Irregular Stress — Strain and Progressive Failure

Contrainte Déformation Irregulière avec Rupture Progressive

E. SOTIROPOULOS Director, Geomechaniki S.A., Athens, Greece
S. CAVOUNIDIS Head Design Dept., Geomechaniki S.A., Athens

SYNOPSIS The stress-strain curves of a sizeable number of samples of an overconsolidated blue marl exhibited irregular behaviour. An important type of irregularity was that the shear stress passed to the softening side of the curve without ever reaching the expected peak shear strength. The possible implications of such behaviour on the mechanism of progressive failure of slopes are examined.

INTRODUCTION

Progressive failure was shown to be one of the principal modes of overconsolidated clay and clay shale slope failures, in the well known work of Bjerrum (1967). An inherent assumption of this mode of failure is that the stress-strain behaviour of the soil is a regular strain softening type of behaviour. Recent investigations of the behaviour of an overconsolidated marl, found in Epirus, Greece, showed that this assumption may not hold for all geological materials.

This paper presents a certain type of irregular behaviour of the marl and attempts an analysis of the implications such a behaviour might have on slope stability, in the context of Bjerrum's progressive failure thesis. Another type of irregular behaviour as well as its possible implications for slope stability was presented elsewhere (Cavounidis and Sotiropoulos, 1980).

ON GEOLOGY

The general area where the marl exhibiting the irregular behaviour was found is in the northwestern part of Greece, about 50 km north of the town of Preveza and about 1 km inland from the Ionian Sea.

The Ionian sea is a geosyncline zone with deep sea sediments. During the Pliocene the area was flooded, possibly because it sank. In the lake created, products of river deposition were gathered, found today in the form of sand-gravels and conglomerates. In the periods of increased lake depth marly clays were deposited. The particular marl which exhibits the irregular behaviour was deposited during the Neogen.

The Ionian zone was folded in the late stages of the alpine orogenesis. Faults run north-south or northwest-southeast dipping towards the west. Tectonic movements are not uncommon in the region, resulting in occasional earthquakes. After the lake was filled with the sediments approximately to their present altitude. Recent faulting has also taken place.

The hilly part of the area contains a number of rather recent, small natural landslides, having an approximately east-west direction of movement that illustrate the general instability of the region. It is on these hills that excavation for a highway prompted a series of slips. The geotechnical investigation of the causes for these slips led to the identification of the irregular behaviour of the marl.

TEST RESULTS

The laboratory tests conducted during the soil investigation at several sites where failures had either occurred or were possible, focussed mainly on the behaviour of an overconsolidated, fissured, blue marl which was identified as the stratum in which most slides originated.

The blue marl appeared in a variety of conditions corresponding to different sites where investigations were carried out. It can be roughly classified as (A) marly clay (Cavounidis and Sotiropoulos, 1979a), (B) marly clay shale (Cavounidis 1979), and (C) marly rock (Cavounidis and Sotiropoulos, 1979b), in ascending order of strength and brittleness.

The first series of tests was conducted on marls (A) and (C). These were routine quick direct shear tests at the natural water content of the marls, adequate for purposes of speedily obtaining approximate results on the strength of the marl in order to prescribe remedial measures, but rather inadequate for the purposes of researching its actual stress-strain behaviour. When the irregularities in the behaviour of the marl were observed in a number of the direct shear tests, the need for better testing was combined with the onset of the investigation of another slide area, corresponding to marl (B). Unlike the samples of marls (A) and (C) which were obtained from boreholes, the samples of marl (B) were large intact samples extracted by hand after an initial excavation of test pits by power shovel. From these, smaller (1.5 in. diam.) samples were prepared and consolidated drained triaxial tests were carried out. For reasons of comparison with the results of tests on samples of marls (A) and (C), quick direct shear tests at the natural water content were also carried out on samples of marl (B). The irregular behaviour subsequently described was observed in a significant percentage of both types of shear tests (6% of tests on marl (A), 10% on marl (B), 20% on marl (C)). Thus it is reasonable to conclude that the irregular behaviour has nothing to do with the method and quality of testing but can only be attributed to the material properties.

