Bn 585/7

Nr. 107

Mitteilungen
des Institutes für Grundbau und Bodenmechanik
Eidgenössische Technische Hochschule Zürich

(vormals: Erdbauabteilung der Versuchsanstalt für Wasserbau und Erdbau an der ETH Zürich)

Herausgegeben von Professor H. J. Lang

Creep in Bündner Schist

von Jachen Huder

Zürich, 1976

Bündnerische Kantonsbibliothek CHUR
G
16.4.80

Reprinted from
LAURITS BJERRUM MEMORIAL VOLUME
Oslo 1976

Creep in Bündner Schist

By
Jachen Huder

Eidg. Technische Hochschule Zürich, Institut für Grundbau und Bodenmechanik,
CH-8006 Zürich, Gloriastrasse 37–39

Synopsis

The Bündner schist (Bündnerschiefer) discussed in this paper is found widely in the canton of Grisons (Graubünden). Its properties as a highly tectonic, metamorphic rock, found in different states of weathering, present an interesting geological-geotechnical problem, which also has far reaching effects on the economy and the communications system.

The schist breaks up into plate-like fragments and, in addition, is subject to weathering so that with time the strength of the material is progressively reduced. Due to movements in a slope the rock fragments can be structurally rearranged, leading to a further reduction in strength. Creep movements may be initiated or accelerated simply by natural means, like erosion, but also as the result of artificial disturbance of slopes. This behaviour of the rock is the cause of extensive creep slopes in the Grisons, with the consequence that day to day life in villages and whole valleys is inconvenienced.

The creep behaviour of Bündner schist is illustrated in this paper mainly by means of the results of tests on the rock in a broken but unweathered condition. The main test employed in the investigation was the triaxial test, the results being supplemented by test results obtained using the direct and ring shear apparatus. Normally, the material exhibits no plasticity. It was tested at degrees of saturation of 100 % and approx. 80 % and at different unit weights. The influence of the maximum fragment size on the shear strength was also investigated. It was found that the strength of the material tested was dependent only on the unit weight of the material and was independent of the fragment size. The effect, whereby the material is shattered, was shown at all round pressures of 50 kg/cm², while the influence of structural readjustment was investigated in the ring shear apparatus. The influence of weathering on the fragment size distribution and thus on the shear strength is shown for a naturally occurring weathered schist.

Practical examples with measurements of deformation demonstrate the importance of the structural behaviour of Bündner schist.

1. Introduction

L. Bjerrum was especially interested in studying creep phenomena in snow and ice in Switzerland after making an exhaustive study of stability problems in Denmark, primarily of railway embankments. The attention of the young Danish engineer was drawn to these problems through the work of R. Haefeli (1940, 1942, 1944). Bjerrum moved to Zürich in 1947 to take up an appointment at the Institute of Hydraulic Research and Soil Mechanics attached to the Swiss Federal Institute of Technology. His interest in snow mechanics waned in the face of the important soil mechanics problems presented in an alpine region, such as is to be found in Switzerland. The varied geological structure from the Jura range across the flat Midlands to the pre-alpine and alpine regions offered Bjerrum a multiplicity of interesting problems to solve. Various problems required urgent attention and could not be postponed, so that hardly any time was left over to pursue studies in a specific branch of soil mechanics. The existence and explanation of residual shear stress and also progressive failure were much under discussion. These ideas were at that time still so new, that confirmation of them was still being sought in nature.

Switzerland does not have soils like London clay or the Norwegian quick clays, whose formations permit the recovery of undisturbed soil samples.

It is not to be wondered at, therefore, that Bjerrum at this time could not accept the hypothesis of progressive failure (Haefeli, 1951, Skempton, 1964, Bjerrum, 1967, 1973). It was too great a step to realize that slides in Molasse or Flysch and also partly in Bündner schist can be much better explained on this assumption. So long as it was not possible to carry out research on the shear strength of Bündner schist the considerations of progressive failure were but speculation. Even today an investigation of the properties of Bündner schist is difficult, as undisturbed samples are scarcely obtainable. Now a closer identification of Bündner schist will be given.

2. **Bündner schist** (Bündnerschiefer)

According to the recent work of H. Bolli and W. Nabholz (1959) and R. Trümpy (1960) Bündner schist is characterized geologically as a Mesozoic rock of pre-

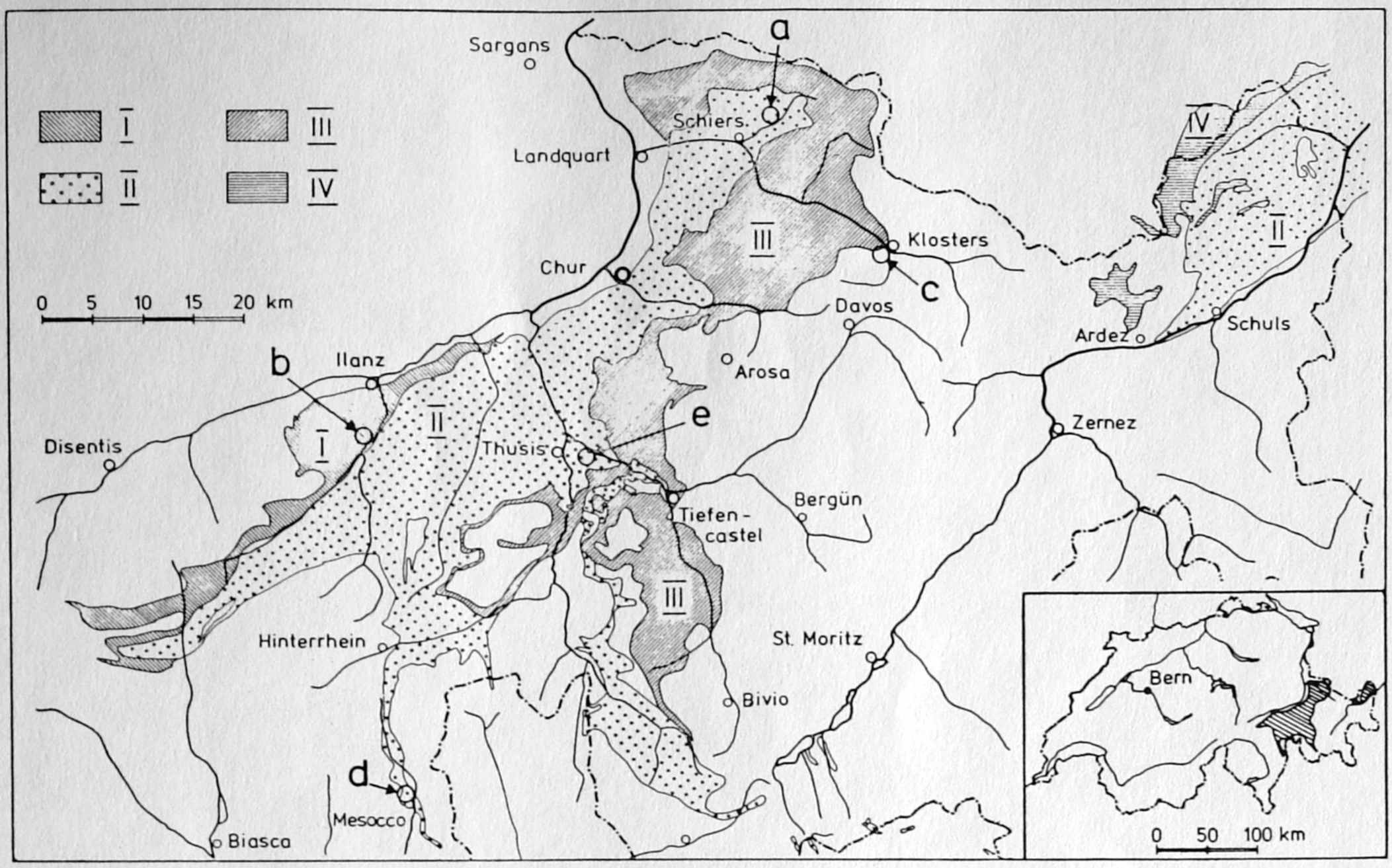


Fig. 1. Distribution of Bündner schist in the Grisons and surrounding areas. Zone I: Gotthard massif Bündner schist. Zone II: Older Bündner schist: Jurassic to Cretaceous. Zone III: Younger Bündner schist: Flysch. Zone IV: Flysch and Limestone. Letters a to e refer to creep slopes described in the text.

dominantly detrital composition (quartz grains, clay and lime), with grain size in the range fine sand to silt, deposited in geosynclines to form later the Pennine nappes. The marine sediments underwent regional (alpine) metamorphism of varying intensity. The sparsity of microfauna and the almost total absence of macrofauna is a feature of Bündner schist, making it difficult to make firm conclusions regarding the stratigraphy. It is classified between the Triassic and Cretaceous systems.

The term Bündner schist is loosely synonymous with the terms 'Schistes Lustrés' and 'Calcescisti'. Geographically it extends beyond the boundaries of the canton Grisons in the Pennine nappe and especially in the following regions: 'Unterengadiner Fenster', Avers – Bergell, Misox – Prättigau – Lugnez; synclines in the region of the Ticino and Simplon nappes, Zermatt, Saas Fee and of Cobin in Valais (Rutsch, 1960). Here, however, our considerations are restricted to the canton Grisons, (Zulauf, 1971) (Fig. 1). But it does not follow that this simplifies matters as the Bündner schist is found in varying degrees of metamorphism and consequently displays great variety in its material properties. In particular, the oriented texture of the rock results in a strongly anisotropic behaviour under the action of stress. Generally, the shear strength is greatly impaired through its coarse-grained and foliated structure, combined with a high content of mica, graphite and carbonaceous substances. Also, the degree of weathering and weatherability of the rock varies according to the composition and subsequent metamorphism. It is precisely this heterogeneity which characterizes Bündner schist (Fig. 2).

The geological processes of denudation, fluvial erosion, rock fall and slides, of removal of material and deposition in all forms is, in mountainous areas, of great technical and, not less, economical significance. It is understandable then that today not only geologists occupy themselves with these processes, but also engineers are forced to study the results of Quaternary geological research, and actively take part in field research projects.

The naturally weathered material is primarily composed of plate-like, flaky fragments. A high mica and graphite content gives rise to distinct flat cleavage surfaces, which, especially in talus slopes with an arrangement of the schist fragments similar to a roof tile arrangement, represent potential failure surfaces. Water seeping into cracks and fissures accelerates the weathering, however, on account of the metamorphism the clay minerals do not exhibit any swelling. Even in Bündner schist in which the metamorphism was not so pronounced it has not been possible to detect any welling or measure any swelling pressure. On the other hand, the presence of water in schist subject to repeated mechanical stressing can considerably reduce the shear strength.

The broad spectrum of the mechanical properties of a Bündner schist is more clearly understood by reference to the mineralogical composition of the rock. Material I is an older Bündner schist (Nollakalkschiefer) from a rock fall in a tunnel at Reischen, south of Thusis (Fig. 1).

Material II originates from the younger Bündner schist (Bündnerschiefer Flysch), from the Brienzerberg rock fall near Tiefencastel.

Table I. Mineralogical composition.

Material		I		II	
		< 2 mm	< 2 μ	< 2 mm	< 2 μ
Unit weight	g/cm³	2.76		2.78	
Carbonate content	%	26		28	
Quartz	%	35		30	
Illite	%	–		20	60
Chlorite	%	10	30	10	20
Muscovite	%	25	60	5	10
Felspar	%	2	10–15	5	5–10
Swelling minerals	%	–		< 2	
Clay minerals with sheet structure	%				5

The mineralogical composition of the fraction less than 2 μ, after removing the carbonates, is given by X-ray diffraction analysis.

The mineralogical composition of the two samples is quite different. Especially worthy of attention is the constituency with respect to illite in Mat. II. Both materials have been the object of an exhaustive geotechnical investigation, the results of which are reported hereafter.

In addition to the geological and mineralogical classification of Bündner schist certain geomorphological considerations are necessary to a better understanding of the material behaviour. The rocks of the catchment area of the Rhine, which we want to consider in more detail here, comprise for the most part Bündner schist and volcanic rocks. The present day sedimentation and depositional processes are on the one hand dependent on the petrographical and lithological composition of the rock, and on the other hand on the nature of the bedding, horizontal or sloping, and the angle between the strike of the beds and the valley direction. Likewise the permeability of the rock exerts a strong influence on the resistance to erosion. Bündner schist offers very little resistance to surface runoff and shows a pronounced tendency to slow creep movements.

These problems were the subject of a thorough investigation by Jäckli (1957). Rivers are the chief agent of erosion and deposition. A typical example of river erosion in Bündner schist is furnished by the Via Mala, a postglacial occurrence (Figs 3 and 4). In Bündner schist erosion is of the order of 4–7.5 mm per year, whereas in

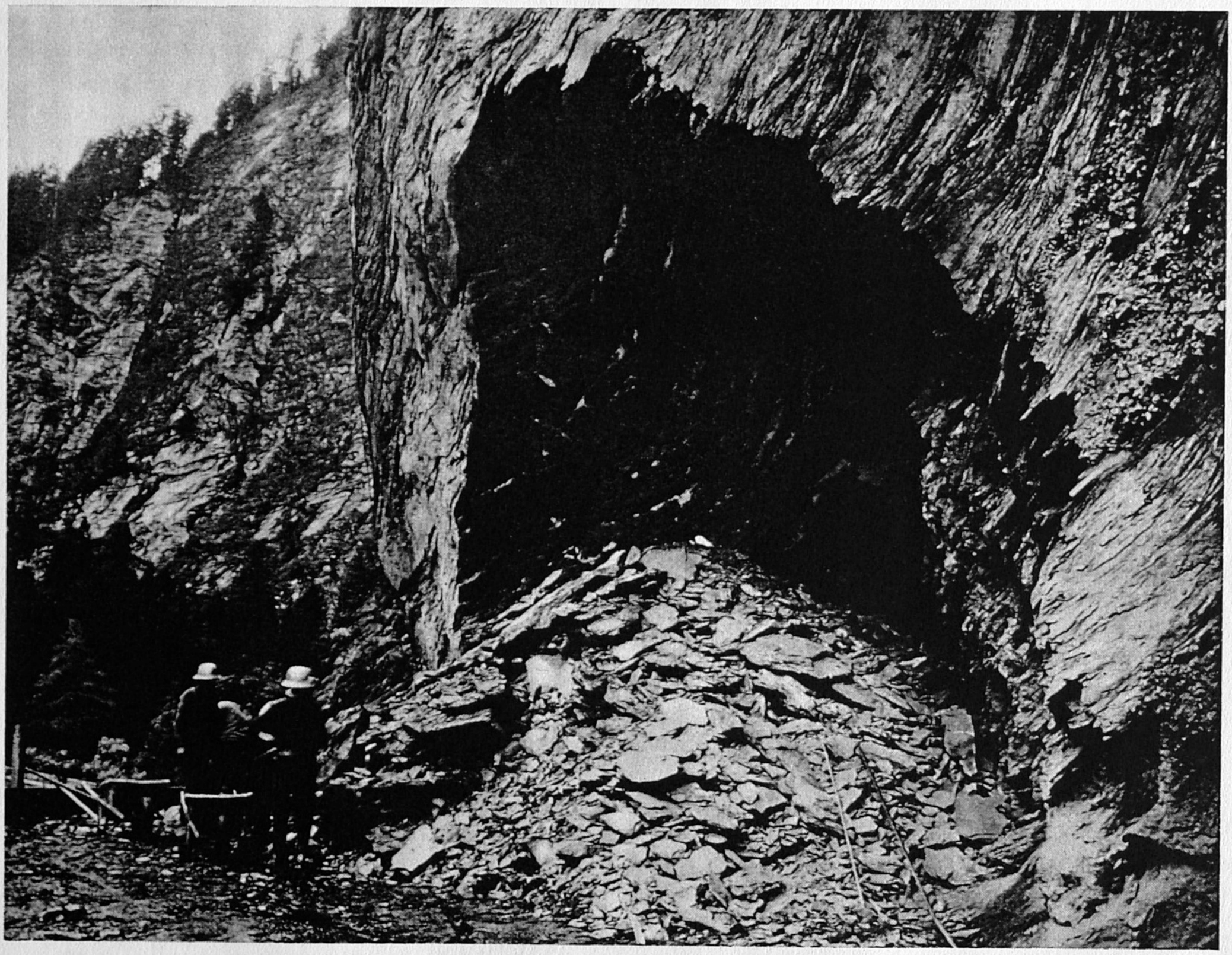


Fig. 2. Bündner schist; bedrock and broken material.
Photo: Crameri, Cantonal Engineering Offices, Chur.

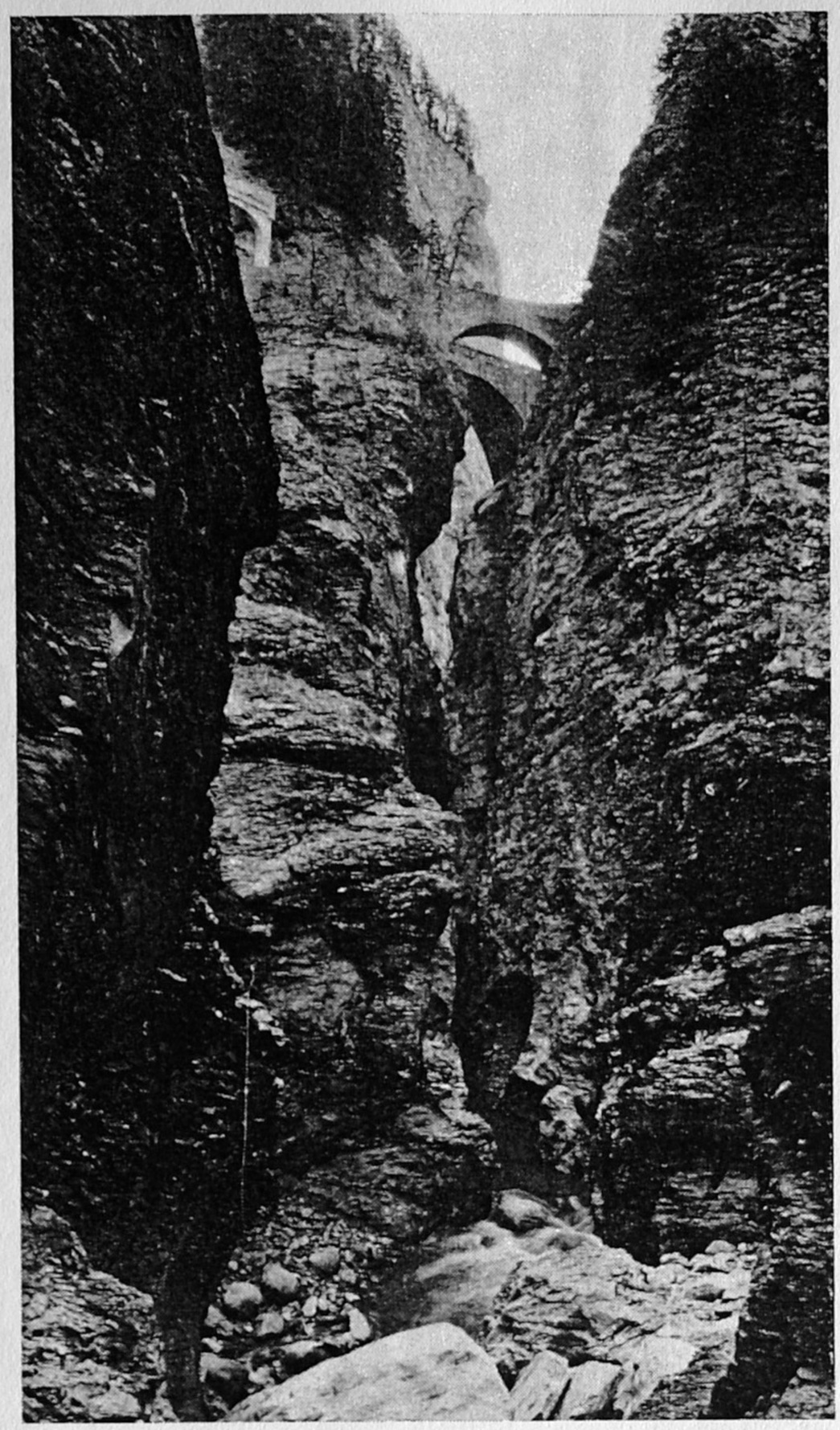


Fig. 3. Via Mala gorge. Post glacial river bed erosion of the Hinterrhein in Bündner schist, south of Thusis.

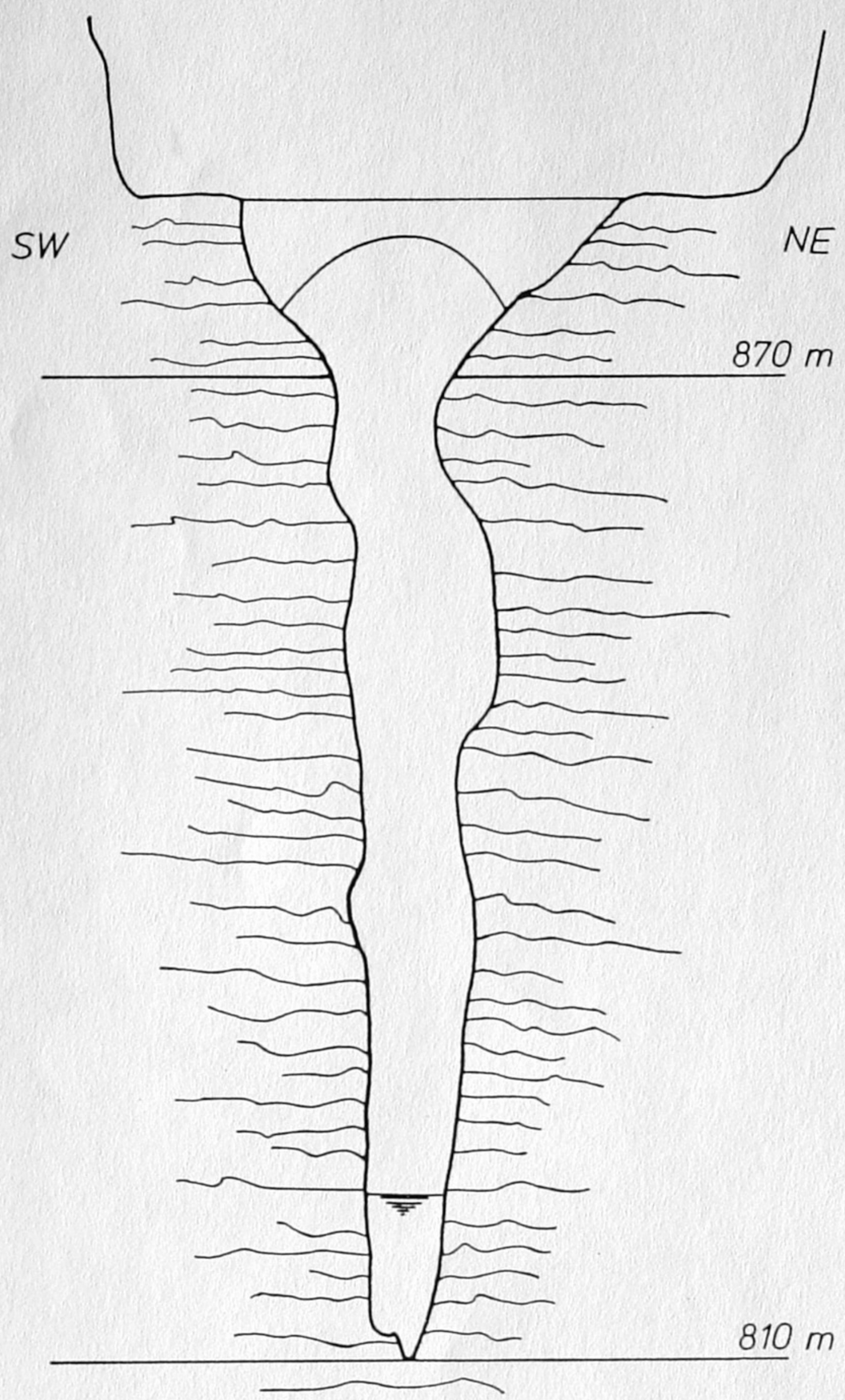


Fig. 4. Via Mala gorge. Depth with respect to the old bridge approx. 70 m, width 2–8 m, and gradient of the river bed approx. 1 in 12.5.

the harder crystalline rocks it amounts to only about 1–3 mm per year, according to the slope and quantities of flowing water. The amount of transported material in the various catchment areas is thus correspondingly large. The most reliable measurements are the long term measurements at the entry of the Rhine into the Lake of Constance (Bodensee). The Rhine with a mixed catchment area extending over nearly 7,000 square kilometres carries yearly 3.65 million cubic metres (i.e. 52.2 m^3/km^2) of transported material. The Nolla in the neighbourhood of Thusis, which has a catchment area of 28 km^2 lying only in Bündner schist, transported before the protection of river banks and afforestation 0.9 million cubic metres of material per year (i.e. 32,000 m^3/km^2). The average flow rate of the Rhine up to its entry into the Lake of Constance is 300 m^3/sec with peak flow rates of 1500 m^3/sec, whereas the Nolla only shows a rate of 5 m^3/sec with peaks in excess of 40 m^3/sec. It may be mentioned that the Via Mala gorge, which lies in the same formation as the Nolla Bündner schist, has been eroded normal to the bedding, while in the Nolla catchment area erosion is more or less parallel to it.

