Geomechanical characterization of residual soils for foundation design of a bituminous faced rockfill dam

Giorgio Pietrangeli
Studio Ing. G. Pietrangeli Srl
Via Cicerone 28 - 00193 Rome
Italy

Alessandro Cacciarini
Studio Ing. G. Pietrangeli Srl
Via Cicerone 28 – 00193 Rome
Italy

Maurizio Calabrese
Studio Ing. G. Pietrangeli Srl
Via Cicerone 28 – 00193 Rome
Italy

Introduction

The Great Ethiopian Renaissance Dam Project, GERD Project, is located on the Blue Nile close to Abay, in Ethiopia, a few kilometers upstream of Roseires Dam, in Sudan (Fig.1).
The reservoir will have a total capacity of about 75 billion of cubic meters and will be created by means of the construction of two dams: the Main Dam and the Saddle Dam. The power plant will have about 6 GW of installed power. The Main Dam is founded on rock and the Saddle Dam is founded mainly on residual soils. This paper describes the method used to estimate the geotechnical parameters of the residual soils in the foundation site of the Saddle Dam.
The saddle dam is a bituminous faced rockfill dam, BFRD, 5 km long and about 70 m high. Its highest typical section is shown in Fig. 2.
The main geological formations of the foundation area are three: schists, femic rocks, granites. Schits are also present with characteristics of phillite. The geomorphology indicates that between the two hills of the dam shoulders, constituted of schist on the left side and granite on the right one, the foundation material derives from the highly decomposed base rock. The dam is founded on residual soil for a length of 3.6 km, and for the remaining portion is founded on rock.
In the early stages, a seismic tomography campaign indicated that the depth of residual soils varies between 0 and 20 m, with a maximum depth of 40 m in the phillitic formation. The following investigation including trenches excavation, boreholes, permeability tests, in-situ tests including Marchetti dilatometer (DMT), Menard pressuremeter (PMT), plate load tests (PLT) and laboratory tests were carried out to gather information about the geotechnical properties of residual soils in the foundation area.
The authors present the findings of the general geology of the foundation area and the results of the interpreted mechanical properties obtained using different tests.

1. Methodology

The approach used to define the geotechnical design parameters of residual soils is the following.

1. More than ten big trenches with different height, varying between 10 and 17 meter and with vertical side-wall, have been excavated. Some trenches, the deepest one too, have been exposed to two rainy seasons without any collapse, resulting in a high values of cohesion. Their mechanical behaviour have been simulated by means of Mohr-Coulomb model, and their stability analysis considering a safety factor equal to 1.1, have permitted to define the minimum values of shear resistance of the residual soils.
2. Twenty DMT profiles and more than ninety PMT tests have been performed until 15 m of depth. The DMT has been performed inserting the blade dynamically, instead of the more common static way, due to the high shear resistance of the material. In all profiles DMT 'feels' this materials like a silty-sand or sandy-silt, thus like a non- cohesive material, providing the friction angle as shear resistance parameter. This means that the interpreted friction angle represents the shear resistance of the material including the cohesion contribute and the friction contribute.
3. Due to the very low degree of saturation, it can be assumed that the soil is in a fully drained condition. In addition with the hypothesis that the residual friction angle of the material is equal to its peak friction angle, it is possible distinguish the contribute of cohesion from that one of friction angle. So reinterpreting the DMT data under this hypothesis and considering the Mohr-Coulomb failure criterion, it is possible to obtain the two parameters, c' and φ', of the materials directly from in situ test.

4. The direct shear tests, executed in laboratory, were used to verify the validity of the hypothesis and to make a comparison between the c' and φ' parameters interpreted by means of in situ tests and those coming out from laboratory. Despite the anisotropy of the residual soils, it is possible to mark the linear variation of cohesion with depth and the good agreement with the in situ estimated drained cohesion.

