Overturning Failure

Overturning Moment about pt. O

Type	Force (kN)		Lever Arm (m)		Moment (kNm)
F_{debris} =	336	x	4.20	=	1410.66
$F_{boulder}$ =	261	x	5.00	=	1306.46
F_{ep1} =	0.00	x	4.16	=	0.00
F_{ep2} =	0.00	x	1.58	=	0.00
F_{ep3} =	34.90	x	0.72	=	25.14
F_{wat1} =	130.64	x	0.72	=	94.10
F_{q1} =	82.01	x	1.58	=	129.61
F_{ep4} =	23.59	x	1.80	=	42.46
F_{wat2} =	28.25	x	1.80	=	50.86
F_{q2} =	103.68	x	2.20	=	228.10
U =	291.11	x	7.67	=	2231.85
			M_o	=	5519.22

Restoring Moment about pt. O

Type	Force (kN)		Lever Arm (m)		Moment (kNm)
W_{stem1} =	172.80	x	3.9	=	673.92
W_{stem2} =	113.79	x	12.70	=	1445.27
W_{stem3} =	76.80	x	0.40	=	30.72
W_{gabion} =	90.00	x	5.00	=	450.00
W_{coping} =	32.50	x	6.05	=	196.60
W_{base1} =	168.00	x	8	=	1344.00
W_{base2} =	60.00	x	2.05	=	123.00
W_{stor1} =	259.20	x	8.5	=	2203.20
W_{stor2} =	29.93	x	11.96	=	358.02
W_{key} =	36.00	x	10.75	=	387.00
F_{SN1h} =	103.37	x	0.5	=	51.68
F_{SN2h} =	103.37	x	1.5	=	155.05
F_{SN3h} =	103.37	x	2.5	=	258.42
F_{SN4h} =	112.76	x	3.5	=	394.67
F_{SN5h} =	0.00	x	4.5	=	0.00
F_{SN1v} =	37.62	x	11.79	=	443.52
F_{SN2v} =	37.62	x	12.37	=	465.24
F_{SN3v} =	37.62	x	12.94	=	486.96
F_{SN4v} =	41.04	x	13.52	=	554.92
F_{SN5v} =	0.00	x	14.10	=	0.00
			M_r	=	10022.17

F.O.S. against Overturning Failure = 1.82 **OK**

C. Check Factor of Safety against Bearing Failure

Vertical Force (N), F_{tv} - $W_{basesoil}$ =	901.81	kN					
Eccentricity e_b, B/2 - (M_r - M_o)/N =	1.96	m	<	B/6 =	2.32	m	**OK**
P = N/(B-2e_b) =	90.30	kPa	<	q_{all} =	174.06	kPa	**OK**

(Appendix A, Geoguide 1)

F.O.S. against Bearing Failure = 1.93 **OK**

Table of Contents Table of Contents Table of Contents Table of Contents

C.4.4 Debris Flow Impacting at the Top of the Barrier

Stabliity check against the impact scenario shown in Figure 2.2(a) of this Technical Note should be carried out as a good practice (Section 2.1(c)). A surge of debris with top level flushing with the wall crest is considered in this design scenario. Debris thickness and debris velocity same as the previous scenarios are adopted. In addition, the same debris impact load and boulder impact are applied.

Partial factor of safety on the shear strength parameters of soil is taken as 1.1 (see Section 5.1(d) of the Technical Note).

The design calculations are presented in the following pages.

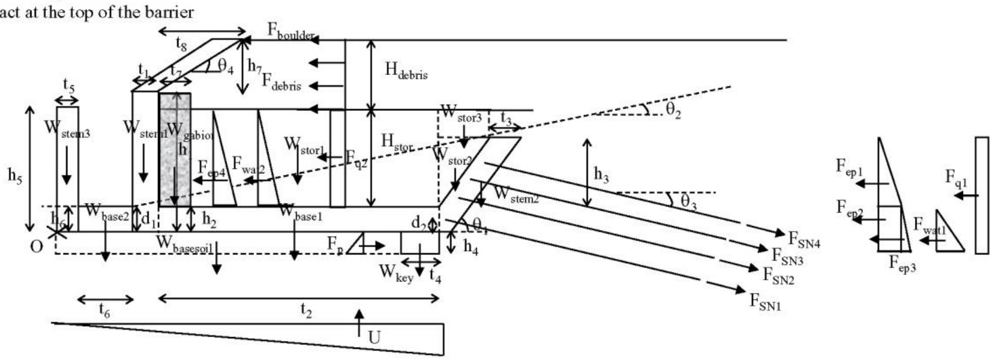
Table of Contents
Table of Contents
Table of Contents
Table of Contents

