DESIGN BASIS FOR STANDARDISED MODULES OF LANDSLIDE DEBRIS-RESISTING BARRIERS

GEO REPORT No. 174

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THE GOVERNMENT OF THE HONG KONG
SPECIAL ADMINISTRATIVE REGION

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**This report was originally produced in August 2003
as GEO Technical Note No. TN 4/2003**

First published, December 2005

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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. These include guidance documents and results of comprehensive reviews. 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 last page of this report.

R.K.S. Chan
Head, Geotechnical Engineering Office
December 2005

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FOREWORD

This report presents the technical basis for the development of a set of standardised modules of debris-resisting barriers to mitigate natural terrain landslide hazards. Suitably conservative standardised modules of debris-resisting barriers have been put forward to suit the typical range of natural hillside profiles in Hong Kong and design events involving a debris volume of up to 600 m^3.

The issue of this report is intended to invite comments for consideration in the further development and refinement of the framework for standardised debris-resisting barriers. A separate report will be prepared to document the recommended framework for application of standardised debris-resisting barriers.

This report was prepared by the Landslip Investigation Division with the support of their landslide investigation consultant, Maunsell Geotechnical Services Ltd. Dr D.O.K. Lo provided useful comments on the proposed design methodology. Professor Oldrich Hungr of the University of British Columbia also reviewed this report and made valuable suggestions.

K.K.S. Ho
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ABSTRACT

New developments on or close to natural hillside, together with the need to react to known natural hillside landslide hazards posed to existing developments, has created a growing demand for natural terrain hazard assessments as well as design and construction of the necessary landslide mitigation works in Hong Kong. The detailed design of a debris-resisting barrier can be a technically demanding and time-consuming process. The development of suitably conservative standardised modules of landslide debris-resisting barriers and a framework based on which these measures may be prescribed for a given site without the need for detailed investigation of the hillside, debris runout modelling and detailed structural design would therefore be beneficial.

A suitably conservative design approach for standardised modules of debris-resisting barriers has been formulated by applying, and where appropriate extending, the methodology given in GEO Report No. 104 (Lo, 2000). Various types of standardised barriers have been developed to suit a range of natural hillside profiles as well as different design events with a debris volume ranging from 50 m^3 to 600 m^3.

This report describes the development of the technical basis for the design methodology for standardised debris-resisting barriers and outlines the key design considerations and assumptions made.

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1. INTRODUCTION

The pressure of new developments on the natural hillsides of Hong Kong, together with the need to react to known hazards for existing developments, has created a growing demand for natural terrain hazard assessments and subsequent design and construction of landslide mitigation works, where necessary. To meet this demand, the Geotechnical Engineering Office (GEO) has published guidelines for the assessment of natural terrain hazards (Ng et al, 2002) and a review of methodologies employed in the design of debris-resisting barriers (Lo, 2000). Some examples of debris-resisting barriers that have been constructed recently in Hong Kong are shown in Plates 1 to 4.

The design of debris-resisting barriers for debris flow mitigation can be a time-consuming process that may be out of proportion with respect to small-scale developments or existing facilities. In the case of landslide emergency works when barriers need to be designed and constructed within a short period, an efficient design approach for standardised modules of mitigation works that facilitates the vetting process is called for.

The development of a suitably conservative set of standardised modules of debris-resisting barriers and a framework whereby these measures may be prescribed for a given site without the need for detailed investigation, debris runout modelling and detailed structural design would therefore be beneficial.

This report presents the technical basis for the development of standardised modules of debris-resisting barriers for mitigation of natural terrain landslide hazards and outlines the design framework for its application. The basic assumptions and the development of the standardised modules of debris-resisting barriers are derived from knowledge obtained from landslide studies, experience of design and construction of debris-resisting barriers in Hong Kong, together with state-of-the-art debris mobility numerical modelling.

2. LOCAL EXPERIENCE

2.1 Landslide Study Data

Based on the GEO's Natural Terrain Landslide Inventory, on average about 300 natural terrain landslides occur every year (Evans & King, 1998). The majority of these landslides are shallow failures within a few metres of the ground surface with short runout distances but some have developed into mobile, channelised debris flows with a long runout distance of up to about 1 km.

Selected natural terrain landslides have been the subjects of detailed studies by the GEO. Compilations of field data from area studies include those carried out by the GEO (e.g. Franks, 1996, Wong et al, 1997) for landslides that occurred on Lantau Island in 1992 and 1993. Recent area studies also include the Tsing Shan Foothill Natural Terrain Landslide Study, which involved the detailed mapping of 117 landslides that occurred in 2000 (MGSL, 2003). Since 1997, systematic landslide studies by the GEO have been implemented which include investigation of significant natural terrain landslides, such as the Pak Shan Wan, Pat Heung, Queen's Hill, Sha Tin Heights, Luk Keng Wong Uk, Outward Bound School, Sham Tsang San Tseun, Leung King Estate and Cloudy Hill, which were carried out by the GEO's landslide investigation consultants. The understanding of the

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diverse range of hillside instability problems has been substantially improved through the landslide studies.

2.2 Notable Debris Flows in Hong Kong

Some of the notable channelised debris flows in Hong Kong include the 1990 Tsing Shan landslide, 1997 Sha Tau Kok landslide, 1999 Sham Tseng San Tsuen landslide and 2001 Lei Pui Street landslide.

The 1990 Tsing Shan debris flow (Plate 5) occurred on 11 September 1990 (King, 1996) and involved an initial failure of about 350 m^3 of weathered granite and colluvium. The debris flow travelled down a steep drainage-line and mobilised a further 20,000 m^3 of primarily loose colluvium. A maximum velocity of about 16.5 m/s was estimated from field superelevation measurements. The debris flow terminated on a platform at the toe of the hillside, approximately 1,000 m (in plan) from the crown of the scar with a travel angle of 21°. The 1990 Tsing Shan debris flow is the largest recent channelised debris flow that has so far been documented in Hong Kong.

The Sha Tau Kok debris flow (Plate 6) occurred in 1997 (Ayotte & Hungr, 1998) and originated as an open hillslope failure that entered a streamcourse where it was channelised and became a debris flow. The total volume of the landslide was about 1,400 m^3. The travel distance of the debris flow was about 850 m in plan, with a travel angle of 21°.

The Sham Tseng San Tsuen debris flow (Plate 7) occurred on 23 August 1999 (FMSWJV, 2000) and originated as an open hillslope failure which became channelised in a streamcourse and developed into a debris flow. The debris demolished a number of squatter dwellings, causing thirteen injuries and one fatality. The total active volume of the debris flow was about 500 m^3 that travelled down the rocky streamcourse in one pulse with only small amounts of deposition and entrainment that were approximately balanced in volume along the debris trail. Velocities of between 7 m/s and 10 m/s were estimated from the superelevation of mud-lines along the streamcourse. The debris flow travelled about 280 m in plan from the crown of the scar, with a travel angle of about 23°. However, the actual mobility of the debris flow could not be established because the debris was obstructed by the densely-packed squatter structures. Subsequently, a reinforced concrete debris-resisting barrier was designed and built at the mouth of the drainage line (Plate 3).

The Lei Pui Street debris flow (Plate 8) occurred on 1 September 2001 on the natural hillside above Lei Pui Street, Kwai Chung (MGSL, 2002). The debris flow was triggered by a translational landslide involving about 250 m^3 of rock and soil that cascaded over a 25 m high cliff and entrained approximately 450 m^3 of colluvium and saprolite below the cliff. The debris flow demolished two inhabited squatter structures that had been vacated less than two hours before the event. The debris was mainly deposited within the lower trail area and a former quarry site, with some outwash entering the adjacent Shek Lei Estate. Detailed field measurements and surveys were carried out within the landslide source area and along the debris trail. Velocities of between 4 m/s and 8 m/s were estimated from the superelevation of mud-lines and an assessment of structural damage along the lower part of the streamcourse. The debris flow had travelled about 320 m in plan from the crown of the scar to the edge of the quarry platform with a travel angle of about 23°. The confluence of the debris flow with a streamcourse draining a larger catchment probably increased the

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mobility of the debris by the injection of additional surface water from the large catchment following breaking of a temporary debris dam at this location. A reinforced concrete debris-resisting barrier was subsequently constructed (Plate 4).

Other notable channelised debris flows include the Liu Pok landslide (King, 1997), 2000 Tsing Shan landslide (King, 2002) and the 2000 Leung King Estate landslides (HCL, 2001).

2.3 Assessment of Landslide Debris Mobility for the Design of Debris-resisting Barriers

2.3.1 General

Assessments of mobility using field data from area studies and selected detailed studies have been undertaken by several authors (e.g. Lau & Woods, 1997; Wong et al, 1997 and Franks, 1996). GEO Report No. 104 (Lo, 2000) summarises the findings and suggests methods for the assessment of landslide mobility for debris-resisting barrier design. The use of travel angle and travel distance versus landslide volume relationships for open hillslope failures and channelised debris flows based on existing data including the recently completed Tsing Shan Foothill Area Natural Terrain Landslide Hazard Study is discussed in Appendix A of this Technical Note.

Within the standardised barrier framework, a channelised debris flow is defined as a landslide involving the movement of debris along a laterally confined path. As broadly defined by Ng el al (2002) and based on case histories reviewed under this study and from consideration of the resolution of 1:1,000 topographic maps, a laterally confined path consists of a distinct channel, drainage line or depression that collects surface runoff during rainfall where the channelisation ratio (i.e. width to depth ratio of the cross section area in a channel/depression) is less than 10 when estimated from a 1:1,000 topographic map with 2 m interval contours or from site measurements or field surveying. The source of the debris may enter into, or originate from within the channel and the channelisation ratio of the potential path must be less than 10 for at least 30% of its length to be classified as a channelised debris flow within the standardised barrier framework.

An open hillslope failure debris path is one where the debris does not mix with a large proportion of surface water and is defined as a flow path with a channelisation ratio of greater than 10 for more than 70% of the flow path.

2.3.2 GEO Report No. 104

Figure 21 of GEO Report No. 104 (Lo, 2000) suggests methods for the assessment of debris mobility for the design of debris-resisting barriers. Both analytical and empirical approaches are covered.

The analytical approach involves the determination of debris mobility using continuum models which have been calibrated against field observations. Based on previous data and the results of back analyses carried out by Hungr (1998) and Ayotte & Hungr (1998), a ϕ-value of 20° irrespective of the debris volume is suggested by Lo (op cit) for the analysis of channelised debris flows using a friction-only rheological model. A turbulence

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coefficient (ξ) of 500 m/s² and a ϕ-value of 11° are recommended for the analysis of channelised debris flows when the Voellmy rheological model is used. For the assessment of open hillslope failures, a friction-only rheological model is suggested, with ϕ-values of 25° and 20° being recommended for debris volumes of less than 400 m^3 and greater than or equal to 400 m^3 respectively.

The empirical approach suggested by Lo (op cit) involves determining the maximum debris velocity and maximum debris thickness, which vary according to four debris volume ranges and applying the friction-only, leading-edge equation of Hungr & McClung (1987) in order to estimate the runout distance and debris velocity within the runout area. The different values of velocity and thickness for the four volume ranges are given in Figure 19 of GEO Report No. 104, which shows the maximum debris front velocity and maximum debris thickness versus total debris volume. These values are mainly based on field measurements and previous back analyses of Hong Kong landslides carried out by Hungr (1998) and Ayotte & Hungr (1998).

2.3.3 Results of Further Back Analyses

Further back analyses of notable natural terrain landslides have been carried out under this study and the results are presented in Appendices A, B and E. These indicate that for both open hillslope failures and channelised debris flows, an analytical model calibrated against available field data would be preferable for the assessment of debris velocity and debris mobility in order to obtain a realistic appreciation of the variation in debris height, velocity and runout distance for slope profiles.

For the analysis of channelised debris flows, the Voellmy rheological model with a maximum ϕ-value of 11.3° is considered appropriate, since this would reduce the risk that the debris mobility may be underestimated for confined channels inclined at between 11° and 20°. For channel angles within this range, the application of the empirical approach suggested in GEO Report No. 104, using the leading-edge calculation with a ϕ-value of 20° for friction model may underestimate the runout distance, while the use of a lower ϕ-value in the Voellmy rheological model would not lead to the debris ceasing motion until a flatter part of the channel is reached.

As three Hong Kong mobile debris flows have been back analysed with ϕ = 5.7° (viz. 1997 Sha Tau Kok landslide, 1999 Sham Tseng San Tsuen landslide and 1993 Tung Chung landslide (landslide No. 5A/2 as designated by Franks, 1996)), a ϕ-value range of between 5.7° and 11° with the Voellmy rheological model is considered appropriate for the development of the standardised barrier framework.

For the moderate volume range of open hillslope failures considered under the standardised barrier framework (i.e. up to 100 m^3), a friction-only model is considered appropriate for the assessment of velocity, with ϕ-values of 30° and 25° assumed for debris volumes of 50 m^3 and 100 m^3 respectively (Appendix A).

Pursuant to the recommendation given in GEO Report No. 104, the estimation of debris velocity and flow height under the standardised barrier framework will be based on the results of back analyses of existing landslides in Hong Kong.

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2.4 Back Analyses of Debris Runout Using Computer Models

2.4.1 General

The chaotic and varied nature of landslides, which commonly involve complex debris movement mechanisms that are not fully understood, present significant challenges in the prediction of landslide debris runout behaviour. A suitable computer model should be calibrated against well-documented, local landslide cases through back analysis before it is used for design purposes. This would allow the applicability and limitations of the model to be ascertained and facilitate a better understanding of the relative significance of the input parameters.

In Hong Kong, the back analysis results for approximately 30 local debris flows and open hillslope failures have been published by various authors including Hungr (1998), Ayotte & Hungr (1998), Chen & Lee (1998), Hungr et al (1999), Lo (2000) and MGSL (2000 & 2002). The relevant data used for calibration of the standardised barrier framework are shown in Tables E2 to E5 in Appendix E.

In general, the back analysis results show broad agreement with the field observations of signs of debris velocity (from the superelevation of the mud-lines), debris thickness and travel distance, indicating that the basic physical equations of motion and relatively simple rheological models can approximate the behaviour of real landslide events in a reasonable manner. In the back analysis, the key parameters are varied in order to achieve a close match with the field data. There can be reasonable confidence in the results of the back analysis where:

(a) the field data are comprehensive and of good quality,

(b) at least two different computer programs are used which provide a good match with the field data, and

(c) the different back analyses results in similar basic rheological parameters.

The standardised barrier design methodology is primarily based on the analyses of past events using Hungr's DAN model and MGSL's Debriflo model to develop a framework for assessing the runout characteristics of different design events (see Appendix E). As a reference, the results of the back analyses of the 1990 Tsing Shan, 1997 Sha Tau Kok and 1999 Sham Tseng San Tsuen debris flows using these two models are summarised in Appendix B. These illustrate that where the data are comprehensive and of good quality, the two models would give similar results and provide a sufficiently good match with the field data. The back analysis of the 2001 Lei Pui Street debris flow using the MGSL Debriflo model is also included in Appendix B as a further example to illustrate the close matching of the field data with the model predictions.

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2.4.2 DAN Model

The DAN model developed by Hungr (1995) is a simplified continuum model based on the principles of conservation of mass, momentum and energy to describe the dynamic motion of landslide debris. A finite difference solution of the governing dynamic equations in a Lagrangian framework is used. The solution is obtained in time steps for a block assembly of elements, with the landslide modelled as a continuum. The effect of lateral confinement and mass changes (i.e. entrainment and deposition) can also be allowed for. The model is capable of determining the debris velocities at different times for a given landslide event and it can also be used to predict debris thickness along the runout trail, provided that the debris width along the flow path is defined. The DAN model has been successfully used on many occasions for the back analysis of landslides (including debris flows) and design of landslide defence measures in other countries.

The DAN model has been used to back analyse a number of landslides in Hong Kong (Hungr, 1998; Ayotte & Hungr, 1998) including some 20 natural terrain landslides with volumes ranging from about 50 m^3 to 26,000 m^3. Various rheological models were used in the analyses, with material coefficients being varied by a process of trial and error to match as closely as possible the actual distribution of the landslide debris. The friction model was reported by Hungr (op cit) as being able to adequately model open hillslope failures in most cases. The Voellmy model was found to be more appropriate for channelised debris flows, and a combination of an apparent friction angle, $\phi = 11.3°$ and a turbulence coefficient, $\xi = 500$ m/s² generally gives a reasonable estimate of the debris mobility. Exceptions were the 1997 Sha Tau Kok debris flow and the 1993 Tung Chung landslide No. 5A/2, where $\phi = 5.7°$ was found to give a better approximation of the actual runout distance. Subsequent back analysis of the 1999 Sham Tseng San Tsuen debris flow with the DAN model also indicates that $\phi = 5.7°$ is more appropriate for well-channelised, mobile wet debris flows in Hong Kong.

2.4.3 Debriflo Model

Following the 1999 Sham Tseng San Tsuen debris flow, MGSL were commissioned by the GEO to design a debris-resisting barrier across the mouth of the drainage line in order to protect the affected squatter village from a 1,400 m^3 design event.

In order to be able to back analyse the 1999 Sham Tseng San Tsuen debris flow and obtain realistic rheological parameters for barrier design, MGSL developed the 'Debriflo' spreadsheet program, using the comprehensive field data from the 1999 event to test the model during its development. A DAN analysis was also carried out by Professor Hungr which confirmed the results of the Debriflo model. After further experience and development, MGSL submitted the program to the Buildings Department (BD) for Government approval under PNAP 79. The computer program was checked in detail by the Special Projects Division of the GEO and approval was given by the BD in December 2002 (BD Reference No. G0126).

The Debriflo program models the leading-edge of the debris front as a single pulse and is based on Newton's second law of motion, with the time-stepping solution logic being similar to the DAN model developed by Hungr (1995) and the "leading edge" equations of Takahashi & Yoshida (1979). The equations satisfy the principles of conservation of mass,

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momentum, energy and continuity of flow for a fluid medium and were developed to allow for a variety of factors, including variations in flow height, slope angle and discharge along the debris path and incorporation of the Voellmy rheological model (Hungr, 1995 and Lo, 2000). A description of the program, based on the approved submission by MGSL to the Special Projects Division is included in Appendix G.

2.4.4 Results of Back Analyses

The results of the back analyses contained in Appendix B indicate that where the field data are comprehensive and of good quality, the two models give similar results, despite some differences in the modelling techniques of the two computer programs. The overall good-fit with the field data and the similar rheological parameters derived from the back analyses give confidence that both computer programs are capable of modelling the behaviour of debris runout in a sufficiently realistic manner.

In the standardised barrier approach, the chaotic nature of landslide debris runout is compensated for by assuming that the maximum debris velocity and debris height versus debris volume correspond to the upper-bound back-analysed results of the different types of previous landslide events, and by 'launching' the debris at its maximum design velocity and height (see Section 4) in order to allow for uncertainties in the initial conditions of the landslide at the source.

3. DEVELOPMENT OF DESIGN BASIS FOR STANDARDISED BARRIERS

3.1 Basic Considerations

The basic considerations in the development of a suitable design framework for standardised modules of debris-resisting barriers are as follows:

(a) The framework to be developed should be based on experience derived from past local events with good quality data.

(b) Appropriate design parameters should be adopted for debris flow and open hillslope failure characteristics which have been verified by back analyses of the local field data.

(c) The framework for barrier design should be suitably simplified, easy to apply and sufficiently flexible to cover the typical range of natural drainage channels and open hillslope characteristics.

(d) The approach to debris flow modelling and structural design should be based on relevant local and international experience and practice.

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(e) In view of the relatively small number of local, well-documented cases from which flow dimensions and velocity can be accurately assessed, an analytical approach is needed to assist in the prediction of debris mobility.

(f) The debris modelling approach should provide good and sufficiently conservative correlations with the available local data and be practicable to implement to deal with the range of design scenarios typically considered.

(g) The barrier structure should be relatively easy to construct and should preferably not involve complex and heavy foundations that require intensive ground investigation and detailed design on a site-specific basis.

(h) For moderate-scale open hillslope failures, a lightweight, flexible structure is preferred in order to minimise foundation loads on the hillside and facilitate construction.

(i) For channelised debris flow barriers, the robustness of the design is enhanced by incorporating a design check for the structure to withstand the impact from an event with double the volume of the design event without uncontrolled collapse and overtopping by the debris. This is to allow for uncertainties in entrainment volume and the possibility of multiple events being channelised into the same drainage line.

3.2 Analysis Methodology

A flow chart that illustrates the methodology developed for the standardised barrier framework is shown in Figure 1. The design debris runout profiles and debris runout modelling methodology together with the barrier options/designs are based on the back analyses of selected debris flows and open hillslope failures in Hong Kong (see Section 2.3 and Appendix B) and the guidelines for debris-resisting barrier design given in GEO Report No. 104. A summary of the main assumptions of the standardised barrier framework is given in Table 1.

The design debris runout profiles and debris runout modelling (see Section 4) provided the basis for spreadsheets which have been developed to cover the full range of design debris runout profiles, design event volumes and two sets of rheological parameters for channelised debris flows. These spreadsheets contain numerical and graphical output for the calculated debris velocity, debris height and run-up and impact forces along the entire design channel profile. A total of 220 analyses were carried out for each of the two sets of rheological parameters (i.e. 440 calculations in total). Testing of the profiles derived from the standardised barrier framework against actual debris flow events indicates that the standardised barrier framework is suitably conservative.

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Detailed structural design of the debris flow-resisting barriers (described in Section 5) was carried out and the maximum load capacity of each barrier was determined. The founding materials of the barriers are taken to meet the minimum shear strength parameters as well as other conditions against bearing capacity and overall instabilities. Details are given in Table 1. A set of barrier design charts was produced for each of the barrier types and different barrier heights in which the minimum barrier length and minimum acceptable distance from the commencement of the runout area is defined for differing debris flow design event volume and channel configurations. The minimum barrier distance shown on the design charts is the largest of the values obtained from consideration of each set of rheological parameters. In this way, the design charts have considered both the upper and lower bound parameters that determine the debris height and debris/boulder impact forces. This means that users need not consider variations in debris mobility within the range of parameters that have been found from the back analyses to be representative of all the previous debris flows in Hong Kong.

A less complicated approach was adopted for open hillslope failures where the landslide debris is modelled as a 'friction-only' lumped mass assuming no turbulence. A set of tables has been developed for a range of slope angles assuming ϕ-values of 25° and 30° for debris volumes of 100 m^3 and 50 m^3 respectively (see Appendix A). The distance along the runout trail whereby the energy of the landslide debris is less than the design capacity of a prescribed tensioned steel mesh fence is given for different slope angle, design event and ϕ-value. Details of the proposed approach are given in Appendix D. The approach enables the designer to make an assessment of whether a barrier is needed or not, taking into account the proximity of the affected facility, the site setting and the design event. In the case whereby a barrier is not needed as far as runout of landslide debris is concerned, the designer is advised to consider whether the potential hazard of boulder 'roll-out' from the landslide debris is a concern and if so, whether a boulder fence to cater for this is warranted or not. For example, Evans & Hungr (1993) suggest that the above hazard should be assessed for a runout path that is steeper than 23° based on their experience with sizeable landslides in Canada. The design of the boulder fence for such a scenario, if considered necessary by the designer, is outside the scope of the present framework.

In terms of mitigation measures for open hillslope failures, tensioned steel mesh fences of up to 2,000 kJ energy capacity are proposed. The designer should ensure that the associated foundations and anchorages could structurally withstand a debris impact corresponding to the energy rating of the fence.

4. DESIGN RUNOUT PROFILES AND MODELLING

4.1 Design Runout Profiles

4.1.1 Channelised Debris Flows

In the standardised barrier design, the debris runout profiles for channelised debris flows consist of three tangent lines. These are used to approximate the actual profile of a potential debris flow path as described in Appendix C which also provides guidelines on how to apply the design profile tangents to a natural drainage line. In the modelling, the upper 34° tangent is used to 'launch' the design debris runout event into the middle and lower tangents. The longitudinal gradient of the upper tangent and cross-sectional profile of the

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design channel is based on a review of previous debris flow events and consideration of typical drainage line profiles in Hong Kong as described in Appendix E.

The procedure for fitting the design profile to a given natural channel as described in Appendix C ensures that the framework can only be applied to debris runout profiles within the upper tangent section that have an overall slope angle of equal to or flatter than 34°. This prevents the framework from being used in cases where the debris runout profile is extremely steep for a long distance. However, the framework may still be used for source areas and irregularities that are steeper than 34° provided that the overall 'angle of reach' or 'energy-line' (Lo, 2000) is equal to or less than 34° between the potential landslide source or the crest of a steep slope segment and the commencement of the middle or lower tangent. This ensures that the velocity upon entry to the next lower tangent will not be higher than that assumed in the present framework.

The maximum height of the upper tangent is also limited to 150 m (Figures C2 and C3, Appendix C) in order to ensure that the standardised barrier framework is restricted to a range of landslide elevations which are similar to those of previous landslides in Hong Kong forming the current database for landslide volumes of less than 600 m³.

The middle tangent, which can be regarded as an approach tangent to the runout area, may vary between 14° and 34° in angle. The modelling of debris travel which forms the basis for the design charts has been carried out for the cases where the minimum length of the middle tangent is 0 m, 25 m and 50 m in order to be able to apply the standardised barrier framework to a wide range of actual channel profiles (see Appendix C).

The lower tangent can be regarded as the runout zone for the debris flow and varies between 2.5° and 12.5°. The minimum distance from the commencement of this zone in which it is acceptable to construct a barrier is defined in the standardised barrier design tables, an example of which is shown in Table C1 of Appendix C. The minimum length of the barrier is also indicated in the standardised barrier tables. In order to ensure that the height of the debris will not exceed the design height derived from the standardised barrier calculations, the base width of the site-specific channel at the prospective site of the barrier must be at least as wide as the minimum barrier length shown in the tables. The designer should also ensure that the channel (within the lower tangent section) behind the debris-resisting barrier should have sufficient retention capacity to contain the design volume of the debris. Otherwise, the barrier needs to be moved forward along the channel to provide the required retention capacity.

Additional qualifying criteria on the use of the standardised barrier framework for channelised debris flows are also proposed to ensure that the field conditions of application will not result in significantly higher discharges of debris than that assumed for the calibrated channels and that the conditions are also within the range of conditions that have previously been encountered in Hong Kong for channelised debris flows with volumes up to 600 m³. These are:

(a) a natural drainage channel with a channelisation ratio of less than 10 (when estimated from 2 m interval topographic contours or site observations) must exist for at least 50 m in horizontal distance above the commencement of the lower tangent, and

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(b) at least one 10 m long segment of the channel within the 50 m zone above the lower tangent must have a channelisation ratio of less than or equal to 5 when estimated from topographic contours, detailed survey plans or site observations.

The above qualifying criteria should ensure that the standardised barrier framework will not be used where fast-moving debris from a nearby open hillslope failure could directly enter the runout area.

4.1.2 Open Hillslope Failures

In the standardised barrier approach, the debris runout profiles for open hillslope failures consist of two tangent lines. These are used to approximate the actual profile of the runout path of the debris from a potential open hillslope failure as described in Appendix D, which also provides guidelines on how to apply the design profile tangents to a given natural hillside. In the modelling, the upper 34° tangent is used to 'launch' the design event into the lower tangent. The longitudinal gradient of the upper tangent is based on a review of previous open hillslope landslide events and consideration of typical open hillslope profiles in Hong Kong as described in Appendix E.

The procedure for fitting the design debris runout profile to a given natural hillside as described in Appendix D ensures that the framework can be applied to natural hillside profiles where the upper tangent section has an overall slope angle that is equal to or less than 34° for the same reasons as given for channelised debris flows in Section 4.1.1. The maximum height of the upper tangent is limited to 80 m (Figure D1, Appendix D) in order to ensure that the standardised barrier framework is restricted to a reasonable range of landslide elevations which are compatible with the relatively moderate debris volumes considered for open hillslope failures.

The lower tangent can be regarded as the runout zone for the debris flow and varies between 6° and 26° in angle. The minimum distance from the commencement of this zone within which it would be suitable to construct a tensioned steel mesh fence is defined in Table D1 of Appendix D.

4.2 Calibration of the Design Channel/Slope and Debris Modelling

4.2.1 Channelised Debris Flows

The calibration of the upper tangent of the design channel with previous data for channelised debris flows in Hong Kong is described in detail in Appendix E. Upper-bound debris velocity and debris height relationships have been established for various design events ranging from 100 m^3 to 8,000 m^3 which represent the range of debris flow events in Hong Kong, previously back analysed. The upper-bound volume is based on the active volume of the 1990 Tsing Shan debris flow at chainage 350 where the maximum field superelevation velocity measurements were made.

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The Debriflo spreadsheet program was used to carry out the calibrations as well as the subsequent detailed modelling for each design event and debris rheology of all the 'three-tangent' design profiles. The modelling approach and parameters used in the back analyses of actual events are thus consistent with the debris mobility modelling design calculations.

The Voellmy rheological model has been used throughout for both the back analyses and the design calculations for channelised debris flows. This model is described in GEO Report No. 104 and is recommended for analytical assessment of channelised debris flows. A Voellmy turbulence factor of 500 m/s^2 was assumed in all cases. As the back analyses of the existing Hong Kong data set indicated that a ϕ-value of 11.3° was appropriate for most debris flows (in matching the debris thicknesses, runout distances as well as debris velocities), while a ϕ-value of 5.7° was representative of the most mobile debris flows analysed in Hong Kong to date (i.e. 1997 Sha Tau Kok and 1999 Sham Tseng San Tsuen debris flows), both sets of ϕ-values were adopted in the design calculations. Each debris runout profile and debris volume case was analysed with each ϕ-value in order to find the most critical conditions in terms of flow height and potential impact forces on the barrier.

In all cases, no entrainment or deposition has been modelled because the active volume of the site-specific landslide at the commencement of the lower tangent should take account of all entrained material. A 5° angle of spread for the channel base width is assumed within the runout segment, which is limited to the top width of the surface of the debris at the point of entry into the runout area. Thus for a 10 m top width, the base width of 1.75 m at the start of the runout area is increased to 10 m (giving a rectangular cross-section) at a distance of 47 m (i.e. $[0.5 \times (10 - 1.75)] \div \tan 5° = 47$ m) from the start of the runout area. A constant rectangular cross-section is assumed beyond this point. The minimum barrier length specified in the design tables is at least the same as the top width of the design channel. It is also at least the same as the lengths assumed in the structural design for each barrier type. Thus the minimum barrier length is the greatest of the top width of the debris flow and the minimum length of the barrier assumed in the structural design. If the barrier needs to be longer to cover a wider channel or to provide sufficient retention capacity of the debris, the debris height would probably be lower than assumed in the design due to additional spreading and therefore the minimum barrier length specified will be conservative.

The above assumptions require the site-specific design volume estimated by the designer who uses the standardised barrier framework to include all debris that may possibly be entrained along the length of the drainage line. It is also assumed that no deposition of debris will occur within the runout zone before reaching the barrier. The assumption of a small amount of spreading of the channel base to the same width as the debris surface width at the commencement of the lower runout tangent requires the designer to ensure that the channel base at the barrier location is at least as wide as the minimum length of the barrier as indicated in the design charts.

4.2.2 Open Hillslope Failures

The calibration of the 34° upper tangent of the design profile with previous data for open hillslope failures in Hong Kong is described in detail in Appendix E. The upper-bound debris velocity versus landslide volume relationship determined from the channelised debris

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flow data (see Figure E2 of Appendix E) also forms a reasonable upper-bound design line for the available data from open hillslope events previously back analysed in Hong Kong (see Figure E3 of Appendix E). The range of volumes of open hillslope failures back analysed by Hungr (1998) and Ayotte & Hungr (1998) varies from about 100 m^3 to 40,000 m^3. This means that the maximum velocity correlation for events less than 100 m^3 would need to be extrapolated from the existing back analysed data. This approach is considered reasonable since the established relationship provides a very good upper-bound fit with the available data as shown in Figure E3, Appendix E.

Unlike the design approach adopted for channelised debris flows, no calibrations for flow height and discharge are necessary in this case because the debris is modelled effectively as a lumped mass which is limited to moderate-scale design events of 50 m^3 and 100 m^3 in volume within the scope of the proposed standardised barrier framework (see also Section 5.2).

5. BARRIER TYPES AND DESIGN CONSIDERATIONS

5.1 Types of Standardised Barriers

A range of barrier types and sizes has been selected for the standardised barrier framework. This is necessary in order to deal with the wide range of conditions that affect the design of barriers, such as the size of the debris flow events, impact loading, run-up heights and debris mobility.

The various types of standardised modules of debris-resisting barriers considered under the framework are:

(a) Type 1 - These comprise reinforced concrete barriers (Figure 2) designed to resist significant impact loads from large-scale events and accommodate the corresponding run-up heights. The Type 1 barrier may be constructed close to the mouth of a drainage line for design events up to 600 m^3.

(b) Type 2 - These comprise gabion units in conjunction with a L-shaped reinforced concrete wall frame (Figure 3), which may be constructed close to the mouth of a drainage line for design events up to 300 m^3.

(c) Type 3 - There are two variants of Type 3 barriers which comprise reinforced gabion units. Type 3A barriers comprise reinforced gabions (Figure 4) whilst Type 3B barriers incorporate a reinforced rockfill core within the gabion units (Figure 5). Vertical steel bars included in the Type 3 barriers are designed to act as dowels to prevent internal sliding failure of the gabion units and maintain structural integrity. Type 3 barriers may be constructed close to the mouth of a drainage line for design events up to 150 m^3.

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(d) Type 4 - These comprise tensioned steel mesh fences (Figure 6) which may be used to mitigate open hillslope failures up to 100 m^3.

5.2 Maximum Design Events for Standardised Barriers and Design Considerations

The maximum design event permitted under the standardised barrier framework (i.e. up to 600 m^3 for channelised debris flows and up to 100 m^3 for open hillslope failures) covers the range of volumes of most of the natural terrain landslides in Hong Kong. Each barrier type and size has been designed for a specific design event, stream/slope profile and impact location within the runout area. The location of a given barrier type needs to be checked to ensure that its design capacity is not exceeded and that there is sufficient retention capacity. This is facilitated by the standardised barrier design tables (refer to example in Table C1 of Appendix C).

For the large-scale events within the scope of the framework (i.e. 600 m^3 design events), the debris velocities and therefore the impacts generated on the debris-resisting structures are likely to be significant and hence, reinforced concrete barriers (i.e. Type 1 barrier) have been specifically designed to resist these events, particularly if the length of the runout zone is limited.

The medium-scale events, i.e. 300 m^3 and 150 m^3, impose smaller impact loads which can be accommodated by less massive structures. For these volumes, two types of reinforced gabion barriers have been developed. The first type (i.e. Type 2 barrier), consisting of a L-shaped reinforced concrete wall frame upon which the gabion core is built, was designed specifically to resist the 300 m^3 events. The second (i.e. Type 3A and Type 3B barrier), smaller type, consisting of gabions with internal steel reinforcement bars, was designed to deal with the 150 m^3 debris flow events.

Each structure was designed to satisfy the constraints imposed by its specific design event, i.e. the total design volume, stream profile, position in the runout zone, run-up height, impact forces and the required minimum barrier length.

In order to ensure that adequate robustness is built into the design process in the light of the potential significant uncertainties in the selection of design events, a robustness check was incorporated into the design. For the relatively 'rigid' structures (i.e. the reinforced concrete walls and reinforced gabion barriers), this consists of checking that the barriers would be able to withstand the effects of an 'extreme' event that corresponds to double the design event volume (e.g. a volume of 1,200 m^3 is taken for the robustness check of a barrier with a design event of 600 m^3). Under the robustness check, the structure must behave in a fail-safe manner.

Tensioned steel mesh fences with a minimum height of 1.5 m have been designed to resist the relatively moderate-scale open hillslope failures (50 m^3 to 100 m^3). The fences consist of wire rope nets supported by steel posts, which are anchored back into the ground with wire rope stays. Such barriers have been reported in the literature as being capable of retaining debris up to a total of 750 m^3 in volume in several impacts (e.g. Rickenmann, 2001). However, the typical volume of landslide debris involved in most of the events was in the range of 100 m^3 to 200 m^3 as far as direct frontal impact loading on the tensioned steel mesh

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fence is concerned. Large-scale debris flume tests of propriety tensioned steel mesh fence (DeNatale et al, 1996) indicate that this type of barrier can retain up to about 10 m³ of debris travelling at velocities up to 9 m/s. However, actual experience of application of such tensioned steel mesh fences to resist the impact of sizeable landslides is very limited and the performance of prototype tensioned steel mesh fences has not been fully verified in the field. Also, the design approach proposed in the literature is largely empirical (e.g. Wartmann & Salzmann, 2002), involving major projection of data obtained from relatively small-scale tests on the key design assumptions (e.g. the duration of impact by debris of a given discharge rate) that can be open to question. In view of the above and following discussion with Professor Hungr of the University of British Columbia, Canada (Hungr, 2002), it is considered unjustified at this stage to have standardised tensioned steel mesh fences to cater for the impact of sizeable landslides. Hence, the maximum design event for this type of barrier has been set at 100 m³.

The detailing of tensioned steel mesh fences has been improved to enhance their robustness. For example, the lateral anchor ropes can be protected from direct impact by boulders carried by the landslide debris at the locations where the ropes are anchored into the ground by the provision of mass concrete deflector blocks (see Figure 6).

5.3 Derivation of Design Impact Loading

The debris and boulder impact loads for rigid barriers were derived in accordance with the recommendation given in GEO Report No. 104. The debris impact loads were determined using the momentum equation factored up using an 'enhancement factor' of 3, as follows:

$$p = 3 \times \rho_d \times v_d^2 \times \sin\beta$$

where ρ_d = density of debris
v_d = velocity of debris
$\sin \beta$ = angle between the velocity vector and surface of the barrier

Boulder impact loads were determined using the Hertz equation divided by a factor of 10 to account for effects of local crushing, as follows:

$$F_{boulder} = \frac{1.14}{10} \times v_b^{1.2} \times \lambda^{0.4} \times m_b^{0.6}$$

$$\lambda = \frac{4 \times \sqrt{r_b}}{3 \times \left(\frac{1-\mu_b^2}{E_b} + \frac{1-\mu_B^2}{E_B} \right)}$$

where m_b = mass of boulder
v_b = velocity of boulder
r_b = radius of boulder
μ_B = Poisson's ratio of barrier
E_B = Young's modulus of barrier

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μ_b = Poisson's ratio of boulder
E_b = Young's modulus of boulder

The impacting boulder was defined as having a diameter equivalent to the debris flow depth at the beginning of the runout area. This depth varies as a function of the stream profile, design volume and debris parameters which determine flow resistance and therefore debris height, and was duly considered in the analyses carried out for the standardised barrier framework.

Debris run-up heights were determined based on the equation given in Section 4.4.4 of GEO Report No. 104. The total debris run-up height derived from the debris modelling for the design event was used to determine the heights required for the barriers:

$$\Delta h = \frac{v^2}{2 \times g}$$

where v = velocity of debris at impact
g = gravitational acceleration

The positions of application of the impact loads were determined such that the worst case effects on the barriers would be covered. For the initial pulse, the debris was assumed to impact on the barrier over a height equivalent to the physical debris depth at that location whilst the boulder impact was assumed to take place near the surface of the debris. For the second pulse where the debris runs up and over the material deposited behind the barrier after the first pulse, the impacting debris was applied over the same height of the debris depth, although in this case it was assumed to act from the top of the barrier down (Figure 7). The boulder was again assumed to act at the top of the impacting debris.

The zone between the bottom of the impacting debris and the base of the wall is assumed to be subjected to static earth, water and surcharge loads from the first pulse of debris. The active earth pressure coefficient (K_a) was assumed to be unity, since the debris will essentially be akin to a thick slurry.

The impact loading on the tensioned steel mesh fences is derived in a different way since these tensioned steel mesh fences rely on large deformations of their key elements to dissipate the energy of impact. Consequently, the equations listed above do not apply to this type of structure. Instead, the resistance capacity of a tensioned steel mesh fence is determined as a function of its energy absorption capacity, which must be greater than the kinetic energy of the impacting material. In the case of open hillslope failures, the impacting mass is assumed to act on the barrier as a lumped mass with no internal deformation. Consequently, the structures are only checked for a single impact.

