The engineering geological study of the Bema landslide in North Italy

Étude géologique du glissement de Bema du nord d'Italie

95

P. Lunardi
University of Parma, Italy

P. Froldi
Rocksoil Ltd, Engineering Geological Survey, Milan, Italy

ABSTRACT: A large old landslide has been reactived during the past heavy rainfall occured in summer 1987 in the Valtellina Valley (North Italy).

The landslide, located below the small built-up area named Bema, has caused several problems relating to the security of the same of Bema and of the town of Morbegno, located downstream from the landslide. Measures for risk reduction and site investigation have been planned.

The paper deals with the engineering geological and rock mechanics analyses carried-out to explore the mechanism and the evolution of landslide, and to choose the best remedial measures.

RESUME: un ancien éboulement de grandes dimensions s'est réactivé à l'occasion des pluies incessantes qui se sont abattues l'été 1987 dans la Valteline dans le nord de l'Italie.
Situé sur le versant au-dessous du village de Bema, l'éboulement a causé de graves problèmes à sa sécurité et à celle de la petite ville de Morbegno située en aval de l'éboulement. Des enquêtes géognostiques et le monitoring de la déformation ont été programmés dans le but d'étudier le phénomène d'instabilité et de réduire les risques pour la population. L'article illustre les analyses géologico-géomécaniques menées pour découvrir le mécanisme de l'éboulement, son évolution possible et, à partir de là, les interventions à adopter pour empecher son évolution.

1 INTRODUCTION

The landscape of the Valtellina Valley in North Italy is characterized by a mountainous terrain well known because of past landslide activity.

The most recent catastrophic landslide, the "Valpola landslide" (Fig. 1) was caused by heavy rainfall in July 1987; another landslide occured later. The "Bema landslide" (Fig. 2) was caused by the same reasons.

This paper deals with the engineering geological analysis of the "Bema landslide", an old landslide reactivated in September 1987 and still prone to instability. Renewed movement of the mass has the potential to form a natural landslide dam. The resulting lake would pose enormous danger to downstream population through the flood caused by overtopping or by failure of the landslide dam.

Safety measures consist of an hydraulic by-pass and surface flood

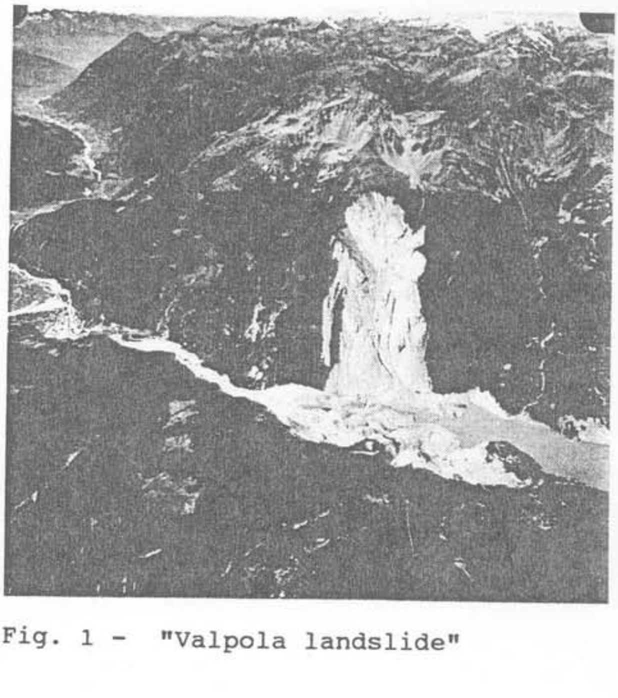


Fig. 1 - "Valpola landslide"

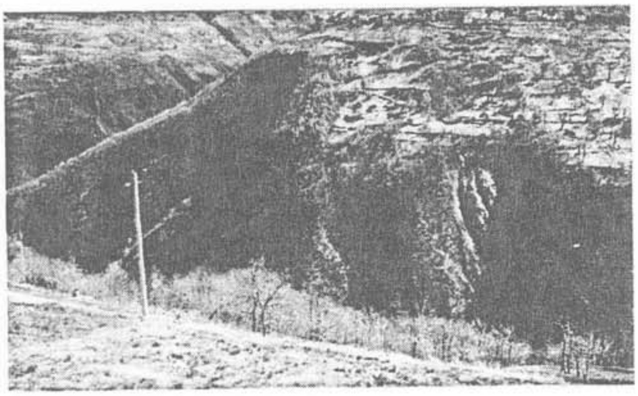


Fig. 2 - "Bema landslide"

control works. The latter involve cutting a rock header on the right of the slide that deflects the stream toward the landslide toe provoking, during the time, strong erosional effects.

A brief assessment of rock cut stability and stabilization has been carried out.

2 GEOGRAPHICAL AND GEOLOGICAL SETTING

The "Bema landslide" is located in the valley of Valgerola Bitto, a tributary of the Adda River (Valtellina Valley), about 150 Km north of Milan (Lombardia, North Italy) (Fig. 3.A).

