Evolution of Slopes in Over-Consolidated Clays

Evolution des Talus dans Argiles Sur-Consolidées

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SYNOPSIS The stability of natural slopes in over-consolidated Italian clays is investigated. The shearing resistance parameters have been measured by laboratory tests, both on undisturbed and remoulded samples, either in fully softened or in the residual state. Both first-time and periodical slides have been examinated; the back-computed angles of shearing strength are compared to the measured ones.

INTRODUCTION

In Northern Italy, a lot of landslides are related to a clayey marine formation, which is named Lugagnano Clay and is widespread along the border between the Apennines and the River Po Basin, from Piedmont to the Adriatic Sea.

After the deposition (Lower Pliocene), these clays were covered by a great thickness of regressive sands and gravels, that were successively eroded; values of the preconsolidation pressure as high as 4000÷5000 kPa have been determined by means of oedometer tests.

The studied area is situated in Southern Piedmont (Alessandria District).

SOIL PROPERTIES

The typical soil profile, relating to a slope near the small town of S. Cristoforo, includes the following sequence (fig. 1):

(a) reddish brown clayey silt, deriving partly from weathering and remoulding of the underlying Lugagnano Clay and including sand and gravel elements , residual from the erosion of the coarse covering (colluvial soil);

(b) light brown silt with clay and traces of sand (in situ weathered Lugagnano Clay);

(c) grey-blue silt with clay (unweathered Lugagnano Clay).

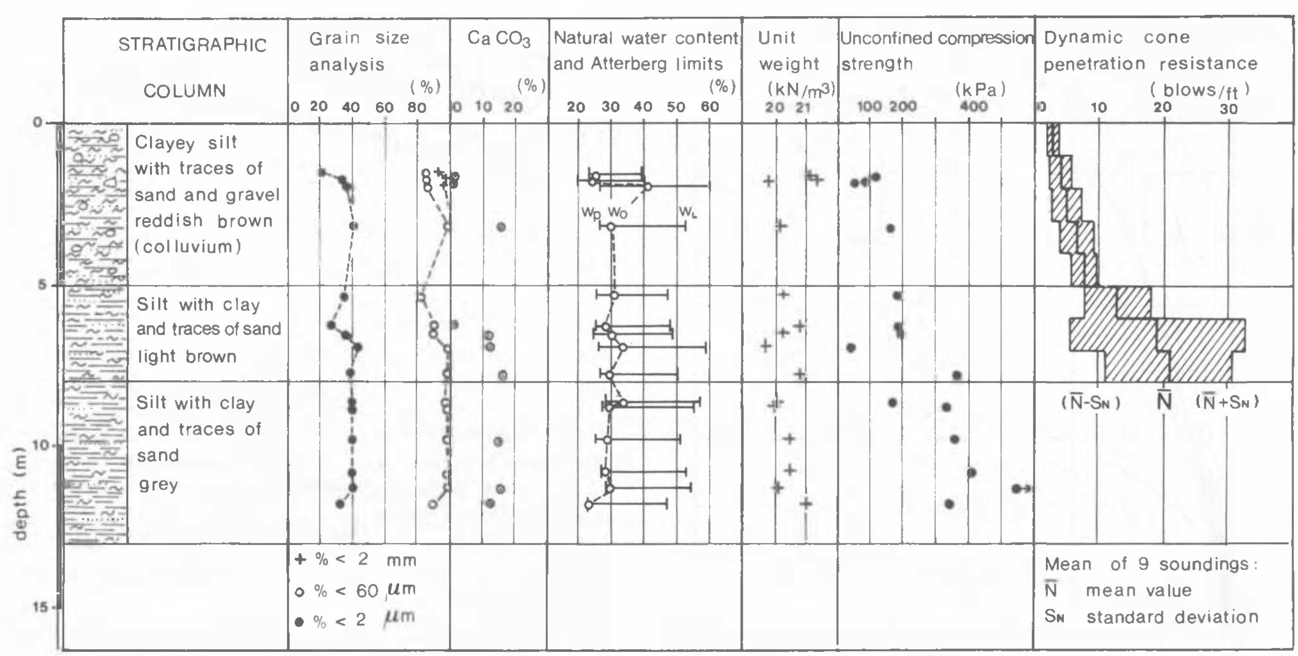

Fig. 1 Soil profile at S. Cristoforo

The limit between (a) and (b) can be recognized also by means of dynamic cone penetration soundings. The test specifications in Italy are: diameter of 60° conical drive point = 51 mm; outer diameter of the casing = 48 mm; hammer weight = 7.3 kN; hammer drop = 0.75 m.