Gully erosion and surface erosion produce a continual change of the terrain; ground water flow is also an important contributing factor. The permeability of intact specimens of Bündner schist is very small, thus the water seeps predominantly through crevices and fissures of the heterogeneous rock system. A large number of piezometric measurements can seldom be used to define a seepage net, as often the terrain only loses water through the fissures to the lower lying boggy zones.

The occurrence of solifluction may not be left out of consideration in Alpine areas. Movements due to solifluction are relatively large. Hegland (1973) using the clinometer measurements of Haefeli (1967) has tried to quantify these movements in different areas. The clinometer tubes of 1.20 metres length are driven into the ground. Surface movements can be found from the changes of slope; values up to 10 cm have been obtained. Absolute movements can be checked, however, only using

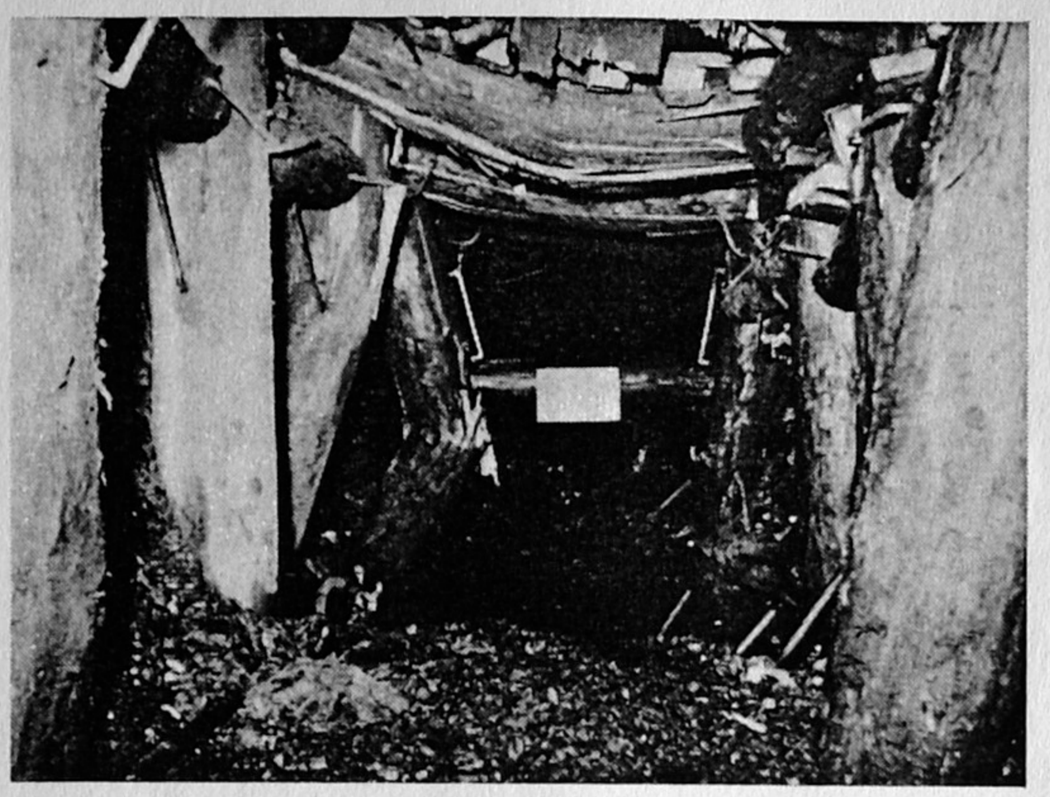


Fig. 5. Tunnels for the Julia hydro-electric power station in creeping (plastic) Bündner schist, to the south of Tiefencastel.
Photo: Prader, Building contractor, Chur.

accurate surveying instruments. Solifluction movements of the same order of magnitude were recorded by Furrer (1954) and Furrer, Bachmann and Fitze (1971) in the Swiss National Park. Here, however, solifluction could only be detected and measured in a belt lying above approximately 2000 metres above sea level; it is a question of earth movements in the form of solifluction sheets.

Topographical measurements are costly and usually only possible within the framework of a building project. Often it is large areas in which deformations are suspected, so that large distances would have to be surveyed.

Mostly, measurements for controlling a building project are only local, in which case their evaluation with respect to a prediction of creep movements is very difficult and somewhat speculative.

Weathering and disturbance of Bündner schist is not only limited to surface zones, but can be effective to very substantial depths, as is impressed upon the engineer by the inspection of such zones. This was the case during the construction of the Julia power station in the neighbourhood of Tiefencastel. The disturbance of the material and the pressures developed are clearly seen from the figure (Fig. 5). The problem could only be overcome by means of sturdy supports. The thickness of the overlying strata was about 200 m.

Even intact Bündner schist can present the engineer with problems in tunnel construction. It has been found that in various tunnel projects where tyre-transport has been used, that after 5000 load cycles, corresponding to an excavation length of around 200 m, the saturated (natural) floor of the tunnel has been softened up to a depth of 1.5 m, necessitating its removal and replacement. More recently, to avoid this, the floor of the tunnel has been concreted as soon as possible after excavation (Weber, 1971).

3. The Shear Strength of Bündner Schist

In geological terms Bündner schist is generally classified in four groups according to the age of the rock. With weathered schists it is at times difficult to distinguish between firm rock and loose rock. This is especially true of samples recovered from boreholes. The weatherability of the rock normally decreases with increasing age of the rock.

The particle size distribution curve of schist in the form of loose rock not only changes from rock to rock, but is also closely related to the mechanical stressing of the material. Freshly broken material – e.g. from rock fall in tunnels – normally exhibits a particle size distribution curve with a coefficient of curvature C_c in excess of 3, and coefficient of uniformity C_u greater than 40.* The material classification – modified Casagrande – is GP (occasionally GW). On the other hand, the material in waste heaps may be classified as GM-ML material, with a curvature less than 1, and a steepness greater than 100. The plasticity is not significant and often the plasticity index cannot be determined. The grain structure is flaky to rod-like with sharp edges. Through intensive mechanical stressing and weathering the material classification can be changed from ML to CL-ML.

The shear strength of fragmented Bündner schist and the waste heap material has been tested in numerous investigations at the Institute for Soil Mechanics and Foundation Engineering (formerly soil mechanics section of the Research Station for Hydraulics and Soil Mechanics) in connection with building projects. An extensive investigation was decided upon for the fill construction at Reischen as part of the new road works of the Via Mala. With the aid of the results of this investigation the different factors controlling the shear strength of Bündner schist could be elucidated.

A. MATERIAL I:
OLDER BÜNDNER SCHIST (NOLLATON SCHIST)

Material I is a grey-black Nollaton schist, an older Bündner schist, deriving from rock falls in a tunnel. It would be described geotechnically as broken up gravel material with a little sand and silt, with very low or no plasticity. The maximum grain size amounts to 100 mm, the coefficient C_c on the grain size distribution curve equals 4, and the coefficient C_u equals 45. The distribution curve lies practically between the Fuller and the EMPA curves. The percentage less than 0.02 mm is 2.5 %.

Separation of coarse fractions from the material supplied, giving maximum grain sizes of 50, 10 and 2 mm, was carried out so that four different materials could be tested, which are denoted by D = 100, D = 50, D = 10 and D = 2 (Fig. 6).

---

* $C_u = \frac{D_{60}}{D_{10}}$ $\qquad$ $C_c = \frac{(D_{30})^2}{D_{10} \cdot D_{60}}$

where:
D_{10}, D_{30}, and D_{60} are the grain-size diameters corresponding respectively to 10, 30, and 60 percent passing on the cumulative grain-size curve.

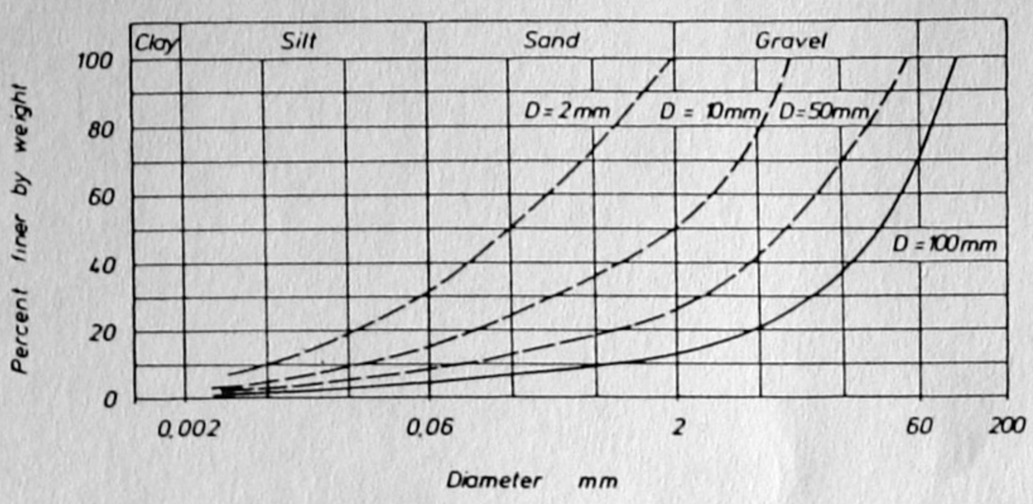


Fig. 6. Grain size distribution curves of materials D = 2, 10, 50 and 100 mm maximum fragment size respectively of old broken Bündner schist.

a) *The Crushing of Bündner Schist*

The degree of crushing, i.e. a shattering or fracturing of the fragments of the schist, was investigated under a high all round pressure of $\sigma_3 = 50$ kg/cm² on the fragments 1–2 mm (Fig. 7).

The results show clearly that a significant shattering of the fragments results from the shearing action. Only when an optimal grain contact surface area is reached, with a grain size distribution curve approximating the Fuller Curve, is there no further test evidence of crushing. However, a further crushing of the material in the failure plane is likely. In naturally occurring Bündner schist, on the other hand, a distribution curve approximating the Fuller curve is rarely found. Normally, in talus slope material consisting of Bündner schist more fines are present, so that the grain distribution curve of material I may be regarded as a limiting case.

The crushing test was carried out as follows: the uniform material of 1–2 mm diameter grain size (curve 1, Fig. 7), of dry density 1.27 t/m³, was tested under a pressure of 50 kg/cm². The high pressure triaxial apparatus permitted a maximum axial strain of 20 %. At 20 % strain the deviator stress was still rising without reaching failure, while the volume of the specimen was constantly decreasing. After dismantling the test apparatus, the grain size distribution of the specimen was obtained (curve 2).

The new material 2 recovered from a large number of tests was also tested at the same pressures in the triaxial test. The density amounts to 1.54 t/m³. Here again the maximum deviator stress is hardly reached at high strains

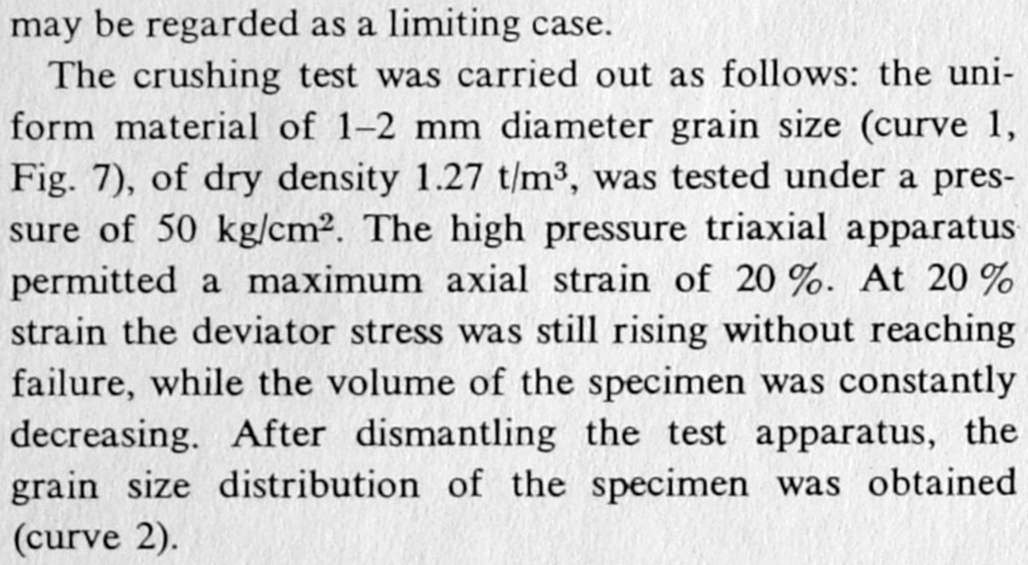


Fig. 7. Grain size distribution for the crushing tests with $\sigma_3' = 50$ kg/cm².

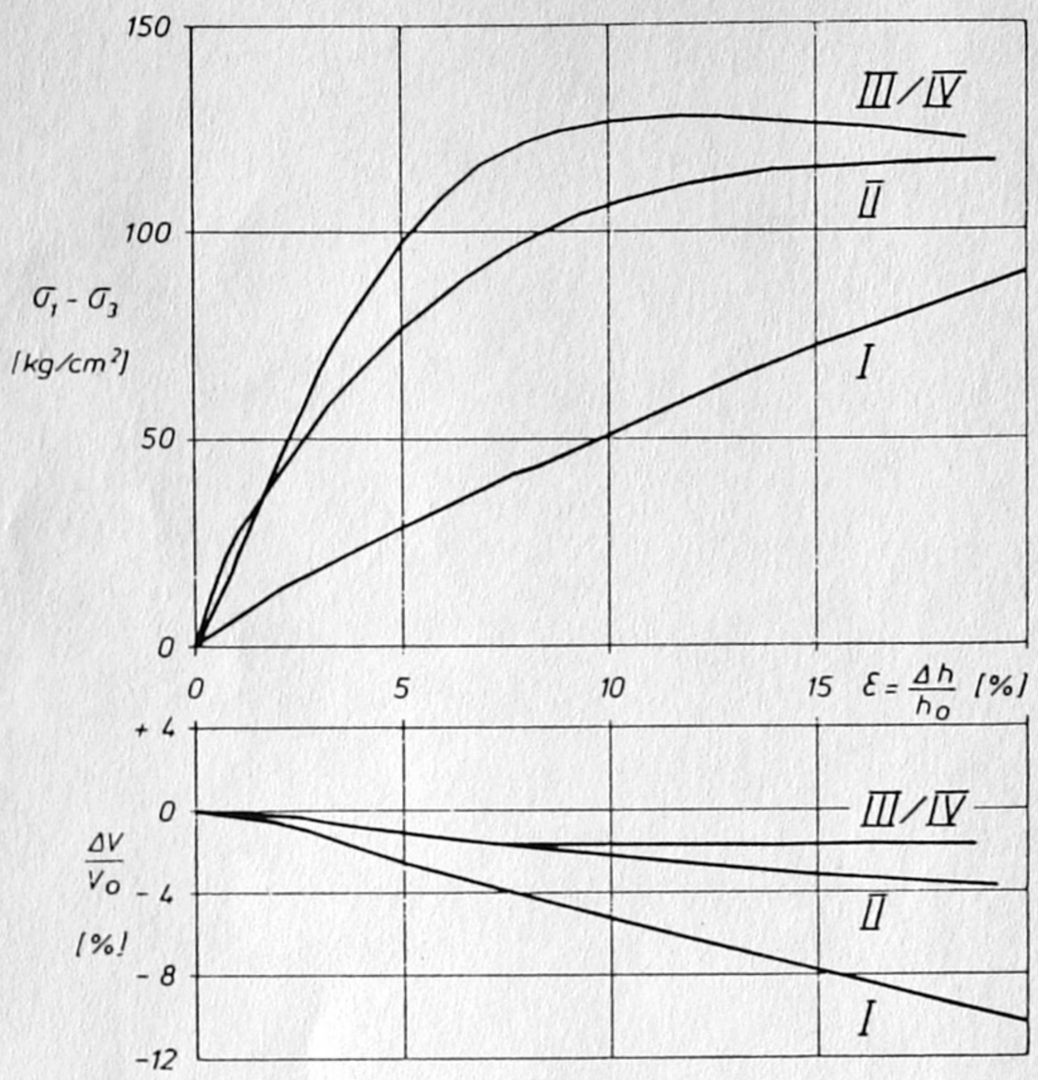


Fig. 8. Stress-deformation curves for triaxial tests with lateral pressure for different volume changes. Series I: Average value for 9 tests, Series II: Average value for 7 tests, Series III: Average value for 5 tests, Series IV: Average value for 2 tests.

($\varepsilon = 18$ %) as shown in Fig. 8. The material behaviour is still characterized by large volume decreases. The third series of tests with material of grain size distribution 4 was obtained once more from tests with material 3. The results, with a density of $\gamma = 1.74$ t/m³ and $\sigma_3 = 50$ kg/cm², were a maximum deviator stress of 127 kg/cm² at 11 % axial strain and a corresponding friction angle of 34° (Fig. 8). At the same value of σ_3 further tests with the same material did not show any marked crushing effect. This effect occurs mainly in the failure plane, so that it is difficult to detect it for individual tests. Only through greater mechanical stressing is it possible to reduce the material size still further. The grain size distribution curve of the crushed rock resembles that of schist in the form of rock fall material.

It follows from the test series III/IV that the shattering of the material by shearing action, especially in the failure plane, takes place with minimum grain contact pressures.

For no volume change at failure the deviator stress falls. At an axial strain ε of 7 %, for $\Delta V/V_o$ held constant, a reduction in the angle of internal friction from 34° to 33° may be observed.

Compaction Tests and Preparation of Specimens

A standard AASHO – Compaction test was carried out on material fraction D = 2. The optimum water content is 11 %, corresponding to a maximum dry density of 2.01 t/m³, and a degree of saturation of 80 %. The corresponding values for a compaction test on the fraction D = 10 were optimum water content 8.5 %, maximum dry density 2.14 t/m³, and degree of saturation 80 %.

In the main investigation the degree of saturation was kept constant at 80 % and the following dry densities were chosen: 1.82, 2.01, 2.14 and 2.24 t/m³.

b) *The Determination of Shear Strength*

Test Procedure

The triaxial tests were of the kind consolidated-undrained with different confining pressures. Pore water pressures and changes of height and volume of specimen were measured during the test. In the test the following parameters were introduced: maximum fragment size, dry density and confining pressure (i.e. cell pressure), which always corresponds to the consolidation pressure.

Fifteen specimens with material fraction D = 2 and nine with D = 10 were compacted statically. For the fractions D = 50 and D = 100 this was not possible on technical grounds. These specimens were compacted in layers to the required density using a vibrator.

The maximum fragment size determined the allowable specimen diameter, the relationship between fragment size and specimen diameter being 1:5. Thus triaxial cells to accommodate the following specimen sizes were used (Fig. 6):

Max grain-size diameter	*Area of triaxial specimen*	*Height of triaxial specimen*
D = 2 mm	F = 10 cm²	h = 7 cm
D = 10 mm	F = 25 cm²	h = 10 cm
D = 50 mm	F = 500 cm²	h = 42 cm
D = 100 mm	F = 1962 cm²	h = 90 cm

c) *Shear Strength*

Various evaluations and representations of individual groups of tests showed for all specimens, that there exists a linear dependence between dry density and maximum deviator stress with confining pressure σ_3 as a parameter. This indicates that the shear strength is dependent only upon the dry density, and not on maximum fragment size.

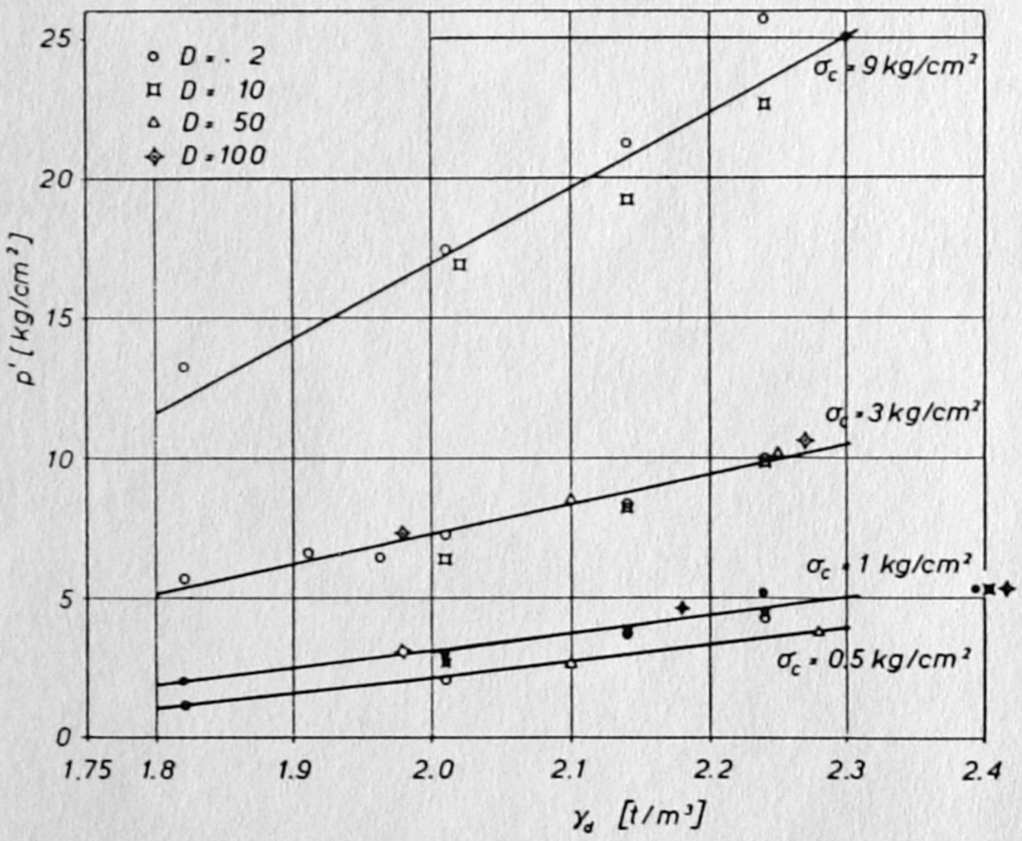


Fig. 9. q as a function of dry density γ_d for triaxial tests with different maximum fragment size and specimen size with lateral pressure $\sigma_c = \sigma_3'$ as parameter.

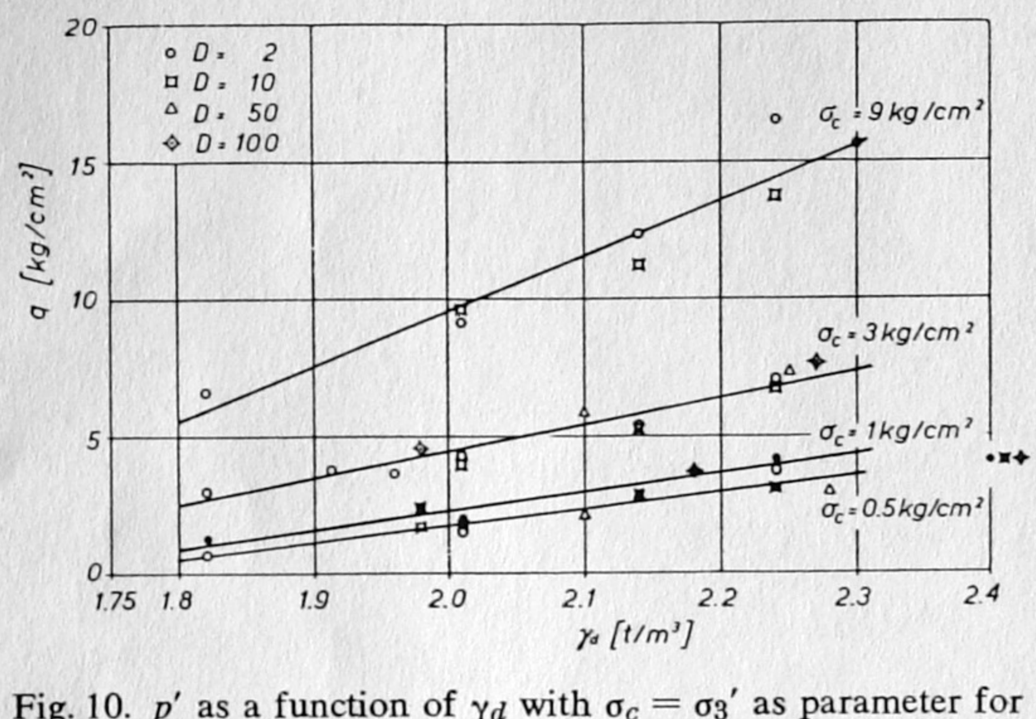


Fig. 10. p' as a function of γ_d with $\sigma_c = \sigma_3'$ as parameter for the same tests as reported in Fig. 9.