A summary of the index properties of the three types of marl is presented in Table 1. It can be observed that marl (B) was less plastic than either marls (A) and (C) and had a smaller cohesion intercept. However, the angles of shearing resistance are in ascending order with marl (A) having the lower value, marl (B) the intermediate and marl (C) the higher.

X-ray diffraction analyses performed in samples of marls (B) and (C) showed that the clay minerals were illite and chlorite. The carbonate content of marl (B) was greater (20% - 25%) than that of marl (C) (15%).

In the majority of the shear tests performed (either direct shear or triaxial) the stress-strain curves for all three types of marl were of regular strain softening type. However, in a number of the tests the stress-strain curve passed to the softening side without ever reaching the anticipated peak. Fig. 1,2 and 3 show typical test results illustrating this behaviour. In particular Fig. 1a presents the results of quick direct shear tests on marl (A). The "truncated peak" behaviour can be seen for the curve corresponding to 200 kn/m^2 normal pressure. The dotted line is an assumption of how the "regular" curve might have looked. Fig. 1b shows how the observed "truncated peak" strength lies below the peak strength envelope. Fig. 2a presents a series of deviatoric stress-axial strain curves obtained during consolidated drained triaxial tests. The curve corresponding to 300 kn/m^2 cell pressure exhibits the truncated peak behaviour. Fig. 2b shows that the Mohr circle based on the truncated peak strength lies below the peak strength envelope. The dotted line is an assumption of how the regular curve might have looked. Fig. 3 presents quick direct shear test results on marl (C) exhibiting the truncated peak behaviour.

DISCUSSION

On the basis of the test results, some of which were presented in the previous section, it can be suggested that it is possible that portions of clayey soil mass, when sheared, may pass to the softening side of the stress-strain curve without ever reaching the peak strength of the material exhibited by different portions of the same mass.

At this point there is no conclusive evidence as to the causes of this behaviour. It seems possible that natural cementation may be a factor (Mitchell 1970, Townsend et al 1969). Somewhat similar behaviour of Keuper marl was referred to as "associated with the structural breakdown which initiates lateral expansion"(Chandler, 1967, p.105). The most probable explanation though, should be centered around the existence of (non-apparent) fissures in the samples. Such fissures were observed on samples of marl (B) which were examined in the scanning electron microscope (Lupini et al, 1980). The orientation of a fissure with respect to the failure plane may greatly influence the strength of the sample (Brown et al, 1977). An internal fissure, or set of fissures, oriented appropriately may produce effects such as the truncated peak.

Drawing on Jaeger and Cook's (1976) analysis of the effect of anisotropy on strength, one can assume that the criterion for failure along a fissure is :

$$\frac{\sigma_1 - \sigma_3}{2} = \frac{c_f + \sigma_3 \cdot \tan\varphi_f}{(1 - \tan\varphi_f \cdot \cot\beta)\ \sin 2\beta} \qquad (1)$$

where σ_1, σ_3 are the principal stresses, c_f= cohesion on the fissure surface, φ_f = angle of shearing resistance on the fissure surface and β is the angle between σ_1 and the normal to the fissure plane. The Mohr-Coulomb criterion for failure through the intact material and for constant σ_3 can be written as :

$$\frac{\sigma_1 - \sigma_3}{2} = \frac{c_{in} + \sigma_3 \cdot \tan\varphi_{in}}{1 - \sin\varphi_{in}}\ \cos\varphi_{in} \qquad (2)$$

where c_{in} = cohesion of the intact material and φ_{in} = angle of shearing resistance for the intact material. Similarly the residual envelope is described by :

$$\frac{\sigma_1 - \sigma_3}{2} = \frac{c_r + \sigma_3 \cdot \tan\varphi_r}{1 - \sin\varphi_r}\ \cos\varphi_r \qquad (3)$$

where c_r = residual cohesion and φ_r = angle of shearing resistance at residual level. It may be assumed that :

$$\begin{aligned} c_{in} &> c_f > c_r \\ \varphi_{in} &> \varphi_f > \varphi_r \end{aligned} \qquad (4)$$