Colour excluded, no distinction can be made, only on the base of identification tests, between (b) and (c). The prevailing lithotype is a medium-high plasticity clay (CL-CH according to the Unified Soil Classification System - fig. 2); the clay fraction is generally inactive (activity index $A_c \approx 0.7$); the content in $CaCO_3$ ranges from 10 to 16 %, so that the soil can be defined as "weakly marly clay"; the unit weight γ is about 20.5 kN/m^3.

Besides than over-consolidated, Lugagnano Clay is also fissured. In slopes, fissuration can induce

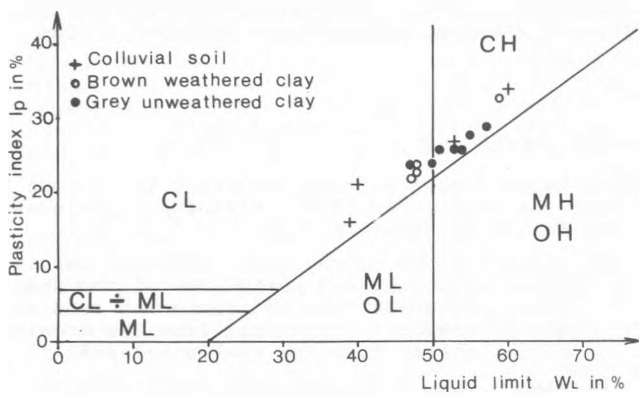

Fig. 2 Plasticity chart

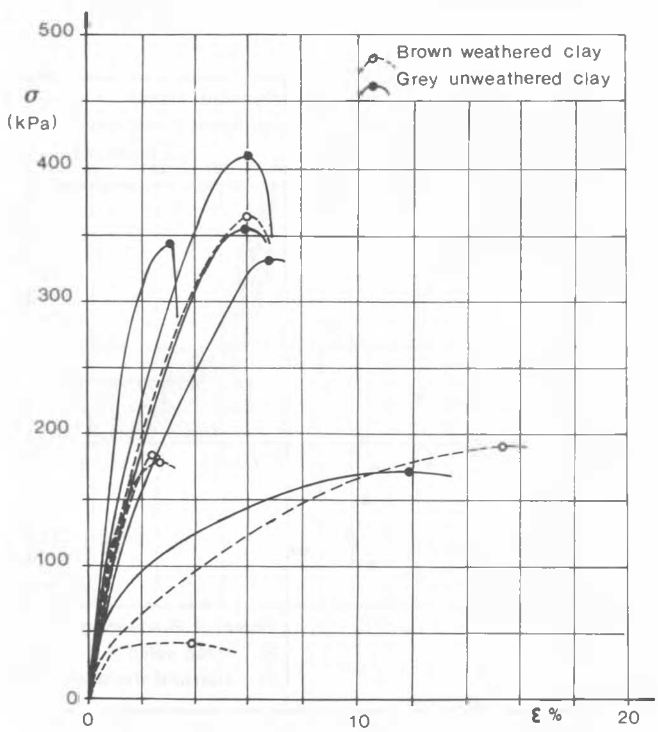

Fig. 3 Results of unconfined compression tests

a process of softening, during which a progressive reduction of shearing strength takes place. However, softening affects not only the weathered horizon, but extends irregularly in depth and can affect also some upper parts of the grey unweathered clay: some results of unconfined compression tests on 80 mm diameter specimens, showing a different stress-strain behaviour between softened and unsoftened samples, are reported in fig. 3.

The shearing resistance (in drained conditions for long-term analyses) has been investigated in the laboratory. The following parameters in terms of effective stresses have been measured:

- the peak values c'_p and ϕ'_p for the unsoftened and unweathered grey-blue Lugagnano Clay (by means of triaxial tests on undisturbed core samples);
- the shearing resistance angle $\phi'_s = \phi'_p$ in fully softened conditions (by means of triaxial tests on remoulded specimens, at initial water content close to liquid limit), the intercept cohesion c'_s resulting negligible or null;
- the residual angle ϕ'_r (by means of direct shear tests along a soil/rock contact, according to the method proposed by Kanji, 1974), the intercept cohesion c'_s resulting approximately zero.

The results that have been obtained for Lugagnano Clay are synthetically represented in fig. 4.

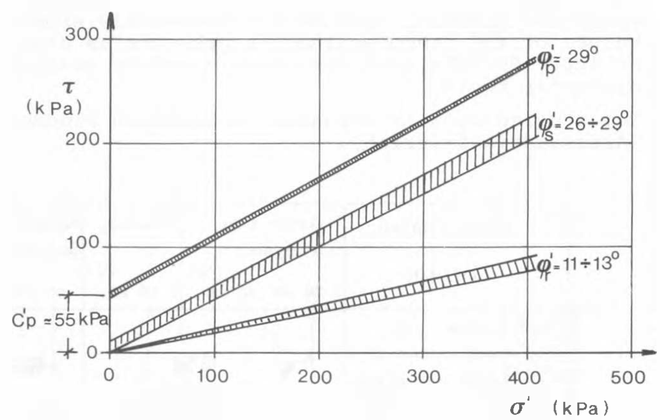

Fig. 4 Envelopes of shearing resistance tests

The values of ϕ'_s are slightly lower than the values that can be evaluated in function of the plasticity index, according to Kenney, 1959, or Bjerrum and Simons, 1960 (fig. 5).