The Tonale Fault Line, running along the Valtellina Valley (Fig. 3.B), divides the Northern Alps into Northern to Southern geological sectors (Desio, 1973).

A wide range of igneous and metamorphic rocks occur in the region; gneiss, schists and serpentines are common in the southern sector where the landslide occured (Fig. 3.C).

The principal tectonic structures occur along three directions: NW-SE, WNW-ESE and NE-SW (less frequent).

The forces still acting along the region today are responsible for the general tectonic uplift that produces a high relief and strong erosion. Landslides occur frequently over much of North Italy.

One of the larger examples of these phenomena are the "Sackung" or, as known in English words, "deep seated mass rock creep" (Sagging) (Forcella, 1984; Forcella et al, 1985). The authors believe that the knowledge and understanding of "Sackung" and of their features are very important in the study of large landslides in the Northern Alps.

The actions causing the formation of "Sackung" are summarized in a simplified manner in Fig. 4 where the variations of the state of stress into the rock mass are also shown.

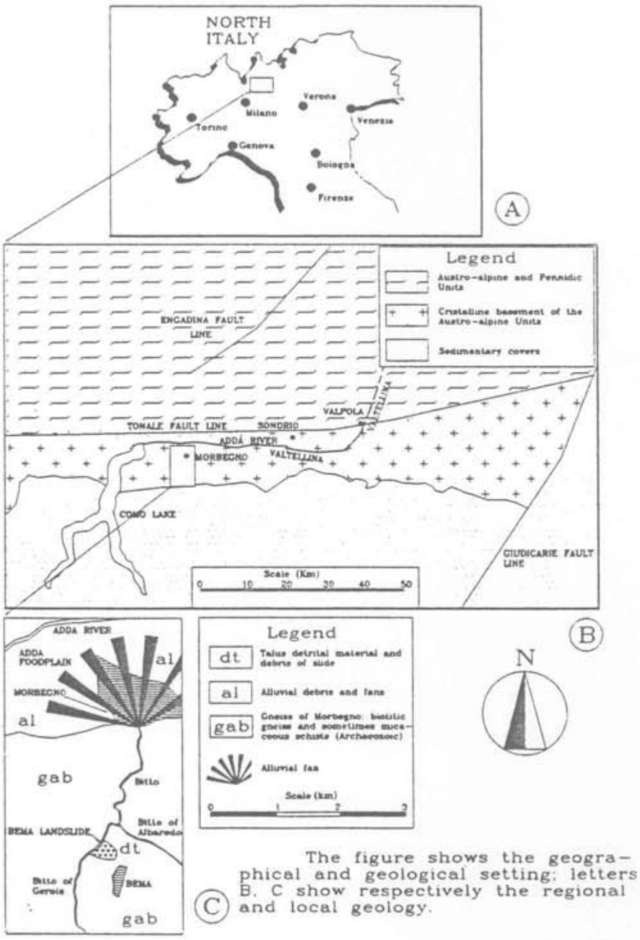


Fig. 3 - Geographical & geological setting

In general, the structural features of "Sackung" (fractures and faults) are causing big landslides (several millions of m^3 of material, Fig. 1, Valpola landslide) often triggered by "rock avalanches".

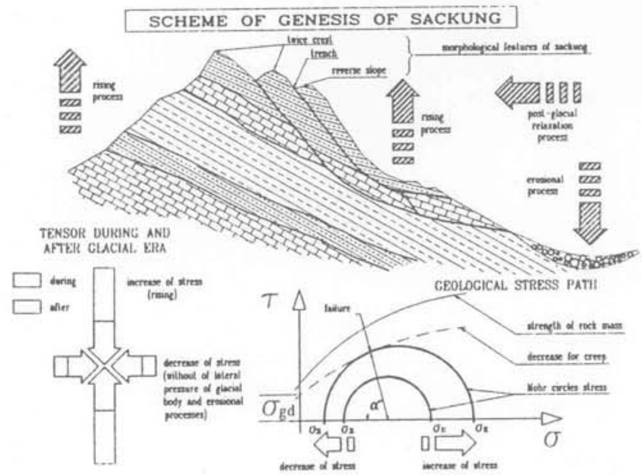


Fig. 4 - Scheme of genesis of Sackung

3 DESCRIPTION OF LANDSLIDE EVENTS

The "Bema landslide" is located in a tributary valley of the Adda River, named the Bitto Valley.

A large alluvial fan occurs at the north of Bitto Valley, on the south margin of the Adda River flood plain; the town of Morbegno is located on the alluvial fan (Fig. 3.C).

About two km south of Morbegno, the valley bifurcates in two distinct valleys: the more eastern is the Bitto of Albaredo Valley and the western is the Bitto of Valgerola Valley where the "Bema landslide" occurs. Near the bifurcation point, an historical landslide can be found; the first landslide event occured in 1872, involving the lower part of the slope.

Witnesses of the historical landslide describe one main collapse followed by minor slides and rock falls. The "Bema landslide" remains unstable with a tendency to retrogress.

