The stabilization of a landslide in a slope of weathered phyllites

P. MAZZALAI *, M. SORANZO **

SUMMARY: The examination of a landslide and the means for its stabilization are presented in the paper. The design choice of structural elements such as anchors, bored micropiles and gabion walls is justified and is related to the geomorphological conditions and to the geotechnical properties of the weathered phyllitic formation present in the site as well as to the safety factor increase.

1. Introduction

A sliding movement, started in 1965 in a built up area in Vetriolo (Province of Trento, Italy), is described in this paper.

An important throughfare and part of the town center were involved in this phenomenon causing serious trouble in the whole area. The mechanical parameters of the materials, involved in the slide, have been determined from direct shear, ring and triaxial tests also of the multistage type. With this investigation peak, remoulded and residual parameters of the shear strength were defined and allowed to interpret the original stability conditions of the slope. The interventions on the slide (i.e.: tie-roads, micropiles, gabion walls), designed and carried out in 1977 and 1978 are reported herein: they have made possible an almost complete stabilization of the urban area and its reutilization.

The effects of the different stabilization technologies are quantified and considered in relation with the safety factor increase.

2. Description of the site

The examined slide took place on a mountain slope located to the south of the health resort, famous for its mineral waters (Fig. 1).

The average elevation of the sliding movement is 1450 m. According to SKEMPTON and HUTCHINSON's classification [1969], it can be described as a translation slide of the slab type.

It possesses the following geometrical features: length =250 m, width = from 40 to 60 m, depth = from 3 to 8 m, slope angle from 25 to 35 degree. The volume of the entire sliding mass is about 55000 m^3.

* Engineer - Agricoltural Faculty - University of Padua.
** Engineer - Engineering Faculty - University of Padua.

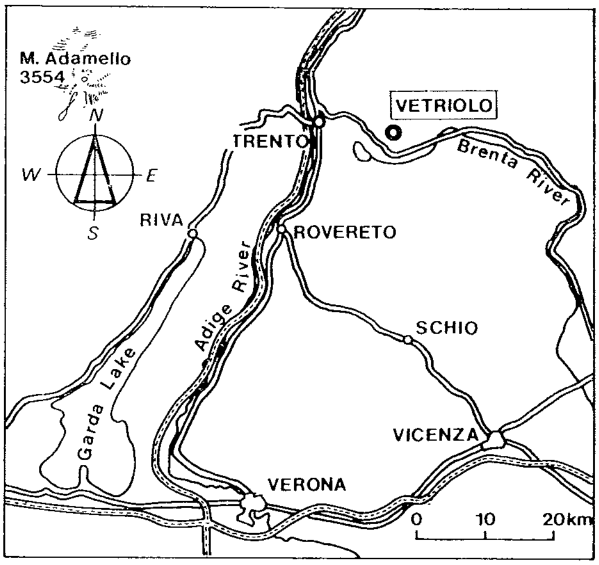

Fig. 1. - Location map of the examined area.

3. Site and geological conditions

The Vetriolo area is situated in the western part of the so-called « crystalline of Cima d'Asta », which is a well-definite geological unit in this southern Alps region.

In this area phyllitic lithotypes emerge, they represent the upper unit of the metamorphitic succession of the southern alpine crystalline basement.

The lower unit consists of paragneiss with associated porphyroides.

Vetriolo phyllitic formation is essentially characterized by quartziferous phyllades and inserted among them, are sericitic, more or less carbonaceous and chloritic phyllites.

The phyllites are often crossed by dykes of basic rocks with irregular courses, frequently associated with hydrothermal manifestations, fluorite and metallic sulfides.

The phyllites present, from a tectonic point of view, a schistosity with a dip direction towards NW and W, this is often disturbed by several fractures and/or fault systems.

These non connective elements can be grouped, according to a chronological order, into:

— irregular mineralized fractures, most likely due to volcanic-tectonic relaxation;

— diaclase and faults, due to orogenic subvertical phenomena and correlated with Valsugana which is considerably tectonicly disturbed in the NW-SE and NE-SW directions;

— gravity-slip-faults, sub-parallel to the slope.

4. Landslip description - Brief history of the phenomenon

The Vetriolo sliding phenomenon, locally known as the « Villa Trento slide » has caused since 1965 serious trouble in the whole area: damages, fissures and cracks to several constructions (Villa Trento, Giori house, retaining wall, sewer system, provincial road). The intensity of the motion seemed to be in relation to the local rainfall. This appened up until 1977-1978, when a decisive intervention of stabilization took place.

As reported by ANDERLE *et al.* [1981], the Vetriolo slope presents a part of the phyllitic metamorphic series which is easily degradable as can be seen from the many areas covered by phyllitic debris which are shown in figure 2.

Substratum weathering leads to the formation of a superficial stratum of a various thickness more or less cohesive, however with poor mechanical properties.

This layer, thicker in compluvium areas, presents sharp movement increases when thaw or plentiful rains occur.