It can be shown geometrically that,for a certain range of angles,peak strength of the intact material will never be mobilized because the failure envelope for the fissure will first be reached. The combined effect of a microfissure, having some inclination with respect to the failure plane and restricted to a part of the sample, together with the strength of the intact part is more complex but must be based on the same principles.

The above may possibly constitute a basis for explaining the observed behaviour. However, a more definite answer may only be obtained after further research and further evidence of similar behaviour in other materials. What is considered important at this point is to explore possible consequences of such behaviour on the stability of slopes. Bjerrum's (1967) model for the progressive failure of overconsolidated clay and clay shale slopes will be followed.

POSSIBLE IMPLICATIONS OF IRREGULAR BEHAVIOUR ON PROGRESSIVE FAILURE

Following the very descriptive illustration of Bjerrum (1967), a slope whose end portion has a truncated peak strength is examined (Fig. 4).

A vertical cut is assumed to be made at the left end of the slope (Fig. 4a). A redistribution of lateral stresses takes place. The total shear stress at point 0 is :

$$\tau_0 = \gamma \ . \ z \ . \ \sin a \ . \ \cos a + k \frac{E}{OA} \qquad (5)$$

where z is the depth of the cut, a the slope angle, k a concentration factor expressing the ratio between maximum and average stress on the plane OA and E is the lateral force. The first term on the right side of eq. (5) is the stress due to the gravity forces. If S_P is the peak strength of the marl and S_{TP} is the truncated peak value call :

$$\lambda_1 = \frac{S_{TP}}{S_P} \quad (\lambda_1 < 1) \qquad (6)$$

the ratio of the truncated peak strength to the maximum peak strength. Then if

$$\tau_0 < \lambda_1 S_P < S_P \qquad (7)$$

the slope is stable. If

$$\tau_0 > S_P > \lambda_1 S_P \qquad (8)$$

where τ_0 refers to the stresses at end of the slope while the stresses upslope are less than the peak strength, this is the classical case of progressive failure. The situation of interest for particular phenomenon is when

$$S_P > \tau_0 > \lambda_1 S_P \qquad (9)$$

In this case a local shear failure will begin at point 0 and will move as far up the slope as P_1' that is it will include all the portion that exhibits a truncated peak strength. It is assumed that the initial stresses before failure have the distribution shown on Fig. 4b, i.e. at point P_1, $\tau_{P_1} > \lambda_1 S_P$. Now shear stresses will initially be reduced to S_{TP} on OP_1. Lateral unloading will cause expansion of the marl. With adequate differential strain across the failure zone the shear stress will be further reduced from S_{TP} to its residual value S_r. If the resulting reduction in stresses is large, this will cause a corresponding large increase of the shear stresses to the right of P_1.

The adjacent block $P_1BB'P_1'$(Fig. 4c) is investigated next. Section BB' is unaffected by the stress conditions at P_1P_1'. If E_P is the passive lateral force acting on P_1P_1' the shear stress at P_1 (Fig. 4d) will be :

$$\tau_{P_1} = \tau_{max} = \gamma \ . \ z \ . \ \cos a \ . \ \sin a + k \frac{E-E_P}{P_1B} \qquad (10)$$

The shear strength to the right of P_1 is assumed to be

$$S = \lambda_2 S_P \quad (\lambda_2 < 1) \qquad (11)$$

If the shear stresses on P_1P_3 are greater than $\lambda_2 S_P$ then the progressive failure continues up to section P_2P_2'. It is assumed that shear stresses to the right of P_2 are less than $\lambda_2 S_P$. The next block can be examined accordingly. If on the other hand the shear stresses to the left of P_1 are less than $\lambda_2 S_P$, or more commonly

$$S_{P_1P_2} = S_P > \tau_{P_1P_2} \qquad (12)$$

there will be no progressive failure. The result would then be merely a loosening of the end part of the slope, which would possibly be cut off completely. Although this by no means is intended to constitute a proof it should be mentioned that such cutting off of end blocks of slopes for no other apparent reason were observed on several occasions in the area investigated.

