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Deliverable 1.9

Recommendations for run out models for use in landslide hazard and risk mapping

Work Package 1.6 - Identification of models best suited for QRA

Deliverable D1.9
Work Package Leader: FUNAB and AMRA

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SUMMARY

This deliverable aims to provide recommendations for users of run out models for susceptibility assessment in Quantitative Risk analysis. It is conceived as a link between model developers and practical users.

We are aware that there exist a large number of models of many types with different accuracy which is currently used for QRA. While some of them are still in a research stage, others have been thoroughly tested and used. This is the case of depth integrated models which present a good combination between computational cost and accuracy.

This Deliverable follows the work done in deliverable D1.7, where the different alternatives concerning mathematical, rheological and numerical models were analyzed.

We devote a first Section to review very succinctly the different categories of models, from the simplest empirical methods based on using the reach angle to the most complex 3D multiphase models incorporating coupling between the solid grains and the pore fluids. There, we state that this report will focus on rational models of continuum type, because they can be applied to a large variety of problems, and indeed, in the case of SPH models, they can simplify to the single block models.

We have left out the discrete element models, because so far, they can only be applied to rock avalanches –with limitations due to the maximum number of particles which can reasonably be used on today's computers. And we haven't included either the block models used to describe rock falls.

The main aim being thus to provide practical recommendations for users of depth integrated codes; we have focused on those aspects which in our opinion are more important.

The topics which we will address are the following:

1. **Recommendations concerning initial conditions**

2. **Recommendations concerning terrain representation**

3. **Recommendations concerning the choice of the rheological model and its parameters**

4. **Recommendations regarding basal erosion**

5. **Recommendations concerning the choice between Eulerian (Finite elements) or Lagrangian (SPH) numerical models**

The approach we have followed consists on present cases which illustrate the difficulties described. It has not been our aim to present a complete set of case studies providing all aspects dealt with in the analysis, but rather to emphasize the topic being addressed.

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CONTENTS

1. INTRODUCTION

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We are aware that there exist a large number of models of many types with different accuracy which is currently used for QRA. While some of them are still in a research stage, others have been thoughtfully tested and used. This is the case of depth integrated models which present a good combination between computational cost and accuracy.

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2. A SHORT STATE OF ART OF RUN OUT MODELS

2.1. INTRODUCTION

Run out models have been presented in several work packages and deliverables of SAFELAND, such as:

- **D1.7**, “Landslide run out: Review of analytical/empirical models for subaerial slides, submarine slides and snow avalanche. Numerical modelling. Software tools, material models, validation and benchmarking for selected case studies”,
- **D1.8**, “Guidelines: Recommended models of landslide triggering processes and run-out to be used in QRA” and
- **D2.4**, **“**Guidelines for landslide susceptibility, hazard and risk assessment and zoning”.

In all of them, especially in D1.7, they have been described and classified. The purpose of this Section is to provide an overview of the group of run out models which are currently used in the framework of QRA analysis.

In general, we can distinguish between **empirical** and **rational** models. Concerning the former group, they are used to estimate travel distances rather than to provide velocities. They may be based on geometrical relations between the slope and the landslide deposits (Hungr and Evans, 1998; Evans and Hungr, 1993; Corominas, 1996; Hunter and Fell, 2003; and Hungr et al., 2005) or on volume change-methods (Cannon, 1993; Fannin and Wise, 2001).

2.2. RATIONAL MODELS

Rational methods are based on the use of mathematical models of different degrees of complexity. They can be classified as follows:

<u>(a) Discrete models</u>. To be used in cases where the granularity of the landslide is important. The simplest case is that of a block, which falls on a slope. Its geometry can be modelled with precision or approximated by a simpler form. The model checks for impacts with the basal surface, applying it a suitable coefficient of restitution. As an example, we can mention the model proposed by Agliardi and Crosta (2003).

On the other extreme, discrete elements have been used to model rock avalanches. The avalanche is approximated by a set of particles of simple geometrical forms (spheres, circles) with ad hoc laws describing the contact forces. The number of material parameters is rather small (friction, sometimes an initial cohesion, and elastic properties of the contact). In many occasions, it is not feasible to reproduce all the blocks of the avalanche, which is approximated with a smaller number of blocks. The spheres (3D) or disks(2D) can be combined to form more complex shapes, and given granulometries can be generated. One main advantage of these methods is their ability to reproduce effects far beyond the reach of continuum models, such as inverse segregation. (Calvetti et al 2000). Discrete element models are suitable for the simulation of rock avalanches, but it is not recommended their use in other situations (flowslides, lahars, mudflows…) because of the rheology of the flowing materials.

(b) Continuum based models. They are based on continuum mechanics, and can include coupling of the mechanical behaviour with hydraulics and thermo mechanics. Here we can consider the following groups.

(b.1) 3D models based on mixture theory. The most complex model category involves all phases present in the flowing material, as solid particles, fluid and gas. Here relative movements can be large, and this group of models can be applied to the most general case. The model is based on the mixture theory. However, due to the great number of unknowns and equations, these models have not been used except when considering the mixture, which is correct for mudflows and rock avalanches. As the geometry is rather complex, no analytical solution exists and it is necessary to discretize the equations using a suitable numerical model, such as finite elements or SPH. As an example, we can mention the work of Quecedo et al 2004 who analyzed the waves generated in reservoirs by landslides. These models are very expensive in terms of computing time, but have to be used in situations where 3D effects are important, as in the case of waves generated by landslides or impact of the flowing material with structures and buildings.

(b.2) Velocity-pressure models (Biot-Zienkiewicz). In many occasions, the movement of pore fluids relative to the soil skeleton can be assumed to be small, and the model can be cast in terms of the velocity of the solid particles and the pore pressures of the interstitial fluids. This is the classical approach used in geotechnical engineering. Again, the resulting model is 3D, and the computational effort to solve is large. Material point models, SPH, and ALE methods, such as used by Sosio et al 2008 can be used, but their field of application is restricted to small-medium runout for practical reasons related to computational effort. One important point is that pore pressures can be fully described.

(b.3) Taking into account the geometry of most of fast propagating landslides, it is possible to use a depth integration approximation. The equations reduce from 3D to 2D, as all variables depend on (x,y), the z information being lost in the integration procedure. This method has been classically used in hydraulics and coastal engineering to describe flow in channels, long waves, tides, etc. In the context of landslide analysis, they were introduced by Savage and Hutter (1991). Since then, they have been widely used by engineers and earth scientists. It is possible too to include information of the basal pore pressure, as done by Iverson et al 2001 and Pastor et al 2008. It is important to notice that even if the results obtained by these models can be plotted in 3D, giving the sensation that is a full 3D simulation, the model is 2D. Moreover, pressures and forces over structures are hydrostatic. Therefore, if this information is needed, it is necessary to couple the 2D depth integrated models with the full 3D model in the proximity of the obstacle. Depth integrated models provide an excellent compromise between computer time and accuracy. They have been used to describe rock avalanches, lahars, mudflows, debris flows and flowslides. Among the many different models it is worth mentioning those of MacDougall and Hungr (2004)

(b.4) Depth integrated models can be still further simplified, as in the case of the so called infinite landslide approaches. Indeed, the block analysis performed in many cases does consist on a succession of infinite landslides evolving over a variable topography. Here, pore pressure dissipation can be included, as done by Hutchinson (1986). Block models have been improved in order to consider the kinematics of several blocks interacting between them, including thermodynamical effects (Pinyol and Alonso 2010, Alonso and Pinyol 2010),

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3. PRACTICAL GUIDELINES

3.1. INTRODUCTION

As it has been explained in the preceding Section, where a concise state of art has been presented, there exist a great variety of models which can be applied for Quantitative Risk Analysis. Most of models provide the susceptibility of a given area to be invaded by a landslide.

This Deliverable complements D1.7 where models have been discussed in depth. There, it was pointed out the excellent compromise between accuracy and time of computation provided by depth integrated models. Here we will concentrate in this category of models, providing additional information regarding how to use them.

Depth integrated models are simple to use in comparison with other models used in geomechanics, such as coupled finite elements or differences methods incorporating complex constitutive equations. Of course, there exist numerous simplified models in geotechnical engineering too, but in some cases they simply cannot be used. As an example we can mention the case of liquefaction of a geostructure induced by an earthquake. Limit analysis methods cannot provide useful answers because (i) failure is not of localized type, and (ii) failure of the material takes place inside Mohr-Coulomb surface. The analyst has to consider the suitability of the type of finite element used, whether it is sensible to mesh size or alignment effects or not, etc.

Depth integrated models are much simpler to use. The description of the material behaviour does not require complex constitutive models incorporating more than 15 parameters used in geotechnical engineering, and in most occasions, 2 or 3 rheological parameters are enough.

However, their use presents some difficulties. The purpose of this Section is to describe them, providing some advise which may be useful to guide scientists using run out models of depth integrated type.

We have classified the problems into the following groups:

(i) Problems related to initial conditions of the mobilized geomaterials, where we can consider the determination of the initial mass of the landsliding material, the resolution we need to describe it, and, in some cases, the pore pressure (at least in the basal surface). Even though this effect is not taken into account by most of users, it can be crucial in the case of flowslides. Indeed, not including it may result on predicting much smaller run out distances –unless the friction angle of the material used in the analysis is altered-

(ii) Description of the topography. It is not the aim of this deliverable –devoted only to depth integrated models- to discuss how topographers obtain digital terrain models, nor the refinement techniques which can be used to provide higher resolution where needed or smaller in other terrain zones. In many occasions the available data has a precision with an associated error which cannot be decreased. The error can be estimated as the product of the second derivative of the surface describing the terrain times the square of the cell size. From a given map, precision can be improved in particular zones by new surveys. Here we will only present the problem of having a cell size larger than required, just to show that cell sizes larger than a critical value will not be able to provide accurate results. The main problem is that this size is difficult to estimate a priori. Rules of thumb such as using 10 points per cross section of channels can help in some cases.

We will pay special attention to what we have called micro-topography, i.e., those details of the terrain which escape the description of most DTMs. This is the case of buildings, curbs, roads, etc, which can divert the flow. Proper modelling of them requires the inclusion of especial elements in the analysis.

(iii) Problems related to the behaviour of the fluidized material. After a general consideration of the influence of the rheological model used, we will describe which model is, in general, more suitable for most common cases of landslides with large run out distances.

(iii) Problems related to interaction between propagating material and the basal surface, i.e., erosion.

(iv) Finally, we will consider very briefly the relative advantages of using structured or unstructured Eulerian meshes (finite elements and differences) and meshless methods such as SPH.

3.2. MODELLING OF INITIAL CONDITIONS

Initial conditions play a crucial role in the results obtained with depth integrated models. This why it is so important to obtain the landsliding mass from the results obtained with a model describing most salient aspects of the triggering process, and this is why special attention has been paid by other SAFELAND teams to fully characterize triggering processes.

The aspects we will consider here will be illustrated by simple examples.

➢ *Sensitivity to initial mass*

In order to present the sensitivity of the results to the initial mass, we will use an example – the Tate's Cairn debris flow of august 2005 – which description has been provided by Hong Kong Geotechnical Office (HKGO). The modelling work which will be presented here consists on (i) the analysis of a past event with a landsliding mass which was given by the Geotechnical Office, and (ii) once the model has been calibrated, make a prediction of the run out of a larger mass. The description of the past debris flow can be found in the report of Maunsell Geotechnical Services Ltd. (2007). In this example we have used the DTM provided by HKGO.

Concerning the source area, it was 36 m long and 22 m wide, with a maximum depth of 5.5 m approximately. The source material consisted of (i) a layer of boulder rich colluvium (or young colluvium) made of slighty sandy silty clay, about 2.9 m thick, and (ii) an old colluvium made of sandy clayey silt.

Figure 3.2.1, taken from Maunsell's report gives an overview of the 2005 debris flow event.

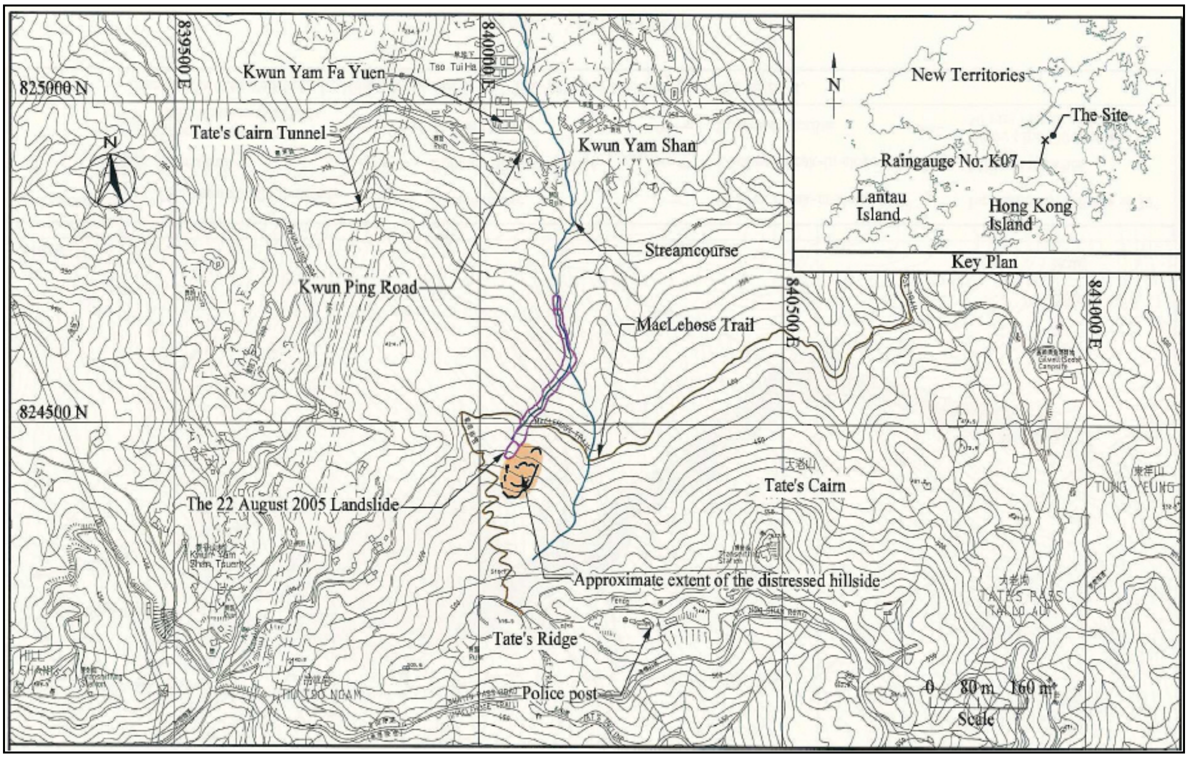

Fig. 3.2.1. Tate's Cairn debris flow in 2005 (after Maunsell 2007)

Concerning the rheological model used, a frictional fluid of Voellmy type has been chosen. The main point here is not the particular model being used, but the effect of the initial mass on the run out.