Figs 9 and 10 were used as a basis to construct the $p' - q$ diagram, Fig. 11, (Huder, 1971).

The points of maximum stress on the $p' - q$ diagram, which are connected by a straight line, may be represented in a $\tau - \sigma'$ diagram using the following transformation.

$$\sin \Phi' = \operatorname{tg} \beta' \text{ and } c' = \frac{a'}{\cos \beta}$$

The friction angles Φ' and the intercepts on the axis c', expressed in terms of effective stresses, are given in Fig. 12 as a function of the dry density γ_d.

The angle of internal friction of material in the loosest conditions was estimated to be $\Phi' = 30°$ for this material and was independent of the maximum fragment size for similar grain size distribution curves. Increasing the density produces a sudden increase in strength, due to the greater structure-interlocking action. A statistical evaluation of the friction angle Φ' indicates in addition a small increase in Φ'. The latter effect is also indicated in Fig. 12.

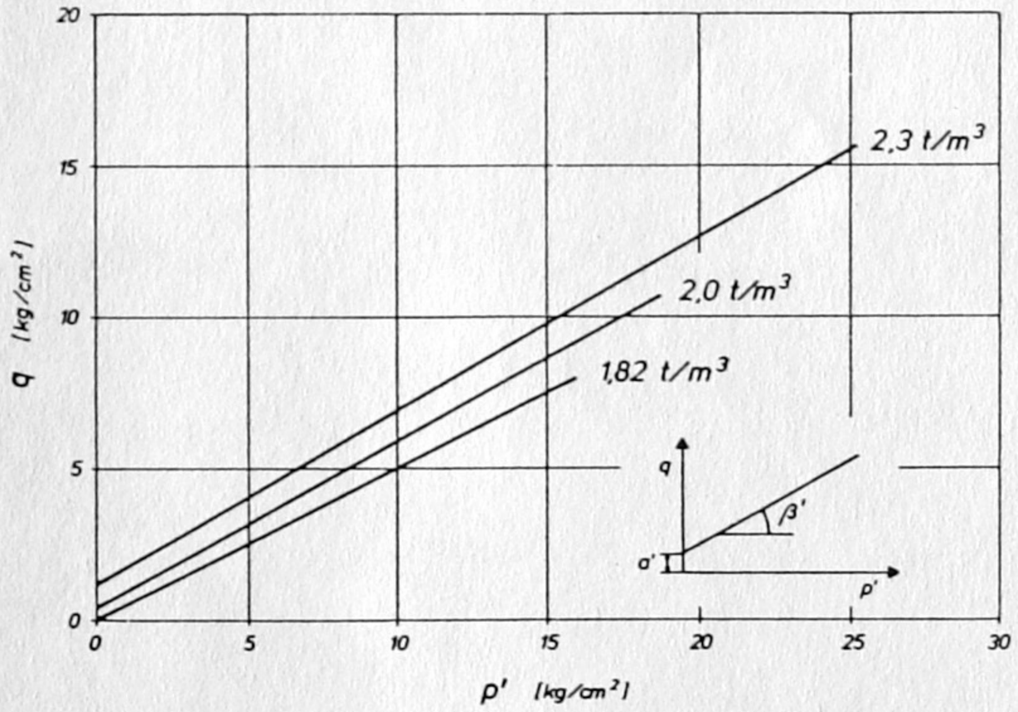
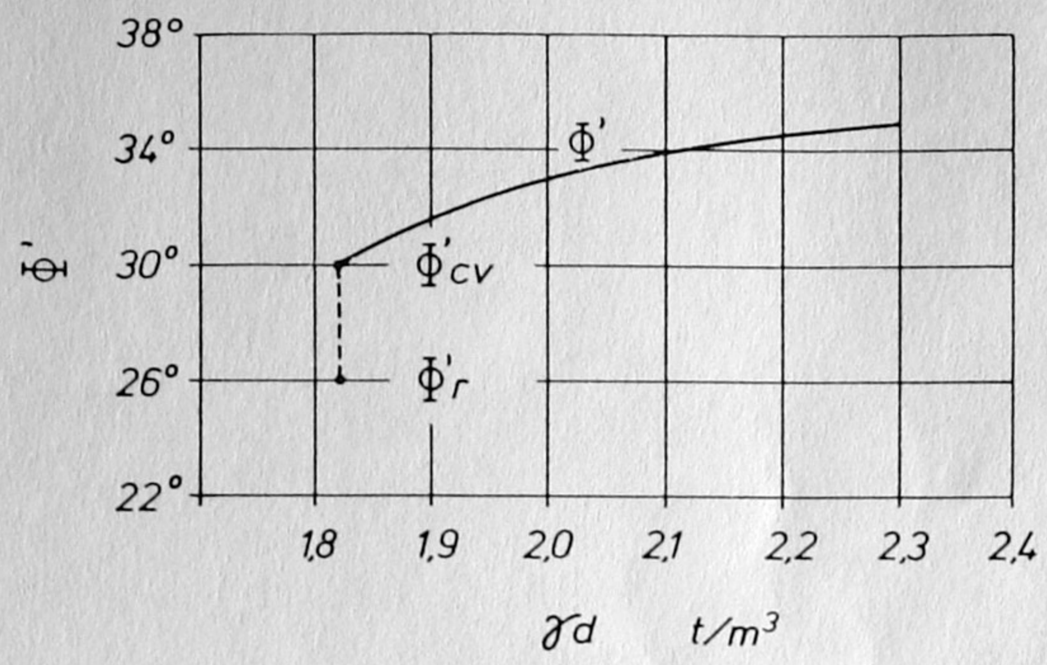


Fig. 11. $p' - q$ diagram for different dry densities γ_d; results obtained from Figs 9 and 10.



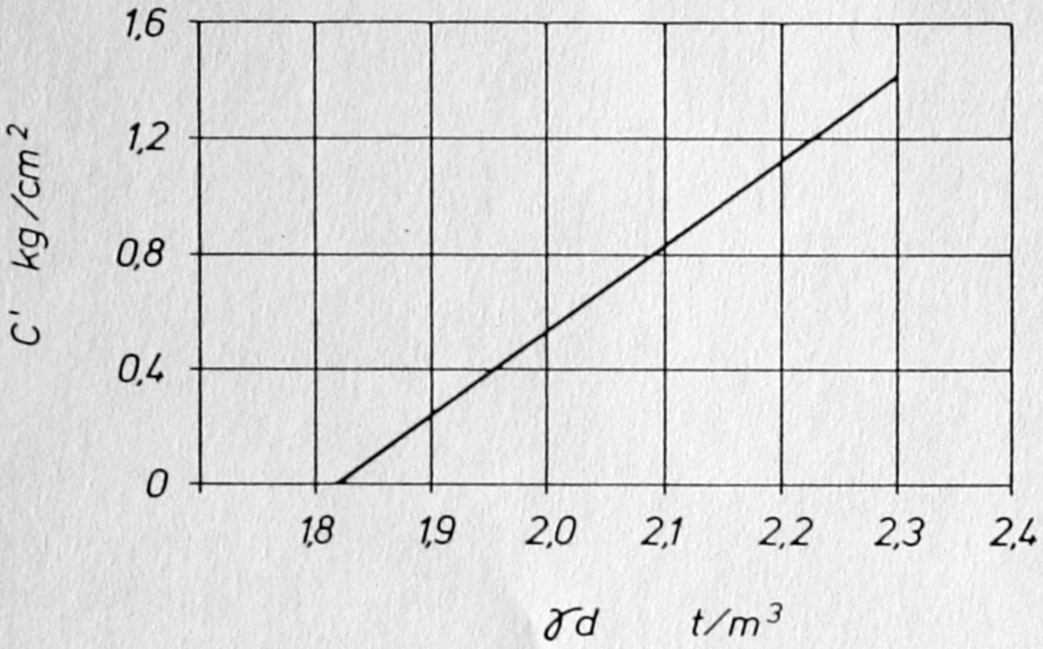


Fig. 12. Friction angle and the τ axis intercept c' for unsaturated specimens of different max. fragment size as functions of γ_d.

It must be emphasized that these relationships (Fig. 12) are only valid for the particular material investigated. For materials with grain size distributions deviating from those obtained here different densities result. Furthermore, the density achieved by Proctor compaction is closely related to the grain size distribution of the material.

As already mentioned, therefore, the investigation of material properties shows that the maximum fragment size in a specimen has no significant influence upon the shear strength. The shear strength is only dependent upon density and the previous stress history.

Compaction and strength property testing of fill materials is usually carried out with maximum fragment sizes in the range 7–10 mm. The effect of larger size fragments is taken into account by a suitable modification of the results. Field densities, due to the inclusion of coarser fragments, are invariably greater than those of laboratory specimens prepared from the separated fine fraction. Due to the greater density the strength of the in-situ material exceeds laboratory determined strength. The influence of the coarse fraction is, however, only present in a limited way as the modification of the results is also governed by physical and geometrical considerations (Huder and Fetz, 1967).

The large (τ-axis) intercepts of the broken material are striking. These values have been obtained from regression curves of the investigated stress regions. With increasing density the material exhibits more and more dilatancy. But this behaviour is stress-dependent, and at high stresses the dilatancy is largely absent, on account of the crushing of the fragments. At smaller stresses, however, the fragments re-align themselves instead of breaking, causing large volume changes. The heterogeneous strength characteristics of the individual fragments have the effect of causing a smooth transition from re-alignment to crushing (i.e. shattering of fragments).

The strength axis intercepts, which are shown as cohesion, only serve the purpose of linearizing the failure envelope, and do not represent the true cohesion of the material, which it is not possible to find with this apparatus. The actual cohesional component at lower stresses is, though, smaller than that obtained by linearizing over a range of higher stresses. It is shown in practice, that the material exhibits an interlocking effect and it is justifiable to reckon with a cohesional component.

The specimens tested have a degree of saturation (S_r) of 80 %, a figure whose order of magnitude was found in many of the slopes investigated. In any case drying out of the material only results in a small increase of shear strength.

The shear strength diagrams (Figs 11 and 12) are based on the peak shear strengths of the individual specimens. These values of shear strength cannot explain the existing slides in Bündner schist material. The post failure reduction of shear strength of the material can, on the other hand, possibly supply an answer to this problem.

Shear Strength under Constant Volume Conditions, i.e. Elimination of the Interlocking Effect

On the basis of stress-deformation tests it has been shown that the post failure strength of the material decreases and only reaches a constant value at large deformations. Higher strengths are obtained at higher densities, so that the post failure reduction of strength is correspondingly greater. The typical behaviour is as follows: at first the volume increases, and then by further shearing the volume remains constant as the specimen reaches a state of large deformation. The final state is in fact independent of the initial specimen density. Since this phenomenon of constant shear strength is reached only at very large deformations, it is often impossible for technical reasons to reach this state experimentally. The tests show, however, that the reduced strength of an overconsolidated material is equal to the shear strength corresponding to the minimum volume.

The conclusion, that $\Phi'_{\text{min.vol.}}$ corresponds approximately to Φ'_{cv}, was also reached for other materials [Bucher, 1975]. If the test values of $(\sigma_1-\sigma_3)$ are corrected using the energy formula [Skempton and Bishop, 1954], the following relationship is obtained:

$$(\sigma_1-\sigma_3)_{\text{corr}} = (\sigma_1-\sigma_3) - \sigma_3 \cdot \frac{1}{V_o} \cdot \left(\frac{dV_o}{d\varepsilon_1}\right)$$

where

$(\sigma_1-\sigma_3)_{corr}$	is the corrected deviator stress,
$(\sigma_1-\sigma_3)$	is the measured deviator stress,
σ_3	is the confining pressure,
V_o	is the initial volume,
$dV_o/d\varepsilon_1$	is the volume change with respect to the decrease in height of specimen.

The deviator stress is corrected by an amount corresponding to the energy required to produce the volume changes. It is assumed that the stored elastic energy may be safely neglected. It is evident from the formula that only for the condition of minimum volume and of constant volume does the correction term disappear.

An exact experimental proof fails firstly because of the limits imposed on the possible deformations by the test apparatus, and secondly because of the falsification of the volume change measurements by the crushing of the fragments and thus the alteration of the given material to a new material with a different grain size distribution.

Due to the great importance of the shear strength of Bündner schist in the form of loose, broken rock fragments, an attempt should be made here to elucidate this behaviour, based upon test results. For this purpose the deformation for approximate constant volume behaviour, are presented. Fig. 13 shows the agreement between q values for constant volume $q_{min.vol.}$ The regression curve is also included in the figure. The agreement in the test results can, in view of the mentioned circumstances, be regarded as good.

The strength of materials in the loosest condition of packing, i.e. the size of Φ'_{cv}, was investigated by means

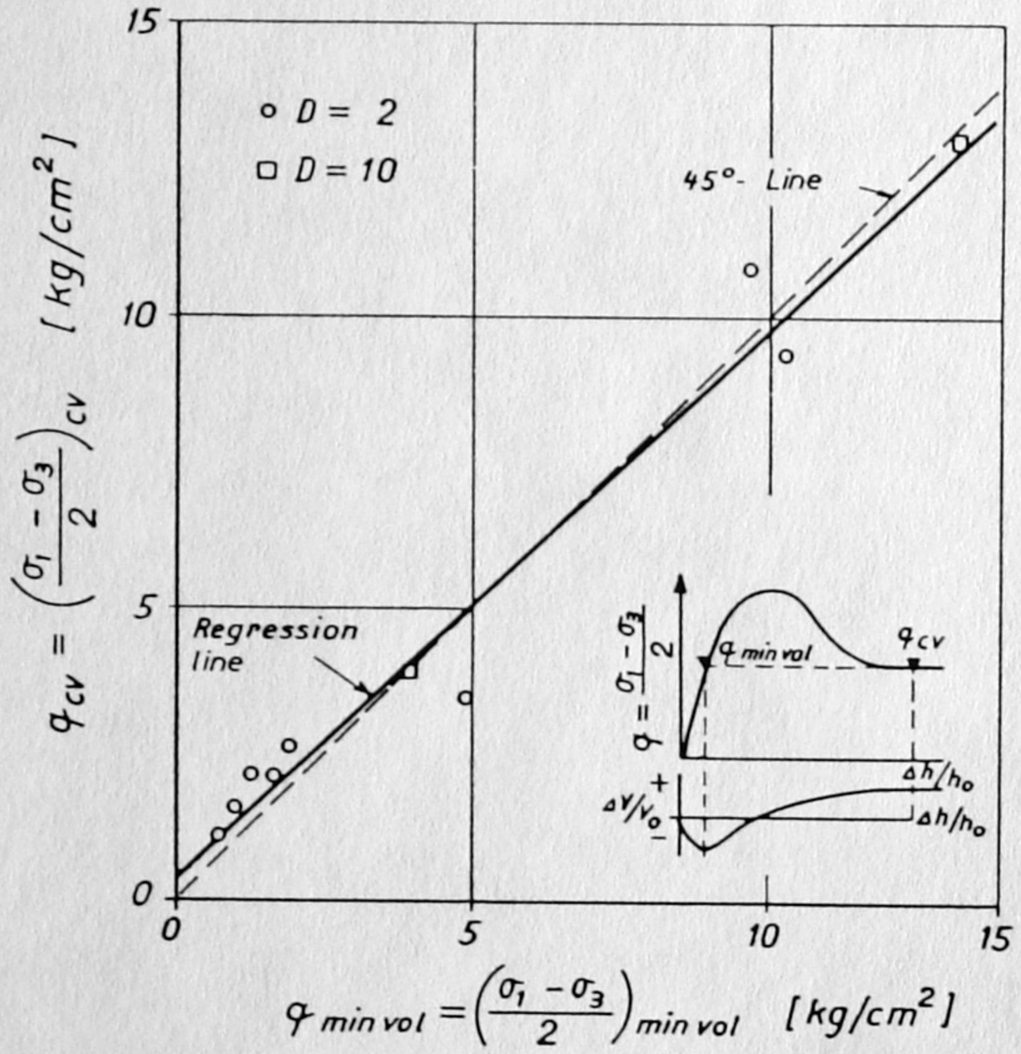


Fig. 13. Relationship between q for constant volume tests and q for minimum volume tests on specimens of material D = 2 and D = 10.

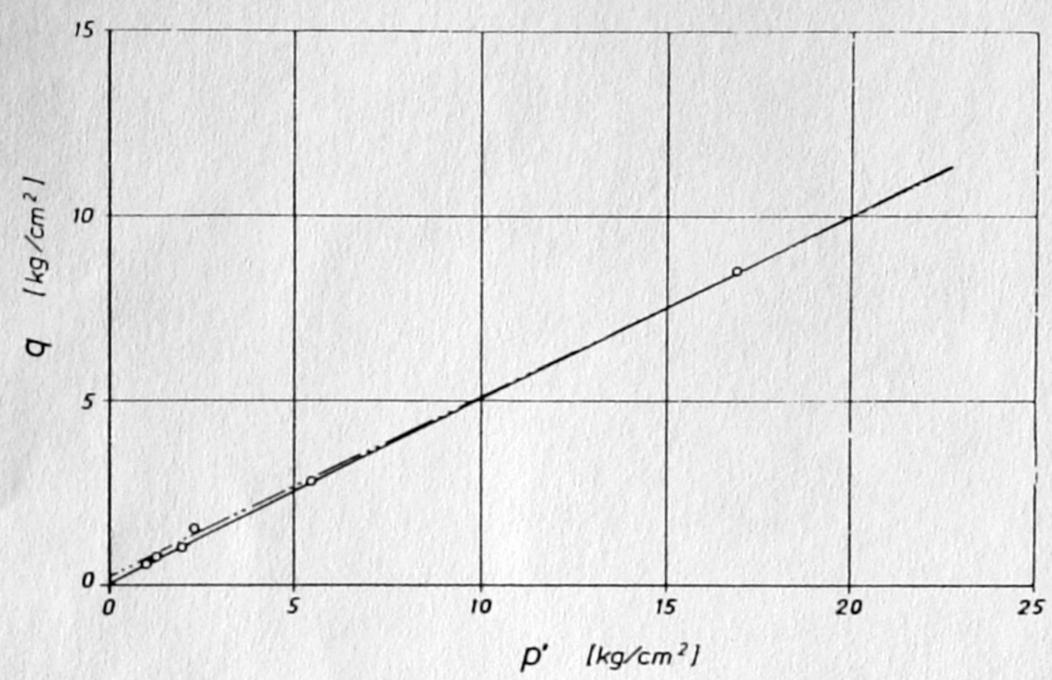


Fig. 14. $p'-q$ diagram for material D = 2 with conditions of minimum volume (q min. vol., full line) and the shear strength for the material in the loose condition (dashed line).

of further triaxial tests on the material D = 2. The results of these tests for the shear strength at $q_{min.vol.}$ and for a density of 1.82 t/m³ (i.e. loosest condition) are shown in Fig. 14. Φ'_{cv} yielded the same friction angle as was obtained in earlier tests, i.e. $\Phi'_{cv} = 30°$.

The formation of a shearing surface and with it the re-arrangement of the plate-like particles requires larger shear strains than are possible using the triaxial apparatus. This possibility was first given by the ring shear apparatus. Unfortunately the maximum particle size D_{max} is limited to 2 mm with this apparatus. The material is sheared at a speed of rotation of 1° per hour under different normal stresses. The testing time may run over several days corresponding to a shear deformation of 50 cm. Results obtained using this apparatus are shown in Fig. 15. It may be concluded from this test that the large shear deformations result in a more effective rearrangement and reorientation of particles, with a friction angle $\Phi'_r = 26°$.

With the assumption that during shear deformations in nature, which may proceed over decades, a change in the particle size distribution curve is produced by

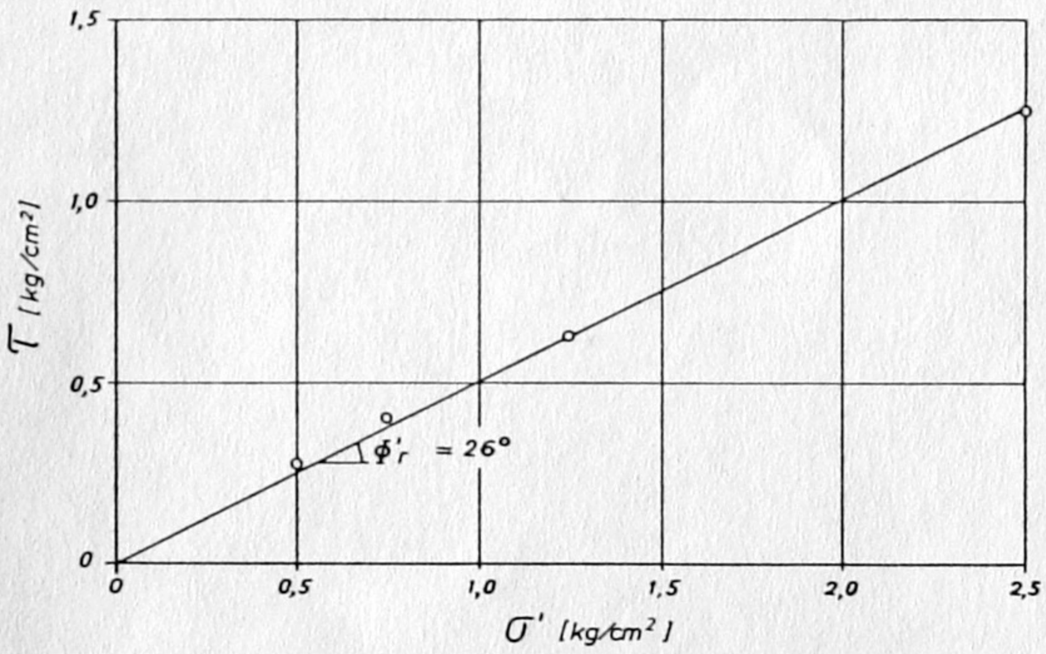


Fig. 15. Shear tests in ring shear apparatus placed in a water container for material D = 2 (broken, unweathered schist). Angle of residual shear strength Φ'_r after a shear deformation of approx. 50 cm.

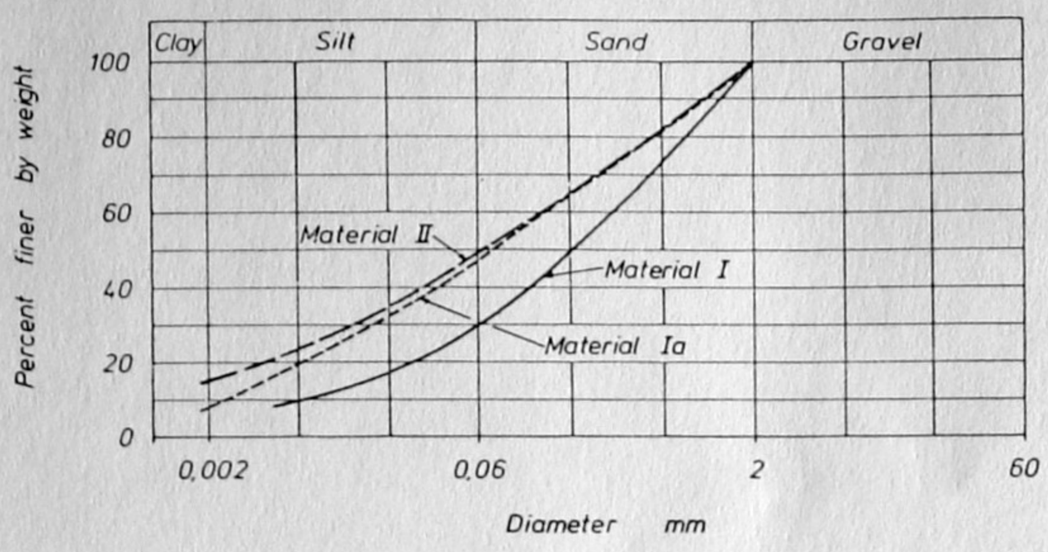


Fig. 16. Grain size distribution of the finer material Ia obtained from material I, with coefficient $C_c = 1$ and coefficient $C_u = 100$, and of material II, which has a plasticity index of 10 %.

weathering action, there is, in turn, an effect on the residual shear strength.

To investigate this behaviour, the material fragments were artificially reduced in size, in such a way that the material resembled material II (Fig. 16). The material did not, however, possess any plasticity.

