POLITECNICO DI TORINO

Whatever is the numerical approach to the study of rock avalanche evolution, obtained results depend on the choice of the value that is assigned to the characteristic parameters of the assumed rheology.
The model DAN has been applied to back analyse a set of case histories of rock avalanches in order to define a range of variations of the above mentioned parameters.

1 BACK ANALYSES

The first step in calibrating a model consists in defining categories of rock avalanches with similar mechanism of failure, material type, volume involved, velocity and slide path characteristics. Subsequently, typical ranges of values of the resistance parameters are defined for each category through back analysis of several real cases.

1.1 Classification criteria

Numerical simulations should provide a useful tool for investigating case histories if some geometrical data and some parameters characteristics of the analysed case are known.
A back analysis procedure gives the possibility of calibrating the model in order to obtain the best value that has to be assigned to each of the required rheological parameters.
Case histories suitable for back analysis were selected from literature and integrated with cases previously back analysed by Hungr (Hungr and Evans, 1996).
To define typical ranges of values of the rheological parameters, a subdivision among cases having different characteristics, and by consequence a different behaviour during the run out phase, has to be created. The first step to be carried out consists in the identification of some general aspects that seem to modify the propagation phase of the unstable mass. The considered categories are the following:
Landslide volume (V). In a landslide phenomenon mobility seems to increase with volume. Compared to a rock fall event ($V<10000\ m^3$), the motion of rock avalanches is more massive and the bulk of rock fragments moves as a semi-coherent flowing mass. The sub-classes in which cases are subdivided as a function of the volume involved are:

- $V<1\cdot10^6\ m^3$
- $1\cdot10^6<V<10\cdot10^6\ m^3$
- $10\cdot10^6<V<50\cdot10^6\ m^3$
- $50\cdot10^6<V<100\cdot10^6\ m^3$
- $V>100\cdot10^6\ m^3$

Slope average angle (α). It is defined as the angle obtainable connecting the uppermost point from which the rock mass broke away and the first point along the considered profile that is placed at the end of a portion of the path having a dip of less than 10° over a distance of at least 100 m. The sub-classes in which cases are subdivided as a function of α are:

- $\alpha\leq5°$
- $5°<\alpha\leq15°$
- $15°<\alpha\leq25°$
- $\alpha>25°$

Unstable sector average angle (β). It is defined as the angle obtainable connecting the uppermost point from which the rock mass broke away and the toe of the surface of rupture. The considered sub-categories are:

- $\beta\leq30°$
- $30°<\beta\leq40°$
- $40°<\beta\leq50°$
- $\beta>50°$

Run out area shape. It introduces three different sub-classes as a function of the shape assumed by the deposit. The considered categories are the following (Nicoletti and Sorriso-Valvo, 1991):

Elongated shape (A). This generally occurs when there is a narrow valley down which the debris is channelized.

Tongue shape (B). This occurs when the moving debris is free from lateral constraints and is able to stop spontaneously when it comes to a wide valley or plan.

T shape (C). This shape results from the crossing of a narrow valley followed by a perpendicular impact against the opposite slope. Run up and partition of the debris are common features.

Material type. For the intact rock pieces, the generalised Hoek-Brown failure criterion for rock jointed masses (Hoek & Brown, 1980) is simplified to:

$$\sigma_1^{'} = \sigma_3^{'} + \sigma_{ci}\left(m_i \frac{\sigma_3^{'}}{\sigma_{ci}} + 1\right)^{0.5} \qquad (2)$$

where σ'_1 and σ'_3 are the maximum and minimum effective stresses at failure respectively, m_i is the value of the Hoek-Brown constant m for the intact rock, 1 and 0.5 are the characteristic constants of the rock mass in case of intact rock, and σ_{ci} is the uniaxial compressive strength of the intact rock pieces.

In order to use the above mentioned criterion for estimating the strength and deformability of jointed rock mass, σ_{ci} and m_i have to be estimated (Hoek & Brown, 1997).

When laboratory tests are not possible, Table 1 can be used to obtained estimate of σ_{ci} and m_i of the intact rock pieces in the rock mass.