We have used a turbulence constant $\xi = 500\, m/s^2$ and $\tan\phi = 0.3$. We have chosen Hungr's erosion model, with an erosion constant of $0.0006\, m^{-1}$. These values were obtained by back analysis of the past event. Once this was done, a second simulation was performed with a larger mass, keeping the same rheological parameters used in the previous analysis.

The results are given in figure 3.2.2, where we have depicted the evolution of the debris flow along time together with a comparison between the computed results and the field observations.

If a similar analysis is performed using a Bingham fluid model, the difference between the results obtained with two different sliding masses is much larger. Figure 3.2.3 provides a comparison between both cases. The rheological parameters were: yield stress 2860 Pa and viscosity coefficient 44.8 $Pa.s$.

Users of run out models should be aware of the fact that the results obtained in the simulations: will present a run out depending on: (i) the initial mass, and (ii) the rheology selected.

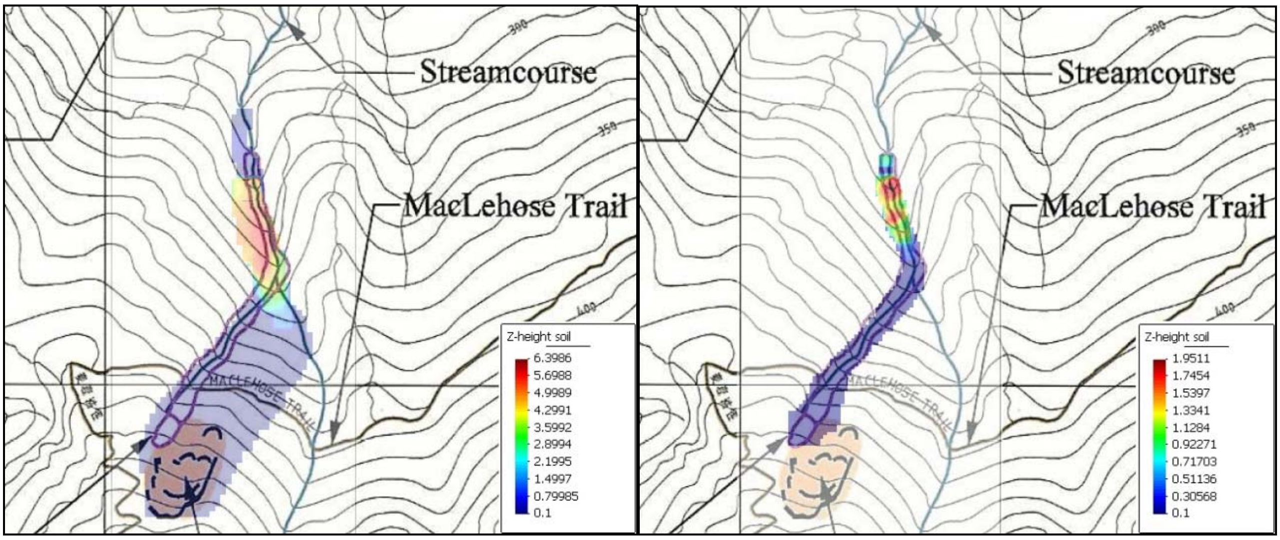

Fig. 3.2.2. Model predictions for Tate's Cairn: comparison between the results obtained with a frictional fluid (Voellmy) for the two initial masses (i) left: 1200 m^3 (ii) right 10000 m^3

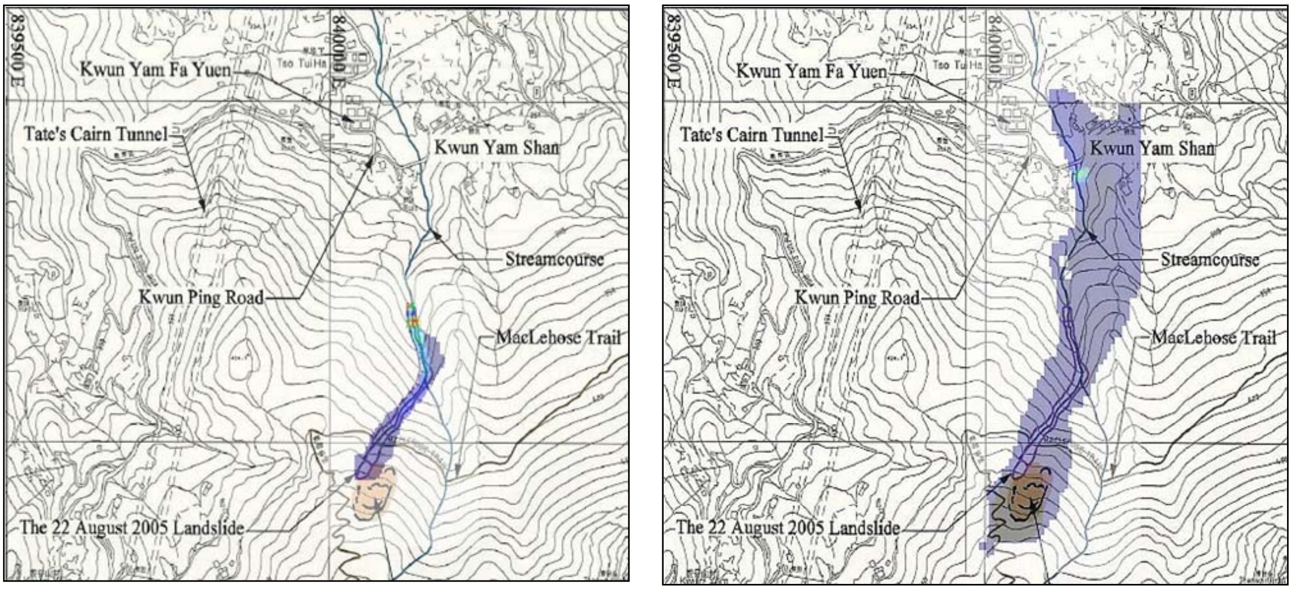

Fig. 3.2.3. Model predictions for Tate's Cairn: comparison between the results obtained with a cohesive fluid model (Bingham) for the two initial masses (i) left: 1200 m^3 (ii) right 10000 m^3

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➢ *Sensitivity to number of points describing the landsliding mass*

The landsliding mass is discretized using a series of nodes in most numerical methods. The accuracy of the simulation depends very much on the number of nodes used by the analyst. In some cases, it is possible to perform simplified analyses with a reduced number of nodes. Indeed, in the limit, using a single material point in SPH is equivalent to the block model.

The best method to assess the validity of the node spacing is to perform a convergence study, where the landsliding mass is discretized with an increasing number of points until the results obtained in the two last analyses are very close to each other, i.e., convergence has been achieved.

To illustrate the method, we will present a case where such analysis was performed. It is a simulation of the 2001 Popocatépl lahar in Mexico (Haddad et al 2010).

The authors discretized the mass using the following set of material points spacing: (10 m, 5 m, 3.14 m and 2.5 m). Figure 3.2.4 shows the four cases considered in the analysis.

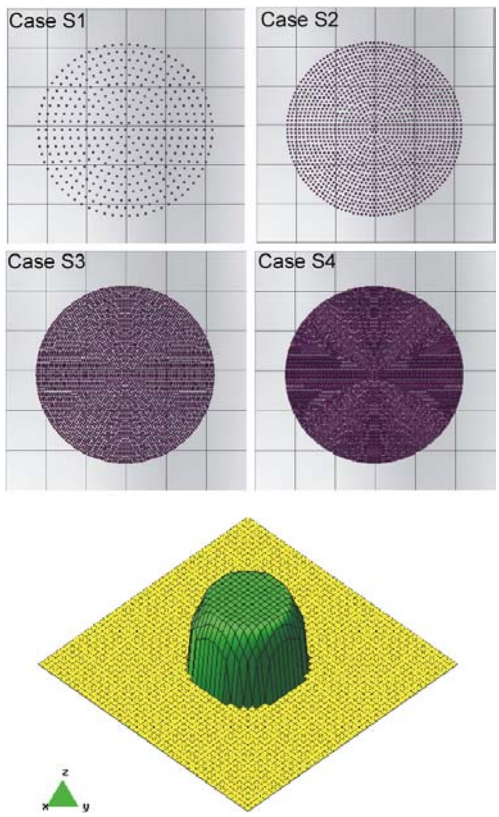

Fig. 3.2.4. SPH representations of the landsliding mass used in the 2001 Popocatépl lahar (Haddad et al., 2010)

The comparison of the results has been done using the distance-time curve for a material point in the mass, and is given in Figure 3.2.5. The orders of magnitude of the run out distance and the time are 7 km, and 1400 s. respectively.

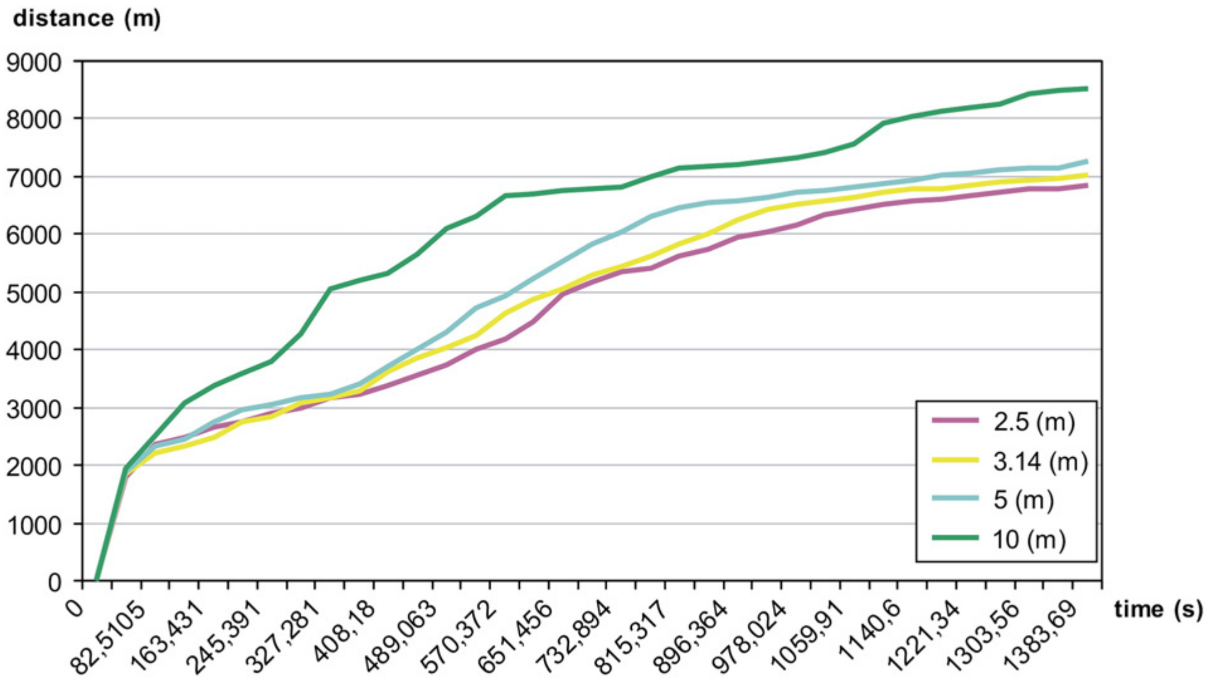

Fig. 3.2.5. Simulated runout for the representative particle corresponding toSPH initial particle spacings of 10, 5, 3.14 and 2.5 m (Haddad et al., 2010)

Convergence is achieved with the 2.5 m spacing. The results of the analyses showed that using smaller number of material points affected much more the velocity than the flow path, and therefore, smaller number of material points could be used for providing estimates to be refined at later stages.

In conclusion, we can recommend users of run out models to perform a convergence analysis with regard to the number of material points or nodes discretizing the landsliding mass whenever possible at the beginning of a series of numerical simulations, whenever possible.

References

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➢ *Basal pore pressure*

In some cases, the triggering process results on the generation of pore pressures in the soil mass, which are responsible for its failure. This happens especially in very loose metaestable materials, as those found on mine dumps and other artificial slopes. The pore pressure results in: (i) a smaller apparent friction angle, and (ii) its variation in time. This effect was described by Hutchinson (1986) in its paper describing the sliding consolidation model.

Few depth integrated codes incorporate this effect currently.

In the case that the application of a previous finite element model has provided information both on the mobilized mass and the initial distribution of pore pressure, it is possible to incorporate the latter on the depth integrated model. Otherwise, the analyst has to think of a reasonable one. When performing back analysis of known events, it is possible to use the information regarding geotechnical properties such as vertical consolidation coefficients, permeability, oedometric modulus, etc. This information can be incorporated in the depth integrated model, and will provide the dissipation of basal pore pressures. However, the initial distribution of the pore pressure has to be obtained by back analysis.