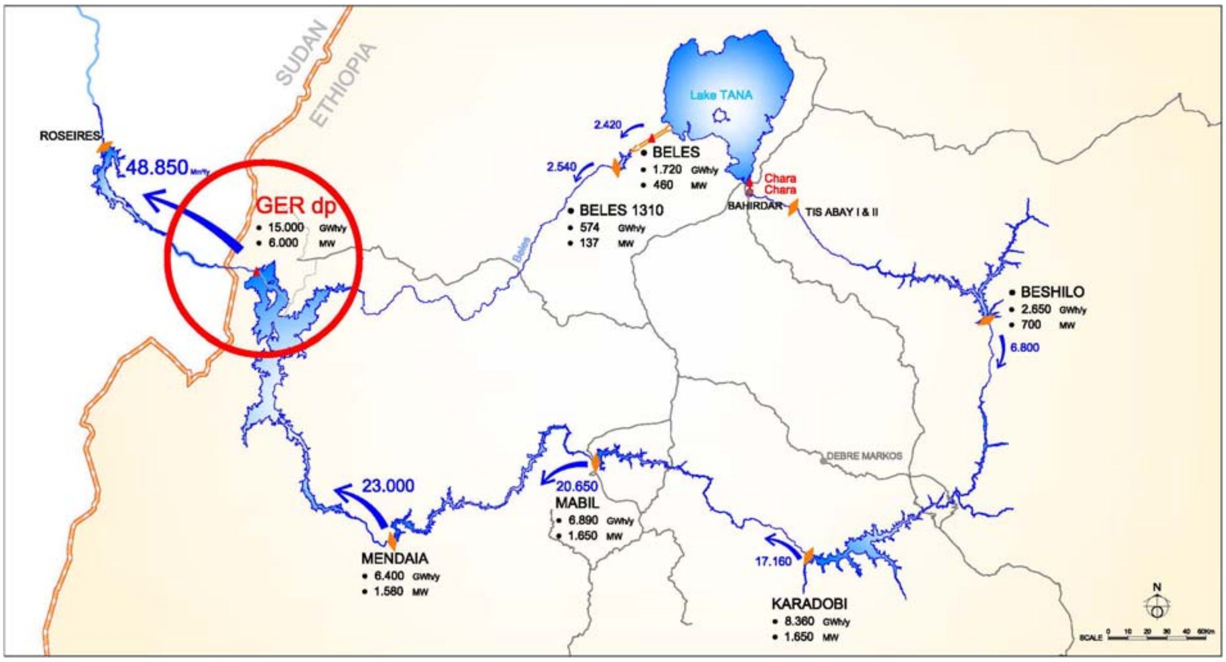

Fig. 1. Map showing location of the GERD project along the blue Nile, Ethiopia.

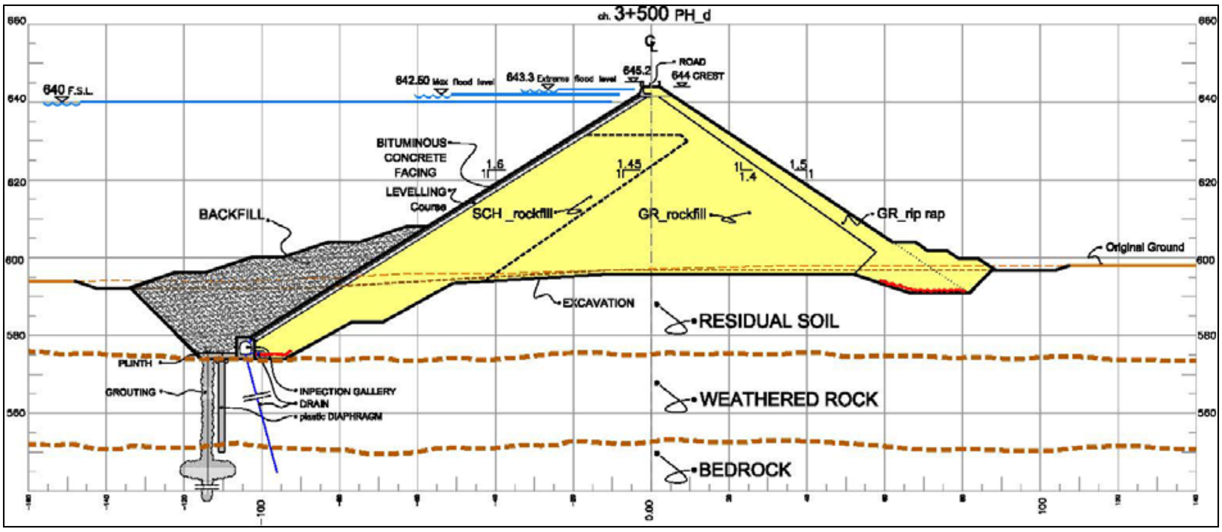

Fig. 2 - GERD project, saddle dam, TYPICAL CROSS SECTION ch. 3+500

2. Geology

The basic geological formations of saddle dam site, shown in Fig. 3, from left to right, are:

- SCHIST with a narrow vein of quartzite (green zone);
- FEMIC rocks, highly decomposed into residual soil, from 5 up to 20 m of depth (cyan zone);
- PHILLITE highly decomposed and crossed by large intrusion of marble, up to 40 m of depth (orange zone);

- GRANITE with a narrow vein of quartzite at the interface Phyllite/Granite present in the dam foundation area with different weathering degree (decomposed / weathered / fresh), from 10 up to 0 m of depth (magenta zone).

The presence of deep residual soil strongly affected the design of the dam. The highest typical cross section, Fig. 2, shows that the residual soil will be excavated only along the upstream and downstream toes in order to reach a soil level with adequate shear resistance and stiffness.