Table of Contents

Geotechnical Engineering Office
Job title: Worked Example for Design of Rigid Debris-resisting Barriers

Stability Checking of Final Barrier (Overflow is not permitted)

Scenario 4 - Refer to Figure 2.2a of this Technical Note.
Debris flow impact at the top of the barrier



N.T.S. (illustration only)

Design Event:
630m^3 debris volume with boulder fall (boulder volume < 1m^3)

Debris Force Determination

impact depth, H_{debris} =	1.6	m	Report No. 104, Figure 21 or DMM model
wall height, h_1 =	6.0	m	
static soil depth, H_{stor} =	4.0	m	Equal to H_{ret}
debris impact velocity, v_d =	6.8	ms^{-1}	Report No. 104, Figure 21 or DMM model
density of debris, r_d =	1800	kg/m^3	
density of gabion, r_g =	1800	kg/m^3	
angle between velocity vector and surface of the barrier, β =	90	deg.	
debris impact force, F_{debris} =	**336**	kN/m	Equation 3 of Section 4 in this TN

Boulder Force Determination

density of boulder r_b =	2600	kg/m^3	
volume of boulder, r_b =	1.00	m^3	
mass of boulder, m_b =	2.60	Mg	
velocity of boulder, v_b =	6.83	ms^{-1}	
radius of boulder, r_b =	0.62	m	
Poisson's ratio of barrier,u_B =	0.18		
elastic modulus of barrier, E_B =	25	GPa	
Poisson' ratio of boulder, u_b =	0.2		
elastic modulus of boulder, E_b =	42	GPa	
Poisson's ratio of gabion blocks,u_B	0.18		
elastic modulus of gabion blocks, l	0.3	GPa	
n =	3.23E+08		
Width of Barrier, L =	11.5	m	
Boulder Impact Force, $F_{boulder}$ =	**261**	kN/m	Equation B1 in Appendix B of this TN with K_c = 0.1

Soil Properties

concrete unit wt., γ_{conc} =	24	kN/m^3	
soil unit wt., γ_s =	18	kN/m^3	
soil saturated unit wt., γ_{sat} =	18	kN/m^3	
water unit wt., γ_{wat} =	9.81	kN/m^3	
ϕ' of back soil =	35	deg.	
c' of back soil =	0	kPa	
partial safety factor =	1.1		
design ϕ' of back soil =	32.5	deg.	
design c' of back soil =	0.0	kPa	
K_a of back soil =	0.28		(Figure 18, Geoguide 1)
K_0 of back soil =	1		
K_p of back soil =	5.5		(Figure 20, Geoguide 1)
wall back friction angle, δ	16.2	deg.	
ϕ' of base soil =	34	deg.	
c' of base soil =	5	kPa	
design ϕ' of base soil =	31.5	deg.	

Barrier Geometry

wall stem 1 thickness, t_1 =	1.2	m
wall stem 1 height, h_1 =	6	m
wall base thickness, h_2 =	1	m
wall base width, t_2 =	7	m
wall stem 2 thickness, t_3 =	1.5	m
wall stem 2 height, h_3 =	3.16	m
key width, t_4 =	1.5	m
key depth, h_4 =	1	m
wall stem 3 thickness, t_5 =	0.8	m
wall stem 3 height, h_5 =	4	m
wall base 2 thickness, h_6 =	1	m
wall base 2 width, t_6 =	2.5	m
embedded depth, d_1 =	1	m
gabion block width, t_7 =	1	m
coping length, t_8 =	3.1	m
coping height, h_7 =	1.1	m

Table of Contents
Table of Contents
Table of Contents

Table of Contents

Geotechnical Engineering Office
Job title: Worked Example for Design of Rigid Debris-resisting Barriers

Stability Checking of Final Barrier (Overflow is not permitted)

design c' of base soil =	4.5	kPa	soil nails above wall toe, d_2 =	0.5	m
K_a of base soil =	0.28	(Figure 18, Geoguide 1)	θ_1 =	60	deg.
K_0 of base soil =	0.48		θ_2 =	17.5	deg.
K_p of base soil =	5.7	(Figure 19, Geoguide 1)	θ_3 =	20	deg.
wall base friction angle, δ_b	31.5	deg.	θ_4 =	20	deg.
wall base cohesion, c_b	4.5	kPa	soil nail vertical spacing =	1	m
			soil nail horizontal spacing =	1.5	m
			F_{SN1} =	110	kN/m
			F_{SN2} =	110	kN/m
			F_{SN3} =	110	kN/m
			F_{SN4} =	120	kN/m
			F_{SN5} =	0	kN/m
			pressure height (at the back slab	5.16	m
			water height (at the back slab)=	5.16	m
			pressure height (at the stem)=	4.00	m
			water height (at the stem)=	4.00	m