On the base of this criterion, the material type classification can subdivide the cases as a function of UCS as indicated in Table 1.

Term	Uniaxial Compressive Strength [MPa]	Point Load Index [MPa]	Field estimate of strength	Examples
R6. Extremely strong	>250	>10	Specimen can only be chipped with a geological hammer	Fresh basalt, chert, diabase gneiss, granite, quartzite
R5. Very strong	100-250	4-10	Specimen requires many blows of a geological hammer to fracture it	Amphibolite, sandstone, basalt, gabbro, gneiss, granodiorite, limestone, marble, rhyolite, tuff
R4. Strong	50-100	2-4	Specimen requires more than one blow of a geological hammer to fracture it	Limestone, marble, phyllite, sandstone, schist, shale
R3. Medium strong	25-50	1-2	Cannot be scraped or peeled with a pocket knife, specimen can be fractured with a single blow from a geological hammer	Claystone, coal, concrete, schist, shale, siltstone
R2. Weak	5-25	*	Can be peeled with a pocket knife with difficulty, shallow indentation made by firm blow with point of a geological hammer	Chalk, rocksalt, potash
R1. Very weak	1-5	*	Crumbles under firm blows with point of a geological hammer, can be peeled by a pocket knife	Highly weathered or altered rock
R0. Extremely weak	0.25-1	*	Indented by thumbnail	Stiff fault gouge

* Point load tests on rocks with a uniaxial compressive strength below 25 MPa are likely to yield highly ambiguous results.

Table 1. Field estimates of uniaxial compressive strength (Hoek & Brown, 1997)

Fahrböschung (δ). It may be translated as the average slope angle of the race course. It is defined as the angle of the line connecting the uppermost point from which the rock mass broke away and the distal point of the deposit. The considered sub-categories are:

$\delta \leq 5°$

$5° < \delta \leq 15°$

$\delta \geq 15°$

Slope characteristics. It subdivides phenomena as a function of the characteristics of the travel path. The existence of ice along all or part of the travel path is considered to modify the behaviour of the mass in comparison with the behaviour that the mass could have in presence of vegetation or of debris. The considered categories are the following:

- Debris
- Rock
- Forest
- Glacier.

1.2 Scheme of analysis

Each case has been subjected to a digital reconstruction of the topography. The slope profile geometry, the profile of the top of the initial mass and the path width are described by a series of x (distance), y (elevation) and z (width) points, respectively.

In most of the back analysed cases, the material rheology was assumed to be frictional on the rupture surface and Voellmy along the travel path.

The main results of each analysis were systematically recorded. Each trial run was assessed by matching the following parameters to the actual values as determined from maps or from the reports of the case histories: total horizontal run out distance, length of the main deposit, mean thickness of debris, flow velocities and flow duration (where available).

Only the rheological parameters and the bulk unit weight of the debris were varied in the analyses. The remaining variables, held constant in all the analyses, were: coefficient of lateral pressure "at rest" 1.0, active lateral pressure coefficient 0.8, passive 2.5.

2 CASES OF INTEREST IN THE INTERREG PROJECT

The cases that in the Interreg IIIA project are object of a specific analysis are:

Val Pola (Italy), Six des Eaux Froides (Switzerland) and Charmonetier (France).

Up to now each of these sites has been analysed considering a frictional rheology for the whole path; besides, an additional analysis of Charmonetier has been carried out considering a Voellmy rheology. As showed in Figure 1, a correct reconstruction of the runout is mainly obtained comparing the real pre- and post- profile of the considered case with the deposit geometrical configuration obtained running the code.