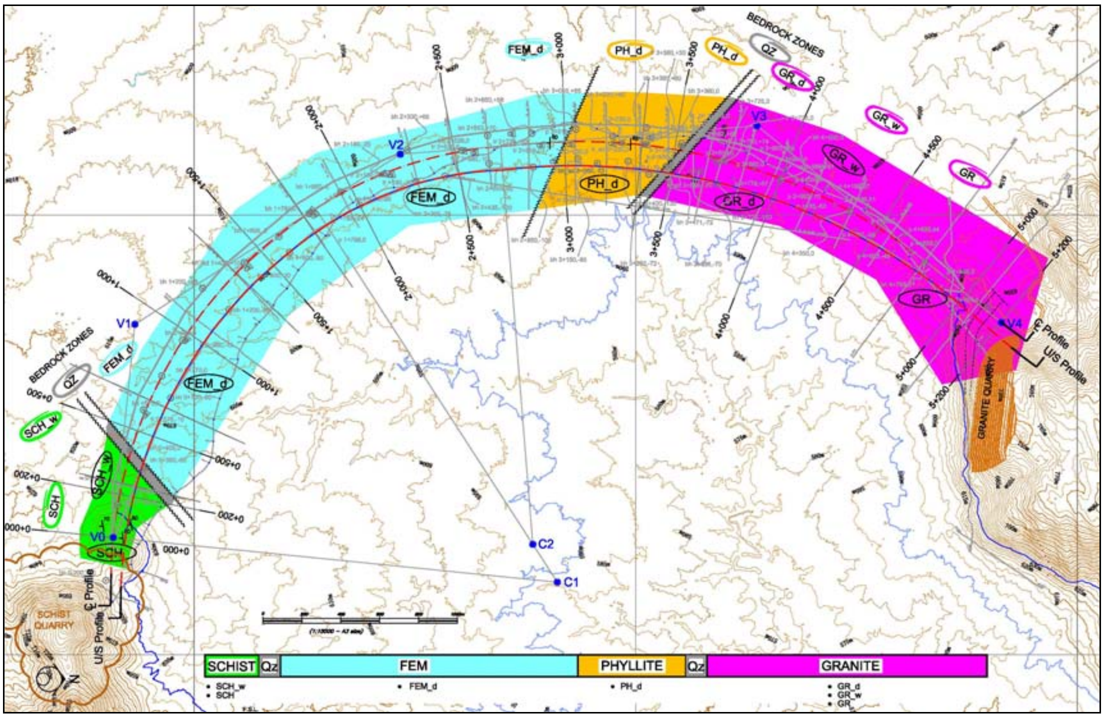

Fig. 3 - GERD project, saddle dam, GEOLOGICAL PLAN

3. Trench Excavation

Fig. 4 (left) shows the vertical side-walls of the deepest excavated trench, 17 m deep, and (right) a particular of the soil structure. The material is similar to a high, over-consolidated clay or a soft rock. Its stability, after two rainy seasons, implies a large cohesion.

Adopting the Mohr-Coulomb failure criterion, considering a safety factor of 1.1 in the stability analysis and a cohesion constant with depth, with the back analysis it has been possible to define the minimum value of shear resistance of the residual soils for the three formations.

Moreover, the operation of trench excavation has permitted the formation of several large soil heaps from which to measure the angle of natural repose, ANR, of the remoulded soil. This parameter assumes an important role in the investigation, providing the key to conceiving a method to estimate the drained parameters, c' and φ', in dry residual soils from in situ tests.

4. Identification Tests

Many samples were taken during trench excavation to execute identification tests for the different geological formations. The samples taken manually, directly from the side-walls of trenches, were treated in the site laboratory. The samples taken by means of a triple core bit mounted on a drill rig were treated in an external laboratory for executing the shear tests.

The identification tests performed are: unit weight, natural water content, specific gravity, degree of saturation, grading and plasticity. The summary of the results are plotted in Figs. 5 and 6.

Grading curve indicates that D_{100} rarely exceeds a few millimetres and the percentage of fines varies in the range of 50 - 80 %. Plasticity chart shows that most of data points are located along the “A line” with plasticity index in the range 5 - 25. The degree of saturation is very low, less than 35 %.

The residual soils, in remoulded state, can be classified according the USCS as:

Class	USCS	Description
FEM	SC / SM	sandy clay / silty sand
PH	SM / ML	silty sand / silt of low plasticity
GR	SM	silty sand