The reader is referred to the papers by Iverson et al 2010 and Pastor et al 2008 where the method is explained.

In conclusion, we can recommend the users of run out models to analyze the available geotechnical information to verify if the triggering of the landslide is a consequence or results on the development of relevant pore pressures. Should this be the case, the model to be applied has to (i) estimate the initial distribution of pore pressure, and (ii) model the propagation incorporating pore pressures and their dissipation.

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3.3. TERRAIN REPRESENTATION

Once the landslide has been triggered, it will propagate downhill. Its speed, path, and run out distance will depend very much on the terrain topography. Therefore, it is important to have a suitable representation of it which can be incorporated in the model.

We will not address here the problem of the techniques used in DTM generation, but we will concentrate on the data required for depth integrated models.

The terrain will be represented by a set of points forming either a structured or a non structured mesh. The latter is often used in finite element analysis, where there is a mesh of 2D elements with nodes where heights of the terrain, together with gradients and curvatures are stored. In the case of SPH methods, there are two meshes, (i) a computational mesh of SPH material points, and (ii) a topographic mesh describing the terrain.

In the case of finite elements, suitable mesh refinement techniques provide iso-error meshes for which the error when interpolating the height of any point of the domain is constant over the mesh.

For SPH models, structured topographic meshes are more suitable, because finding to which cell belongs a given point is immediate.

There is, therefore, a first indicator of the precision of the mesh, which is the product of the second order derivative of the basal surface height by the square of the mesh size, but this is not enough. Much research is still needed here.

As a rule of thumb, we can suggest that at least 10 points are used to discretized canyons and gullies channelling the flow.

We will consider here two aspects: (i) how the cell size may influence the results obtained in depth integrated models, and (ii) how micro-topographical features have to be properly accounted in the analysis.

➢ *Sensitivity to DTM grid spacing*

We will present here the case of the 2000 Tsing Shan debris flow, which happened in Hong Kong on the 14th of April. This analysis is based on the data provided by Hong Kong Geotechnical Office, and the report by King (2001).

This debris flow took place following rains which triggered more than 50 landslides in the area. The accumulated rainfall was 160 mm. The terrain was vegetated, and consisted of colluvial boulders. One important feature of this event is the strong erosion which made the initial mass to increase from 150 to 1600 cubic meters. Figure 3.3.1, taken from King, provides two general views of the debris flow. One important aspect is the bifurcation of the flow which can be observed in the pictures.

Fig. 3.3.1. General view of the 2000 Tsing Shan debris flow (King 2001)

The most salient feature of the debris flow –which can be seen in the picture- is the bifurcation into two branches. The simulation has been performed with a frictional model incorporating a consistent viscosity which has been presented in deliverable D1.7. The fluidized soil is considered to be a dilatant viscoplastic frictional fluid with a basal friction given by the law

$$\tau_b = \sigma_v^{'} \tan\phi + \frac{25}{4}\mu_{CF}\frac{\bar{v}^{2}}{h^2}$$

where $\sigma_v^{'}$ is the effective vertical stress, ϕ the effective friction angle, ρ the mixture density, $\bar{v}$ the depth integrated velocity, h the depth of the flow, and μ_{CF} a non Newtonian viscosity coefficient with units $Pa.s^2$. In the present simulation we have chosen a friction angle with $\tan\phi = 0.18$ and $\mu_{CF} = 0.00133\,Pa.s^2$.

Another salient feature of this particular debris flow was the strong erosion, which made the initial mass to increase from 150 to 1600 m^3.

Modelling of erosion will be addressed at another section of this text, and several alternatives will be shown there. In this case, we have chosen a simple law proposed by Hungr, using an erosion coefficient of $0.0082\ m^{-1}$.

Concerning the DTM, we used a first grid spacing of 5 m.

The computed path depicted in Fig. 3.3.2 (a) and (b) shows the branching. There, it can be seen that the deepest deposit was formed at the end of the lower south branch, with a maximum depth of 1.8 m. Regarding velocities, there is no available information.

The model predicts a time of propagation close to 120 s. Considering that the runout was 900 m in the lowest branch, the average velocity is close to 30 km/h. The report provides a total mass deposited in the south branch of 500 m^3, while the computation provides a value of 525 m^3. Concerning the total volume of eroded soil, the report estimates it as 1600 m^3, while the computations provide a value of 1550 m^3.

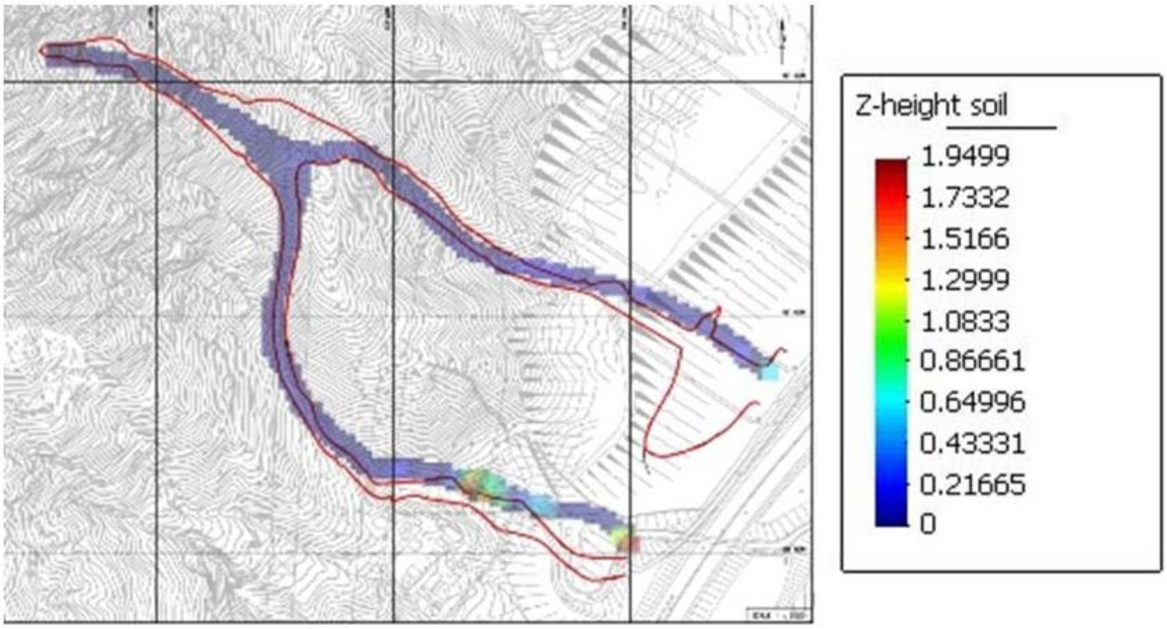

Fig. 3.3.2.(a). Shan debris flow: Model predictions vs. field observations with a DTM grid spacing of 5 m

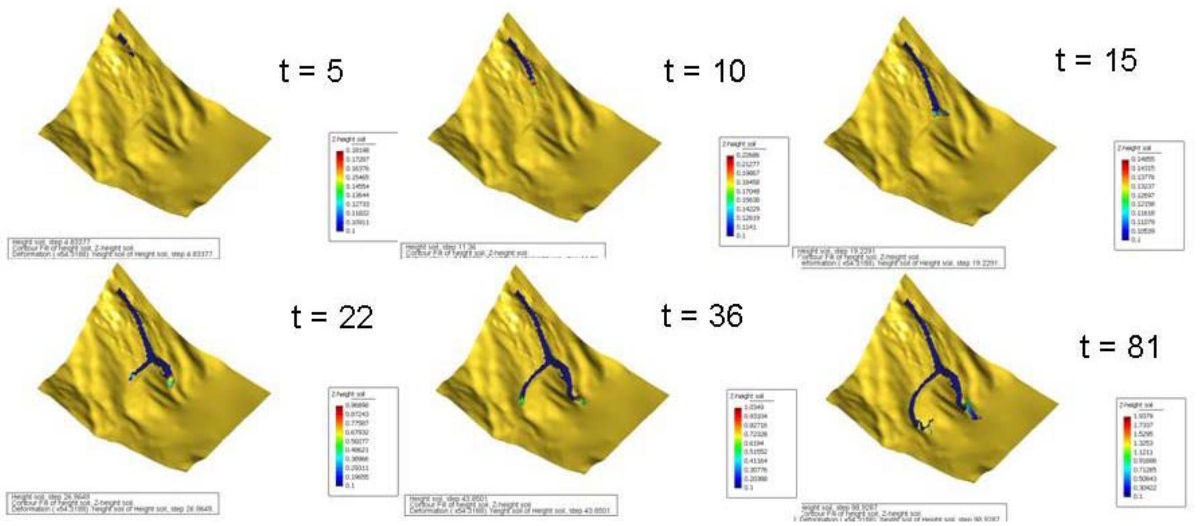

Fig. 3.3.2.(b). Shan debris flow: Model predictions at different times with a DTM grid spacing of 5 m

In order to illustrate the effect of the DTM size, we have run this case keeping all parameters the same but varying the DTM grid spacing, which now is 10 m.

The results are given in Fig. 3.3.3, where we can see how the model predicts only one branch, instead of the two branches observed in the reality.

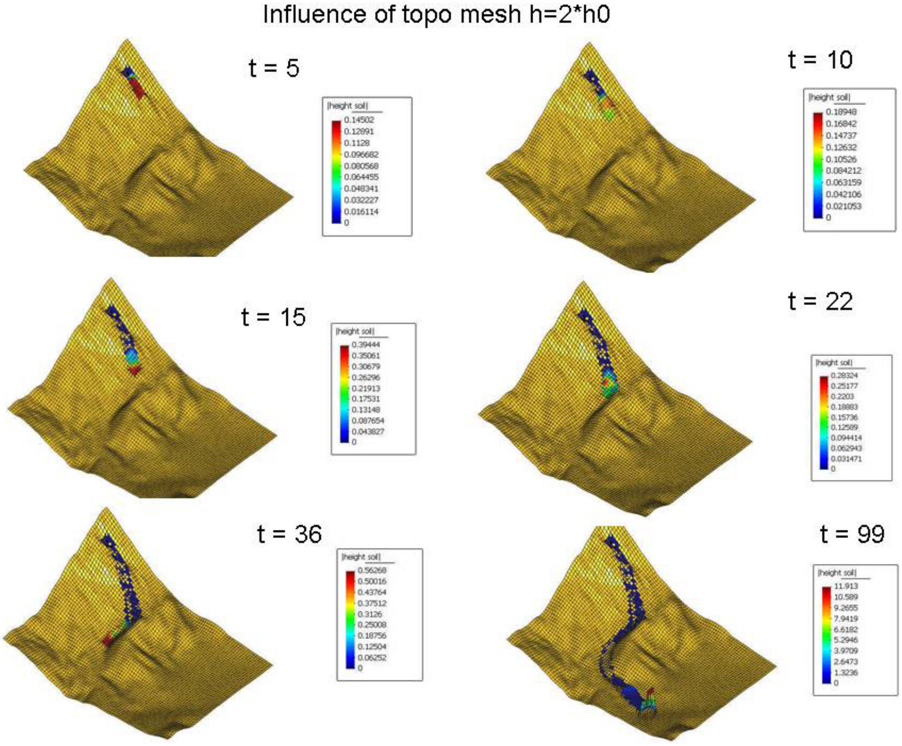

Fig. 3.3.3. Tsing Shan debris flow: Unrealistic Model predictions vs. field observations with a DTM grid spacing of 10 m

In conclusion, users of run out models are strongly recommended to make sure that the precision of the DTM being used in the analysis is enough. While error indicators are available for given DTMs, they have not been used by practitioners, and are not included in depth integrated codes. There is, therefore, a first indicator of the precision of the mesh, which is the product of the second order derivative of the basal surface height by the square of the mesh size, but this is not enough. Much research is still needed here. As a rule of thumb, we can suggest that at least 10 points are used to discretized canyons and gullies channelling the flow.

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➢ *Sensitivity to micro topographic features*

In addition of the DTM cell size, there are elements with characteristic sizes smaller than DTM grid spacing which can affect the propagation path. This is the case of buildings, curbs, roads, etc., which can divert the flow. Proper modelling of them requires the inclusion of especial elements in the analysis.

Just to show the effect of such elements in the flow, we will present here an interesting case for which there is information kindly provided by the Hong Kong Geotechnical Office, a debris flow which occurred in Lo Wai in August 2005. This debris flow originated from water overflowing from a catch water which was blocked by a landslide during a rainstorm.

The information used to elaborate the model was (i) the report prepared by Maunsell Geotechnical Services (MGS) 2005, together with the digital terrain models provided by Hong Kong Geotechnical Engineering Office.

For the sake of completeness, we include here a figure from the MGS report which provides a very good general description of the scenario. Figure 3.3.4 provides an aerial view where the catch water has been sketched, together with the drainage lines followed by the overflowing water.