Val Pola **Six des Eaux Froides**

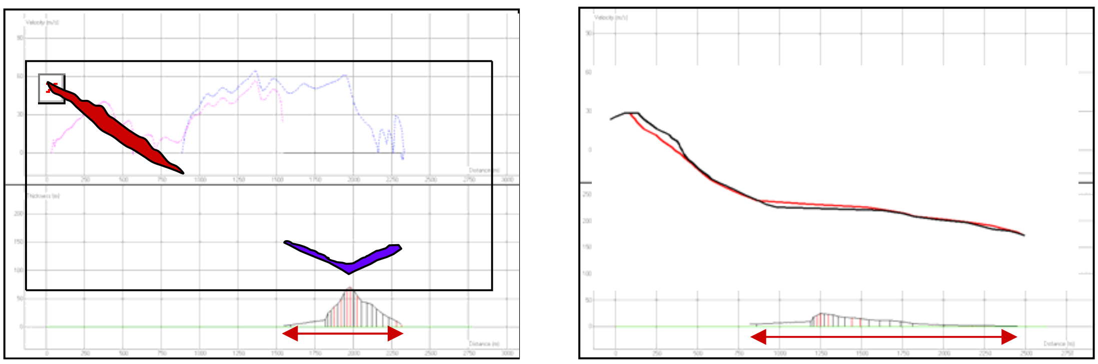

Figure 1. Reconstruction of runout geometry.

The best match parameters are summarized in Table 2.

	Frictional	**Voellmy**
Val Pola	$\varphi = 22°$	
Charmonetier	$\varphi = 39°$	$\tan\varphi = 0.72$ $\xi = 300\ m/s^2$
Six des Eaux Froides	$\varphi = 17°$	

Table 2. Results obtained.

3 CASE HISTORIES SELECTED FROM LITERATURE

Case histories selected from literature were considered well documented for our purposes if the collected data allowed us to have a topography, containing information about pre- and post- collapse configuration of the slope, and a profile along the path of the movement.

The code calibration has been more accurate when detailed data characterising the propagation phase were supplied (i.e. velocity, deposit depth).

In Table 3 are summarized the best match parameters for the analysed cases integrated with results obtained by Hungr (Hungr & Evans, 1996).

As previously mentioned, most of the analyses carried out assume simultaneously a frictional rheology in correspondence of the rupture surface and a Voellmy rheology along the travel path, while results obtained by Hungr & Evans (1996) derive from analyses in which either a frictional rheology or a Voellmy rheology is assumed.

The application of the constant frictional rheology gives results that have the tendency to predict excessive thinning of the deposits in the distal part and to overestimate velocities, while the Voellmy rheology produces most consistent results in terms of debris spreading and distribution as well as velocity data, the same results were obtained by Hungr (Hungr & Evans, 1996).

Four of the reported cases were analysed through the three above mentioned rheological combinations: Dusty Creek, Madison Canyon, Pandemonium Creek and Rubble Creek. The simultaneous application of the two considered rheologies underlines that the influence of the frictional rheology, if used only on the rupture surface, is negligible in comparison with the analysis based on the constant Voellmy model. Thus, the values assigned to the friction and turbulence coefficients are approximately the same obtained if only the Voellmy rheology is applied. This allows the recent analyses to be compared with those carried out earlier by Hungr and Evans (1996).