5.4 Stability Considerations

The stability of the debris-resisting barrier, which includes sliding resistance, overturning resistance and the induced bearing pressures, has been checked for the various loading conditions. For the reinforced concrete barrier, the critical loading condition with respect to the sliding resistance is when the barrier is subjected to the impact from the first

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pulse of debris impact. This is due to the fact that there will only be a minimal amount of debris to enhance (through the action of its self weight), the sliding resistance of the barrier. The second pulse, when the barrier is already partially filled with debris and is impacted at the top, is usually critical for overturning and bearing capacity. Owing to the uncertainty on whether the full base friction will be mobilised at impact, a reduction in the beneficial effects of the impacting debris self-weight has been accounted for. Only 50% of the impacting debris self weight is considered as a beneficial effect for base friction when checking the sliding resistance of the reinforced concrete barriers.

The reinforced gabion barriers, on the other hand, are only checked for the second pulse impact load case since these do not rely on the debris self weight to resist sliding and such a conditions would be more critical.

Stability checks have been carried out for the design event as well as for an extreme scenario with a volume corresponding to twice that of the design event. The latter check is for enhancing the robustness of the barrier scheme given the potential uncertainties in the assessment of an appropriate design event). Under the normal design event scenario, the factors of safety against sliding, overturning and bearing should all be above unity. Since the impact loads are of very short duration (see Section 5.3) and bearing in mind the conservatism already built into the various assumptions as described above, a computed factor of safety of unity is considered to be adequate for the present purposes. Under the extreme event scenarios considered for the robustness check, overturning and bearing failure modes should still satisfy the same criterion. However, in this case the barrier is allowed to slide forward when impacted by a volume that is twice that of the design event. The maximum allowable translational movement under this extreme condition has been set at 1.5 m.

The stability checks do not take account of any potential uplift. It is considered that this would have been overly conservative when assessing the sliding resistance under dynamic impact and it would have made the barriers excessively large and costly to construct. It is assumed that the groundwater level is maintained at a minimum of 1 m below the founding level of the barrier. In order to prevent high groundwater levels from adversely affecting the stability of the barriers, installation of subsoil drainage measures may be prescribed in instances where high water levels are anticipated or where the founding material is not relatively free-draining (i.e. with a coefficient of permeability of less than 10^{-5} m/s). These measures may consist of placing, say, a 300 mm thick layer of rockfill and 100 mm diameter subsoil drain pipes at 3 m spacing under the base of the barrier.

The designer prescribing the standardised debris-resisting barrier should assess whether the ground at the founding level would have the minimum unfactored parameters of $c' = 0$ kPa, $\phi' = 35°$ and $\gamma = 19$ kN/m³. To prevent overall instability and bearing capacity failure, especially if the barrier is to be constructed on sloping ground, the designer also needs to assess the condition of the barrier site against these two failure modes. In these assessments, an ultimate bearing pressure of 300 kPa at the founding level over the whole area of the base is assumed. This ultimate bearing pressure is to account for the self-weight of the barrier structure as well as the debris, including the dynamic impact load of the debris. Where appropriate, replacement of weak materials or, for example, soil reinforcement may be needed to achieve the required ultimate bearing capacity.

Tensioned steel mesh fences rely on large displacement of the structures to dissipate the impact energy and therefore sufficient clearance must be ensured between the fences and

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the affected facilities. Given the working principle of tensioned steel mesh fences, sliding, overturning and bearing checks do not apply. However, the designer needs to check the overall stability of any sloping ground, considering the static loading of the debris built up behind the barrier. The tensioned steel mesh fences are rated according to the impact energy they are able to resist. All the elements in the system have been designed so that they can adequately dissipate the corresponding level of impact energy. The designer prescribing the tensioned steel mesh fence should check if the ground anchorage of the steel mesh fence has sufficient capacity commensurate with the energy rating of the fence.

5.5 Drainage Considerations

Debris-resisting barriers to mitigate debris flows are inherently built in drainage lines and consequently adequate provisions have to be made to allow water flow to take place effectively and safely, under normal conditions and after a debris flow event has occurred.

Such provisions are highly site specific and as such standard details which cover all scenarios cannot be pre-determined. A schematic surface drainage layout is shown in Figure 8 and general considerations as to the best practice to be adopted in designing the drainage provisions are given below. The final choice of the most suitable arrangement to be adopted will be the responsibility of the designer and will be heavily dependent on the site setting under consideration and any site formation works which need to be carried out to accommodate the barriers.

The main considerations to be taken into account when designing the arrangement of the surface drainage system are:

(a) Any surface water flow from the drainage line should be collected and directed away from the barrier in such a way as to prevent any ponding upslope of the barrier, reduce the possibility of high groundwater levels around and under the barrier and reduce the potential for erosion of the material at the base of the barrier structure.

(b) Drainage pipes, culverts or channels built under the barriers should be avoided as far as possible to avoid potential blockage. The surface drainage provisions should preferably be directed around the structures and designed to fulfil their role even following a debris flow event. The clearance and repair of these drainage provisions should be simple and should not obstruct the access to the debris retention area.

In practice, it would be advisable to consult the Drainage Services Department prior to construction of any such drainage provisions.

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5.6 Other Considerations

Where the width of a drainage line at the prospective location of a barrier is greater than the minimum barrier length specified, it is important that full retention of the debris is provided by extending the barrier to the edges of the drainage line so that the barrier will not be by-passed by the debris. For barriers with a length that is greater than 1.5 times the minimum length specified, a low section of 80% of the standardised barrier height not longer than the total length of the barrier minus the minimum barrier length could be provided to ensure that any possible over-topping of debris will take place at a location that does not jeopardise the safety of any facilities downslope.

Aesthetics and environmental considerations are other important aspects which need to be considered carefully by the designer.

The environmental impact arising from the construction of debris-resisting barriers should be carefully considered since these could be built on natural terrain and across drainage lines and may be subject to the scrutiny of the Environmental Protection Department. Therefore, the designer should refer to the relevant regulations and respective government departments to verify whether, depending on the site under consideration, an environmental impact assessment is required.

From an aesthetics point of view, some of the standardised modules of barrier structures that form part of the framework may be considered visually obtrusive, depending on the site locality. The designer should give due consideration to the necessary mitigation measures to minimise visual impact. Trees and planters could be provided to hide the structures from direct view or to mitigate visual impact. Suitable surface finishes or facing could be considered to make the structure more visually appealing.

6. DESIGN CONSIDERATIONS FOR THE PRESCRIPTION OF STANDARDISED BARRIERS

The following are the advantages of using the standardised barrier framework over conventional design methods:

(a) Practical and technical benefits in allowing geotechnical professional practitioners to implement natural terrain landslide risk mitigation measures for debris flows and open hillslope failures, based on back analyses of past debris flow and open hillslope failure events in Hong Kong and suitably conservative design assumptions to cater for the degree of uncertainty in the design process.

(b) The framework provides for standardised modules of mitigation works as typical design provisions or contingency provisions which can be quickly applied and facilitate site layout design.

(c) Savings in time and human resources by eliminating the need for detailed ground investigation on the hillside, debris

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runout modelling, structural design and rigorous external checking procedures. These savings can be substantial and the design and checking processes made much more efficient, especially when there is a need is to provide the mitigation works within a short period of time or as emergency measures following landslides.

The design assumptions incorporated in the standardised barrier frameworks are based on back-analysed data from past landslides in Hong Kong and with due consideration of the significant uncertainties involved in runout and impact characteristics. Suitable simplification and appropriately conservative assumptions have been made regarding the debris runout profile, channel shape and flow behaviour, etc. Given the flexibility in the applicability to a range of site condition and the efficient design process, the costs of the standardised barriers may be higher than structures designed on the basis of a detailed site-specific assessment with detailed investigation and analysis. However, the additional cost and safety margin will be offset by the cost and time savings in obviating the need for detailed investigation and analysis and the corresponding resources input.

The framework is applicable to a maximum design event volume of 600 m^3 and a range of channel configurations. The framework is expected to be able to cover most of the situations likely to be encountered in practice. The design event must be carefully determined by a suitably experienced and qualified geotechnical professional in accordance with the guidelines given in Special Project Report No. SPR 1/2002 (Ng et al, 2002). As the volume of the design event reaching the site of the barrier should account for the potential entrained material in addition to the failure volume at some areas, adequate examination of the characteristics of the drainage line would be necessary for the assessment of the appropriate design event.

Schematic details of the drainage provisions for stream-flow around the barrier are provided within the standardised barrier framework. Given that the requirements will be highly site-specific, it is necessary for designers to ensure that adequate drainage provisions are made for the site setting under consideration.

Provided that designers acknowledge and work within the above issues, the standardised barrier framework could be adopted as a relatively rapid and conservative approach for the determination of a barrier to be used as natural terrain landslide mitigation works for a site-specific situation.

7. SCOPE OF APPLICATION

7.1 General

The standardised barrier framework has been developed based on a combination of the back analyses of, and detailed observations on, previous natural terrain landslides that have occurred in Hong Kong. The aim is to be able to apply the framework to as many site settings as possible in Hong Kong.

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7.2 Scope of Application

A suitable standardised debris-resisting barrier framework has been developed to enable barriers to be prescribed to mitigate natural terrain landslide hazard as urgent protective works following landslides, as prescriptive mitigation works or as permanent mitigation measures, without the need for detailed investigation and elaborate design analyses. The standardised barrier modules may also be used as preliminary design to facilitate assessment of site layout and cost estimate. As such, there can be savings in respect of time and human resources.

The standardised barrier framework provides an efficient approach for prescribing suitable mitigation measures for small developments (e.g. small houses in NT) subject to small to moderate scale design events where the conventional approach involving detailed design of landslide mitigation works would be technically demanding and time-consuming.

Guidance on the application of the technical framework described in this report and the required input by qualified geotechnical professionals will be documented in a separate report.

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Table 1 - Summary of Assumptions of the Standardised Barrier Framework (Sheet 1 of 2)

Consideration	Assumptions
Design Events	For channelised debris flows, design events volumes of 150 m³, 300 m³ and 600 m³ are considered. For open hillslope failures, design event volumes of 50 m³ and 100 m³ are considered. The design event volume must be assessed by a suitably qualified geotechnical professional in accordance with SPR 1/2002 and must include all potentially entrained material.
Past Experience	Previous back analyses of Hong Kong debris flows indicate that the Voellmy rheological model can be used to realistically approximate field conditions for channelised debris flows (see Section 2 and Appendix E). These parameters have been adopted for the standardised barrier framework. ξ = 500 m/s² appears to give the closest-fit for most back-analysed cases ϕ = 11.3° (typical), ϕ = 5.7° (for very wet and mobile flows) For open hillslope failure, the friction-only model can be used with ϕ = 30° and ϕ = 25° for design event volumes of 50 m³ and 100 m³ respectively. Extreme runout distances for these respective debris volumes are 75 m and 120 m measuring from the lower edge of the source area of the failures (see Appendix A).
Design Debris Runout Profiles	The design channel configurations are defined in Appendices C and D for channelised debris flows (three-tangent system) and open hillslope failures (two-tangent system) respectively. For channelised debris flows, the following limitations are applied: • the height of the upper tangent is limited to 150 m • the average depth of the source is less than 2 m • a reasonable degree of channelisation exists above the runout area (see Section 4.1.1 (b) & (c)) • the minimum angle of spreading of the base width of the debris runout channel in the lower tangent is 5°, and the top width of the debris runout channel at the commencement of the lower tangent is the minimum base and top width of the runout channel at the barrier location, i.e. width of the channel at the barrier location must not be less than the dimension shown in the design charts (the flow width and design barrier length).
Debris Initial Runout Conditions	Constant velocity, height and discharge are assumed within the upper ‘launching’ tangent for the design events. These have been determined by calibration of the upper tangent channel with the results of back analyses and field observations (Appendix E). The calibrated conditions reflect the upper-bound values determined from back analyses of previous events in Hong Kong.

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Table 1 - Summary of Assumptions of the Standardised Barrier Framework (Sheet 2 of 2)

Consideration	Assumptions
Barrier Height and Impact Loading	The barrier height assumed in the framework is equal to the flow height plus the run-up height calculated in accordance with the recommendation given in GEO Report No. 104 for barriers with vertical backs. Refer to Section 5.3 for derivation of impact forces. See Appendix F for typical calculations and load cases for each type of barrier.
Founding Stratum	For barriers resisting channelised debris flows, the ground conditions at founding level of the barrier site have shear strength parameters and unit weight equal to or better than the following: $c' = 0$ kPa, $\phi' = 35°$, $\gamma = 19$ kN/m³ Groundwater level should be maintained at minimum 1 m below founding level. Subsoil drain should be provided where high groundwater level is expected or where the founding materials not free draining. The designer should check against bearing capacity failure and overall instability, especially if the barrier is to be constructed on sloping ground. An ultimate bearing capacity of 300 kPa at the founding level over the whole area of the base should be considered in the overall stability and bearing capacity assessments to account for the self-weight of the barrier structure and the debris as well as the impact load of the debris. For open hillslope failures, the associated foundations and anchorages shall be determined by the designer to withstand a debris impact corresponding to the energy rating of the tensioned steel mesh fence.
Robustness	The calibrated velocity and discharge for each design volume reflect the upper-bound values determined from back analyses of previous natural terrain landslide events in Hong Kong. In the case of channelised debris flows: • Each barrier is designed to withstand the impact from a debris flow with double the volume of the design event without catastrophic failure (e.g. a 600 m³ design event barrier shall be able to withstand the impact from a 1,200 m³ event without collapse). Under the impact by this extreme event, up to 1.5 m sliding movement of the barrier is tolerated. • The assumptions made when considering a second debris impact at the very top of the barrier reflect the most extreme conditions that the barrier can be subjected to for the design events considered (see Appendix F). • In accordance with GEO Report No. 104, a multiplying factor 3 has been applied to the momentum equation based on consideration of an equivalent fluid for the assessment of debris impact pressure on the barrier (see Section 5.3). • When assessing the resistance to sliding under impact from moving debris, it is assumed that only half of the self-weight of the impacting debris will contribute to enhance the shearing capacity at the base of the barrier structure. For open hillslope failures, additional robustness is provided by protecting the lateral anchor ropes of the tensioned steel mesh fence from direct impact by boulders at the point where the ropes were anchored to the ground by the construction of mass concrete deflector blocks as shown in Figure 6.

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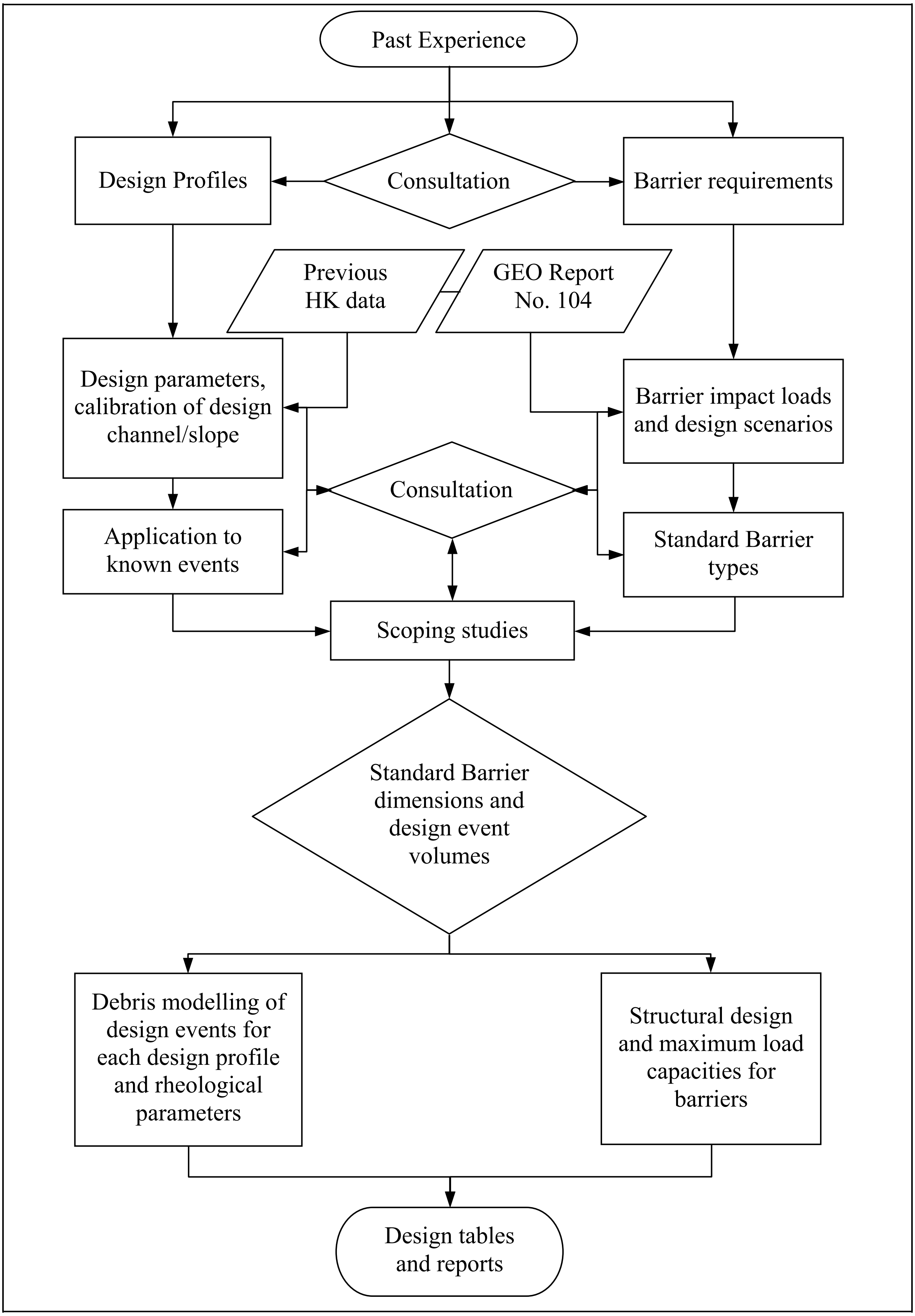

Figure 1 - Development of Standardised Barrier Framework

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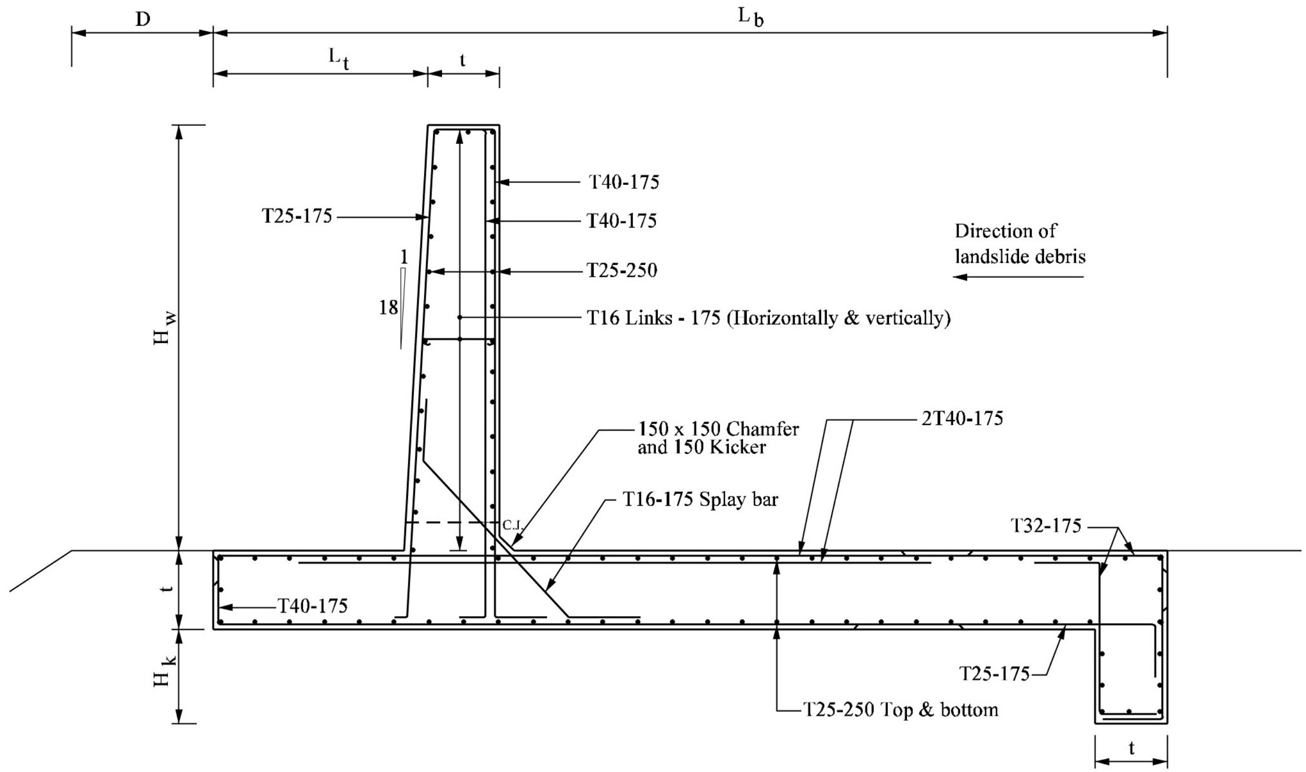

Type 1A Standard Barrier

D	Dimensions (m)				
	H_w	L_b	L_t	t	H_k
> 4.5 m	3.5	12.5	0.5	1.5	0
	4.0	14.0	0.5	1.5	0
	4.5	14.5	0.5	1.5	0
⩽ 4.5 m	3.5	13.0	0.5	1.5	0
	4.0	15.0	0.5	1.5	0
	4.5	15.5	0.5	1.5	0

Type 1B Standard Barrier

D	Dimensions (m)				
	H_w	L_b	L_t	t	H_k
> 6.5 m	3.5	10.5	1.5	1.5	1
	4.0	11.5	1.5	1.5	1
	4.5	12.5	1.5	1.5	1
⩽ 6.5 m	3.5	11.0	1.5	1.5	1
	4.0	12.0	1.5	1.5	1
	4.5	13.0	1.5	1.5	1

Notes :
(1) All dimensions are in mm.
(2) All structural concrete shall be grade 40 D to CS1.
(3) Steel reinforcement shall be grade 460 high yield steel to BS 4449.
(4) Cover to reinforcement shall be 50 mm.
(5) All lap and anchorage length shall be equal to 46 times bar diameter.
(6) Concrete finish shall be specified by the engineer.
(7) 75 mm grade 20 blinding layer shall be provided.
(8) In all cases, dimension "D" should not be less than 1.5 m.

Figure 2 - Reinforced Concrete Barrier (Type 1 Barrier)

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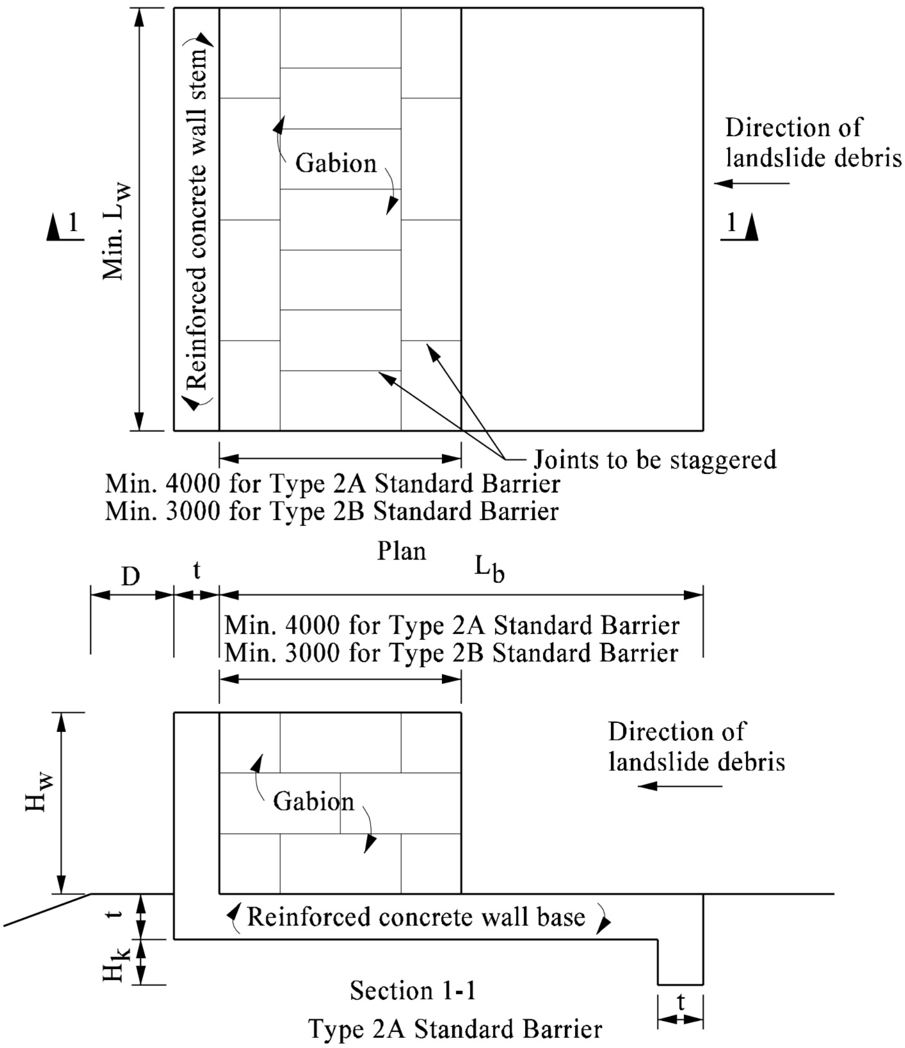

Type 2A Standard Barrier

	Dimensions (m)				
	H_w	L_w	L_b	t	H_k
D > 2 m	3.0	10.0	7.75	0.75	0
	3.5	10.0	8.25	0.75	0
	4.0	11.0	9.25	0.75	0
D ⩽ 2 m	3.0	10.0	8.50	0.75	0
	3.5	10.0	9.00	0.75	0
	4.0	11.0	10.25	0.75	0

Type 2B Standard Barrier

	Dimensions (m)				
	H_w	L_w	L_b	t	H_k
D > 4.5 m	3.0	10.0	7.75	0.75	1.0
	3.5	10.0	8.25	0.75	1.0
	4.0	11.0	9.25	0.75	1.0
D ⩽ 4.5 m	3.0	10.0	8.50	0.75	1.0
	3.5	10.0	9.00	0.75	1.0
	4.0	11.0	10.25	0.75	1.0

Notes :
(1) All dimensions are in mm.
(2) In all cases, dimension "D" should not be less than 1.5 m.

Figure 3 - Reinforced Gabion Barrier (Type 2 Barrier)

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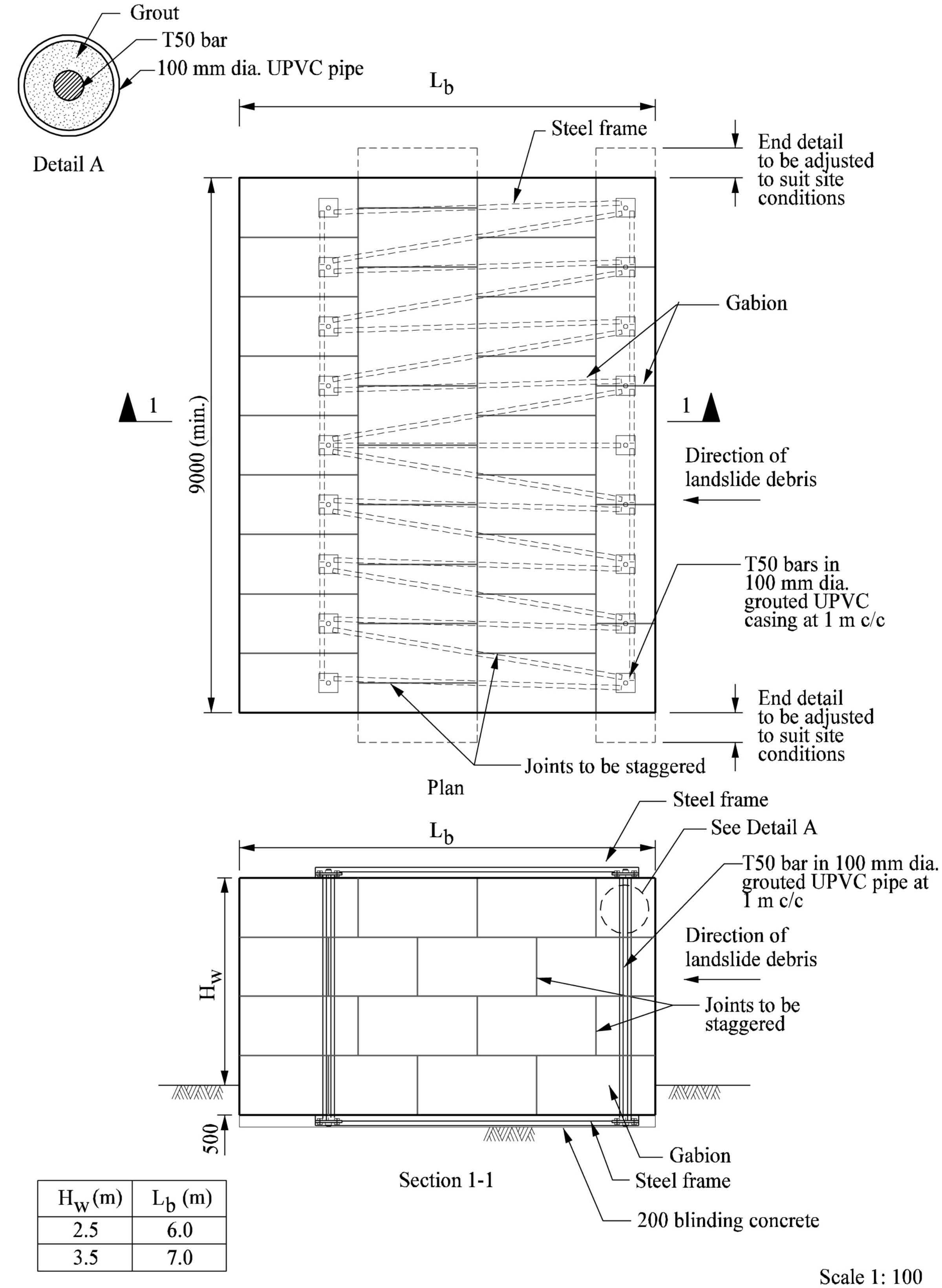

H_w (m)	L_b (m)
2.5	6.0
3.5	7.0

Note : All dimensions are in mm.

Figure 4 - Reinforced Gabion Barrier (Type 3A Barrier)

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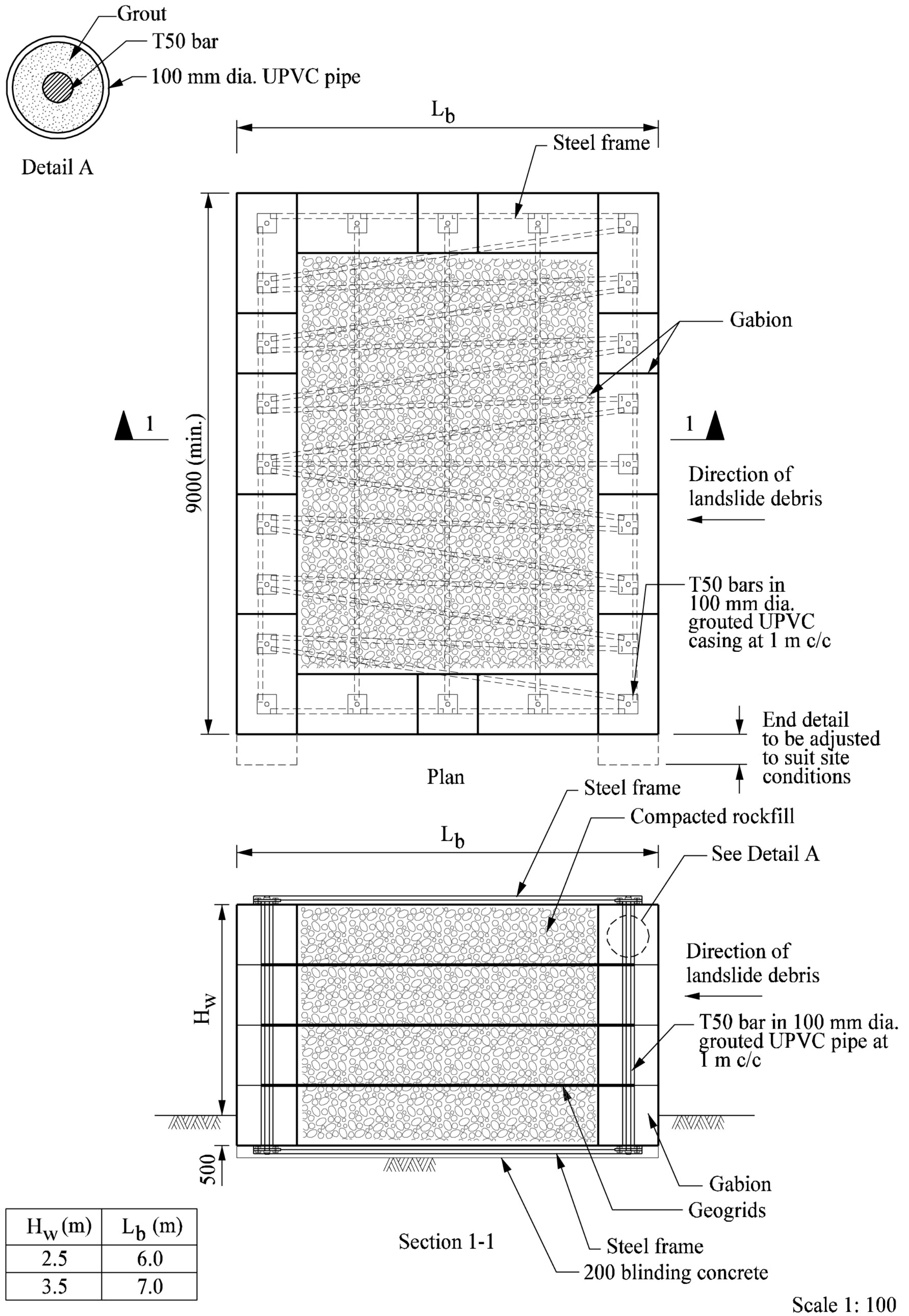

H_w (m)	L_b (m)
2.5	6.0
3.5	7.0

Note : All dimensions are in mm.

Figure 5 - Reinforced Gabion/Rockfill Barrier (Type 3B Barrier)

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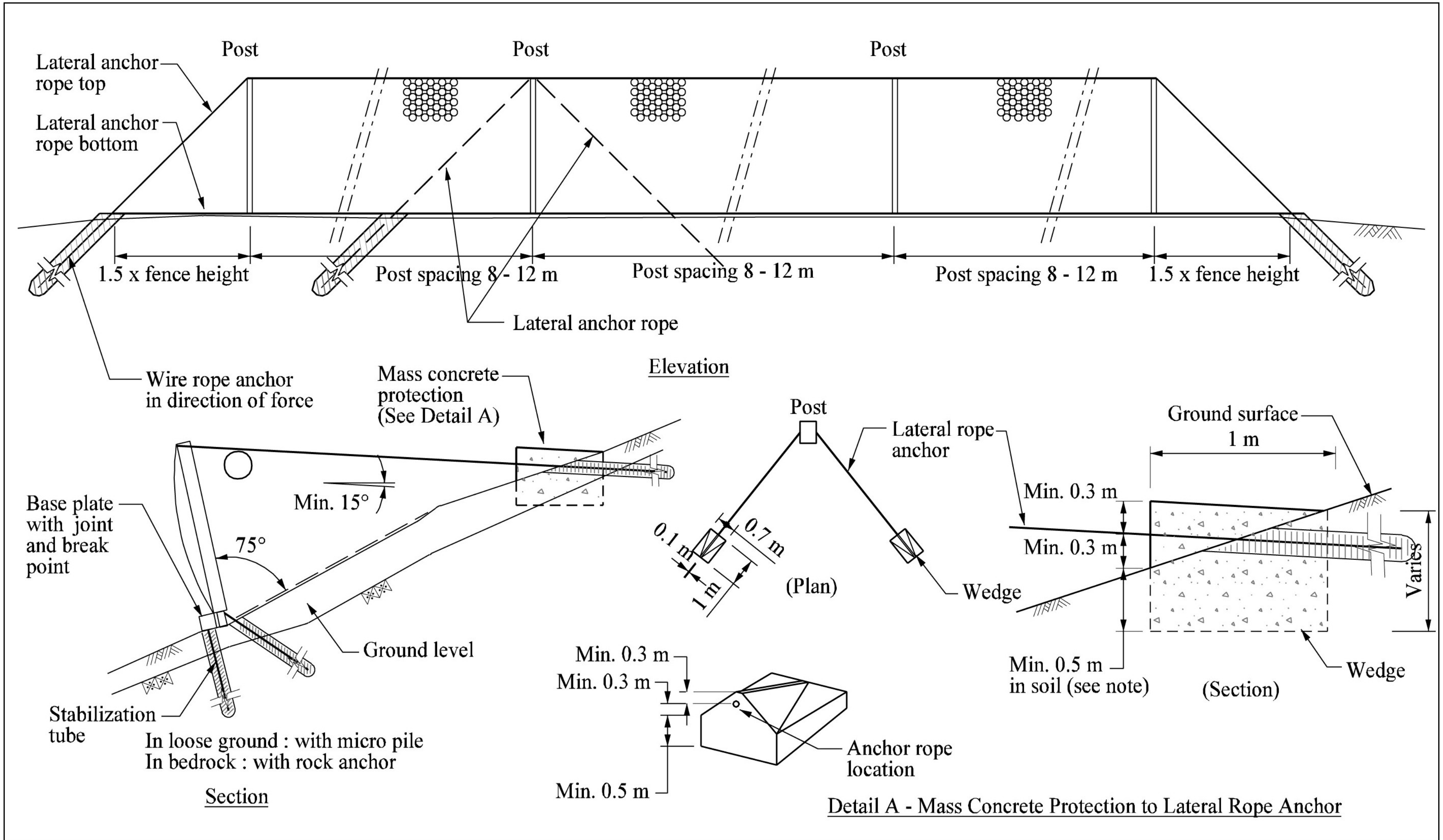

Note : Depth of mass concrete block may be reduced where rockhead is shallower than 1.0 m depth provided that 'L' shaped dowels are installed to anchor the block.