Several other possibilities exist. If, for example, although the strength on P_1P_3 is greater than the shear stresses, at some portion $P_3P_3'P_4P_4'$ intermediate between sections P_1P_1' and BB'(Fig. 4c) the peak strength is again a truncated one, $\lambda_3 S_P'$ and the stresses on P_3P_4 are greater than $\lambda_3 S_P'$ then the stresses on the surface P_3P_4 will initially fall to a $\lambda_3 S_P$ value. This in turn will create a concentration of stresses on both sides of the block $P_3P_3'P_4P_4'$. This concentration may possibly trigger a mechanism of failure if lateral expansion is possible. This would be a more complex form of progressive failure. The same type of failure can possibly be initiated in the case where a portion with a truncated peak strength is not situated at the very end portion of the slope but near enough so that the shear stresses will still be greater than S_{TP}.

If in the above cases not enough lateral expansion was allowed and a progressive failure was not initiated, then this could lead to the creation of a predetermined slip surface in the slope, ready to initiate a failure mechanism if new conditions for lateral unloading subsequently appeared.

CONCLUSIONS

Shear tests on samples of a blue marl from Epirus exhibited irregularities in the stress -

strain curves of a sizeable percentage of the samples tested. An important irregularity, presented herein, was that of the stresses passing to the softening side of the stress-strain diagram without ever reaching the anticipated peak strength.

It is thought at this stage that the most probable explanation of such a behaviour must lie in the existence of small, non-apparent fissures. The curves obtained could be the result of the combination of the influence of the shear strength of a fissure contained in part of the failure plane and the shear strength of the intact material.

An examination of possible consequences such a behaviour might have on slope stability in the context of the progressive failure paradigm proposed by Bjerrum (1967) led to the conclusion that if part of a clay slope is characterized by this behaviour a slide may be initiated without the shear stresses exceeding what normally would be expected to be the peak strength value.

REFERENCES

Bjerrum, L. (1967). Progressive failure in slopes of over-consolidated plastic clay and clay shales. ASCE, J. S.M.F.D., (93), No. SM5, May, 3 - 49.

Brown, E.T., Richards, L.R. and Barr, M.V. (1977). Shear strength characteristics or the Delabole Slates. Proc.Conf.Rock Engg., Un. Newcastle-upon-Tyne, 33 - 51, England.

Cavounidis, S. (1979). An irregular stress-strain behaviour. Proc.7th Eur.Conf.Soil Mech.Found.Engg., (4) 83, Brighton.

Cavounidis, S. and Sotiropoulos,E. (1979a). A report on slides on slopes of marly clay. Proc.6th A.R.C., (1), 209 - 212, Singapore.

Cavounidis, S. and Sotiropoulos, E. (1979b). Strain softening marly rock. Proc.4th Cong. I.S.R.M., (1), 63 - 67, Montreux.

Cavounidis, S. and Sotiropoulos,E. (1980). Hypothesis for progressive failure in a marl. ASCE, J.Geotech.Div., June, 659 - 671

Chandler, R.J. (1967). The strength of a stiff clay. Proc.Geotech.Conf., (1), 103 - - 108, Oslo.

Jaeger, J.C. and Cook, N.G.W. (1976). Fundamentals of rock mechanics. 2nd Ed., Chapman and Hall Ltd., London.