Catastrophic rainfall events in July and September 1987 caused the reactivation of the "Bema landslide" (Fig. 2).

About two million m^3 of material has been detached and travelled from the top to the bottom of the slope. Continuing creep of the old landslide body in the same area has resulted in a local filling of bed river.

The authors recognized that a landslide dam could form, resulting in flood hazards for the town Morbegno downstream.

Four types of remedial measures could be undertaken to reduce the risk of damage to Morbegno:

1. remove the landslide dam to prevent the formation of a lake;
2. prevent any erosional processes by flood control works;
3. stabilize the toe of the landslide detrital body;
4. remove the rock header deflecting flow on the toe of unstable slope.

4 GEOLOGY OF BITTO VALLEY

The Bitto Valley is in an area within the Meridional Alps Complex. Locally the outcrops show the crystalline basement composed of metamorphic rocks. Muscovitic schists and biotitic paragneiss, the latter named "Gneiss of Morbegno", are present.

The landscape morphology shows the features typical of an ancient glacial region in rapid uplift; here there are a strong relief, deep valleys such as Bitto Valley, with "V"-shaped profile (fluvial incision) in the lower part and a "U"-shaped profile (glacial incision) in the upper part, and few detrital

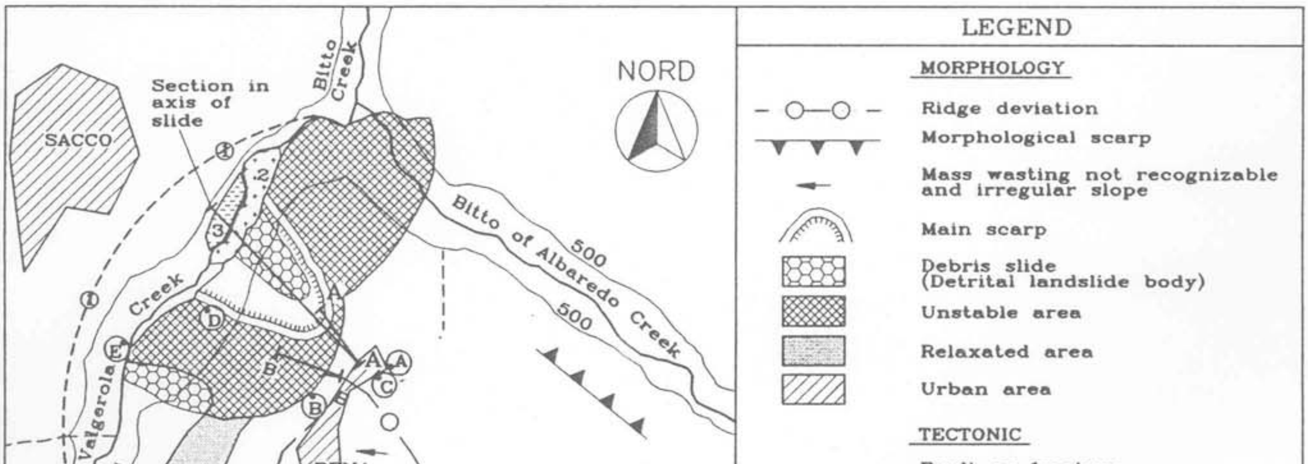


Fig. 5 - Morphological and investigation map

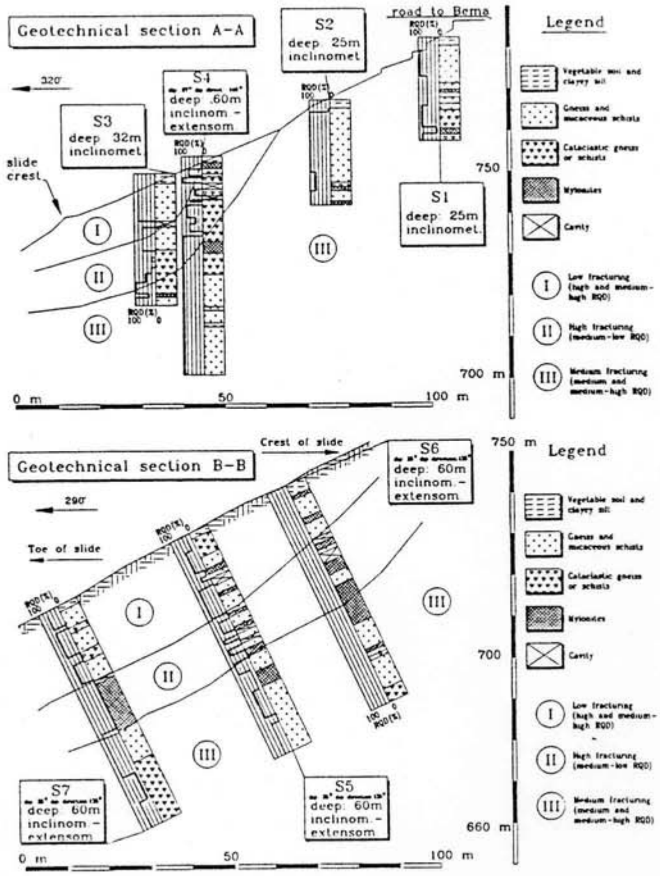
bodies, mainly consisting of morainic deposits.