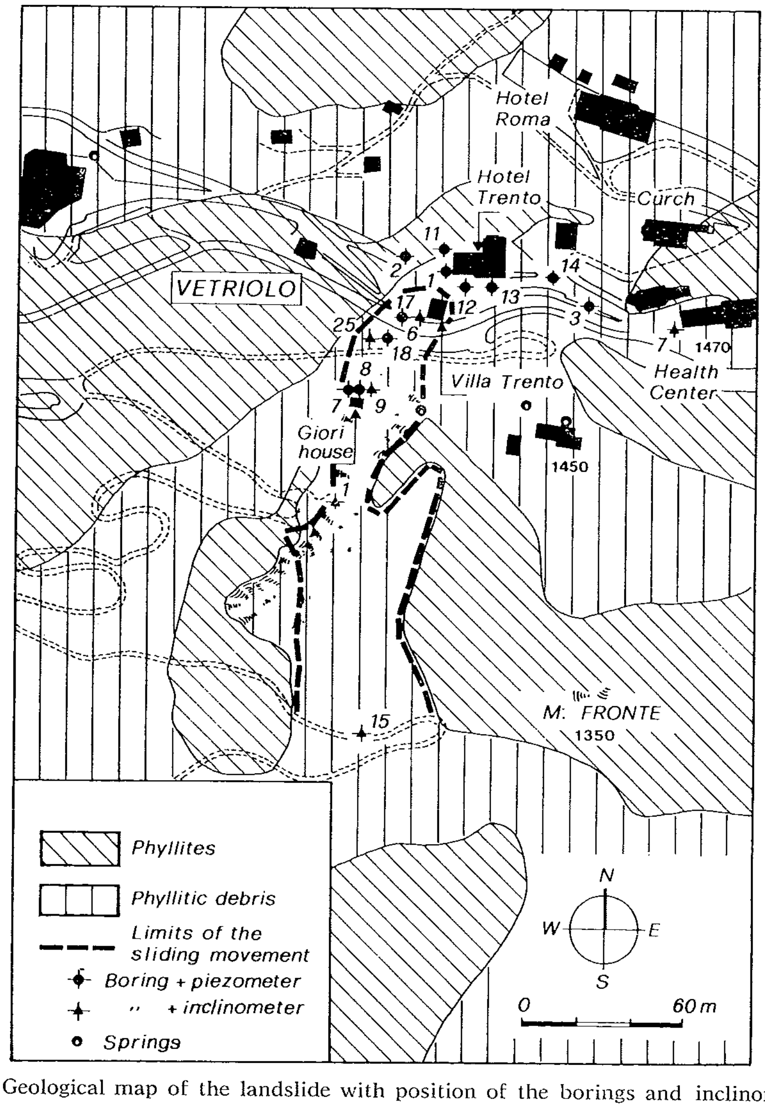

Fig. 2. - Geological map of the landslide with position of the borings and inclinometers.

In this particular case several landslides (of various extension) happened in a rather limited area: « Health center », « Tabaccheria », « Albergo Panarotta » are the local denominations of the various sliding sites.

All the mentioned landslides developed slowly and involved buildings and roads.

Not long ago (1976), following the Friuli earthquake, which devasted part of North-Eastern Italy, a large phenomenon started, with landslides in the Vetriolo built-up area and more limited landslides in the forest located under the town.

A short chronicle of « Villa Trento » lanslide development and stabilization is reported herein:

— spring 1965: a first settlement of the provincial road and nearby buildings was observed, during a rain and thaw period. The road was repaired with a reinforced concentre arch slab acting as a retaining wall;

— autumn 1966: the whole of the Trento province suffered a violent flood. Plentiful rainfalls caused a collapse in the sewer system retaining wall displacement, Villa Trento differential settlements, Giori house translation;

— 1968-1973: drainages and retaining walls were built into the upper part of the landslide and surface drains in the lower part;

— 1975: the slope movement starts again (due to a great increase in the rainfall); this continued until autumn 1976. During this period the upper of the landslide clearly involved Villa Trento;

— 1976-1979: definitive stabilization interventions are designed and carried out:

— drainages, in the upper part of the landslide that are able to intercept flowing water;

— reinforced concrete double beam, linking a series of micropiles and tie-rods: the lower beam also supported the gabion wall, which, in turn, holds up the provincial road embankment;

— retaining gabions and surface drainage in the lower part of the landslide (below the Giori house).

Before carrying out this work, an extensive investigation program took place both in the site and in the laboratory (the test results of which are reported below).

The micropiles construction started at the beginning of July 1977, and it was anticipated by large movements of the reference point situated on Villa Trento garage. This particular point moved 9 cm, along the landslide axis, the week before the stabilization was carried out, and only 5 cm, in the following 5 months.

The mass-movement was entirely stopped, around the main road and the Villa Trento, at the end of the intervention.

On the contrary the sliding movement has continued its motion on the lower part of the landslide, cutting (in Autumn 1977) two inclinometers situated between the road and the Giori house.

In 1978 the stabilizing effects of drainages and gabions, carried out on the lower part of the landslides, were noticeable.

Inclinometric measurements, taken near the Villa Trento, confirm the intervention validity, as much as the movements of the upper part of the landslides were equal to 1.9 cm in June '78 - December '78 period, and to 3 mm or less in March-June '79 period.

At the present time the movement is nearly equal to 1-3 mm/year.

The zone of the main stabilization intervention, given by the combined effects of tie-rods, micropiles and gabion wall, as well as the position of a drainage gabion wall inserted in the middle part of the landslide is reported in the general plan (fig. 3).

5. **Field investigation**

Since 1977 systematic field studies of the slip area and surrounding site have been undertaken in several stages.

The final investigation consisted of three ground surveys, which were repeated in May 1977, June 1978 and June 1979, two trial pits and seventeen continuous borings in which ten undisturbed samples were obtained with the Osterberg's piston sampler.

A first important finding was that the depth of the weathered sheets decreased from 8 to 3 m going from higher to lower elevation within the landslip.

Ground elevations and displacements were measured before and after the construction of the remedial works. In this period of time (first 8 months of 1977) a maximum displacement of 12 centimeters along the axis of the landslide has been recorded.