Table 1 – USCS Residual Soil Classification

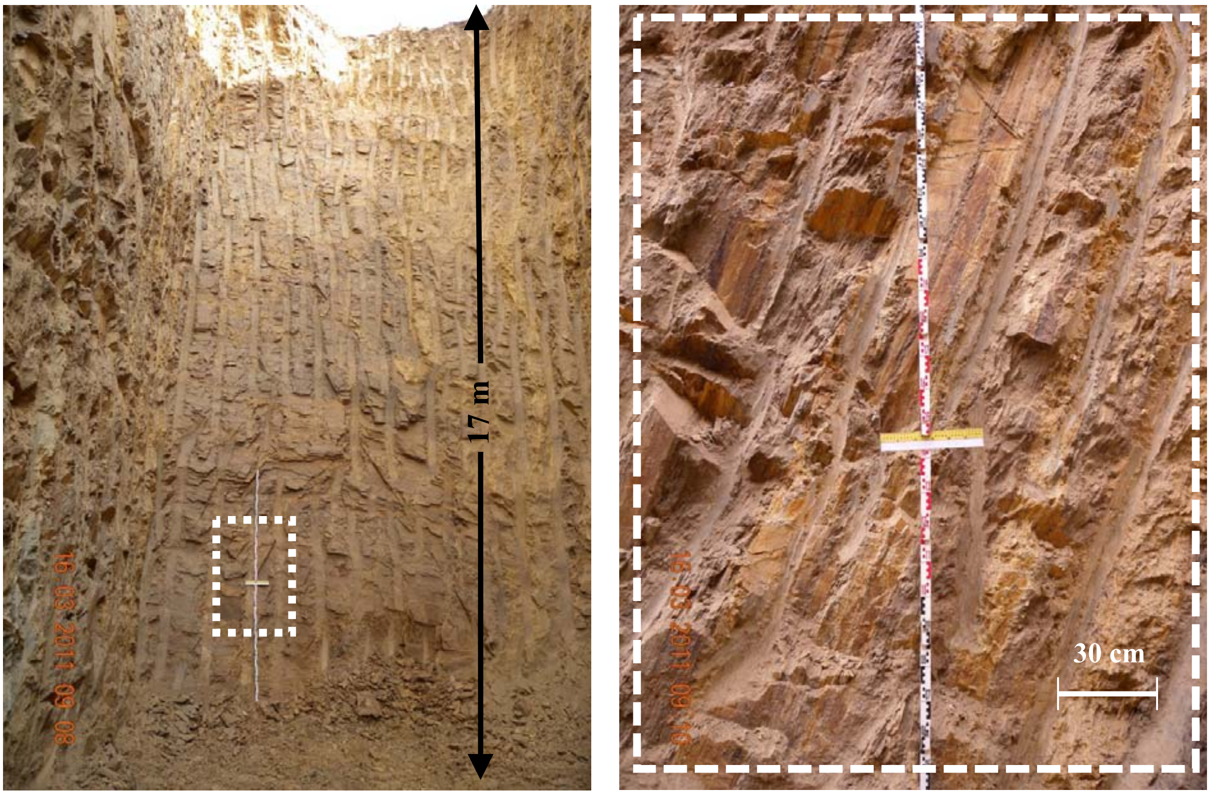

Fig. 4 - Trench in Phillitic formation

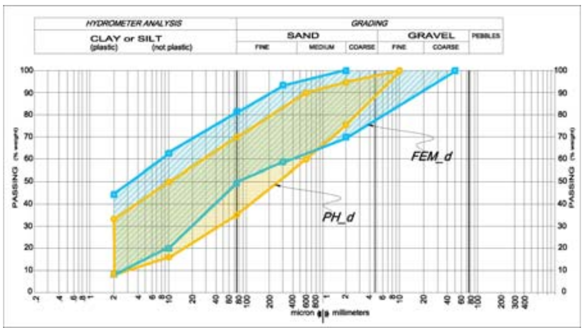

Fig. 5 – Residual soils grading from femic and phillite

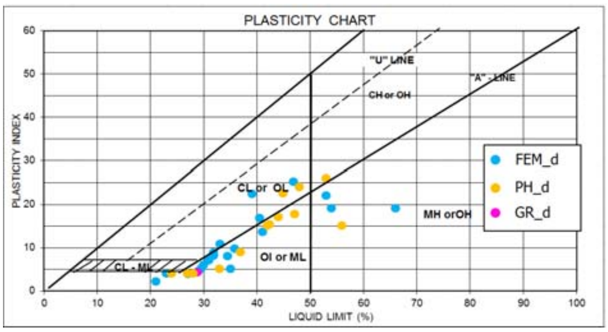

Fig. 6 – Plasticity Index summary graph

5. In Situ Tests

During the selection of in situ tests to use for the investigation, SPT and CPT were discarded. The quality of the results of SPT tests are not significant because this test is strongly influenced by the type of equipment, the operator and the site characteristics. In addition, with the CPT test it is not possible to reach the required depth, about 10-15 m, by means of the static thrust of a ballasted penetrometer, due to the stiffness and strong resistance of the residual soils.

The chosen in situ tests were Menard pressuremeter (Menard, 1956), PMT, and Marchetti dilatometer (Marchetti, 1980), DMT. The plate load test, PLT, was also executed because it is a simple test to perform, easy to control and does not require any calibration. In contrast it requires the excavation of a large trench in order to collect only few test data as a function of depth. The limit of this test is identified in the disturbance induced in the soil during the excavation phase for the preparation of the test area. The PLT has been adopted to provide a common basis of comparison for the PMT and DMT which estimate the stiffness of the soil also, in order to confirm the accuracy of PMT and DMT.

Both PMT and DMT have been performed to estimate the mechanical properties of soils, in particular the oedometric modulus, M, and the angle of shear resistance in the hypothesis that the material is purely frictional (c'=0). Of course considering the high value of cohesion highlighted by the trenches stability, the friction angle estimated by PMT and DMT contains the friction contribute and the cohesive contribute of the shear resistance. For this reason the friction angle provided by in situ tests has been indicated with the symbol φ^* in the diagrams.

A first comparison of the geotechnical interpreted parameters can be made observing in the unit weight, γ, estimated from DMT and measured in laboratory (Fig. 7), and the oedometric modulus, M, from different in situ tests (Fig. 8).