A. Check Factor of Safety against Sliding Failure

Force from Debris and Water

Type									Force (kN)	F_{h1} (kN)	F_{v1} (kN)
F_{debris} =								=	336	336	0
$F_{boulder}$ =								=	261	261	0
F_{ep1} =	0.5	x	0.28	x	18.00	x	0.00 ^2	=	0.00	0.00	0
F_{ep2} =	0.28	x	18.00	x	0.00	x	5.16	=	0.00	0.00	0
F_{ep3} =	0.50	x	0.28	x	8.19	x	5.16 ^2	=	30.54	30.54	0
F_{wat1} =	0.50	x	9.81	x	5.16 ^2			=	130.64	130.64	0
F_{q1} =	2.44	x	27	x	0.28	x	5.16	=	95.17	95.17	0
F_p =	0.25	x	5.7	x	8.19	x	1.00 ^2	=	-11.6708	-11.67	0
F_{ep4} =	0.50	x	1.00	x	8.19	x	4.00 ^2	=	65.52	65.52	0
F_{wat2} =	0.50	x	9.81	x	4.00 ^2			=	78.48	78.48	0
F_{q2} =	1.60	x	27	x	1.00	x	4.00	=	172.80	172.80	0
										1159	0.00

Force from Soil Nail

Type		Force (kN)	F_{nh} (kN)	F_{nv} (kN)
F_{SN1} =	=	110	-103.37	37.62
F_{SN2} =	=	110	-103.37	37.62
F_{SN3} =	=	110	-103.37	37.62
F_{SN4} =	=	120	-112.76	41.04
F_{SN5} =	=	0	0.00	0.00
			-423	153.91

Self Weight and Uplift Pressure

Type									Force (kN)	F_{h2} (kN)	F_{v2} (kN)
W_{stem1} =	1.2	x	6.0	x	24			=	172.80	0	172.80
W_{stem2} =	1.5	x	3.16	x	24			=	113.79	0	113.79
W_{stem3} =	0.8	x	4.0	x	24			=	76.80	0	76.80
W_{gabion} =	1	x	5.0	x	18			=	90.00	0	90.00
W_{coping} =	1.2	x	1.1	x	24			=	32.50	0	32.50
W_{base1} =	1	x	7	x	24			=	168.00	0	168.00
W_{base2} =	1	x	2.5	x	24			=	60.00	0	60.00
W_{stor1} =	6	x	4.00	x	18			=	432.00	0	432.00
W_{stor2} =	0.5	x	1.82	x	3.16	x	18	=	51.91	0	51.91
W_{stor3} =	1	x	1.82	x	0.84	x	18	=	27.56	0	27.56
$W_{basesoil}$ =	1	x	10	x	18			=	180.00	0	180.00
W_{key} =	1.5	x	1	x	24			=	36.00	0	36.00
U =	0.5	x	5.16	x	9.81	x	11.5	=	-291.11	0	-291.11
										0	1150.25

Total Sliding Force (F_{th}), F_{h1} + F_{h2} =	1159	kN
Total Vertical Force (F_{tv}), F_{v1} + F_{nv} + F_{v2} =	1304.16	kN
Total Sliding Resistance, F_{tv} x tan δ_b + c_b x wall base - F	1274.83	kN

Geotechnical Engineering Office
Job title: Worked Example for Design of Rigid Debris-resisting Barriers

Stability Checking of Final Barrier (Overflow is not permitted)

F.O.S against Sliding Failure = 1.10 **OK**

B. Check Factor of Safety against Overturning Failure

Overturning Moment about pt. O

Type	Force (kN)		Lever Arm (m)		Moment (kNm)
F_{debris} =	336	x	5.80	=	1948.06
$F_{boulder}$ =	261	x	6.60	=	1724.53
F_{ep1} =	0.00	x	4.16	=	0.00
F_{ep2} =	0.00	x	1.58	=	0.00
F_{ep3} =	30.54	x	0.72	=	22.00
F_{wat1} =	130.64	x	0.72	=	94.10
F_{q1} =	95.17	x	1.58	=	150.40
F_{ep4} =	65.52	x	2.33	=	152.88
F_{wat2} =	78.48	x	2.33	=	183.12
F_{q2} =	172.80	x	3.00	=	518.40
U =	291.11	x	7.67	=	2231.85
			M_o	=	7025.33