Test samples were placed in the ring shear apparatus in the loosest condition without any compaction and saturated. Then the material was preloaded at a normal stress of $\sigma = 4$ kg/cm². After the pore water pressures had dissipated, the saturated material was unloaded to 0.55 kg/cm² and slowly sheared at constant volume at a rate of 1° per hour over a period of 3 weeks. The change of shear stress with shear deformation is shown in Fig. 17. The shear stress curve rises, rapidly and reaches a clearly defined maximum at a deformation of 6.1 mm and then falls quickly and levels out slowly to the value of the residual shear stress. Certain points a ÷ e have been inserted on the curve in order to clarify these points better in the τ–σ' diagram, (Fig. 18). During the test the normal stresses increased substantially, preventing a volume increase in the specimen. The stress curve is represented as a vector I in the same way as in the triaxial test (Casagrande and Hirschfeld, 1960; Huder, 1963), where the letters a ÷ e on the vector I correspond to the points on the shear-deformation diagram. The tangent to this curve gives a friction angle $\Phi' = 23°$ and an 'interlocking' resistance $c' = 0.30$ kg/cm².

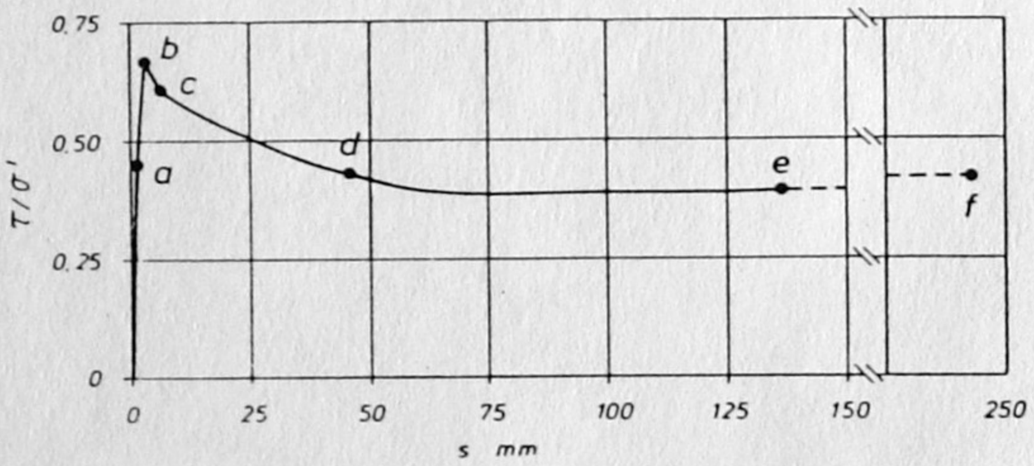


Fig. 17. Ring shear test on material Ia preloaded to 4 kg/cm² and then tested at the reduced pressure of 0.55 kg/cm² under constant volume conditions. τ/σ' as a function of shear deformation s of the test curve I, Fig. 18.

After reaching the shear deformation of 6.1 mm corresponding to maximum shear stress, the normal stress continues to rise whereas the shear stress falls. In this phase of shear, i.e. the post failure stage, wherein volume increases are prevented by allowing normal stresses to build up, the behaviour is shown up clearly in the vector curve. The residual shear strength is reached at $\Phi'_r = 23°$. The reduction of shear strength is associated with disintegration of the fragments and reorientation parallel to the maximum shear stress direction and a consequent tendency to volume increase. This is also apparent from the shearing behaviour during slow successive unloading of the specimen at the end of the test (see the $\tau - \sigma'$ diagram, Fig. 18).

If the material is placed in the apparatus as in the above described test, but not preloaded, the vector curve II results. The normal stress was kept constant during the test. The small decrease of normal stress is to be traced back to the correction of the friction in the apparatus. Of interest is the result that for a loose condition, as is required for the installation of saturated samples in the ring shear apparatus, the same friction angle of residual strength Φ'_r is obtained.

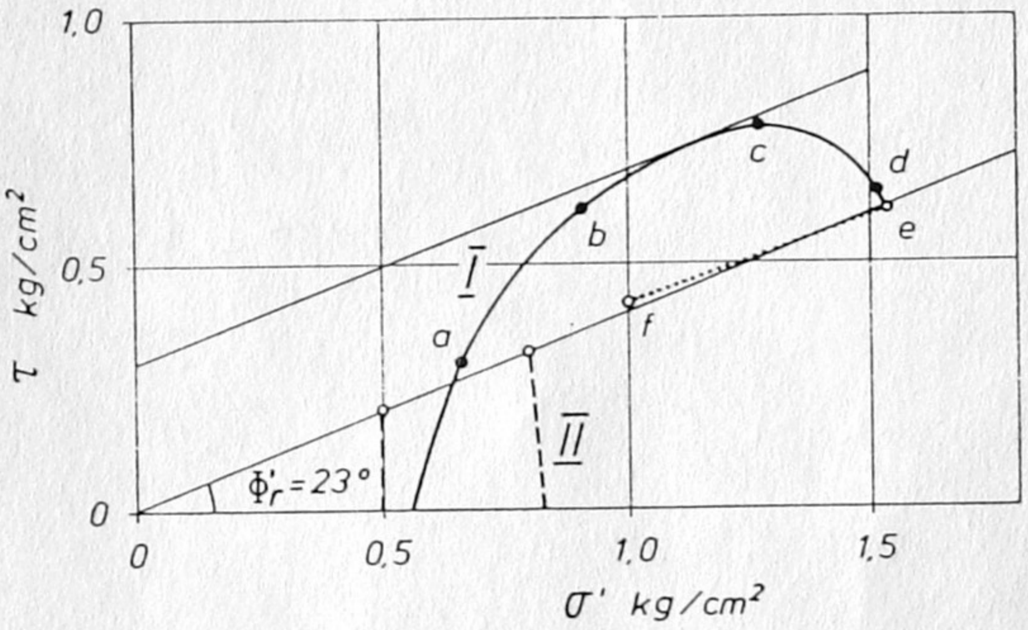


Fig. 18. $\tau - \sigma'$ diagram for shear tests Fig. 17. Curve I: Material in dense condition; Curve II: Material in loose condition.

In further tests the angle Φ'_r was determined using the direct shear apparatus (Bjerrum and Landva, 1966; Skempton and Petley, 1968). With this apparatus large shear deformations can only be achieved by reversals of the shear movement. Since particle orientation was already present, shearing was carried out in three different tests at constant volume with an initial normal loading, $\sigma = 1.0$ kg/cm². The tendency to volume decrease for samples in a loose condition made it necessary to compensate for the volume decrease by allowing the normal stress to reduce by a factor of 40 % until the particles in the new shear plane had adjusted and reoriented themselves, and then the normal stress was permitted to increase again. In Fig. 19 the influence of dilatancy on the shear reversals is also shown in vector form. The shear path in the direct shear apparatus is too

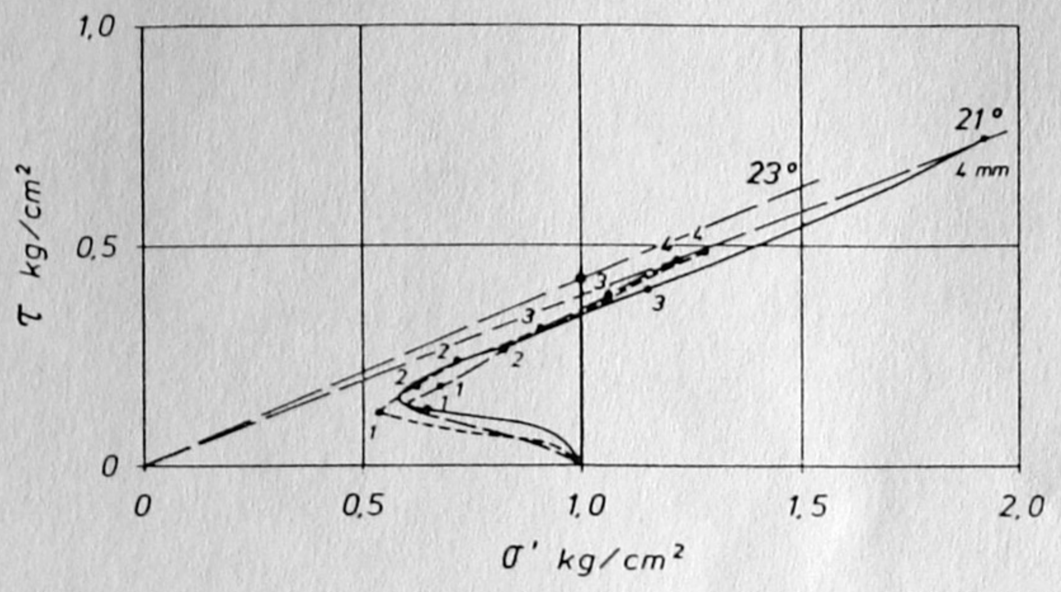


Fig. 19. $\tau - \sigma'$ diagram for direct shear tests on material Ia. After many reversals the shear surface was formed and further stress reversals produced no volume change in the specimen. The numbers shown on the vector curves signify the shear deformation in millimetres.

short to mobilize the complete friction angle $\Phi'_r = 23°$. The prevention of volume increases requires a greater normal stress than was applied in the initial stage of the test.

Both these tests and those in the ring shear apparatus exhibit a strong dilatancy effect, despite the fineness of the material. Once the shear surface has been formed the friction angle remains constant for the same direction of shearing, both for loading and unloading. If on the other hand the direction of shearing is reversed, a partial reorientation of the particles is needed, as follows from the results of the direct shear test. This behaviour is accompanied by dilatancy effects.

B. MATERIAL II:
YOUNGER BÜNDNER SCHIST (FLYSCH)

Material I, which has previously been described, comes from a tunnel roof fall on the Via Mala. It is therefore unweathered. The Bündner schist scree material has suffered various degrees of weathering over the years, and this in turn has an effect on the particle size distribution curve, i.e. the material possesses a continually increasing fine fraction, but does not possess any appreciable plasticity. The material described below (Material II, Fig. 16) is representative of a weathered young schist or Bündner schist Flysch, i.e. of the rock series III and IV (according to Fig. 1). In contrast to the older Bündner schists this material contains about 20 % illite. These clay minerals introduce a little plasticity into the material behaviour. The plasticity index is approximately $I_p \approx 10\,\%$.

The influence of weathering and the particle size distribution on the shear strength properties is shown for the following material, which is representative of many other materials tested.

The particle size distribution curve for material II is shown in Fig. 16. To allow comparison of materials I and II the corresponding curve for material I is also given. The relatively sharp increase of fine components may be clearly seen. In contrast to material I, it was decided not to test all size components. Only the components smaller than 2 mm were tested, as it was shown for material I that the size of the components has no direct influence on the shear strength. In this case the larger components are more completely embedded in the matrix of finer particles. The tests were carried out in the triaxial apparatus, in the direct shear apparatus and in the ring shear apparatus. Thus a direct comparison of the results was possible. Both saturated and unsaturated specimens were investigated. The setting up of the test with saturated specimens was made by means of the time consuming procedure of vacuum installation (Thut, 1969). The test results show clearly that the increase in shear strength of dense, unsaturated specimens depends not only on the interlocking effect, but also on the negative pore water pressures. The $p' - q$ diagram, with the parameter 'dry density', indicates that the material has a friction angle $\Phi' = 23°$ (Fig. 20). An increase in friction angle with the density of the unsaturated material was not registered. For this material the coarser grains are, however, better embedded in the finer fraction than in material I. The smaller friction angle is to be attributed to the mineralogical composition of the material.

Whereas for material I the increase of shear strength is largely dependent upon the interlocking effect, for material II the contribution of the negative pore water pressures is important. Fig. 20 shows the shear strength of the 80 % saturated material as a function of the dry density, γ_d. The friction angle Φ' is equal to the friction angle at constant volume Φ'_{cv}, which for material II is independent of density. It was also confirmed in this investigation that the angle Φ'_{cv} corresponds to $\Phi'_{\text{min.vol.}}$

For saturated samples, the influence of the negative pore pressures on the shear strength was investigated using back pressures in the pore water system. As would be expected for this slightly cohesive material, a pronounced intercept c' is obtained. This value of apparent cohesion c' is, however, reduced by more than 50 % as the degree of saturation increases from 80 %–100 %.

During the shearing process, material II exhibits a much smaller dilatancy effect than materials I and Ia respectively. This follows from the following test results obtained by using the ring shear apparatus. The saturated

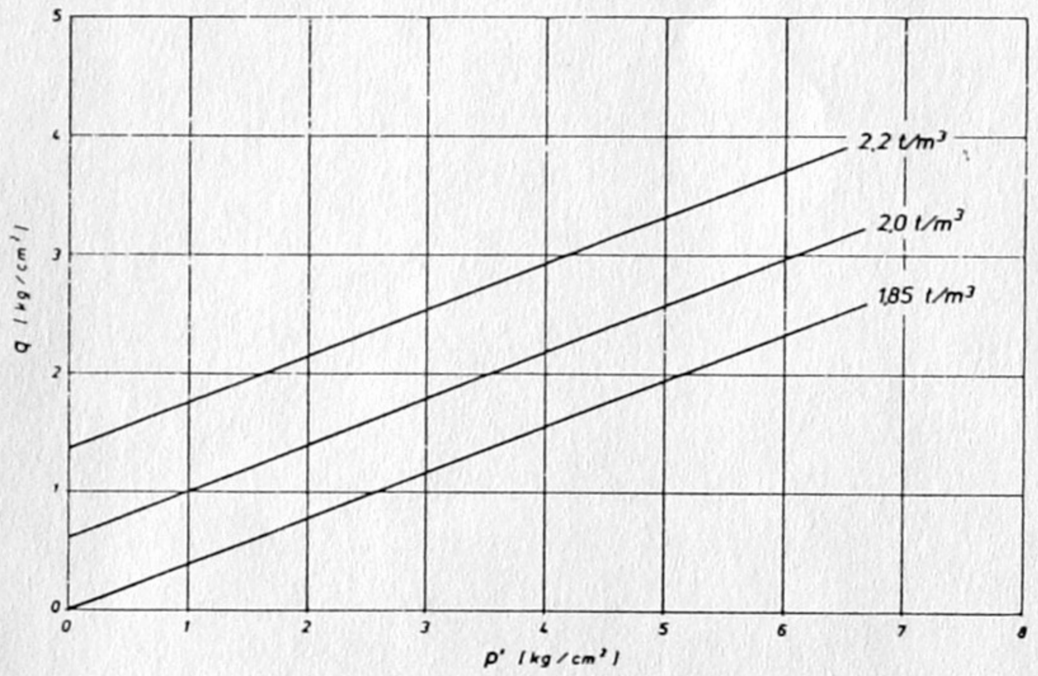


Fig. 20. $p' - q$ diagram of material II with the parameter γ_d, the dry density.

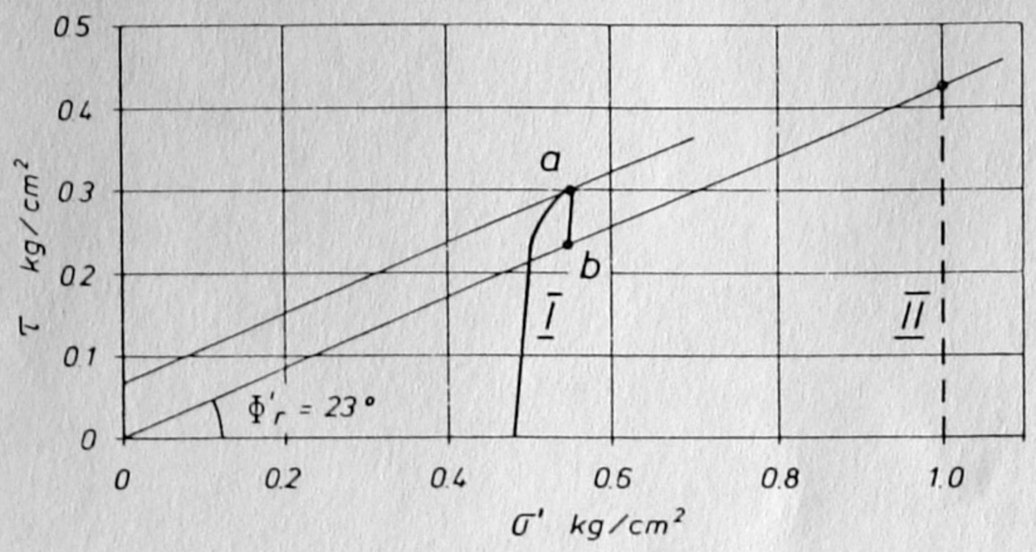


Fig. 21. Ring shear test on material II, preloaded to 4 kg/cm² unloaded to $\sigma' = 0.475$ kg/cm² and stressed at constant volume; see curve I. The same Φ'_r is obtained for the loose condition and for constant σ' curve II.

material preloaded to 4 kg/cm² was sheared at practically constant volume under a normal loading of 0.475 kg/cm². A peak shear stress is represented by $\Phi' = 23°$ and $c' = 0.06$ kg/cm² (Fig. 21). The corresponding values for material I were $\Phi' = 23°$ and $c' = 0.30$ kg/cm². The differences of behaviour are not only to be seen in the τ/σ'-stress path-diagram (Fig. 22), but also from the vector curve I of the test results illustrated in Fig. 21. For material I the normal stress increased threefold during the test (Fig. 18), but for material II only a small increase of normal stress was registered. The residual shear strength was reached for the loosest material condition with a friction angle of $\Phi'_r = 23°$. The test is also represented vectorially by curve II, Fig. 21.

As with material I a series of direct shear tests was also carried out. The same result for the angle of residual strength was obtained, viz. $\Phi'_r = 23°$. This value was reached after many reversals of the direction of shearing. For constant volume conditions the deformation was also represented by a vector curve, but this curve was considerably less characteristic than that obtained for material I (see Fig. 23). This behaviour is also an indication of the smaller dilatancy of this material compared with material I. The reduction of shear strength of material II proceeds in a different manner to that of material Ia. According to the results of the triaxial tests, after overcoming the interlocking action, an angle of friction Φ' is obtained which is equal to Φ'_{cv}, where $\Phi'_{cv} = 23°$ is also equal to the angle of residual strength. In the case

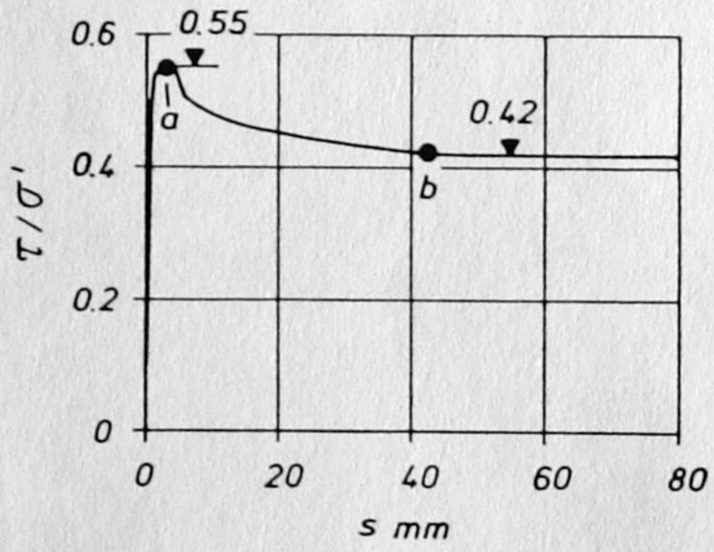


Fig. 22. τ/σ' as a function of the shear deformation s of the test, curve I, Fig. 21.

of the coarser material Ia, however, the failure plane must first be formed, which involves crushing and reorientation of the flaky micaceous fragments, before the state of residual shear strength is reached. In Fig. 24 test results are presented, which have been statistically evaluated. c' has been estimated as a function of γ_d by means of a regression curve with a correlation coefficient $r = 0.95$.

In the loosest condition with a dry density $\gamma_d = 1.85$ t/m³ the material exhibits a friction angle at constant volume, $\Phi'_{cv} = 26°$. With increasing dry density the shear strength increases rapidly. This increase is attributable to the component c', the interlocking effect and the apparent cohesion due to incomplete saturation, which is explicitly given with c' as a function of γ_d in Fig. 24. The angle of internal friction Φ' is equal to Φ'_{cv}, which for material II is independent of density. It was also found that $\Phi'_{cv} = \Phi'_{\text{min.vol.}}$ An overloading of the material leads to a reduction of shear strength, the shear deformation producing a reorientation of the particles, which leads to $\Phi'_r = 23°$. If saturation occurs, the behaviour is characterized by

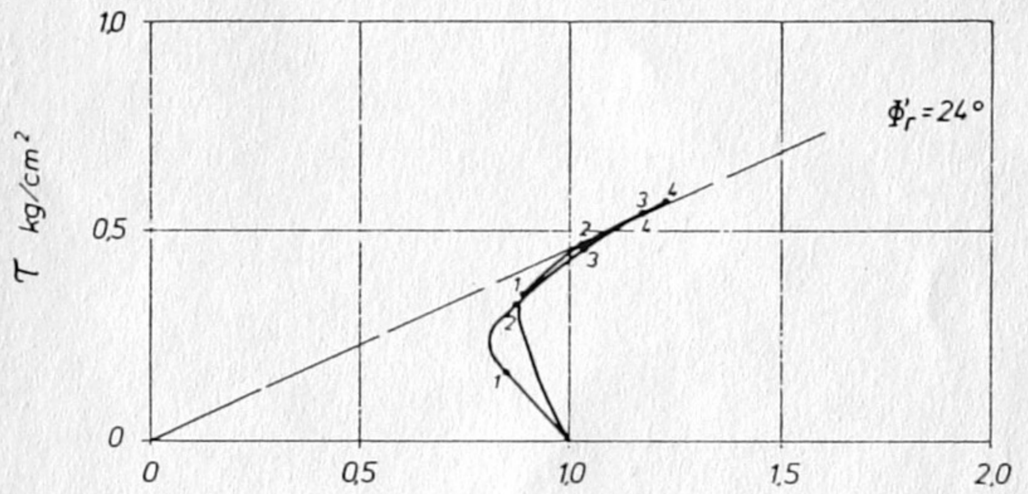


Fig. 23. $\tau - \sigma'$ diagram for the direct shear test. After reaching a constant value of τ/σ' the movement takes place with no volume change of the specimen. See Fig. 19.

the elimination of the negative pore water pressures, and thereby of the reduction of c'. It is interesting to observe that the angle of residual strength Φ'_r is independent of the mineralogical-petrographical composition for the two materials.

a) *Factors Influencing the Shear Strength*

Bündner schist is a marine sediment that has been subject to varying degrees of metamorphism, and has been moderately to highly overstressed in mountain building. In its natural formation it is, on the whole, unsaturated. The highly tectonic stressing of the rock has given rise to fissures and a loosening of the material depending on the structure, so that the rock occurs mainly as a compact heap of fragments. The lateral expansion produced in the rock facilitates the ingress of water which in turn activates the weathering process. At the same time the erosional effects associated with valley formation and the accompanying destressing of the rock play an im-

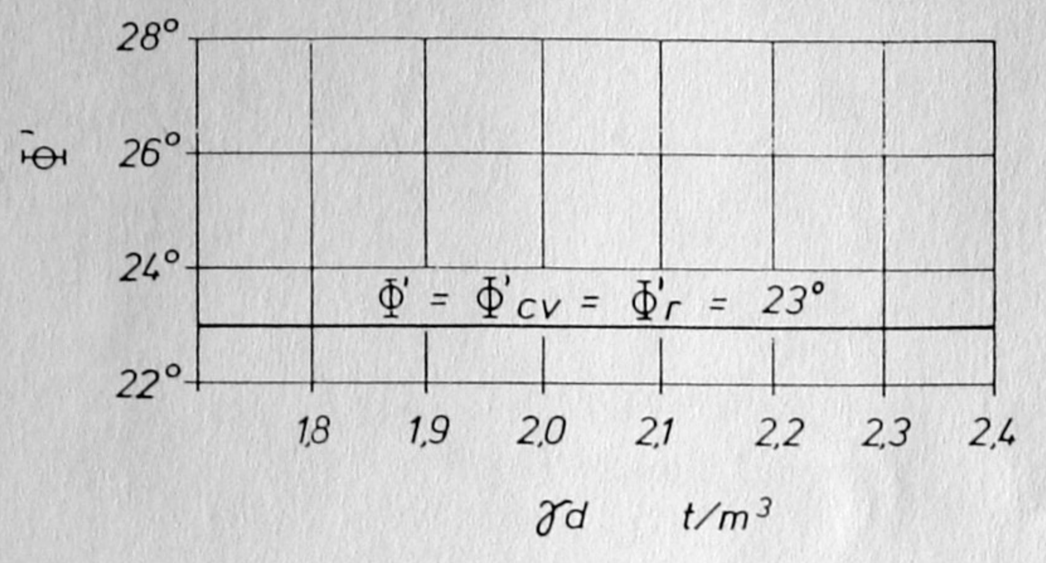


Fig. 24. Φ' and c' as functions of dry density γ_d for material II.

portant part. The high weatherability of the scree material resulting in continuous compression due to crushing of material at the highly stressed, weathered, fragment contact points is one reason for the initiation of creep movements. The creep movements and the continuing weathering action lead to an acceleration of the movemovements. Weathering is more active in younger Bündner schist (Flysch) and becomes more and more important with the passage of time, and combined with the increasing decarbonisation produces a 'remoulding' of the material.