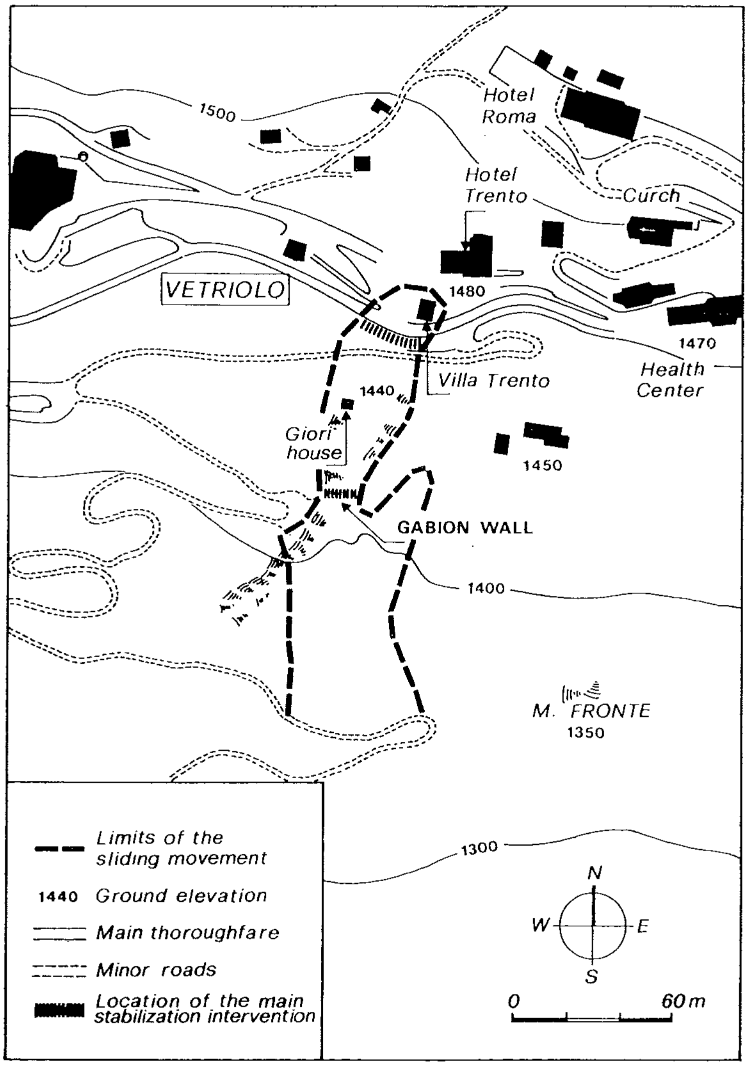

Fig. 3. - General plan of the landslide with position of the stabilization works.

Displacements of this order have caused the failure of two inclinometers (n. 25 and 9) at the depth of 4 and 5 m below the ground level (see fig. 2).

The greatest instability phenomena were located near the head of the slip that is they are in relation to the thicker part of the alluvial sheet.

6. **Properties of the weathered soil**

The rocky formation (sericitic phyllites) is covered by the « in situ » soil which is its weathering product.

Visually it appears as a chaotic compound of lithic jointed elements of various dimensions (which can be cut through the joints with a knife) which are included into a finer matrix of brown-silver soil.

This matrix is quite slippery to the touch but it does not seem to possess significant plasticity.

The soil classification tests (according to BS : 5930-1981) give a grain size distribution ranging from a coarse gravel to clay.

The fuse of the 12 grain size tests performed, which reveals a well graded soil, is reported in fig. 4.

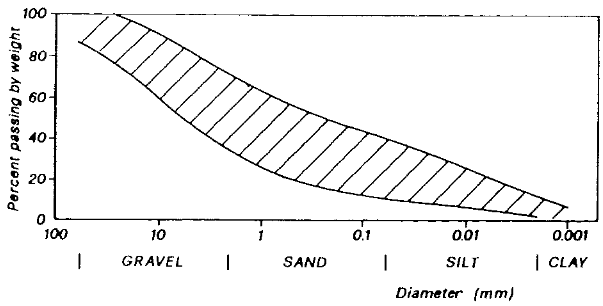

Fig. 4. - Grain size distribution range for a set of 12 specimens of weathered phyllites.

The grains smaller than the No. 40 sieve are silty and clayey soil of low plasticity (ML-CL) that is they form from 20 to 55 percent in weight of the overall samples.

The arithmetic mean values of the index properties were found to be LL = 29.5 percent and PL = 22.8 percent, their standard deviation were 2.4 and 2.5 respectively.

The natural moisture content ranges from 5 to 15 percent and these low figures are due both to uncomplete saturation and to the granular part of the entire soil samples to which these values refer.

The unit weight of soil has a mean value of $22 kN/m^3$ and has been evaluated on samples of which the weight was greater than 35 N.

Furthermore at the interface of the weathered eluvial sheet with the bedrock the soil results to be more homogeneous and the coarser grains disappear.

In this zone ground water is present and has an elevation of some tens of centimeters.

The Coulomb's failure equation

$$\tau = c' + \sigma' \cdot \tan \varnothing' \qquad (1.1)$$

was used to define the maximum and residual shear strength and to interpret shear tests performed on samples obtained from trial pits and borings.

At the geotechnical laboratory of the University of Padua, consolidated — undrained triaxial compression tests were carried out on samples bored in the failure zone which was indicated by the inclinometric-readings.

Because of the complex soil-fabric it has not been possible to take specimens with a diameter smaller than the sampler diameter (D = 10 cm).

In order to obtain some further information a triaxial compression was carried out by means of the multi-stages technique; two shearing stages followed two consolidation stages on the same specimen [KENNEY and WATSON, 1961; ANDERSON, 1974, I-II]. It was possible to prepare only four specimens in a set of fifteen samples because of the soil brittle nature.

The stress-strain curves are reported in fig. 5. All the specimens underwent a saturation process with back pressure ranging from 100 to 150 kN/cm^2 in order to reach a value $B = \Delta u/\Delta\sigma_3 = 0.95$ prior to the shearing stage.