A field survey has been carried out to locate the main morphological and structural features. It has been found that landslides are more frequent on the eastern side of the valley. In fact, the larger mass-wasting phenomena are located at places where the erosion-resistant rock outcrops deflect water flow toward the toe of the slope.

The strong erosional processes triggered by these phenomena constitute, with the structural features, the principal cause of slope instability. Further studies by the authors have revealed that the rock deflectors are frequently located near fault zones, where tectonically broken material, triggered out, leave the sound rock masses in the header shape.

The landslide site is characterized, from the structural point of view, by a pattern of elements (faults and shear zones) with WNW-ESE alignments (Fig. 5) linked with regional field of fractures.

On the right side of the valley, several relaxed areas have been found, especially near and around the landslide sectors (Sackungen). Locally the schistosity is near vertical with its strike in the direction of N-S with a light trend toward 80°/70° (dip and dip direction).



Figg. 6,7 - Geognostic sections

5 LANDSLIDE INVESTIGATION

The slide body is extended from 380 m to 700 m in altitude for a width of about 300 m at the toe; planimetrically it shows a shape from subtrapezoidal to shell, for an area of about 60,000 m^2. The detrital body occupies the half lower part of this concavity.

Three main kinds of survey have been planned:
- geognostic explorations by bore logging, water pump test and seismic refraction;
- monitoring explorations by inclinometer and rod extensometer devices in boreholes;
- geomechanical-structural recognition by field tecniques.

Fig. 5 shows the site of the surveys. Most of the investigations have been carried out above the head of the slide area to recognize the local underground conditions and to prevent further progressive back-evolution of the slide. The final aim was the knowledge of the safety conditions of overhanging built-up area (Bema).

For the first survey three vertical and four slightly inclined boreholes have been carried out drilled with continuous coring and 5 seismic refraction lines; two geotechnical sections were prepared (Figg. 6,7).

In several cases, during drilling it has been necessary to grout the hole because of a time-expensive loss of circulation water. A large amount of grout mixture was lost in underground cavities. The main reasons of these underground openings can be found in the stress relaxation of rock mass.

Locally there has been identified three conditions, characterized by different degrees of fracturing;
- first, superficial level, thick 15-20 m: RQD generally ranged from 50 to 60%;
- second level: it shows a strong decrease of RQD (often 0-20%) till it shows a mylonitic-cataclastic material; the thickness of this level is about 15 m.
  In this level most of the underground cavities have been found;
- third level or local bedrock (sensu latu); here, it has been found a quick enhancement of RQD value (RQD > 30%) except some zones where the fracturing is at a higher level (RQD < 20-30%).

From these data it has been possible to visualize a realistic scenario of slide evolution: the fracturing conditions increase toward actual upper zone of slide and sliding surfaces in sound rock mass are considered unlikely. Thereby it has been understood that the mechanism of failure could not be any sliding movement instead of what stated by the reports available at the time (Pozzi & Sfondrini, 1981). It was clear that another mechanism of failure was to be found.

The second kind of survey, seismic refraction, did not add any further information: the seismic velocities of compressional waves in the rock beneath the detrital level (4-5 m) could not be distinguished. The velocities ranged from 1,7 to 1,9 Km/sec, indicating a strongly relaxed rock mass.

Useful information was provided by structural field analysis carried out in several points of the area.

The structural data, dip and dip direction of discontinuity planes, have been processed by microcomputer (Fig.8).

Three main families of discontinuities have been found: Sc (schistosity), K1 and K2 (fractures). The borehole structural data show the same orientations, confirming the structural pattern recorded in outcrops near the slide area. In this way it has been possible to make a final assessments of the mechanism of mass wasting: a slide triggered by rock-toppling phenomenon as can be observed in a the sketch (Fig. 8).