Figure 6 - Tensioned Steel Mesh Fences (Type 4 Barrier)

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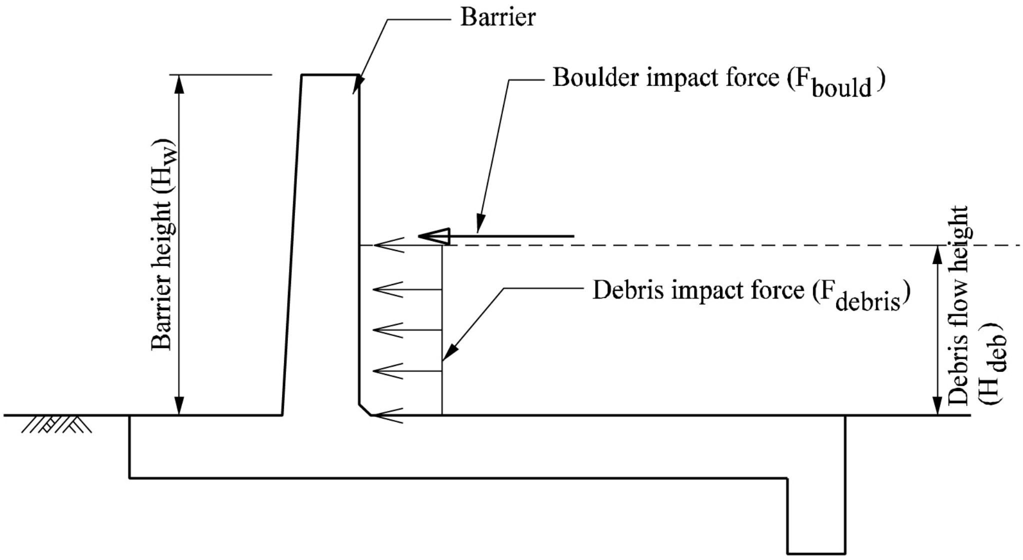
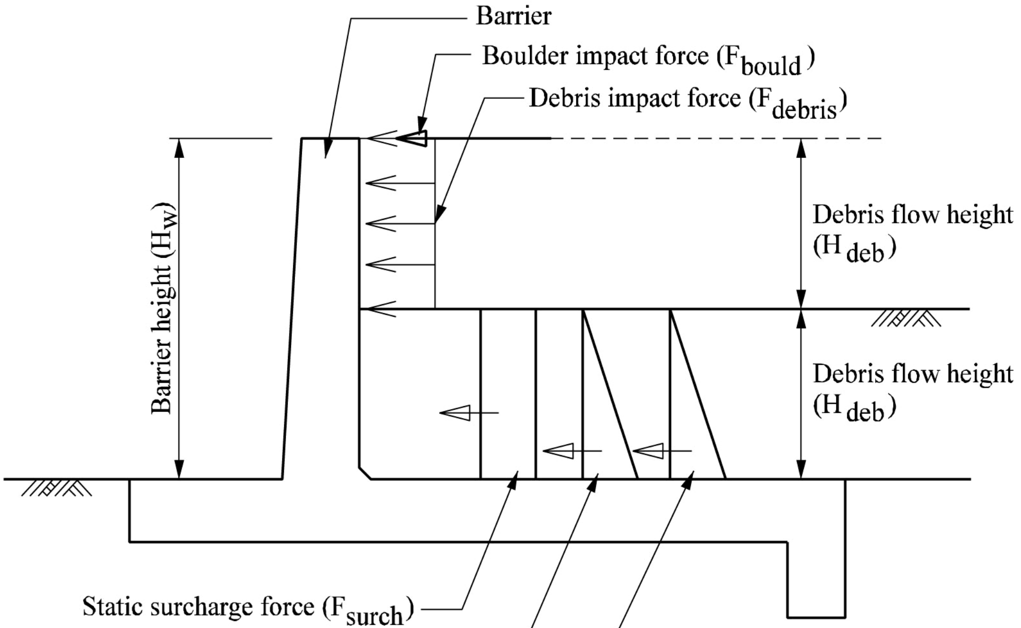


(a) Assumption of Impact from the First Pulse



(b) Assumption of Impact from the Second Pulse

Note : The above assumptions are applicable to Type 1 to Type 3 barriers.

Figure 7 - Assumption of Impact Loads on Debris-resisting Barrier

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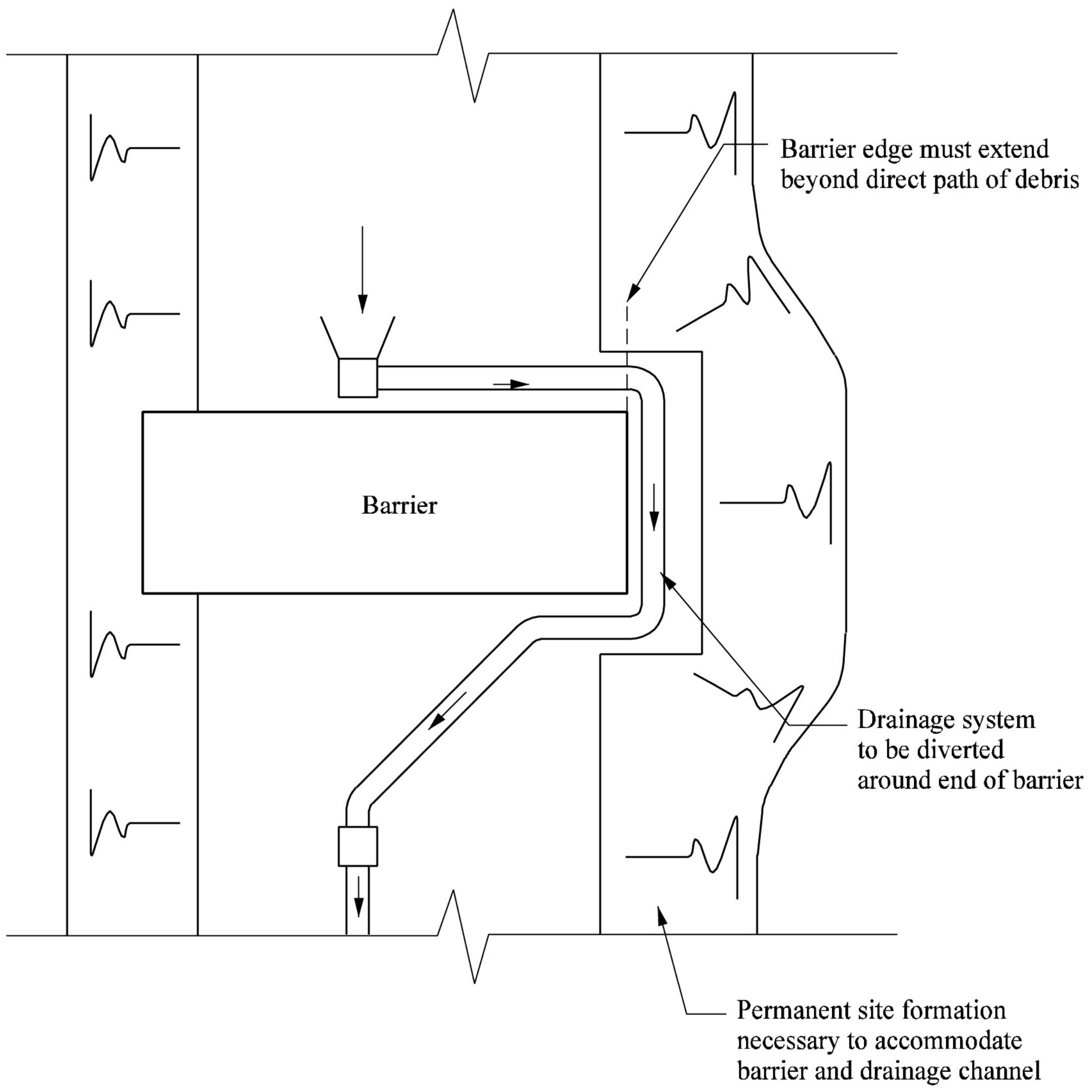

Drainage Arrangement Around Barrier Base

Figure 8 - Schematic Drainage Layout

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Plate 1 - Debris-resisting Barriers above Leung King Estate

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Plate 2 - Debris-resisting Barrier at Cyberport Development

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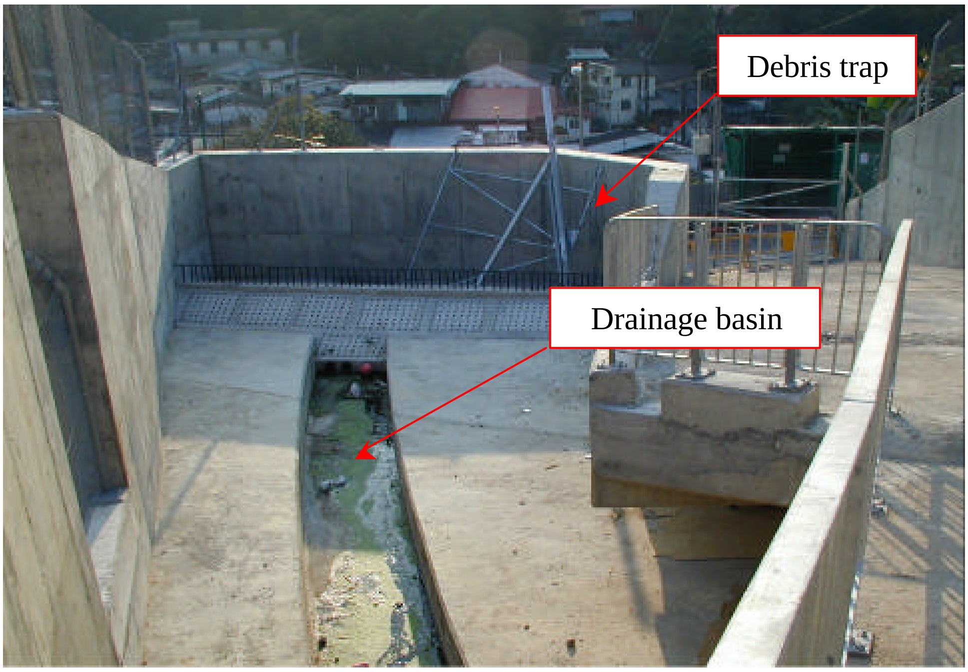

Plate 3 - Debris-resisting Barrier at Sham Tseng San Tsuen

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Plate 4 - Debris-resisting Barrier above Lei Pui Street

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Plate 5 - Oblique Aerial View of the 1990 Tsing Shan Debris Flow
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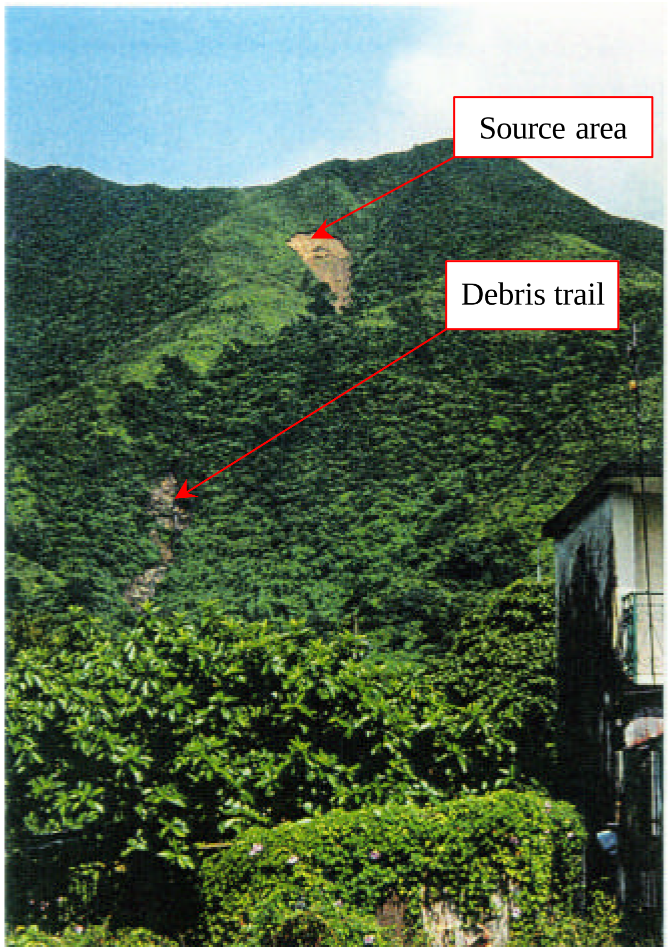

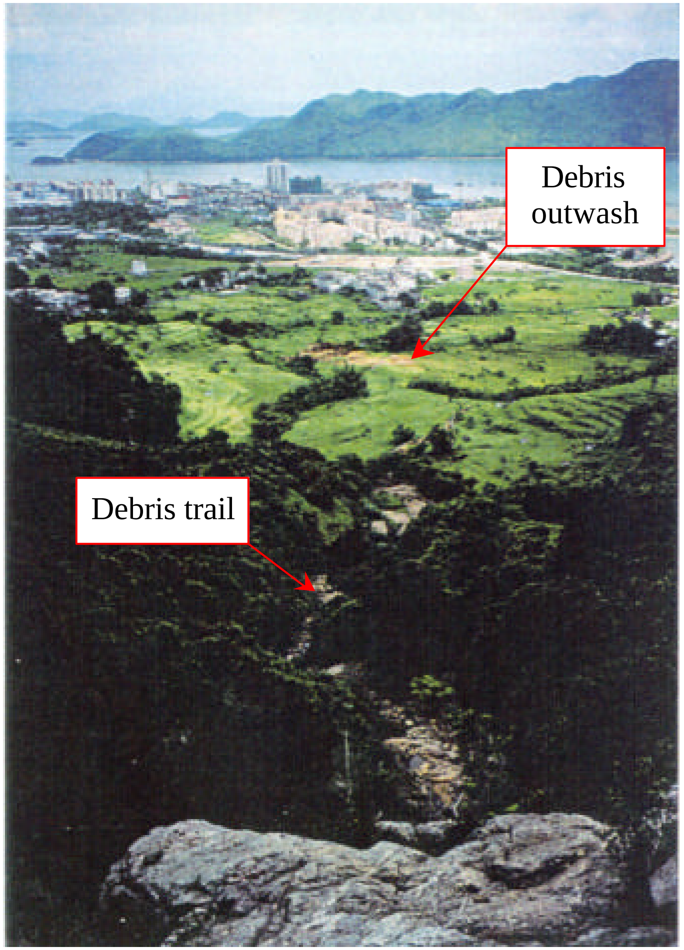

Plate 6 - Views of the 1997 Sha Tau Kok Debris Flow

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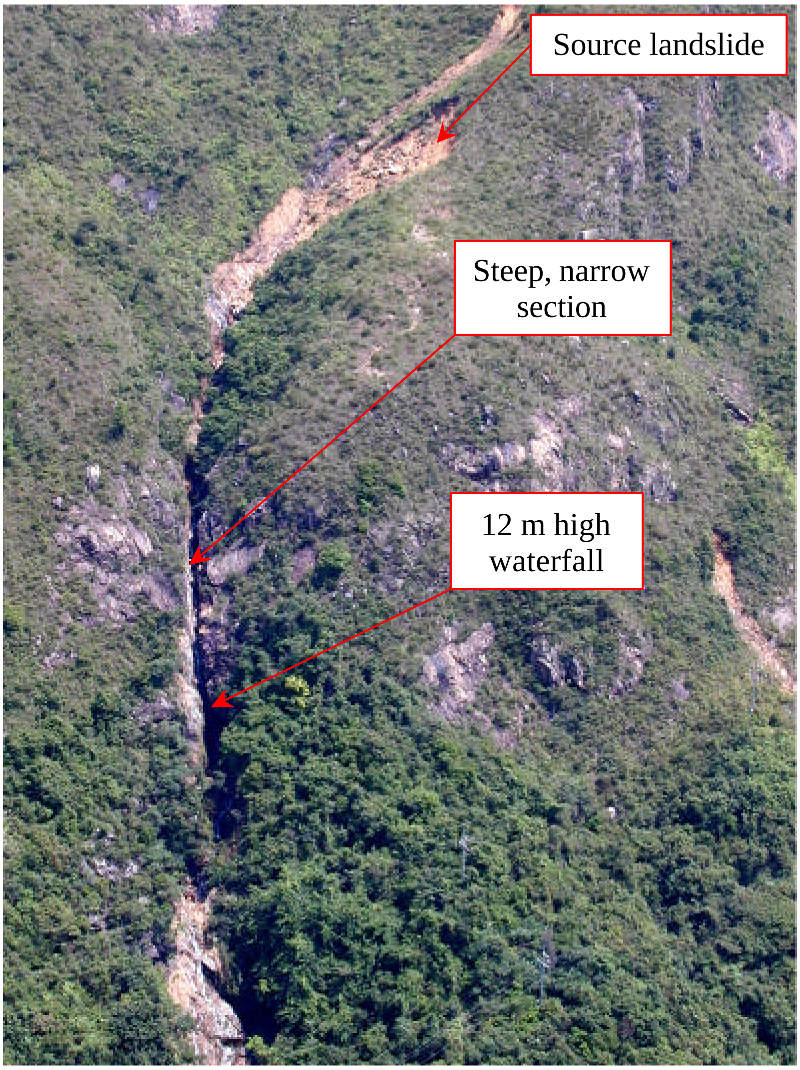

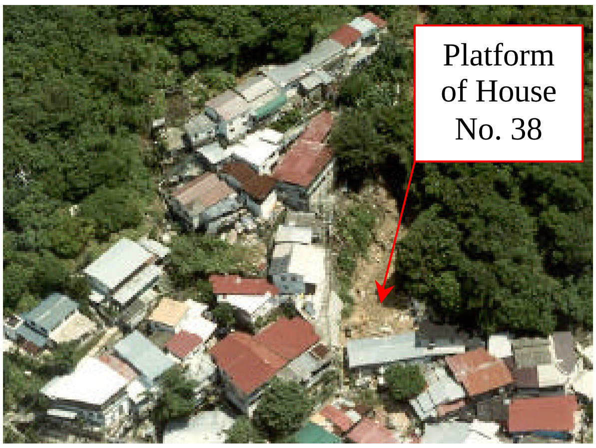

Plate 7 - Oblique Aerial Views of the 1999 Sham Tseng San Tsuen Debris Flow
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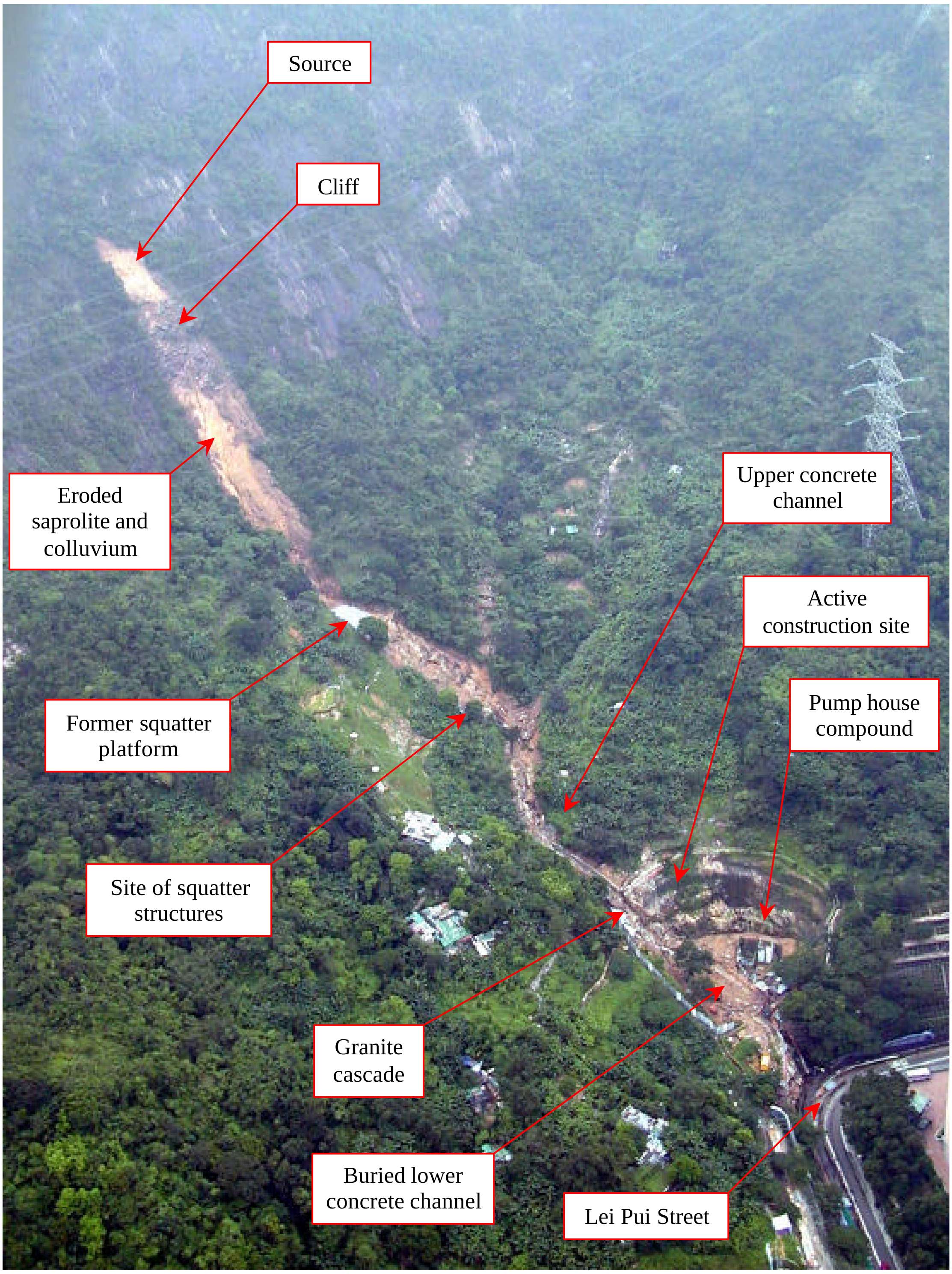

Plate 8 - Oblique Aerial View of the 2001 Lei Pui Street Debris Flow
(Photograph taken on 10 September 2001)

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APPENDIX A

TRAVEL ANGLE AND TRAVEL DISTANCE VERSUS LANDSLIDE VOLUME

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A.1 TRAVEL ANGLE AND TRAVEL DISTANCE FOR OPEN HILLSLOPE FAILURES

Figures A1 and A2 show the travel angle and travel distance vs. landslide volume for open hillslope failures in Hong Kong based on data from Wong et al (1997), the Tsing Shan Foothills Natural Terrain Landslide Study (MGSL, 2002b), Hungr (1998) and Ayotte & Hungr (1998). Despite the scatter in the data, there are good trends indicating a lower travel angle and larger distance of travel with increasing landslide volume.

Variations in the range of travel angle and travel distance for a particular landslide volume are mainly due to local influencing factors which are best summarised by the key findings of the study on debris mobility of the 1993 Lantau landslides (Wong et al, 1997):

(a) The type of landslide and debris movement (e.g. planar failure vs. channelised flow) is important.

(b) The travel distance of landslide debris appears to be principally a function of the failure mechanisms, properties of the material that control the failure and whether channelisation and significant entrainment of debris can occur.

(c) Debris runout appears to be affected by the scale of the failure which could affect the mechanisms of debris movement.

(d) The apparent angle of friction between the debris and the underlying material can be much lower than the angle of shearing resistance of the slope-forming materials.

In the case of slopes with well-defined, flat-lying areas at their foot, the travel angle concept can provide a reasonable resolution in predicting debris travel distance. However, in the case of natural terrain landslides, the downslope angle may be comparatively steep with only a small difference between the travel angles. In such circumstances, the use of a travel angle to predict debris mobility is likely to be unreliable. In the case of most open-hillside failures, much of the debris is deposited on the hillside and one possible improvement in the prediction of debris travel is to use a combination of travel angle and upper-bound travel distance, both related to the different mechanisms and scale of failure.

A pragmatic approach advocated by Wong & Ho (1996) is by means of empirical observations based on good quality data and a rational classification of the landslide/debris movement mechanisms with allowance made for possible increase in debris mobility with landslide volume.

Building on this approach, and using good quality data (i.e. the data-set represented in Figures A1 and A2), it is possible to determine a probable lower-bound travel angle and probable upper-bound travel distance for open-hillside landslides. Consideration of such factors as the failure type and the channelisation potential, average slope angle, roughness and vegetation characteristics of the potential debris path may allow a smaller range of probable travel angles and travel distances to be predicted within the extremes of the upper and lower bounds.

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For example, the current data-set comprised mainly landslides with the debris trail vegetation cover consisting of grass with only a few scattered trees or shrubs. For potential landslides of moderate volume and where the debris runout area is covered with dense trees or shrub, the movement of the landslide debris is likely to be affected and the landslide would have a higher travel angle and shorter travel distance than a landslide with a smooth (e.g. grass covered) debris path which is concave in cross-section.

The potential landslide mobility can be described from the four correlation lines shown in Figures A1 and A2 which would correspond to ‘very favourable’, ‘average’, ‘adverse’ and ‘extreme’.

In recognition that further work needs to be conducted to assess the data in terms of local factors which influence the runout distance of landslides within the current data set, it is proposed to adopt the ‘extreme’ correlation line for travel distance vs. landslide volume shown in Figure A2 for the initial prediction of runout distance for open hillslope failures within the standardised barrier framework. For the moderate-scale open hillslope failure design events of 50 m^3 and 100 m^3 considered within the framework, Figure A2 indicates that the corresponding ‘extreme’ runout distances are about 75 m and 120 m respectively, measured from the lower edge of the source area of potential failures.

From consideration of the travel angle range and the clustering of data points shown in Figure A1, it is proposed that the ϕ-values used for assessment of the mobility of the landslide debris are 30° and 25° for debris volumes of 50 m^3 and 100 m^3 respectively.

A.2 TRAVEL ANGLE AND TRAVEL DISTANCE FOR CHANNELISED DEBRIS FLOWS

The travel angle and travel distance method discussed above for open hillslope failures should be used with caution for channelised debris flows because of the higher potential for entrainment and the lower ϕ-value which according to previous back analyses by Hungr (1998), Ayotte & Hungr (1998) and MGSL (2000), is between approximately 6° and 11°. In theory, if the debris remains confined in a channel, which is inclined at an angle only slightly higher than ϕ the debris may continue to move downhill. It is therefore recommended that an angle of reach or travel distance approach should not be relied upon for the assessment of channelised debris flow mobility within the standardised barrier framework. As there are also much less data available for channelised debris flows on which to correlate travel angle and travel distance with debris volume, an analytical approach has been used for the prediction of travel distance for channelised debris flows.

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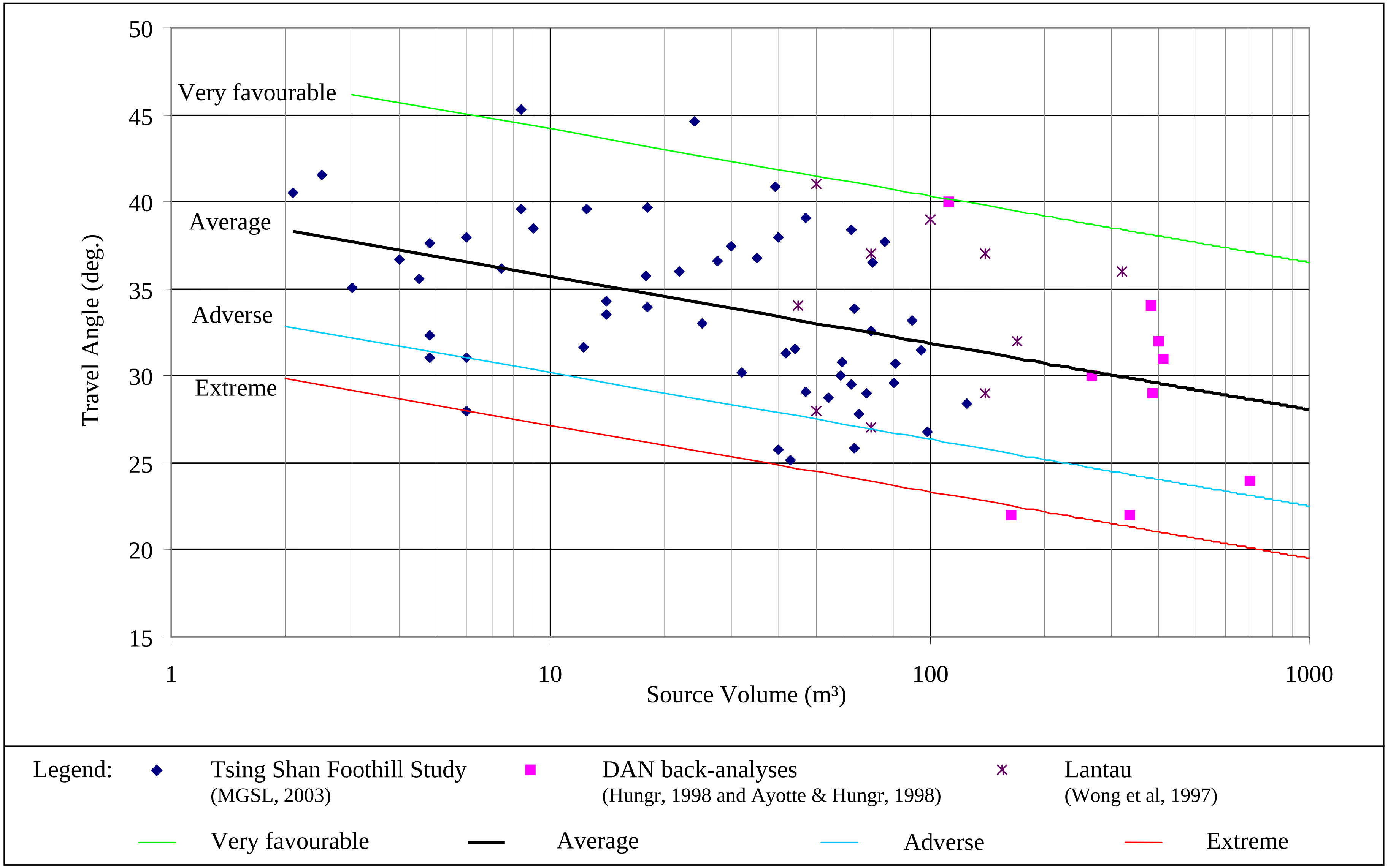

Figure A1 - Travel Angle Vs. Landslide Volume for Open Hillslope Failures

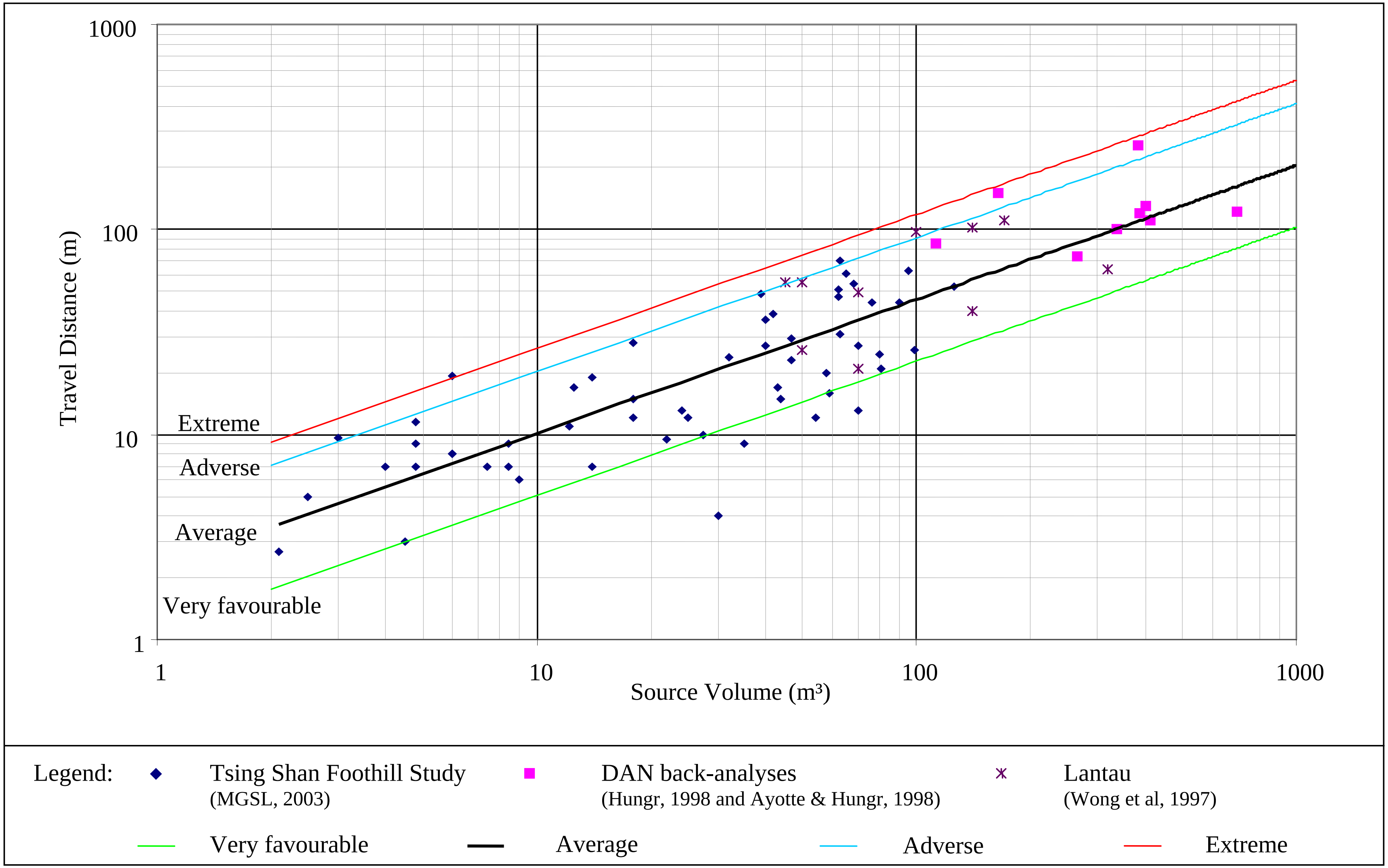

Figure A2 - Travel Distance Vs. Landslide Volume for Open Hillslope Failures

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APPENDIX B

RESULTS OF BACK ANALYSES OF THE 1990 TSING SHAN, 1997 SHA TAU KOK, 1999 SHAM TSENG SAN TSUEN AND 2001 LEI PUI STREET DEBRIS FLOWS

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B.1 INTRODUCTION

This Appendix describes the back analyses of four debris flows in Hong Kong. Three of these have been back analysed using both the DAN model and the Debriflo model, while the recent Lei Pui Street debris flow has been back analysed using the Debriflo model only.

B.2 1999 SHAM TSENG SAN TSUEN DEBRIS FLOW

The field data collected from this event are considered to be amongst the most reliable in Hong Kong for the testing of the basic functions of a computer program which models debris flows because:

(a) the data are very comprehensive and comprise field measurements checked by detailed surveying,

(b) the debris flow did not involve significant entrainment or deposition along most of the debris trail, thereby providing a good check on the applicability of the basic model without having to consider any approximations which have to take into account the influence of entrainment and deposition, and

(c) the debris flow was confined within a rocky channel which was not significantly deepened during the course of the debris flow event, thereby providing some certainty that the height of the debris marks along the ravine represent the true height of the largest debris pulse.

The debris flow (Plate 7) was back analysed by MGSL and Hungr using the Debriflo and DAN programs respectively. The same field data were used, and both back analysis methods gave ϕ = 5.7°. The Debriflo program assumed a basic ξ = 500 m/s^2 which is varied in the program to simulate the effect of the changing channel cross-section along the irregular drainage line (actual range of 180 to 480 m/s^2). A best-fit profile for the DAN analysis was obtained by assuming a constant ξ = 200 m/s^2.

The results of the back analyses are shown in Figures B1 and B2. From Figure B1 it can be seen that:

(a) The velocity profiles provide a good match with the velocities calculated from field superelevation measurements.

(b) The calculated average flow height profiles provide a good match with the average height profile derived from field measurements.

Some minor differences in profiles given by the two programs are due to the following differences in the modelling techniques:

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(a) In the DAN program, sharp changes in vertical gradient need to be 'smoothed-out' to maintain numerical stability between the discretised blocks of the debris flow model. In the Debriflo program, no smoothing is necessary to maintain numerical stability. This results in the Debriflo output giving sharp peaks in velocity at 'waterfalls', while the DAN output gives more subdued peaks.

(b) Given that the Debriflo program directly models the increased frictional and turbulent effects of deep, narrow cross-sections, while the DAN program uses a constant parabolic shape factor to approximate a channel, the flow height is generally higher and the velocity lower (and more in line with field observations) in the Debriflo output at narrow, confined sections than it is in the DAN output.

Despite the minor differences in the modelling techniques of the two programs, the overall good fit with the field data and similarly derived rheological parameters gives confidence that both programs have approximated the behaviour of this debris flow in a realistic manner.

B.3 1990 TSING SHAN DEBRIS FLOW

The field data collected from this event are considered to be very reliable and comprehensive (King, 1996). However, the debris flow occurred over a period of at least one hour, and involved successive pulses of debris that gradually entrained material and enlarged the eroded channel, which probably obliterated some of the evidence left by the initial pulse.

As the accurate modelling of such a complex event would defy most computer programs and in any event, would not be possible due to the fragmentary nature of the field evidence for each pulse, the debris flow was back analysed as a single pulse, in which the maximum debris heights recorded along the debris trail were matched during the modelling.

The debris flow was back analysed by MGSL and Hungr using the Debriflo and DAN programs respectively. The same field data were used, and both models assumed rheological parameters of $\phi = 11.3°$ and $\xi = 500$ m/s². For the initial conditions, the DAN analysis commenced the flow from Chainage 200 at the location of the parent landslide (Plate 5), while the Debriflo analysis commenced the flow from the trigger landslide at Chainage 20, with gradual entrainment along the debris path to match the flow heights from field measurements.

The results of the back analyses are shown in Figures B3 and B4. From Figure B3 it can be seen that:

(a) The velocity profiles are similar and provide a good match with the velocity calculated from field superelevation measurements at CH 350.

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(b) The calculated average flow height profiles are very similar and correlate well with the average height profile derived from field measurements.

Minor differences in the velocity profile given by the two models are due to the differences in the modelling techniques described above. Despite these differences, the overall good fit with the field data and similar results using the same rheological parameters gives confidence in the two models, even though the debris flow was conservatively assumed to have occurred as a single pulse.

B.4 1997 SHA TAU KOK DEBRIS FLOW

This event (Plate 6) was modelled by Ayotte & Hungr (1998) using field data collected by Ayotte. However, the only published field data available for further back analysis consist of 1:1000 scale topographic plans on which the edges of the flow path have been marked.

The debris flow was back analysed by MGSL and Hungr using the Debriflo and DAN programs respectively, and both models assumed rheological parameters of ϕ = 5.7° and ξ= 500 m/s² along the channelised part of the debris trail. The initial open hillslope section of the landslide between the drainage line (Ch 170) and the source was modelled by both programs assuming a friction-only rheological model.

The results of the back analyses are shown in Figures B5 and B6. From Figure B5 it can be seen that:

(a) The overall runout distance obtained by the two models is similar.

(b) The velocity profiles are similar near to the source, but the Debriflo velocity along the channelised part of the trail is generally much higher than that obtained in the DAN analysis.

(c) The calculated average flow height profile for the Debriflo analysis is generally much higher along the channelised part of the trail than the flow height profile obtained from the contours and plotted edges of the debris trail shown in the published data.

The differences between the two models appear to be for two main reasons:

(a) A large amount of debris is shown as being deposited close to the scar in the Ayotte & Hungr results. Therefore, the active volume assumed in the DAN analysis in the channelised section would be much less than the reported 1400 m³ that is assumed in the Debriflo model. This results in much higher velocities and flow heights when the Debriflo model is run.

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(b) The channel cross-section profiles used in the Debriflo analysis are based on poor topographic data without the benefit of field measurements or surveys. The channel profiles can therefore only be expected to roughly approximate the actual eroded channel profile.

A check with the Debriflo model assuming a similar velocity and flow height to the DAN analysis from the commencement of the channelised section at Ch 170 gives a velocity and flow height very similar to the DAN results. This example illustrates the importance of the quality and completeness of the input data for the back analysis.

B.5 2001 LEI PUI STREET DEBRIS FLOW

This event (Plate 8) was analysed using the Debriflo model, based on the high quality data obtained from field measurements and surveying. The initial mass after commencement of failure was modelled as a sheet of saturated solids inclined at 41° with a saturated bulk density of 2400 kg/m³. A ϕ-value of 18.6°, zero turbulence and a velocity of 0.01 m/s were assumed to represent the initial conditions.

The parameters of the debris were gradually changed to simulate the breaking up and mixing of the mass as it cascaded down the cliff face. After Chainage 50, it is assumed that the mass was a mixture of failed colluvium, boulders and entrained material with a saturated unit weight of 1970 kg/m³, ϕ = 11.3° and ξ = 500 m/s².

Several analyses were carried out, with the debris parameters and volume of deposition adjusted to obtain best-fit velocity and calculated average debris height profiles with the field measurements of velocity and average debris height.

The results of the back analyses are shown in Figures B7 and B8. From Figure B7 it can be seen that the velocity profile provides a good match with the velocities calculated from field superelevation measurements and structural damage calculations at various points along the debris trail. At the same time, the calculated flow height profile provides a good match with flow height field measurements.

In order to match the field estimates of velocity and debris height below Chainage 190, approximately 80 m³ of active debris was assumed to be lost from the debris front and the ϕ-value reduced to about 8°. On site, it was observed that about 12 m³ of fresh debris consisting of boulders up to 1 m in length was scattered up a major drainage line for about 15 m. A much larger amount of bouldery debris front material would have been initially deposited in the mouth of the drainage line in the form of a temporary levee (later removed by a small dam-break). The removal of bouldery material and the mixing with flood-water from the much larger catchment of the drainage line is considered likely to have increased the mobility of the initial debris front.

Owing to the good correlation with the field data, the model is considered to represent a good approximation of the mobility of the initial debris front.