The entire set of triaxial compression were

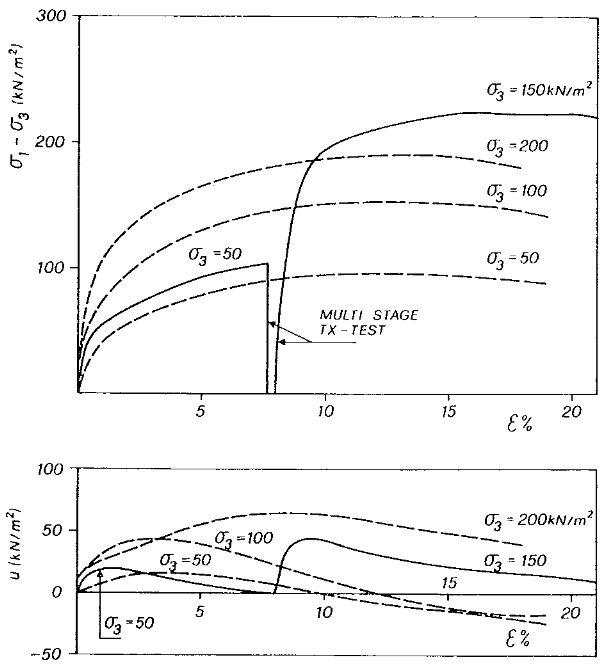

Fig. 5. - Stress-strain curves for triaxial compression C.I.U. test performed on specimens drilled within the shear zone.

interpreted by means of the normalized failure criterion:

$$\frac{\sigma_1 - \sigma_3}{\sigma'_3} = \text{Maximum} \qquad (1.2)$$

determined from the stress paths of fig. 6.

The interpretation of values of relation 1.2 (i.e.: the tangent through the origin to the curves of fig. 6) enables us to evaluate the shearing stress relation of this partially softe-

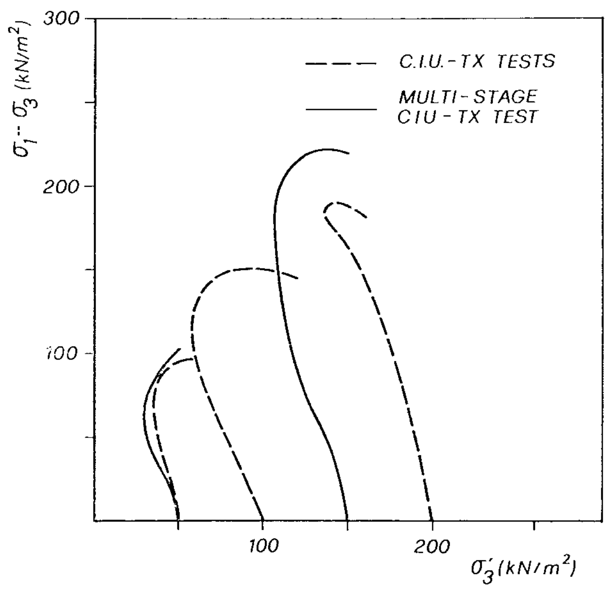

Fig. 6. - Deviator stresses vs. effective confining stresses for the triaxial compression C.I.U. tests.

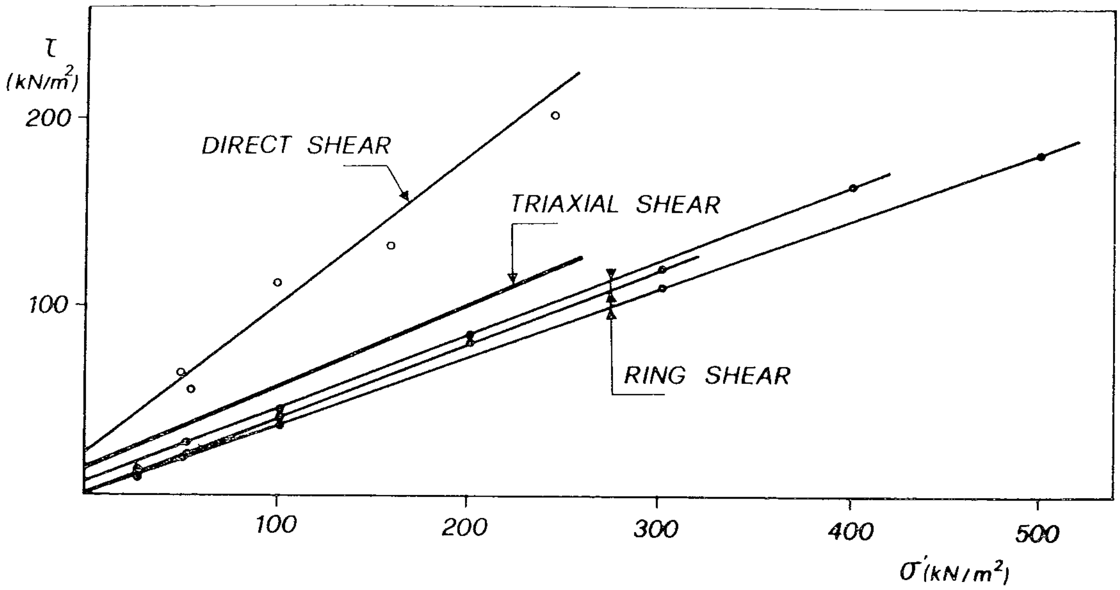

Fig. 7. - Summary of the failure envelopes determined with the various shear tests performed on specimens drilled within or above the shear zone.

ned soil which, in terms of effective stresses is expressed by:

$$\tau_1 = 14 + \sigma' \cdot \tan 22^\circ \quad (\mathrm{kN/m^2}) \quad (1.3)$$

The less weathered soil above the shearing zone was undoubtedly more resistant; the shear parameters defined in drained conditions [GIBSON and HENKEL, 1954] by means of Casagrande's shear box (diameter of the box = 10 cm) were found to be:

$$\tau_2 = 22 + \sigma' \cdot \tan 38^\circ \quad (\mathrm{kN/m^2}) \quad (1.4)$$

At last the BROMHEAD's anular shear apparatus [1979] was utilized to measure the residual strength. This determination was performed on remoulded soil smaller than the No. 40 sieve and gave, for soil both within or above the shearing zone:

$$\tau_r = 4 + \sigma' \cdot \tan 20.5^\circ \quad (\mathrm{kN/m^2}) \quad (1.5)$$

Due to well known non linear behaviour of the residual shear stress with increasing normal stress, the equation (1.5) refers to $\sigma' = 88$ kN/m² that is equal to the average stress normal to the shear surface [COTECCHIA and FEDERICO, 1980; PICARELLI, 1980; LUPINI *et al.*, 1981]. The results of all the shear tests carried out with the procedures and equipments described above are summarized in fig. 7.