The same correlation for the measured values of ϕ'_r is fairly in agreement with the relationship that has been proposed by Kanji, 1974; the correlation between ϕ'_r and the content in clay fraction agrees with the results published by Blondeau, 1973 (fig. 6).

The common simplification of assuming zero for c'_s and c'_r in stability analyses is endorsed by the test results.

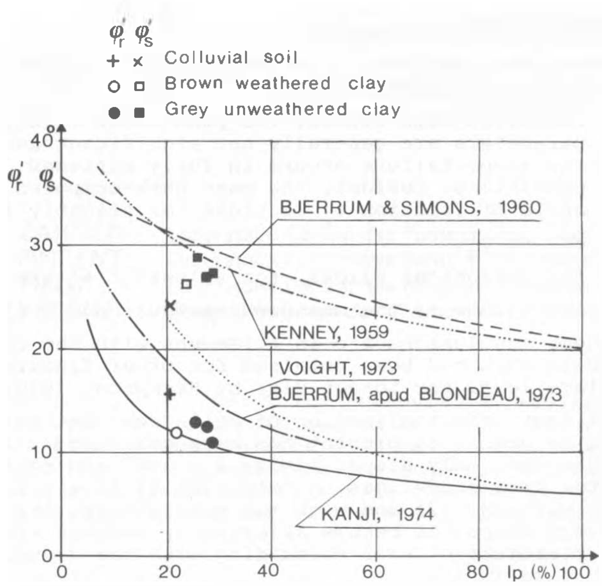

Fig. 5 Shear resistance angles ϕ_s' and ϕ_r' versus plasticity index

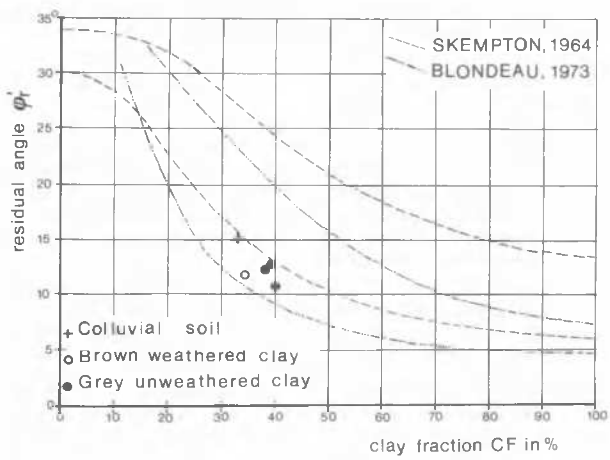

Fig. 6 Residual angle ϕ_r' versus content in clay fraction

PERIODICAL SLIDES

As a first example of stability analysis, a periodical slope movement near S. Cristoforo, concerning the safety of an important oil pipeline, can be selected.

The situation of the hillside is represented by the section in fig. 7. The angle of slope to horizontal β varies from 4° near the bottom of the valley up to about 16° near the crest of the hill; in the slide area $\beta \simeq 8°$.

The dimensions of the sliding area are:
- maximum length L = 200 m;
- maximum breadth B = 230 m.

The subsoil was investigated by means of 3 borings and 9 dynamic penetration soundings; the soil profile is the same which is represented in fig.1.

The trend of the slip surface was recognized directly into trial pits and interpolated from the results of geotechnical investigations. As the depth D is slightly varying from 6 to 9 m, the sliding mass involves not only the colluvial soil and the weathered clay, but also part of the grey unweathered clay.

Open-tube piezometers were inserted into bore- and penetration-holes. Measurements went on for 6 months and allowed to sketch a possible maximum piezometric level, indicating that the flow of water is roughly parallel to the slope.

In these seepage conditions, the factor of safety for an infinite slope in cohesionless saturated soil sliding along a plane parallel to the ground surface is (Skempton and De Lory, 1957)

$$F = \frac{(\gamma - m\,\gamma_w)\cos\beta\,\tan\phi_m'}{\gamma\sin\beta} \qquad (1)$$

where: γ = unit weight of saturated soil;
γ_w = unit weight of water;
m = ratio between height of water level above the slip surface and height of the sliding stratum;
ϕ_m' = mean effective angle of internal friction along the slip surface.