The shear strength of a disturbed Bündner schist consists of a frictional and a cohesional component, where the latter is traceable to an interlocking of the fragments. The overstressing of the material, either artificially through mechanical action or naturally through weathering action, progressively reduces the cohesional component of strength (Schmertmann and Osterberg, 1961). The shattering of fragments, due to the initially compact condition, produces a volume increase. Thus the conditions favouring a reduction of strength to the value of the residual strength are provided. The latter is estimated at $\Phi'_r = 23°$. The micaceous flakes in materials I and II plus the clay minerals in material II are responsible for this strength reduction. Despite similar grain size distribution the two materials are different with respect to their plasticity and dilatancy behaviour during shearing. Material I in contrast to material II has a pronounced dilatancy behaviour, as shown in both the ring shear tests and the direct shear tests under constant volume conditions. The dilatancy effect with material I is so great that it is questionable whether residual strength parameters should be investigated using the direct shear apparatus under constant volume conditions.

The increase of plasticity must also have an influence on the shear strength of the material. Saturation of material II causes a slight decrease in strength. This is clear from the tests and must not be neglected. The formation of the material by the lateral expansion and loosening of a schist layer gives rise to unsaturation. It can be assumed, that by continual seepage saturation is achieved, which, however, was not possible in laboratory tests. The decrease in permeability with increase in plasticity has, on the other hand, a counter effect. The behaviour of the tunnel for the Julia power station (Tiefencastel) cannot be explained on any other grounds than that the mass of creeping material is saturated. The water not only has an influence on the weathering action but also directly reduces the strength when saturation eliminates the negative pore water pressures. Thus the effects of water must not be regarded as secondary.

4. Examples of Creep Slopes

A few case histories are given to illustrate the various effects of creep in Bündner schist, with regard to its cause, the nature of the movements, and its economic importance.

Case histories (a) to (e) are referred to on the geological map (Fig. 1). Here a short summary is given of these case histories, and in the succeeding sections each case history is handled separately in some detail.

The creep movements of the slope at Schuders (a) have been caused primarily by the progressive toe erosion of the slope. As a result, the material of the rock slope is loosened due to a disturbance of the rock formation. Fissures formed in the upper parts of the slope cause a total reduction of the tensile strength, so that small sections of the slope become unstable, and their movement is accelerated until finally they reach new positions of stability. During this time the access to the valley is interrupted because of the material blocking up the roadway. The primary cause of these movements (i.e. the toe erosion) must be remedied.

The creep movements at Lugnez (b) have been made well-known since 1887 through the size of the area affected and the rates of movement as established by the results of the land survey. However, damage is relatively seldom recorded, and the maintenance costs for roads and for the water supply system for the villages is not above normal. The cause of the movements is here again due primarily to toe erosion of the slopes, but the creep behaviour is only plausible if water movements in the slope are taken into account. A depth of creep zone of 50–100 meters may be assumed for this case.

To what depth creep movements can occur is shown by the creep slope at Klosters (c). The slope, which lies to the south of Klosters, was well known on account of

the slides that had occurred there, but the extent of the creep movements first became apparent in connection with the survey carried out for the loop-tunnel. Unfortunately, measurements were first begun when new buildings in the area were already, after only a few years, showing signs of deformation and damage.

The Cebbia slope (d) was known to be subject to creep movements, but no information was available as to the actual magnitude of the deformations involved. During the planning stage for the construction of a new road to traverse the slope, a thorough check was made on the movements of the slope, and it was possible to find a constructional solution to the problem, taking full account of the geotechnical aspects.

The creep movements at Schin (e) serve as an illustration of the substantial creep movements that a small additional loading may cause, if the geotechnical properties of Bündner schist are given too little consideration. Only by means of extensive remedial measures was it possible in this case to bring the creep movements slowly to a halt.

a) *The Slide at Schuders*

Of all the numerous slide zones in Bündner schist region, that at Schuders is best known, not on account of its extensiveness but rather because of its activity. The attention of the public was drawn to the serious consequences of a sudden slide here through the newspapers and in Parliament.

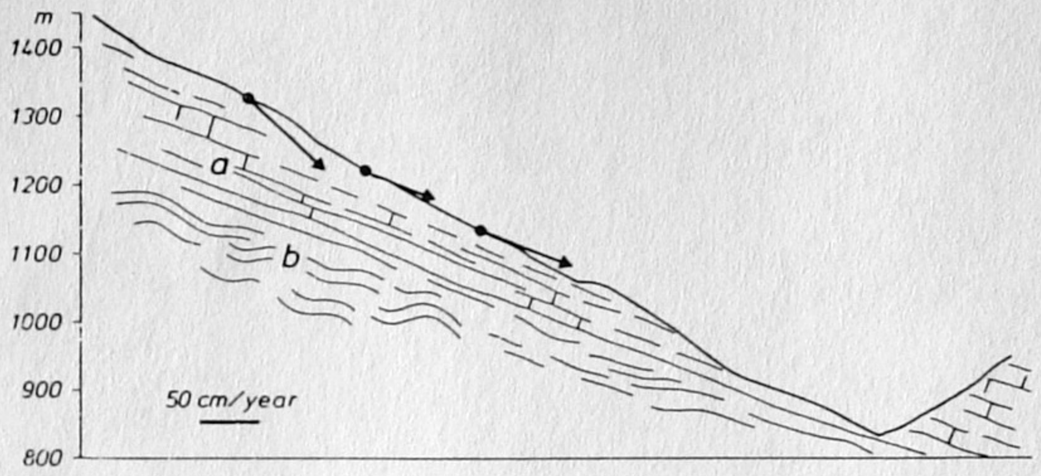


Fig. 26. Schematic geological cross-section (Nänny, 1948). a) Sassauna Series: Limestone, abundant sandy limestone, schist; b) Valzeina Series: Argillaceous schist, thinly bedded siliceous limestone. The vectors of velocities are shown as mean-values for the years 1951–1956.

The area lies on the northern slope of the Schraubbach, a valley branching off to the right of the main valley of Prättigau, and the slope itself borders directly on the village of Schuders. The breadth of the sliding zone is about 700 metres and extends upwards to the sharp crevice at 1400 metres above sea level and downwards to the Schraubbach gorge at approximately 800 metres. Fig. 25 gives a good picture of the slide area.

The angle of inclination of the slope α averages between 30° and 33°. The highly tectonic schist lies discordant to the slope. It is weathered to different depths (see Fig. 26 for a schematic representation by Nänny, 1948) and is covered by a thin layer of humus.

The first signs of movement date from the year 1902. The continuous deformations since then have not only caused the devastation of forestry and cultivated land, but have also rendered houses to be uninhabitable and destroyed cattle sheds. The road to Schuders constructed in 1931 was closed to traffic for several years despite the costly inconveniences this entailed. The repeated corrections to the road necessitated a check to be made on deformations, and thus gave the opportunity to make periodic measurements following the progressive movements of the slope. To prevent landslides affecting the lower lying communities, especially that of Schiers, constructional work had to be carried out in earlier years (1896, 1910, 1926) on the highly eroded banks of the river Schraubbach.

The area of the slide at Schuders may be clearly seen in Fig. 27: the light areas show up the tension cracks and lower down are the areas of surface erosion, which completely lack vegetation. The new road, built in 1931, is also distinguishable where a bridge crosses the Salgina gorge – a famous 'Maillart' bridge (Bill, 1949) shown in the bottom left hand corner of the figure – and in the adjoining slope construction with the crossing of the sliding zone to the village Schuders. The deformations in the slope were measured over a considerable period of time by means of a topographical survey. Kobolt (1960) has described the complications met with in the

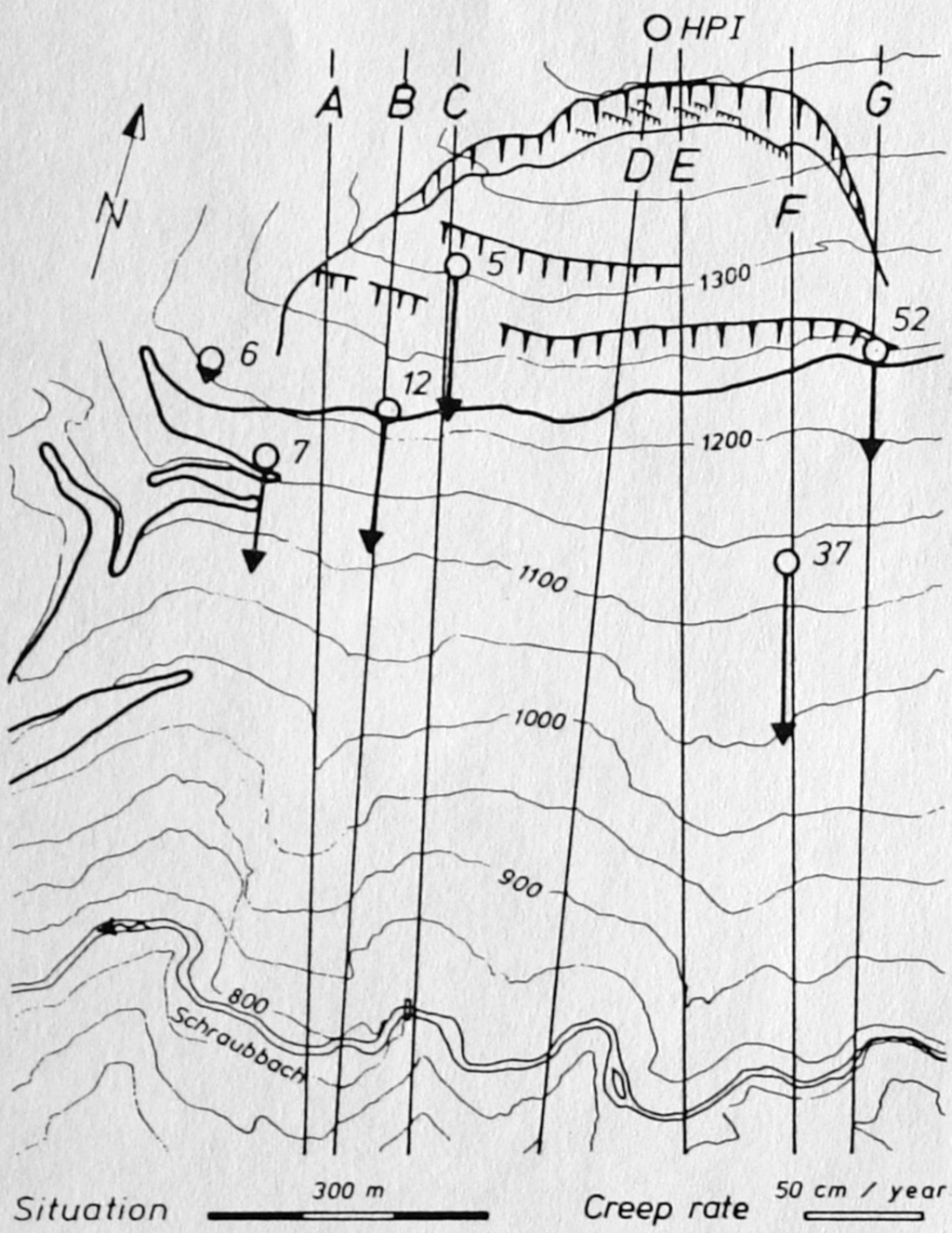


Fig. 25. Plan of the area of earth slides showing the selected survey points and the pronounced 'set' caused by the slide. The displacements at the survey points are shown on the plan for the period 1951–1956. The barrier constructed in the river bed is also shown, cross-section C.



Fig. 27. General view of the earth slides area at Schuders. Lower left: Highway leading to the Maillart bridge over the Salgina gorge.
Photo: Crameri, Cantonal Engineering Office, Chur.

evaluation of the measured results if the survey is carried out over a long period of time; the fixed points on the opposite slope move themselves at a rate of about 5 mm a year.

Fig. 26 shows a geological section through the slope. The vectors for the rate of creep for the period 1951–1956, i.e. the period when the greatest deformations started – have been superimposed on the geological section. The survey points are given in Fig. 25. The creep rates are also shown in this figure. Since the investigation was primarily concerned with the behaviour of the road, the survey points have been concentrated in the area adjacent to the road. The land far below the roadway is rather inaccessible and the line of sight to points in this area is somewhat obscured. To check on the movements photogrammetric measurements were made in 1946, 1958, and 1964. Using this method it has been possible to observe the mass movements in the area, movements extending well outside the area enclosed by the survey points. The movements are shown in Fig. 28 for the geological profiles of Fig. 25. The deformations of the profiles are clearly seen from this figure. In the period 1958–1964 lower-lying parts of the slope were affected, giving a progressive downhill movement of the whole slope. Recently, however, these movements have stabilized themselves. If one compares this fact with the directions of the creep velocity vectors, it is apparent that in the zone of tension cracks the vectors are steeper than the inclination of the slope, whereas towards the valley bottom the reverse is true, but the creep rates are smaller. Measurements are lacking far away from the area through which the road passes, but indications are that small movements occur even down to the river bed. Some survey points have been lost owing to large movements in the course of time.

A better analysis of the creep behaviour can be obtained from the creep rate diagram. The first movements to be noticed could not have been very large, probably amounting to no more than 5–10 mm per year.

The creep movements have been noticed particularly since constructing the road. To what extent the road works, with the construction of hairpin bends and with cuttings, have caused a local acceleration of movements, is not possible to find out afterwards. When measurements were begun in 1942, movements of more than 1 m/year were registered. Since 1945 a large number of measuring points have been set up. Fig. 29 clearly shows, with the changes of elevation and position, the acceleration of the movements since 1945, with creep rates of up to 10 m/year.

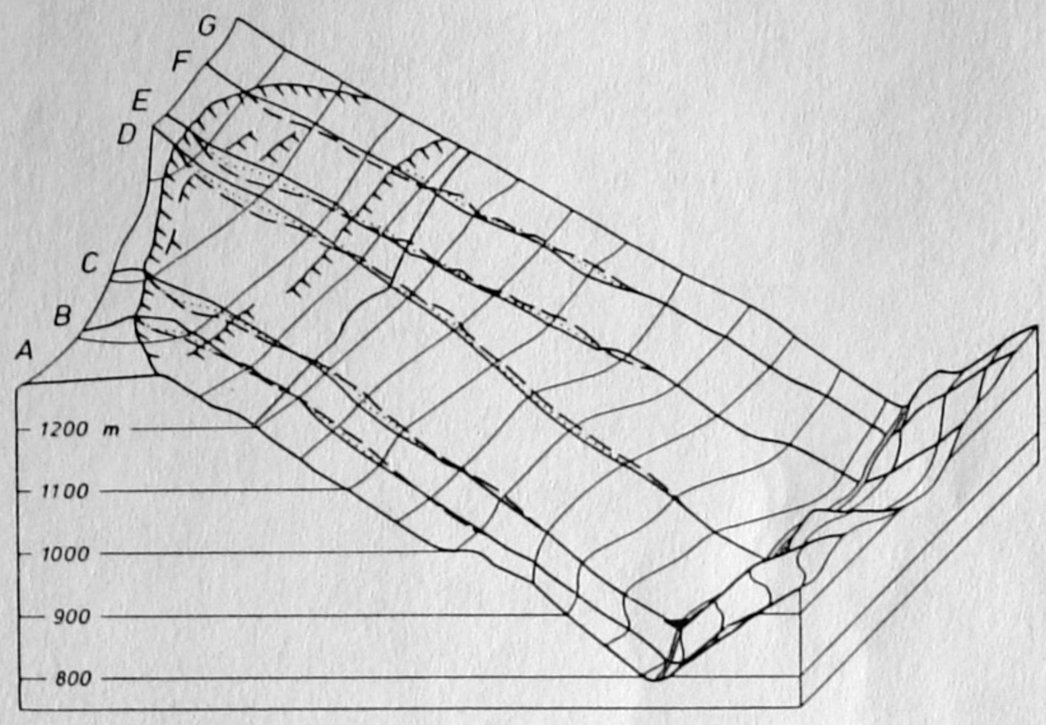


Fig. 28. Cross-sections A to G from Fig. 25, traversing the area of the slides. The magnitudes of the displacements were obtained by means of photogrammetrical methods. The movements of the masses of the second-order slide are shown. The full line shows the situation in 1964, the dotted line in 1958, and the dashed line in 1964.

It is not easy to analyse the complicated nature of the movements. Much has been written on this subject, especially with regard to the ground water conditions in the slope. That the ground water contributed importantly to the movements when they were at their greatest, is obvious. The cracks opened up by the movements facilitate a greater ingress of water leading to a reduction of shear strength. But this hypothesis by itself does not explain the starting up of the movements.

Due to erosion in the valley bed more and more layers are exposed. The firmer the layers, for example of a limestone bed, the greater the unloading energy of tectonic origin is. The release of energy is accompanied by the removal of strain, which can affect weaker zones of the schist, and produce progressive (i.e. leading backwards and upwards from the toe of the slope) creep failure.

Since the slope is more or less inclined at the angle of residual shear strength of the schist, and the toe of the slope no longer provides enough support, progressive failure takes place in the slope and may ultimately stretch upwards to the top of the slope exploiting the presence of weaker zones and existing failure planes. Thereby sections of the soil mass previously held together by tensile forces are now loosened and set into motion. If, in addition, there is an ingress of water into the cracks and fissures, the motions may be accelerated. This condition had probably been reached in 1951 and accelerated motions occurred until 1958, with peak rates of creep of 10 m/year. At this time the mass movements were of such magnitude that they balanced each other out, the upper half becoming stabilized on account of the new position reached by the lower half. This is a clear consequence of sliding in nature, and it may be partly recognized in the section profiles. The re-positioning of the sliding mass increases the computed safety factor by about 5 %, which suffices at least for a while to bring the slide to rest in a secondary stage of motion. The creep rates in the slope have decreased since then, and values corresponding to the period 1945–1951 have been recorded.

The movements in the period 1956–1960 may be explained in this manner. Whether or not these movements can already be described as slides is an open question. The masses cannot, however, be regarded as stable, as the slightest disturbance of the slope will set them in motion again.

The motions may thus be characterized as progressive failure caused by river erosion of the toe of the slope, whereby a progressively larger wedge of soil begins to move and finally comes to rest in a more stable position.

According to the engineer responsible for repairs to the road no observable movements are at present taking place in the slope.

For the purpose of remedial works a sum of 8 million Swiss francs was allocated by the Government and the cantonal authorities. The erosion had to be brought under control so that the damage did not become irreparable. It is hoped to achieve stabilization through a series of barriers, the construction of which was started in 1971 and is still proceeding. The high reinforced concrete barriers help to stabilize the slope by preventing further erosion. The highest of these is 18 m high and was completed and put into operation in the middle of December 1974 (see Fig. 25, profile C). By these means the bed of the river was raised 14 m, and behind the barrier a considerable build-up of material occurs. The river diversion was accomplished by means of a conduit in rock through the left bank of the river. Thereby further erosion of the right bank (i.e. on the side where creep movements occur) is hindered. According to a personal

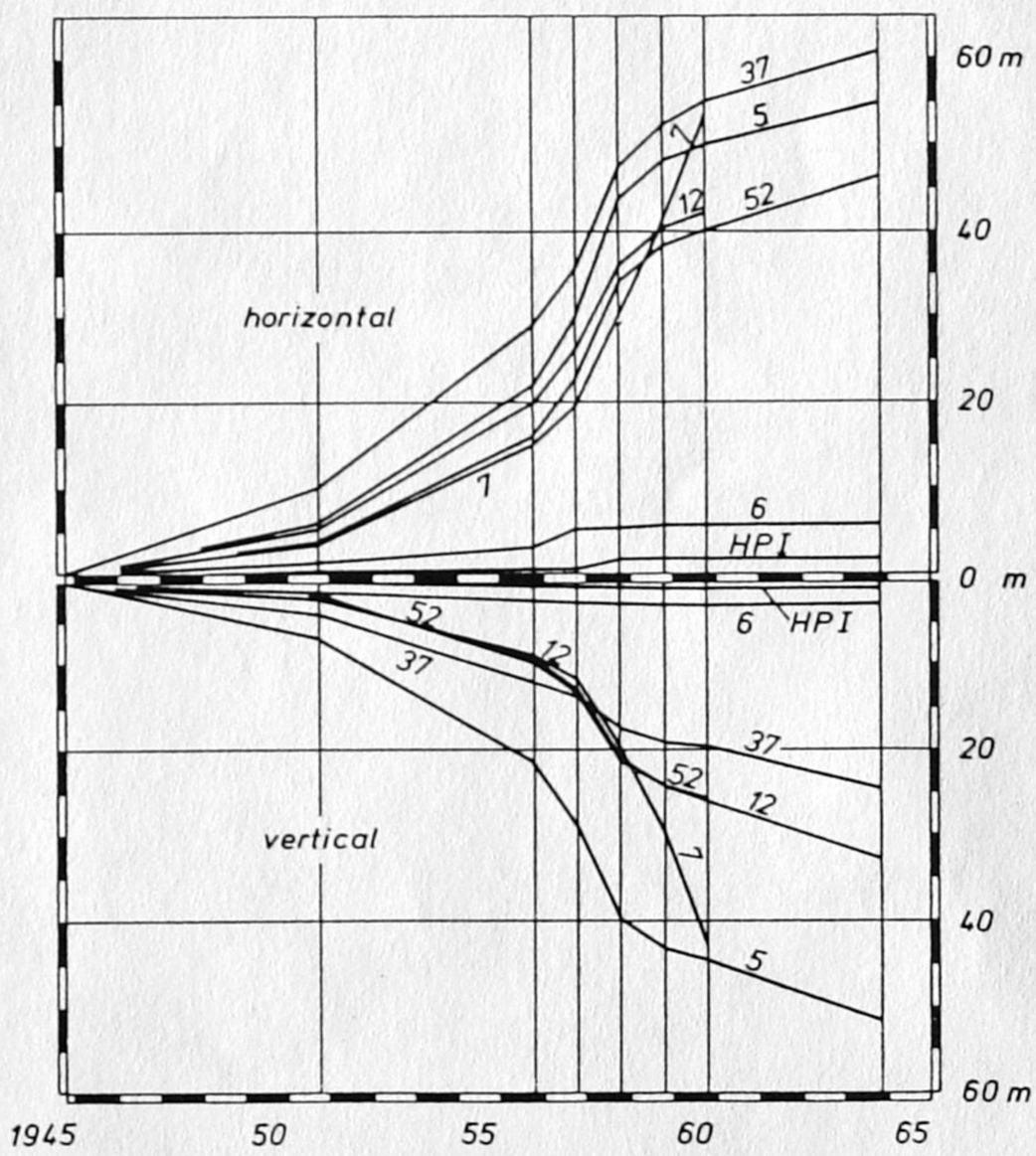


Fig. 29. Creep rate diagrams for horizontal and vertical movements of various 'fixed' points on the slope.

communication of the consulting geologist, the slope is already showing no signs of further movement. Drainage of the slope has also contributed to the cessation of secondary creep movements.

At the time of the greatest movements some springs ran dry, which lately have reappeared, now that the slope movements have ceased.

Measured deformations can be explained broadly on the basis of the geological structure. The average dip of the layers is about 22°–26°. Through the metamorphism the rock was overstressed, which caused fracturing and in turn made the rock more susceptible to weathering. The continual erosion exposed more and more layers; these experienced stress release resulting in further movements at the toe of the slope. On predisposed failure surfaces the resistance (i.e. angle of friction) is reduced to Φ'_r. By removing the base support greater sections of the slope are made to 'hang' on the remainder of the slope. On account of the tensile stresses the rock fractures, as it yields to the deformation, and cracks are formed. The failure zones propagate backwards up the slope and emerge in its upper part. In some sections there is a complete loss of tensile strength so that they are solely dependent on the shear resistance on the layer interface; this, however, is insufficient to maintain equilibrium. The new movements reduce the shear resistance even further, so that greater accelerations occur, as was the case in the years 1958–1960. Through the new configuration of the sliding masses the movements have slowed down and apparently the slope is once more stable. A permanent stability of the whole slope is, however, only possible when the cause of the creep movements, i.e. the erosion work in the river bed, is prevented.