7. Stability analysis

A « post mortem » analysis of stability has been performed utilizing the BELL's method [1969] which has been introduced in a computer programme [BARLA *et al.*, 1974; COLLESELLI *et al.*, 1979]. This method of stability analysis is based upon the limit equilibrium assumptions and refers to the equilibrium of forces and moments of the full mass which slides over a surface of general shape. The water table was taken to be from 0.5 to 1.0 m above the sliding surface as pointed out by the few measures recorded from the piezometers before their cutting off.

Considering the full slide, as defined by slope indicators, and the shear parameters given by the equations (1.3), (1.4), (1.5), the results of the stability computations are reported in Table I.

TABLE I
STABILITY ANALYSIS OF THE FULL SLOPE

Shear parameters c' (kN/m²)	Ø' (degrees)	Position of specimen in the soil mass	Type of specimen	Safety factor
c' = 14	Ø' = 22	within the shearing zone	undisturbed	1.03
c' = 22	Ø' = 38	above the shearing zone	»	1.90
c'_r = 4	Ø' = 20.5	within the shearing zone	remoulded	0.77

The extremely different values of the safety factors associated with the various parameters give an idea both of the progress of the weathering process and remouldment within or above the shearing zone and of the importance of considering the complete samples (i.e.: from gravel to clay) when studying the strength of a soil of this type. Furthermore the strong difference in strength given by different types of tests performed on intact specimens can not be explained by means which are related only to the test type and test technique and thus must be somehow related to the partial remoulded soil and to the weathering process of the soil mass at different levels in the same mass. These preliminary analysis allowed us to choose equation (1.3) as the one representing the shear strength acting on the sliding surface at failure.

Figure 8 shows that the entire sliding mass was in a state of very low stability which has been defined through the ratio τ_l/τ_{ss} where τ_{ss} is the shearing stress acting on the assumed shear surface and $\tau_l/\tau_{ss} = 1$ defines failure.

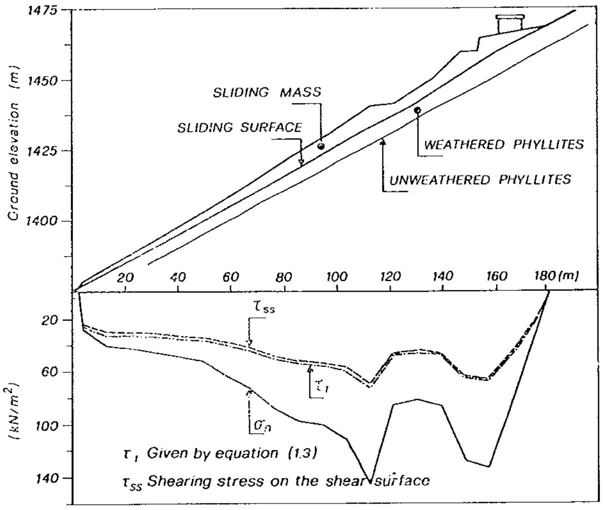

Fig. 8. - Longitudinal section of the landslide. Stresses on shear surface.

8. **Stabilization methods**

The intervention design was concentrated on the upper part of the slide since that was where the most important buildings and the main road were localized.

It was decided to choose a flexible and combined stabilization system also capable to hold the 7 m high road-embankement.

The stabilization work is composed of tie rods and bore micropiles both reaching the unweathered rock formation and connected between themselves with a reinforced concrete beam which holds a gabion wall.

The gabion wall has been preferred to other types of stiffer retaining structures since it could bear greater horizontal and vertical deformations of its substructures (i.e.: the connection concrete beam).

The main characteristics of the constructed work are reported in fig. 9.

Each anchor, which has an inclination of an angle α with the sliding surface, was prestressed with a traction force R giving a stabilizing force S parallel to the shear surface, equal to:

$$S = R\left[\sin\left(\frac{\pi}{2} - \alpha\right) + \right.$$
$$\left. + \cos\left(\frac{\pi}{2} - \alpha\right) \tan \varnothing'\right] \qquad (2.1)$$

The inclination $\alpha = 60°$ of the anchors, that is about 30° below the horizontal line, was selected in order to reach a sufficient penetration, into the unweathered rock [Hanna and Littlejohn, 1969], in order to mobilize the required resistance.

In equation (2.1) the two stabilizing terms within the square brackets represent the contribute of S parallel or orthogonal to the failure surface.

The upper row of anchors has transferred to the soil a force per meter R = 100 kN/m while the two lower rows together were responsible of a force R = 500 kN/m. Each single anchor carried a traction force of 300 kN in the upper row (length of anchors = 15 m) and 600 kN in the lower row (length of anchors = 20 m).

Furthermore in the stability calculations the stabilizing shear forces given by equation (2.1) were applied upon the failure surface on a strip surface of which the width was estimated from the elastic distribution of the stresses generated by the forces induced by the anchors.

A further stabilizing contribute was given by the nailing effect offered by the steel tube reinforcement of the bore micropiles; such piles are injection piles of the tubfix type [Mascardi, 1968; Koreck, 1978, Diamanti, 1980].

The resulting shear force acting per meter on the sliding surface was of 40 kN/m and 80 kN/m on the upper and lower piles-row respectively.