Lupini, J.F., Hight, D.W. and Cavounidis, S. (1980). Some observations of microfabric and their role in understanding soil behaviour. Riv.Ital.Geot., Italy (to be published).

Mitchell, R.J. (1970). On the yielding and mechanicalstrength of Leda clays. Can.Geot. J., (7), 59 - 69.

Townsend, D.L., Sangrey, D.A. and Walker, L.K. (1969). The brittle behaviour of naturally cemented soils. Proc.,7th Int.Conf. Soil Mech.Found.Engg., (1), 411 - 417, Mexico City.

Table 1

Average Properties of Marls

Marl Type	Percent smaller than 2μ grain size	Liquid Limit %	Plastic Limit %	Plasticity Index %	Natural Water Content %	Dry Unit Weight kN/m³	Void Ratio	Unconfined Compressive Strength kN/m²	Peak Cohesion intercept kN/m²	Peak Angle of Shearing resistance	Residual Cohesion intercept kN/m²	Residual Angle of Shearing resistance
A		52	18	34	19	17.5	0.55	430	120	31°	30	22°
B	31	39	16	23	12.6	19.0	0.44		35	44°	0	31°
C		46	16	30	11.4	19.9	0.36	1730	185	55°	55	34.5°

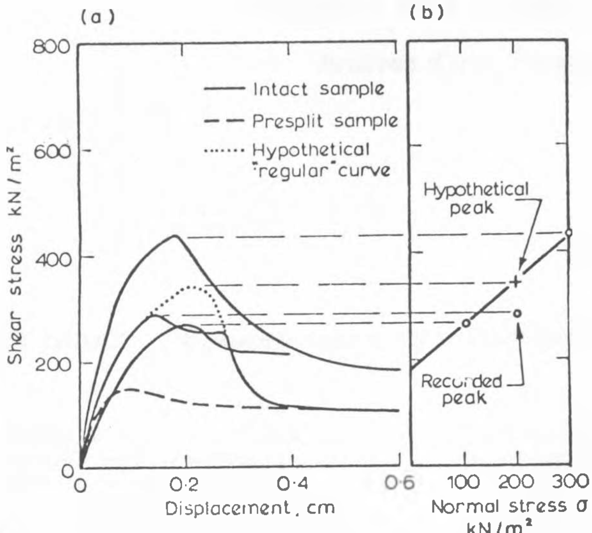

Fig.1.Direct Shear Test Results on Marly Clay Sample(A)(at Natural Water Content Showing Truncated Peak Behaviour (a)Stress-Displacement Curves (b)Strength Envelope