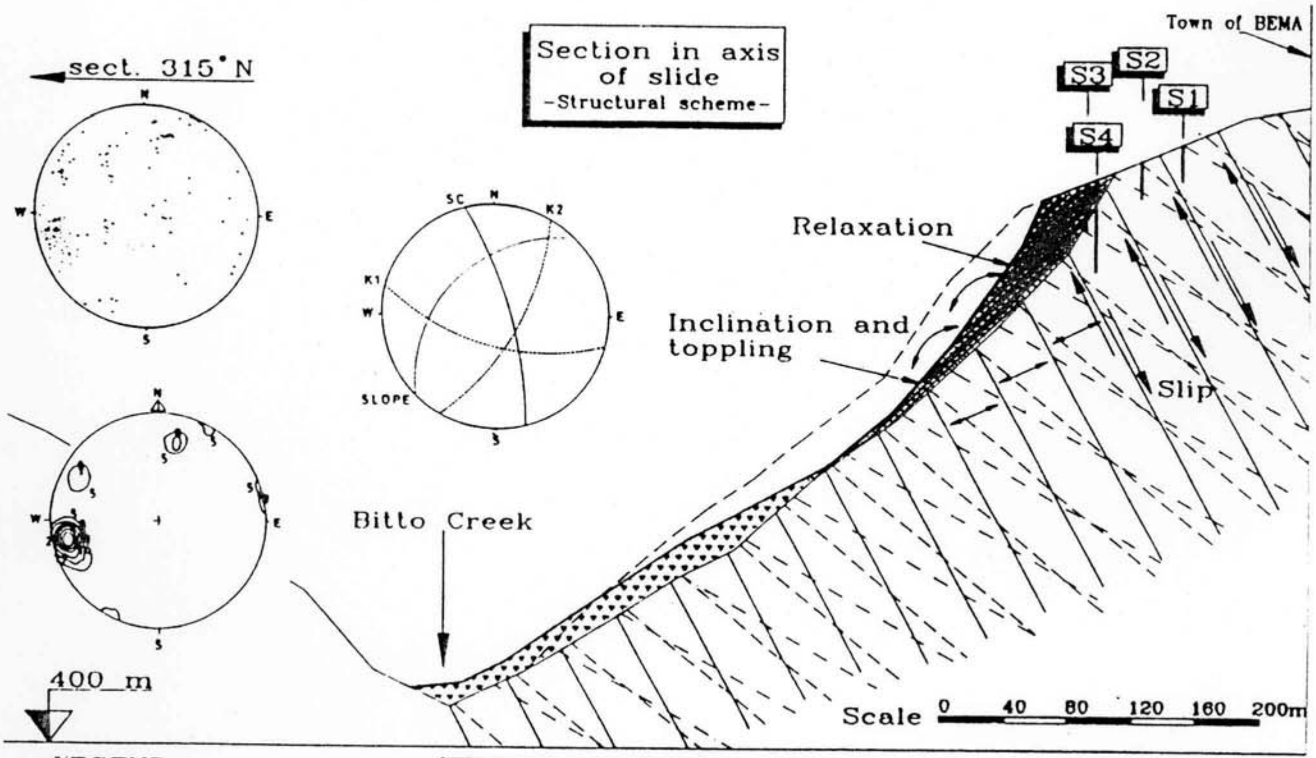
Keeping in mind the structural features, it is possible to draw an engineering analogy, that the behaviour of the rock is similar to that in beam deflexion (Fig. 8).

After the initial failure (tensile stress) the mass wasting becomes a rock fall from the upper part of slide, while in the lower part the whole detrital body becomes a jumbled blockly rock mass. The size of these blocks is very variable; they ranged from more than one meter to small gravel and sand.

The lithological composition of the landslide mass is not only metamorphic but also morainic and alluvial.

6 RISK-REDUCTION MEASURES

To avoid other risks for the town downstream from the slide, a multipurpose hydraulic project has been planned after removal of the mass slide away from the bed of Bitto Creek. The long-time safety project against natural hazards has been planned through the following remedial works (Fig. 5):



LEGEND

—— Sc
– – – K1
- - - K2

Slide body
Relaxation zone
Low fracturing

Geomechanic features						
Set	Dip	Dip D	Spa.(m)	Fill.	JRC	JCS
Sc	75°	75°	0.30-0.50	Silt	4-8	50-80
K1	60°	195°	0.20-0.75	Nothing	8-16	30-100
K2	65°	120°	0.40-0.60	Nothing	8-10	80-90

Drilling bore-holes		
Number	Deep (m)	Devices
S1	20.05	Inclinom.
S2	25.20	Inclinom.
S3	32.10	Inclinom.
S4	60.00	Incl.-Ext.

Fig. 8 - Section in axis of slide

- direct measures:
  - hydraulic by-pass to divert possible overflow away from the toe of slide;
  - geotechnical works at the toe of the slide to increase its stability (loading the toe, by ground improvements and retaining structures);
- indirect measures:
  - cutting of rock header to create a new way for water flow minimizing further erosional process at the toe of the slide;
  - canalization and regulation of Bitto Creek into artificial channel to minimize erosional damages and improve the flow;
  - surface drainage works above and around the upper contour of the slide to minimize damaging infiltration of surface water into a highly pervious fractured mass.

The remainder of this paper deals in particular with the rock-mechanics aspect of cutting the rock header and the related stabilization works.

Initially it was believed that no strong and continous plane of weakness existed throughout the rock mass. However cleaning of the slope and the rock surface revealed a very continous surface of weakness.

Because the designed rock-cut is, for geometrical reasons, steeply inclined (≈ 80°), rock reinforcement was necessary to assure the stability of the new cutting slope. To improve efficiency and decrease the costs of these reinforcing works, great emphasis was put on the engineering geological survey.

7 ENGINEERING GEOLOGICAL SURVEY AND ROCKSLOPE DIAGRAM

The geometrical and geomechanical properties of discontinuities in the rock header are listed in Table 1.