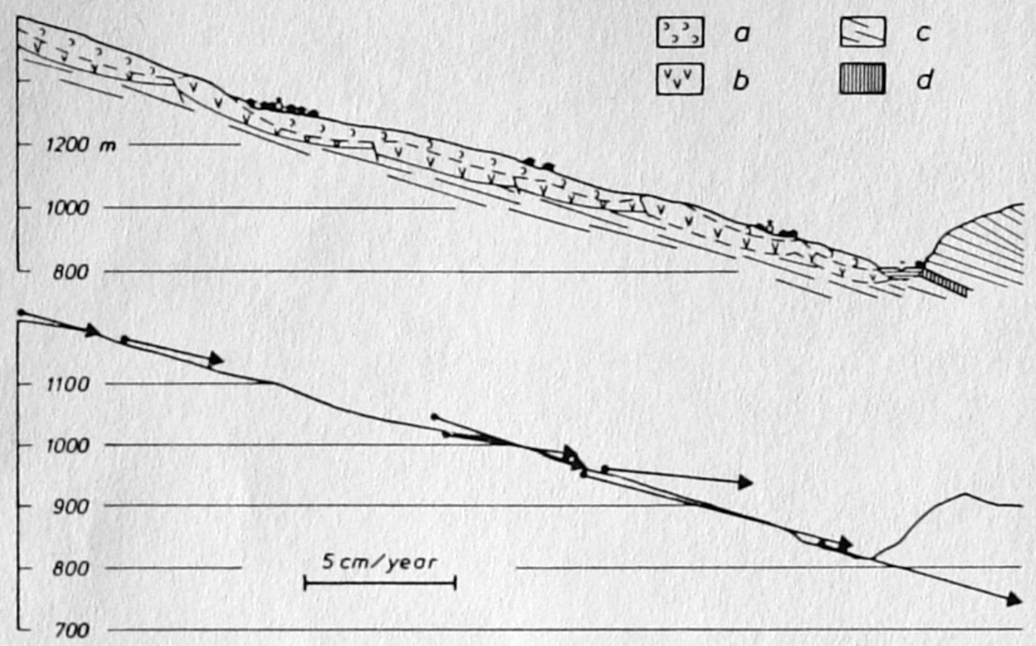


Fig. 30. Schematical geological cross-section with the bedding layers. a) Talus slide – rock nearly always participatory in the slide. Sliding surface not known. b) Disturbed Bündner schist; thickness of the disturbed area not known. c) Bündner schist (argillaceous schist, siliceous limestone). d) Triassic rocks (Rauwacke, gypsium). (Winterhalter, 1948). The surface displacement vectors are also given.

b) *The Creep Slope at Lugnez*

The creep movements in the valley of the Lugnez, a tributary of the Rhine, have been known for some time. Although the size of the area is much greater than that at Schuders, it has not received a great deal of attention because the movements are steady and affect large surface areas. The difference in behaviour between this creep zone and that at Schuders lies primarily in the geological structure and the hydrological relationships of the left side of the valley of the Lugnez at the level of the village Peiden.

The Bündner schist is relatively flat, dipping in general at an angle of about 15° in the south easterly direction, as may be seen from the section in Fig. 30. The layers are approximately conformable with the slope. In places, on top of the Bündner schist, there are fairly thick deposits of loose scree material and moraine. The history of the valley formation is, however, not simple. Today various interactions may be recognised in the landscape, such as between glacial deposition, glacial and water erosion, and slides. These relations have not been completely explained up to the present day.

The Bündner schist in this area is composed of thin plate-like, sandy-calceraceous deposits with numerous joints, containing quartz infillings. The rock is rather hard, though also fairly brittle, and contains numerous, embedded, thin sheets of mudstone, which are often found in a soft remoulded state.

This geological structure, with its softened mudstone lenses, favours the formation of slip surfaces. In a compact condition the Bündner schist is practically impermeable. Since, however, the mudstone layers contain many fissures, and the structure is disturbed by the movements, it is not only possible for water to penetrate the upper layers, but also to substantial depths.

The average gradient over the total height of the slope from the river bed at elevation 800 m to the top at elevation 2060 m is 15 degrees. Certain parts, however, are inclined at about 30 degrees, so that below the village of Peiden the average inclination is about 25 degrees (see Fig. 31). The question, how instabilities are to be explained here, may now be posed. With the inclination of the layers parallel to the slope the assumption of a disturbance of the material leading to a reduction of shear strength is not necessary. The rearrangement of the grains is likewise much in evidence, due to the schist structure. The movements in the slope as a whole can only be explained by the additional factor of water action in the fissures, since the smallest value of Φ'_r come across in Bündner schist until now is 23°. With this value of frictional angle it is only possible to explain the local movements. With a pore water pressure of approximately one-third the overburden pressure i.e. B= 0.33, it is, on the other hand, possible to explain the instability of the whole of the slope.

The cause of the instability does not lie, in this case, entirely in the erosion of the toe of the slope. The movements do indeed propagate upwards from the foot of the slope, but they are greatly assisted by the foliation of the material and the water in the fissures, which must be under appreciable pressure. If, however, one analyses the movements of isolated points, one realizes that these points have displacement vectors with magnitudes, inclin-

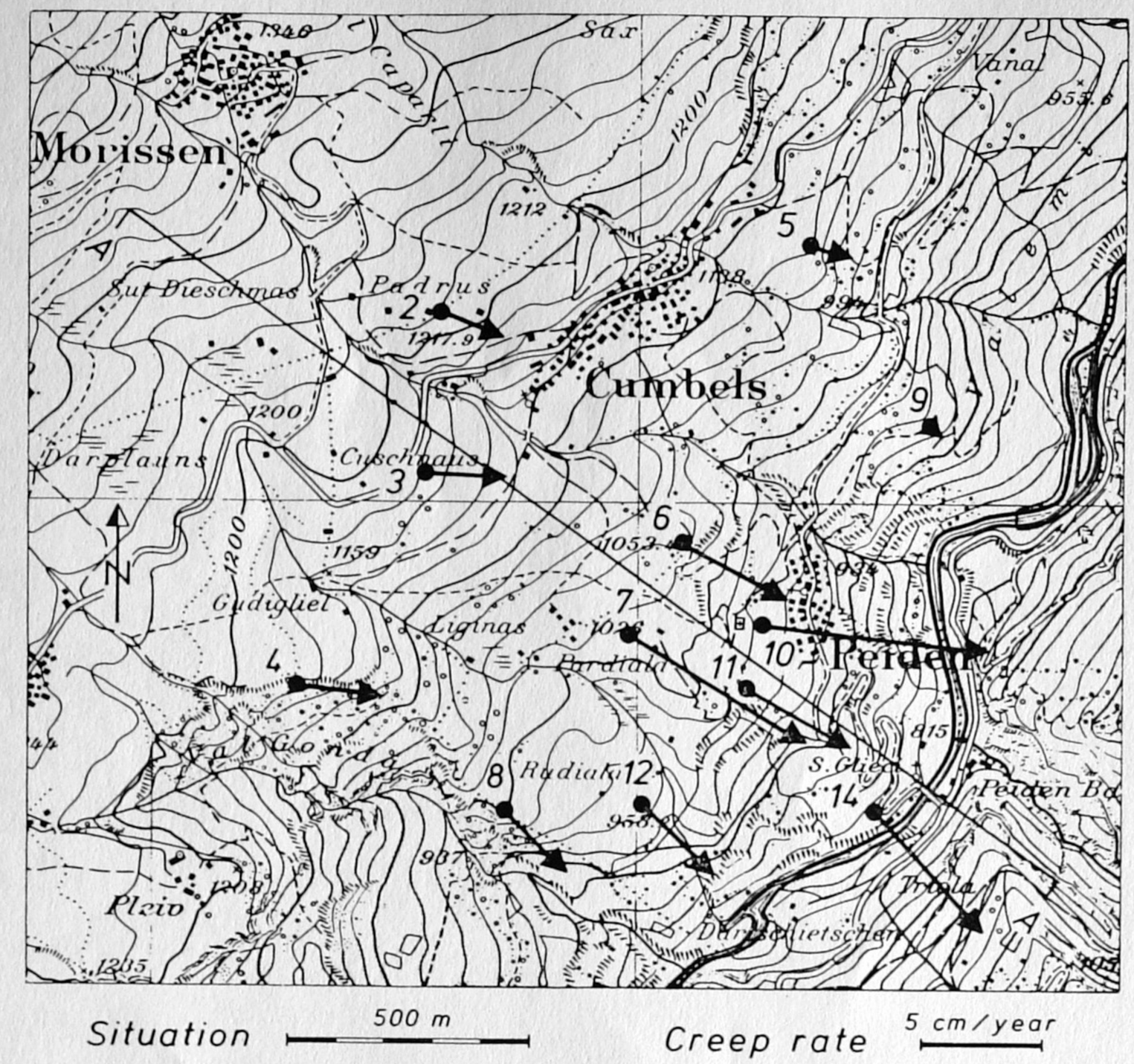


Fig. 31. Plan of a part of the slide area with creep rate vectors.
Reproduced with permission of the Topographical Survey of Switzerland of 8th Oct. 1974.

ations and, to a certain extent, also directions different from one another. Over the whole slope there are, in addition, tension and compression zones with different creep rate vectors, and also neutral zones in which the tensile and compressive forces are equalized.

These characteristics are easily recognizable in the present-day landscape. The creep zone extends to the full height of the slope up to a height of 2064 m above sea level and over a breadth of several kilometres. As seen from the rear side of the mountain the depth of the sliding surface is in places 60–200 metres.

These and other observations permit an estimate of the depth of the tension cracks. The estimated depth is 50–100 m. But these depths unfortunately have not been reached in site investigation borings. The deepest borings were only 30 m into the scree material. Also, the measurement of pore water pressures is restricted on cost grounds, as the depths of interest extend to 100 m. An estimate of the water pressures must suffice. In order to carry out an exact study of the creep movements a very extensive area would have to be surveyed, also depthwise. The area involved is 25 square kilometres or about a thousand million ($1 \cdot 10^9$) cubic metres of material undergoing movements. (Figs 31 and 33).

This area has been surveyed since 1887. In the intervening period a large number of control points have been set up, so that a fairly clear picture is available with regard to the creep rates at the surface of the slope. The number of measuring points has been greatly increased since 1947. The creep rates in individual zones have remained more or less constant since 1887, but they vary greatly from zone to zone, with rates of up to 37 cm per

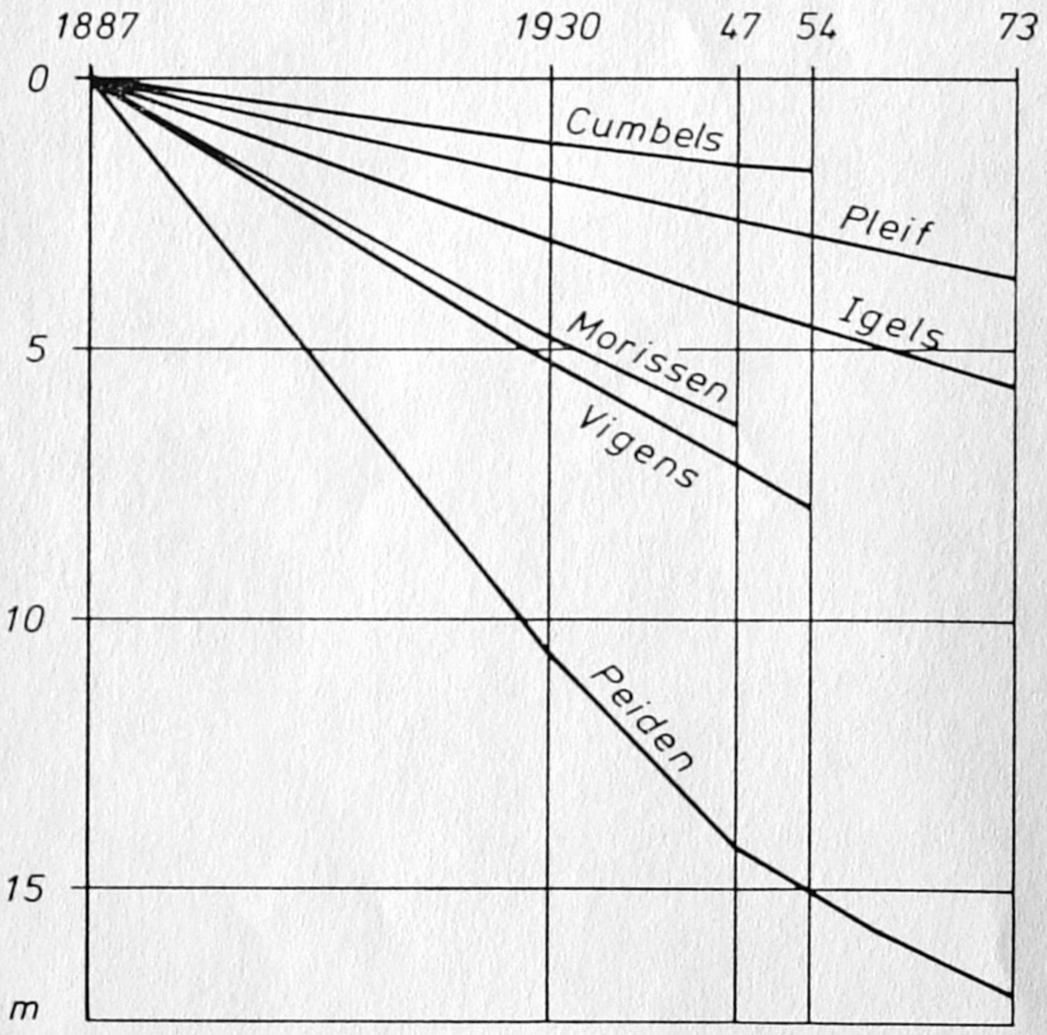


Fig. 32. Horizontal displacements of churches in the slide area over the period 1887–1973.

year (see Fig. 32). The variation in the creep rate, if continuity is pre-supposed, is explainable as follows: there is a deep-seated sliding surface, the exact depth of which is difficult to ascertain, but it must, if the volume of the sliding mass is constant, taper to form a wedge, so that in the lower half of the slope the creep rate is greater. The basic creep rate, i.e. of the whole of the sliding mass, is shown in Fig. 30. A secondary sliding mass, which overlies the basic sliding surface, takes in the zone of Peiden and also of the village of Morissen. In addition, a contribution to the creep rate due to solifluction must be included, and this produces the rather complicated distribution of creep velocity mentioned above (see the section, Fig. 30). This alone explains the nature of the creep movements of the slope at the village of Peiden, which, without there being outward signs of deformations in the buildings themselves, including the church, is slowly moving. The church has moved over 15 metres in the period of 1887 to 1973, but the 25 m high church tower still remains vertical. The other buildings in the village, too, show no evidence of differential settlements or deformations. Cracks, which have appeared in walls, could be explained equally well by other means; they are mainly in houses that are more than 300 years old. This secondary movement, which is recognizable by two abrupt changes in the surface profile, i.e. by a 'set', may be clearly seen at a point on the roadway, where large depressions appear, and also at a point above the village on the walls of the cemetery; where the first 'set' crosses the cemetery, the walls have been displaced by as much as a metre.

In comparison to the total movements of the village the secondary deformations are indeed small. But associated with these deformations are very large compressive forces as indicated by measurements at the points of the three horizontal polygons following contour lines (cf an open traverse) and a line up the slope normal to the contour lines. (See the situation in Fig. 31, in which a few points with the vectors have been entered.)

These polygons were surveyed by the Geodetical Institute at the ETH in the years 1955 – 58 – 60 – 73. The survey points are 50 metres apart, but for sake of simplicity only a few of the points have been shown. In the plan the displacement vectors for the period 1960–73 are shown (horizontal components), while the total displacements for the same period are shown in the sections (see Fig. 30). It may be seen from Fig. 32 that the movements are regular, so that to illustrate the direction of movement only the vectors obtained for one period of measurement need be given. The maximum deviation of

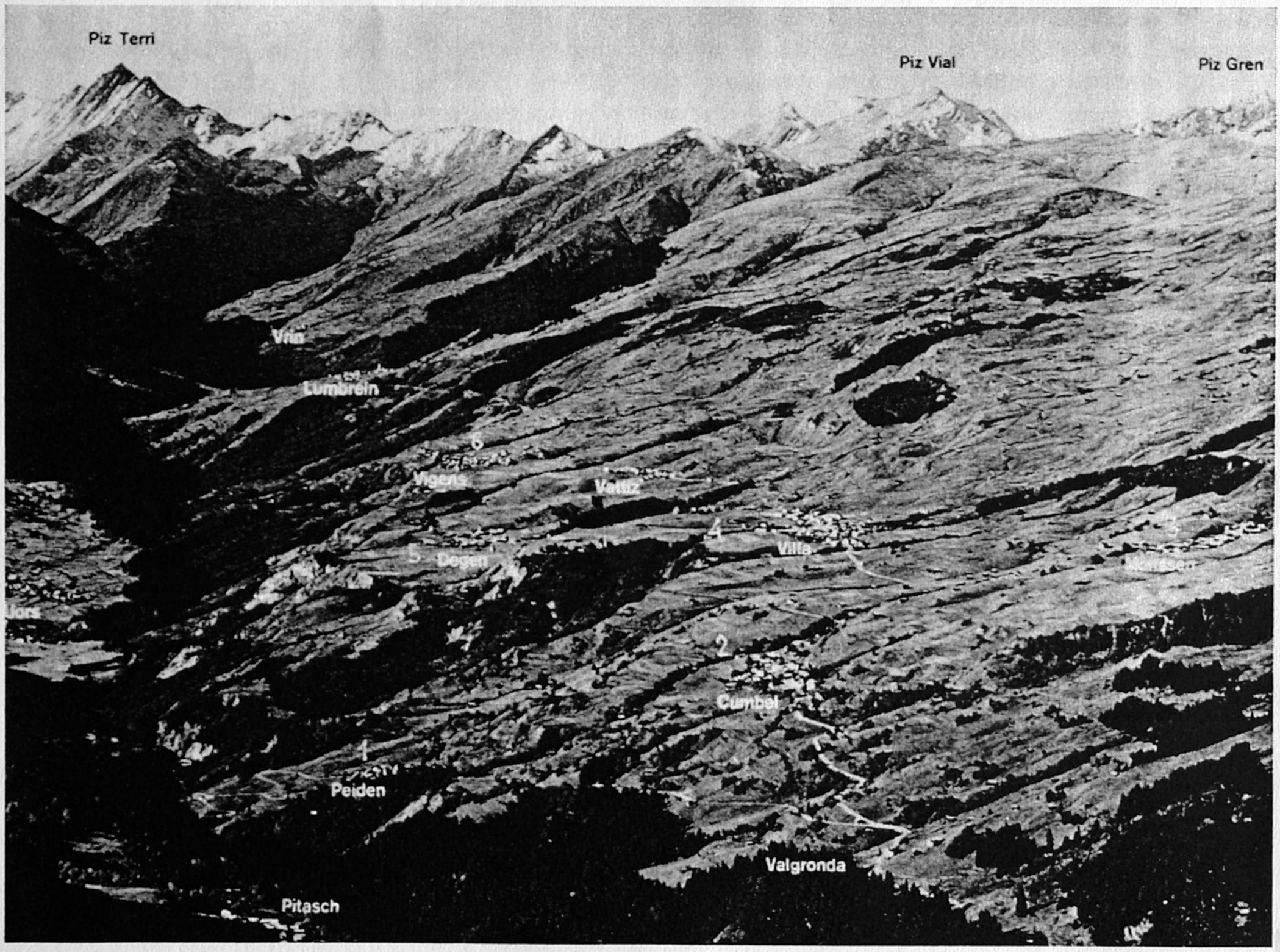


Fig. 33. Photograph of the whole of the slide area, including the villages of Peiden (1), Cumbels (2), Morissen (3), Villa (4), Degen (5) and Vigens (6).

the horizontal direction with respect to the previously recorded value amounts to 10 degrees. The evaluation of all measuring points confirms the above statement, that the displacement directions remain practically constant.

The relative change of deformation between two points on the surface has a maximum value of $^1/_{50}$ per cent/year. This value is only occasionally reached, but should it apply to a stretch for example of 10 m length, it would give a deformation of 2 cm over a period of 10 years. This amount would have caused more damage to buildings than is in fact to be found today.

The different rates of flow and directional deviations can be explained by the presence of sub-regions of flowing material, in which different pressures build up, depending on the obstacles impeding motion.

Also the role of water in the fissures may not be neglected. Several springs reach the surface in the lower parts of the slope, which cause further wetting of the whole area, and even lead to small mudflows. The water, that must be present in the slope in order for the creep deformations to take place, emerges lower down the slope.

Remedial measures for the slope is a costly affair, and the advice, where to begin, is not easily given. In that the average inclination is only about 15 degrees, and the angle of residual shear strength Φ'_r is about 23°–26°, pore water pressures must in this case play an important part Thus a permanent remedial measure could be brought about by dewatering the slope on the one hand, and protecting the river bed from erosion on the other hand. The analysis of movements shows that the depth of the sliding surface is appreciable, so that dewatering must also be carried out to great depths to be successful. On account of the extensiveness of the sliding zones, the dewatering system would also have to be installed over large areas. The protection of the river bed from erosion would complete the remedial measures. Such a remedial proposition, though well intentioned, would certainly be rejected on the grounds of the enormous costs involved.

c) *Creep Slopes at Klosters*

Whereas for various creep slopes a large number of deformation measurements for points at the slope surface are available, measurements of deep-seated deformations are exceptional. One exception is the creep slope at Klosters. This slope to the south west of the district of Klosters lies at the edge of the Bündner schist zone (see Fig. 34). The old slide area comprises about 2 sq.km with a base length of approximately 2 km, extending from 1180 m to 2200 m above sea level. The rock fall material consists of Serpentine and is rich in limestone material (Dolomite boulders). The creep movements of this slope have been discussed by Haefeli and others, regard being paid to the various factors involved and in connection with the remedial works on the famous Maillart bridge, carried out in 1930 (Haefeli, 1947 and 1964, Haefeli et al., 1953, Bill, 1949).

A loop tunnel of the Rhaetic Railway was built into this slope in 1930 in order to do away with the terminal



Fig. 34. Plan of the loop tunnel and the bridge over the river Landquart with creep rate vectors of surface points G, H, J, K. S as well as the points in the tunnel 1 to 18. Reproduced with permission of the Topographical Survey of Switzerland of 8th Oct. 1974.

situated on the right side of the valley. The slope was well-known geologically as an old slide area, and signs of movements were available, since the old railway bridge had to be continually re-positioned. The yearly horizontal movements were 5 mm. However, the consequence of these movements and their significance to new constructions were not realized. For a new construction across the river Landquart a Maillart bridge was chosen. The bridge is of reinforced concrete, a ribbed arch construction with a span of 30 m and a rise of 7.9 m, connected on both sides to framed structures, giving a total length of 75.4 m. The bridge is curved in plan and is skew to the direction of the river (see Fig. 34). The new narrow gauge railway line leaves the station and passes directly over the Landquart bridge into the loop tunnel 400 m long with a radius of 125 m. That insufficient consideration was given to potential movements may be concluded from the choice of bridge.

Already in 1938 the first remedial measures had to be carried out on the bridge. In connection with this work the Institute for Soil Mechanics and Foundation Engineering was consulted. An extensive observation net with fixed points was set up and survey measurements were carried out periodically, first by the Topographical Survey of Switzerland and later by the Geodetical Institute of the ETH, Zurich.

After 4 years of observation it was decided to strengthen the bridge by means of a heavy concrete compression girder spanning between the abutments, which is supported by hanging from the ribbed arch. The work was described by Haefeli (1947). Control measurements

on the bridge and the terrain were continued. In addition, in 1952 and 1956 measurements were carried out on the tunnel with 18 points on both sides, after that repair work was also necessary on the tunnel. The most recent measurements date from the year 1968.