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B.6 REFERENCES

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Figure B1 - Velocity and Height of 1999 Sham Tseng San Tsuen Debris Flow

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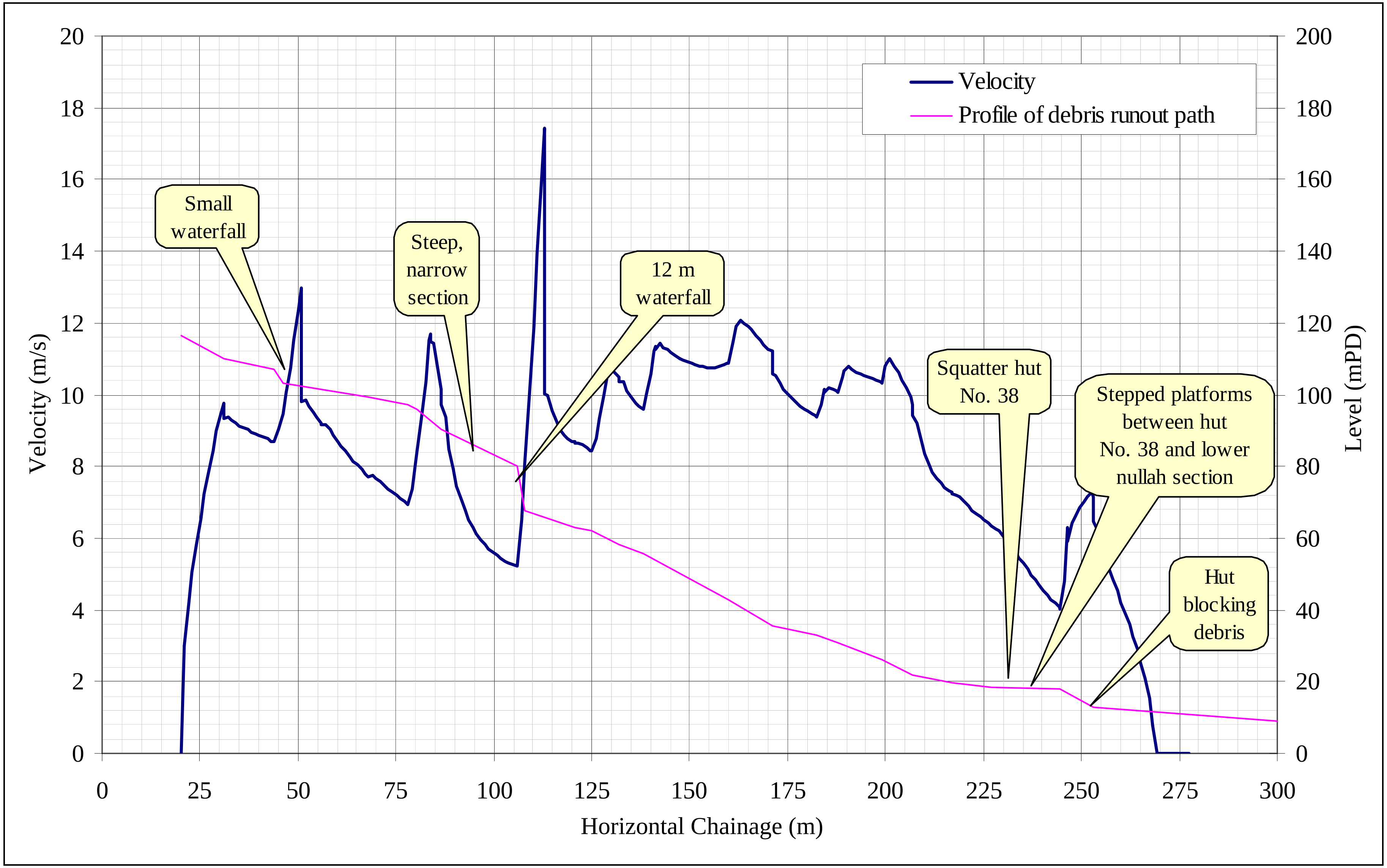

Figure B2 - Velocity and Profile of 1999 Sham Tseng San Tsuen Debris Flow

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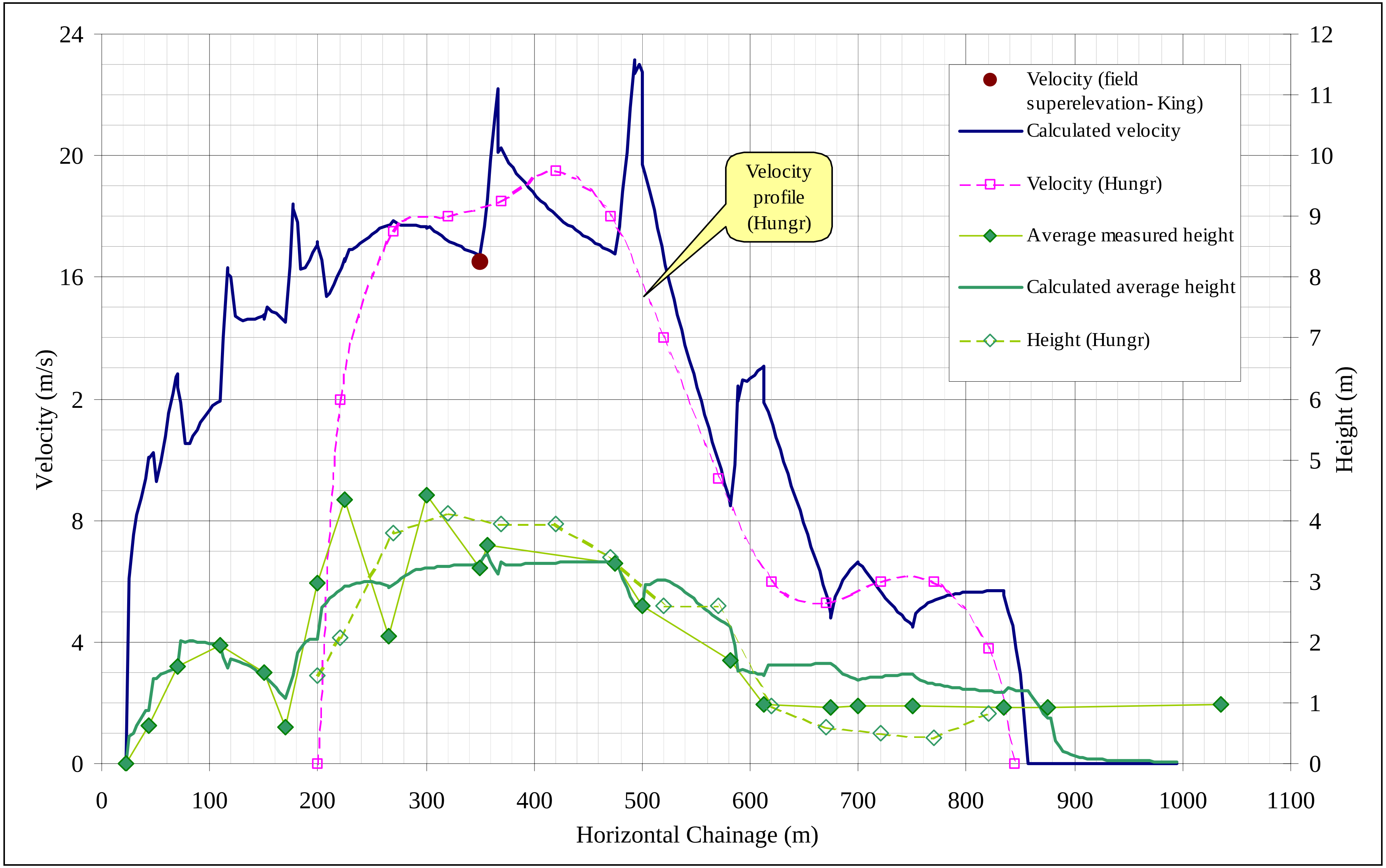

Figure B3 - Velocity and Height of 1990 Tsing Shan Debris Flow

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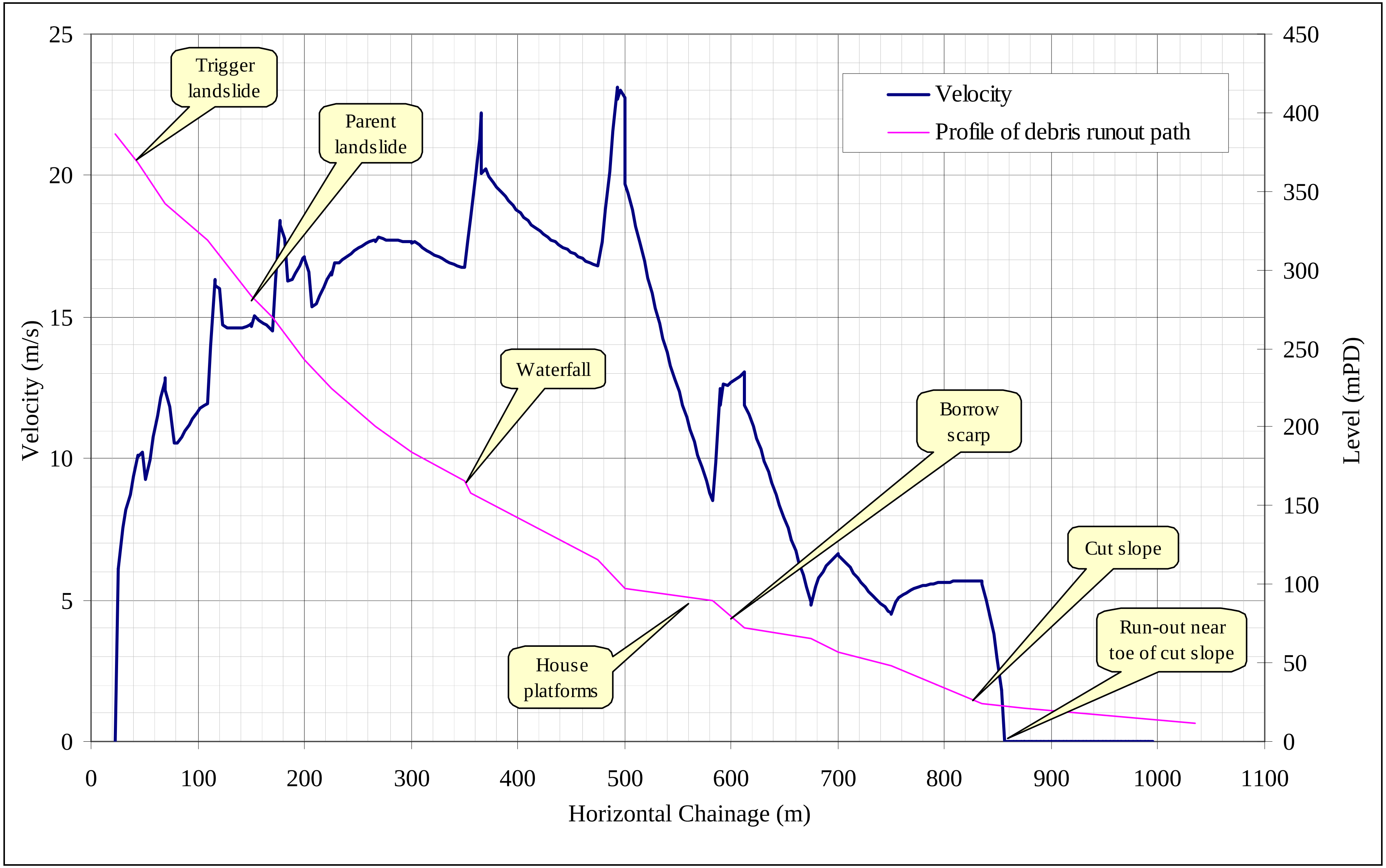

Figure B4 - Velocity and Profile of 1990 Tsing Shan Debris Flow

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Figure B5 - Velocity and Height of 1997 Sha Tau Kok Debris Flow

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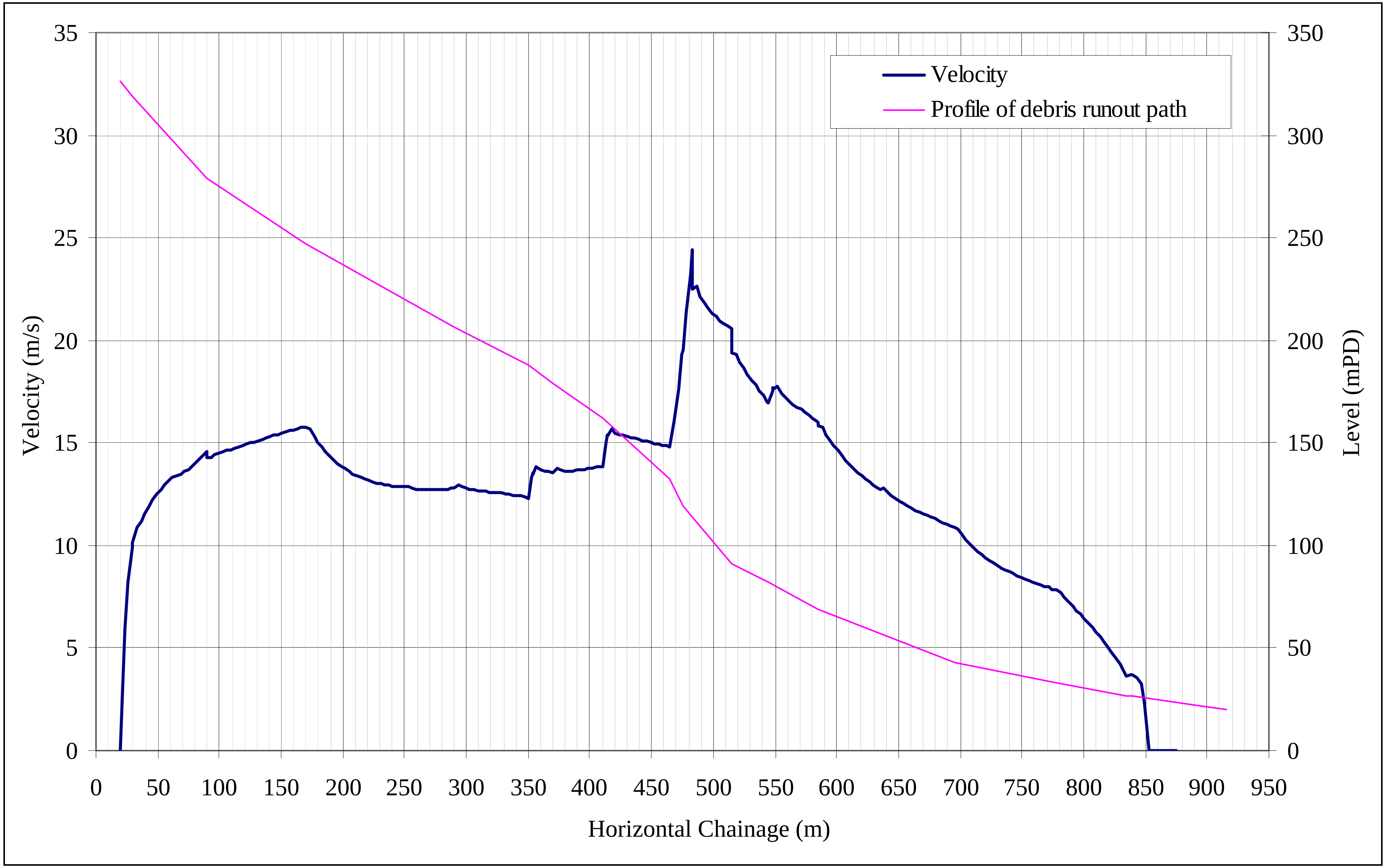

Figure B6 - Velocity and Profile of 1997 Sha Tau Kok Debris Flow

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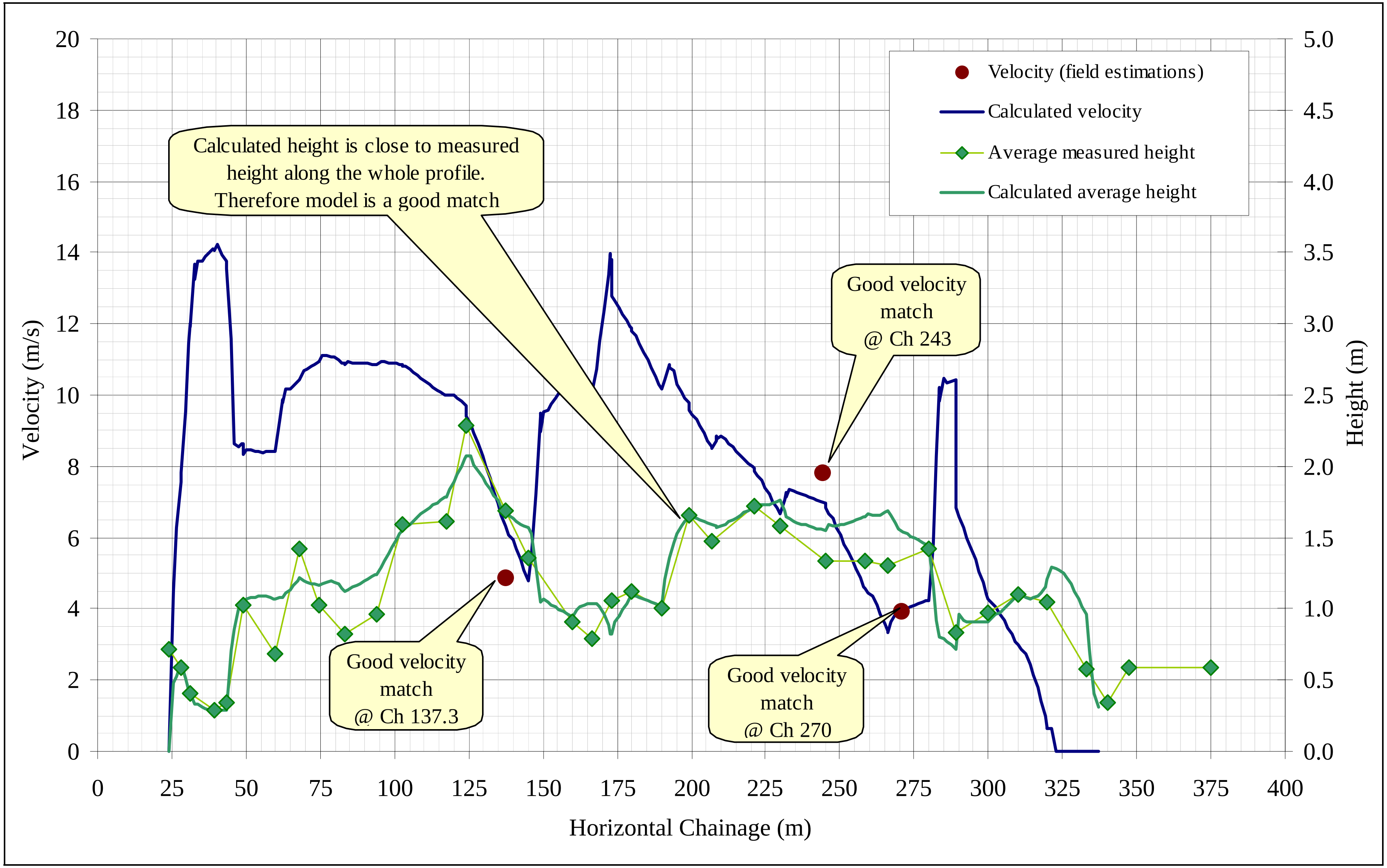

Figure B7 - Velocity and Height of 2001 Lei Pui Street Debris Flow

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Figure B8 - Velocity and Profile of 2001 Lei Pui Street Debris Flow

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APPENDIX C

DETERMINATION OF THE DEBRIS RUNOUT DESIGN PROFILE FOR CHANNELISED DEBRIS FLOWS

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C.1 INTRODUCTION

The results of back analyses and debris height observations of previous debris flows in Hong Kong have been taken into consideration in determining the worst-case debris height and velocity for a given design event of a certain volume. The calibrated results have then been applied to a series of generalised design debris runout channels of constant cross-section, each comprising three tangents of varying inclinations. The debris height, velocity, impact forces and debris run-up height have then been computed for each design profile in accordance with GEO Report No. 104 (Lo, 2000).

The structural capacity of each of the standardised barriers has been determined by conventional structural calculations, and the capacities were then compared with the debris flow forces and run-up height within the run-out area for each design profile. A set of barrier design charts has then been prepared in which the minimum acceptable distance from the start of the run-out area and minimum barrier width are given for each design profile. An example of a barrier design chart is shown in Table C1.

This Appendix describes the procedures and compliance conditions for determining the design profiles that are applicable to the site-specific ground profile under consideration and the potentially suitable barrier locations within the run-out area.

C.2 GLOSSARY OF TERMS

The terms used in this Appendix relating to the determination of the design profiles have the meanings given below:

Conditions for Template Fitting. The four conditions for template fitting are defined in Figures C2 and C3. These conditions ensure that the design profiles selected to model the ground profile and the determination of potentially acceptable barrier locations will be suitably conservative.

Design Profile. The Design Profile to determine the selection of a suitable barrier from the barrier design tables (refer to Table C1). The Design Profile comprises a 34° upper tangent, and middle and lower tangents that vary in angle as shown in Figures C2 and C3. Any combination of tangents that can be fitted to the ground profile, and which meets the Conditions for Template Fitting, may constitute a Design Profile. The final, optimised Design Profile is that which results in the most favourable barrier options with reference to the design tables.

Ground Profile. The Ground Profile is the profile of the actual drainage line or debris flow path under consideration, as extended from the highest potential site of instability to the lowermost point within the boundary of the subject site.

Node Point. Node Point Nos. 1 and 2 lie at the intersection of the lower tangent with the middle tangent and the intersection of the middle tangent with the upper tangent respectively (see Figures C2 and C3). Node Point No. 1 marks the start of the run-out area.

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Step. A step in the Ground Profile within the lower tangent area (i.e. runout area or runout zone) that is steeper than the lower tangent being considered for incorporation into the Design Profile.

Template. Templates A and B are shown in Figures C2 and C3. The templates reflect all channel configurations used to prepare the barrier design tables.

Three Tangent System. The system by which the Design Profile is derived from the actual Ground Profile by application of either Template A or B and the Conditions for Template Fitting.

C.3 METHODOLOGY

The standardised barrier method relies on the application of the Three Tangent System to produce a simplified debris runout Design Profile from a 'best-fit' of the actual Ground Profile using the methodology shown in the flow chart in Figure C1.

In order to find the optimum Design Profile, the template is moved along the Ground Profile while keeping Node Point No. 1 co-incident with the Ground Profile. The optimum Design Profile will be that which results in the most favourable barrier options with reference to the barrier design tables.

The application of the Three Tangent System to an actual stream channel profile is demonstrated in Examples 1 to 4 in Figures C4 to C10. The complying segments within the lower tangent run-out area are highlighted in blue in the Figures.

Example No. 1 shows a case where there are no acceptable segments of the lower tangent because the overall angle of the Ground Profile in the upper tangent area is steeper than assumed within the standardised barrier framework.

Example No. 2 shows a case where the overall angle of the Ground Profile beneath the upper tangent and middle tangent is acceptable and where a short length of the lower tangent is also potentially acceptable for the location of a barrier. In this case, the angle of the middle tangent is the same as the upper tangent, and so the length of the middle tangent can be taken as zero for the purposes of determining a suitable barrier from the barrier design tables.

Example No. 3 shows a case where there are no acceptable segments of the lower tangent because the overall angle of the Ground Profile in the middle tangent area is steeper than assumed within the standardised barrier framework.

Example No. 4 shows a case where the overall Ground Profile beneath the upper and middle tangents is acceptable. A significant portion of the Ground Profile within the lower tangent run-out area is also acceptable for the potential location of a barrier. The acceptable area is located where the overall and local Ground Profile angles within the lower tangent area are equal to or flatter than the angle of one of the particular lower tangents that can be used to form the Design Profile. In this case, the irregularities and Steps in the lower tangent area lead to different tangent angles being applicable at different distances from the commencement of the run-out area (Node Point No. 1). For this example, four Design

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Profiles need to be referenced in the barrier design tables, all having a common middle tangent of 26°.

The most favourable barrier options will usually be determined from a compromise between increasing the run-out distance to the barrier within the Lower Tangent, reducing the angles of the complying Lower and Middle Tangents, and increasing the length of the Middle Tangent. The suitable locations for barriers will also be limited by the physical constraints of the site.

The maximum height of the upper tangent is limited to 150 m (Figures C2 and C3) in order to ensure that the standardised barrier framework is restricted to a range of landslide elevations, which cover the elevation range of previous landslides in Hong Kong that form the current database for volumes less than 600 m^3.

Additional limitations on the use of the standardised barrier framework for channelised debris flows are also proposed to ensure that the field conditions of application will not result in significantly higher discharges than that assumed for the calibrated channels and that the site conditions lie within the range of conditions that have previously been encountered in Hong Kong for channelised debris flows with volumes up to 600 m^3. These are:

(a) a natural drainage channel with a channelisation ratio of less than 10 (when estimated from 2 m interval topographic contours or site observations) must exist for at least 50 m in horizontal distance above the commencement of the lower tangent, and

(b) at least one 10 m long segment of the channel within the 50 m zone above the lower tangent must have a channelisation ratio of less than or equal to 5 when estimated from topographic contours, detailed survey plans or site observations.

These limitations should ensure that the standardised barrier framework will not be used where fast-moving debris from a nearby open-hillside failure could directly enter the runout area.

C.4 REFERENCES

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Table C1 - 4.5 m High Type 1 Barrier - 150 m³ Design Volume

Design Volume = 150 m³

Barrier Height = 4.5 m

Lower Tangent Angle (deg.)	Middle Tangent Characteristics															
	Angle = 30°			Angle = 26°			Angle = 22°			Angle = 18°			Angle = 14°			
	Length			Length			Length			Length			Length			
	0 m	25 m	50 m	0 m	25 m	50 m	0 m	25 m	50 m	0 m	25 m	50 m	0 m	25 m	50 m	
2.5	6	6	6	6	6	4	6	4	2	6	2	0	6	0	0	Barrier Position
	16	16	16	16	16	16	16	16	16	16	16	16	16	16	16	Min Barrier Length
5	7	7	6	7	6	6	7	4	4	7	2	0	7	0	0	Barrier Position
	16	16	16	16	16	16	16	16	16	16	16	16	16	16	16	Min Barrier Length
7.5	9	9	7	9	7	6	9	6	4	9	4	0	9	0	0	Barrier Position
	16	16	16	16	16	16	16	16	16	16	16	16	16	16	16	Min Barrier Length
10	11	9	9	11	9	7	11	7	6	11	4	0	11	0	0	Barrier Position
	16	16	16	16	16	16	16	16	16	16	16	16	16	16	16	Min Barrier Length
12.5	12	11	11	12	9	9	12	7	6	12	5	0	12	0	0	Barrier Position
	16	16	16	16	16	16	16	16	16	16	16	16	16	16	16	Min Barrier Length

Note: The following minimum requirements should be satisfied:

- The founding material should have the following minimum parameters:

  $c' = 0$ kPa
  $ø' = 35°$
  $\gamma = 19$ kN/m³

- The bearing capacity and overall stability should also be checked by the designer, especially if the barrier is to be constructed on sloping ground. An ultimate bearing capacity of 300 kPa at the founding level over the whole area of the base should be considered in the overall stability and bearing capacity assessments to account for the self-weight of the barrier structure and the debris as well as the impact load of the debris.

- The groundwater level should be maintained at least 1 m below founding level.

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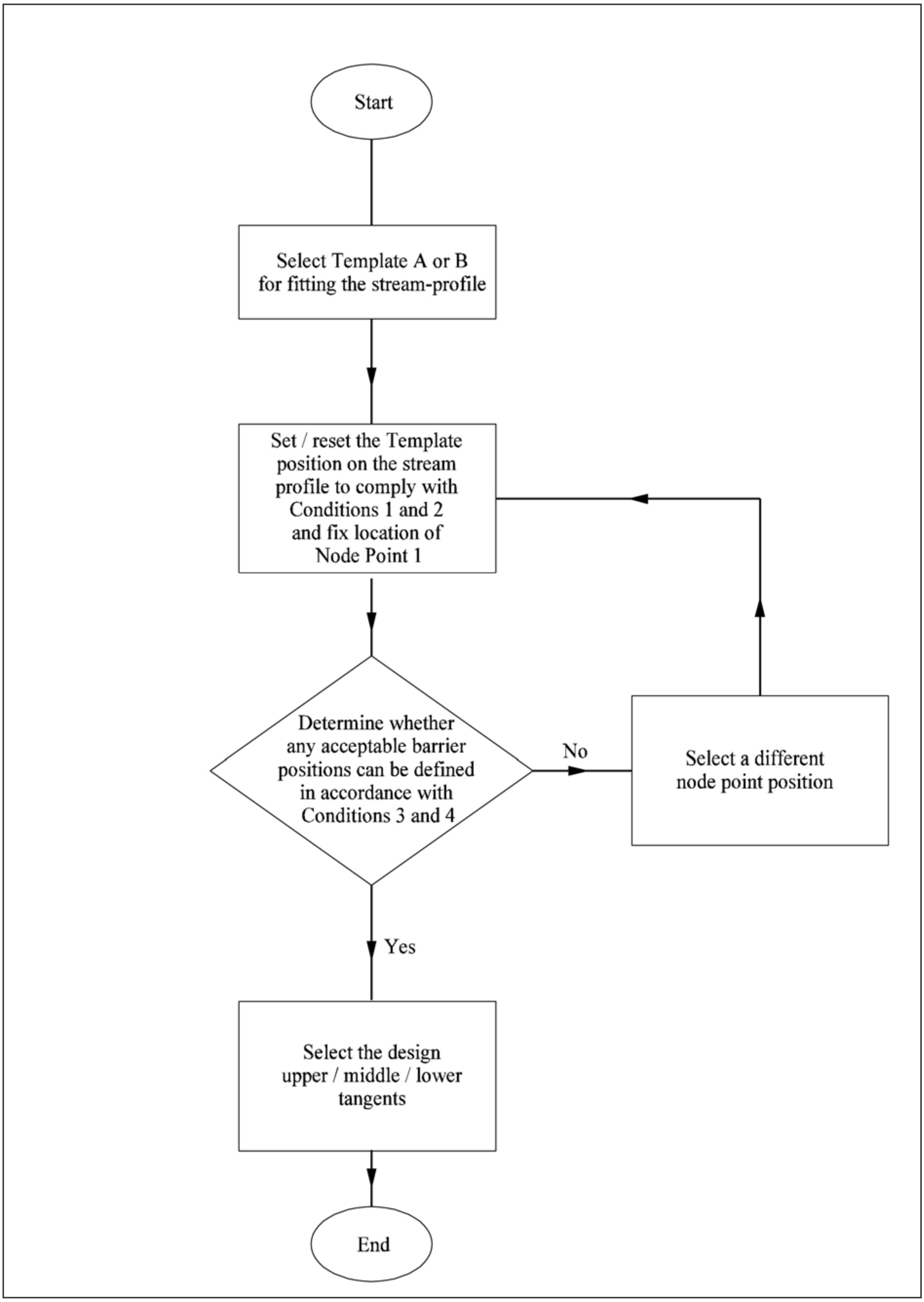

Figure C1 - Flowchart for Testing Compliance of the Ground Profile with the Three Tangent System

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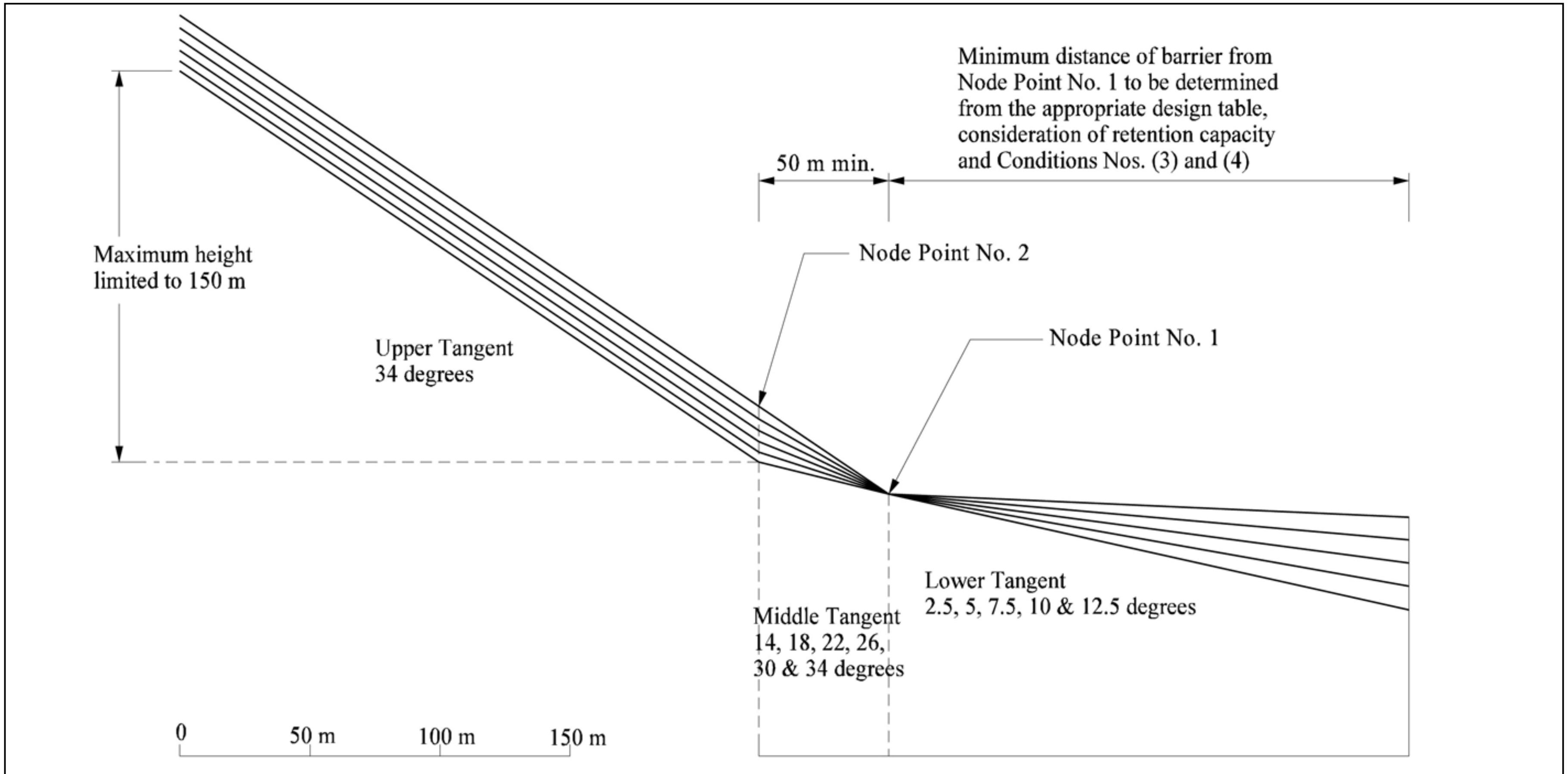

Conditions for Template Fitting:

(1) No parts of the Upper or Middle Tangent lines shall be below the ground profile (Upper Tangent length to be extended as far as the crown of the potential source).
(2) Node Point No. 1 must co-incide with ground level.
(3) Segments of the Lower Tangent over which a barrier can be positioned must lie below the ground profile and must be inclined steeper than or equal to the gradient of the local ground profile.
(4) Steps in the ground profile in the Lower Tangent area are allowable provided that Condition No. (3) is complied with and that the distance of the barrier from the toe of the step shall not be less than the plan length or vertical height of the step (whichever is greater).

Figure C2 - Template A (50 m min. Middle Tangent)

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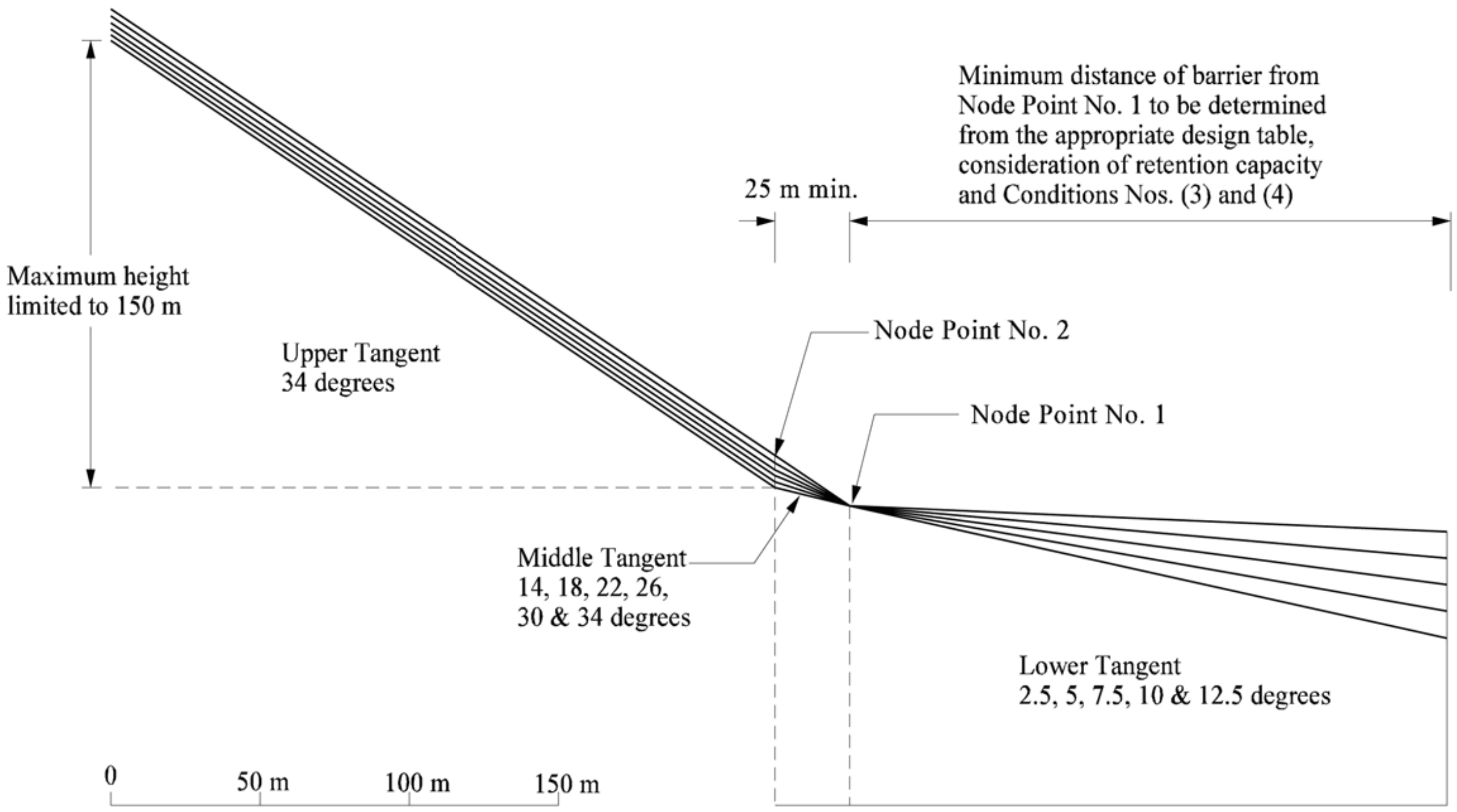

Conditions for Template Fitting:

(1) No parts of the Upper or Middle Tangent lines shall be below the ground profile (Upper Tangent length to be extended as far as the crown of the potential source).

(2) Node Point No. 1 must co-incide with ground level.

(3) Segments of the Lower Tangent over which a barrier can be positioned must lie below the ground profile and must be inclined steeper than or equal to the gradient of the local ground profile.

(4) Steps in the ground profile in the Lower Tangent area are allowable provided that Condition No. (3) is complied with and that the distance of the barrier from the toe of the step shall not be less than the plan length or vertical height of the step (whichever is greater).