In particular, it has been observed nine principal structures named J1 through J9 that are important for their orientation and persistence.

The orientations of the geological structures show high scattered pattern typical in such lithological environment (paraschists and gneiss).

By stereographic structural analysis the kinematically possible failures have been identified.

The three-dimensional mode of failure predicted by the analysis was a wedge failure on two discontinuity planes. Wedge failures of this type have been analyzed by Hoek & Bray (1977).

To estimate the strength along the discontinuities a simple back-analysis of wedges recognized in situ has been carried out.

Table 1. Geometrical and geomechanical properties of planes of weakness

JOINT SET OR SINGLE JOINT	Dip, Dip Dir. (°)	Persistency (m)	Frequency (n°/m)	Opening (cm)	Filling	JCS (MPa)	JRC
MainFault J1	45/70	>10	/	10 - 50	Catacl. fault breccia	/	/
Faults	45/70	Several m (3 - 5)	0.2 - 0.5	0 - 10	Catacl. or clay	/	/
K1	70/150	Several m (2 - 4)	0.2 - 0.5	closed	rare	50-70	6-12
K2 Schistos.	85/300	Several m	3 - 5	closed	rare	40-60	4-8
J2	85/85	>10	/	closed	/	/	/
J3	30/70	>10	1 - 2	closed	/	/	/
J4	55/230	>10	0.4 - 0.5	closed	/	/	/
J5	40/70	>10	0.4 - 0.5	closed	/	/	/
J6	85/125	>10	0.6 - 1	closed	/	/	/
J7	55/145	>10	/	closed	/	/	/
J8	50/155	>10	1	closed	/	/	/
J9	40/70	>10	0.4	closed	/	/	/

Hoek & Bray define the safety factor of the wedge, neglecting the existence of tension-crack, as:

$$FS(3d)=SIN\beta/(SIN1/2zeta)\cdot TAN\phi/(TAN\alpha) \quad (1)$$

where the symbols used are listed in Fig. 9 and FS (3d) is the safety factor in three-dimensional analysis.

The two terms can be simplified assuming:

$$SIN\beta/(SIN1/2zeta)=K \quad (2)$$

and

$$TAN\phi/(TAN\alpha)=FS(2d) \quad (2)$$

and substituting equations (2) into (1);

$$FS(3d)=K.FS(2d) \quad (3)$$

where FS(2d) is the safety factor in two-dimensional analysis.

In a conservative way, the structural data have been processed by the safety factor just described to seek, by a back-analysis, the shear strength of discontinuities. The following simplified assumptions have been made:
- FS(3d) equal to unity for wedge recognized in field;
- only frictional shear strength;
- absence of water table and dynamic forces.

The geometrical features of wedges and the results of the back-analysis of factors of safety are summarized in Tables 2 and 3.

A strength criterion depending on surface roughness of rock planes has been assumed (Patton 1966, Rengers, 1970, Barton, 1971, 1973, 1988, Barton & Bandis, 1980, Barton & Choubey, 1977).