Restoring Moment about pt. O

Type	Force (kN)		Lever Arm (m)		Moment (kNm)
W_{stem1} =	172.80	x	3.9	=	673.92
W_{stem2} =	113.79	x	12.70	=	1445.27
W_{stem3} =	76.80	x	0.40	=	30.72
W_{gabion} =	90.00	x	5.00	=	450.00
W_{coping} =	32.50	x	6.05	=	196.60
W_{base1} =	168.00	x	8	=	1344.00
W_{base2} =	60.00	x	2.05	=	123.00
W_{stor1} =	432.00	x	8.5	=	3672.00
W_{stor2} =	51.91	x	12.11	=	628.60
W_{stor3} =	27.56	x	12.41	=	342.15
W_{key} =	36.00	x	10.75	=	387.00
F_{SN1h} =	103.37	x	0.5	=	51.68
F_{SN2h} =	103.37	x	1.5	=	155.05
F_{SN3h} =	103.37	x	2.5	=	258.42
F_{SN4h} =	112.76	x	3.5	=	394.67
F_{SN5h} =	0.00	x	4.5	=	0.00
F_{SN1v} =	37.62	x	11.79	=	443.52
F_{SN2v} =	37.62	x	12.37	=	465.24
F_{SN3v} =	37.62	x	12.94	=	486.96
F_{SN4v} =	41.04	x	13.52	=	554.92
F_{SN5v} =	0.00	x	14.10	=	0.00
			M_r	=	12103.70

.O.S. against Overturning Failure = 1.72 **OK**

C. Check Factor of Safety against Bearing Failure

Vertical Force (N), F_{tv} - $W_{basesoil}$ =	1124.16	kN					
Eccentricity e_b, B/2 - (M_r - M_o)/N =	2.43	m	>	B/6 =	2.32	m	**NOT OK**
P = N/(B-2e_b) =	124.42	kPa	<	q_{all} =	185.42 (Appendix A, Geoguide 1)	kPa	**OK**

F.O.S. against Bearing Failure = 1.49 **OK**

Table of Contents
Table of Contents
Table of Contents
Table of Contents

C.4.5 Overtopping by Debris Surge

The overtopping scenario as shown in Figure 2.2(b) of this Technical Note is considered. The shear stress which gives rise to the drag force is determined by Equation (2.1) of the Technical Note.

$$\tau = h \rho g \tan \phi_e \quad \text{...} \quad (C.1)$$

where h = maximum debris thickness (= 2.1 m, see also Figure C3)

$\tan\phi_e$ = equivalent coefficient of friction at the interface of the overtopping debris surge and the deposited debris and should be taken as $\tan \phi + v^2 / (h\ \xi)$ where v is design debris velocity (= 6.8 m/s) and h is the maximum debris thickness (= 2.1 m), ϕ and ξ are the apparent friction angle and turbulent coefficient adopted in the debris mobility analysis respectively

The length of active wedge behind the barrier is adopted in calculating the drag force. Partial factor of safety on the shear strength parameters of soil is taken as 1.1.

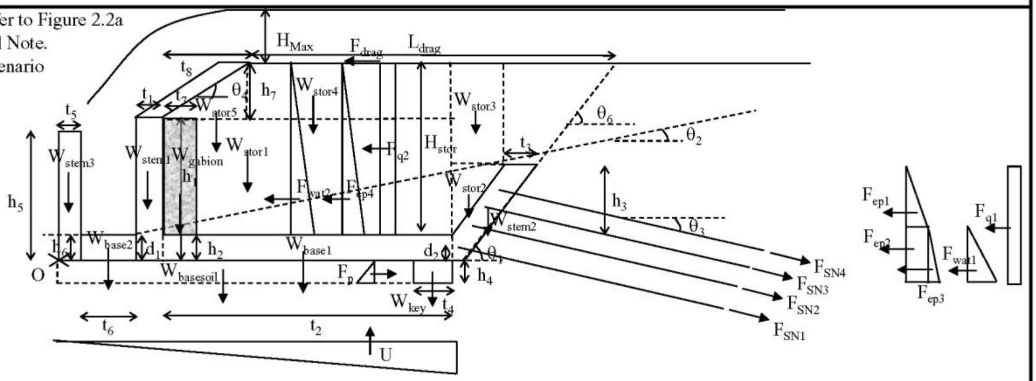
The design calculations are presented in the following pages.