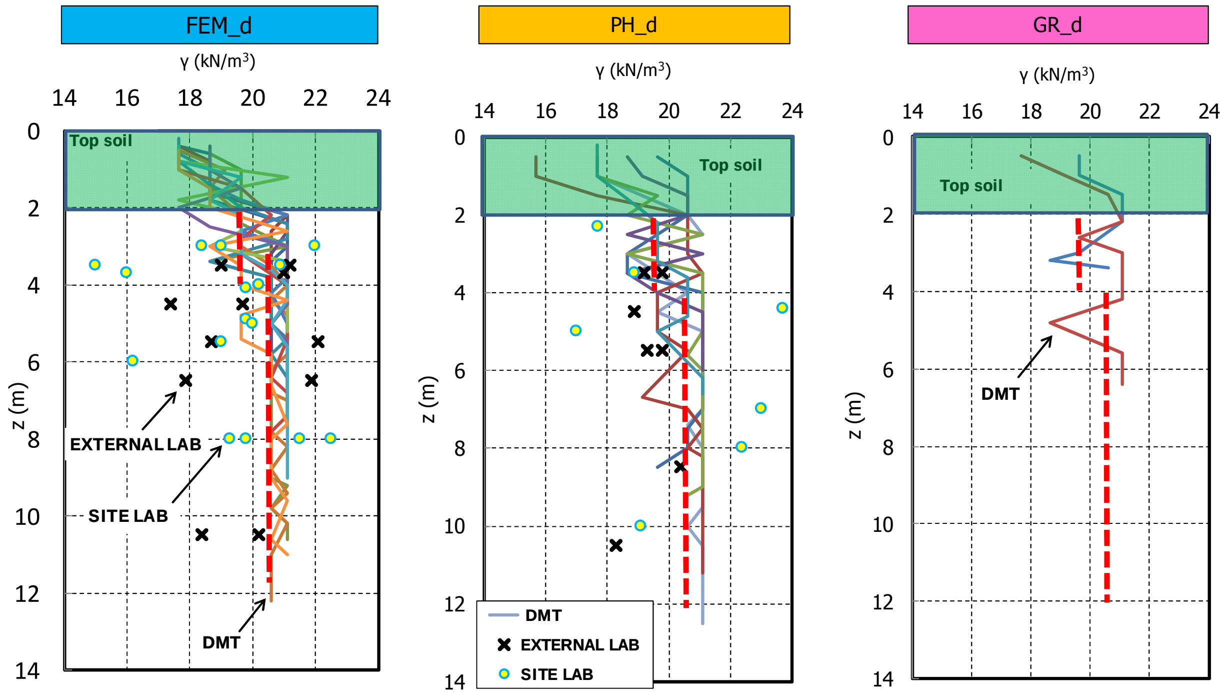

Fig. 7 – Unit weight from DMT and LAB

The unit weight estimated by DMT varies in a narrow interval with respect to that one obtained in laboratory. It seems quite similar for the different formations.

As shown in Fig. 7, for the dam design it has been assumed a unit weight equal to:

- 19,5 kN/m^3 for up to 4 m of depth;
- 20,5 kN/m^3 for depth over 4 m;