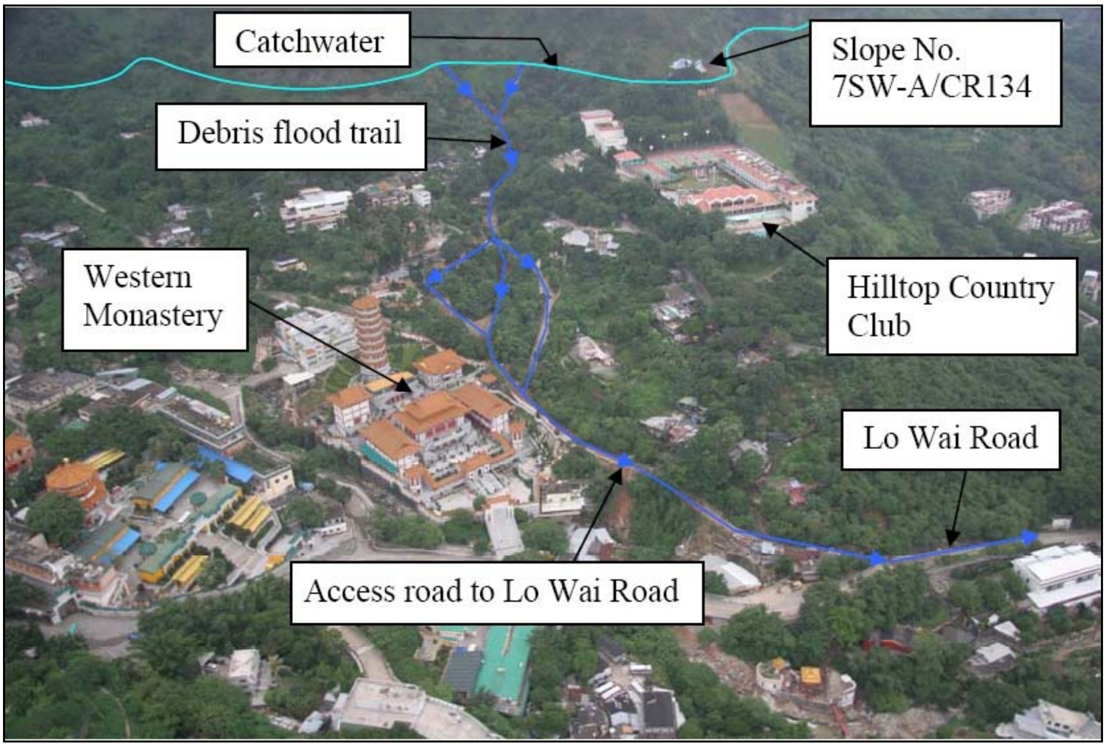

Fig. 3.3.4. General view of Lo Wai debris flow (MGS 2005)

One of the main difficulties found in the analysis has been to implement in the SPH model developed by the authors an algorithm able to inject nodes in a suitable manner, together with the knowledge of the input hydrograph. To obtain it in an accurate manner, it is necessary to model a section of the channel with the blocking landslide. In this way, it would have been possible to reproduce the upstream moving wave and its arrival to both weirs. By applying a suitable weir formula, we would have been in position to obtain the hydrograph.

We have therefore done the approximation of modelling only the northern weir, as both of them join close to the catch water channel. Of course, there will be errors in the drainage line of the southern channel. The second approximation has consisted in using a hydrograph of constant height of 0.15 m during 300 s, which is the time taken for the flow to travel the computational domain.

The grid size used in the DTM has been 5m.

A first analysis was performed just by using the topographic information provided in the DTM. Concerning the rheology, it was assumed that the debris flow consisted on pure water evolving into a Bingham fluid as it was eroding the channel. Details have been given in Pastor et al. 2007.

We have plotted both observations and the computed results in Figure 3.3.5, where we can see how the flow follows a path different from that observed.

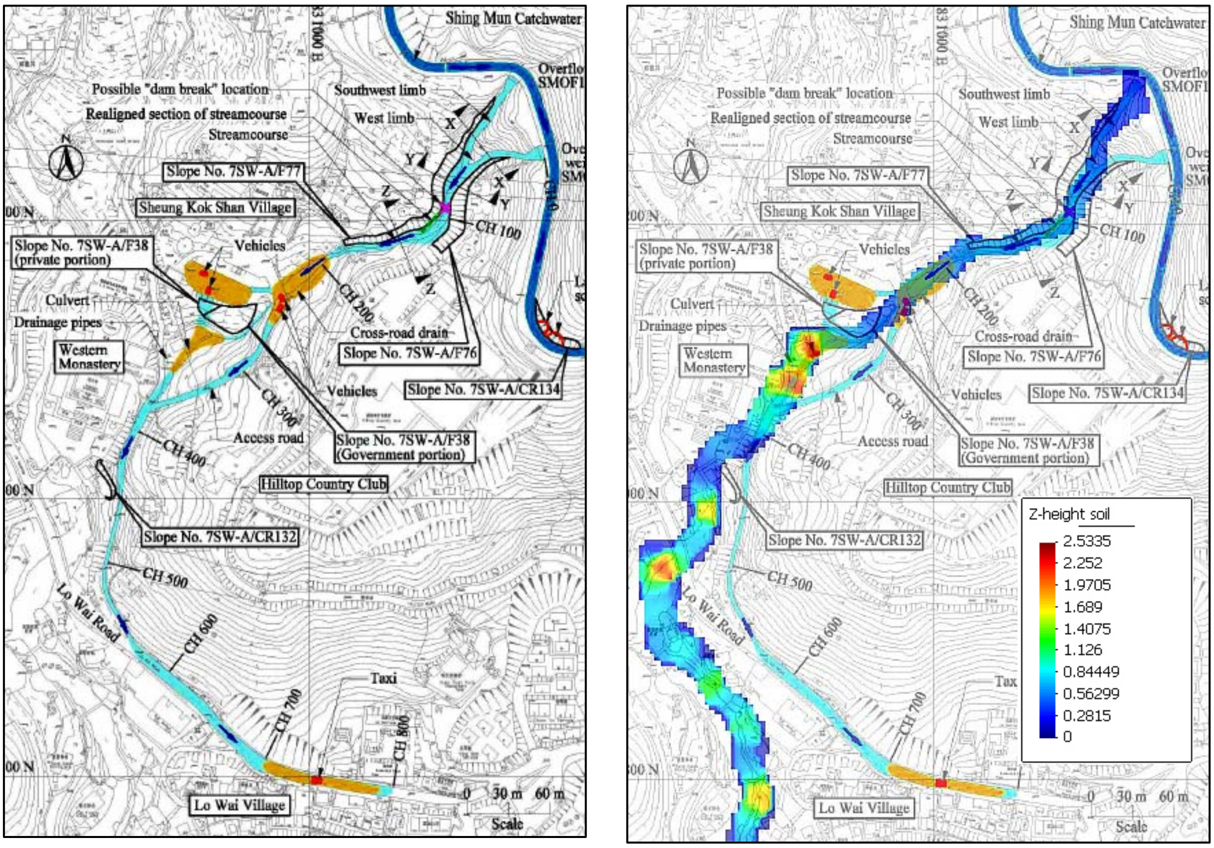

Fig. 3.3.5. Lo Wai debris flow. Computation without channeling barriers: Observation vs. Computed results

The reason for this disagreement is the existence of elements with characteristic sizes smaller than DTM grid spacing which such as buildings, curbs, roads, etc, which can divert the flow. In order to take them into account, we have included special barriers, made of a series of nodes which interact with those of the flowing material whenever the distance between them is smaller than a given tolerance which can be chosen as half the topographic grid size.

Figure 3.3.6 shows the location of the computational barriers and the new computed results are shown in Fig.3.3.7.

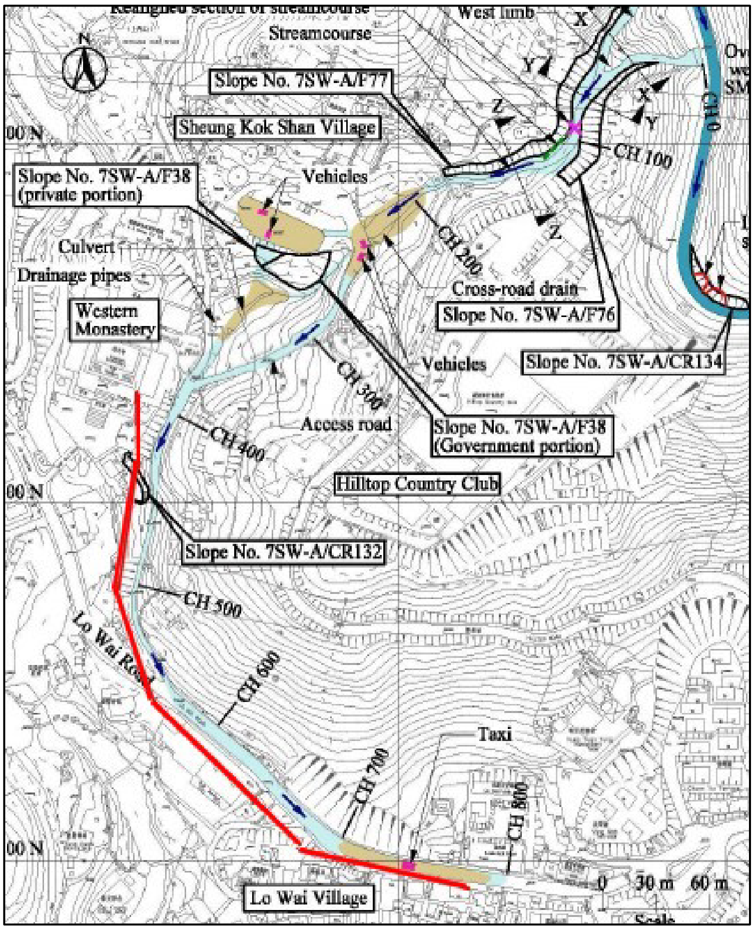

Fig. 3.3.6. Lo Wai debris flow. Computation Location of the computational channelling barriers

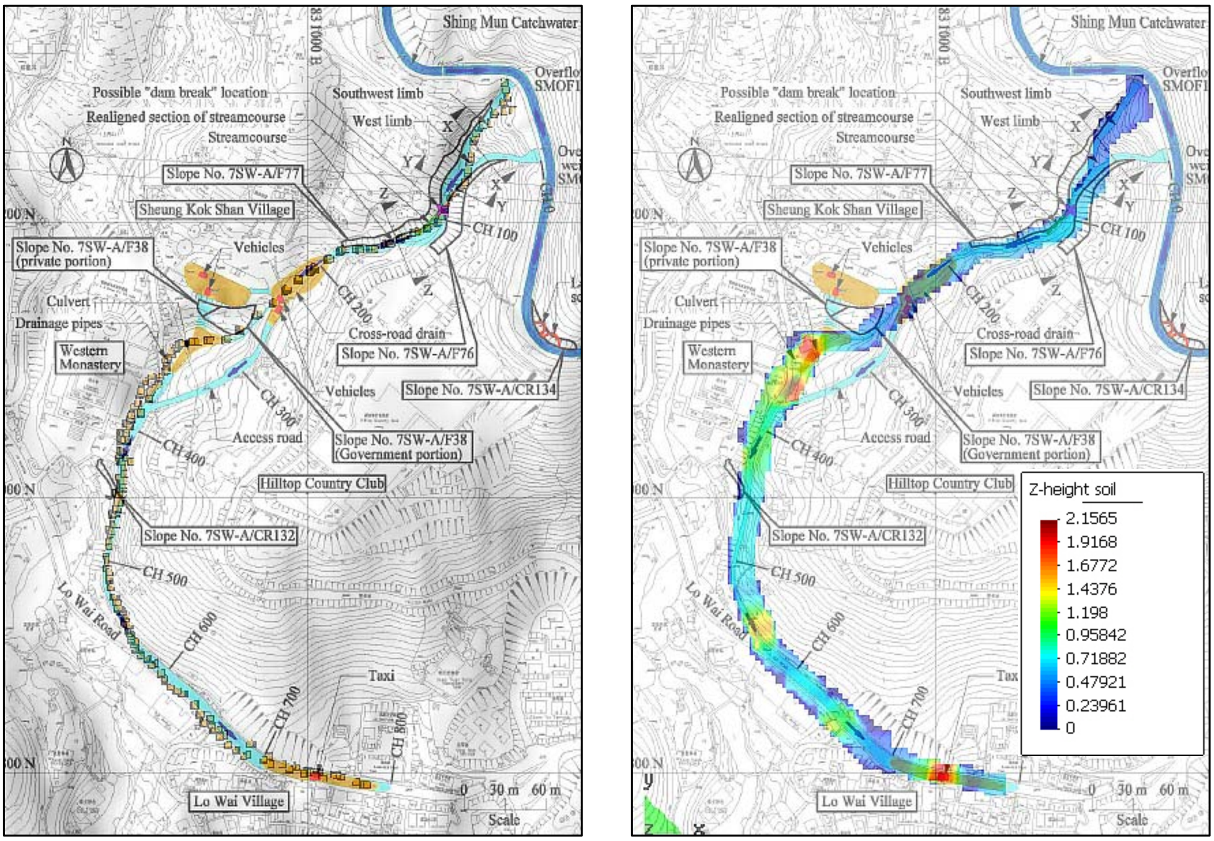

Fig. 3.3.7. Lo Wai debris flow. Computation with channeling barriers: Observation vs. Computed results

In conclusion, we can say that users of run out models should consider the possibility of including barriers whenever there exist in the path elements such as houses, roads, curbs, embankments, etc., which cannot be properly described by the DTM used.

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3.4. MATERIAL BEHAVIOUR: RHEOLOGY

➢ *Introductory remarks: choosing the rheological model*

One of the main difficulties encountered when modelling run out of landslides is the selection of the rheological model and its material parameters. In some cases it will be possible to obtain the properties in a rheometer, as in the case of muddy materials without large granular particles. Indeed, the maximum particle size which can be used in a rheometer can be estimated as 1/20 to 1/50 of its width, as happens in the classical triaxial tests too. In the case of rock avalanches, it is difficult –if not impossible- to use rheometers, and rheological parameters have to be estimated in a different way.

A possible approach consists of (i) finding similar landslides which have already occurred, (ii) determining by back analysis the material parameters, and (iii) extrapolating them to the case being analyzed. This is correct whenever the rheological model is of the correct type. By this we mean that the model we have decided to use is physically sound. It is not logical, for instance, to use cohesive fluid models to rock dry rock avalanches. In some cases, the situation is not that clear, and scientists trying to model a past event have to decide which model to use.

We will illustrate the situation with two examples. First, we will consider the case of Aberfan, where on the 21st of October 1966 a flowslide developed at a tip of loose colliery waste. It propagated downhill and into the village of Aberfan, causing 144 deaths. The material was loose coalmine waste, which was deposited by end tipping. The height of the tip was 67 m from the toe, and the natural slope of natural terrain was 128. The failure mechanism and geotechnical properties have been described by Bishop (1973) where the interested reader can find a detailed description.