Cases	Classification Criteria							Rheology			
								Frictional		Voellmy	
	$V \cdot 10^6$ [m^3]	α [°]	β [°]	S	M	δ [°]	C	φ [°]	r_u	μ	ξ [m/s^2]
Avalanche Lake N. *	160			C	Dolostone/Carbonite	8	Forest	10		0.1	1000
Avalanche Lake S. *	40			C	Dolostone/Carbonite	10	Forest	11		0.1	500
Blackhawk	283	9.5		B	Marble	6.3	Rock	12	0.4	0.04	1000
Chaos Jumbles	44	8.7	45	B	Dacite	8.7	Forest	11		0.08	1450
Diablerets *	30			A	Limestone	12.4		20		0.2	450
Dusty Creek	7	17.2		B	Dacite/ Pyrocl.rocks	21.3	Forest/Debris	22		0.22	200
Dusty Creek *	7	17.2		B	Dacite/ Pyrocl.rocks	21.3	Forest/Debris	21		0.2	200
Elm *	10	19.7	50	B	Slate	14.6	Forest	14		0.14	500
Flims *	0.1			B	Limestone	28		23		0.21	500
Frank *	36.5			B	Limestone	13.5	Forest	16		0.1	700
Goldau *	35			B	Conglomer.	10.2		12		0.1	500
Gros Ventre *	38			B	Sandstone	8.5		16		0.22	500
Hope *	60	22	30	C	Metamorphic rock	16.2	Forest	16.2		0.1	500
Huascaran	75	14	53	A	Granodiorite	14	Glacier/Rock	14.5	0.1	0.05	500
Kennedy River *								23		0.1	500
Kofels	2200	17.6	22	B	Gneiss	10.2		17.5		0.03	1000
Lake of the Woods *								20		0.24	200
La Madeleine	100	19.1		C	Limestone/Schist	19	Forest	12		0.24	1900
Madison Canyon	28	30	38.7	C	Gneiss/Schist	14.6	Forest	25		0.18	700
Madison Canyon *	28	30	38.7	C	Gneiss/Schist	14.6	Forest	16		0.2	500
Mayunmarca *	1600		35	B	Sandstone/Siltstone	12.4	Rock	12.4		0.1	500
Mount Cayley *	0.74	18.6	38	A	Pyroclastic tuff	19					
Mount Granier *	210			B	Limestone/Marl	11.3		12		0.09	1000
Mount St Helens *	2300						Forest	9.5		0.08	500
Mystery Creek *	35			B	Diorite	17.2	Forest	17		0.1	600
Ontake *	34			A	Pyroclastic rock	6.4		8		0.03	200
Pandemonium Creek	6.7	13.2	48	A	Gneiss	13.2	Glacier/Rock	13		0.1	1000
Pandemonium Creek *	6.7	13.2	48	A	Gneiss	13.2	Glacier/Rock	13		0.1	1000
Rockslide Pass	493	13.4	14	B	Dolostone/Limestone	8.5	Forest	9		0.08	800
Rubble Creek	33	8.5	32	A	Dacite	8.5	Forest	8.6		0.05	140
Rubble Creek *	33	8.5	32	A	Dacite	8.5	Forest	13		0.07	100
Sherman Glacier *	13.3			A	Sandstone/Siltstone	10.2	Glacier	10		0.03	1000
Texas Creek	45	35	35	C	Argillite	16	Forest	25		0.08	1700
Turbid Creek	0.94					19		17.5		0.1	300
Val Pola *	35	32.2	34	C	Gneiss/Diorite	24.7	Forest	16		0.1	300

* Hungr & Evans, 1996

Table 3. Results of the back analyses

V: Involved volume M: Material type
α: Slope average angle δ: Fahrböschung
β: Unstable sector average angle C: Slope characteristics
S: Run out area shape (A: elongated shape, B: tongue shape, C: T shape)

It is observed that the obtained distribution of resistance parameters is influenced by the run out area shape. In Table 4 it is underlined as all the considered parameters assume a mean value that increases changing from A shape to C shape .

	φ [°]			μ [-]			ξ [m/s^2]		
shape	A	B	C	A	B	C	A	B	C
min	8	9	10	0.03	0.03	0.08	100	200	500
mean	12.5	15.9	16.4	0.08	0.12	0.14	548.7	704.2	912.5
max	20	22	25	0.20	0.22	0.24	1000	1450	1900

Table 4 Distribution of resistance parameters as a function of the run out area shape.

4 CONCLUSIONS

The DAN code allows to simulate the main features of all the considered case histories.

The main limitation of the applied model is due to the fact that it reduces a complex and heterogeneous three dimensional problem into an extremely simple formulation. In any case, the simplicity of the model is an advantage in making possible an immediate and rapid numerical simulation of real cases. It also allows the choice among different rheologies, some of which are particularly simple, reducing the number of mechanical parameters that have to be defined.

To obtain useful guidelines to the choice of values to assign to resistance parameters of a potential landslide, provided it belongs to one of the above defined categories, the back analysis of a considerable number of cases is required. The obtainable results should be useful for the prediction of rock avalanche propagation for the purpose of hazard assessment.

Further developments of the present approach envisage the increase of the number of back analysed cases and the interpretation of the obtained results through the research of possible correlations and of ranges of variation of the resistance parameters as a function of the different proposed criteria of evaluation.

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