Figure C3 - Template B (25 m min. Middle Tangent)

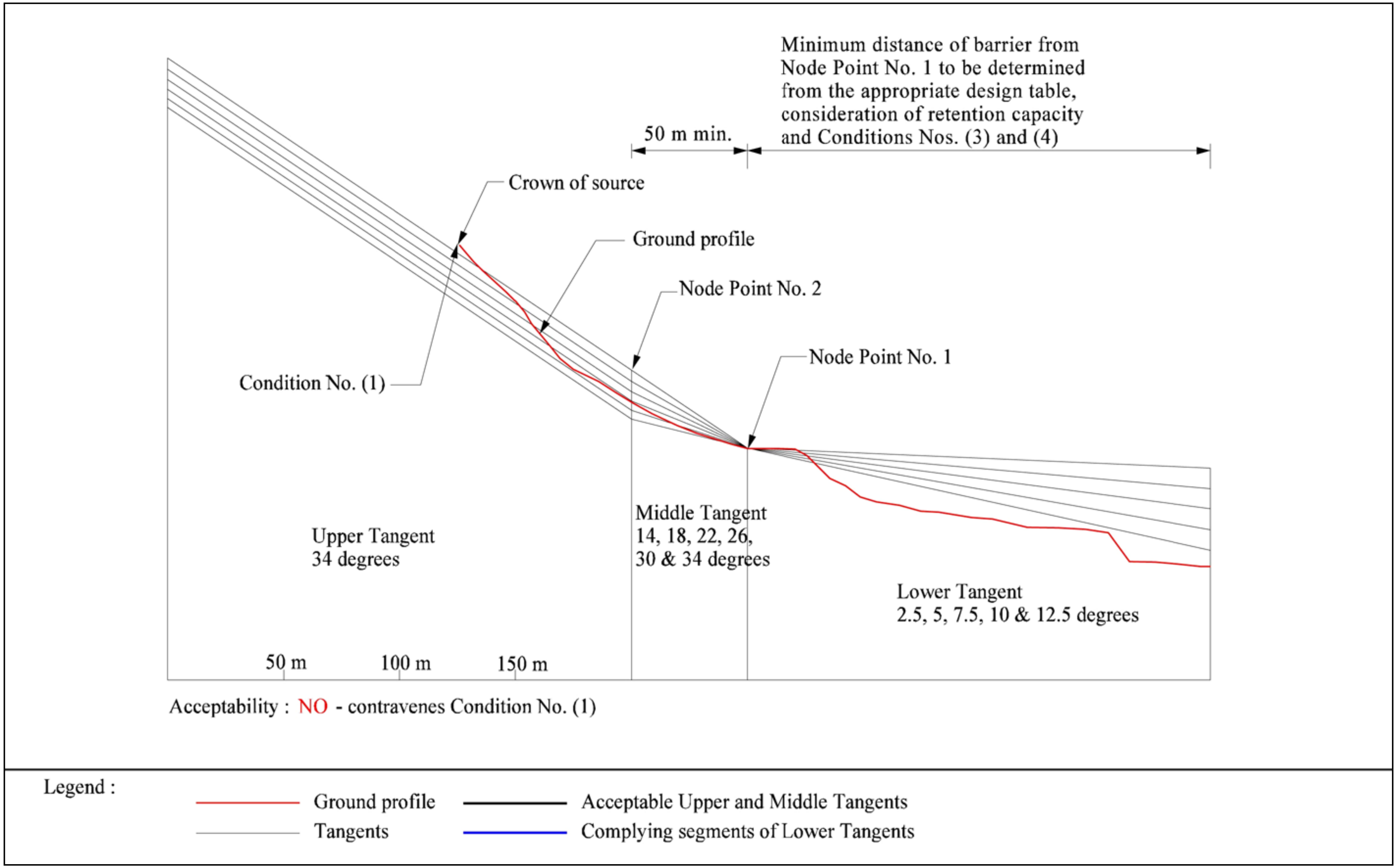

Figure C4 - Fitting of Debris Runout Design Profile with Template A (Example No. 1)

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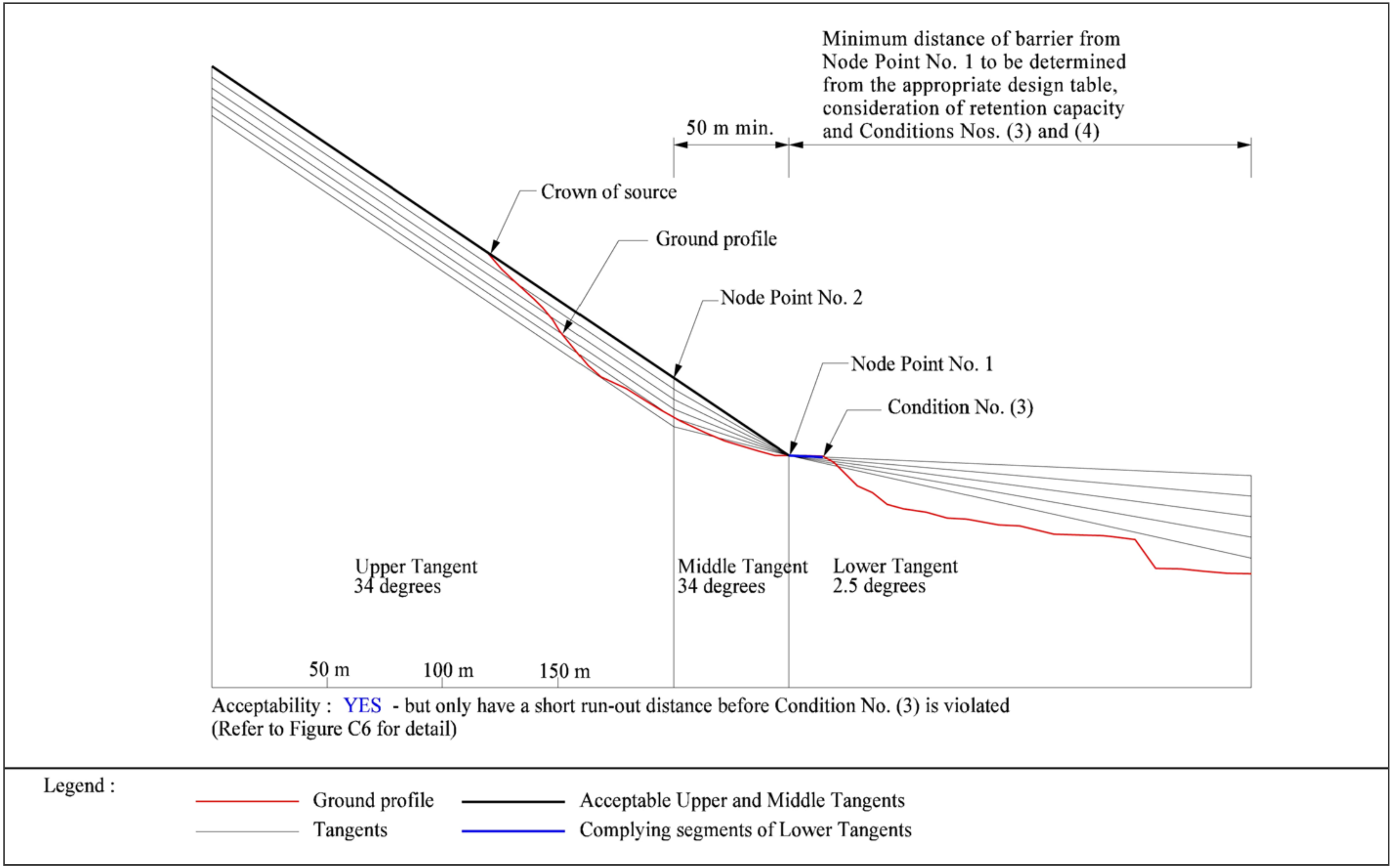

Figure C5 - Fitting of Debris Runout Design Profile with Template A (Example No. 2)

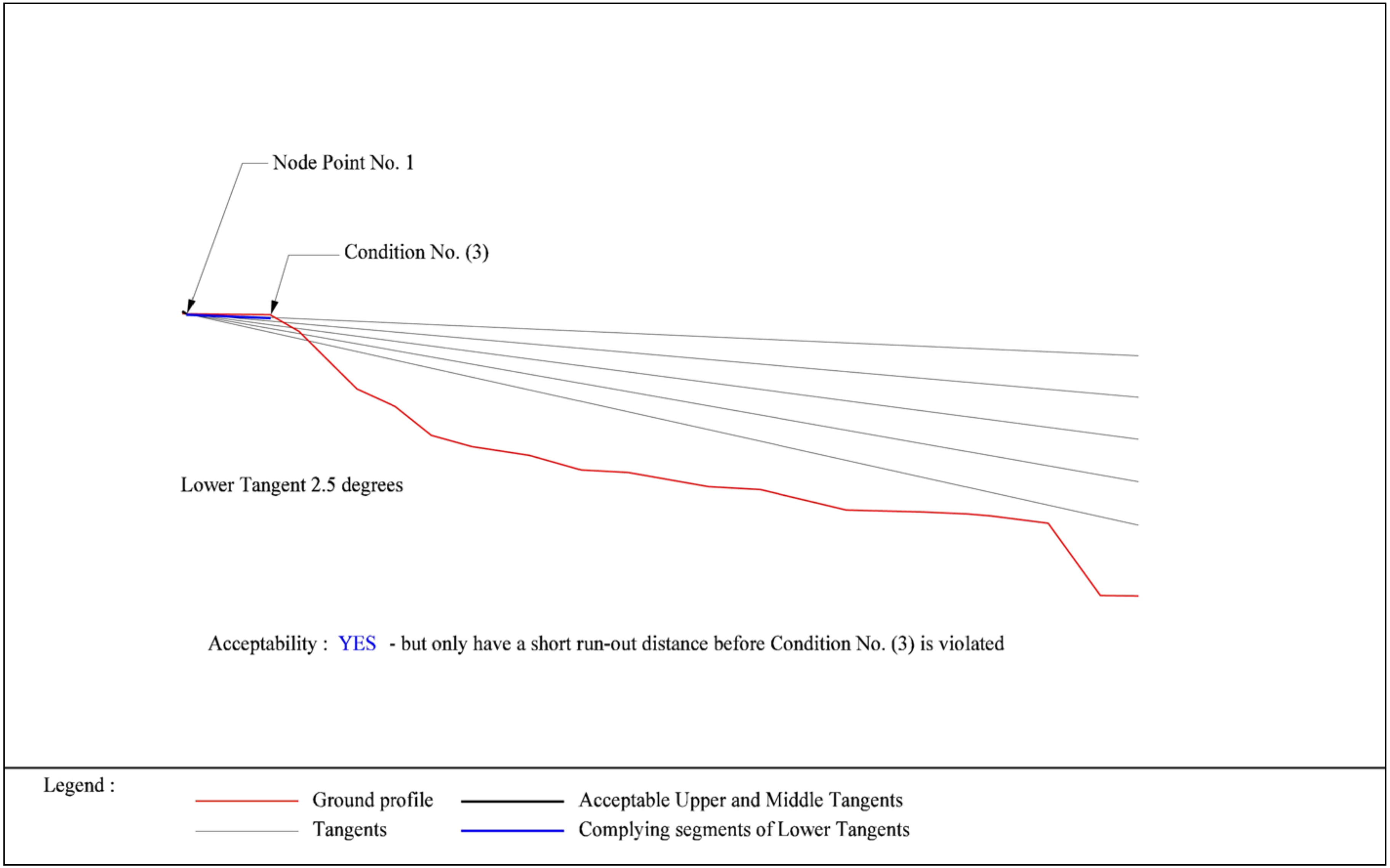

Figure C6 - Fitting of Debris Runout Design Profile with Template A, Close-up at Lower Tangent (Example No. 2)

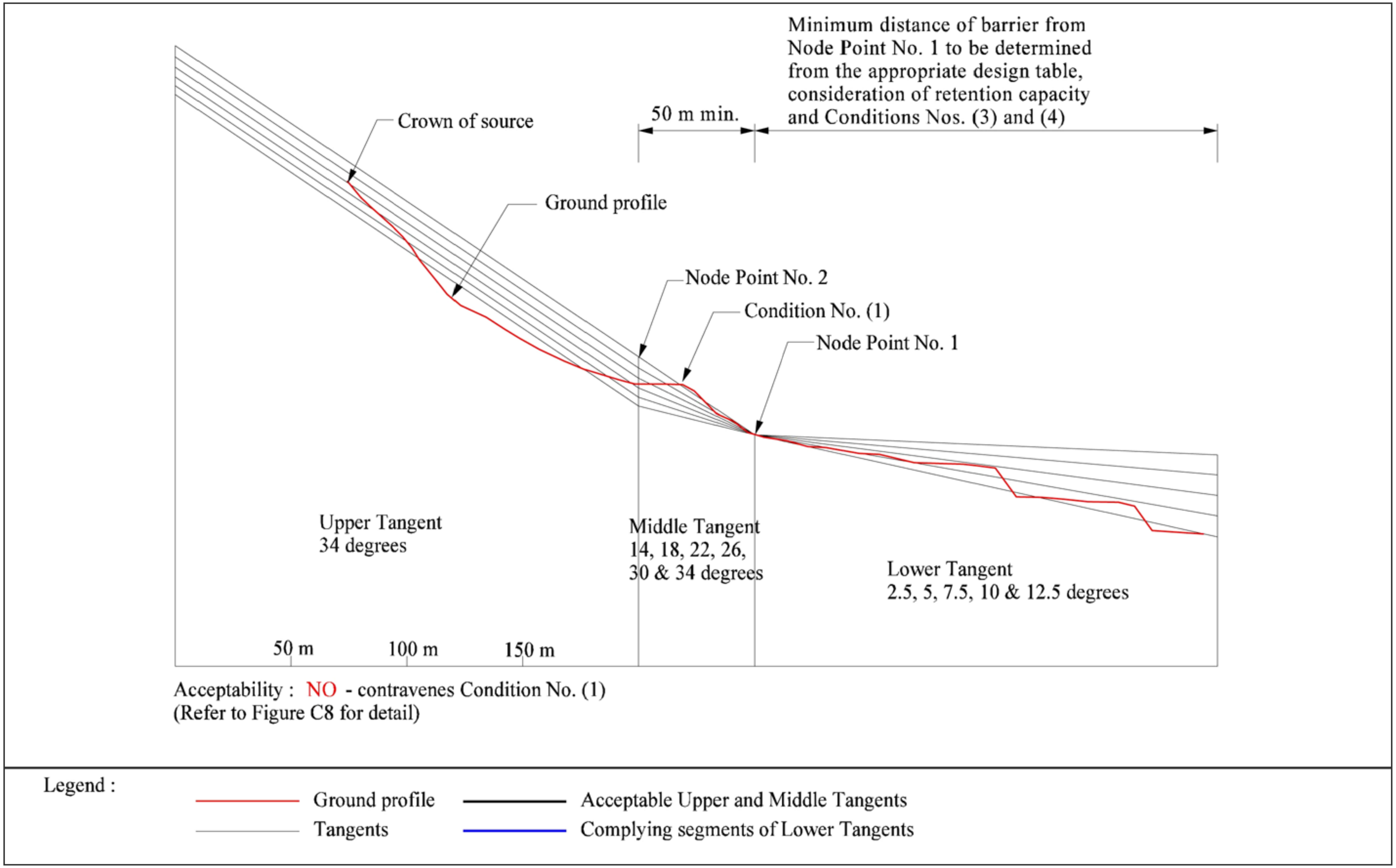

Figure C7 - Fitting of Debris Runout Design Profile with Template A (Example No. 3)

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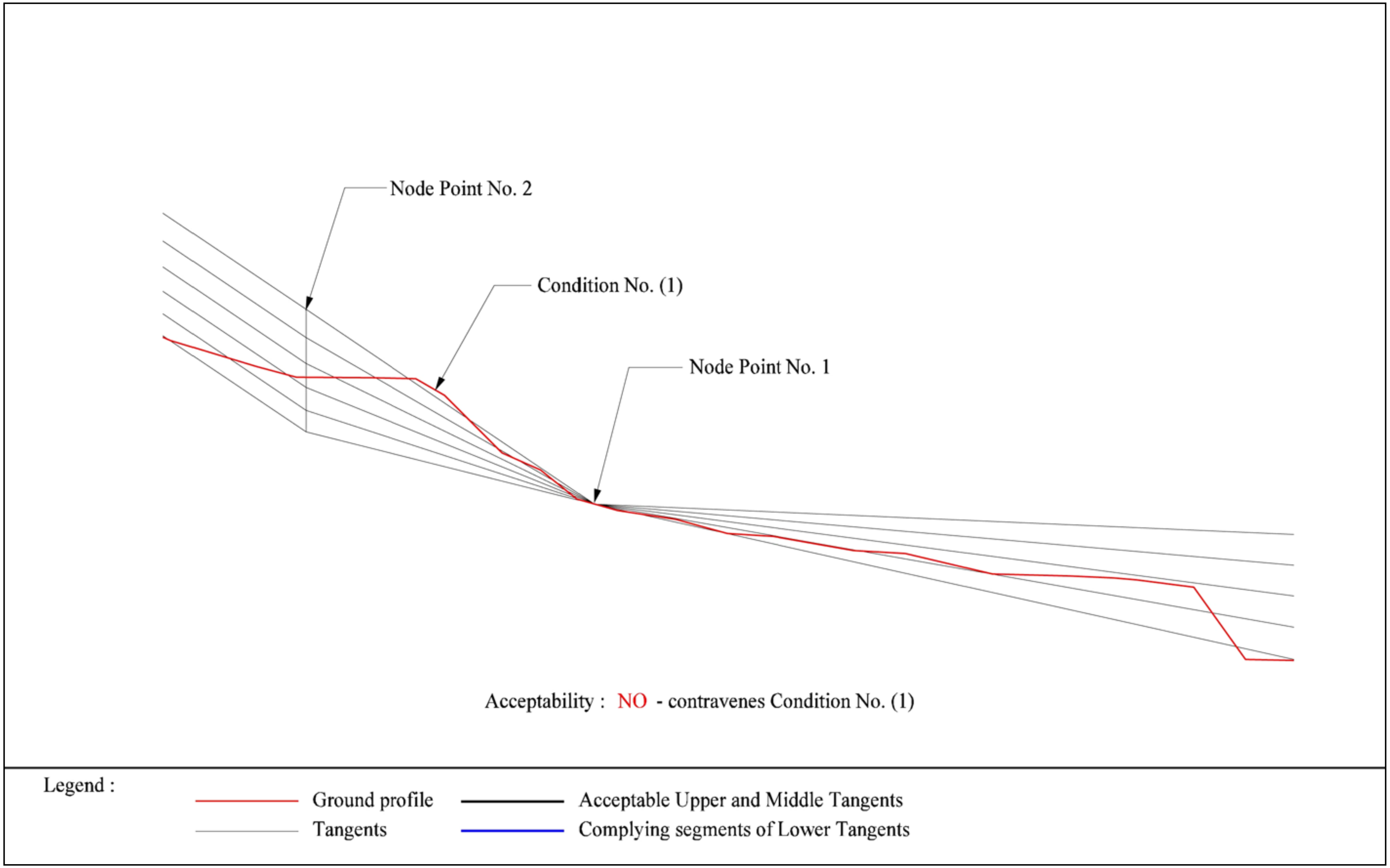

Figure C8 - Fitting of Debris Runout Design Profile with Template A, Close-up at Lower Tangent (Example No. 3)

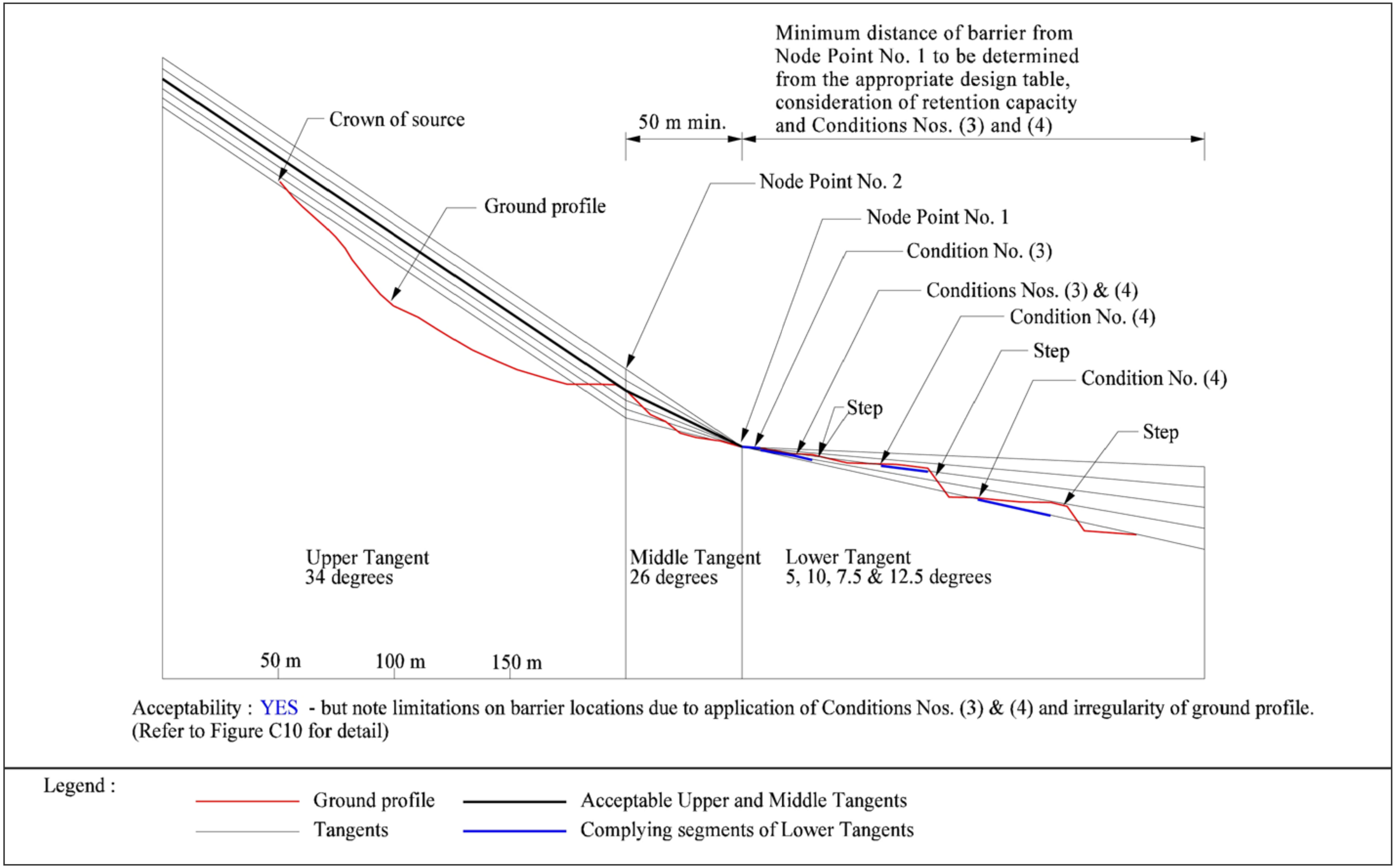

Figure C9 - Fitting of Debris Runout Design Profile with Template A (Example No. 4)

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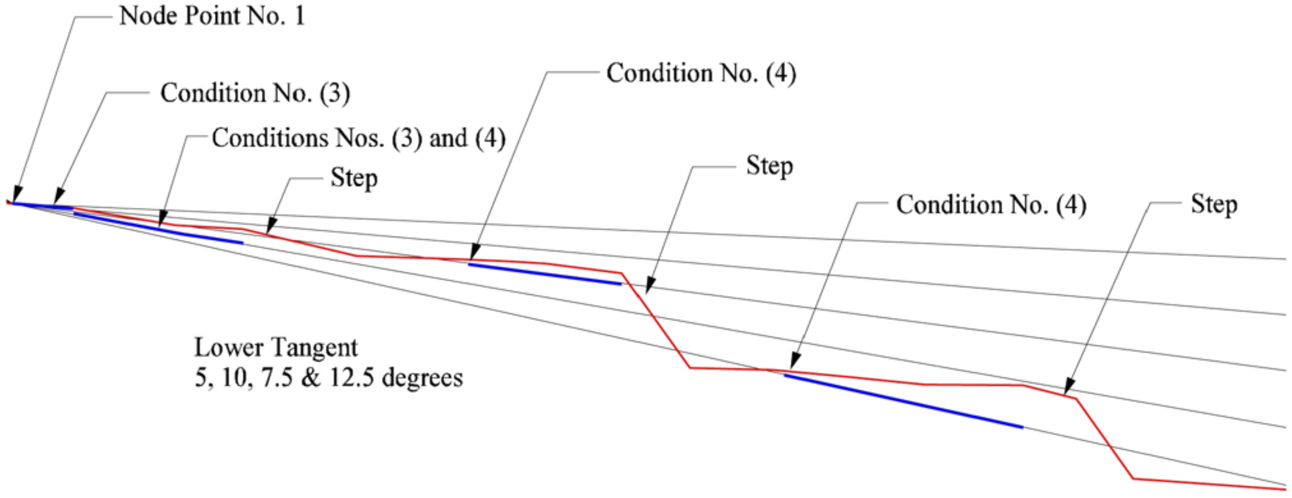

Acceptability : YES - but note limitations on barrier locations due to application of Conditions Nos. (3) & (4) and irregularity of ground profile.

Legend :

Ground profile — Acceptable Upper and Middle Tangents

Tangents — Complying segments of Lower Tangents

Figure C10 - Fitting of Debris Runout Design Profile with Template A, Close-up at Lower Tangent (Example No. 4)

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APPENDIX D

DETERMINATION OF THE DEBRIS RUNOUT DESIGN PROFILE FOR OPEN HILLSLOPE FAILURES

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D.1 INTRODUCTION

The results of back analyses of previous open hillslope failures in Hong Kong have been taken into consideration in determining the worst case debris velocity for a given design event of a certain volume. The calibrated results have then been applied to a series of generalised 'design' slope profiles, each comprising two tangents (as opposed to three tangents in the case of channelised debris flows), with the lower tangent varying in inclination. The debris velocity and kinetic energy of the impacting debris has been computed for each runout slope using the lumped-mass, friction-only approach as recommended in GEO Report No. 104 (Lo, 2000).

The rated structural capacity of a standardised tensioned steel mesh fence was compared with the kinetic energy of the impacting debris for each design profile. A set of barrier design charts have been prepared in which the minimum acceptable distance from the start of the run-out area is given for each design profile. As an illustration, an example of a barrier design chart is shown in Table D1.

This Appendix describes the procedures and compliance conditions for determining the design profiles that are applicable to the site-specific ground profile under consideration and the potentially suitable barrier locations within the run-out area.

D.2 GLOSSARY OF TERMS

The terms used in this Appendix relating to determination of the design profiles have the meanings given below:

Conditions for Template Fitting. The four conditions for template fitting are defined in Figure D1. These conditions ensure that the design profiles selected to model the ground profile and the determination of potentially acceptable barrier locations will be suitably conservative.

Design Profile. The Design Profile to determine the selection of a suitable barrier from the barrier design tables (refer to Table D1). The design profile comprises a 34° upper tangent and a lower tangent that varies in angle as shown in Figure D1. Any combination of tangents that can be fitted to the ground profile, and which meets the Conditions for Template Fitting, may constitute a Design Profile. The final, optimised Design Profile is that which results in the most favourable barrier options with reference to the design tables.

Ground Profile. The Ground Profile is the profile of the actual hillside under consideration, as extended from the highest potential site of instability to the lowermost point within the boundary of the subject site.

Node Point. Node Point No. 1 lies at the intersection of the lower tangent with the upper tangent (see Figure D1). This point marks the start of the run-out area.

Step. A step in the Ground Profile within the lower tangent area that is steeper than the lower tangent being considered for incorporation into the Design Profile.

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Template. The template for open-hillside failures is shown in Figure D1. The template reflects all design slope configurations used to prepare the barrier design tables.

Two Tangent System. The system by which the Design Profile is derived from the actual Ground Profile by application of the design Template and the Conditions for Template Fitting.

D.3 METHODOLOGY

The standardised barrier method relies on the application of the Two Tangent System to produce a simplified Design Profile from a 'best-fit' of the actual Ground Profile using the same methodology as shown in the flow chart in Figure C1, Appendix C for channelised debris flows.

In order to find the optimum Design Profile, the template is moved along the Ground Profile while keeping Node Point No. 1 co-incident with the Ground Profile. The optimum Design Profile will be that which results in the most favourable barrier options with reference to the barrier design tables.

The application of the Two Tangent System to an actual hillside profile follows the same principles as those demonstrated in Examples 1 to 4 in Figures C4 to C10 of Appendix C for the upper and lower tangents.

For the moderate-scale open hillslope failure design events of 50 m^3 and 100 m^3 considered within the framework, Figure A2, Appendix A indicates that the corresponding 'upper bound' (extreme) runout distances are about 75 m and 120 m respectively measuring from the lower edge of the source area of the failures (Appendix A). The recommended runout distances (measured from the lower edge of the source area of potential landslides) to be used for assessment purposes are 75 m and 120 m for open hillslope failure design events of 50 m^3 and 100 m^3 respectively. Beyond these distances, it is not considered necessary to construct a barrier to arrest coherent landslide masses. It should, however, be recognised that individual boulders from the debris front may travel further than the distance of the coherent mass. The designer is advised to consider whether the potential hazard of boulder 'roll-out' from the landslide debris is a concern and if so, whether a boulder fence to cater for this is warranted or not. For example, Evans & Hungr (1993) suggest that the above hazard should be assessed for a runout path that is steeper than 23° based on their experience with sizeable landslides in Canada. The design of the boulder fence for such scenario, if considered necessary by the designer, is outside the scope of the present framework.

The maximum height of the upper tangent is limited to 80 m (Figure D1) in order to ensure that the standardised barrier framework is restricted to a reasonable range of landslide elevations which are compatible with the relatively moderate debris volumes considered for open hillslope failures.

D.4 REFERENCES

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Table D1 - Minimum Distance from Node Point No. 1 to Type 4 Barrier

50 m³ Event with Friction Angle of 30° (1,000 kJ Tensioned Steel Mesh Fence)						
Lower Tangent Angle (°)	6	10	14	18	22	26
Distance (m)	6	7	8	11	16	31

50 m³ Event with Friction Angle of 30° (2,000 kJ Tensioned Steel Mesh Fence)						
Lower Tangent Angle (°)	6	10	14	18	22	26
Distance (m)	3	4	5	7	10	19

100 m³ Event with Friction Angle of 25° (2,000 kJ Tensioned Steel Mesh Fence)					
Lower Tangent Angle (°)	6	10	14	18	22
Distance (m)	11	14	19	29	67

Note: Barriers should be provided within the zone where the potential runout distance of open hillslope failures between the crown of the landslides and the affected facility is less than 75 m and 120 m respectively as measured from the lower edge of the area of potential open hillslope failure for debris volumes of 50 m³ and 100 m³ respectively.

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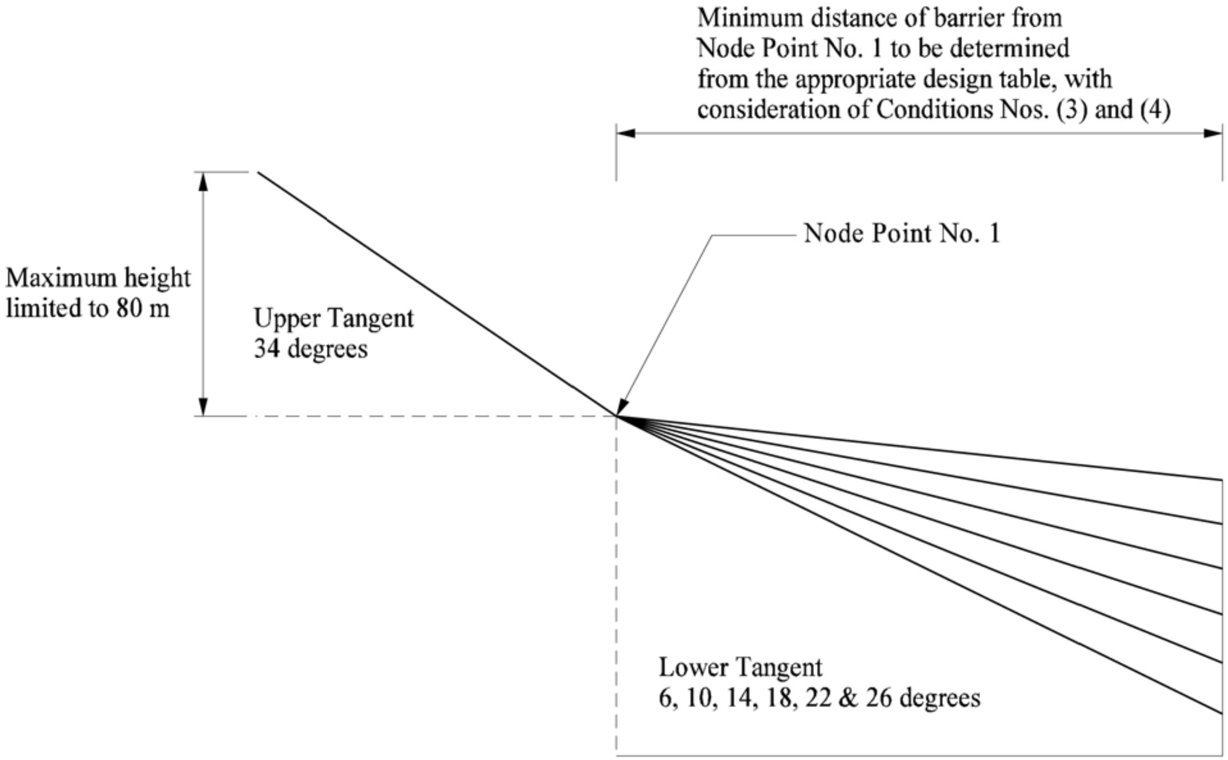

Conditions for Template Fitting:

(1) No parts of the Upper or Lower Tangent lines shall be below the ground profile (Upper Tangent length to be extended as far as the crown of the potential source).

(2) Node Point No. 1 must co-incide with ground level.

(3) Segments of the Lower Tangent over which a barrier can be positioned must lie below the ground profile and must be inclined steeper than or equal to the gradient of the local ground profile.

(4) Steps in the ground profile in the Lower Tangent area are allowable provided that Condition No. (3) is complied with and that the distance of the barrier from the toe of the step shall not be less than the plan length or vertical height of the step (whichever is greater).

Figure D1 - Template for Open Hillslope Failures

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APPENDIX E

CALIBRATION OF DESIGN PARAMETERS WITH EXISTING DATA

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E.1 INTRODUCTION

A flowchart showing the methodology for the determination of the design channel/slope characteristics and calibration of the design parameters with existing data is shown in Figure E1.

In order for the standardised barrier framework to be applicable to a wide range of cases while still remaining practicable to develop, a single standard design channel gradient for channelised debris flows and a single gradient for open hillslope failures have been adopted for the upper tangent of the design profiles. In addition, the cross-sectional shapes for channelised debris flows and open hillslope failures need to be standardised in order to limit the geometrical permutations to a reasonable number for development of the framework.

The maximum velocity and height of the debris within the channel is taken to be dependent upon the design event volume, channel gradient and rheological parameters of the debris. The rheological properties and maximum velocity vs. debris volume for the upper tangent design channel were based upon a review of previous back analysed incidents in Hong Kong. The relevant data used for calibration of the design channel are shown in Tables E2 and E3.

Mobility analyses using MGSL's Debriflo program were then carried out to determine the design height of the debris for each set of rheological parameters which is compatible with the maximum velocity in the upper tangent design channel.

The results were then compared with the existing data on debris flow height and discharge in Hong Kong and found to produce a good, upper-bound fit with the data.

A comparison of the results with the empirical guidelines for maximum velocity and debris height vs. design event volume given in GEO Report No. 104 shows that the calibrated results are broadly compatible, with the design velocity being an upper-bound to the velocity range indicated in empirical guidelines.

E.2 UPPER TANGENT DESIGN CHANNEL/SLOPE GRADIENT

From a review of past incidents and typical drainage line/slope profiles in Hong Kong and consideration of the limits of applicability of a standardised design framework, an upper tangent gradient of 1.5 (H) : 1.0 (V) (34°) is considered to be appropriate for design.

Local slope angles steeper than 34° are not uncommon along drainage lines in Hong Kong, but the overall gradient along most channel and slope profiles is usually less than 34° (refer to examples in Appendix B). In order to base the framework on the results of previous back analyses carried out in Hong Kong, the calibration channel/slope gradient needs to be similar to the actual profiles in the back analysed data set at which the maximum velocity was calculated. The choice of a 34° calibration channel achieves this objective while still giving a reasonably steep upper tangent that can be applied to a wide range of hillside and drainage line longitudinal profiles in Hong Kong.

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E.3 CHANNELISED DEBRIS FLOWS

E.3.1 Upper Tangent Design Channel Cross-section

The channel cross-section chosen for design consists of a 1.75 m wide horizontal base with a side slope gradient of 1.75 (H) : 1.0 (V), i.e. 30°. This cross section is representative of the average channel dimensions in terms of channelisation ratio (surface width of the debris divided by the maximum depth of the debris, (Ng et al, 2002) of previous debris flows in Hong Kong (Ayotte & Hungr, 1998). The design channel gives channelisation ratios that vary from about 4.0 to 5.0, depending on the height of debris.

Smaller channelisation ratios will create additional frictional drag and turbulence relative to the cross-sectional area (larger perimeter to area ratio), thereby reducing the velocity and increasing the debris height for a fixed discharge rate. Larger channelisation ratios will reduce the frictional drag and turbulence relative to the cross-sectional area, thereby increasing the velocity and reducing the debris height for a fixed discharge rate.

Given that debris runout and impact force are functions of both velocity and debris height (refer to the leading-edge equation in Lo (2000) and the impact velocity equation given in Section 5.3 of this Technical Note), moderate variations in channelisation ratios are unlikely to significantly influence barrier design.

E.3.2 Data for Back Analysed Debris Flows

The available velocity, flow height and discharge rate data vs. debris volume for those Hong Kong debris flows back analysed by Hungr (1998), Ayotte & Hungr (1998) and MGSL (2000 & 2002) are shown in Figure E2.

Definitions of the terms used in Figure E2 and the key considerations in the selection of the data are given below:

(a) The maximum debris volume is the maximum volume of debris (including entrainment) that passed the point where the maximum velocity has been recorded.

(b) In assessing the maximum back-analysed velocity (V_{max}), care has been taken to ignore sections of channel or slope that appear to be influenced by the initial conditions or steep, local gradients such as 'waterfalls'.

(c) The maximum debris height has been measured at the same point as V_{max}. A meaningful correlation between debris height and velocity can only be achieved by measuring the maximum debris height at the same location as the maximum debris velocity.

(d) The discharge rate at the location where V_{max}, was measured is the sectional area normal to the slope multiplied by V_{max}.

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Figure E2 indicates that there is a general trend of increasing debris velocity, height and discharge rate with increasing debris volume.

E.3.3 Determination of Rheological Parameters For Design

E.3.3.1 Rheological Model

The Voellmy rheological model has previously been used by Hungr (1998), Ayotte & Hungr (1998) and MGSL (2000 & 2002) for the back analysis of debris flows in Hong Kong. This model is also recommended in GEO Report No. 104 for the analytical modelling of debris flows.

E.3.3.2 Turbulence Factor (ξ)

From the back analysis results, a ξ-value of 500 m/s^2 has been shown to be typical for Hong Kong conditions and is recommended in GEO Report No. 104. This value has been adopted for the calibration of the 'design' channel and is considered to be suitably conservative when applied to the straight and relatively unconfined 'design' channels under the standardised barrier framework.

E.3.3.3 ϕ Parameter

The results of the back analyses indicate that the range of ϕ-values lies between 5.7° and 11.3°. A ϕ-value of 11.3° is considered to be appropriate for most debris flow events in Hong Kong where the potential for mixing with a proportionately large amount of water and channelisation in ravine-type streamcourses is limited. A ϕ-value of 5.7° is considered appropriate for the potentially highly mobile flows in confined streamcourses with a large amount of surface water (e.g. 1997 Sha Tau Kok and 1999 Sham Tseng San Tsuen debris flows).

In order to cater for both scenarios, calibrations and detailed calculations have been carried out for both ϕ-values. The higher shear resistance flows will have a greater debris depth for the same velocity than the lower resistance flows, leading to larger boulders being carried in the debris. The lower resistance flows will be smaller in depth, but will have a longer run-out.