Table of Contents
Table of Contents
Table of Contents
Table of Contents

Geotechnical Engineering Office
Job title: Worked Example for Design of Rigid Debris-resisting Barriers

Stability Checking of Final Barrier (Overflow is permitted)

Scenario 5 - Refer to Figure 2.2a of this Technical Note.
Over-topping scenario



Design Event: N.T.S. (illustration only)

630m^3 debris volume with boulder fall (boulder volume < 1m^3)

Debris Force Determination

Maximum debris thickness, H_{Max} =	2.1	m	Report No. 104, Figure 21 or DMM model
wall height, h_1 =	6.0	m	
static soil depth, H_{stor} =	6.1	m	
debris impact velocity, v_d =	6.8	ms^{-1}	Report No. 104, Figure 21 or DMM model
density of debris, r_d =	1800	kg/m^3	
density of gabion, r_g =	1800	kg/m^3	
angle between velocity vector and surface of the barrier, β =	90	deg.	
length of drag, L_{drag} =	10.5	m	
debris drag force, F_{drag} =	**86**	kN/m	Equation 1 of Section 2 in this TN

Boulder Force Determination

Boulder Impact Force, $F_{boulder}$ =	**0**	kN/m	Equation B1 in Appendix B of this TN with K_c = 0.1

Soil Properties				Barrier Geometry		
concrete unit wt., γ_{conc} =	24	kN/m^3		wall stem 1 thickness, t_1 =	1.2	m
soil unit wt., γ_s =	18	kN/m^3		wall stem 1 height, h_1 =	6	m
soil saturated unit wt., γ_{sat} =	18	kN/m^3		wall base thickness, h_2 =	1	m
water unit wt., γ_{wat} =	9.81	kN/m^3		wall base width, t_2 =	7	m
ϕ' of back soil =	35	deg.		wall stem 2 thickness, t_3 =	1.5	m
c' of back soil =	0	kPa		wall stem 2 height, h_3 =	3.16	m
partial safety factor =	1.1			key width, t_4 =	1.5	m
design ϕ' of back soil =	32.5	deg.		key depth, h_4 =	1	m
design c' of back soil =	0.0	kPa		wall stem 3 thickness, t_5 =	0.8	m
K_a of back soil =	0.28	(Figure 18, Geoguide 1)		wall stem 3 height, h_5 =	4	m
K_0 of back soil =	1			wall base 2 thickness, h_6 =	1	m
K_p of back soil =	5.5	(Figure 20, Geoguide 1)		wall base 2 width, t_6 =	2.5	m
wall back friction angle, δ	21.7	deg.		embedded depth, d_1 =	1	m
ϕ' of base soil =	34	deg.		gabion block width, t_7 =	1	m
c' of base soil =	5	kPa		coping length, t_8 =	3.1	m
design ϕ' of base soil =	31.5	deg.		coping height, h_7 =	1.1	m
design c' of base soil =	4.5	kPa		soil nails above wall toe, d_2 =	0.5	m
K_a of base soil =	0.28	(Figure 18, Geoguide 1)		θ_1 =	60	deg.
K_0 of base soil =	0.48			θ_2 =	17.5	deg.
K_p of base soil =	5.7	(Figure 19, Geoguide 1)		θ_3 =	20	deg.
wall base friction angle, δ_b	31.5	deg.		θ_4 =	20	deg.
wall base cohesion, c_b	4.5	kPa		θ_6 =	**51**	deg.
ϕ' of debris =	11	deg.		soil nail vertical spacing =	1	m
turbulance coefficient, ξ	500	m/s^2		soil nail horizontal spacing =	1.5	m
equivalent ϕ_e' of debris =	13.4	deg.		F_{SN1} =	110	kN/m
design equivalent ϕ_e' of debris=	12.2	deg.		F_{SN2} =	110	kN/m

Table of Contents
Table of Contents
Table of Contents
Table of Contents

Geotechnical Engineering Office
Job title: Worked Example for Design of Rigid Debris-resisting Barriers

Stability Checking of Final Barrier (Overflow is permitted)

$\tan \phi_e' = \tan \phi' + 0.5 \times v^2/(h\xi)$

max $\phi_e' = 20°$

F_{SN3} =	110	kN/m
F_{SN4} =	120	kN/m
F_{SN5} =	0	kN/m
pressure height (at the back slab	5.16	m
water height (at the back slab)=	5.16	m
pressure height (at the stem)=	6.13	m
water height (at the stem)=	6.13	m