The stabilizing effect of all the combined reinforcements is very low when the full body of the sliding mass is considered. In this case

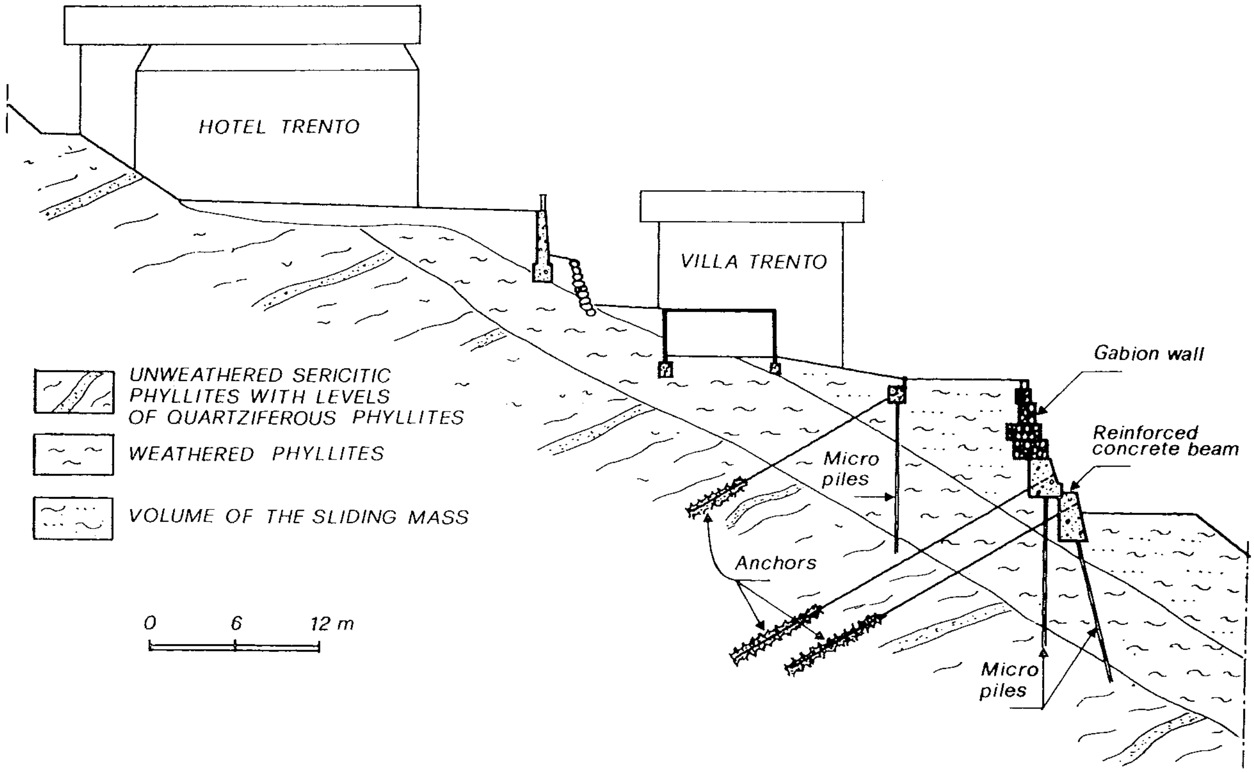

Fig. 9. - Cross section showing the stabilization works.

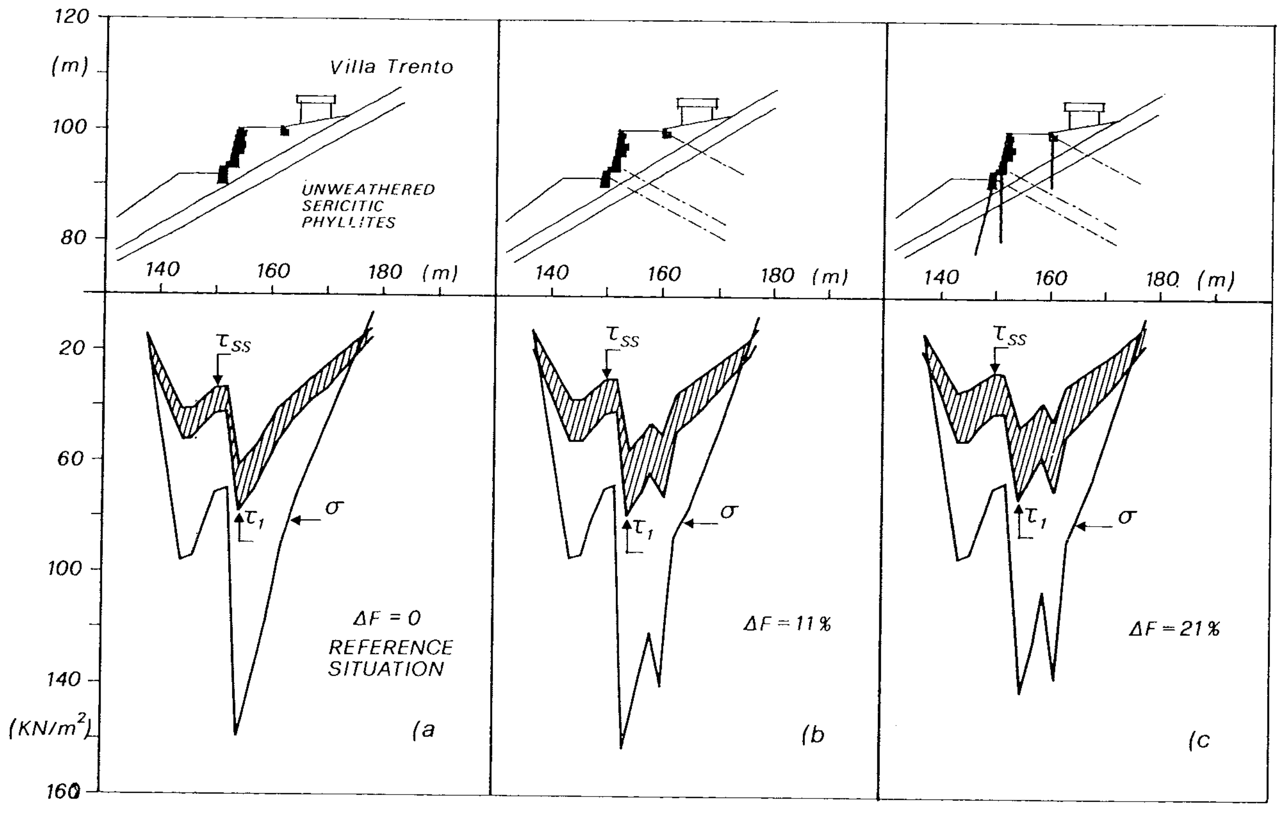

Fig. 10. - Evaluation of the stabilization effects given by the anchors and micropiles on the upper part of the landslide.