One of the main difficulties when modelling the Aberfan flowslide is the role of pore pressures. The coal debris was a material composed of solid, fluid and gas phases, with a strong interaction between them. The role of pore pressures was of paramount importance when the flowslide was triggered, and during propagation they played an important role too. Therefore, from geotechnical considerations, this flowslide has to be modelled using a frictional fluid with coupling of pore pressures which can dissipate during the propagation, as done by Pastor et al. (2001).

However, Aberfan flowslide has been modelled assuming a Bingham fluid rheological law for the debris. For instance, Jeyapalan et al. (1983) and Jin & Fread (1997) obtained results that fitted the observations well choosing a yield stress $\tau_y = 4794\ Pa$, and a viscosity $\mu = 958\, Pa.s$. Even if the results were good, it is possible to argue that waste coal was not fully saturated, and the material was frictional.

The problem does not comes at the backwards prediction stage, but when trying to extrapolate the results obtained in them to another situation, either on the same location but with a larger initial mass, or on a different location with a different topography.

To illustrate it we will consider the case of Tate's Cairn debris flow which has been considered in Section 2 of this report. There, we have seen how run out was influenced by the size of the landsliding mass. Moreover, this influence was stronger in the case of Bingham fluids.

The example was part of a benchmarking activity proposed by Hong Kong Geotechnical Office, (Ko and Li 2008). Participants were asked to perform (i) the back analysis of a past event, obtaining the rheological parameters which fitted better the observations, and (ii) use them to predict a debris flow with an initial mass 10 times larger.

The authors have used two different models: (i) a frictional fluid, and (ii) a Bingham fluid model. In both cases, the agreement between simulations and observations was satisfactory for the backwards case. However, the predictions of both models for the new case with a much larger mass were absolutely different, with a much larger run out distance predicted for the Bingham fluid case. (see figures 3.2.2 and 3.2.3).

The question is how to choose a rheological model, and the answer is not easy if there is not available data.

The following recommendations can be used:

- **Rock avalanches will have to be modelled with frictional fluid like models.**
- **Mudflows are recommended to be modelled with cohesive fluid models such as Bingham, for instance.**
- **Flowslides are usually generated in very loose metastable materials, where pore pressures generated in the triggering process have largely contributed to the failure. Therefore, frictional fluids with coupling of pore pressures are recommended here. Please, notice that pore fluids refer both to water in saturated soils and to air in unsaturated or dry materials. There exists liquefaction in dry materials, the air playing the role of water in the process. Their dissipation time is very small, but what matters is the relative size of consolidation and propagation times. The case of Las Colinas was first analyzed by Pastor et al (2001) assuming coupling between solid skeleton and air.**
- **Debris flows are quite a general class of phenomena, the material being a mixture of solids and fluids. If the fluid is of muddy type, the rheological model will have to be of cohesive type, while if not, frictional type of fluids will be suitable**.

➢ *Cohesive fluid models (Bingham) for mudflows*

In the case of mudflows, the material model to be used is a cohesive fluid model, such as Bingham or any extension of it like Herschel-Bulkley model.

In the context of landslides, Bingham models have been used to model mixtures of fine grained soils with high water contains. Several researchers have provided values which can illustrate theirs ranges.

Rickenmann and Koch (1997) have taken values of τ_y and μ in the ranges 100-800 Pa and 400-800 Pa·s respectively, for simulations of debris flows which occurred in Kamikamihori (Japan) and Saas (Switzerland) valleys. They concluded that the use of Bingham models results on higher velocities of propagation than those predicted by other models.

It is also interesting to consider the values reported by Jan (1997), obtained from other works:

Author	$\rho\left(Kg/m^3\right)$	$\tau_y\left(Pa\right)$	$\mu\left(Pa.s\right)$
Jan		100 to 160	40-60
Johnson	2000 to 2400	60; 170 to 500	45
Sharp& Nobles	2400		20-60
Pierson	2090	130-240	210-810

Jin and Fread (1997) reported the following values for different sites:

Site	$\rho\left(Kg/m^3\right)$	$\tau_y\left(Pa\right)$	$\mu\left(Pa.s\right)$
Anhui	1570	38	2.1
Aberfan	1764	4794	958
Rudd Creek	1575	956	958

One typical example of mudflows is that caused by failing of a tailings dam. We will include here an example reported by Jeyapalan, Duncan and Seed (1983).

Jeyapalan, Duncan and Seed (1983) report the flow of liquefied tailings from Gypsum Tailings Impoundment in East Texas which took place in 1966. The impoundment was rectangular in plan, and had reached a height of 11 m by the time failure took place. The slide was caused by seepage at the toe of the slope, and affected a length of 140 m of the dyke, extending 110 m into the impoundment lagoon. The released material travelled 300 m before stopping, with an average velocity of 2.5 to 5 m/s. A section of the flow by a vertical plane perpendicular to the dyke at the centre of the breach is given in Fig. 3.4.1.

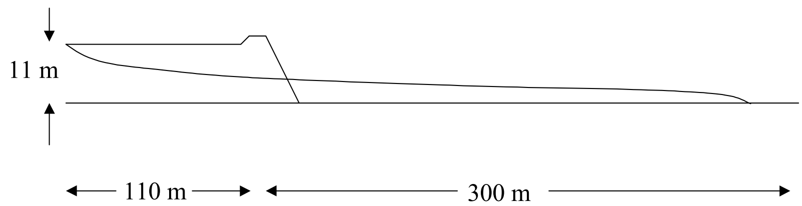

Fig. 3.4.1. Central section of the failure at Gypsum Tailings impoundment

Properties of the tailings have been taken from Jeyapalan et al (1983) who assumed that the behaviour of the flowing material could be described by a Bingham model. The tailings were classified as "non-plastic silts", with D_{50} = 0.07, a uniformity coefficient close to 3, density of particles 2450 Kg/m^{3}and an average water content of 30%. The yield stress was determined from simple slope stability analysis as $\tau_y = 10^3$ Pa, and the viscosity was taken as $\mu = 50$ Pa·s. Finally, density of the tailings was assumed to be $\rho = 1400$ Kg/m^3.

The finite element mesh used in the analysis is shown in Fig. 3.4.2. The characteristic element size is 7 m. We have modelled the 350 m length dyke, removing the 140 m long section which failed. Therefore, the model assumes that collapse of the dyke takes place instantaneously. The plain onto which the tailings flow has been limited for computational reasons, imposing absorbing conditions at the artificial boundaries.

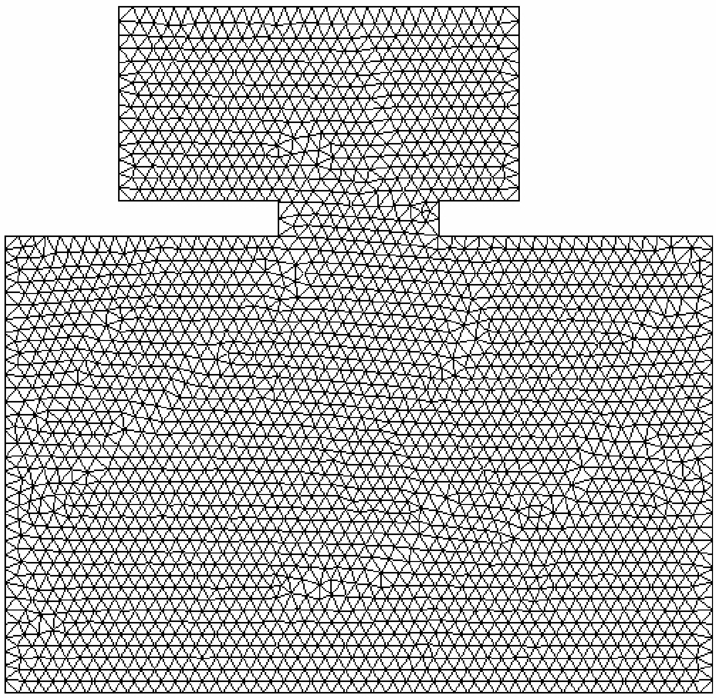

Fig. 3.4.2. Finite element mesh used in the analysis

Figure 3.4.3 shows the contours of tailings depth on the plane at times 0, 30, 60, 90 and 120 s. The vertical scale has been enlarged by a factor of ten in order to better describe the properties of the flowslide. It is interesting to see the development of a transient jump at t = 30 s which propagates backwards. This jump is also seen in Figure 3.4.4, where free surface profiles along the vertical plane passing by the centerline of the breach are given.

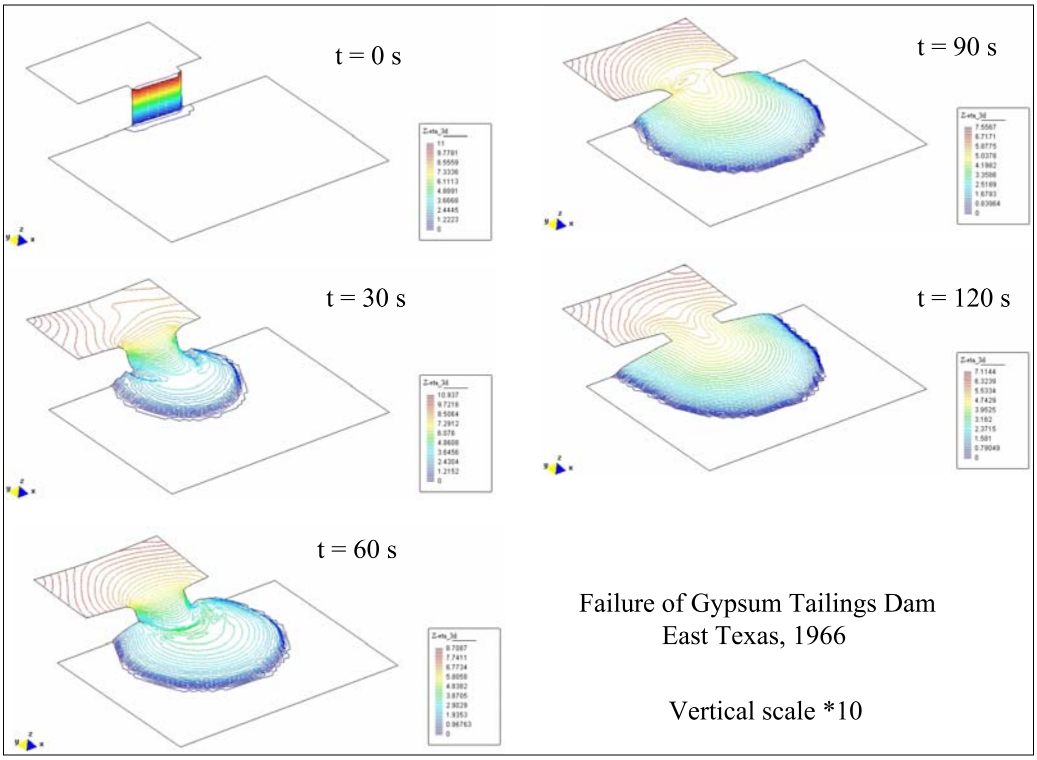

Fig. 3.4.3. Run out of Gypsum tailings dam failure

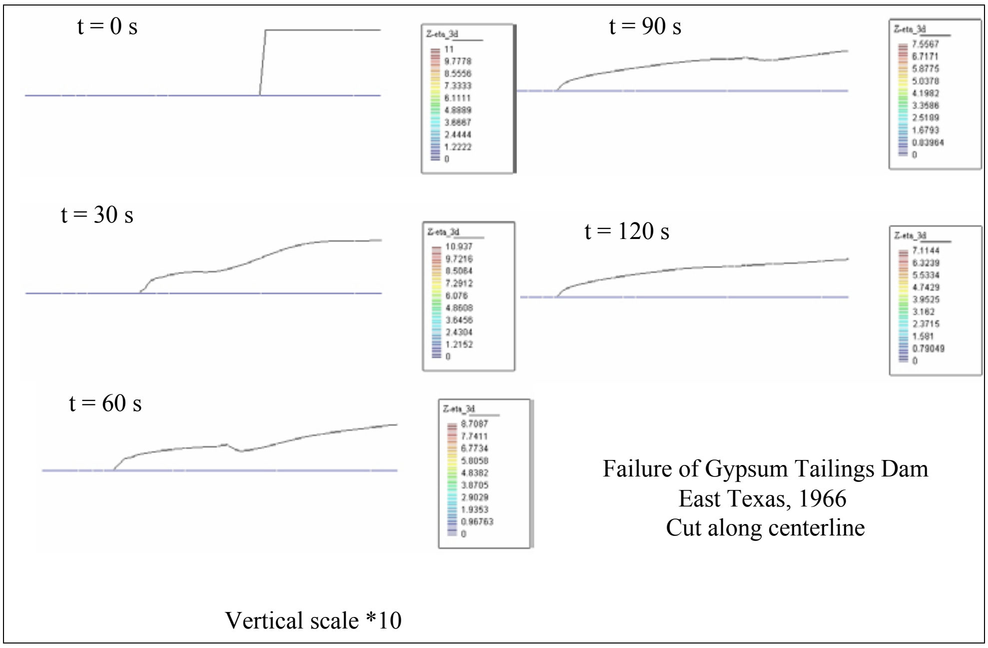

Fig. 3.4.4. Gypsum tailings dam failure: profiles along central section

The results agree well with observed values of inundation distance (300 m) , freezing time (60-120 s) and mean velocity in the range of (2.5-5.0 m/s) However, the flowslide progressed back into the pond about 110 m, while in the simulation reaches a longer distance.

- ***Frictional fluid models for rock avalanches and debris flows***

In the cases of rock avalanches and granular debris flows, the behaviour of the landsliding soil can be modelled by simple frictional fluid models. The main material parameter is the friction angle. However, the users of depth integrated run out models have to take care because when performing the depth integration of momentum and constitutive equations, the resulting closure term is the basal friction, i.e., the friction between the landslide mass and the basal surface. In some occasions, this can be very much different of the friction angle of the material. As an example, it is worth mentioning the case of rock avalanches travelling over glaciers, where the friction angle can be very much smaller.