E.3.4 Field Measurements of Maximum Debris Depths

The data extracted from Wong et al (1997) are shown in the maximum debris height vs. maximum volume plot in Figure E2. Although no velocities are reported, the data can be used as a reference against which the calibrated debris height vs. maximum debris volume can be compared.

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E.3.5 Upper-bound Maximum Debris Velocity for the Design Channel

A plot of the maximum velocity vs. maximum debris volume for back-analysed channelised debris flows is shown in the upper part of Figure E2. The design line for velocity vs. maximum debris volume to be adopted for the calibration calculations for the standardised barriers is also shown in the same figure. The maximum design velocity varies from 9.8 m/s for a volume of 100 m^3 to 19.2 m/s for a volume of 8,000 m^3. The design line adopted represents an upper-bound to the back-analysis data and the velocity versus debris volume relationship suggested in GEO Report No. 104 (also indicated in Figure E2). The maximum velocity vs. debris volume relationship adopted for the standardised barrier framework is therefore considered to be suitably conservative.

E.3.6 Calibration Results

The Debriflo spreadsheet program was used to calibrate the maximum vertical height in the 34° design channel with the upper-bound velocity design line shown in Figure E2 for a range of debris volumes. The channel cross-section and the two sets of rheological parameters used in the analyses are the same as those described in this Appendix.

For the calibration, the vertical debris height was adjusted until the calculated velocity matched that of the velocity design line shown in Figure E2 and remained constant along the length of the design channel. The vertical debris height needed to provide sufficient thrust to propel the debris at the design velocity in the design channel for a given volume is taken to be the debris height adopted for design. The results of the calibrations for both sets of parameters are summarised in Table E1, while the calibrated height lines vs. debris volume are shown in the middle plot in Figure E2.

E.3.7 Comparison of Calibrated Values with the Existing Data

The results of the calibration are considered to provide a reasonable match with the existing data-set (with the exception of the 1990 Tsing Shan results for 8,000 m^3 in Figure E2), primarily on the basis that the discharge rates from the existing back analyses plot below the 11.3° calibration line shown on the lower plot in Figure E2. This indicates that within the range of the design events covered by the standardised barrier framework (i.e. 150 m^3 - 600 m^3 for channelised debris flows), the combination of design velocity and calibrated debris height vs. debris volume relationships chosen for design will produce a discharge rate in the upper tangent design channel that is an upper bound to the discharge rates at maximum velocity derived from the existing back analysis results.

The back-analysed debris height for the 680 m^3 2001 Lei Pui Street debris flow plots above the calibrated height versus debris volume relationship shown in the middle plot of Figure E2, primarily because the debris velocity from the back analysis results is much lower than the upper bound velocity design line shown in the upper plot in Figure E2. A lower velocity in natural channels for a given discharge rate results in an increase in debris height to maintain the discharge rate compatible with the upstream discharge. As the discharge rate for this result lies between the 11.3° and 5.7° discharge rate calibration lines shown in the lower plot of Figure E2, the result is not considered to be anomalous because the discharge rate is less than the upper bound discharge rate for this volume established from the

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calibration exercise. It is also considered probable that the three debris height points from the existing field data (Wong et al, 1997) that plot above the debris height calibration lines in the middle plot of Figure E2 are also likely to be due to the actual debris velocity at the points of measurement being considerably lower than the velocity assumed for design.

The 1990 Tsing Shan debris flow (with an active volume of 8,000 m^3 at chainage 350) involved a significant amount of entrainment that took place over a relatively long time, resulting in the actual depth of the main debris pulse being uncertain. The final base-width of the channel is also much wider than the standardised barrier design channel which contributes to a higher discharge rate than shown by the calibrated results. As the Tsing Shan debris flow involved a series of pulses with deepening of the channel by gradual entrainment, the back analysis of this event as a single debris pulse is questionable. It is also noted that the standardised barrier framework will not consider design event volumes greater than 600 m^3.

E.3.8 Comparison with GEO Report No. 104

The maximum empirical velocity and maximum debris height guidelines from GEO Report No. 104 are shown on the plots in Figure E2. The calibrated results are broadly compatible, with the design velocity being an upper-bound to the empirical guidelines for design event volumes greater than about 130 m^3.

E.3.9 Conclusions

The calibrated design channel gives suitably conservative design values for debris velocity and debris height over the range of design event volumes considered within the standardised barrier design framework. The adoption of these values for the modelling of debris in the straight and regularly shaped design channel is likely to err very much on the conservative side where irregularly shaped, natural channels are considered.

E.4 OPEN HILLSLOPE FAILURES

E.4.1 Data for Back-analysed Open Hillslope Failures

The available debris velocity vs. debris volume data on Hong Kong open hillslope failures back analysed by Hungr (1998) and Ayotte & Hungr (1998) are shown in Table E4 and Figure E3.

Definitions of the terms used in Figure E3 and the key considerations in the selection of the data are given below:

(a) The maximum debris volume is the maximum volume of debris mobilised above the point where the maximum velocity has been recorded.

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(b) In assessing the maximum back-analysed velocity (V_{max}), care has been taken to ignore sections of channel or slope that appear to be influenced by the initial conditions or steep, local gradients such as 'waterfalls'.

E.4.2 Upper-bound Maximum Debris Velocity for the Design Slope

From Figure E3, it can be seen that a reasonable upper-bound maximum debris velocity vs. maximum debris volume relationship for back-analysed open hillslope failures is obtained by adopting the same design line as for channelised debris flows (Figure E2).

The range of volumes of open hillslope failures back analysed by Hungr (1998) and Ayotte & Hungr (1998) varies from about 100 m^3 to 40,000 m^3. This means that the maximum velocity correlation for events less than 100 m^3 has to be extrapolated using the logarithmic relationship shown in Figure E3. This approach is considered reasonable since the established relationship provides a good upper-bound fit with the available data.

From Figure E3, the maximum design velocities for design event volumes of 50 m^3 and 100 m^3 are 8.3 m/s and 9.8 m/s respectively.

E.4.3 Comparison with GEO Report No. 104

The empirical debris velocity and debris height guidelines given in GEO Report No. 104 are shown in Figure E3. The design line adopted here represents an upper-bound to that recommended in GEO Report No. 104 for design event volumes greater than about 130 m^3. The extrapolation of the design line for design event volumes of 50 m^3 and 100 m^3 results in the design velocities adopted in the standardised barrier framework being between 7% and 21% lower than the constant velocity of 10.5 m/s suggested for design events of less than 400 m^3 in GEO Report No. 104 which is considered too conservative for volumes less than 100 m^3.

E.4.4 Conclusions

The design velocities adopted in the standardised barrier framework for design events of 50 m^3 and 100 m^3 are considered to be suitably conservative in that they fit the upper-bound velocity trend indicated by the back-analysed results.

The fact that the design velocities adopted are lower than the maximum velocity recommended in GEO Report No. 104 for relatively large volumes up to 400 m^3 is not considered to be significant because the difference in velocity is not great and that the design events considered within the standardised barrier framework for open hillslope failures (up to 100 m^3) are relatively small.

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Table E1 - Summary of Calibration Results for Channelised Debris Flows in 34° Design Channel

$\phi = 11.3^{o}$, $\xi = 500$ m/s²						
Design Volume (m^3)	Design Velocity from Fig. E2 (m/s)	Calibrated Vertical Debris Height (m)	Calibrated Area Normal to Slope (m^2)	Calibrated Discharge Rate (m^3/s)	Calibrated Width (m)	Channelisation Ratio
100	9.77	1.04	3.08	30	5.39	5.2
150	10.64	1.3	4.34	46	6.30	4.8
300	12.13	1.8	7.31	88	8.05	4.5
400	12.75	2.07	9.22	118	9.00	4.3
600	13.62	2.4	11.84	161	10.15	4.2
1000	14.72	2.9	16.41	241	11.90	4.1
1200	15.12	3.1	18.44	278	12.60	4.1
1400	15.45	3.26	20.15	310	13.16	4.0
2000	16.21	3.6	24.03	389	14.35	4.0
8000	19.20	5.29	48.27	927	20.27	3.8
$\phi = 5.7^{o}$, $\xi = 500$ m/s²						
Design Volume (m^3)	Design Velocity from Fig. E2 (m/s)	Calibrated Vertical Debris Height (m)	Calibrated Area Normal to Slope (m^2)	Calibrated Discharge Rate (m^3/s)	Calibrated Width (m)	Channelisation Ratio
100	9.77	0.85	2.28	22	4.73	5.6
150	10.64	1.02	2.99	32	5.32	5.2
300	12.13	1.42	4.99	60	6.72	4.7
400	12.75	1.62	6.10	78	7.42	4.6
600	13.62	1.95	8.35	114	8.58	4.4
1000	14.72	2.4	11.84	174	10.15	4.2
1200	15.12	2.6	13.58	205	10.85	4.2
1400	15.45	2.74	14.67	226	11.34	4.1
2000	16.21	3.1	18.44	299	12.60	4.1
8000	19.20	4.5	35.91	689	17.50	3.9

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Table E2 - Available Velocity and Flow Height Data for Back-analysed Debris Flows

Maximum Volume m³	Maximum Velocity (Vmax) m/s	Max. Debris Height @ Vmax. m	Angle of Reach @ Vmax deg.	Local Upslope Angle deg.	Channelisation Ratio -	Sectional Area Normal to Slope @ Vmax m²	Discharge (Q) @ Vmax m³/s	ϕ deg.	Turbulence Factor ξ m/s²	Location	Source #
140	6.5	0.45	29.7	31	7.8	1.05	7	11.3	500	Pat Sin Leng (No. 2)	2
150	8.7	0.89	24.7	29.2	7.3	3.90	34	11.4	500	Pak Sha Wan	2
300	8	0.6	27	27	15.0	3.60	29	11.3	500	Liu Pok	2
350	9.5	0.9	33.7	30	12.2	6.60	63	21.8	500	Lantau (JK-529)	2
500	10.7	2.26	28.5	30	2.7	4.93	53	5.65	500	Sham Tseng San Tsuen	3
500	12	1.85	28	26	2.7	6.20	74	5.65	200	Sham Tseng San Tsuen	5
680	9.7	3.79	35	16	2.1	16.00	155	11.3	500	Lei Pui St. (ch124)	3
1400	10	1.5	26.6	18.6	6.7	10.00	100	5.71	500	Sha Tau Kok	2
1400	14.5	1.8	23.4	28.4	5.1	7.63	111	5.65	500	Sha Tau Kok	3
8400	18.3	6	32	20.3	4.5	108.00	1980	11.3	500	Tsing Shan (ch350)	1
8400	16.7	7.25	32	20.3	3.7	87.75	1470	11.3	500	Tsing Shan (ch350)	3
8400	16.55	6.8	32	20.3	3.7	89.10	1470	-	-	Tsing Shan (ch350)	4

Note: # Source:
1. Hungr (1998) - DAN analyses
2. Ayotte & Hungr (1998) - DAN analyses
3. MGSL Debriflo analyses
4. King (1996) - Estimated from field measurements
5. DAN analysis by Hungr

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Table E3 - Field Data for Channelised Debris Flows from Wong et al (1997)

Landslide No.	Debris Volume (m^3)	Maximum Debris Depth (m)	Angle of Reach to Measured Section (degrees)	Local Upslope Angle (degrees)	Channelisation Ratio
A13A	100	0.5	37	37	4.77
B11M	125	0.5	33	28	10.14
A18B/C	150	1	31	25	3.17
A2	195	1	33	30	4.06
A10A-D	265	1	27	24	3.52
A18A	290	1	35	27	3.45
A1B	295	2.2	33	22	3.05
B5A	340	2	38	38	1.88
B9	360	0.8	35	40	9.33
A5A	460	0.8	29	22	5.46
A17	465	2.5	37	34	0.49
B1	690	1	31	27	6.29
B4E	905	2	41	41	2.05
B7M	1260	2	24	-	3.40
B2M	1420	2	28	33	2.04

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Table E4 - Available Velocity and Flow Height Data for Back-analysed Open Hillslope Failures

Maximum Volume m³	Maximum Velocity (Vmax) m/s	Max. Debris Height @ Vmax. m	Angle of Reach @ Vmax deg.	Local Upslope Angle deg.	Channelisation Ratio -	Sectional Area Normal to Slope @ Vmax m²	Discharge (Q) @ Vmax m³/s	ϕ deg.	Turbulence Factor ξ m/s²	Location	Source #
112	8	1	45.9	47	10.0	6.8	54	41	-	Tung Chung (6A1)	2
164	10	1.5	28	28	3.3	5.0	50	24	-	Luk Keng	2
266	8	0.9	38.6	38	28.9	18.4	147	34	-	Tung Chung (5A10)	2
287	9.9	0.6	28.1	33	30.8	9.3	92	28	-	Lantau (JK-515)	2
337	7.5	0.8	26	24	17.5	10.2	77	25	-	Tai Mong Tsai	2
384	12	1.5	33.7	38.6	9.3	16.4	197	31	-	Lantau (JK-410)	2
400	12.9	1.6	35	43.8	10.4	19.3	249	28	-	Pat Sin Leng (No. 1)	2
411	10	0.8	28.4	28	9.5	5.4	54	29	-	Lantau (A6)	2
687	9	0.9	26.6	26.6	7.8	5.6	51	25	-	Tung Chung (5A13)	2
2068	16.5	1.1	26	23	12.7	14.2	234	23	-	Lantau (C1)	1
2500	12	2.8	34.6	38.7	9.4	57.5	690	20	-	Sau Mau Ping	1
23061	14.8	3	26	26	24.0	194.1	2870	11.3	200	Shum Wan	1
40068	12.6	1.5	25	43	40.0	65.8	830	23	-	Po Shan Road	1

Note: # Source: 1. Hungr (1998) - DAN analyses
2. Ayotte & Hungr (1998) - DAN analyses

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Table E5 - Field Data from Wong et al (1997) and the Tsing Shan Foothills Natural Terrain Landslide Study (Year 2000 Landslides) for Open Hillslope Failures

Landslide No.	Debris Volume (m^3)	Maximum Debris Depth (m)	Angle of Reach to Measured Section (degrees)	Local Upslope Angle (degrees)	Source
B3B	30	0.5	40	-	Wong et al (1997)
B3A	30	0.5	45	-	
A13B	45	0.5	31	-	
A3	50	0.3	30	-	
A11	50	0.7	45	-	
B13A	50	0.6	30	-	
A1A	60	0.3	32	-	
A9	60	0.8	42	-	
A2	65	1	32	-	
B7A	70	0.3	32	-	
A4	95	0.3	29	-	
A8	100	0.25	39	-	
A16A	105	0.6	37	-	
A12	135	0.3	39	-	
A5B	140	0.6	35	-	
B12	140	2	31	-	
A14	170	1.5	32	-	
A7	190	0.2	40	-	
A18A	210	1	32	-	
B10	210	0.6	35	-	
A1B	230	1.5	32	-	
A5A	240	0.3	42	-	
A15	320	1.5	33	-	
A6	400	0.5	31	-	
11	5	0.15	38	42	Year 2000 Landslides of Tsing Shan Foot Hill Study (MGSL, 2003)
70	5	0.1	32	32	
117	6	0.25	28	25	
115	8	0.3	48	40	
69	12	0.3	32	27	
6	13	0.3	39	37	
7	14	0.4	35	30	
29	14	0.4	33	30	
8	18	0.4	37	10	
10	18	0.15	34	33	
4	25	0.3	33	30	
98	39	0.2	41	43	
45	40	0.5	44	47	
93	40	0.5	29	20	
19	42	0.3	36	39	
12	59	0.3	31	25	
107N	62	0.3	35	34	
107S	62	0.6	38	39	
14	63	0.15	35	30	
24	63	0.5	26	15	
23	65	0.3	32	30	
53	68	0.4	30	30	
55	70	0.5	33	34	
18	71	0.5	39	25	
63	81	0.4	31	20	
52	90	0.4	33	28	
100	95	0.8	32	27	
71	99	0.4	27	32	
15	125	0.2	31	28	

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E2	Velocity, Height and Discharge Rate Vs. Maximum Debris Volume for Channelised Debris Flows	121
E3	Maximum Velocity Vs. Maximum Debris Volume for Open Hillslope Failures	122

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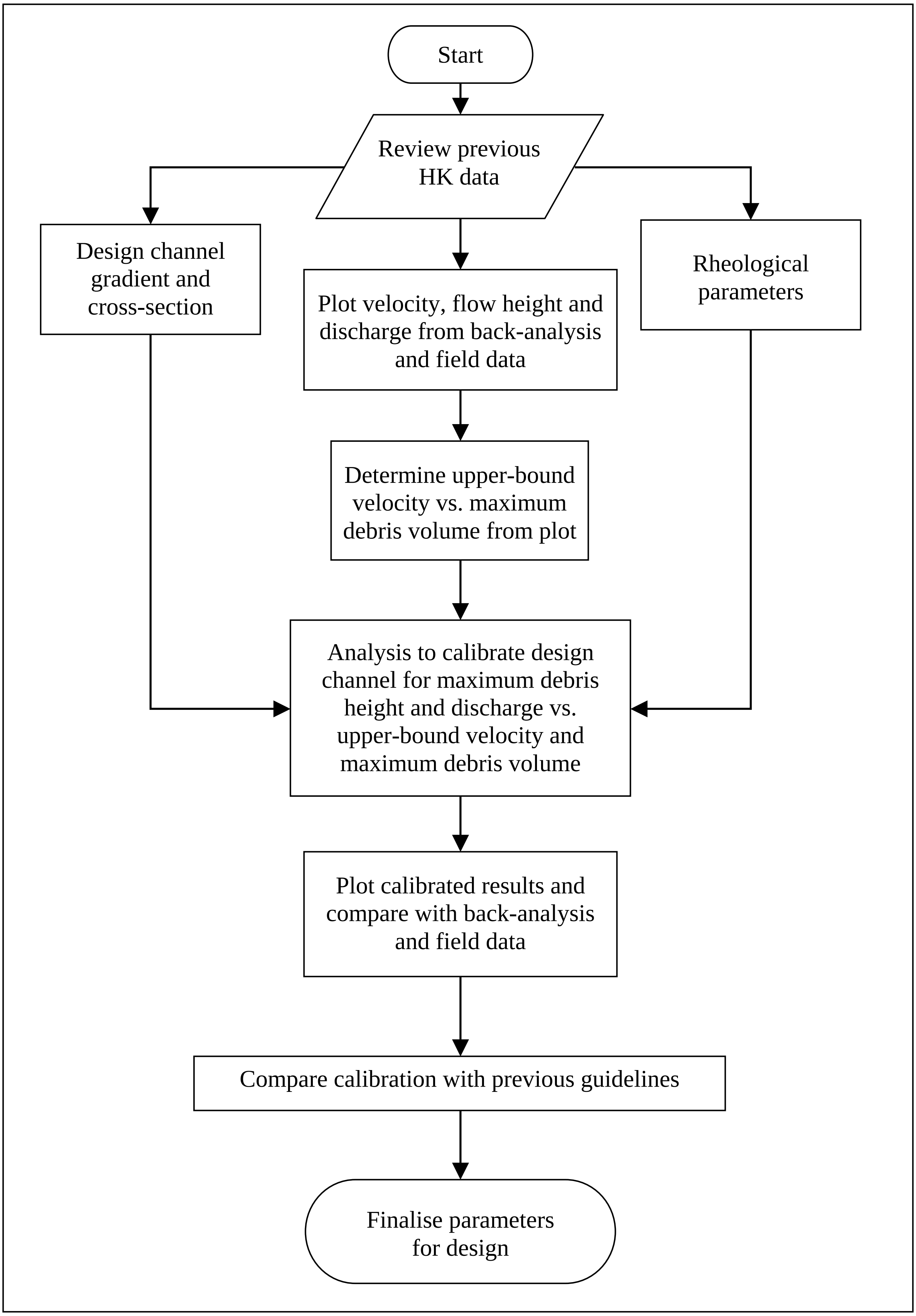

Figure E1 - Methodology for Calibration of Design Channel/Slope

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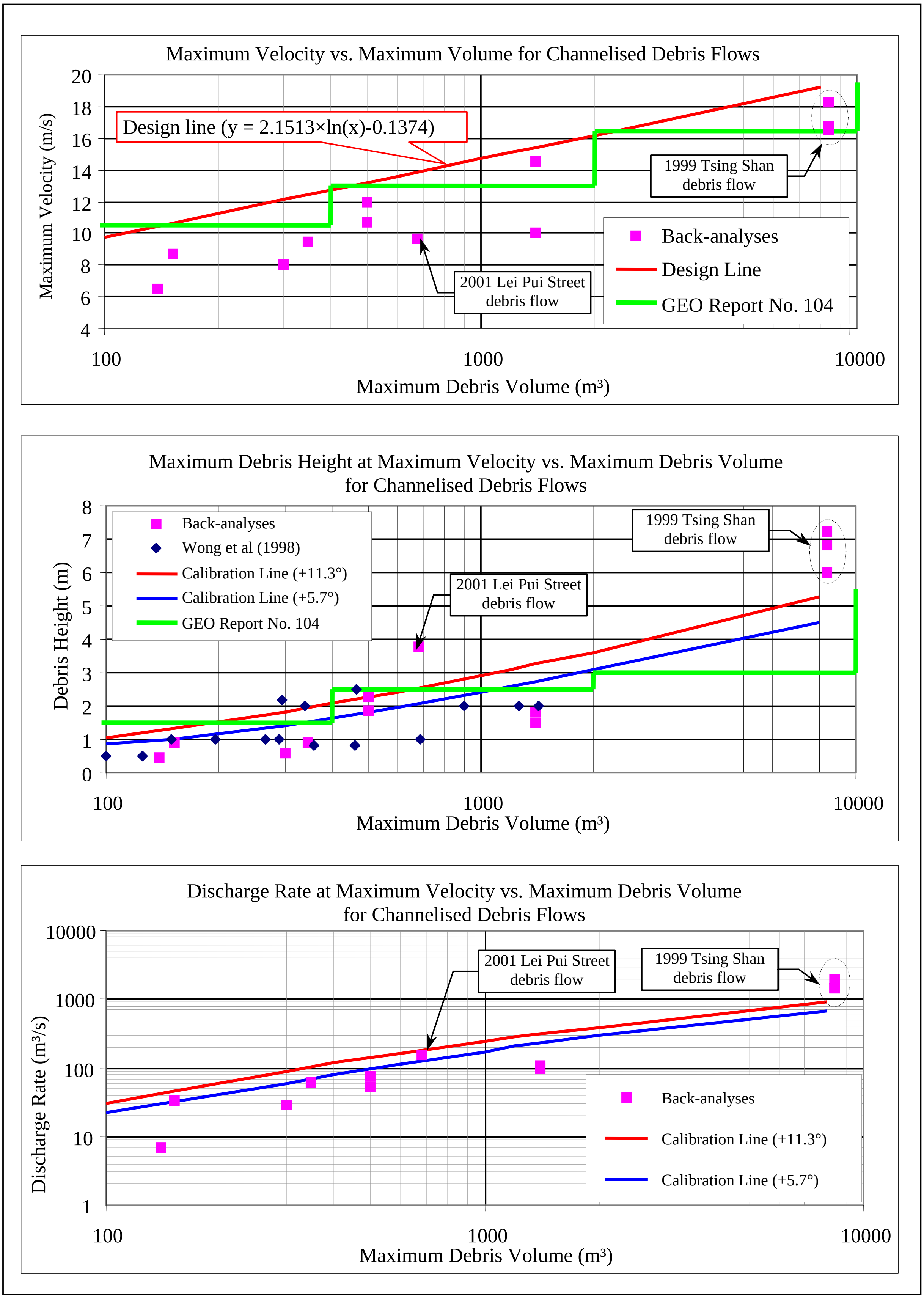

Figure E2 - Velocity, Height and Discharge Rate Vs. Maximum Debris Volume for Channelised Debris Flows

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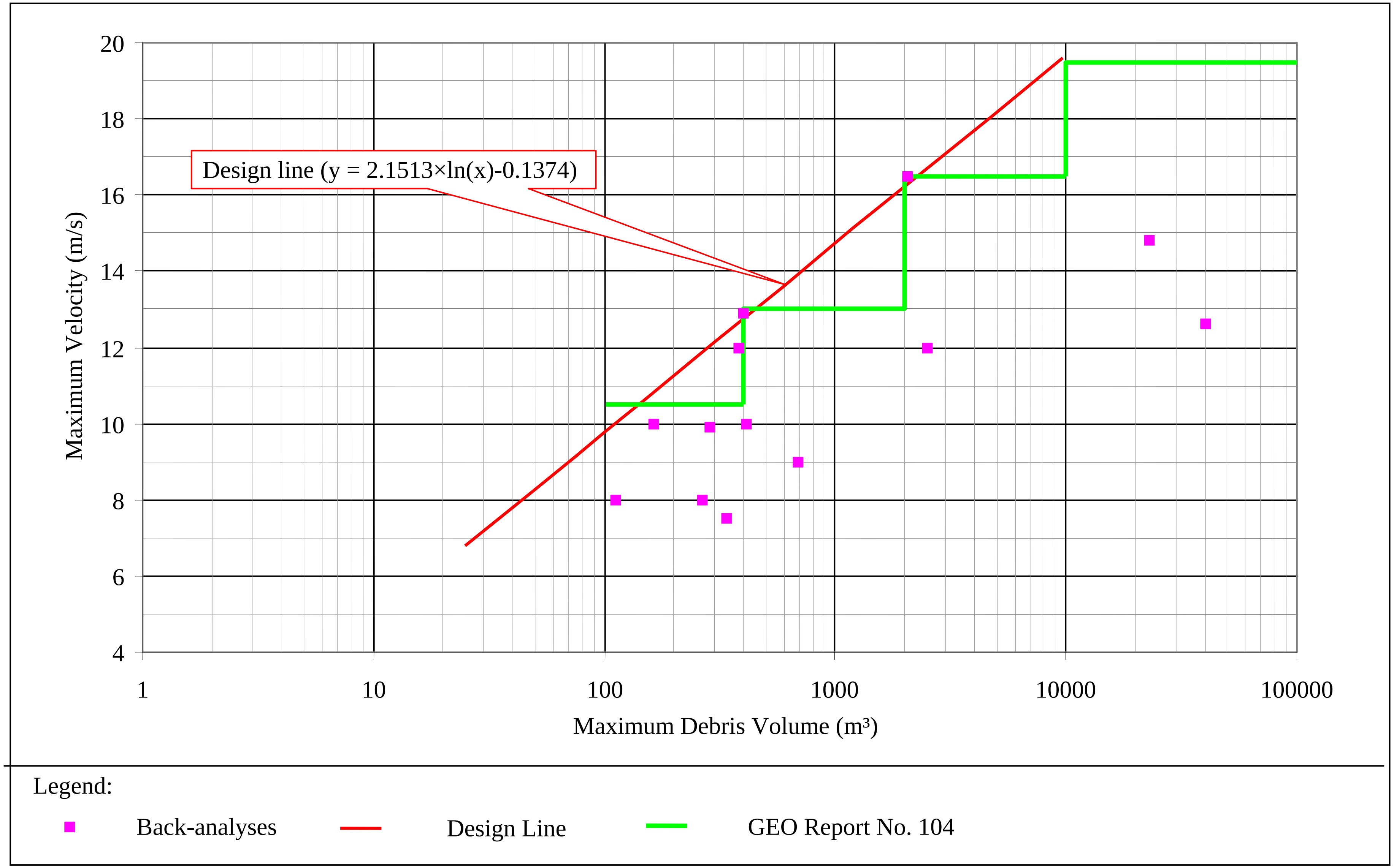

Figure E3 - Maximum Velocity Vs. Maximum Debris Volume For Open Hillslope Failures

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APPENDIX F

SAMPLE CALCULATIONS FOR STANDARDISED BARRIER DESIGN

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DESIGN CALCULATIONS FOR

4.5 m HIGH TYPE 1A BARRIER

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Checking Stability of Structure

Type 1A - 4.5 m with sloping ground in front - 1st pulse 50% weight

Type 1A - 4.5 m with sloping ground in front - 2nd pulse 50% weight + lower static soil

The design events are:

1 - 600 m³ debris volume with boulder from 600 m³ boulder

Boulder diameter =	2.528	m	
Impact Depth, H_{deb} =	2.974	m	
Wall Heigh, H_w =	4.5	m	
Static Soil Depth, H_{sta} =	1.526	m	
Debris Impact Velocity, V_1 =	5.281	ms^{-1}	
Debris Impact Force, F_{debris} =	4432	kN	(total)
Boulder Impact Force, F_{bould} =	5025	kN	for RC (total)
Length of Debris, L_{deb} =	10.641	m	

2 - 1200 m³ debris volume with boulder from 600 m³ boulder

Boulder diameter =	2.528	m	
Impact Depth, H_{deb} =	3.496	m	
Wall Heigh, H_w =	4.5	m	
Static Soil Depth, H_{sta} =	1.004	m	
Debris Impact Velocity, V_1 =	6.951	ms^{-1}	
Debris Impact Force, F_{debris} =	10141	kN	(total)
Boulder Impact Force, F_{bould} =	6986	kN	for RC (total)
Length of Debris, L_{deb} =	12.77	m	

For event 1, we have to satisfy the following checks:

* FOS > 1 Sliding
* FOS > 1 Overturning
* FOS > 1 Bearing

For event 2, we have to satisfy the following global geotechnical checks:

* FOS > 1 Sliding or
  If Sliding Resistance < Sliding Force, then check movement during the dynamic impact phase to ensure sliding movement < 1.5 m
* FOS > 1 Overturning
* FOS > 1 Bearing

Elastic Modulus of Concret Barrier E_{bc} = 25 GPa

Elastic Modulus of Gabion Barrier E_{bg} = 0.3 GPa

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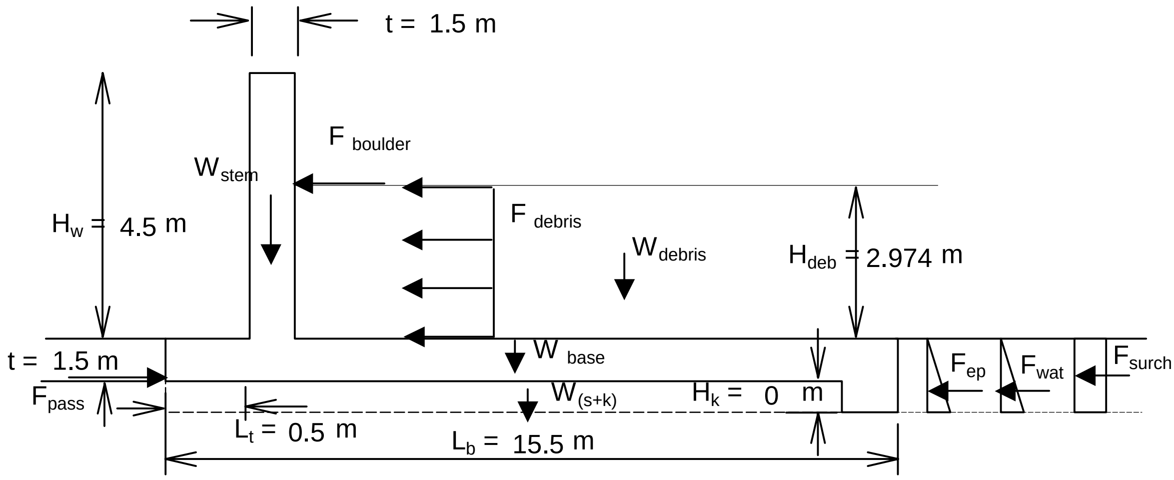
Stability Check for 600 m³ Event (with boulder in 600 m³ event)

Type 1A - 4.5 m with sloping ground in front - 1st pulse 50% weight



Assumptions:

γ_{soil} =	19.7 kN/m³	γ_{con} =	24 kN/m³
γ_{wat} =	9.81 kN/m³	γ_{gab} =	18 kN/m³
Base Friction Angle ø =	35 °	$K_a = K_o$ =	1
		K_p =	0.00
Debris Length, L_{deb} =	10.641 m		
Debris Height, H_{deb} =	2.974 m		
Static Debris Height, H_{sta} =	0 m		
Wall Length, L_w =	16 m	Concrete thickness, t =	1.5 m
Wall Height, H_w =	4.5 m	Length of Toe, L_t =	0.5 m
Wall Base Length , L_b =	15.5 m	Height of Key, H_k =	0 m

Mark	Force (kN)	Arm (m)	Moment at toe (kNm)
F_{debris} =	4432.0	2.987	13238.4
F_{bould} =	5025.0	4.474	22481.9
F_{ep} =	178.0	0.500	89.0
F_{wat} =	176.6	0.500	88.3
F_{surch} =	935.1	0.750	701.4
W_{stem} =	2592.0	-1.250	-3240.0
W_{base} =	8928.0	-7.750	-69192.0
W_{stor} =	0.0	-8.750	0.0
W_{debris} =	8416.3	-8.750	-73643.0
$W_{(s+k)}$ =	0.0	-	-
F_{pass} =	0.0	-	-

(-ve denote stabilising moment)

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Stability Check for 600 m³ Event (with boulder in 600 m³ event)

Type 1A - 4.5 m with sloping ground in front - 1st pulse 50% weight

Checking Sliding Resistance

The disturbing force $F_{slid} = F_{debris} + F_{bould} + F_{ep} + F_{wat} + F_{surch}$

$\mathbf{F_{slid}}$ = **10746.7 kN**

The resisting force $F_{res} = (W_{stem}+W_{base}+W_{stor}+W_{debris}/2+W_{(s + key)}) \times \tan\phi + F_{pass}$

$\mathbf{F_{res}}$ = **11013.0 kN**

and $\mathbf{FOS_{slid}}$ = **1.025 > 1** **OK in Sliding**

Checking Overturning Resistance

The disturbing moment M_o = Moment due to $(F_{debris} + F_{bould} + F_{ep} + F_{wat} + F_{surch})$

$\mathbf{M_o}$ = **36598.9 kN**

The restoring moment Mr = Moment due to $(W_{stem}+W_{base}+W_{stor}+W_{debris})$

$\mathbf{M_{res}}$ = **146075.0 kN**

and $\mathbf{FOS_{over}}$ = **3.991 > 1** **OK in Overturning**

Checking Bearing Pressures

The eccentricity, e of the resultant vertical force is:

$$e = (L_b / 2) - (M_{res} - M_o) / F_{vertical}$$

with $F_{vertical} = W_{stem} + W_{base} + W_{stor} + W_{debris}$

M_{res} =	146075.0	kNm	or	9129.7	kNm/m
M_o =	36598.9	kNm	or	2287.4	kNm/m
F_{vert} =	19936.3	kN	or	1246.0	kN/m

e = 2.259 m

By Figure A1 in Geoguide 1, the effective length of the base, L_b' is :

$L_b' = L_b - 2e =$ 10.983 m

The maximum bearing stress is found as:

$\mathbf{q_{max}} = F_{vert} / (L_b' \times L_w)$

= 113.5 kPa < 300 kPa **OK in Bearing**

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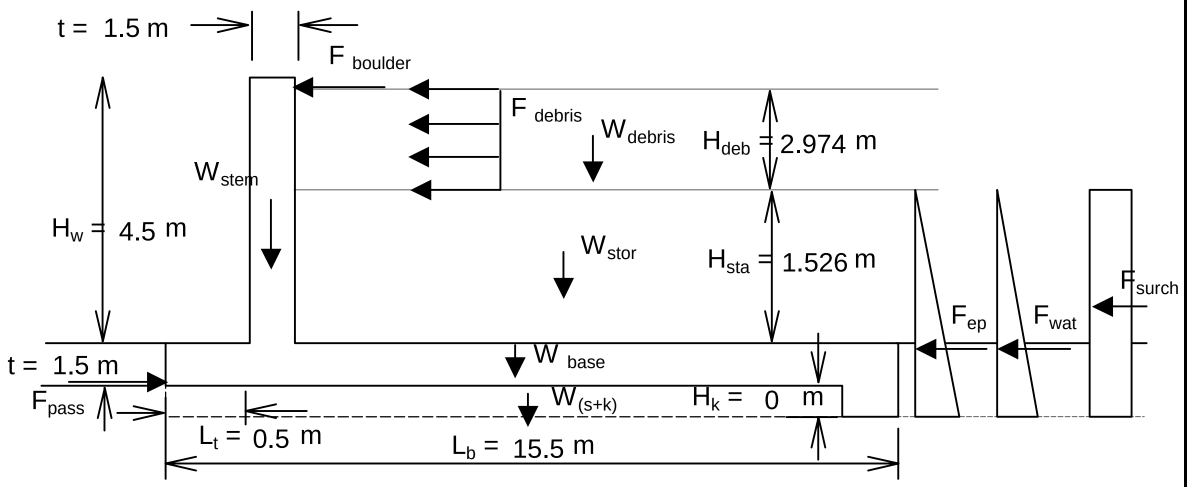
Stability Check for 600 m³ Event (with boulder in 600 m³ event)

Type 1A - 4.5 m with sloping ground in front - 2nd pulse 50% weight + lower static soil



Assumptions:

γ_{soil} =	19.7 kN/m³	γ_{con} =	24 kN/m³
γ_{wat} =	9.81 kN/m³	γ_{gab} =	18 kN/m³
Base Friction Angle ø =	35 °	$K_a = K_o$ =	1
		K_p =	0.00
Debris Length, L_{deb} =	10.641 m		
Debris Height, H_{deb} =	2.974 m		
Static Debris Height, H_{sta} =	1.526 m		
Wall Length, L_w =	16 m	Concrete thickness, t =	1.5 m
Wall Height, H_w =	4.5 m	Length of Toe, L_t =	0.5 m
Wall Base Length , L_b =	15.5 m	Height of Key, H_k =	0 m

Mark	Force (kN)	Arm (m)	Moment (kNm)
F_{debris} =	4432.0	4.513	20001.6
F_{bould} =	5025.0	6.000	30150.0
F_{ep} =	724.5	1.009	730.8
F_{wat} =	718.6	1.009	724.8
F_{surch} =	1886.5	1.513	2854.3
W_{stem} =	2592.0	-1.250	-3240.0
W_{base} =	8928.0	-7.750	-69192.0
W_{stor} =	6493.4	-8.750	-56817.6
W_{debris} =	8416.3	-8.750	-73643.0
$W_{(s+k)}$ =	0.0	-	-
F_{pass} =	0.0	-	-

(-ve denote stabilising moment)

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Stability Check for 600 m³ Event (with boulder in 600 m³ event)

Type 1A - 4.5 m with sloping ground in front - 2nd pulse 50% weight + lower static soil

Checking Sliding Resistance

The disturbing force $F_{slid} = F_{debris} + F_{bould} + F_{ep} + F_{wat} + F_{surch}$

F_{slid} = 12786.6 kN

The resisting force $F_{res} = (W_{stem}+W_{base}+W_{stor}+W_{debris}/2+W_{(s + key)}) \times \text{Tan}\,\phi + F_{pass}$

F_{res} = 15559.7 kN

and **FOS_{slid} = 1.217 > 1** **OK in Sliding**

Checking Overturning Resistance

The disturbing moment M_o = Moment due to $(F_{debris} + F_{bould} + F_{ep} + F_{wat} + F_{surch})$

M_o = 54461.5 kN

The restoring moment Mr = Moment due to $(W_{stem}+W_{base}+W_{stor}+W_{debris})$

M_{res} = 202892.6 kN

and **FOS_{over} = 3.725 > 1** **OK in Overturning**

Checking Bearing Pressures

The eccentricity, e of the resultant vertical force is:

$e = (L_b / 2) - (M_{res} - M_o) / F_{vertical}$

with $F_{vertical} = W_{stem} + W_{base} + W_{stor} + W_{debris}$

M_{res} = 202892.6 kNm or 12680.8 kNm/m

M_o = 54461.5 kNm or 3403.8 kNm/m

F_{vert} = 26429.8 kN or 1651.9 kN/m

e = 2.134 m

By Figure A1 in Geoguide 1, the effective length of the base, L_b' is :

$L_b' = L_b - 2e$ = 11.232 m

The maximum bearing stress is found as:

$q_{max} = F_{vert} / (L_b' \times L_w)$

= 147.1 kPa < 300 kPa **OK in Bearing**

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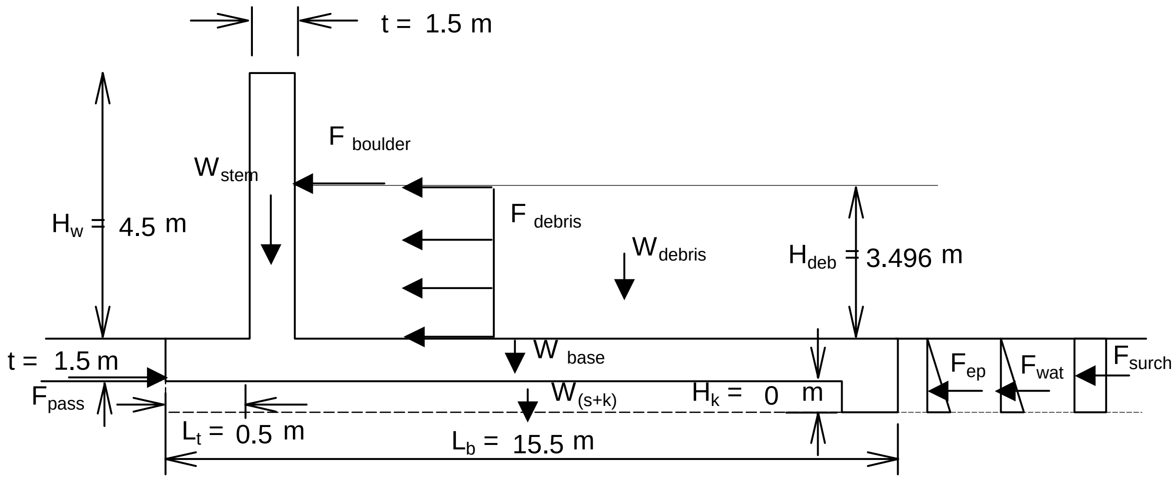
Stability Check for 1200 m³ Event (with boulder in 600 m³ event)

Type 1A - 4.5 m with sloping ground in front - 1st pulse 50% weight



Assumptions:

γ_{soil} =	19.7 kN/m³	γ_{con} =	24 kN/m³
γ_{wat} =	9.81 kN/m³	γ_{gab} =	18 kN/m³
Base Friction Angle ø =	35 °	$K_a = K_o$ =	1
		K_p =	0.00
Debris Length, L_{deb} =	12.77 m		
Debris Height, H_{deb} =	3.496 m		
Static Debris Height, H_{sta} =	0 m		
Wall Length, L_w =	16 m	Concrete thickness, t =	1.5 m
Wall Height, H_w =	4.5 m	Length of Toe, L_t =	0.5 m
Wall Base Length , L_b =	15.5 m	Height of Key, H_k =	0 m

Mark	Force (kN)	Arm (m)	Moment (kNm)
F_{debris} =	10141.0	3.248	32938.0
F_{bould} =	6986.0	4.996	34902.1
F_{ep} =	178.0	0.500	89.0
F_{wat} =	176.6	0.500	88.3
F_{surch} =	1319.2	0.750	989.4
W_{stem} =	2592.0	-1.250	-3240.0
W_{base} =	8928.0	-7.750	-69192.0
W_{stor} =	0.0	-8.750	0.0
W_{debris} =	11873.1	-8.750	-103889.2
$W_{(s+k)}$ =	0.0	-	-
F_{pass} =	0.0	-	-

(-ve denote stabilising moment)

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Stability Check for 1200 m³ Event (with boulder in 600 m³ event)

Type 1A - 4.5 m with sloping ground in front - 1st pulse 50% weight

Checking Sliding Resistance

The disturbing force $F_{slid} = F_{debris} + F_{bould} + F_{ep} + F_{wat} + F_{surch}$

$\mathbf{F_{slid}}$ **= 18800.8 kN**

The resisting force $F_{res} = (W_{stem}+W_{base}+W_{stor}+W_{debris}/2+W_{(s + key)}) \times \text{Tan}\ \phi + F_{pass}$

$\mathbf{F_{res}}$ **= 12223.2 kN**

and $\mathbf{FOS_{slid}}$ **= 0.650 < 1** **Movement check is required**

Checking Overturning Resistance

The disturbing moment M_o = Moment due to $(F_{debris} + F_{bould} + F_{ep} + F_{wat} + F_{surch})$

$\mathbf{M_o}$ **= 69006.7 kN**

The restoring moment Mr = Moment due to $(W_{stem}+W_{base}+W_{stor}+W_{debris})$

$\mathbf{M_{res}}$ **= 176321.2 kN**

and $\mathbf{FOS_{over}}$ **= 2.555 > 1** **OK in Overturning**

Checking Bearing Pressures

The eccentricity, e of the resultant vertical force is:

$e = (L_b / 2) - (M_{res} - M_o) / F_{vertical}$

with $F_{vertical} = W_{stem} + W_{base} + W_{stor} + W_{debris}$

M_{res} =	176321.2	kNm	or	11020.1	kNm/m
M_o =	69006.7	kNm	or	4312.9	kNm/m
F_{vert} =	23393.1	kN	or	1462.1	kN/m

e = 3.163 m

By Figure A1 in Geoguide 1, the effective length of the base, L_b' is :

$L_b' = L_b - 2e =$ 9.175 m

The maximum bearing stress is found as:

$\mathbf{q_{max}} = F_{vert} / (L_b' \times L_w)$

= 159.4 kPa < 300 kPa **OK in Bearing**

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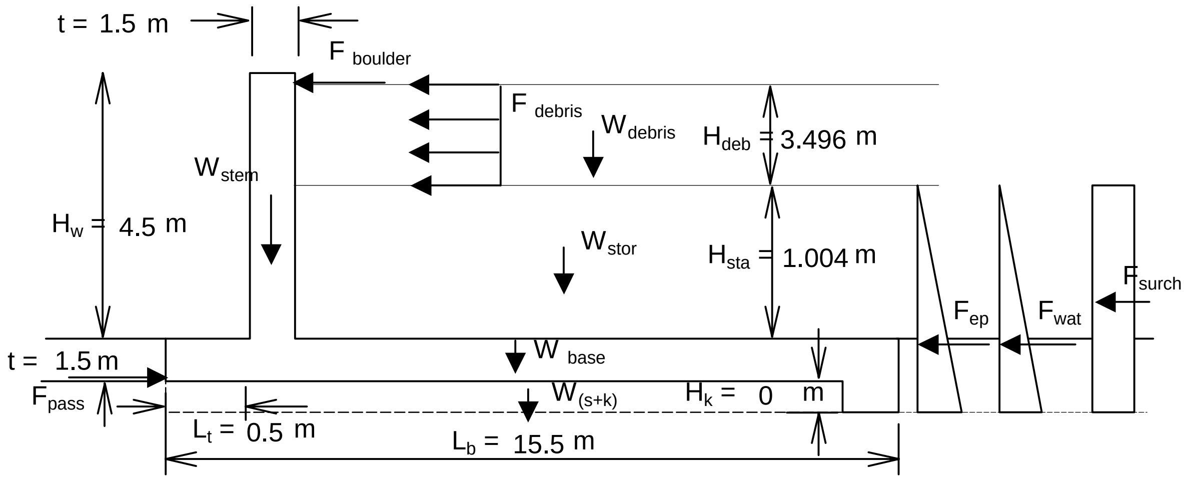
Stability Check for 1200 m³ Event (with boulder in 600 m³ event)

Type 1A - 4.5 m with sloping ground in front - 2nd pulse 50% weight + lower static soil



Assumptions:

γ_{soil} =	19.7 kN/m³	γ_{con} =	24 kN/m³
γ_{wat} =	9.81 kN/m³	γ_{gab} =	18 kN/m³
Base Friction Angle ø =	35 °	$K_a = K_o$ =	1
		K_p =	0.00
Debris Length, L_{deb} =	12.77 m		
Debris Height, H_{deb} =	3.496 m		
Static Debris Height, H_{sta} =	1.004 m		
Wall Length, L_w =	16 m	Concrete thickness, t =	1.5 m
Wall Height, H_w =	4.5 m	Length of Toe, L_t =	0.5 m
Wall Base Length , L_b =	15.5 m	Height of Key, H_k =	0 m

Mark	Force (kN)	Arm (m)	Moment (kNm)
F_{debris} =	10141.0	4.252	43119.5
F_{bould} =	6986.0	6.000	41916.0
F_{ep} =	496.1	0.835	414.1
F_{wat} =	492.1	0.835	410.7
F_{surch} =	2202.2	1.252	2757.2
W_{stem} =	2592.0	-1.250	-3240.0
W_{base} =	8928.0	-7.750	-69192.0
W_{stor} =	4272.2	-8.750	-37381.9
W_{debris} =	11873.1	-8.750	-103889.2
$W_{(s+k)}$ =	0.0	-	-
F_{pass} =	0.0	-	-

(-ve denote stabilising moment)

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Stability Check for 1200 m³ Event (with boulder in 600 m³ event)

Type 1A - 4.5 m with sloping ground in front - 2nd pulse 50% weight + lower static soil

Checking Sliding Resistance

The disturbing force $F_{slid} = F_{debris} + F_{bould} + F_{ep} + F_{wat} + F_{surch}$

F_{slid} = 20317.4 kN

The resisting force $F_{res} = (W_{stem}+W_{base}+W_{stor}+W_{debris}/2+W_{(s + key)}) \times \text{Tan}\ \phi + F_{pass}$

F_{res} = 15214.6 kN

and **FOS_{slid} = 0.749 < 1** **Movement check is required**

Checking Overturning Resistance

The disturbing moment M_o = Moment due to $(F_{debris} + F_{bould} + F_{ep} + F_{wat} + F_{surch})$

M_o = 88617.5 kN

The restoring moment Mr = Moment due to $(W_{stem}+W_{base}+W_{stor}+W_{debris})$

M_{res} = 213703.1 kN

and **FOS_{over} = 2.412 > 1** **OK in Overturning**

Checking Bearing Pressures

The eccentricity, e of the resultant vertical force is:

$$e = (L_b / 2) - (M_{res} - M_o) / F_{vertical}$$

with $F_{vertical} = W_{stem} + W_{base} + W_{stor} + W_{debris}$

M_{res} =	213703.1 kNm	or	13356.4 kNm/m
M_o =	88617.5 kNm	or	5538.6 kNm/m
F_{vert} =	27665.3 kN	or	1729.1 kN/m

e = 3.229 m

By Figure A1 in Geoguide 1, the effective length of the base, L_b' is :

$L_b' = L_b - 2e =$ 9.043 m

The maximum bearing stress is found as:

$$q_{max} = F_{vert} / (L_b' \times L_w)$$

= 191.2 kPa < 300 kPa **OK in Bearing**

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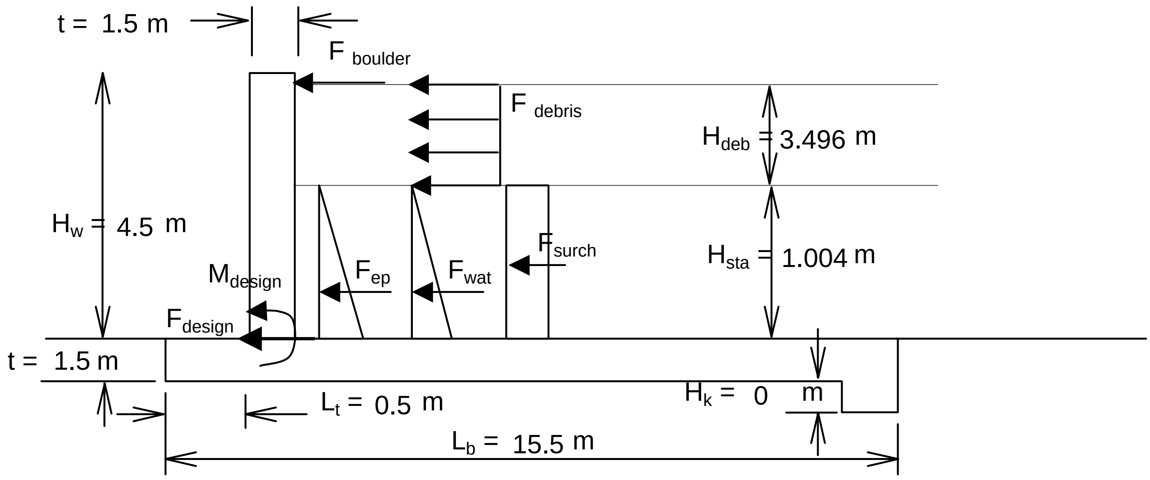
Wall Stem Design for 1200 m³ Event (with boulder in 600 m³ event)

Type 1A - 4.5 m with sloping ground in front - 2nd pulse 50% weight + lower static soil



Assumptions:

γ_{soil} =	19.7	kN/m³	γ_{con} =	24	kN/m³
γ_{wat} =	9.81	kN/m³	γ_{gab} =	18	kN/m³
Base Friction Angle ø =	35	°	$K_a = K_o$ =	1	
			K_p =	0.00	

Debris Length, L_{deb} =	12.77	m
Debris Height, H_{deb}=	3.496	m
Static Debris Height, H_{sta}=	1.004	m

Wall Length, L_w =	16	m	Concrete thickness, t =	1.5	m
Wall Height, H_w =	4.5	m	Length of Toe, L_t =	0.5	m
Wall Base Length , L_b =	15.5	m	Height of Key, H_k =	0	m

Mark	Force (kN)	Arm (m)	Moment (kNm)
F_{debris} =	10141.0	2.752	27908.0
F_{bould} =	6986.0	4.500	31437.0
F_{ep} =	63.7	0.335	21.3
F_{wat} =	63.1	0.335	21.1
F_{surch} =	883.0	0.502	443.3

Design Shear Force

The design shear force $F_{design} = F_{debris} + F_{bould} + F_{ep} + F_{wat} + F_{surch}$

F_{design} = 18136.8 kN over the debris length

or **F_{design} = 1420.3 kN per m run**

Design Bending Moment

The design shear force M_{design} = Moment due to ($F_{debris} + F_{bould} + F_{ep} + F_{wat} + F_{surch}$)

M_{design} = 59830.7 kN over the debris length

M_{design} = 4685.3 kN per m run

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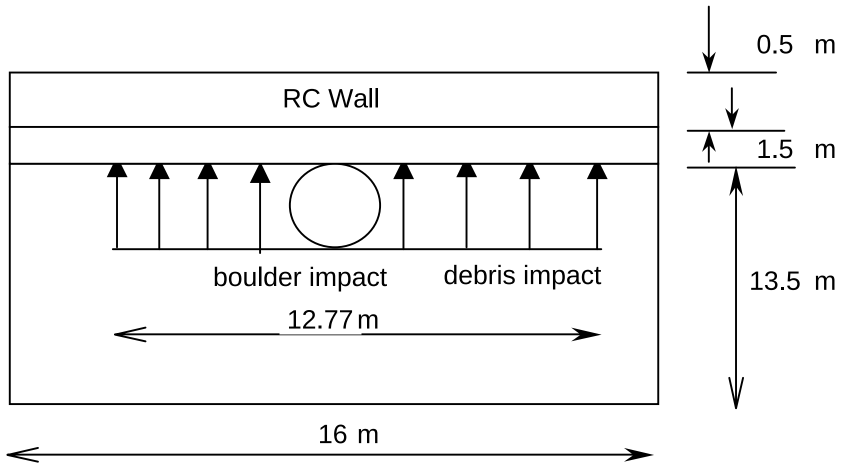
Checking Displacement of Barrier under Impact Load

The impact load from the debris and the boulder are assumed to give rise to a certain amount of displacement of the barrier. This will dissipate the energy from the impact, however the displacement has to be kept within limits to ensure the wall integrity.

Typical wall layout:



Assumption:

The debris and boulder impact loads are assumed to be transferred through the whole of the barrier structure. Hence, the displacement will apply to **the whole of the Barrier structure** .

The 1200 m³ events are checked as this will give rise to the most significant displacement.

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Derivation of Debris Mass Impacting the Wall for 1200 m³ Event

Type 1A - 4.5 m with sloping ground in front - 1st pulse 50% weight

Type 1A - 4.5 m with sloping ground in front - 2nd pulse 50% weight + lower static soil

The debris impacts the wall at a velocity, V_1, and the impact force calculations have yielded a debris impact force, Fdebris:

	1st pulse	2nd pulse
V_1 =	6.951 m/s	6.951 m/s
F_{debris} =	10141 kN	10141 kN
$F_{static} = F_{ep}+F_{wat}+F_{surch}$ =	1673.8 kN	3190.4 kN
$F_{impact\ total}$=	11814.8 kN	13331.4 kN

In order to determine the mass of the debris acting on the wall at impact, we use Newton's law of motion, assuming that the debris stops completely under impact:

$$\mathbf{F = M_{debris} * a} \qquad \textbf{(Eq. 1)}$$

We assume that the arrest will be done within a time period, Δt:

Δt = 0.5 s

The decceleration (negative acceleration) can be written as:

$a = \Delta V / \Delta t$

or also $a = (V_1 - V_{final}) / \Delta t$

and here we have **a = 13.902 m/s²**

Rearranging **Eq. 1** in function of the debris mass, yields:

$M_{debris} = F_{debris} / a$

and, 1st pulse $\mathbf{M_{debris\ 1}}$ **= 849.87 Mg**

2nd pulse $\mathbf{M_{debris\ 2}}$ **= 958.95 Mg**

<u>Boulder Mass Determination</u>

The assumptions are:

$\rho_{boulder}$ = 26.5 kN/m³

or $\rho_{boulder}$ = 2701.33 kg/m³

Boulder dia. = 2.528 m

The boulder mass is therefore:

$M_{boulder} = 4 * \pi * \rho_{boulder} * (Dia/2)^3 / 3$

$\mathbf{M_{boulder}}$ **= 22.85 Mg**

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Derivation of Debris Mass Impacting the Wall for 1200 m³ Event

Type 1A - 4.5 m with sloping ground in front - 1st pulse 50% weight

Type 1A - 4.5 m with sloping ground in front - 2nd pulse 50% weight + lower static soil

Wall Mass Determination

The assumptions are:

	Wall Length, L_w =	16	m	
	Wall Height, H_w =	4.5	m	
	Wall Thickness, t =	1.5	m	
	Base length, L_b =	15.5	m	
	Density of the Wall =	24	kN/m^3	(for RC wall)
or	Density of the Wall =	2446.48	kg/m^3	
	$M_{barrier}$ =	**1174.31**	**Mg**	

Mass of Static Soil Determination

The assumptions are:

1st pulse

	Wall Length, L_w =	16	m
	Thickness of Static Soil, H_{sta} =	0	m
	Wall heel length, L_b - L_t - t =	13.5	m
	Density of the soil =	19.7	kN/m^3
or	Density of the Wall =	2008.15	kg/m^3
	M_{static} =	**0.00**	**Mg**

2nd pulse

	Wall Length, L_w =	16	m
	Thickness of Static Soil, H_{sta} =	1.004	m
	Wall heel length, L_b - L_t - t =	13.5	m
	Density of the soil =	19.7	kN/m^3
or	Density of the Wall =	2008.15	kg/m^3
	M_{satic} =	**435.50**	**Mg**

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Calculate Movement of Structure under Impact Load

We will use the equation of conservation of momentum to determine the total movement under load. The equation is written as:

$$M_1 \times V_1 = M_2 \times V_2$$

or in terms relevant to this case:

$$\mathbf{(M_{debris} + M_{bould}) \times V_1 = (M_{debris} + M_{bould} + M_{barrier} + M_{static} + M_{(s+key)}) \times V_2} \qquad \textbf{(Eq.2)}$$

where,

M_{debris} = Mass of debris impacting
M_{bould} = Mass of boulder impacting
$M_{barrier}$ = Mass of barrier undergoing movement
M_{static} = Mass of static soil
$M_{(S+Key)}$ = Mass of Soil in front of shear key
V_1 = Velocity of debris and boulder before impact
V_2 = Velocity of debris, boulder and barrier after impact

We assume that the boulder, the debris and the barrier and the soil in front of the shear key move together and come to rest at a distance **S** after impact, the movement is governed by the following equation:

$$V_{final}^2 = V_2^2 + 2 \times a \times S \qquad \textbf{(Eq.3)}$$

where,
V_{final} = Final Velocity
V_2 = Initial Velocity of the Boulder, Debris, Barrier System
a = Acceleration
S = Distance Moved

Since we want to find the distance moved before the system stops we have:

V_{final} = 0 m/s and, $0 = V_2^2 + 2 \times a \times S$

We can therefore rearrange the **Eq. 3** in function of the acceleration and we get:

$$\mathbf{a = -V_2^2 / (2 \times S)} \qquad \textbf{(Eq.3a)}$$

Rearranging **Eq. 2** in function of V2 and replacing it in **Eq. 3a**, yields:

$$\mathbf{a = -[(M_{debris} + M_{bould}) \times V_1 / (M_{debris} + M_{bould} + M_{barrier} + M_{static} + M_{(S+Key)})]^2 / (2 \times S)} \qquad \textbf{(Eq.3b)}$$

From Newton's Law of Motion:

$$\mathbf{F = m \times a}$$

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Calculate Movement of Structure under Impact Load

Therefore, the force applied to the system can be found as:

$$\mathbf{F = (M_{debris} + M_{bould} + M_{barrier} + M_{static}) \times a}$$

Replacing the acceleration term from **Eq. 3b**, yields:

$$\mathbf{F = (M_{debris} + M_{bould} + M_{barrier} + M_{static}) \times ((M_{debris} + M_{bould}) \times V_1)^2 / (M_{debris} + M_{bould} + M_{barrier} + M_{static} + M_{(S + Key)})^2 / (2 \times S)} \quad \textbf{(Eq.4)}$$

The base friction afforded by the wall, due to the self weight of the soil and wall, and the passive resistance will provide the resisting force to the impact and hence slow the system down, the force is then:

$$\mathbf{F_{frict + pass} = (M_{barrier} + M_{bould} + M_{debris} + M_{static} + M_{(s + key)}) \times g \times \tan\phi + F_{pass}} \quad \textbf{(Eq.5)}$$

Equating the Force in **Eq. 5** into **Eq. 4** and rearranging in term of **S** gives:

and rearranging to find **S**, gives:

$$\mathbf{S = \{ (M_{debris} + M_{bould}) * V_1)^2 \times (M_{debris} + M_{bould} + M_{barrier} + M_{static}) \} / \{ 2 \times (M_{debris} + M_{bould} + M_{barrier} + M_{static} + M_{(s + key)})^2 \times (M_{debris} + M_{bould} + M_{barrier} + M_{static} + M_{(s + key)}) \times g \times \tan\phi + F_{pass} \}} \quad \textbf{(Eq. 6)}$$

	1st Pulse Impact Load			2nd Pulse Impact Load		
Eq. 6 with	M_{debris} =	849.87	Mg	M_{debris} =	958.95	Mg
	M_{bould} =	22.85	Mg	M_{bould} =	22.85	Mg
	$M_{barrier}$ =	1174.31	Mg	$M_{barrier}$ =	1174.31	Mg
	M_{static} =	0.00	Mg	M_{static} =	435.50	Mg
	$M_{(s + key)}$ =	0.00	Mg	$M_{(s + key)}$ =	0.00	Mg
	F_{pass} =	0.00	kN	F_{pass} =	0.00	kN
	V_1 =	6.951	m/s	V_1 =	6.951	m/s
	ϕ =	35	^{o}	ϕ =	35	^{o}
	g =	9.81	m/s^2	g =	9.81	m/s^2
	S_{1st} =	**0.64**	**m**	**S_{2nd} =**	**0.50**	**m**

*** Maximum allowable displacement in Double Event is 1.5 m**

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APPENDIX G

DESCRIPTION OF THE DEBRIFLO PROGRAM
BY MAUNSELL GEOTECHNICAL SERVICES LIMITED
(EXTRACTED FROM THE DEBRIFLO SUBMISSION TO
THE SPECIAL PROJECTS DIVISION OF THE GEO)

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G.1 AREAS OF APPLICATION

The program is applied to model debris flow fronts where debris composed of soil, rock and water flow along an inclined channel. The program models the leading-edge of the debris front as a single pulse and can be used as a predictive tool and also as a tool to back-analyse past debris flow events.

G.2 BRIEF DESCRIPTION OF THE PROGRAM, ASSUMPTIONS AND THEORY

The program is an "Excel" workbook consisting of a number of linked worksheet modules as described in the user manual. The rheologic model adopted in the program is the Voellmy model, which considers turbulence and frictional resistance (refer Table 6 of GEO Report No. 104). When a very high turbulence coefficient (ξ) is input (i.e. very low turbulence), the rheologic model effectively becomes friction-only.

The program is based on Netwon's 2nd law of motion and the logic is similar to the DAN model developed by Hungr (1995) and the "leading edge" equations of Takahashi and Yoshida (1979). The equations satisfy the principles of conservation of mass, momentum, energy and continuity of flow for a fluid medium and were developed to allow for a variety of factors, including variations in flow height, slope angle and discharge along the debris path.

The spreadsheet expresses the governing equation of the debris flow front, based on the input data in the Section Properties Module, Channel Properties Module and the initial condition variables defined in the Calculation Module. Examples of the input and output modules are shown in Annex A.

The program simulates the passage of a debris front by considering the effects of flow resistance, slope angle and thrust immediately behind the debris front. The effect of waterfalls where the debris falls along a parabolic curve with no reaction from the ground surface has been incorporated. The influence of entrainment and changes in cross-section and bends in the debris trail are also simulated.

The spreadsheet divides the potential length of the path into the segments defined in the Channel Properties Module in terms of chainage and elevation. Each segment comprises several sub-segments, with 300 sub-segments being used in total so that the passage of the debris front is modelled in very small increments of distance (and hence time).

From the initial height (dependent on debris volume and cross-section defined in the input modules) and velocity input, the spreadsheet first calculates the upstream discharge rate and then iterates each line to find the velocity, height and discharge rate compatible with the slope gradient and calculated upstream forces, height and discharge.

The iteration continues until all calculated forces and flow dimensions on each line are in balance in accordance with the Debriflo equation, and there is no change greater than 0.005 in any of the calculated variables in the spreadsheet.

When the program is run, an equivalent trapezium is calculated for each cross-section, and the average height corresponding to the calculated thrust vs. flow resistance is derived. When conducting a predictive calculation, the wetted surface of each cross-section must be

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adjusted so that the average height derived from the input data provides a reasonable match with the average height calculated by the program.

If a back-analysis is being carried out, the parameters chosen must result in a reasonable correlation with any reliable field estimated velocities, and the calculated average height for each cross section should match reasonably well with the average height derived from field measurements of mud-lines at the same chainages. In addition, the overall runout distance should also match with the field observations. Although ϕ or ξ can be adjusted to achieve a field-estimated superelevation velocity at any particular point, both factors need to be adjusted to provide reasonable matches with the overall measured height and velocity profiles along the debris path. A reduction in friction results in higher velocity but lower flow height (and vice versa), while an increase in turbulence results in lower velocity but higher flow height (and vice versa). In cases where there is no change in flow volume, there is only one combination of parameters that will adequately satisfy the field measurements. A very small velocity of about 0.01 m/s is input to simulate the initial conditions at the lip of the source, and the initial height is chosen to reflect the initial thrust exerted by the average depth of the depleted mass.

If the spreadsheet is used for predictive purposes, a set of parameters must be chosen, preferably based on parameters derived from back-analyses of previous debris flows in Hong Kong. In this case, the cross-sections of the channel are input and the extent of the wetted profiles is then adjusted so that the average heights derived from the wetted profiles match the average heights calculated in the spreadsheet. The initial conditions can be modeled in the same way as for a back-analysis or, a 'launching' channel can be assumed with the initial debris depth and velocity corresponding to the 'design volume' upper-bound values derived from previous back-analyses of debris flows in Hong Kong (standard barrier framework).

G.3 PROGRAM LIMITATIONS AND RANGE OF INPUT PARAMETERS

The intrinsic two-dimensional nature of the Debriflo model means that, unless any deposition input in the spreadsheet exceeds the assumed initial volume plus all entrained material, the supply behind the front is considered to be infinite. This limitation results in conservative estimations of runout when no deposition is assumed as the debris front is decelerating.

In the calculation module of the program, the shape of the debris front must be defined in terms of the maximum height/average height ratio. For typical cases in Hong Kong, it is recommended that a rectangular shape (h_u/h_{av} = 1.0) is selected for the following reasons:

(a) For the back-analyses of previous events in Hong Kong, a rectangular shape (h_u/h_{av} = 1.0) was assumed because the debris flow fronts were very bouldery, and are likely to have been approximately rectangular in profile due to the higher frictional resistance (and therefore steeper gradient) of the front.

(b) Assuming this shape gives results that correspond well with field observations and Hungr's independent analyses using the DAN program.

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(c) If the parameters derived from the Debriflo back-analyses are used to predict the behaviour of flows of similar composition using the Debriflo spreadsheet, then a rectangular profile for the debris front should also be input.

(d) The "Leading-edge" model described in Table 7 of GEO Report No. 104 (derived from Hungr et al., 1984 and Takahashi & Yoshida, 1979) is often used to predict velocity in the runout area. This model assumes that the debris front profile is rectangular. It is therefore less ambiguous when comparing results if a rectangular profile has been used in the Debriflo calculations.

However, for the analysis of very low-friction flows such as mud flows (none has so far been back-analysed in Hong Kong), it is recommended that the debris front is assumed to be parabolic when using the Debriflo spreadsheet.

The chaotic nature of debris flows means that any computer program can only be expected to provide an approximate simulation. Considerable professional engineering judgement must be applied when selecting parameters and initial conditions, and must be based on local and international experience of back-analysing these complex events. Although a certain range of input parameters can be recommended based on previous back-analyses, it is still up to the experienced professional to determine the most appropriate parameters and conditions that suit the problem at hand. The range of parameters given below should not, therefore, be treated as absolute limits, and combinations of parameters outside these ranges may occasionally be applied to suit specific site conditions provided that adequate justification is given.

Geometry Module

Input Data	Limitations and Recommended Value	Reference/Comment
Chainage	Location of section	Must be in consecutive order
U	Horizontal coordinate to define the reference point in a section	Absolutely vertical or overhanging sides should not be input.
V	Vertical coordinate to define the reference point in a section	
Wet	X for the reference point below the debris	Defines the height of debris in the channel

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Channel Module

Input Data	Limitations and Recommended Value	Reference/Comment
Horiz CH	Location of section (m)	Must be in consecutive order
Level	The elevation in mPD of a section	
Sigma	Bulk density of the non-suspended boulders 2400 kg/m³ typical.	Should give a reasonable overall bulk density (gamma) for the debris and a reasonable Sf (tanϕ) which is typically 0.2 for channelised debris flows, but may be as low as 0.1 for very wet debris flows. Alternatively, Sf may be directly input for high-friction cases up to ϕ = 30°. GEO Report 104, Bagnold (1954), Hungr, 1985 Hungr (1998), Ayotte & Hungr (1998), MGSL Debriflo backanalyses and Standard Barrier calibrations
Ro	Bulk density of the slurry medium 1300 - 1460 kg/m³ typical range. 1000 kg/m³ to model effective stress when 100% non-suspended solids.	
c	Percentage of non-suspended boulders in the debris front. 33% to 66% typical. 100% for friction-only slide.	
Alpha	Angle of friction of the non-suspended boulders. 30° typical.	
ksi	Turbulence factor ksi = 500 m/s^2 (typical), 200 m/s^2 (high turbulence, 10000 m/s^2 (effectively negligible turbulence)	GEO Report 104 Hungr (1998), Ayotte & Hungr (1998), MGSL Debriflo backanalyses and Standard Barrier calibrations
ksi_f	Power to shape factor for computation of variable ξ in channel of variable shape. 1.0 typical when Sf = 0.2 1.5 typical when Sf = 0.1 0 for ξ independent of channel shape.	MGSL Debriflo backanalyses and Standard Barrier calibrations
Radius	Radius of curvature between two sections. Use 10000 or -10000 to model the straight line	MGSL Debriflo backanalyses and Standard Barrier calibrations

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Calculation Module

Input Data	Limitations and Recommended Value	Reference/Comment
Job Title	Title of the analysis	
$h_{initial}$	Initial effective average height of debris. Can be proportional to average depth of failed mass or based on an upper bound value proportional to design event volume if a 'launching' channel is used.	Takahashi & Yoshida (1979), MGSL Debriflo backanalyses and Standard Barrier calibrations
$v_{initial}$	Initial velocity of debris flow $v_{initial}$ = 0.01 m/s when the model starts from the lip of the landslide scar or can be based on an upper bound value proportional to design event volume if a 'launching' channel is used.	MGSL Debriflo backanalyses and Standard Barrier calibrations
h_u/h_{av}	Ratio of max height to average height of debris. It is used to define the shape of debris front For parabola $h_u = 1.5\ h_{av}$ For rectangle $h_u = h_{av}$ (recommended) For triangle $h_u = 2\ h_{av}$	Hungr (1995) MGSL Debriflo backanalyses and Standard Barrier calibrations
k	Coefficient of effective lateral pressure in debris k =1 (recommended)	Hungr (1995)
n	Coefficient of gradual setting of fines n = 1 (recommended)	Hungr et al. (1984)
Rad k	Radius of superelevation Must be 0.3	MGSL Debriflo backanalyses
Damping	Must be 0.9	MGSL Debriflo backanalyses
Conv factor	To reduce numerical instability Conv factor = 0.01 to 0.99	Faster convergence with higher value. If numerical instability results, try a lower value.
Tolerance for 'Landing'	Help the program to determine the debris landing position to reduce numerical instability. 0.0 m to 1.0 m (typical)	Any value within this range that results in numerical stability is acceptable, but preferably the smaller, the better

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G.4 CODES OF PRACTICE AND VERIFICATION

The Debriflo program satisfies the principles of conservation of mass, momentum, energy and continuity of flow for a fluid medium and has been calibrated against field observations of previous debris flows in Hong Kong. The program therefore satisfies the criteria outlined in GEO Report No. 104 for models which are suitable for the analytical determination of debris mobility.

The spreadsheet has been validated against simple models where the results can be checked by hand-calculation. Three verification examples are shown in Appendix B of the User Manual.

The model has also been directly validated against known case histories, where good quality and unambiguous field data are available. Appendix C in Volume 2 of the User Manual describes the back-analyses of the 1999 Sham Tseng San Tsuen debris flow and the 1990 Tsing Shan debris flow. The Debriflo back-analyses produce a good fit with the data and the independent DAN analyses.

G.5 REFERENCES

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ANNEX A

EXAMPLES OF INPUT AND OUTPUT

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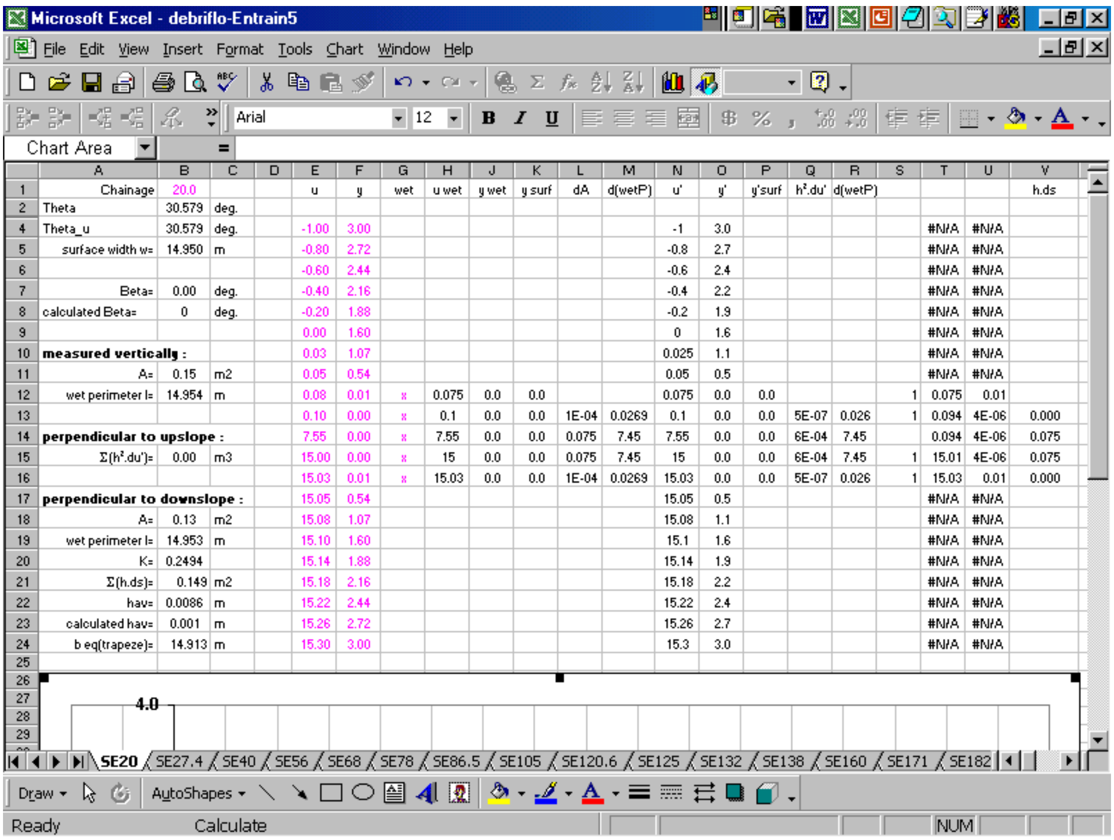
A.1 DATA INPUT

The cells in magenta colour are required to be input by user, other cells will be calculated by the spreadsheet. The data input is divided into three modules, section properties module, channel properties module and calculation module.

A.1.1 Section Properties Module

The geometry of a section is represented by a series of points (u, y) joined by straight lines. The sections must be input as measured on a vertical plane

Variable	Description	
	INPUT	
Chainage	Section location. It is recommended to use the lip of the landslide scar as a reference point (with an initial velocity near zero)	
U	Horizontal coordinate of reference point	
Y	Vertical coordinate of reference point	
Wet	x	The reference point is below the surface of debris flow
	Blank	The reference point is above the surface of debris flow



	A	B	C	D	E	F	G	H	J	K	L	M	N	O	P	Q	R	S	T	U	V
1	Chainage	20.0			u	y	wet	u wet	y wet	y surf	dA	d(wetP)	u'	y'	y'surf	$h^2.du'$	d(wetP)				h.ds
2	Theta	30.579	deg.																		
4	Theta_u	30.579	deg.		-1.00	3.00							-1	3.0					#N/A	#N/A	
5	surface width w=	14.950	m		-0.80	2.72							-0.8	2.7					#N/A	#N/A	
6					-0.60	2.44							-0.6	2.4					#N/A	#N/A	
7	Beta=	0.00	deg.		-0.40	2.16							-0.4	2.2					#N/A	#N/A	
8	calculated Beta=	0	deg.		-0.20	1.88							-0.2	1.9					#N/A	#N/A	
9					0.00	1.60							0	1.6					#N/A	#N/A	
10	**measured vertically :**				0.03	1.07							0.025	1.1					#N/A	#N/A	
11	A=	0.15	m2		0.05	0.54							0.05	0.5					#N/A	#N/A	
12	wet perimeter l=	14.954	m		0.08	0.01	x	0.075	0.0	0.0			0.075	0.0	0.0			1	0.075	0.01	
13					0.10	0.00	x	0.1	0.0	0.0	1E-04	0.0269	0.1	0.0	0.0	5E-07	0.026	1	0.094	4E-06	0.000
14	**perpendicular to upslope :**				7.55	0.00	x	7.55	0.0	0.0	0.075	7.45	7.55	0.0	0.0	6E-04	7.45		0.094	4E-06	0.075
15	$\Sigma(h^2.du')$=	0.00	m3		15.00	0.00	x	15	0.0	0.0	0.075	7.45	15	0.0	0.0	6E-04	7.45	1	15.01	4E-06	0.075
16					15.03	0.01	x	15.03	0.0	0.0	1E-04	0.0269	15.03	0.0	0.0	5E-07	0.026	1	15.03	0.01	0.000
17	**perpendicular to downslope :**				15.05	0.54							15.05	0.5					#N/A	#N/A	
18	A=	0.13	m2		15.08	1.07							15.08	1.1					#N/A	#N/A	
19	wet perimeter l=	14.953	m		15.10	1.60							15.1	1.6					#N/A	#N/A	
20	K=	0.2494			15.14	1.88							15.14	1.9					#N/A	#N/A	
21	Σ(h.ds)=	0.149	m2		15.18	2.16							15.18	2.2					#N/A	#N/A	
22	hav=	0.0086	m		15.22	2.44							15.22	2.4					#N/A	#N/A	
23	calculated hav=	0.001	m		15.26	2.72							15.26	2.7					#N/A	#N/A	
24	b eq(trapeze)=	14.913	m		15.30	3.00							15.3	3.0					#N/A	#N/A	

Figure A.1.1.1 - Section Properties Module

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Variables	Description
	OUTPUT
u wet	Horizontal coordinate of the wetted section
y wet	Vertical coordinate of the wetted section
y surf	Debris surface level
DA	Area between 2 consecutive coordinates
d(wetP)	Wetted perimeter between 2 consecutive coordinates
u’	Equivalent u used for calculation
y’	Equivalent y used for calculation
y’surf	Equivalent debris surface level

The spreadsheet will generate the shape of channel for data checking purposes. Figure A.1.1.2 is a typical output of the section properties module. The blue line is the geometric shape of the channel and the magenta is the wet boundary. The black line represents the equivalent idealized trapezoidal channel which has the same thrust. The idealized section is used when calculating the interrelationships between vertical height, flow resistance and cross-sectional area.