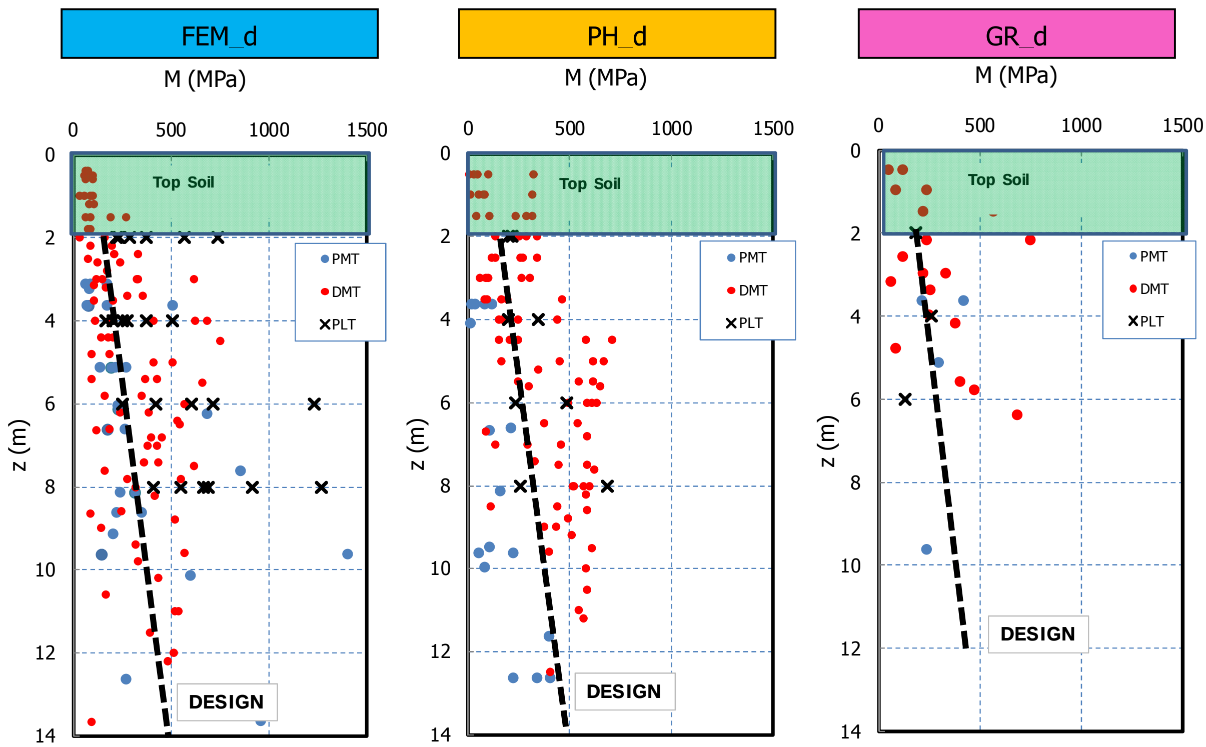

Fig. 8 – Oedometric modulus from in situ tests

The oedometric modulus ranges in the same interval for all the formations. A certain scattering can be noticed in the data due to the anisotropy, caused by developing of the weathering phenomenon. In particular the PMT seems to provide lower values than DMT and PLT, mostly in phillitic formation. The modulus from laboratory results very low, at least one order of magnitude less, affected by disturbance in sampling and transportation. It can be assumed that oedometric modulus varies linearly with the depth from a minimum of 200 MPa @ 4 m to a max of 500 MPa @ 12 m. The equation is: M (MPa) = 30xZ(m)+80 MPa

6. Drained Parameters, c' and φ', From In Situ Tests

In residual soil, weathering phenomenon can be defined as the physical, chemical and biological reactions that decompose a rock massif in increasingly smaller grains with lesser attraction forces between them. This means that the cohesion highlighted by the stability of the deep trenches is not similar to that one of an alluvial soil (Vaughan, 1988).

Considering the residual soil as a solid matrix material, instead of a mass of granular sediments joined all together, it is possible to formulate the hypothesis that the shear resistance is mainly given by the cohesion and, after the mechanical collapse of the main structure, the peak friction angle of the material is the same of the residual one. It follows that the residual friction angle can be expressed by the angle of natural repose, ANR, already determined in situ during the trenches excavation.

The very low degree of saturation of the soil allows to consider the DMT and PMT in situ tests fully drained. This assumption, with the previous hypothesis and adopting the Mohr-Coulomb failure criterion, allows the drained cohesion from the DMT data to be estimated.

The cohesion for the residual soil of the three geological formatiosn has been obtained from the DMT using the three steps, illustrated in the Fig. 9:

1. At the considered depth draw, on the Mohr Coulomb plane, the straight line of failure φ* starting from axes origin, obtained from DMT;

2. From the vertical effective stress value, estimated by DMT, draw the Mohr's circle, tangent to the straight line and with horizontal effective stress greater that the vertical one. This last condition descend by the fact that Kd parameter of DMT, similar to the over consolidation ratio, OCR, indicates that the material is like an over-consolidate material.
3. Draw the tangent to the Mohr circle , with slope equal to ANR, and read the c' on the intercept with the vertical axis.

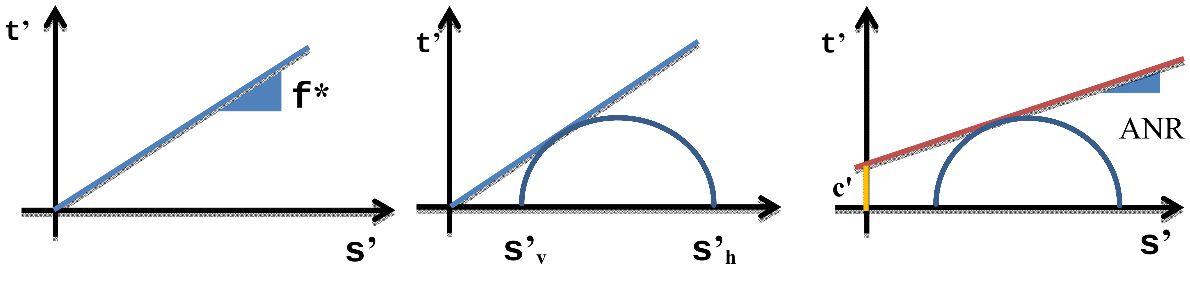

Fig. 9 – Determination of cohesion from DMT test

7. Laboratory Direct Shear Test

The direct shear test has been chosen, for its simplicity and reliability with respect to the triaxial test, to verify in laboratory the method used to estimate the drained parameters of shear resistance from in situ tests. Starting from a total number of 25 samples, only 16 resulted of high quality class. Due to the anisotropy of the material every test was conducted shearing five specimens for each sample, instead of three.