Friction angle can be determined – if there is field information available – by considering the angle of reach which is defined as the *"angle of the line connecting the highest point of the landslide crown scarp to the distal margin of the displaced mass*", although care has to be taken because there exists a dependence on the rock avalanche volume showing that large landslides display lower angles of reach.

Basic frictional fluid can be extended to account for effects such as turbulence of the flow. As an example, we can mention Voellmy's model which provides basal friction terms of the form:

$$\tau_b = \sigma_v^{'} \tan\phi + \rho g \frac{\overline{v}^2}{\zeta}$$

where $\sigma_v^{'}$ is the effective normal stress on the basal surface, ϕ the friction angle, ρ the mixture density, g the acceleration of gravity, $\overline{v}$ the depth averaged velocity, and ζ a turbulent viscosity coefficient with units m/s^2.

As an example of the use of frictional models to model an avalanche travelling through a glacier, we have chosen a case which has been reported in the literature for which there is good information available, the Thurwieser rock avalanche.

This case is a rock avalanche which occurred in the Central Italian Alps the 18th September 2004. The location was the south slope of Punta Thurwieser, and it propagated through Zebrú valley. Its propagation path extended from 3500 m to 2300 m of altitude, with a travel distance of 2.9 Km. The rock avalanche involved 2.2 million cubic meters. The information concerning this avalanche, including a detailed digital terrain model has been provided by Sosio and Crosta (2007). Figure 3.4.5 from Sosio and Crosta (2008), and Sosio et al (2007) provide a general view of the avalanche and its location.

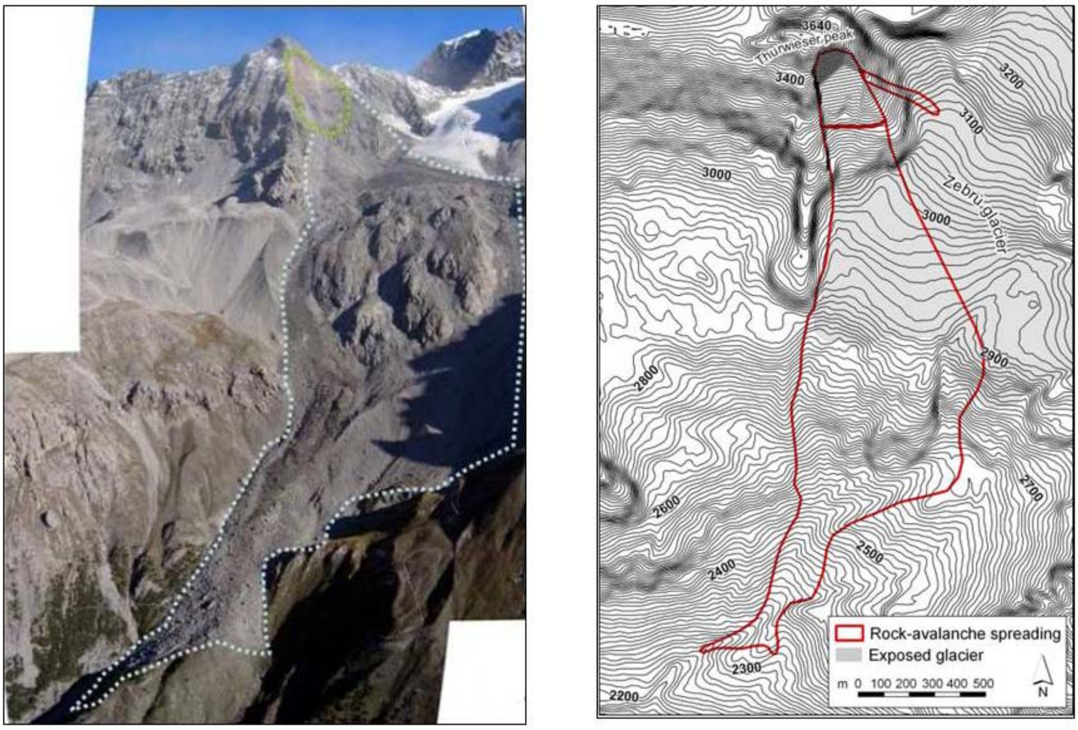

Figure 3.4.5 General view of Thurwieser rock avalanche (Sosio and Crosta 2007)

This avalanche presents several modeling difficulties, such as crossing of terrains of different materials, such as the Zebrú glacier. There, the basal friction is very small, and erosion of ice and snow is possible. This entrained material can melt due to the heat generated by basal friction, providing extra water, and probably originating basal pore pressures. We have used here a simple frictional model including Voellmy turbulence. Concerning erosion, we have used the law proposed by Hungr 1995. The rheological parameters chosen are: $\phi = 26^o$, Voellmy coefficient 1000 $\mathrm{m/s^2}$, erosion coefficient 0.00025 $\mathrm{m^{-1}}$.

The results are given in Figures 3.4.6 and 3.4.7, where we have plotted the avalanche evolution along time and the computed final extension together with the observed in the field.

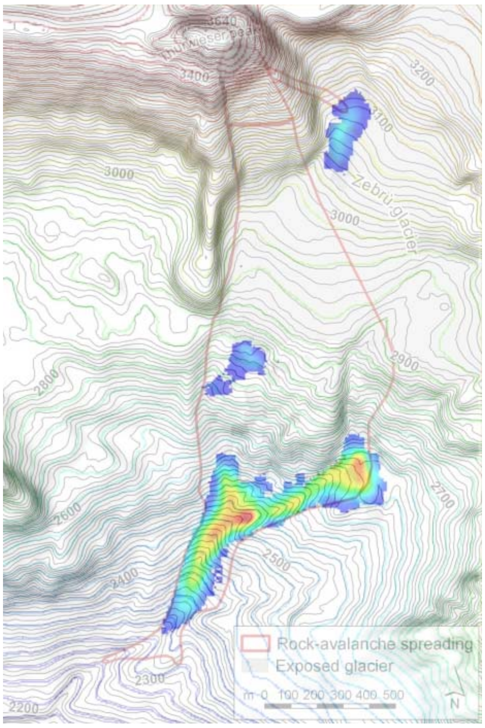

Figure 3.4.6. Thurwieser avalanche after 80 seconds with friction angle 26 º: computed results (colourful isolines and deposit height) versus field measurements (black isolines and red line for the spreading)

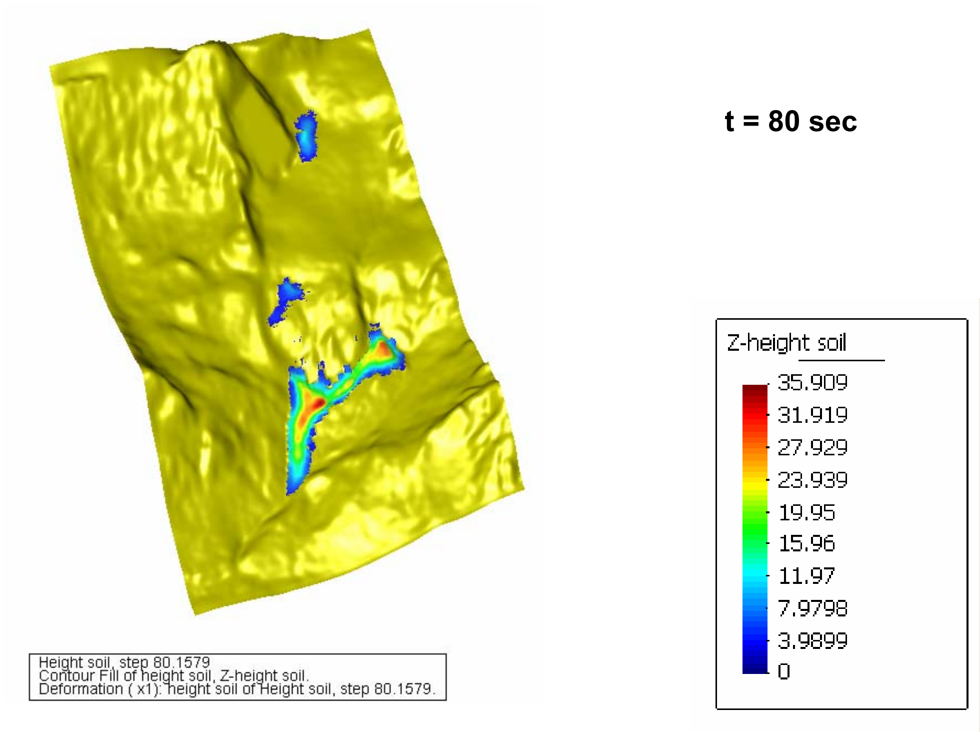

Figure 3.4.7: Thurwieser propagation after 80 seconds with friction angle 26 º: computed results

➢ *Coupled models with basal pore pressure dissipations for flowslides*

Flow slides present frequently an important coupling between soil skeleton and pore fluids. The aim of this section is to propose some recommendations related to how to identify those problems where pore pressures are going to be important.

In many occasions, flowslides are associated to very loose metaestable materials, as it happens in the cases of hydraulic fills, mine dumps and tailings dams. Failure uses to be of diffuse (Darve and Laouafa 2000) instead of localized type. **In order to assess the possibility of this type of failure, it is necessary to carefully analyze the results of consolidated undrained (CU) triaxial tests performed in laboratory. It is important to notice that the tests should be of CU type and not of consolidated drained type, because liquefaction is clearly seen for the former**.

As an example of the required type of tests we include in Fig. 3.4.8 tests performed by Dawson and co workers on the materials found on Fording Greenhills, where in May 1992 the Cougar 7 dump failed. According to Dawson et al. (1998), the mobilized mass consisted on some 200000 cubic meters which propagated downhill a distance of 700 m before stopping. The dump which failed had a height close to 100 m.

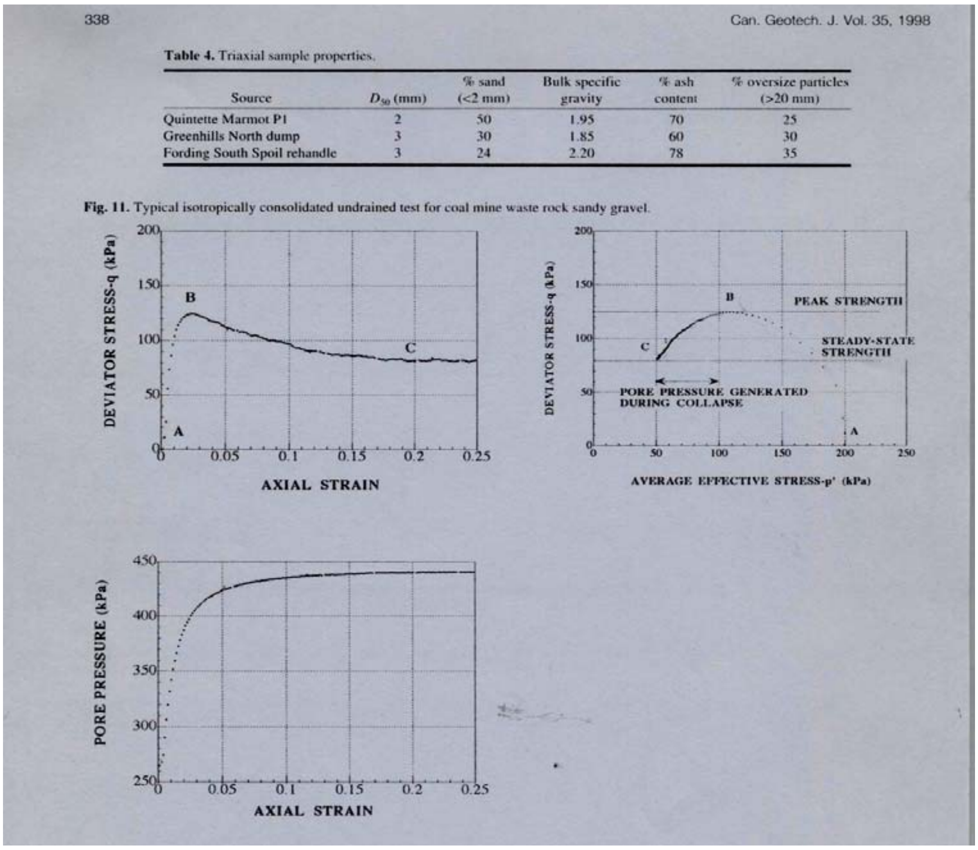
338 Can. Geotech. J. Vol. 35, 1998

Table 4. Triaxial sample properties.

Source	D_{50} (mm)	% sand (<2 mm)	Bulk specific gravity	% ash content	% oversize particles (>20 mm)
Quintette Marmot P1	2	50	1.95	70	25
Greenhills North dump	3	30	1.85	60	30
Fording South Spoil rehandle	3	24	2.20	78	35

Fig. 11. Typical isotropically consolidated undrained test for coal mine waste rock sandy gravel.

Fig. 3.4.8. Typical CU Triaxial test on Cougar Hill materials (Dawson et al.1998)

If we look at the stress path shown on the upper right picture, we find there the typical pattern of liquefaction, and it is possible to evaluate from there the residual friction angle, which is the one which will control the flow. It is important to notice that the deviatoric stress vs. strain curve will not confirm the occurrence of liquefaction, as similar patterns are found on softening materials.

The laboratory tests performed on Aberfan materials were of CD type, and we can only conclude that the material had a tendency to compact, without a positive confirmation of liquefaction.

The first conclusion is that from CU triaxial tests we can ensure the possibility of liquefaction and obtain at the same time the residual friction angle.