A. Check Factor of Safety against Sliding Failure

Force from Debris and Water

Type										Force (kN)	F_{h1} (kN)	F_{v1} (kN)
F_{debris} =									=	86	86	0
$F_{boulder}$ =									=	0	0	0
F_{ep1} =	0.5	x	0.28	x	18.00	x	0.00 ^2		=	0.00	0.00	0
F_{ep2} =	0.28	x	18.00	x	0.00	x	5.16		=	0.00	0.00	0
F_{ep3} =	0.50	x	0.28	x	8.19	x	5.16 ^2		=	30.54	30.54	0
F_{wat1} =	0.50	x	9.81	x	5.16 ^2				=	130.64	130.64	0
F_{q1} =	5.07	x	27	x	0.28	x	5.16		=	197.71	197.71	0
F_p =	0.25	x	5.7	x	8.19	x	1.00 ^2		=	-11.6708	-11.67	0
F_{ep4} =	0.50	x	1.00	x	8.19	x	6.13 ^2		=	153.79	153.79	0
F_{wat2} =	0.50	x	9.81	x	6.13 ^2				=	184.21	184.21	0
F_{q2} =	2.10	x	27	x	1.00	x	6.13		=	347.48	347.48	0
											1118	0.00

Force from Soil Nail

Type		Force (kN)	F_{nh} (kN)	F_{nv} (kN)
F_{SN1} =	=	110	-103.37	37.62
F_{SN2} =	=	110	-103.37	37.62
F_{SN3} =	=	110	-103.37	37.62
F_{SN4} =	=	120	-112.76	41.04
F_{SN5} =	=	0	0.00	0.00
			-423	153.91

Self Weight and Uplift Pressure

Type									Force (kN)	F_{h2} (kN)	F_{v2} (kN)
W_{stem1} =	1.2	x	6.0	x	24			=	172.80	0	172.80
W_{stem2} =	1.5	x	3.16	x	24			=	113.79	0	113.79
W_{stem3} =	0.8	x	4.0	x	24			=	76.80	0	76.80
W_{gabion} =	1	x	5.0	x	18			=	90.00	0	90.00
W_{coping} =	1.2	x	1.1	x	24			=	32.50	0	32.50
W_{base1} =	1	x	7	x	24			=	168.00	0	168.00
W_{base2} =	1	x	2.5	x	24			=	60.00	0	60.00
W_{stor1} =	6	x	6.13	x	18			=	661.86	0	661.86
W_{stor2} =	0.5	x	1.82	x	3.16	x	18	=	51.91	0	51.91
W_{stor3} =	1	x	1.82	x	2.97	x	18	=	97.48	0	97.48
$W_{basesoil}$ =	1	x	10	x	18			=	180.00	0	180.00
W_{key} =	1.5	x	1	x	24			=	36.00	0	36.00
U =	0.5	x	5.16	x	9.81	x	11.5	=	-291.11	0	-291.11
										0	1450.02

Total Sliding Force (F_{th}), $F_{h1} + F_{h2}$ =	1118	kN
Total Vertical Force (F_{tv}), $F_{v1} + F_{nv} + F_{v2}$ =	1603.93	kN
Total Sliding Resistance, F_{tv} x tan δ_b + c_b x wall base - F_i	1458.65	kN

F.O.S against Sliding Failure = 1.30 **OK**

Table of Contents
Table of Contents
Table of Contents
Table of Contents

Geotechnical Engineering Office
Job title: Worked Example for Design of Rigid Debris-resisting Barriers

Stability Checking of Final Barrier (Overflow is permitted)

B. Check Factor of Safety against Overturning Failure

Overturning Moment about pt. O

Type	Force (kN)		Lever Arm (m)		Moment (kNm)
F_{debris} =	86	x	7.13	=	611.47
$F_{boulder}$ =	0	x	9.23	=	0.00
F_{ep1} =	0.00	x	4.16	=	0.00
F_{ep2} =	0.00	x	1.58	=	0.00
F_{ep3} =	30.54	x	0.72	=	22.00
F_{wat1} =	130.64	x	0.72	=	94.10
F_{q1} =	197.71	x	1.58	=	312.47
F_{ep4} =	153.79	x	3.04	=	467.95
F_{wat2} =	184.21	x	3.04	=	560.52
F_{q2} =	347.48	x	4.06	=	1412.19
U =	291.11	x	7.67	=	2231.85
			M_o	=	5712.54