in fact there is an increase of the safety factor of only 2 percent. On the other hand the stability of the upper part has been compilated by a stability analysis; the results are represented in fig. 10. In this case the safety factor increased by 11 percent when only the forces given by the anchors were taken into consideration and by 21 percent if the effect of the anchors and micropiles are considered jointly.

From fig. 10 it is also clear how much the shear resistance increase in the treated zone as well as the position of the shear strength increase.

The effect of the stabilization is well documented by the displacements - depth diagram given by inclinometer N. 6 (fig. 11) which, over two years, gave displacements after stabilization of a few millimeters.

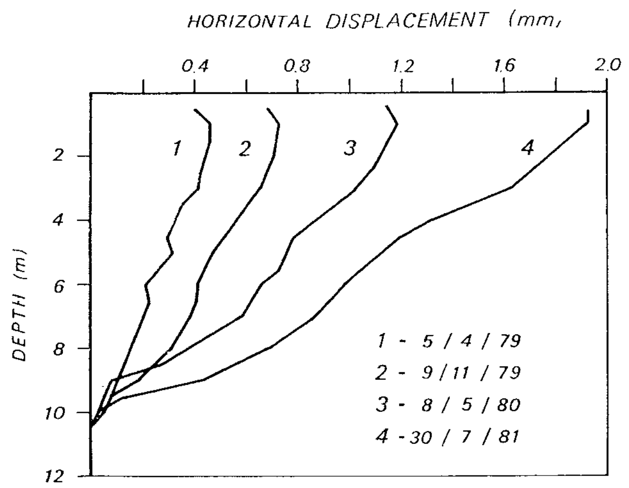

Fig. 11. - Horizontal displacement vs. depth for inclinometer no. 6 measured over a period of more than 2 years after the taking place of the stabilization works.

9. Conclusions

The stabilization of the Vetriolo landslide, examined in this paper, represents a typical example of intervention on a slope in the Alpine region of Italy.

The process of decision making, before starting the design stage, required deep investigations of the nature of soil and rock formations, of the hydrogeological conditions as well as of the main shear strength characteristics which have never been studied before with regard to the weathered phyllites of this area.

The original topographic conditions imposed to stabilize a steep slope where it was difficult to use heavy machinery.

In order to stabilize only a minor, nevertheless important, part of the full slide (i.e. the upper part from the main thoroughfare to the Trento Hotel) it was decided to design a stabilization work able to increase significantly the shear strength of the soil in the considered area.

This was possible utilizing a combined system of structural elements of great flexibility (i.e.: anchors and micropiles) which increased the stress normal to the shear surface inducing a precompression stage into the soil.

This stabilizing system has the further advantage not to alter the existing water flow.

Because of the flexibility of the substructure it was required to utilize a flexible superstructure to hold the road embankment; the gabion wall was then incorporated in the design.

The stabilization increased the safety factor of the upper part of the sliding mass by 21 percent, this was considered acceptable by both the designers and contractor and, in fact, the upper part of the slide has almost stopped its movement (see fig. 12).

In spite of the fact that only superficial drainage was built in the lower part of the landslide this has considerably slowed down its motion since the stabilization of the upper portion of the slide even if the overall safety of the entire slope was increased by only 2 percent.

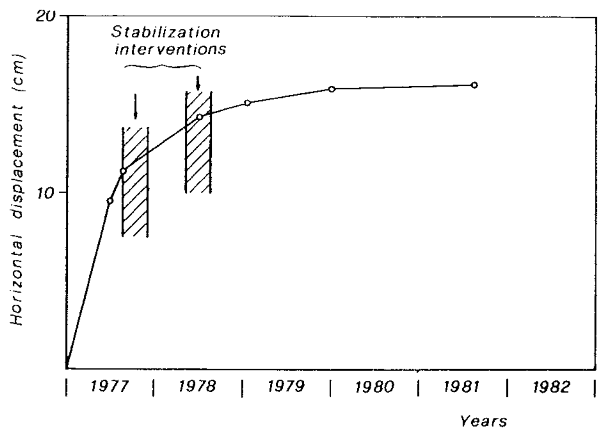

Fig. 12. - Horizontal displacement of a topographic bench mark located on the edge of Villa Trento.

ACKNOWLEDGMENT

The authors are endebted to the Geological Service of the Provincia Autonoma di Trento and expecially to Dr. Mario Nardin of the same Service who has supplied the geological news given in the paper as well as many historical knowledge.

The mechanical tests have been carried out at the Geotechnical Laboratory of the Engineering Faculty of the University of Padua.

REFERENCES

ANDERLE R., MAZZALAI P., NARDIN M. (1981) - *Indagine comparativa degli eventi franosi nel Trentino nei periodi 1966-67 e 1976-77.* Proc. of Economia Trentina, vol. 1, pp. 55-82.

ANDERSON W. F. (1974), I - *Factors influencing the measured properties of Glasgow regional fill.* Ground Engineering, vol. 7, n. 2, pp. 20-27.

ANDERSON W. F. (1974), II - *The use of multi-stage triaxial test to find the undrained strength parameters of stony boulder clay.* Proc. Inst. Civil Engrs., Part. 2, n. 57, pp. 367-372.