Next, we have to obtain from classical oedometric tests the coefficient of consolidation, which is given by:

$$\frac{d\bar{P}_1}{dt} = -\frac{\pi^2}{4h^2} c_v P_1$$

where P_1 is the basal pore pressure, c_v the consolidation coefficient, and h the depth of the flowslide at the considered material point. The consolidation coefficient depends on the oedometric modulus E_m and the permeability k_w as:

$$c_v = \frac{E_m k_w}{\rho_w g}$$

It is interesting to note that we can obtain a characteristic consolidation time from above parameters as:

$$T = \frac{4h^2}{\pi^2 c_v}$$

This characteristic time is a most relevant parameter, because

(i) if it is much larger than the expected propagation time, the process can be considered as undrained, and coupling analysis will not be necessary. We will have to estimate an apparent friction angle which will take into account the effect of the generated pore pressure –assumed constant in the propagation phase.

(ii) If the time of consolidation is much smaller than the expected time of propagation, the analysis can be done "drained", i.e., without being necessary to study the coupling.

(ii) if this characteristic time of consolidation is of the same order than that of propagation, the analysis has to be of coupled type.

In the case of Fording Greenhills, the consolidation characteristic time was estimated as 68 s from the laboratory data provided by Dawson et al.

In order to provide some insight of the dissipation of pore pressures during propagation, we depict in Figures 3.4.9 and 3.4.10 both the propagation and the dissipation of pore pressure.

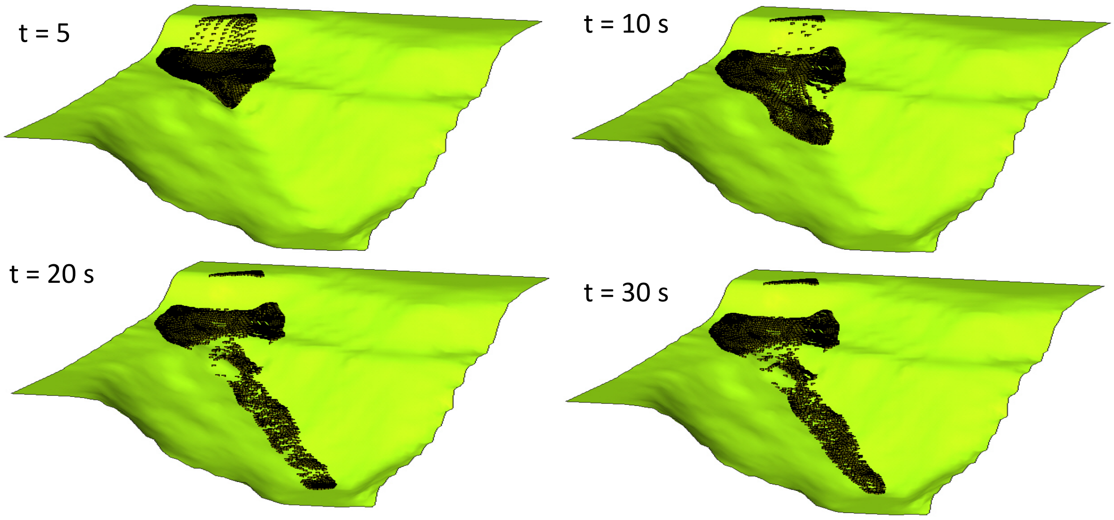

Fig. 3.4.9.(a). Position of SPH nodes at different times during propagation

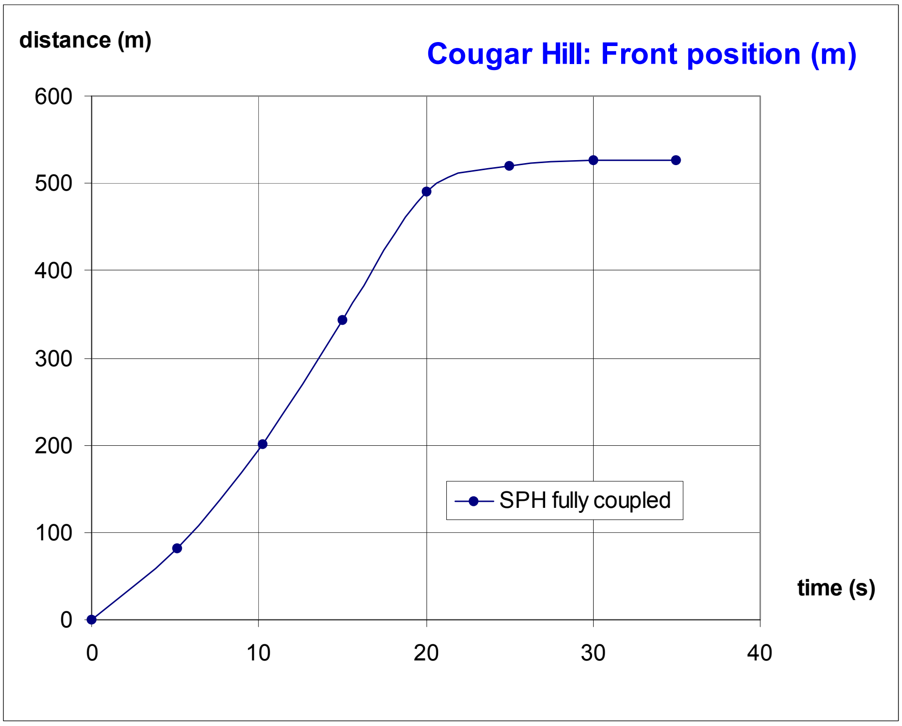

Fig. 3.4.9.(b). Evolution of front position with time

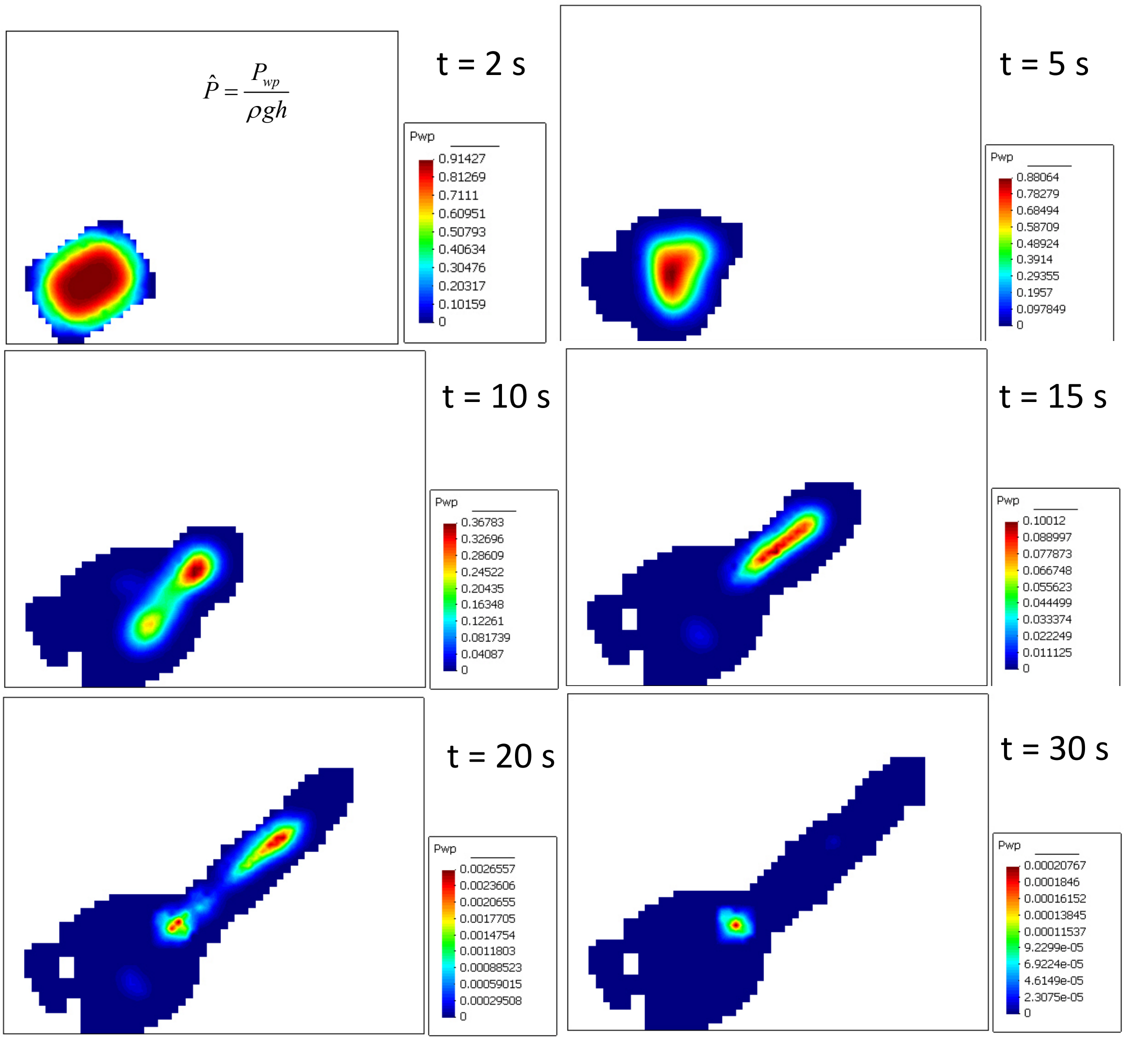

Fig. 3.4.10. Evolution of pore pressures (relative to that causing liquefaction)in Cougar Hills

It is most interesting to notice the relation between the speed of propagation and the basal pore pressures. Indeed, the flowslide has come to rest at the moment the pore pressures have completely dissipated.

As conclusions for flowslides analysis, we can state that:

(i) from CU triaxial tests we can ensure the possibility of liquefaction and obtain at the same time the residual friction angle.

(ii) Dissipation of pore pressures depends on geotechnical parameters which can be easily obtained in standard tests.

(iii) The convenience or even necessity of performing a coupled analysis depends on the relative values of the propagation and consolidation times.

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3.5. BASAL EROSION

Basal erosion plays a fundamental role in many landslides. While there exists today a number of empirical formulas providing an estimation of erosion for depth integrated models, there is a lack of sound theories able to relate in a consistent way the properties of the landsliding mass and the hydro mechanical characteristics of the basal surface over which it flows.

A consistent way of modelling the process has been proposed by Crosta et al. (2008). We depict in Figure 3.5.1 taken from Deliverable D.1.7 the results obtained with a ALE model able to reproduce large displacements of a soil mass.

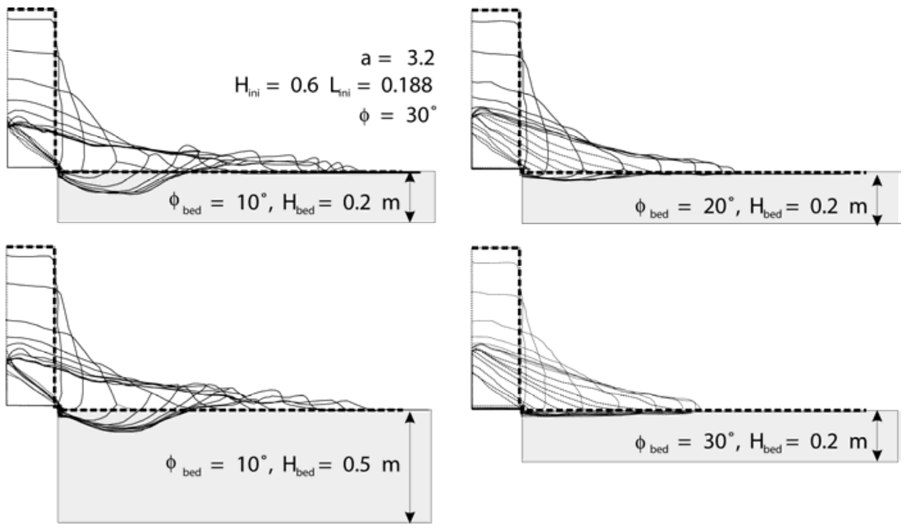

Figure 3.5.1. Results of the numerical models for the collapse of a granular step (Hini = 0.6 m; Lini = 0.188 m; a = 3.2, see Table 1) including the presence of an erodible basal layer (light grey). The layer thickness and internal friction angle have been varied. Time step between each profile is 0.1 sec. The interface between static and moving material is also represented for each instantaneous profile. A completely different behavior is observed for the interface when a low shear strength basal layer is simulated: no deposition is simulated along the interface and no increase in static sector is observed till final arrest.

It is also worth mentioning here the approach proposed by Issler and Johannesson (2010), where a novel and consistent erosion law has been introduced.

Depth integrated codes use to implement simple erosion laws – lacking the consistency of the afore mentioned approaches.

As an example, we can mention the following: the Hungr erosion law (Hungr 1995), the modified Egashira erosion law (Egashira 1993) and a new proposed erosion law (Blanc 2008). All of them have been described in Blanc (2008). What it is important to notice is that approximated erosion laws cannot provide extremely accurate results. However, their accuracy is thought to be acceptable.

➢ ***Description of the Hungr erosion law (Hungr 1995)***

In the Hungr's erosion law, the erosion rate increases in proportion to the flow depth, resulting in an exponential growth of the debris flow with displacement. Although this law is empirical, it has a physical basis. Indeed the changes in the stress conditions leading to a failure in the bottom of the flow path and with this the entrainment of the material are related to the changes in the total bed-normal stress and thus with the flow depth.

This empirical law is based on a input parameter, E_s, given by the user of the model. E_s is a displacement erosion rate, the so called growth rate. This parameter represents the bed-normal depth eroded per unit flow depth and unit displacement. The dimension of this parameter is L^{-1}. It is worth mentioning that this parameter is different to the time dependant erosion rate, e_r and it is independent to the flow velocity. For example when E_s is constant and takes the value 0.01, the debris flow volume increases by 1% when it travels 1 meter.

Hungr's law consists of the relation between the erosion rate e_r and the growth rate E_s :

$$e_r = E_s \times h \times v \tag{1}$$

where h is the flow depth and v is the depth averaged flow velocity (Hungr, 1995).