Restoring Moment about pt. O

Type	Force (kN)		Lever Arm (m)		Moment (kNm)
W_{stem1} =	172.80	x	3.9	=	673.92
W_{stem2} =	113.79	x	12.70	=	
W_{stem3} =	76.80	x	0.40	=	30.72
W_{gabion} =	90.00	x	5.00	=	450.00
W_{coping} =	32.50	x	6.05	=	196.60
W_{base1} =	168.00	x	8	=	1344.00
W_{base2} =	60.00	x	2.05	=	123.00
W_{stor1} =	661.86	x	8.5	=	5625.79
W_{stor2} =	51.91	x	12.11	=	628.60
W_{stor3} =	97.48	x	12.41	=	1209.92
W_{key} =	36.00	x	10.75	=	387.00
F_{SN1h} =	103.37	x	0.5	=	51.68
F_{SN2h} =	103.37	x	1.5	=	155.05
F_{SN3h} =	103.37	x	2.5	=	258.42
F_{SN4h} =	112.76	x	3.5	=	394.67
F_{SN5h} =	0.00	x	4.5	=	0.00
F_{SN1v} =	37.62	x	11.79	=	443.52
F_{SN2v} =	37.62	x	12.37	=	465.24
F_{SN3v} =	37.62	x	12.94	=	486.96
F_{SN4v} =	41.04	x	13.52	=	554.92
F_{SN5v} =	0.00	x	14.10	=	0.00
			M_r	=	14925.26

.O.S. against Overturning Failure = 2.61 **OK**

C. Check Factor of Safety against Bearing Failure

Vertical Force (N), F_{tv} - $W_{basesoil}$ =	1423.93	kN					
Eccentricity e_b, B/2 - (M_r - M_o)/N =	0.48	m	<	B/6 =	2.32	m	**OK**
P = N/(B-2e_b) =	110.04	kPa	<	q_{all} =	346.87	kPa	**OK**
					(Appendix A, Geoguide 1)		

F.O.S. against Bearing Failure = 3.15 **OK**

Table of Contents
Table of Contents
Table of Contents
Table of Contents

C.5 References

GEO (1993). *Guide to Retaining Wall Design (Geoguide 1) (2nd Edition).* Geotechnical Engineering Office, Hong Kong, 258 p.

GEO (2008). *Guide to Soil Nail Design and Construction (Geoguide 7)*. Geotechnical Engineering Office, Hong Kong, 97 p.

Lo, D.O.K. (2000). *Review of Natural Terrain Landslide Debris-resisting Barrier Design (GEO Report No. 104)*. Geotechnical Engineering Office, Hong Kong, 91 p.

Table of Contents
Table of Contents
Table of Contents
Table of Contents

GEO PUBLICATIONS AND ORDERING INFORMATION
土力工程處刊物及訂購資料

A selected list of major GEO publications is given in the next page. An up-to-date full list of GEO publications can be found at the CEDD Website http://www.cedd.gov.hk on the Internet under "Publications". Abstracts for the documents can also be found at the same website. Technical Guidance Notes are published on the CEDD Website from time to time to provide updates to GEO publications prior to their next revision.

部份土力工程處的主要刊物目錄刊載於下頁。而詳盡及最新的土力工程處刊物目錄，則登載於土木工程拓展署的互聯網網頁http://www.cedd.gov.hk 的"刊物"版面之內。刊物的摘要及更新刊物內容的工程技術指引，亦可在這個網址找到。

Copies of GEO publications (except geological maps and other publications which are free of charge) can be purchased either by:

<u>Writing to</u>
Publications Sales Section,
Information Services Department,
Room 402, 4th Floor, Murray Building,
Garden Road, Central, Hong Kong.
Fax: (852) 2598 7482

<u>or</u>
- Calling the Publications Sales Section of Information Services Department (ISD) at (852) 2537 1910
- Visiting the online Government Bookstore at http:// www.bookstore.gov.hk
- Downloading the order form from the ISD website at http://www.isd.gov.hk and submitting the order online or by fax to (852) 2523 7195
- Placing order with ISD by e-mail at puborder@isd.gov.hk

讀者可採用以下方法購買土力工程處刊物(地質圖及免費刊物除外)：

<u>書面訂購</u>
香港中環花園道
美利大廈4樓402室
政府新聞處
刊物銷售組
傳真: (852) 2598 7482

<u>或</u>
- 致電政府新聞處刊物銷售小組訂購 (電話：(852) 2537 1910)
- 進入網上「政府書店」選購，網址為 http://www.bookstore.gov.hk
- 透過政府新聞處的網站 (http://www.isd.gov.hk) 於網上遞交訂購表格，或將表格傳真至刊物銷售小組 (傳真：(852) 2523 7195)
- 以電郵方式訂購 (電郵地址：puborder@isd.gov.hk)

1:100 000, 1:20 000 and 1:5 000 geological maps can be purchased from:

Map Publications Centre/HK,
Survey & Mapping Office, Lands Department,
23th Floor, North Point Government Offices,
333 Java Road, North Point, Hong Kong.
Tel: (852) 2231 3187
Fax: (852) 2116 0774