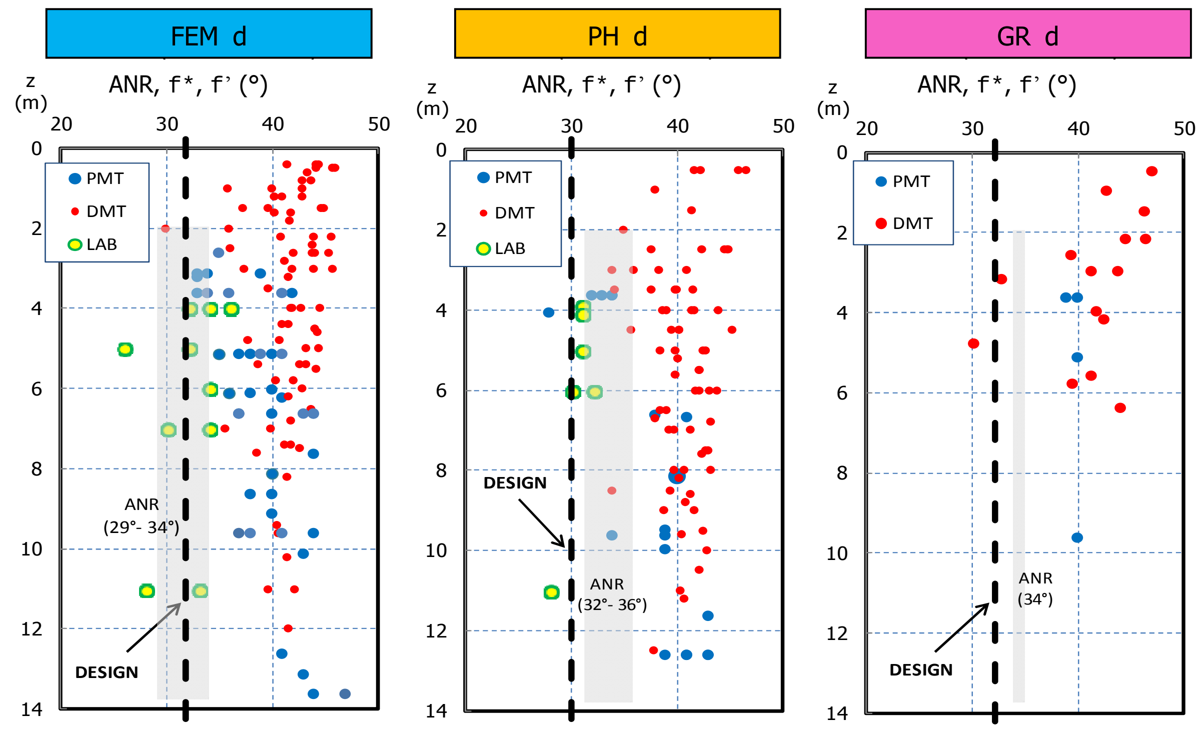

Fig. 10 – Friction angles from in situ and laboratory tests

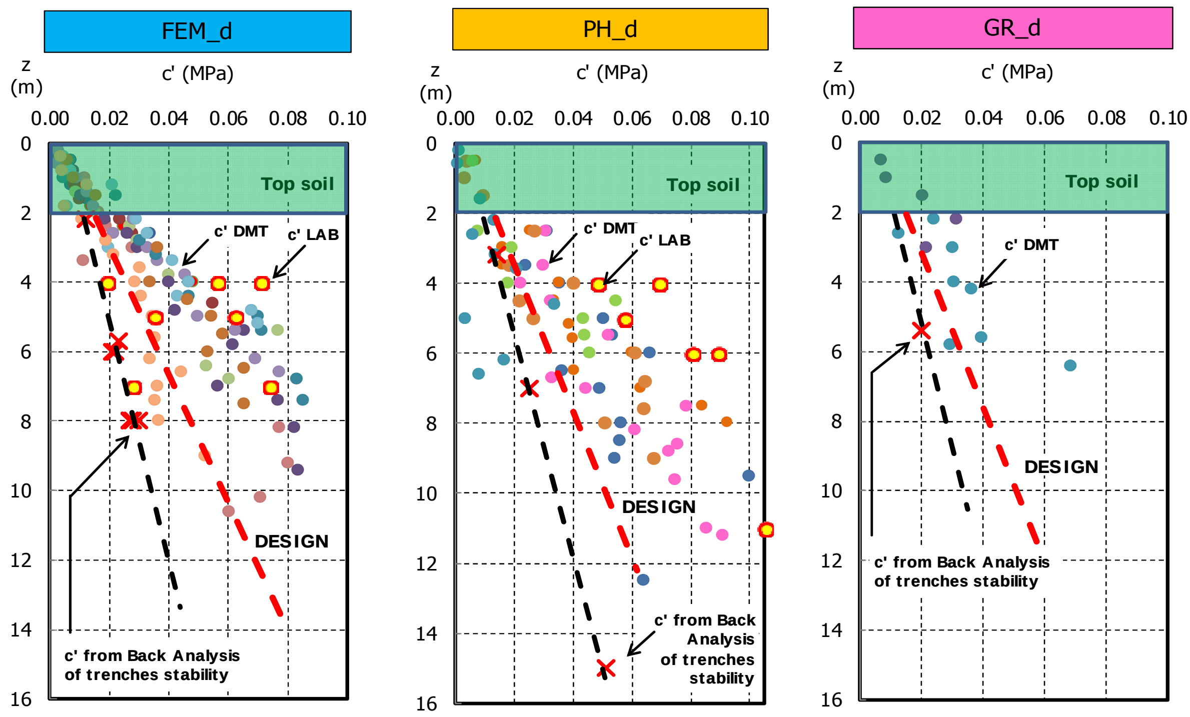

Fig. 11 – Cohesions from in situ and laboratory tests

The tests have proven that the peak friction angle is quite similar to the residual friction angle, discarding by no more than 1-2 degrees. This result can be assumed to be a good confirmation of the hypothesis, considering the difficulty in taking good quality samples with triple core bit equipment.
The cohesion measured in the direct shear test is in the same range of variation of the values estimated by means of ANR and DMT tests.
Figs. 10 shows the field values of friction angle ANR, measured in situ on the heaps, the friction angle φ*, as interpreted in situ by PMT and DMT, and the peak friction angle φ', measured in laboratory. It can be noted that ANR and φ' are in the same range, and that φ* is greater than φ' as expected because it 'contains' the contribution of cohesion.