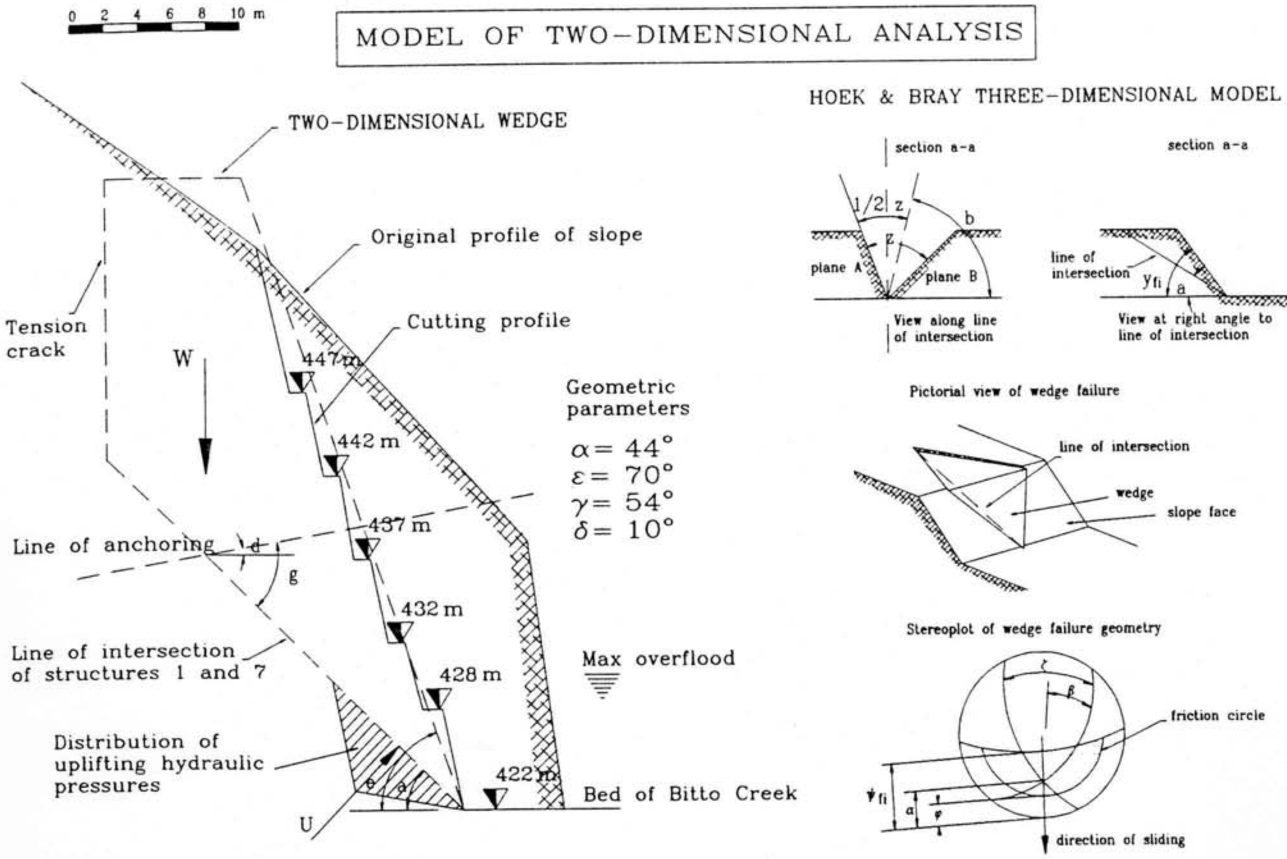


Fig. 9 - Model of two-dimensional analysis

Table 2. Geometrical properties of wedges

WEDGES	STRUCTURES	psi fi	α	β	1\2 zeta	FS(2d) TAN φ	K
1	1,7	78°	44°	77°	62°	1.143	1.103
2	5,7	78°	39°	76°	62°	1.357	1.098
3	5,8	80°	37°	86°	61°	1.514	1.141
4	7,3	78°	30°	68°	60°	1.854	1.071
5	3,8	80°	28°	71°	62°	2.014	1.071
6	1,8	82°	40°	86°	60°	1.373	1.152

Table 3. Results of back - analysis

WEDGES	FS for $\phi = 23°$	FS for $\phi = 25°$	FS for $\phi = 30°$	Fs for $\phi = 35°$
1	0.485	0.532	0.660	0.800
2	0.576	0.632	0.783	0.950
3	0.642	0.706	0.874	1.060
4	0.785	0.865	1.070	1.298
5	0.855	0.939	1.163	1.410
6	0.582	0.640	0.793	0.961

Moreover, processing the geometrical features of joints by Barton's theory (1973) and Tse & Cruden (1979) analysis (Z2 parameter) (Froldi & Mantovani, 1990), for large scale problems it was found that:

$JRC_{(l)} = 9 \div 12.5$
$JRC_{(n)} = 3 \div 6$
$\phi_{(b)} = 23° \div 25°$
$JCS = 50$ Mpa

where:

$JRC_{(l)}$=laboratory value
$JRC_{(n)}$=in situ value
$\phi_{(b)}$=basic friction angle
JCS=Joint Compressive Strength

The parameter JRC has been found by

$$JRC=32.2+32.47\log Z2$$

$$\text{where } Z2=[1/M(Dx)^2 \sum_{i=1}^{m}(y_{i+1}-y_1)^2]$$

The values that appear in this expression have been identified by geometrical analysis (Tse & Cruden, 1979) of roughness profiles.

To find JRC for natural scale (Bandis, Lumsden & Barton, 1981) the following equations has been applied:

$JRC_{natural\ block}/JRC_{laboratory} =$

$\bar{a}^{\circ}_{natural\ block}/\bar{a}^{\circ}_{laboratory}$

and since

$$JRC_{nb} = \frac{\bar{a}^{\circ}_{nb} \cdot JRC_{lab}}{\bar{a}^{\circ}_{lab}}$$

where the meaning of the parameters M, y, and a° are described by Tse & Cruden (1979) and Bandis et al. (1981).

The main parameters found by back-analysis reveal a good approximation to those described above. Appropriate strength parameters can be estimated by Barton's law:

$\tau_{mob}=\sigma_n \cdot TAN\phi_{mob}=$

$\sigma_n \cdot TAN[\phi_b+JRC \cdot (\log(JCS/\sigma_n))$

$\phi_{mob}=[\phi_b + JRC(\log(JCS/\sigma_n))]$

The Hoek & Bray two-dimensional analysis gives reasonable results because K is near unity and FS(3d) is approximately equal to FS(2d).