By putting into (1) F = 1, the mean angle of internal friction at equilibrium can be evaluated by the expression

$$\tan\phi_m' = \frac{\gamma\tan\beta}{\gamma - m\,\gamma_w} \qquad (2)$$

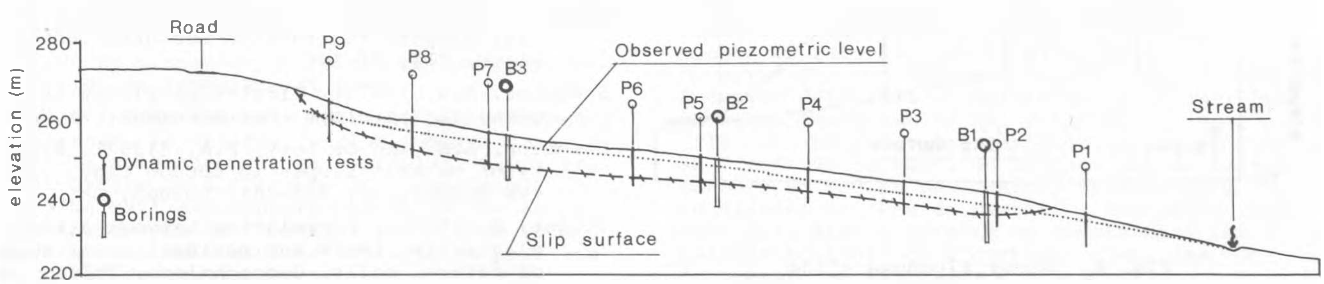

Fig. 7 S. Cristoforo slide

For γ = 20.5 kN/m³, β = 8°, m = 5m:8m = 0.625, it results ϕ'_m = 11.4°; this value agrees well with the lowest measured values of ϕ'_r (11÷13°).

A more refined analysis can take into account the curvilinear sliding surface, which is represented in fig. 7.

By means of Janbu's simplified method of slices (Janbu et al., 1956), a mean value of ϕ'_m = 11.5° can be computed, that is very close to that one obtained by eq. (2).

FIRST-TIME SLIDES

A first-time slide has been also analysed, that is situated about 1 km far from the previously described area, at the source of a small stream tributary of River Albedosa (fig. 8).

The subsoil composition is similar to that one in fig. 1, with the difference that the colluvial cover is only 3 m thick.

The hillside had never been affected by other preceding slides. However, the stream source had been causing a continuous erosion at the head of the valley; so, the increase of the mean angle of slope produced in 1978 a sudden movement along a curvilinear surface (fig. 8).

The sliding mass was about 70 m long and 45 m wide; a geomorphological survey, the inspection of a drainage trench and 3 borings allowed to reconstruct the trend of the slip surface and the maximum piezometric level at failure (in part coinciding with the ground surface): the maximum heigth of the sliding mass was about 14 m.

The slope failure couldn't anyhow be justified by the measured peak shear strength parameters (c'_p = 55 kPa, ϕ'_p = 29° - cfr. fig. 4); so, the back-analyses were carried out for fully softened conditions (c'_s = 0).

With such hypothesis, a mean effective angle of internal friction along the whole slip surface can be computed (ϕ'_m = 24°), that is rather lower than the measured values (ϕ'_s = 26÷29°).

Likely, the agreement would be closer if a residual angle $\phi'_r < \phi'_s$ could be admitted whithin the superficial colluvial soil.

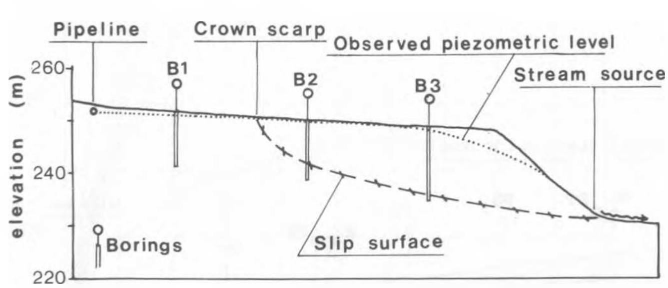

Fig. 8 River Albedosa slide

CONCLUSIONS

Stability analyses on natural slopes in Lugagnano Clay strengthened that:

- for first-time slides, the peak shear strength parameters are generally not significant and the slope failure occurs in fully softened conditions; besides, the mean back-computed angle of friction ϕ'_m is close (or slightly lower) to ϕ'_s;
- for periodical slides, the values of ϕ'_m are very close to the measured residual angle ϕ'_m.

These conclusions are in agreement with the results that had been obtained for other fissured clays (e.g. for London Clay by Skempton, 1970).

At last, the inclination of hillsides that seem to be stable at present has been examined. In practice, only slopes having β < 5÷6° are really free from landslides. This fact is in accordance with eq. (2) and with the possibility, for every slope, to become affected by seepage with a piezometric level coinciding with the ground surface (m = 1).

AKNOWLEDGEMENTS

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