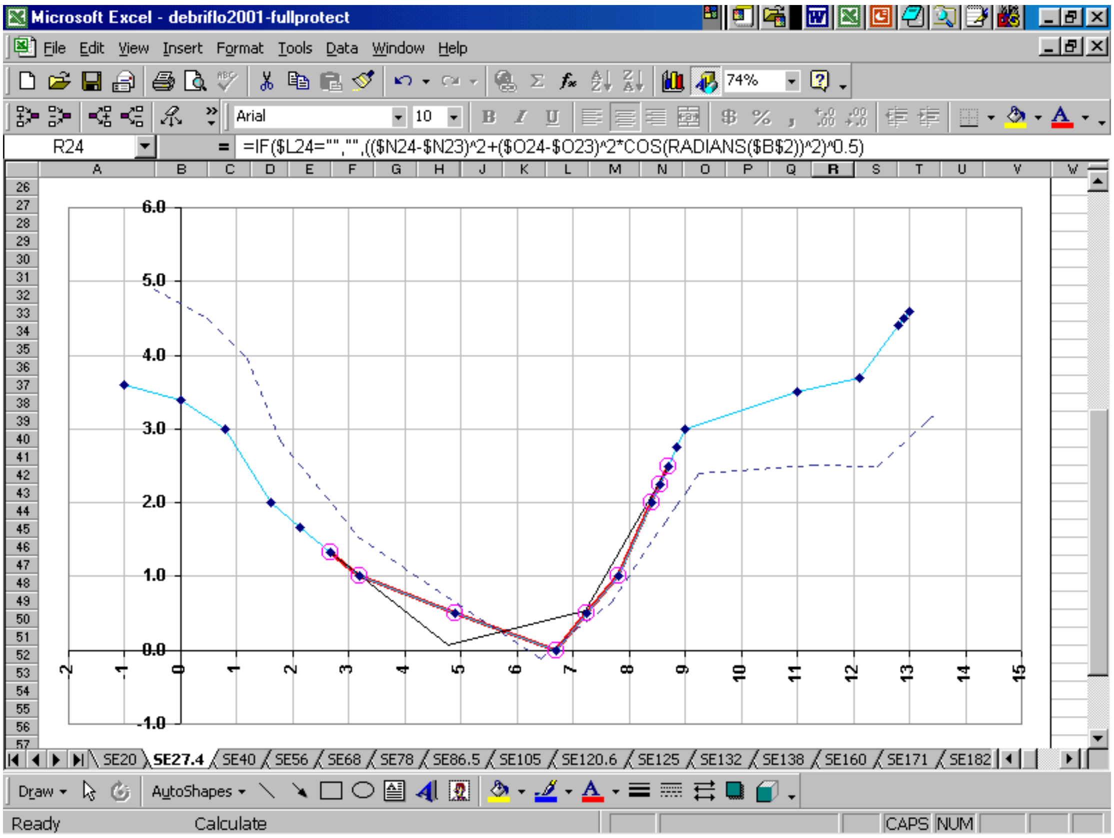

Figure A.1.1.2 - Geometric Shape of Section

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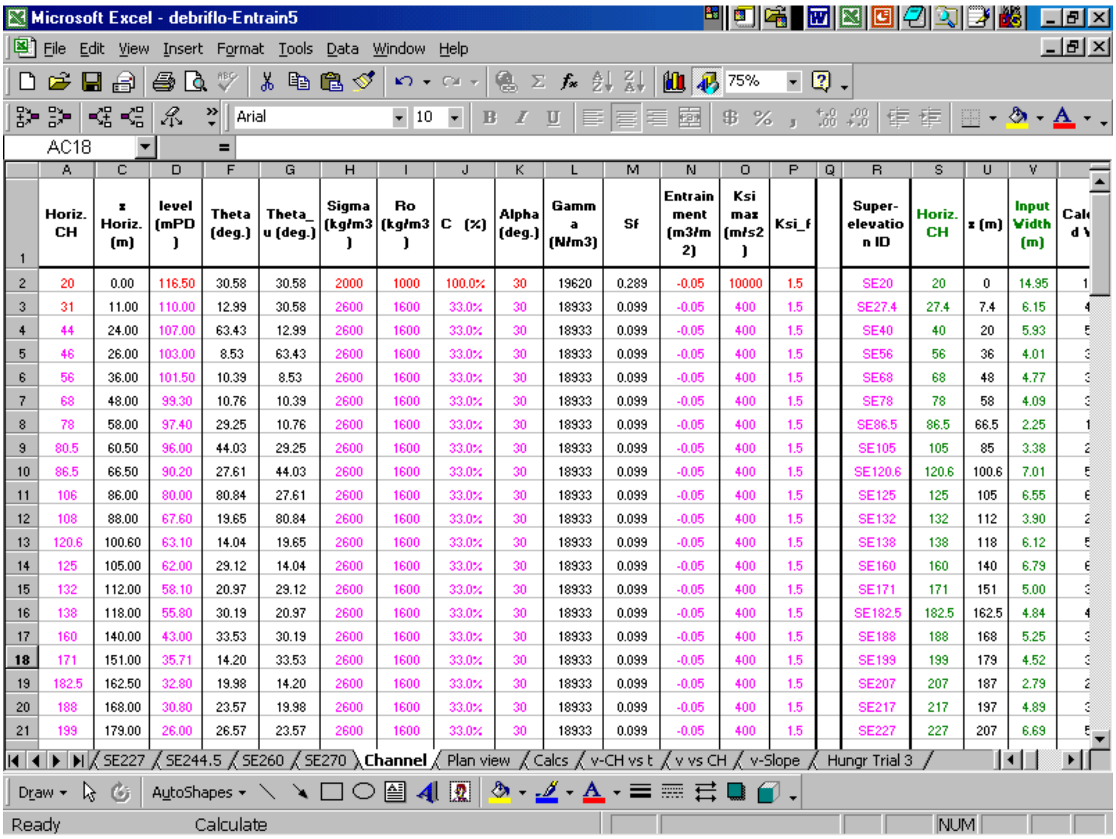
A.1.2 Channel Properties Module

	Variables	Description
		INPUT
Definition of slope Segments	Horiz CH	Horizontal chainage mark (meter)
	level	The elevation of a section (mPD)
	sigma	Bulk density of the debris (kg/m³)
	Ro	Bulk density of the slurry (kg/m³)
	c	Volumetric content of the non-suspended particles (%)
	Alpha	Intergranular angle of friction of the non-suspended particles (degrees)
	Entrainment	Volume of debris entrained (m³/m²) (+ve indicates entrainment, -ve indicates deposition)
	ksi	Turbulence factor (m/s²)
	ksi_f	Power to shape factor for computation of ξ (enter 0 for ξ independent of channel shape) ξ_c is the turbulence factor used in the program $\xi_c = \xi_{input}/(P/w)\ \xi_f$
Summary table	Super elevation ID	The name of worksheet containing channel cross-section geometry data
Horizontal bends	CH	Chainage of the section
	Radius	Radius of curvature between two sections. Use very large radii (such as 10000 or –10000) in order to model straight line



Microsoft Excel - debriflo-Entrain5

	Horiz. CH (A)	z Horiz. (m) (C)	level (mPD) (D)	Theta (deg.) (F)	Theta_u (deg.) (G)	Sigma (kg/m3) (H)	Ro (kg/m3) (I)	C (%) (J)	Alpha (deg.) (K)	Gamma (N/m3) (L)	Sf (M)	Entrainment (m3/m2) (N)	Ksi max (m/s2) (O)	Ksi_f (P)	(Q)	Super-elevation ID (R)	Horiz. CH (S)	z (m) (U)	Input Width (m) (V)
2	20	0.00	116.50	30.58	30.58	2000	1000	100.0%	30	19620	0.289	-0.05	10000	1.5		SE20	20	0	14.95
3	31	11.00	110.00	12.99	30.58	2600	1600	33.0%	30	18933	0.099	-0.05	400	1.5		SE27.4	27.4	7.4	6.15
4	44	24.00	107.00	63.43	12.99	2600	1600	33.0%	30	18933	0.099	-0.05	400	1.5		SE40	40	20	5.93
5	46	26.00	103.00	8.53	63.43	2600	1600	33.0%	30	18933	0.099	-0.05	400	1.5		SE56	56	36	4.01
6	56	36.00	101.50	10.39	8.53	2600	1600	33.0%	30	18933	0.099	-0.05	400	1.5		SE68	68	48	4.77
7	68	48.00	99.30	10.76	10.39	2600	1600	33.0%	30	18933	0.099	-0.05	400	1.5		SE78	78	58	4.09
8	78	58.00	97.40	29.25	10.76	2600	1600	33.0%	30	18933	0.099	-0.05	400	1.5		SE86.5	86.5	66.5	2.25
9	80.5	60.50	96.00	44.03	29.25	2600	1600	33.0%	30	18933	0.099	-0.05	400	1.5		SE105	105	85	3.38
10	86.5	66.50	90.20	27.61	44.03	2600	1600	33.0%	30	18933	0.099	-0.05	400	1.5		SE120.6	120.6	100.6	7.01
11	106	86.00	80.00	80.84	27.61	2600	1600	33.0%	30	18933	0.099	-0.05	400	1.5		SE125	125	105	6.55
12	108	88.00	67.60	19.65	80.84	2600	1600	33.0%	30	18933	0.099	-0.05	400	1.5		SE132	132	112	3.90
13	120.6	100.60	63.10	14.04	19.65	2600	1600	33.0%	30	18933	0.099	-0.05	400	1.5		SE138	138	118	6.12
14	125	105.00	62.00	29.12	14.04	2600	1600	33.0%	30	18933	0.099	-0.05	400	1.5		SE160	160	140	6.79
15	132	112.00	58.10	20.97	29.12	2600	1600	33.0%	30	18933	0.099	-0.05	400	1.5		SE171	171	151	5.00
16	138	118.00	55.80	30.19	20.97	2600	1600	33.0%	30	18933	0.099	-0.05	400	1.5		SE182.5	182.5	162.5	4.84
17	160	140.00	43.00	33.53	30.19	2600	1600	33.0%	30	18933	0.099	-0.05	400	1.5		SE188	188	168	5.25
18	171	151.00	35.71	14.20	33.53	2600	1600	33.0%	30	18933	0.099	-0.05	400	1.5		SE199	199	179	4.52
19	182.5	162.50	32.80	19.98	14.20	2600	1600	33.0%	30	18933	0.099	-0.05	400	1.5		SE207	207	187	2.79
20	188	168.00	30.80	23.57	19.98	2600	1600	33.0%	30	18933	0.099	-0.05	400	1.5		SE217	217	197	4.89
21	199	179.00	26.00	26.57	23.57	2600	1600	33.0%	30	18933	0.099	-0.05	400	1.5		SE227	227	207	6.69

SE227 / SE244.5 / SE260 / SE270 / Channel / Plan view / Calcs / v-CH vs t / v vs CH / v-Slope / Hungr Trial 3

Figure A.1.2.1 - Channel Properties Module

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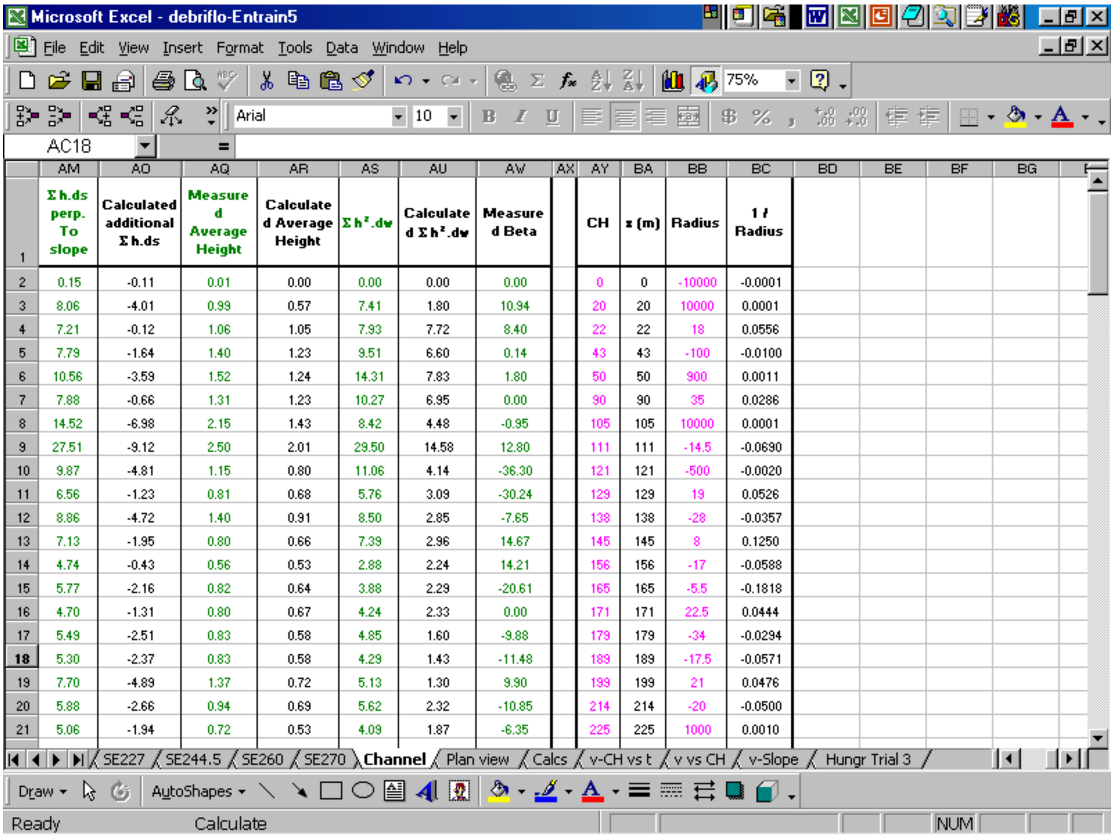

	AM	AO	AQ	AR	AS	AU	AW	AX	AY	BA	BB	BC
1	Σh.ds perp. To slope	Calculated additional Σh.ds	Measured Average Height	Calculated Average Height	Σh².dw	Calculated Σh².dw	Measured Beta		CH	z (m)	Radius	1 / Radius
2	0.15	-0.11	0.01	0.00	0.00	0.00	0.00		0	0	-10000	-0.0001
3	8.06	-4.01	0.99	0.57	7.41	1.80	10.94		20	20	10000	0.0001
4	7.21	-0.12	1.06	1.05	7.93	7.72	8.40		22	22	18	0.0556
5	7.79	-1.64	1.40	1.23	9.51	6.60	0.14		43	43	-100	-0.0100
6	10.56	-3.59	1.52	1.24	14.31	7.83	1.80		50	50	900	0.0011
7	7.88	-0.66	1.31	1.23	10.27	6.95	0.00		90	90	35	0.0286
8	14.52	-6.98	2.15	1.43	8.42	4.48	-0.95		105	105	10000	0.0001
9	27.51	-9.12	2.50	2.01	29.50	14.58	12.80		111	111	-14.5	-0.0690
10	9.87	-4.81	1.15	0.80	11.06	4.14	-36.30		121	121	-500	-0.0020
11	6.56	-1.23	0.81	0.68	5.76	3.09	-30.24		129	129	19	0.0526
12	8.86	-4.72	1.40	0.91	8.50	2.85	-7.65		138	138	-28	-0.0357
13	7.13	-1.95	0.80	0.66	7.39	2.96	14.67		145	145	8	0.1250
14	4.74	-0.43	0.56	0.53	2.88	2.24	14.21		156	156	-17	-0.0588
15	5.77	-2.16	0.82	0.64	3.88	2.29	-20.61		165	165	-5.5	-0.1818
16	4.70	-1.31	0.80	0.67	4.24	2.33	0.00		171	171	22.5	0.0444
17	5.49	-2.51	0.83	0.58	4.85	1.60	-9.88		179	179	-34	-0.0294
18	5.30	-2.37	0.83	0.58	4.29	1.43	-11.48		189	189	-17.5	-0.0571
19	7.70	-4.89	1.37	0.72	5.13	1.30	9.90		199	199	21	0.0476
20	5.88	-2.66	0.94	0.69	5.62	2.32	-10.85		214	214	-20	-0.0500
21	5.06	-1.94	0.72	0.53	4.09	1.87	-6.35		225	225	1000	0.0010

Figure A.1.2.2 - Channel Properties Module

A.1.3 Calculation Module

Variables	Description
	INPUT
Job Title	Input the job title
$h_{initial}$	Input the initial effective height of debris flow (m)
$v_{initial}$	Input the initial velocity of debris flow (m/s). It is recommended to set to a small value such as 0.01 m/s if the model starts from the lip of the landslide scar.
h_u/h_{av}	Input ratio of maximum height of debris to average height of debris For parabola $h_u = 1.5h_{av}$ For rectangle $h_u = h_{av}$ For triangle $h_u = 2h_{av}$
k	Coefficient of effective lateral pressure within debris
n	Coefficient of gradual setting of fines (k and n are assumed to be 1.0)
rad k	Radius of superelevation permanently set to 0.3
damping	Permanently set to 0.9 to reduce the numerical instability in superelevation calculation rad k and damping are used to calculate the change of superelevation angle when horizontal bend radius changes
conv. factor	Input factor to reduce numerical instability during iteration. It is recommended to be 1.0 initially. May be reduced to not less than 0.1 if

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Variables	Description
	large oscillations do not converge (endless iteration of the spreadsheet).
Tolerance for “Landing”	When the angle of incidence of the debris to the slope is very small at the landing point after a waterfall, the spreadsheet may endlessly iterate between two possible landing chainages. Inputting a small value such as 0.01 m to 0.03 m for the tolerance may help the program to choose one of the two possible chainages.
	OUTPUT
CH	Chainage
theta	Gradient of slope at entrance point of a section
thetau	Gradient of slope at exit point of a section
input width	Width of the debris trial
calc’d width	Calculated width based on hav
Base width	Base width of the debris trial
Side Angle	Side angle of the channel used for calculation
Input section area	Section area from Channel Properties Module
calc’d section area	Calculated section area based on hav
Input wet perimeter	Wet perimeter from Channel Properties Module
calc’d wet perimeter	Calculated wet perimeter based on calculated width
bu	Upstream base width
wu	Upstream width
deltau	Upstream side angle
hu	Maximum height of debris
vu	Initial velocity of the debris
mPD slope	Elevation of the Chainage point
mPD check	Bottom level of the debris
Debris location	Location of debris Ground Debris is flowing along the slope Air Debris is falling from a waterfall
mPD base	Bottom level of debris
mPD top	Top level of debris
v	Calculated velocity
v0 Hz	Horizontal component of the velocity
x cumul	Horizontal distance from starting Chainage
x	Horizontal distance from the changing point
t	Time required to travel each section of the slope
t cumul	Time required to travel from starting Chainage
plan area	
friction area	Average wet area to calculate friction force
Entrainment	Material enter or exit the debris trail +ve Entrainment -ve Deposition
Sf	Coefficient of friction (total-stress)

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Variables	Description
ksi	Turbulence factor
h av	Average height of debris
Q	Flowrate of debris
Qu	Entry flowrate
V past	Check on the volume of material
Orient	Flow direction (radians)
Easting	Easting of the Chainage point
Northing	Northing of the Chainage point

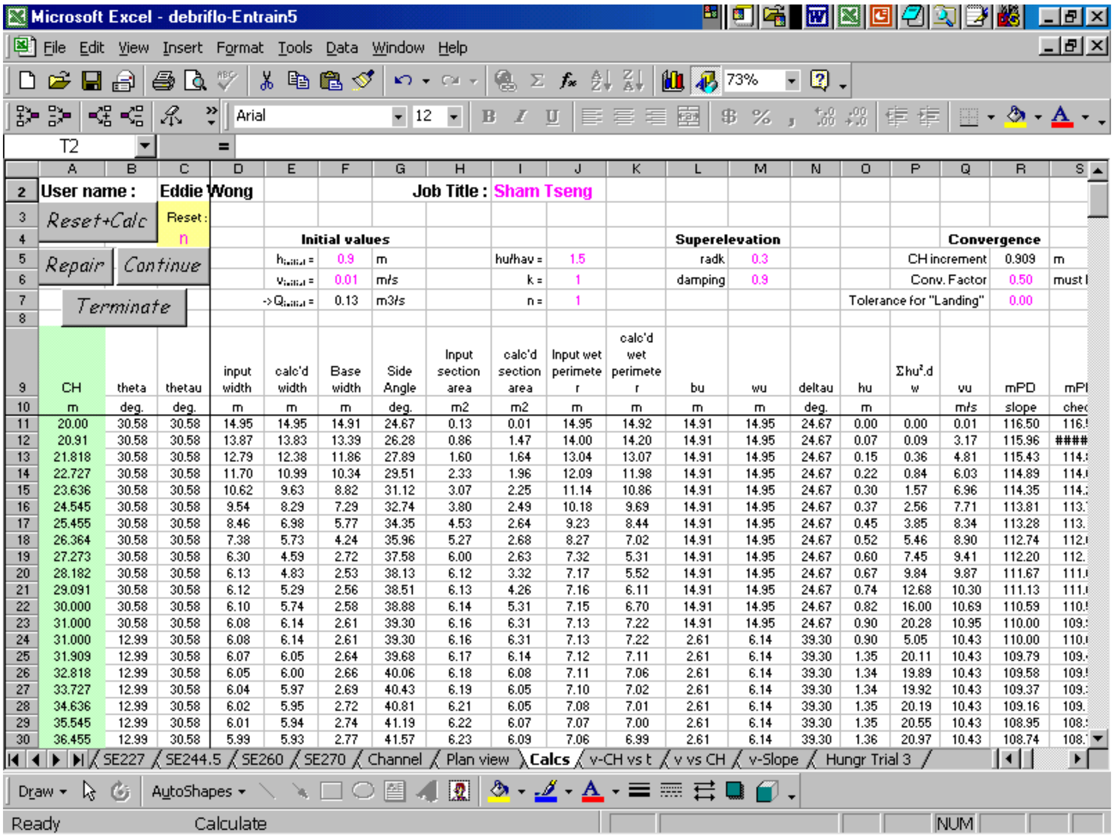

Figure 1.3.1 - Calculation Module

A.2 OUTPUT

To calculate the result, the [Reset + Calc] bottom in Calcs worksheet should be pressed. The numerical results are in Calcs worksheet.

Button	Description
[Reset +Calc]	Reset and start the calculation
[Repair]	To be used to repair numerical instabilities
[Continue]	Restart the calculation
[Terminate]	Does one single iteration to refresh all charts

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Plots of velocity vs time, chainage vs time and velocity vs slope profile will be graphed. The plan view of the channel will also be printed. Figures A.2.1 to A.2.5 are typical plots given by the spreadsheet.

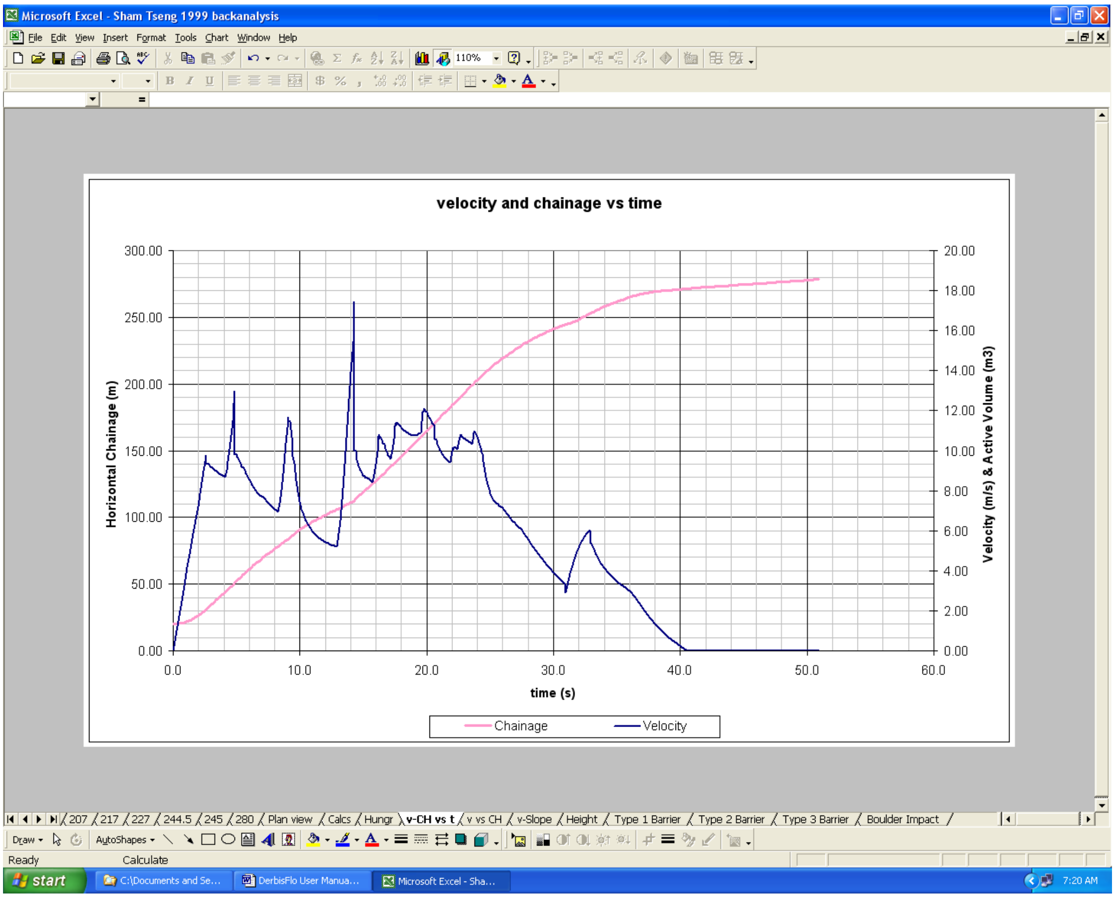

Figure A.2.1 - Velocity and Chainage vs Time

The blue line shows the velocity of the debris front against time and the magenta line indicates the traveling time of the debris front. The flatter part of the curve shows the stopping chainage of the debris. This agrees with the velocity-time curve. Figure A.2.1 shows the debris stopped at chainage 270 at 40 seconds.

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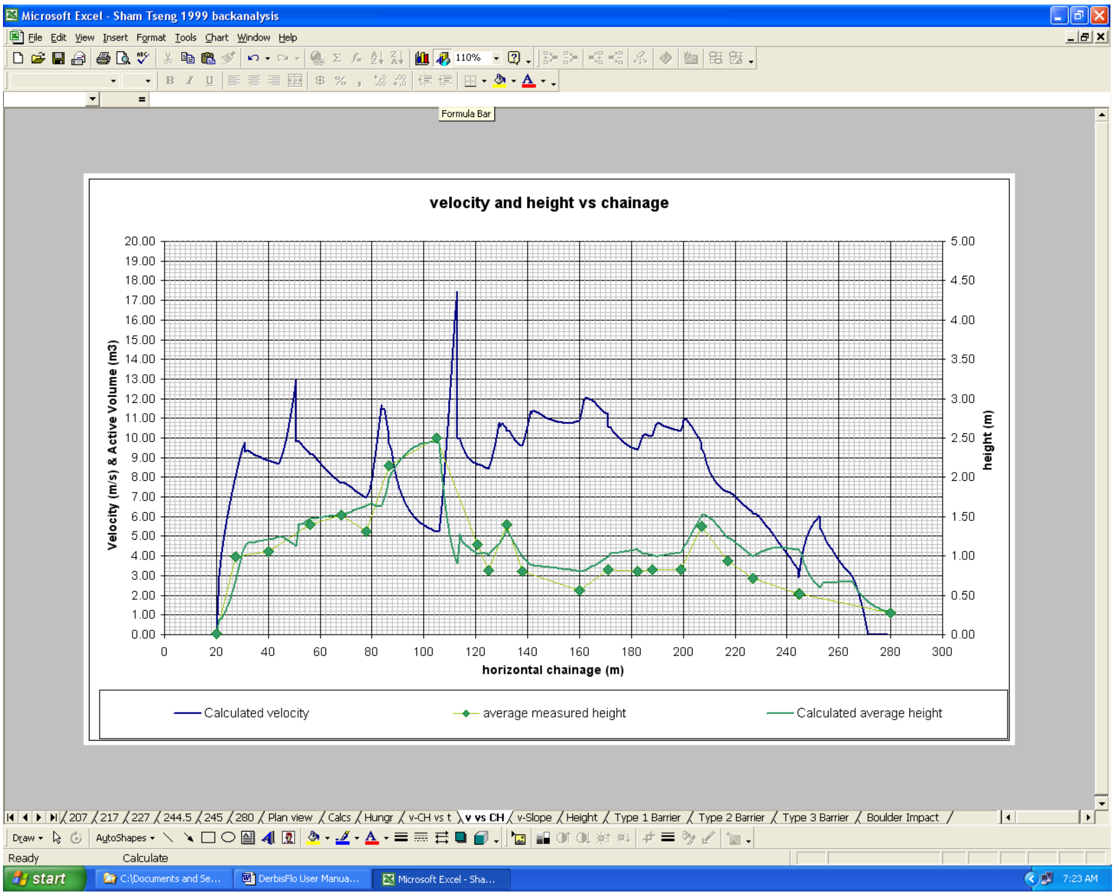

Figure A.2.2 - Velocity and height vs chainage

Figure A.2.2 shows the velocity profile and the profiles of the average measured height from the input data and the average calculated height. Checks of the velocity profile with field estimations of superelevation velocity provide a good check on the validity of any back-analysis. A further check on the validity of the results can be made by comparing the height profiles. The calculated heights must be close to the input heights.

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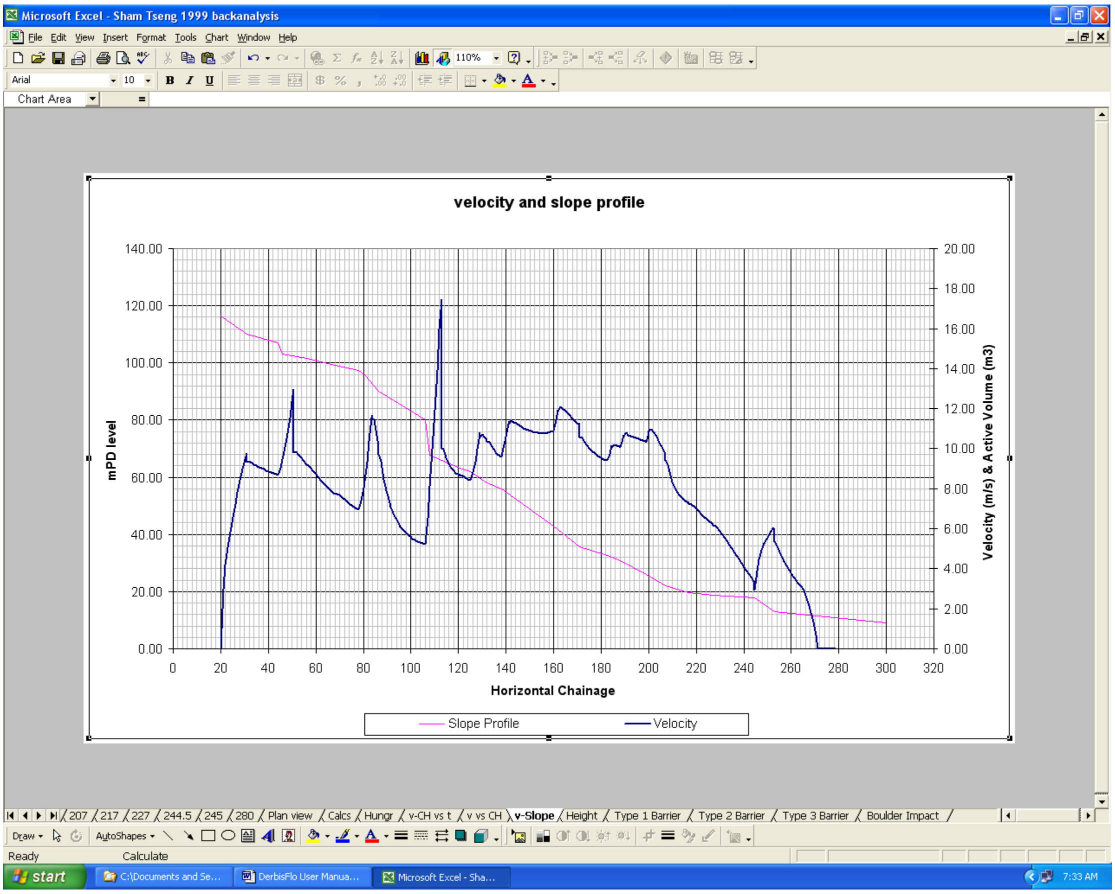

Figure A.2.3 - Velocity vs Slope Profile

This figure shows the relationship between the velocity and slope profile. The blue line is the velocity curve and magenta is the slope profile.

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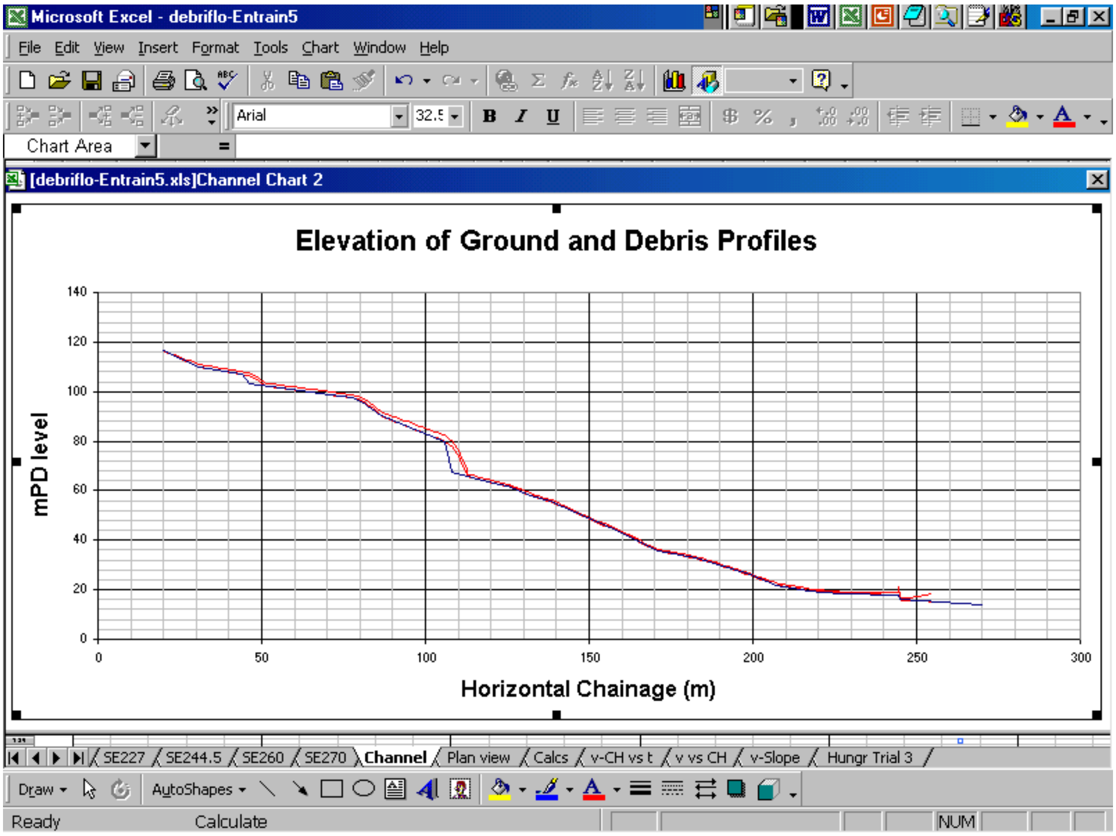

Figure A.2.4 - Debris Flow Profile

The red lines are the calculated top and bottom levels of the debris. The blue line is the slope profile.

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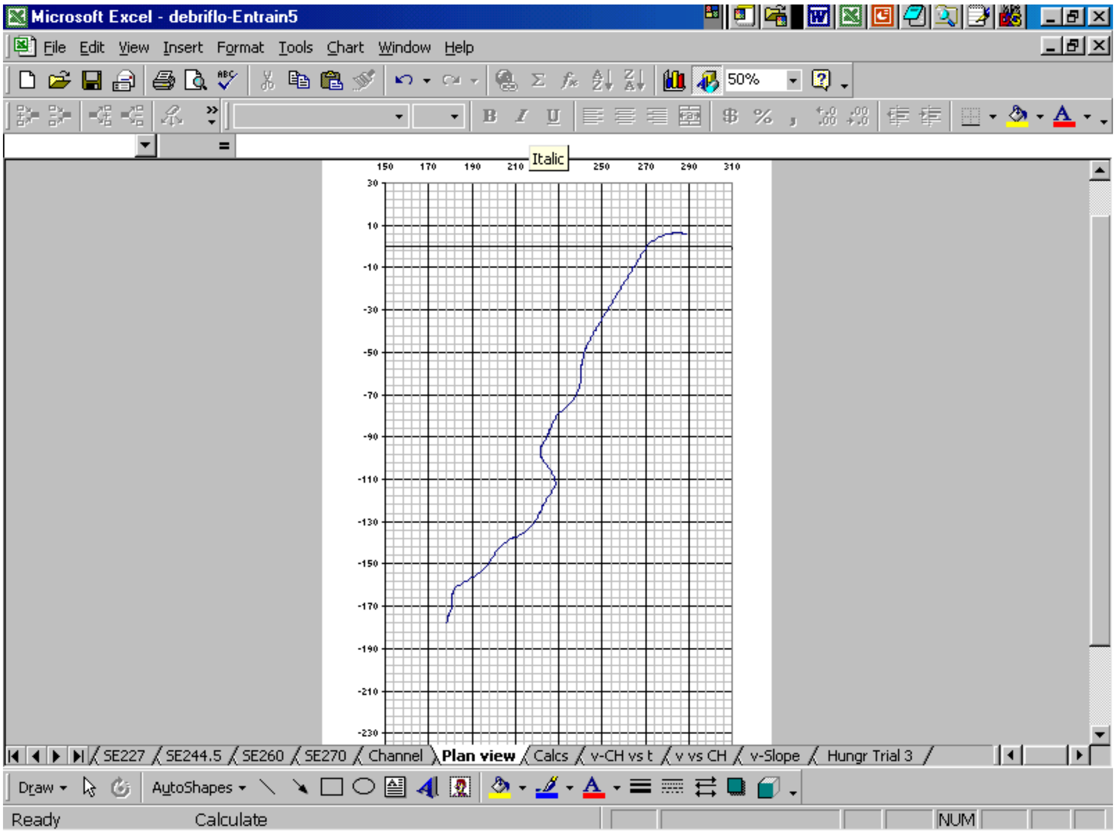

Figure A.2.5 - Plan View of Channel

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

GEOTECHNICAL MANUALS

Geotechnical Manual for Slopes, 2nd Edition (1984), 300 p. (English Version), (Reprinted, 2000).

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.

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Geoguide 1	Guide to Retaining Wall Design, 2nd Edition (1993), 258 p. (Reprinted, 2000).
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.

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Geospec 1	Model Specification for Prestressed Ground Anchors, 2nd Edition (1989), 164 p. (Reprinted, 1997).
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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. 1/96	Pile Design and Construction (1996), 348 p. (Reprinted, 2003).
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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

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