BARLA G., CORNERI C., CRAVERO M., GIANI G. P. (1974) - *L'applicazione dei metodi dell'equilibrio limite nello studio dei pendii naturali e delle scarpate.* Ass. Mineraria Subalpina, Suppl. Bull., vol. 11, n. 3-4, 197 pp.

BELL J. M. (1968) - *General slope stability analysis.* J. Soil Mech. Found. Div., vol. 94, SM 6, pp. 1254-1270.

BROMHEAD E. N. (1978) - *A simple ring shear apparatus.* Ground Eng., vol. 12, pp. 40-44.

BS: 5930 (1981) - *Code of practice for site investigations.* BSI, London, 147 pp.

DIAMANTI L. (1980) - *Pali trivellati di piccolo diametro.* XIV Ital. Conf. Soil Mech., vol. 1, pp. 153-162.

COLLESELLI F., COSTANTINI C., MONTANARI F. (1979) - *Stabilità allo scivolamento di pendii, fondazioni ed opere di sostegno.* Coop. Libr. Ed., Univ. di Padova, 185 pp.

COTECCHIA V., FEDERICO A. (1980) - *Sulla dipendenza della resistenza al taglio residua drenata di terreni coesivi dal livello di sforzo normale efficace.* XIV Conv. Naz. di Geotecnica, Firenze, vol. 2, pp. 317-324.

GIBSON R. E., HENKEL D. J. (1954) - *Influence of duration of tests at constant rate of strain on measured « drained strength ».* Geotechnique, vol. 4, n. 1, pp. 6-15.

HANNA T. H., LITTLEJOHN G. S. (1965) - *Retaining wall tieback.* The Consulting Engineer, Part I, pp. 50-53, Part II, pp. 49-52.

KENNEY T. C., WATSON G. N. (1961) - *Multiple-stage triaxial test for determining c' and Ø' of satured soils.* Proc. Int. Conf. Soil Mech., Paris, vol. I, pp. 191-195.

KORECK W. (1978) - *Small diameter bored injection piles.* Ground Engineering, vol. 11, n. 4, pp. 14-20.

LUPINI J. F., SKINNER A. E., VAUGHAM P. R. (1981) - *The drained residual strength of soil.* Geotechnique, vol. 31, n. 2, pp. 181-213.

MASCARDI C. (1968) - *Micropali di elevata capacità portante.* Proc. Riv. Ital. di Geotecnica, vol. 2, n. 4, pp. 189-196.

PICARELLI L. (1980) - *Sulla resistenza residua di argille varicolori dell'Italia meridionale.* XIV Conv. Naz. di Geotecnica, Firenze, vol. 2, pp. 301-309.

SKEMPTON A. W., HUTCHINSON J. (1969) - *Stability of natural slopes and embankment foundation.* Proc. 7th Int. C.S.M.F.E., Mexico, State of the Art Report, pp. 291-340.

SOMMARIO

Stabilizzazione di una frana in un pendio di filladi alterate.

L'articolo presenta una sintesi degli studi eseguiti per interpretare e stabilizzare una frana ubicata nel territorio comunale di Vetriolo (Trento) (fig. 1).

Il movimento franoso, che avvenne entro una coltre di alterazione di una formazione di filladi quarzifere, interessò un pendio a quota media 1450 ed un volume di circa 55000 m^3 posto al disotto del centro abitato. I dissesti furono osservati nell'area dal 1965 e provocarono danni alla strada provinciale, alle infrastrutture idrauliche e ad una serie di costruzioni civili (Villa Trento, Casa Giori - fig. 2). Una serie di interventi minori, eseguiti dal 1965 al 1973 per stabilizzare la frana, dimostrarono la loro inefficacia nel tempo ed indussero ad un maggior approfondimento nell'esame delle caratteristiche dell'ammasso.

Diciassette sondaggi e due trincee esplorative permisero di individuare la morfologia del substrato, di prelevare un numero sufficiente di campioni da sottoporre a prove geotecniche e di inserire nel corpo in frana strumenti di controllo (piezometri ed inclinometri).

Fu così anche definita la potenza della coltre di alterazione delle filladi sericitiche variabile da 8 a 3 m da monte verso valle.

I parametri della resistenza al taglio di picco, residua e corrispondente ad uno stato di parziale rammollimento furono misurati in laboratorio con varie tecniche ed apparecchiature.

L'espressione $\tau = 14 + \sigma' \cdot \tan 22^{\circ}$ (kN/m^2) valutata con l'apparecchiatura triassale su campioni saturati, consolidati e non drenati nella fase di taglio, eseguita questa con misura della pressione neutrale, fu adottata, in base ad un'analisi a ritroso, per rappresentare la resistenza mobilitata sul piano di taglio (v. c. 6 e 7 e tab. I).

Disponendo di queste nuove acquisizioni si potè procedere alla scelta del metodo di intervento, del tipo d'opera e al suo dimensionamento. Fu deciso di aumentare la resistenza al taglio della parte sommitale del movimento franoso (la parte interessata dalle opere ingegneristiche) inducendo contributi tensionali stabilizzanti sul piano di taglio.

Ciò si ottenne utilizzando elementi di grande flessibilità: tiranti e micropali tra loro connessi con travi in calcestruzzo armato.

La flessibilità di questi elementi richiese l'impiego di una struttura deformabile ad essi sovrapposta a sostegno del rilevato della strada provinciale (fig. 9).

L'intervento, nel suo complesso, fornì un incremento del coefficiente di sicurezza del 21% nella parte sommitale del pendio, direttamente interessata dalle opere di stabilizzazione, mentre decisamente minore fu l'effetto stabilizzante valutato sull'intero pendio.

Questi valori furono ritenuti soddisfacenti dal committente e dai progettisti; in effetti, una serie di misure di spostamento orizzontale (fig. 11 e 12) testimoniano la forte decelerazione del movimento franoso.