- ***Description of the modified Egashira's erosion law (Egashira1993)***

Egashira's law is based on flume tests, as well as numerical and dimensional analyses. Egashira assumes that the bed slope is always adjusted to its equilibrium in case of debris flows travelling over an erodible bed.

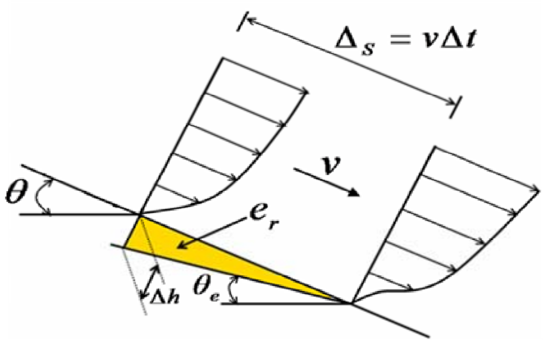

Figure 3.5.2 Sketch depicting main variables

Referring to the figure 3.5.2, the mass conservation law of eroded material yield can be applied:

$$e_r \Delta_s = e_r v \Delta t = c_* v \Delta h \tag{2}$$

Where:

- c_* is the sediment concentration by volume of bed sediment (of the non moving layer),
- θ is the bed slope,
- θ_e is the equilibrium bed slope, and
- the other magnitudes are have already been defined previously.

From the equation 2, the next step is obtained:

$$\frac{e_r}{v} = c_* \frac{\Delta h}{\Delta s}$$

From here, Egashira derived his erosion law, substituting in the last equation the term $\theta - \theta_e = \arctan\left(\frac{\Delta h}{\Delta_s}\right)$:

$$e_r = c_* v \tan\left(\theta - \theta_e\right)$$

with:

$$\theta_e = \tan^{-1}\left\{\frac{(\sigma - \rho)c}{(\sigma - \rho)c + \rho}\tan\phi\right\}$$

where:

- σ the mass density of the sediment particle
- ρ the mass density of the water
- c the sediment concentration of the debris flow by volume, and
- ϕ the internal friction angle of the bed approximated by the basal friction angle $\tan\phi_b$.

With Egashira's law, the user of the model has to input two parameters which are:

- c, the sediment concentration of the debris flow by volume
- c_*, the sediment concentration by volume of bed sediment (of the non moving layer).

Experimentally, Takahashi (1992) has proved that the sediment concentration of the debris flow (c) cannot exceed the value $0.9c_*$. Therefore the following has to be always verified:

$$c < 0.9c_*$$

Implemented into the 2D SPH depth integrated model the Egashira's erosion law has to be modified in order to get realistic results. In fact the value of the erosion rate given by Egashira has to be multiplied by an empirical factor, K, and thus it becomes:

$$e_r = Kc_* v \tan\left(\theta - \theta_e\right).$$

- ***Description of a new erosion law (Blanc 2008)***

A new erosion law based on a combination of the Egashira's and Hungr's law has been proposed. This law would have the following form:

$$e_r = K \times v \times h \times \left(\tan\theta\right)^{2.5}$$

where:

- K is an empirical parameter
- θ is the slope

- v is the flow velocity
- h is the flow depth

This type of law would allow calculating erosion rates taking into account the slope as well as other variables (e.g. flow velocity and flow depth). This law is only a proposition and should be tested in laboratory and in the SPH depth integrated model before to be validated. The exponent 2.5 is purely empirical. This law has been introduced in order to represent the variation of the erosion processes along the path. In fact the erosion processes take place mainly in the first two stages of the debris flow (initiation and propagation) while the debris flow do not erode the bed in the deposition area.

We depict in figure 3.5.3 the results obtained with these different erosion laws in the simulation of a debris flow event which took place in Tsing Chan (Hong Kong) in 1990. Erosion was important, because the volume of the deposited material was about 20000 m^3 while the initiation volume was only about 350 m^3.

It can be observed that in all the calculations with Hungr's law, the volume increases rapidly in the very last part which correspond to the deposition area where no erosion take place (cf. the 1990 and 2000 Tsing Chan debris flow). Thus while Hungr's law is able to predict well the final volume, this law is not able to evaluate the volume along the flow path.

The modified and proposed erosion law give better results because with these laws most of the debris is accumulated before reaching the last part of the path. As a consequence the erosion occurs mainly in the first two stages: the initiation and the propagation area (cf. the 2000 Tsing Chan debris flow).

In the 1990 Tsing Chan debris flow, the law proposed by Blanc (2008) is the law closest to reality. In fact there is a great increase of the volume along the initial 50% of the flow path and then the volume increases at a lower rate.

The main conclusion of the comparison is that the slope of the terrain is the parameter which plays the most important role in the erosion processes, together with availability of the erodible material; in consequence it should be integrated into the erosion laws. Otherwise, errors will be introduced, but the quality is still acceptable.

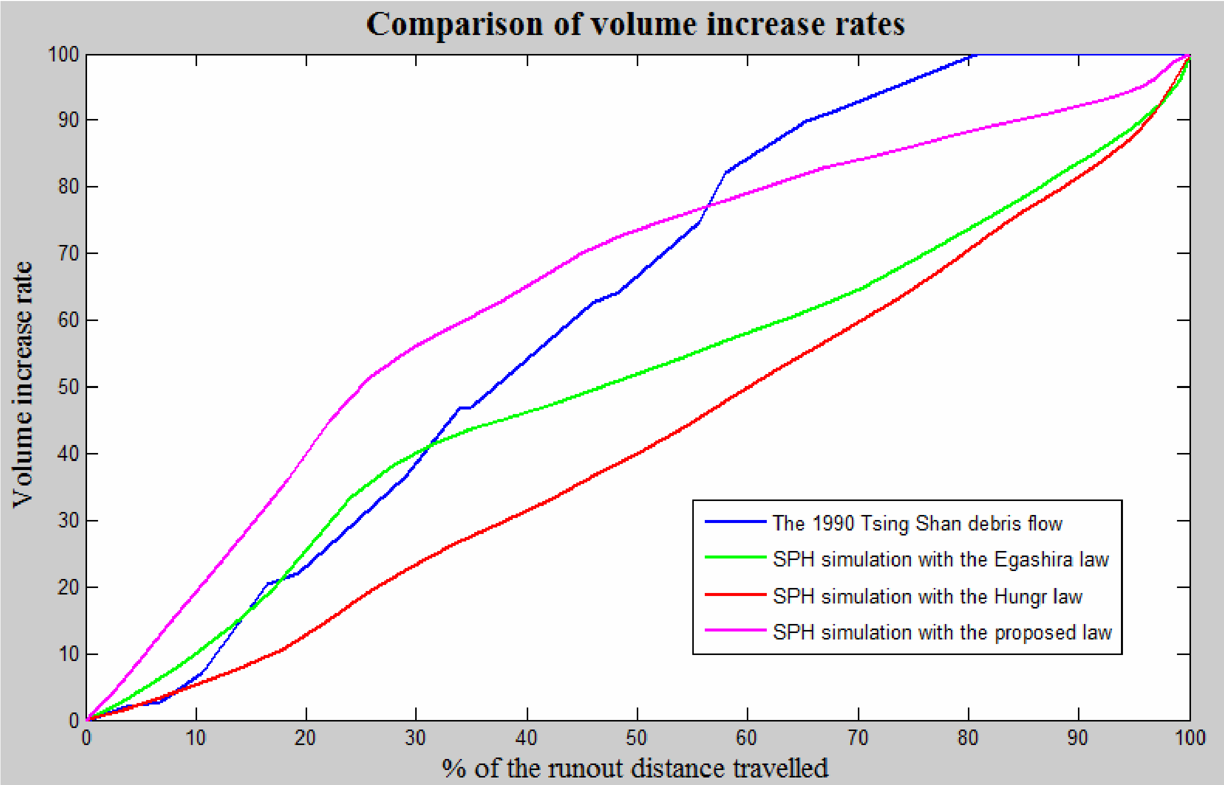

Fig.3.5.3 Erosion in the 1990 Tsing Chan debris flow (Blanc 2008)

Hungr's law (red) provides a good estimate of the final amount of eroded material, because this is precisely the value used for the calibration. On the other hand, both the Egashira and Blanc laws provide results of good quality.

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3.6. EULERIAN (FINITE ELEMENTS) VS. LAGRANGIAN (SPH) MODELS

So far, we have presented recommendations related to the importance of initial conditions, the accuracy of the DTM, the rheology, and the erosion laws. Most of them apply to both Eulerian and Lagrangian numerical methods, such as finite elements and SPH, just to mention one example from each group.

While both approaches are of similar quality in most of the cases, they present relative advantages and disadvantages which we will describe here.

Users of depth integrated models will find one approach more interesting than the other in some cases, and vice versa.

The main advantage of meshless methods, when compared to Eulerian finite elements or finite volume, is the computational cost. In fact, the time of computation is lower than that of classical, Eulerian finite elements, because the computational grid is separated from the structured terrain mesh used to describe terrain topography. The authors of this report have measured differences in computer time of ratios close to 1:30 in favor of SPH when analyzing the propagation of lahars in the Popocatépl volcano. The reason is that only a very small part of the topography was occupied by the propagating lahar. In the case of finite elements, all nodes have to be active, hence the much larger computing cost. In other occasions, as for instance, when studying the propagation of a mudflow originated by the failure of a tailings dam, computing times are more similar (ratios close to 1:5) Here the reason is that most of the computational domain was occupied by the flow.

Another aspect which favors SPH methods is the mass conservation, which is enforced in a more effective way. Eulerian finite element models of landslide propagation over long distances suffer from a loss of mass which is much larger than that found in SPH methods. The reason is that Eulerian methods use to make zero heights smaller than a threshold value to avoid numerical instabilities.

On the other hand, simulation of walls containing a fluid is much easily dealt with finite elements than with SPH methods, which do require special techniques due to their boundary deficiency problem.

Finally, one important limitation of SPH methods arises when using hydrographs to apply boundary conditions related to the incoming flow in a domain. Indeed, the solution in SPH is to inject nodes, but then we need to apply initial conditions on them, and not boundary conditions.

In conclusion, both finite elements and SPH techniques present relative advantages and disadvantages. The user of depth integrated codes has to decide which one is preferable in each particular case, for which he can follow the hints provided above.

3.7. CONCLUSIONS and RECOMMENDATIONS

This report has the main objective of providing the users of depth integrated models some recommendations concerning their use.

The major conclusions described in the preceding Sections are the following:

(1) Recommendations concerning initial conditions

- Users of run out models should be aware of the fact that the results obtained in the simulations, will present a run out depending on: (i) the initial mass, and (ii) the rheology selected
- They are advised to perform a convergence analysis with regard to the number of material points or nodes discretizing the landsliding mass whenever possible at the beginning of a series of numerical simulations, whenever possible.
- We do recommend them to analyze the available geotechnical information to verify if the triggering of the landslide is a consequence or results on the development of relevant pore pressures. Should this be the case, the model to be applied has to (i) estimate the initial distribution of pore pressure, and (ii) model the propagation incorporating pore pressures and their dissipation.

(2) Recommendations concerning terrain representation

- Users of run out models are strongly recommended to make sure that the precision of the DTM being used in the analysis is enough. While error indicators are available for given DTMs, they have not been used by practitioners, and are not included in depth integrated codes. There is, therefore, a first indicator of the precision of the mesh, which is the product of the second order derivative of the basal surface height by the square of the mesh size, but this is not enough. Much research is still needed here. As a rule of thumb, we can suggest that at least 10 points are used to discretized canyons and gullies channeling the flow.

- We suggest that users of run out models should consider the possibility of including barriers whenever there exist in the path elements such as houses, roads, curbs, embankments, etc., which cannot be properly described by the DTM used

(3) Recommendations concerning the choice of the rheological model and its parameters

- Concerning the choice of a particular rheological model, we suggest the following:
    - Rock avalanches are advised to be modeled with frictional fluid like models.
    - Mudflows are recommended to be modeled with cohesive fluid models such as Bingham, for instance.
    - Flowslides are usually generated in very loose metastable materials, where pore pressures generated in the triggering process have largely contributed to the failure. Therefore, frictional fluids with coupling of pore pressures are recommended her. Please, notice that pore fluids refer both to water in saturated soils and to air in unsaturated or dry materials. There exists liquefaction in dry materials, the air playing the role of water in the process. Their dissipation time is very small, but what matters is the relative size of consolidation and propagation times.
        - Debris flows are quite a general class of phenomena, the material being a mixture of solids and fluids. If the fluid is of muddy type, the rheological model will have to be of cohesive type, while if not, frictional type of fluids will be suitable.
- Concerning the special case of flowslides, it is important to note that:
    - CU triaxial tests can verify the possibility of liquefaction and obtain at the same time the residual friction angle.
    - Dissipation of pore pressures depends on geotechnical parameters which can be easily obtained in standard tests.
    - The convenience or even necessity of performing a coupled analysis depends on the relative values of the propagation and consolidation times

(4) Regarding basal erosion, it is important to take into account the following:

- The slope of the terrain is the parameter which plays the most important role in the erosion processes; in consequence it should be integrated into the erosion laws. Otherwise, errors will be produced, but the quality is still acceptable Simple laws, such as Hungr's provide enough accurate results if the final eroded volume is known.

(5) Recommendations concerning the choice between Eulerian (Finite elements) or Lagrangian (SPH) numerical models

- As far as the choice between finite elements or SPH numerical models is concerned, both present relative advantages and disadvantages which the user of a numerical model has to know before choosing the particular model he will use for a particular case or group of cases. In most relevant cases of long run out landslides, SPH models present clear advantages: (i) They are computationally cheaper, (ii) mass conservation is enforced in a more efficient way.
- The cases where finite elements have advantages are: (i) presence of walls in the domain, enclosing reservoirs (ii) use of hydrographs.