The position and change of height of points on the ground surface is shown in Fig. 35 for the years 1939 to 1966 and 1968 respectively. The creep rate was constant for all points. The deformations produced in the bridge are restricted by the remedial measures taken. The creep rate vectors of the points are shown in Fig. 34, together with the vectors for the bridge for the period 1959–1966. To the same scale the tunnel deformations for a few selected points are given, an average value for the two positions adjacent to the axis being taken. Ground movements for an overburden of more than 50 m still continue at a rate of 4 mm/year and increase with decrease in the overburden. The creep movements are not, however, restricted to shallower depths. Differences occur in the directions and inclinations of the deformation vectors. Still greater, though, are the differences for the pairs of points on the side walls of the tunnel, but here the deformability of the tunnel lining must also be taken into

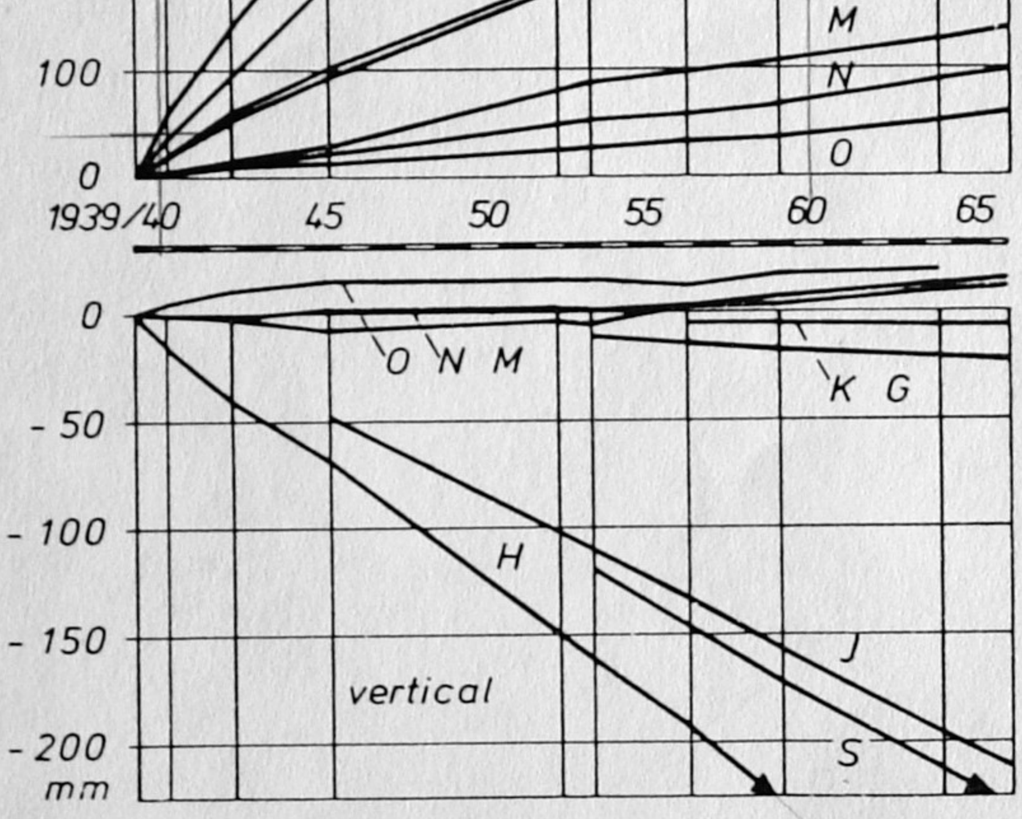


Fig. 35. Horizontal and vertical changes of position from the year 1939. Number of measurements up to and including the year 1966: 9 measurements.

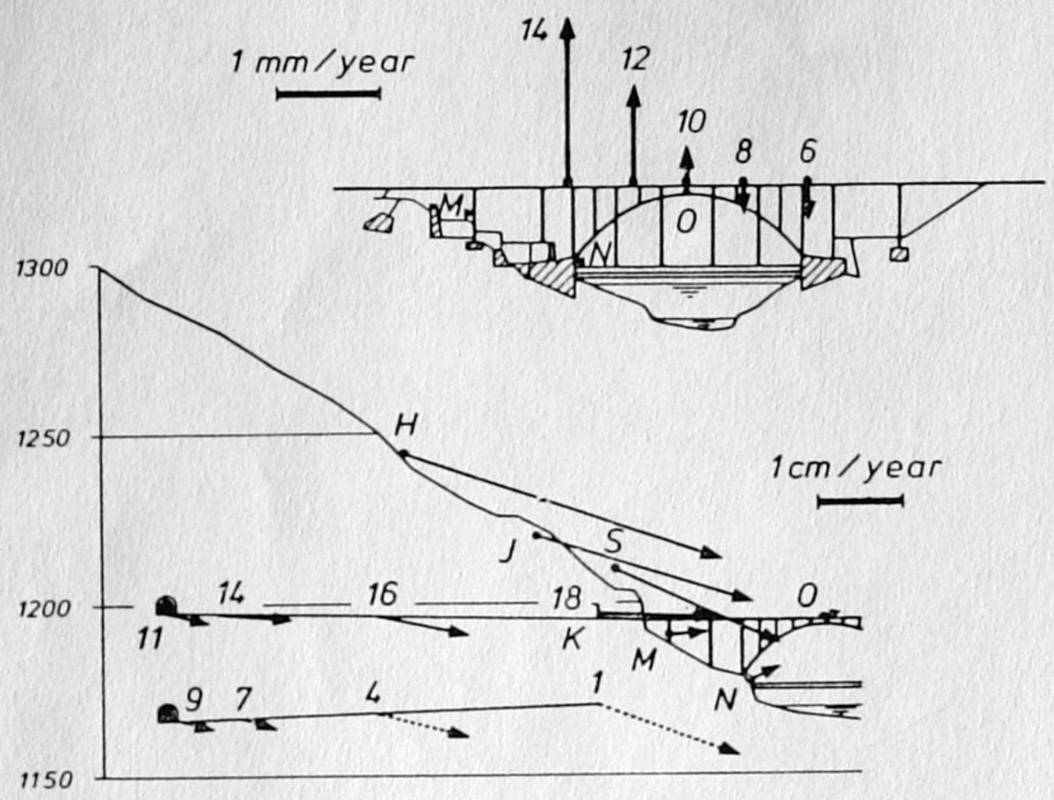


Fig. 36. Cross-section A – A, Fig. 34, through the slope. The creep rate vectors and the vertical deformations of points 6 to 14 on the bridge are given in the section.

consideration. A check on the mutual change of position of the pairs of points is lacking.

The deformations at the tunnel portals, points 1 and 18 respectively, do not agree with those of the points on the terrain surrounding the tunnel, with respect to magnitude and inclination. The inclination of the deformation vector is 6° measured to the horizontal, with a rate of $v = 1.75$ cm/year. Point 3 has practically the same rate of deformation, but the inclination β corresponds more to that of the ground surface, $\alpha = 27°$. Both the magnitude and inclination of the vector movements decrease with depth below ground surface, for instance point 9 exhibits the values $v = 0.43$ cm/year and $\beta = 17°$. At point 18, however, where the overburden is less, $v = 1.1$ cm/year, and $\beta = 2°$.

The direction and rate of deformation of the surface of the slope is indicated at points G, H, J and S; the direction is the same as that of the inclination of the slope. Below the surface of the slope the measurements at the points 1–18 give a better indication of the general direction of the slope movements. The rates of deformation of surface points are also about 1 cm/year greater than those recorded at the tunnel portals (Fig. 36). It is quite possible that this excess rate is that contributed by solifluction. This would be true above all for the point H on the steeper part of the slope. An interesting result is obtained for the inclination of the vector β at the point identified as S, i.e. $\beta = 25°$. Points J, H and G have values of 20°, 8°, and 8° respectively. For the sake of completeness it should be mentioned that the river is dammed 120 m downstream of the bridge, and that erosion of the river bed is thereby hindered.

The movements are of lesser magnitude the nearer to the river, and their inclination is somewhat flatter or even upwards directed (i.e. β becomes negative). These changes are conditioned by the influence of the opposite side of the valley. The bridge itself, with the added stiffness due to the girder, has not suffered large deformations. The compression load in the girder increased to 800 tons in 1952 and thereafter it has remained constant.

The point M situated at the middle support of the framed structure on the left bank of the river has a rate of deformation of $v = 0.74$ cm/year, with $\beta = -14°$ (i.e. directed upwards), while point N on the left abutment of the bridge has the values $v = 0.31$ cm/year and $\beta = -28°$. The directions of the vectors shown in the plan (Fig. 36) also show that the bridge is not moving with the creeping mass.

The movements in the slope were confirmed by using precision instruments. The change of inclination, whereby the right abutment is settling slightly and the left abutment rises by an amount five times larger, is $^1/_{150}$ per cent/year. There is in addition a twisting of the bridge about the left abutment amounting to $^1/_{50}$ per cent/year.

The remedial measures carried out on this concrete bridge have for the time being saved the structure from excessive damage. The movements in the slope are continuing, however, so that from a long term point of view it is unlikely that the bridge will be entirely free of troubles. It is scarcely possible to perform remedial work on the slope, since dewatering is unlikely to help. This may be supposed from observations made in the loop tunnel, which penetrates deeply into the creep slope, but as a dewatering system has no influence upon the behaviour of the slope movements.

d) *The Slope at Cebbia*

The construction of the new St. Bernhardin route necessitated the crossing of the slope at Cebbia, above Mesocco (see Fig. 1). The slope is inclined at 30 degrees on the average. The original cantonal road largely avoided the steeper parts of the slope. Maintenance work on the road had shown that even for minor cuts into the hillside appreciable movements had to be reckoned with. However, the extent of the sliding and creep movements were only a matter of conjecture.



Fig. 37. View down the mountain side towards Mesocco showing the bridge construction traversing the slope at Cebbia.
Photograph: Crameri, Cantonal Engineering Office, Chur.

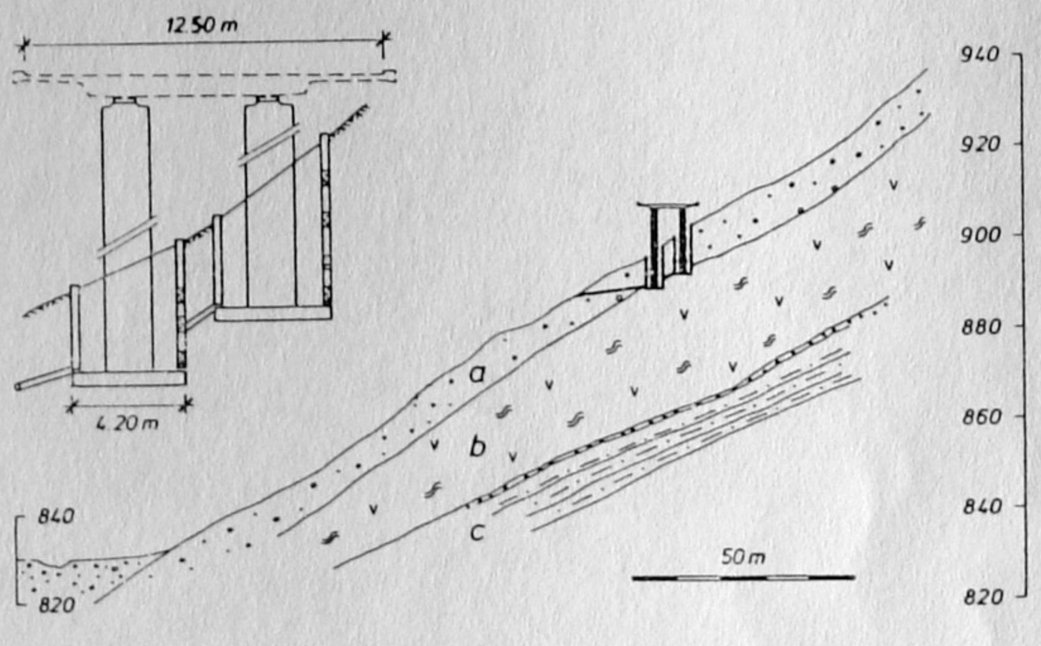


Fig. 38. Section through the slope at Cebbia above Mesocco showing the geological structure. a) Talus slope. b) Disturbed Bündner schist. c) Paragneiss. – Details of the bridge profile and of the foundations are also given.

The geological structure of the slope is given in Fig. 38. The thickness of material to firm rock amounts to approximately 25 m: Paragneiss is overlain by disintegrated material consisting of Bündner schist, which in turn is covered by scree material 4–7 metres thick. A large number of borings were made for the site investigation, but even so it was difficult to distinguish the transition from scree material to Bündner schist.

The Bündner schist may be characterized as disintegrated remoulded material, disintegration being due to the lateral expansion following reduction of side support. In the material a layered structure is still recognizable. The material is compact and varies in thickness from 17 to 20 metres. Within it, water bearing layers were encountered, which on occasion give rise to springs at the surface of the slope. The water in these layers is not under any sizable pressure, but in the highly-weathered transition zone between Bündner schist and Paragneiss, considerable water pressure was encountered and the transition zone was highly water bearing. Investigations showed that artesian water is confined to this zone only. In a drainage layer in the overlying Bündner schist this pressure is reduced and the further overlying layers are not influenced.

Already at the project stage, as the geotechnical site investigation was carried out, it was arranged to have the slope surveyed over a period, which extended to five years. During this time measurements were periodically made at a large number of points on the site. The results did not indicate large deformations taking place. However, it was deemed wise to construct a bridge running parallel to the valley side (see Fig. 37 and Fig. 38 showing the pier supports 18 m apart) instead of the original proposal with retaining walls and revetment walls. For safety reasons the support was constructed so that the bridge can be re-positioned, although differential settlements of the supports can be tolerated, before readjustment is necessary. The bridge foundations were put well into the disintegrated Bündner schist. This necessitated excavation timbering to a depth of 8 or more metres through the scree material into Bündner schist. To protect the bridge piers from possible build up of pressure due to slipping of the scree material over the Bündner schist, it was recommended that the pier and the shaft lining be separated from one another. For each support a 4.2 m diameter shaft was constructed in 2 m stages depthwise. The shaft lining was concreted immediately after excavation for each stage. The circular foundation slabs could be made bigger depending on the prevailing ground conditions. By means of this building procedure a disturbance of the slope could be largely prevented. The space between the pier and the shaft lining may be inspected and it is simply closed up by a cover. In addition, filter-stones were set in the shaft wall and the foundation area at times dewatered by means of a horizontal boring. (Stampf, 1971)

Since the construction of the bridge in 1969 its behaviour has been observed by using a system of survey measurements. Sizable displacements were measured showing a linear variation in a logarithmic plot, reaching a max. value of 2 cm after 5 years. These have slowly petered out, whereas still no movements were registered in the slope itself. (Arioli, 1971). The correctness of the precautions taken here is also shown by laboratory test results. The angle of internal friction Φ'_r was on the average found to be 28°, and as a result of the weathering of the material the interlocking effect has been destroyed. c', the cohesion intercept, was found to be 0.15 kg/cm². The angle Φ'_{cv} for constant volume was approximately 26°, and Φ'_r decreased to 23°. In that all slope movements have been prevented and the additional loading has been kept to a minimum, the latent danger of new deformations was avoided. During the excavation of the shaft no pre-existent sliding surfaces pointing to older slip movements could be detected.

The stability investigation of the slope with the estimated shear strength showed clearly its stable condition, so that it is hardly possible that zones with reduced shear strength are present, i.e. the existence of pre-existent sliding surfaces is excluded. Parts of the slope which are steeper than $\alpha = 45°$ have at most a height of 18 m, for $\alpha = 40°$ a height of 26 m, and for $\alpha = 35°$ a maximum height of 55 m. These values lead to stable slope conditions. The safety of the bridge itself has been achieved by placing the foundations at adequate depths.

If the engineer, on the basis of previous observations, has knowledge of the creep movements of a slope, it is possible for him, by choosing the appropriate constructional method and by using special precautions, to traverse such a slope by means of a bridge. On account of the relatively deep foundations for the supporting piers, which are constructed so as to be separate from and independent of the shaft wall, possible creep movements in the slope cannot cause a transfer of load to the piers. This form of construction is often used in creep slopes, but an exact prior knowledge of the size of slope movements is a prerequisite for this method. This somewhat costly constructional form does, however, have the advantage, that at any time the shaft is accessible down to foundation level, making possible its inspection at any time.

e) *Slides in the Vicinity of the Schin Highway*

During the construction of the Schin highway from Thusis to Tiefencastel the old road was rerouted in several places, so that various corrections of the old route were necessary. New retaining walls and some new bridges across gorges had to be constructed. Over a long stretch the road cuts through slope with a previous history of slides. The material of the slope is an old Bündner schist. The dimensions of the slide caused by the erosion of the Albula encompasses an area of around 700 m breadth stretching from the river bed 400 meters or so up the hillside. The depth of the sliding mass is estimated at about 50 metres, measured normal to the slope, as based upon the results of a test tunnel and the railway tunnel (see the situation Fig. 39, and the profile Fig. 40).

The maintenance of the old road, which was unstabilized, required no special work which pointed to movements in the slope. As a result no special measures, such as a check on the movements of the slope using surveying instruments, were introduced. In the area of the Caseller gorge, on which our interest centres, a new wall was added to the old retaining wall. The new wall constructed in front of the old one caused an additional loading of the ground. The gorge itself was crossed by a bridge. Measurements were carried out to check the behaviour of the new highway. These were made in 1963, and indicated small values of relative settlement throughout.

For a long time nobody gave any consideration to the question of deformations, but a few years after the end

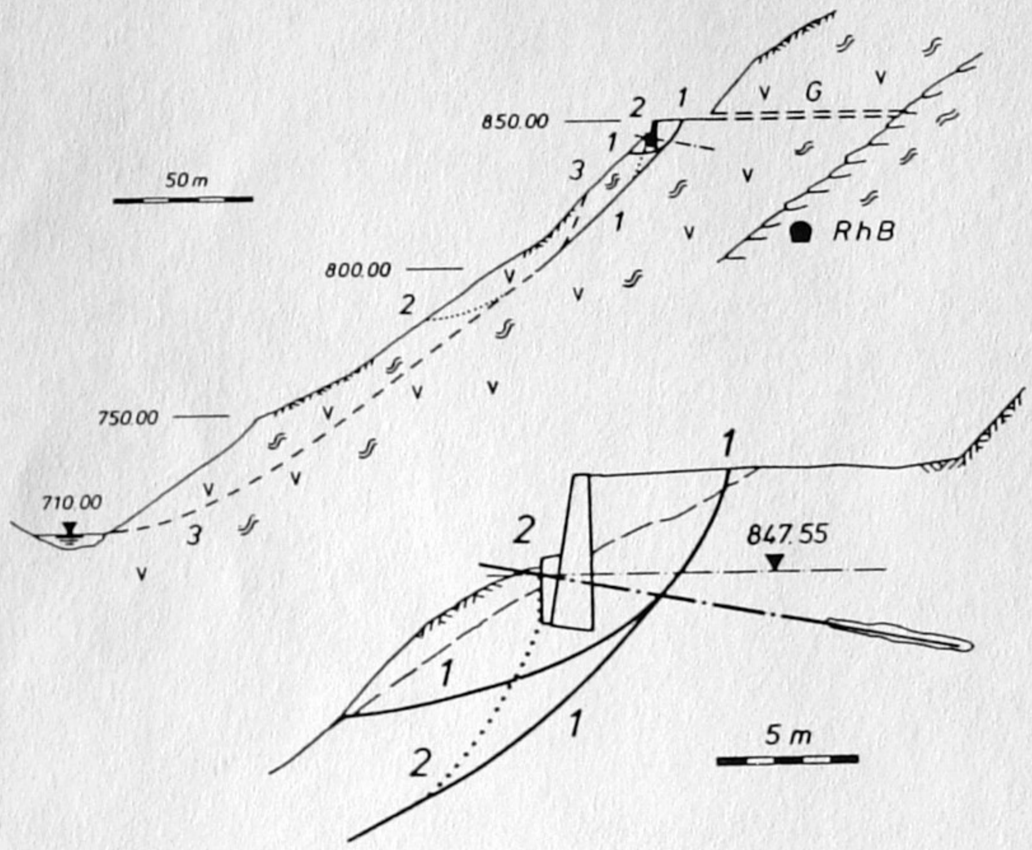


Fig. 40. Section (A – A, Fig. 39) through the slope, with sketch of the geological structure. Recognisable are the tunnel of the Rhaetic railway RhB and the test tunnel G.
Details of the cross-section of the new highway showing also the old profile (dashed line). 1. First slide before the wall was secured with the use of ground anchors. 2. Slide after construction of the anchors. 3. The 'set' after removing part of the wall and backfill and using anchors on the bridge abutments and on the pier foundations.

of construction the first cracks in the road surface began to appear. Survey measurements were then intensified and it was found that the movements at the top of the retaining wall were assuming continually increasing proportions. In the period May to the middle of June 1967 the rate of deformation reached a value of 7 mm/day with a vector inclination of 40°, which is about 5° steeper than the average inclination of the slope. The total displacement amounted to 30 cm. It was decided to remedy the initial, purely local deformation of the wall (Slide 1, Fig. 40) with alluvial anchors. Thus eleven pre-stressed anchors were constructed at intervals of 5 m at the end of June 1967. This also improved the stability of the wall with respect to sliding by about 20 %. For a while the deformations were checked, but periodic survey measurements showed that deformations were continuing at the reduced rate of 1 cm/month. In January 1968 the movements began to increase once again and the rate of deformation was soon doubled. A further rise in the rate of deformation, bringing it to 4 cm/month, was registered in February. Then in the second half of March the rate of deformation showed a further rise to 7 mm/day. At that time more cracks appeared in the road surfacing. Since the use of ground anchors another 30 cm deformation of the wall has occurred. By means of the anchors the wall was prevented from sliding, but the ground under the foundation for the wall has settled about 2 m (primarily due to slide 2). With the onset of heavy rainfall and the continual melting of snow on the slope an acceleration of movements was produced. On the highway itself values of 8 cm/day were measured, the vector of displacement being parallel to the slope. The moving ground stood out more and more and cracks in the road

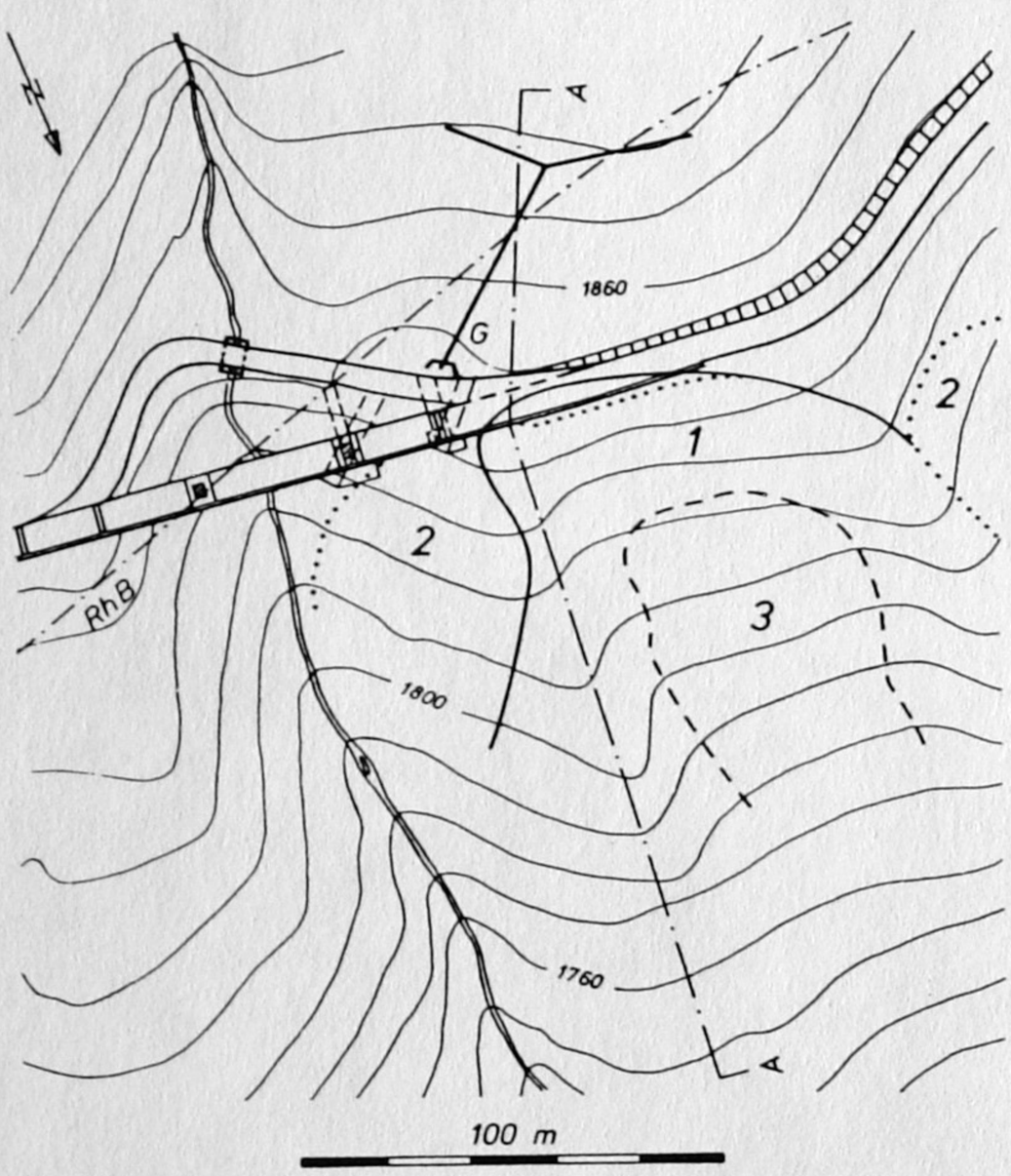


Fig. 39. Plan showing the route of the old roadway and the route with the bridge across the gorge. The dashed line is the axis of the Rhaetic railway (Rh. B.). The full line is the test tunnel. The numbers 1 to 3 indicate the phases of the slowly developing slide.

surface were opened up, as may be seen in the photograph, Fig. 41. At this point in time new steps for ensuring the safety of the wall were urgent. First of all a control was made of the tensile forces in the anchors, whereby it was established that the anchors were already stretched to the stress limit of the steel. Under these circumstances it was impossible from the constructional standpoint to unload the old anchors or insert new anchors. The danger of the anchors failing was so great, that it was decided to remove the fill material behind the wall and the wall itself above the level of the anchors. This step led to an unloading of the anchor. To avoid lengthy restrictions to the traffic using the road, a temporary wooden construction was made and after a short period, traffic was allowed once more over this part of the road. A thorough investigation was deemed necessary. A trial boring was made and the bore log showed that disintegrated Bündner schist was present to a depth of 70 metres. The schist was on the whole fairly dry, but certain water bearing layers were present. The underlying intact rock was wet, but did not evidence water seepage. The borehole, despite the small amounts of seepage in the schist, was built to function as a dewatering system. Some water did flow out from below the wall, but the quantity was relatively small. The stress relief due to removing part of the wall and the fill material produced an immediate cessation of deformation in the roadway, but other parts of the slope were still affected by creep movements. Due to progressive failure, the movements were propagated up the slope from the level of the roadway and downwards to the Albula. The possibility is not excluded that the large deformations in the slope blocked up the natural dewatering processes and pore water pressures were developed, which in turn contributed to the instability of the slope.