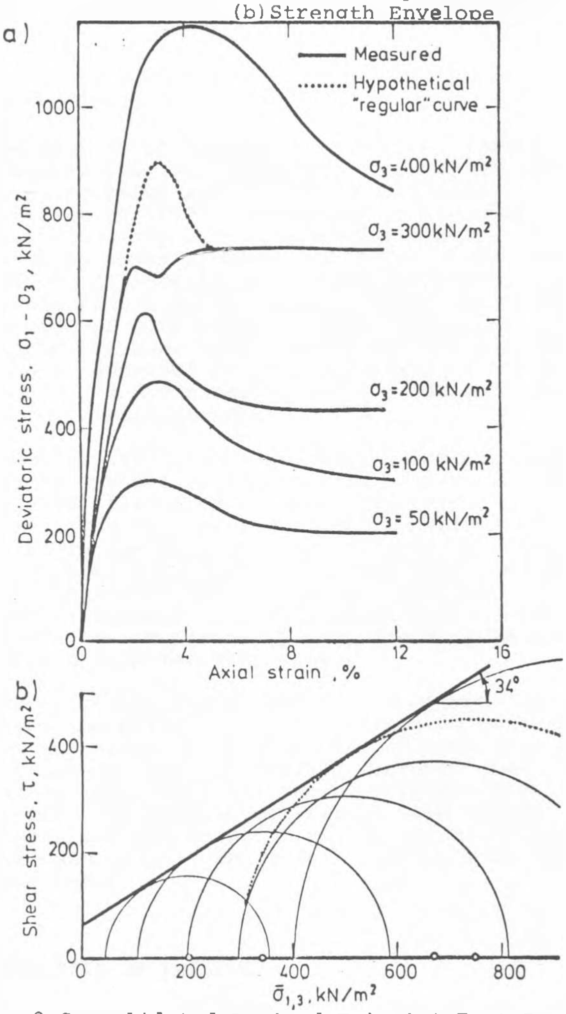

Fig.2.Consolidated Drained Triaxial Test Results on Marly Clay Shale(B)Sample Showing Truncated Peak Behaviour (a)Deviatoric Stress-Axial Strain Curves (b)Strength Envelope

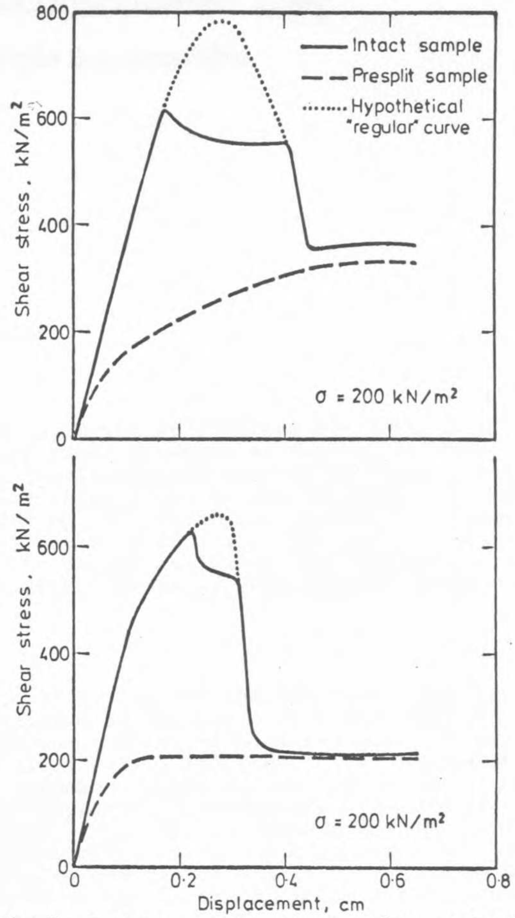

Fig.3. Direct Shear Test Results on Marly Rock (C) Samples Showing Truncated Peak Behaviour

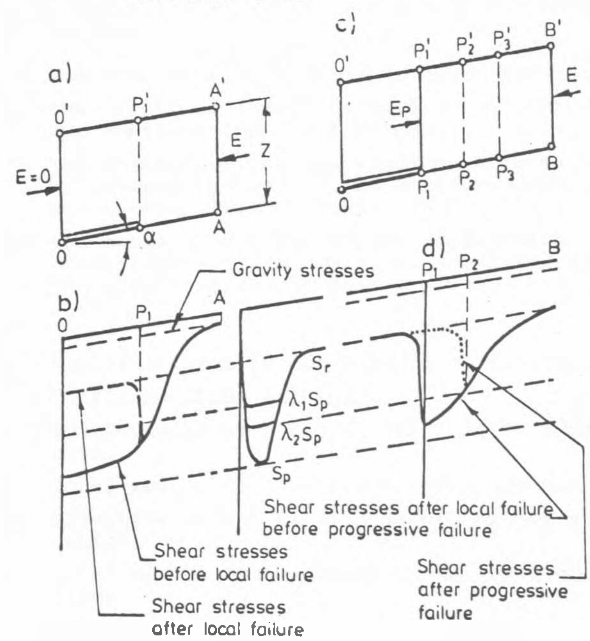

Fig.4. Principle of Progressive Failure of a Slope with Truncated Peak Strength.