Fig. 11 plots the variability of cohesion with depth from:

- the back analysis of trenches;
- the DMT data elaborated as a function of ANR, according to the approach of Fig. 9;
- laboratory shear tests.

It can be noted that the cohesion estimated by DMT for the phillite formation, is a little bit lower than that measured in laboratory. This is probably due to an over estimation of ANR in phillite, visible in the comparison of ANR and φ' from laboratory (Fig. 10).

These results seem to confirm the possibility to estimate c' and φ' parameters from in situ tests, especially from DMT test, in a dry residual soil.

8. Conclusions

The design parameters are shown in the following table.
The reasons of the conservative approach adopted are twofold:

1. this structure is of paramount importance;
2. residual cohesion measured in situ is not that of saturated soil.

	FEM_d					PH_d					GR_d				
z	γ	M	n	c'	f '	γ	M	n	c'	f '	γ	M	n	c'	f '
(m)	(kN/m^3)	(MPa)	(-)	(kPa)	(°)	(kN/m^3)	(MPa)	(-)	(kPa)	(°)	(kN/m^3)	(MPa)	(-)	(kPa)	(°)
0 - 2	TOP SOIL					TOP SOIL					TOP SOIL				
2 - 4	19.5	200		20		19.5	200		15		19.5	200		15	
4 - 6		250		30			250		25			250		25	
6 - 8		300	0.25	40	32		300	0.25	35	30		300	0.25	35	32
8 - 10	20.5	350		55		20.5	350		45		20.5	350		45	
10 - 12		400		65			400		55			400		55	
12 - 14		500		75			500		65			500		65	

Table 2 – Geotechnical design parameters

9. References

1. **ASTM D 1194-72**. Standard Test Method for "Bearing Capacity of Soil for Static Load on Spread Footings", 1977.
2. **Blight, G.E.** “Mechanics of residual soils”. Balkema, Rotterdam, 1997.
3. **Little, A.L.** “The engineering classification of residual tropical soils”. Proceedings Special Session, VII International Conference on Soil Mechanics and Foundation Engineering, Vol.1, Mexico City, 1969.
4. **Marchetti, S.** “In-situ tests by flat dilatometer”. J. Geotechnical. Engineering Division ASCE,106, GT3, 1980.
5. **Menard, L.** “The Menard Pressuremeter, Interpretation and application of pressuremeter test results to Foundation Design”. Sols Soils, 1975.
6. **Vaughan, P.R.** “Characterizing the mechanical properties of in-situ residual soils”. Proceedings 2nd International Conference Geomechanics in Tropical Soils, Singapore, Vol. 2, 1988.
7. **Viana da Fonseca, A.** “Correlating in-situ parameters from different testing procedures in Oporto residual soil from granite”. Geotechnical Engineering of Hard Soils-Soft Rocks. Proceedings 2nd International Symposium, Vol. 1, Ed. Anagnastopoulos et al.A.A. Balkema, Rotterdam, 1993.
8. **Viana da Fonseca, A. and Coutinho, R. Q.** “Characterization of residual soils”. Keynote paper – 3rd International Conference on Site Characterization. Taiwan, 2008.
9. **Wesley, L.D.**. “Engineering classification of residual soils”. 'Geomechanics inTropical Soils'. Proc. Sec. Int. Conf., Singapore, Vol. 1, pp. 77-84. A.A. Balkema, Rotterdam, 1988.

The Authors

G. Pietrangeli, a civil hydraulic engineer, is the founder and managing director of Studio Pietrangeli, a leading European engineering consultancy firm, based in Italy, specialised in the design of dams and hydropower projects. Over the past 50 years he has been personally responsible for finding innovative solutions and using non-conventional methodology to ensure successful implementation of the firm’s entire range of engineering services. His numerous projects include the feasibility, design and/or technical supervision of construction of well over 180 dams and 40 hydropower projects worldwide, particularly in Africa.

A. Cacciarini graduated in Geological Science from the University of Florence and has 35 years’ experience as a geologist, hydrogeologist and photo-interpreter of aerial photos and satellite images. Since 2003, as Chief Geologist at Studio Pietrangeli, he has been directly responsible for many of the geological studies related to dam and hydropower projects carried out by the firm in Africa and South America. Before joining SP he worked for many years with Telespazio and was involved in numerous missions for onsite verification of satellite interpretation. He also gained significant academic experience as a lecturer in photointerpretation, cartography and tele-detection of satellite imagery.

M. Calabrese, graduated in civil engineering at the University of L’Aquila, has acquired more than 20 years of experience in the geotechnical engineering, engaged in the area of characterization of soils from in situ and laboratory tests for special foundations, stabilization of slopes, deep excavation. He has been researcher in geotechnics at the Faculty of Engineering of the University of L’Aquila, from 2002 to 2006. As consultant in Studio Pietrangeli, he is involved in the GER dp (Ethiopia).