To find the design anchoring force, the equation of Londe (Londe, 1965, Londe et al., 1969) has been taken, as suggested by Hoek & Bray:

$$T(2d)=FS(2d) \cdot T_d-T_r\ /(SIN\ gamma \cdot TAN\phi_{mob}+ +FS(2d) \cdot COS\ gamma) \quad (4)$$

for active forces where:
T(2d) = anchoring force for two-dimensional analysis
gamma = angle between line of anchor and sliding plane
T_d = driving force along sliding plane
T_r = resistance force along sliding plane

Equation (3) can be rewritten

$$FS(2d) = \frac{FS(3d)}{K} \quad (5)$$

and substituted into equation (4) to yield:

$$T(3d)=((Fs(3d)/K) \cdot T_d-T_r)/(SIN\ gamma \cdot \cdot TAN\phi_{mob}+(Fs(3d)/K) \cdot COS\ gamma) \quad (6)$$

or

$$T(3d)=Fs(3d) \cdot T_d-K \cdot T_r/(K \cdot SINgamma \cdot TAN\phi_{mob} +FS(3d) \cdot COS\ gamma) \quad (7)$$

for active forces.

The first analysis is made to assess the approximate intensity of reinforcing anchors for a kinematically possible wedge failure.

This method makes it possible to estimate quickly and simply the anchoring forces for local conditions, decreasing excessive conservation in the design calculations.

8 SUMMARY AND CONCLUSIONS

A major landslide (about 2 million of cubic metres), failed in a tributary valley (Bitto Valley) of Valtellina (North Italy) during heavy rainfall in Summer 1987, forming a natural dam across the Bitto Creek.

Immediately it was realised that enormous risks were created for the downstream town of Morbegno.

A detailed engineering geology study was performed to understand the slide and to plan remedial measures.

Engineering geological survey and field analysis have been able to document and quantify the main features of landslide; in particular, erosion of the toe of the slope due to stream flow directed by a rock header has been identified as one of the main causes for slope failure.

In excavating the rock header, a rapid two-dimensional analysis to resolve a three-dimensional problem has been developed.

ACKNOWLEDGEMENTS

We thank very much all those that have helped in the preparation of this paper, in particular Dr. Ing. Sannazzari, Technical Director of CogefarImpresit, the Company executing these works, to what we are very grateful for permission to publish the data.

REFERENCES

Barton N., 1971. "A relationship between joint roughness and joint shear strength" Proc. Int. Symp. on Rock Mechanics. Nancy 1-8.

Barton N., 1973. "Review of a new shear strength criterion for rock joints" Eng. Geol. 7: 287-332.

Barton N., 1988. "Some aspects of rock joint behaviour under dynamic conditions" 2° Ciclo di conferenze di meccanica e ingegneria delle rocce, MIR 88, Politecnico di Torino, 17: 1-14.

Barton N. & Bandis S., 1980. "Technical note: some effects of scale on the shear strength of joints" Int. J. Rock Mech. Min. Sci. & Geomech. Abstr. 17: 69-73.

Barton N. & Choubey V., 1977. "The shear strength of rock joints in theory and practice" Rock Mech. 10: 1-65.

Bandis S., Lumsden A.C., Barton N., 1981. "Experimental studies of scale effects on the shear behaviour of rock joints" Int. J. Rock. Mech. Min. Sci. & Geomech. Abstr. 18: 1-21.
Desio A., 1973. "Geologia dell'Italia", UTET.
Farmer I., 1983. "Engineering behaviour of rocks" Chapman & Hall, London.
Forcella F., 1984. "Brevi note sulla tettonica gravitativa di versante nelle Alpi Centrali" Boll. Soc. Geol. It., n° 103, pagg. 689-696.
Forcella F., Gallazzi D., Montrasio A., Notarpietro A. 1985. "C.N.R.: Progetto Finalizzato Geodinamica - Sottoprogetto neotettonica".
Froldi P. & Mantovani S., 1990. "An experimental-statistical approach to study the shear strength of rock joints", Proceeding Sixth International Congress of International Association of Engineering Geology - 6/10 August 1990 Amsterdam - Netherlands".
Hoek E. & Bray J.W., 1977. "Rock slope engineering. Istitution of mining and metallurgy, London.
Londe P., 1965. "Un méthode d'analyse a' trois dimensions de la stabilitée d'une rive rocheuse" - Annales des Ponts et Chausseés - Paris, 37-60.
Londe P., Vigier G. and Vormeringer R., 1969. "The stability of rock slopes, a three dimensional study" - J. Soil Mech. and Foundation Div. ASCE.
Patton F.D., 1966. "Multiple mode of shear failure in rock" 1° Congress Int. Soc. Rock Mech., Lisbona, 1: 509-513.
Pozzi R. & Sfondrini G., 1981. "Caratteri generali della franosita' in provincia di Sondrio", C.N.R., Fondazione per i problemi montani dell'Arco Alpino, Pubbl. n° 88.
Tse R. & Cruden D.M., 1979. "Estimating joint roughness coefficients" Int. J. Rock Mech. Min. Sci. & Geomech. Abstr., 16: 303-307.
Rengers N., 1970. "Influence of surface roughness on the friction properties of rock planes" Proc. 2° Cong. of Int. Soc. for Rock Mechanics, Belgrade, 1: 1-31.