讀者可於下列地點購買1:100 000、1:20 000及1:5 000地質圖：

香港北角渣華道333號
北角政府合署23樓
地政總署測繪處
電話: (852) 2231 3187
傳真: (852) 2116 0774

Requests for copies of Geological Survey Sheet Reports and other publications which are free of charge should be directed to:

<u>For Geological Survey Sheet Reports which are free of charge</u>:
Chief Geotechnical Engineer/Planning,
(Attn: Hong Kong Geological Survey Section)
Geotechnical Engineering Office,
Civil Engineering and Development Department,
Civil Engineering and Development Building,
101 Princess Margaret Road,
Homantin, Kowloon, Hong Kong.
Tel: (852) 2762 5380
Fax: (852) 2714 0247
E-mail: jsewell@cedd.gov.hk

<u>For other publications which are free of charge</u>:
Chief Geotechnical Engineer/Standards and Testing,
Geotechnical Engineering Office,
Civil Engineering and Development Department,
Civil Engineering and Development Building,
101 Princess Margaret Road,
Homantin, Kowloon, Hong Kong.
Tel: (852) 2762 5346
Fax: (852) 2714 0275
E-mail: thomashui@cedd.gov.hk

如欲索取地質調查報告及其他免費刊物，請致函：

<u>免費地質調查報告</u>:
香港九龍何文田公主道101號
土木工程拓展署大樓
土木工程拓展署
土力工程處
規劃部總土力工程師
(請交:香港地質調查組)
電話: (852) 2762 5380
傳真: (852) 2714 0247
電子郵件: jsewell@cedd.gov.hk

<u>其他免費刊物</u>:
香港九龍何文田公主道101號
土木工程拓展署大樓
土木工程拓展署
土力工程處
標準及測試部總土力工程師
電話: (852) 2762 5346
傳真: (852) 2714 0275
電子郵件: thomashui@cedd.gov.hk

Table of Contents
Table of Contents
Table of Contents

MAJOR GEOTECHNICAL ENGINEERING OFFICE PUBLICATIONS
土力工程處之主要刊物

GEOTECHNICAL MANUALS

Geotechnical Manual for Slopes, 2nd Edition (1984), 302 p. (English Version), (Reprinted, 2011).

斜坡岩土工程手冊(1998)，308頁(1984年英文版的中文譯本)。

Highway Slope Manual (2000), 114 p.

GEOGUIDES

Geoguide 1	Guide to Retaining Wall Design, 2nd Edition (1993), 258 p. (Reprinted, 2007).
Geoguide 2	Guide to Site Investigation (1987), 359 p. (Reprinted, 2000).
Geoguide 3	Guide to Rock and Soil Descriptions (1988), 186 p. (Reprinted, 2000).
Geoguide 4	Guide to Cavern Engineering (1992), 148 p. (Reprinted, 1998).
Geoguide 5	Guide to Slope Maintenance, 3rd Edition (2003), 132 p. (English Version).
岩土指南第五冊	斜坡維修指南，第三版(2003)，120頁(中文版)。
Geoguide 6	Guide to Reinforced Fill Structure and Slope Design (2002), 236 p.
Geoguide 7	Guide to Soil Nail Design and Construction (2008), 97 p.

GEOSPECS

Geospec 1	Model Specification for Prestressed Ground Anchors, 2nd Edition (1989), 164 p. (Reprinted, 1997).
Geospec 3	Model Specification for Soil Testing (2001), 340 p.

GEO PUBLICATIONS

GCO Publication No. 1/90	Review of Design Methods for Excavations (1990), 187 p. (Reprinted, 2002).
GEO Publication No. 1/93	Review of Granular and Geotextile Filters (1993), 141 p.
GEO Publication No. 1/2006	Foundation Design and Construction (2006), 376 p.
GEO Publication No. 1/2007	Engineering Geological Practice in Hong Kong (2007), 278 p.
GEO Publication No. 1/2009	Prescriptive Measures for Man-Made Slopes and Retaining Walls (2009), 76 p.
GEO Publication No. 1/2011	Technical Guidelines on Landscape Treatment for Slopes (2011), 217 p.

GEOLOGICAL PUBLICATIONS

The Quaternary Geology of Hong Kong, by J.A. Fyfe, R. Shaw, S.D.G. Campbell, K.W. Lai & P.A. Kirk (2000), 210 p. plus 6 maps.

The Pre-Quaternary Geology of Hong Kong, by R.J. Sewell, S.D.G. Campbell, C.J.N. Fletcher, K.W. Lai & P.A. Kirk (2000), 181 p. plus 4 maps.

TECHNICAL GUIDANCE NOTES

TGN 1	Technical Guidance Documents

Table of Contents
Table of Contents
Table of Contents
Table of Contents