The average inclination of the slope amounts to 35°, which equals the friction angle of the slope material. The investigation of samples for shear strength showed that there was a cohesion in addition to a friction angle, as was obtained with the material from Reischen. It must be assumed, however, that the long term movements have caused a realignment of fragments parallel to the direction shear. Whether or not the slope has experienced earlier deformations cannot be said with certainty. The observed movements, however, seem to indicate that the instability of the wall and the adjoining slopes was caused by the extra loading due to the retaining wall construction. The zone of deformation in the slope extends to include the area of the bridge abutments and the pier foundations had to be strengthened by the construction of a number of ground anchors to save the bridge. In this way, the movements of the bridge were brought under control.

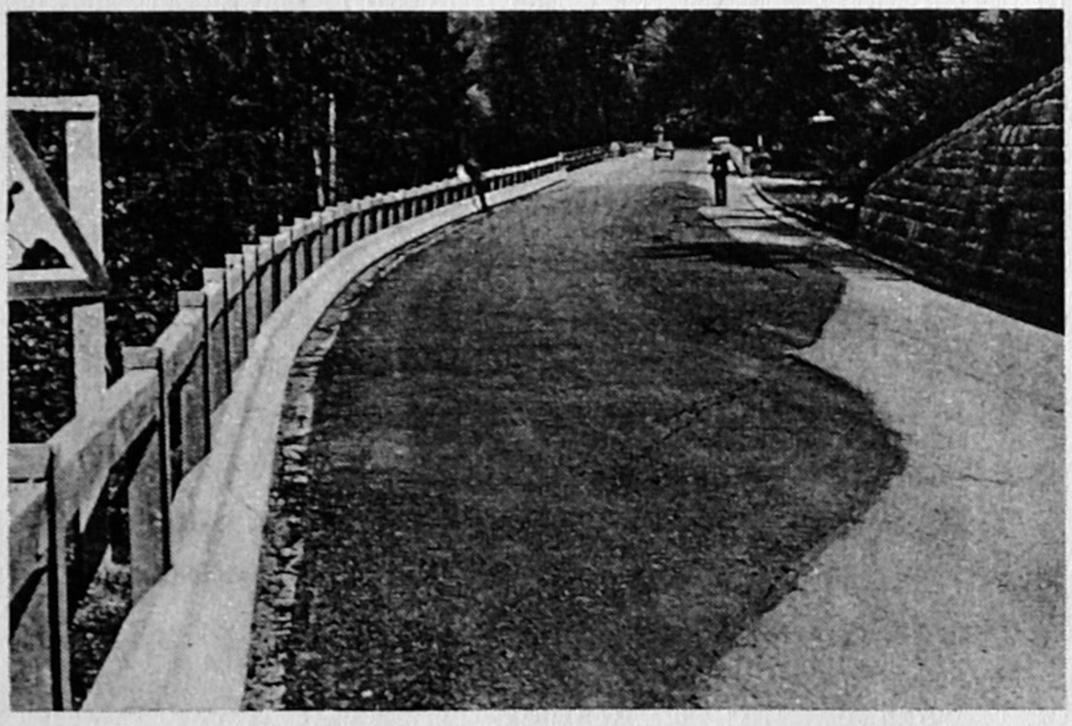


Fig. 41. Corrections to the highway after the first improvement work.

In that the trial boring (used as a dewatering system) and piezometers did not produce any sizable quantities of water, a general dewatering scheme would hardly have been successful. It was decided, however, to protect the river bed against further erosion of the toe of the slope. At the same time barriers were built into the slope and founded on firm rock, to provide an additional resistance to slope movements. The work extended over a period of approximately 6 years. It is hoped that the remedial works carried out will be a permanent success, so that the provisional measure adopted, i.e. the use of the wooden construction, may be replaced by a permanent bridge structure. Today the movements amount to about 1 cm/year.

Despite the known, unsafe ground conditions the reconstruction of the old road was preferred to other alternatives, such as a tunnel through rock bypassing this area, which was the solution adopted for the railway line built at the turn of the century. The considerations that led to the present building programme were, above all, the good condition of the old road subgrade and the adaptability of the route to the terrain. One was conscious of the fact, however, that by choosing the cheaper solution some surprises had to be expected, which did in fact happen. To reduce the risks, disturbances to terrain were kept to a minimum during construction, for example in the redistribution of loads on or the addition of loads to the slope. In spite of these measures, 10 per cent of the 700 m long stretch of new road fell victim to the risks discussed above.

The occurrence of creep movements is a consequence of the defective knowledge of the shear properties of Bündner schist prevalent at that time. The new retaining wall was dimensioned on the assumption of a friction angle of the material of 35°, corresponding to the inclination of the slope. For the later remedial works with ground anchors the same assumptions were made, i.e. the friction angle (of 35°) could be fully mobilized by the anchor forces. It was not realized that the shear strength of Bündner schist at large deformations is greatly reduced, endangering local stability and leading to progressive failure in the slope.

The remedial measure of using ground anchors was only of limited success. Renewed movements overstressed the anchors necessitating an unloading of the wall structure by the removal of material behind the wall. To bring the deformations under control extensive remedial work

was necessitated. Once the movements have completely settled down, a bridge will be constructed here to replace the present provisional structure.

In *retrospect,* it may be concluded from the *five case records,* that the reduction of shear strength of a Bündner schist results from disintegration and weathering of the rock and a rearrangement of the structure at large deformations and occurs over a long period of time. Instabilities or creep movements can be explained on the basis of progressive failure. What triggers off the instabilities is as a rule to be sought in the erosion at the foot of the slope, as Bjerrum (1967) assumed for clay landslips.

Events which take place only slowly are hardly perceptible, as for example the lateral expansion of the material brought on by the erosional activity at the base of the slope. Artificial disturbances of the slope can, on the other hand, cause a sudden release of forces and produce large deformations even in rocks (Feld, 1965).

Bündner schist, on account of tectonic activities in the past, has been highly overstressed and potential sliding surfaces have aready been formed. Thus the material as it occurs naturally meets the requirements for the reduction of shear strength outlined above. Such preexistent sliding surfaces may be observed in hand specimens also, but their dating is not possible. (Morgenstern and Tchalenko, 1967). The prerequisites for creep behaviour in a slope is mainly determined by the geology, so that a joint investigation by geologists and engineers is necessary. (Terzaghi, 1950).

Theoretically the development of creep zones requires substantial straining, which may be well recognized by zones in which compressed material has piled up in potential sliding areas such as, for example, heaps of material on the periphery of the sliding mass and humps or a rippled surface appearing at or near the foot of the mass. However, hidden preexistent sliding surfaces may well lead to unsuspected creep behaviour.

Creep slope deformations are not so spectacular as 'mud rock flows' (German: 'Blockströme'). This is illustrated by the mud rock flow at Val Sassa, south of Zernez in the Swiss National Park. The tongue of the mud rock flow lies at a height of 2060 m above sea level, which is for the engineer somewhat distant from his building site and its ground conditions. The length of the flow is about 1 kilometre with a width of just 80 m and a thickness of about 10 m. With a constant slope of 12.5° the yearly movement amounts to 50 cm. The surface material may be described as broken, angular gravel-sand fragments with blocks (boulders), but with depth the silt fraction becomes more and more important. Such is the material description according to Girsperger (1973) and Damaradzki (1951).

Instead of the German word 'Blockströme' – literally 'block flows' – it would be geotechnically more appropriate to describe the phenomena as 'Siltströme' ('silt flows'). In this way the flow movements could be better explained, for with an inclination of terrain of 12.5° in the saturated state a value of residual strength $\varphi'_r = 24°$ would be fitting for this silt. The nature of the movements is clearly shown in the air photograph Fig. 42: the blocks at the surface of the flowing mass which are carried along by it have indeed contributed to the description of this phenomena named 'Blockströme'. The English description: 'mud rock flows' fits the phenomena better in that the presence of a fine fraction is indicated.

If signs of movement in a Bündner schist slope are not visibly detectable, but geologically quite probable, there exists the possibility of obtaining a reliable picture of the movements after a relatively short period of survey observation, because of the substantial yearly movements usually encountered. The deformations are about ten times as great as the corresponding creep deformations in a clay slope.

5. Closing Remarks

Knowledge of the properties of Bündner schist, both in its natural and artificially compacted condition, is of interest to the engineer. In extensive areas the rock must be used as a building material. Scree material consisting of broken fragments of Bündner schist has, if artificially compacted, a high shear strength. For the life of a structure there is hardly any likehood of a substantial reduction of shear strength due to weathering action. In estimating the stability of slopes and the prevailing state of stress it is important to take into consideration the cohesive component due to the interlocking effect in the material, which is dependent on the state of compaction achieved.

Artificial compaction can give certain difficulties depending on the crushing strength of the grains. With greater fraction of fine components the material becomes impermeable and during compaction the water content must be kept within close limits to the optimum water content. Tests show that with a water content of only 2 % above optimum the successful compaction of the material is questionable and difficulties may be encountered in driving vehicles over the material.

In the case of the stability of natural slopes, on the other hand, attention must be paid to the degree of fragmentation of the components if the interlocking action is to be correctly estimated. Indications of creep behaviour show that in the zone affected by creep movements calculations should be carried out on the basis of the residual shear strength.

Likewise the influence of pre-existent slip zones, in which the plate-like particles show a preferred orientation (with a consequent loss of the interlocking effect), should not be underestimated in natural slopes.

The degree of weathering of the material must also be considered, since this has a great influence on the particle size distribution. The susceptibility to weathering action is greater if the schist is of more recent origin. Further, the greater the fine fraction the smaller the shear strength.



Fig. 42. Aerial photograph of the mud rock flow at Val Sassa, south of Zernez.
Reproduced with permission of the Federal Directorate of Catastral Survey.

Particular attention must be paid to these observations if the stability of a slope is to be investigated. In addition to the topographical relations, the geological and geotechnical relationships must also be known, whereby the direction of stratification, the petrography and the degree of weathering are decisive factors. Should the slope already be in movement it is advisable to show special care, since due to progressive movements the particles are reoriented and the creep process may accelerate.

In slopes on the point of movement where no motion as yet occurs (i.e. there is a state of neutral equilibrium), changes of stress must be caused with great care. The consequences of inadvertent interference in the stability of a slope could be catastrophical years or even decades later. For instance, a retaining wall improperly backfilled could be severely damaged many years afterwards, due to the slowly increasing creep pressures.

All the results given here are expressed in terms of effective stress parameters. In computations carried out with the help of these results, pore water pressures (measured by means of piezometers) must be considered.

The shear strength of Bündner schist is of a complex nature. The difficulty of appraising the strength properties is indeed well known to the engineer. Schists that seem to be alike may exhibit different behaviour, the one giving firm, stable properties, while slopes consisting of the other material are made unstable by the slightest disturbing force.

An exact appraisal of the shear strength properties requires, in addition, the careful observation of the stability behaviour of natural slopes. The experienced eye may detect creep movements in slopes, since for a reduction of strength substantial deformations are necessary according to the degree of weathering of the material. These deformations may be of the order of metres. Thus a check by means of survey measurements can provide useful information. Such field observations carried out hand in hand with geotechnical laboratory investigations provide the engineer with the necessary data for computations.

The natural factors present in the given case histories play a varied role. Thus in the case of Schuders it is definitely the erosion of the foot of the slope by the river Schraubbach which leads to the instabilities. For this reason, the first remedial work was concerned with the protection of the river bed and banks from erosion. In Peiden, on the other hand, the chief factor controlling creep movements is the geological structure of the valley sides. On account of bedding planes parallel to the slope and natural events, certain planes of weakness in the material of the slope are already present. But here again the toe erosion of the slope must not be overlooked. The Cebbia slope was known to be a slope on the point of moving (i.e. safety factor equal to unity). Experiences in the maintenance of the cantonal highway had already shown that slight disturbances to the slope could have serious consequences. A check on creep movements over a period of 5 years brought to light no deformations worth mentioning, so it was thought to be safe to commence the construction of the bridge. A bridge construction was preferred to a retaining wall because it involved less disturbance to the slope. That this solution was the right one has been shown by the absence of subsequent deformations in the slope. In the vicinity of the Schin highway, however, a record of movements was nonexistent, so that a large retaining wall was placed up against the old wall in the road improvements scheme. Some years went by before the first signs of instability were noticed. These, however, increased quite rapidly.

The first measures to remedy the movements by using ground anchors were of temporary success. The check on the tendon loads carried out some time afterwards, however, revealed that the anchors were overstressed so that the only remaining solution was to remove some of the backfill material and a part of the wall itself. The deformations in the slope later affected the bridge structure as well. They extend downwards to the river Albula, where it was decided to protect the slope from further erosion. Today the measurements show that the safety measures carried out at the foot of the slope have given a certain degree of success.

Normally it is not possible to make any definite statement about the depth of a slide, i.e. about which zones are affected. Mostly sliding is accompanied by creep movements, and usually there is a superposition of solifluction deformations. It has been shown again and again that tension and compression zones are present, with neutral zones in between, and whole villages can undergo movements without suffering any damage. The loop tunnel at Klosters shows clearly that such phenomena are largely attributable to the geological relationships. Here the deformations decrease substantially with depth, and are mainly due to the weathering characteristics of the rock slope.

The case histories presented in this paper are only meant to illustrate typical ways in which creep deformations are developed. The number of areas which are undergoing creep movements is greater than one would imagine. Movements of the order of centimetres to decimetres/year are often encountered in these areas. A ready made solution to prevent further movements in areas subject to creep slides cannot unfortunately be proffered. Each slide area and each creep slope requires a special investigation.

Acknowledgements

It is my desire to thank each of those persons who in one way or another contributed to the success of this study. I should like to give especial mention to the following: the cantonal office for civil engineering works, Grisons, under the supervision of chief engineer, H. Fuhr, Esq., who placed important plans, photographs, and records of measurements at my disposal. In the course of the years numerous investigations have been carried out for the above office at the Institute of Soil Mechanics and Foundation Engineering (IGB) of the ETH, Zurich (formerly designated VAWE). The results of these investigations were drawn upon for this study. Also, without the numerous measurements made by members of the Institute for "Geodäsie und Photogrammetrie" under the leadership of Professors F. Kobold and R. Conzett, the creep movements could scarcely have been followed so extensively. The measurements were carried out by students as part of their course work. To my co-workers at the IGB, under the direction of Professor H. J. Lang, I extend my thanks for their support and help in both the carrying out of tests and the evaluation of results, namely to Dr. F. Bucher and Mr. T. Ramholt. Special thanks are due also to Dr. E. G. Prater who translated the manuscript into English.

References

Arioli, R. (1971): *Neue Vermessungsaufgaben im Dienste der Sicherheit im Strassenbau*. Strasse und Verkehr, Vol. 57, Nr. 6, p. 235–236.

Bill, M. (1949): *Robert Maillart*. Verlag für Architektur AG, Erlenbach-Zürich.

Bolli, H. und Nabholz, W. K. (1959): *Bündnerschiefer, ähnliche fossilarme Serien und ihr Gehalt an Mikrofossilien*. Eclogae geologicae Helveticae, Vol. 52, p. 237–270.

Bjerrum, L. and Landva, A. (1966): *Direct simple-shear tests on a Norwegian quick clay*. Norwegian Geotechnical Institute. Publication Nr. 70, 20 p.

Bjerrum, L. (1967): *Progressive failure in slopes of overconsolidated plastic clay and clay shales.* (Third Terzaghi Lecture). Proc. ASCE, Vol. 93, No. SM 5, p. 3–49.

Bjerrum, L. (1968): *Kriechen von Böschungen in vorbelasteten Tonen.* Schweizerische Bauzeitung, Vol. 86, No. 31, p. 545–547.

Bjerrum, L. (1973): *Problems of soil mechanics and construction on soft clays and structurally unstable soils.* General Report, Session 4. Proc. 8, Int. Conf. SMFE, Moscow, Vol. 3, p. 111–159.

Bucher, F. (1975): *Restscherfestigkeit natürlicher Böden, ihre Einflussgrössen und Beziehungen als Resultat experimenteller Untersuchungen.* Diss. ETH, Zürich.

Casagrande, A. and Hirschfeld, R. C. (1960): *Investigation of stress-deformation and strength-characteristics of compacted clays.* Harvard soil mechanics series, No. 61, 45 p. Cambridge, Mass.

Domaradzki, J. (1951): *Blockströme im Kanton Graubünden.* Diss. Universität, Zürich.

Feld, J. (1965): *The factor of safety in soil and rock mechanics.* Proc. 6, Int. Conf. SMFE, Montreal, Vol. 3, p. 185–197.

Furrer, G. (1954): *Solifluktionsformen im Schweizerischen Nationalpark.* Kom. Schweiz. Naturforschende Gesellschaft zur wiss. Erforschung des Nationalparkes Bd. IV.

Furrer, G., Bachmann, F. und Fitze, P. (1971): *Erdströme als Formelemente von Solifluktionsdecken im Raume von Mont Chavagl/Mont Buffalora.* Komm. Schweiz. Naturforschenden Gesellschaft.

Girsperger, S. (1973): Private communication.

Haefeli, R. (1940: *Zur Mechanik aussergewöhnlicher Gletscherschwankungen.* Schweiz. Bauzeitung, Vol. 115, No. 16.

Haefeli, R. (1942): *Spannungs- und Plastizitätserscheinungen der Schneedecke.* Schweizer Archiv für angewandte Wissenschaft und Technik. Vol. 8, No. 9–12.

Haefeli, R. (1944): *Erdbaumechanische Probleme im Lichte der Schneeforschung.* Schweiz. Bauzeitung, Vol. 123, No. 2, 4 und 5.

Haefeli, R. (1947): (Mohr, C., Haefeli, R., Meissen, L., Waltz, Fr., Schaad, W.) *Umbau an Landquartbrücke der Rhätischen Bahn in Klosters.* Schweiz. Bauzeitung, Vol. 65, No. 1–3.

Haefeli, R. (1951): *Investigation and measurements of the shear strengths of saturated cohesive soils.* Géotechnique, Vol. 2, Nr. 3, p. 186–208.

Haefeli, R., Schaerer, Ch. and Amberg, G. (1953): *The behaviour under the influence of soil creep pressure on the concrete bridge built at Klosters by the Rhaetien Railways Company, Switzerland.* Proc. of the Third Int. Conf. on Soil Mechanics and Foundation Engineering, Zürich, Vol. 2, p. 175–179.

Haefeli, R. (1964): *Creep and progressive failure in snow, soil, rock and ice.* Proc. 6, Int. Conf. SMFE, Montreal, Vol. 3, p. 134–148.

Haefeli, R. (1967): *Ein neuer Klinometer zur Erfassung von Bauwerks- und Geländebewegungen.* Strasse und Verkehr, Vol. 53, No. 11, p. 554–555.

Hegland, R. (1973): Private communication.

Huder, J. (1963): *Bestimmung der Scherfestigkeit strukturempfindlicher Böden.* Eidgenössische Archmische Hochschule, Zürich. Versuchsanstalt für Wasserbau und Erdbau. Mitteilung, No. 58, 35 p.

Huder, J. and Fetz, L. B. (1967): *Shear strength and density of talus material used in the Goescheneralp Dam.* Proc. Europ. Conf. Oslo, 1967, Vol. 1, p. 205–207.

Huder, J. (1971): *Scherfestigkeit eines gebrochenen Bündnerschiefers.* Strasse und Verkehr, Vol. 57, No. 6/71, p. 215–220.

Jäckli, H. (1957): *Gegenwartsgeologie des bündnerischen Rheingebietes.* Geotechnische Serie, Lieferung 36, Schweiz. Geotechn. Komm. 136 p.

Kobold, F. (1960): *Methoden und Ergebnisse der in den Jahren 1956 bis 1959 im Rutschgebiet von Schuders durchgeführten Verschiebungsmessungen.* Schweiz. Zeitschrift für Vermessung, Kulturtechnik und Photogrammetrie, No. 10, 21 p.

von Moos, A. und Haefeli, R. (1962): *Schweizerische Probleme auf dem Grenzgebiet von Bodenmechanik, Geologie und Glaziologie.* Int. Baugrundkursus, 1961, Essen. Technische Hochschule Aachen. Institut für Verkehrswasserbau, Grundbau und Bodenmechanik. Mitteilung, No. 25, p. 237–264.

Morgenstern, N. R. and Tchalenko, J. S. (1967): *Microstructural observations on shear zones from slips in natural clays.* Proc. Europ. Conf. Oslo, 1967, Vol. 1, p. 147–152.

Nänny, P. (1948): *Zur Geologie der Prättigauschiefer zwischen Rhätikon und Plessur.* Diss. Universität, Zürich.

Rutsch, R. F. (1960): *Lexique Stratigraphique International.* Vol. I, Europ. Fascicule 7, Suisse.

Schmertmann, J. H. and Osterberg, J. O. (1961): *An experimental study of the development of cohesion and friction with axial strain in saturated cohesive soils.* ASCE, Research Conf. on Shear Strength of Cohesive Soils Boulder, Colo., p. 643–694.

Skempton, A. W. and Bishop, A. W. (1954): In *Soils.* Building Materials, their elasticity and inelasticity. M. Reiner, North-Holland Publishing Company, Amsterdam. Chapter X, p. 417–482.

Skempton, A. W. (1964): *Long-term stability of clay slopes.* 4. Rankine Lecture, Geotechnique, Vol. 14, No. 2, p. 77–102.

Skempton, A. W. and Petley, D. J. (1968): *The strength along structural discontinuities in stiff clays.* Proc. Europ. Conf. Oslo, 1967, Vol. 2, p. 29–46.

Stampf, W. (1971): *Brückenbau in Graubünden.* Strasse und Verkehr, Vol. 57, No. 6, p. 223–227.

Terzaghi, K. (1950): *Mechanism of landslides.* Engineering Geology. Geol. Soc. of America. Application of geology to engineering practice (Berkey Volume), p. 83–123.

Thut, A. (1969): *Experimentelle Untersuchung der Scherfestigkeit trockenfester Böden und Gesteine am Beispiel von künstlich hergestellten und zusammenhängenden $CaCO_3$-Proben.* Diss. Eidg. Technische Hochschule, Zürich. Versuchsanstalt für Wasserbau und Erdbau. Mitteilung No. 81, 68 p.

Trümpy, R. (1960): *Paleotectonic evolution of the Central and Western Alps.* Bull. Geol. Soc. of America, Vol. 71, p. 843–908.

Weber, E. (1971): *Tunnelgeologie in Graubünden.* Strasse und Verkehr, Vol. 57, No. 6, p. 212–214.

Winterhalter, R. U. (1948): *Geologischer Bericht.* Expertenkomm. zur Sicherung der Ortschaft Peiden. Unpublished.

Zulauf, R. (1971): *Ueber die Bündnerschiefer im Bündnerland und Umgebung.* Strasse und Verkehr, Vol. 57, No. 6, p. 207–212.