

Engineering Works Affected by Soft Rocks

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SUMMARY: Weak rock is usually defined as the intact rock presenting strength below about 25 MPa. Lately, more and more engineering projects are being built on sites dominated by soft rocks, as hard rocks might be absent in the region or the best geological sites have been already utilized. Many authors include in the category of soft rocks those rock masses that although presenting initially strengths above that upper limit, may be transformed in soft rocks under the effect of accelerated weathering, or behave as a weak rock mass with strong deformation due to high degree of jointing, high ground pressures, high temperatures, etc. Since soft rocks are usually less studied, there is a lack of knowledge about their properties and little confidence in their use. As a result, it is usual to adopt very conservative parameters for them, resulting commonly in additional economical expenses. The paper presents three cases (a dam foundation, a slope and an underground opening) to exemplify the influence of soft rocks in engineering works

KEYWORDS: Rock Mechanics, Soft rocks, Dam foundation, Slope failure, Tunnel collapse.

1 INTRODUCTION

Soft rocks are a critical material since it presents a series of problems. First of all they are intermediate between soil and hard rock and many time they can not be tested neither in soil mechanics lab due to its higher resistance, nor in rock mechanics lab as they are too soft to be trimmed and tested, very often crumbling before it can be tested. Secondly, and as a consequence, there is often a lack of adequate testing equipment for such materials. Additionally, there are many difficulties in sampling soft rocks, as conventional rotary drillings often destroy partially or totally the rock core - even triple barrels may not be suitable. Sometimes its failure is not according to the expected behavior, with intact rock failure at the same time as along discontinuities, representing a mix of traditional soil and rock failure.

As a result, soft rocks are little studied and there is little confidence on their properties to be utilized in important engineering works. Therefore, usually conservative parameters are adopted, for the sake of safety, but very often

against the economy.

Therefore, it is important the effort to investigate soft rocks to know their properties, explain their behavior, and try to find some index property that would allow to forecast their behavior. The lessons learned in problem solving in diverse engineering workings in soft rock would be a highly valuable mean to gather further experience in the matter.

Besides some research at Universities, the attempts to systematically study soft rocks have been made by the geotechnical societies. ISSMGE had a Soft Rock Commission under the leadership of L. Dobereiner, but after his death the commission slowed down and was later discontinued. In Brazil a compilation of sedimentary rocks of the Paraná Basin was coordinated by J. Oliveira Campos, which was published by ABGE (Brazilian Assoc. Eng. Geology). A multinational commission of Soft Rocks of the Paraná Basin was organized by Prof. J. J. Bosio Ciancio from Paraguay, who was initially its chairman. That commission was later converted in a Regional Working Group of IAEG, having produced an Interim Report with several contributions, but was not continued by

the new President Elect.

Under our suggestion, Dr. P. Pinto, President of the ISSMNGE proposed a new Joint technical Committee on Soft Rocks (JTC-7), jointly sponsored by ISSMGE, ISRM and IAEG, which started working but unfortunately was dismissed about 2 years later, with all other JTCs, except JTC-1 on Slope Stability. In 2011 during the ISRM International Congress, this Author proposed the establishment of a Technical Commission on Soft Rocks which was accepted by the new President, Dr. Xia-Ting Feng and the ISRM Board. The commission is being active, and information can be obtained in the ISRM site. It is expected that interested experts join the commission.

2 WHAT IS SOFT ROCK

The limits in strength of what can be considered soft rocks is somewhat variable according to various authors, but generally it is considered that their upper limit of the intact rock is about 25 MPa.

The lower limit, distinguishing from soil, is more questionable. Terzaghi & Pech (1967) consider that SPTs above 50 and a UCS (unconfined compression strength) above 0.4 MPa the material has more rock like characteristics than soil. Rocha (1975) considers as rock the material that does not crumble when immersed in water, and suggests the UCS of 2MPa for rocks. Dobereiner (1984, in Dobereiner, 1987) proposes an UCS of 0,5 MPa, which practically coincides with the value proposed by Rocha.

Another criteria proposed to define the transition between soil and rock was presented by Baud & Gamblin (2011) by means of the limit pressure in pressuremeter testing, indicating values from 2 to 10 MPa depending on the elastic modulus to the limit pressure ratio, as reproduced in Figure 1.

The establishment of limits between soft rock and soil or hard rock is somewhat questionable. When verifying whether all rock types follow the theoretical relationship between porosity and dry

density, it was seen that there is a gradual transition of data from hard rock to soft rock and to soil, as shown by the plot of dry density vs. porosity in Figure 2 (Kanji & Galván 1998).

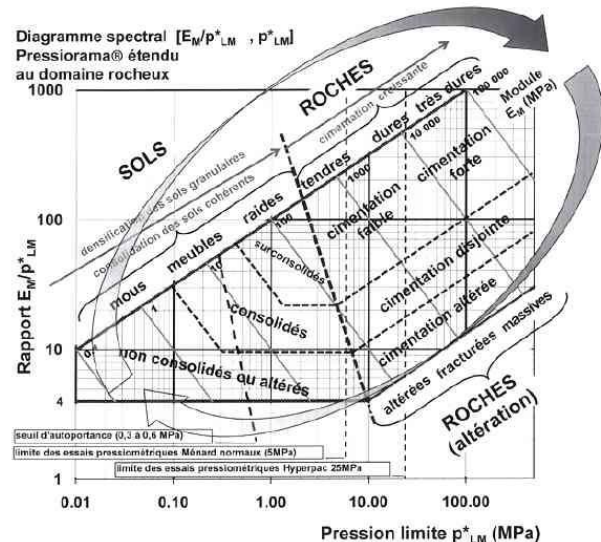


Figure 1. Pressuremeter Limit Pressure as a function of the Elastic Modulus to the Limit Pressure ratio (Baud & Gamblin, 2011).

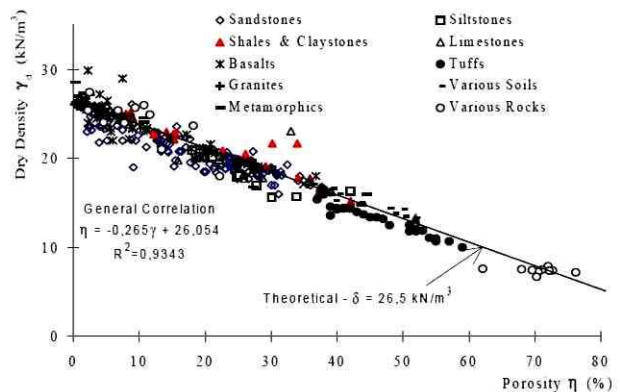


Figure 2. Plot of dry density vs porosity along the theoretical line for various rock types (Kanji & Galván, 1998)

It is important to mention that Deere & Vardé (1986) have also considered that rock masses can be considered weak not only by the low strength of the intact rock, but also rock masses of hard rock but containing structural features of low strength as discontinuities, voids, etc, making the whole mass weak. They have used the term "weak rock" for these cases.

In the same way, it has been accepted

nowadays that rock masses of harder intact rocks but that can deteriorate quickly becoming soft, or intensely jointed or at great depth subjected to high pressures may present high deformation and therefore will behave as soft rocks. This means that the concept of low strength intact rock is extended also to the rock mass. The examples given in this paper relate to both intact rock and weak rock mass.

The rock types that usually constitute intact soft rocks are mentioned in Table 1.

Table 1. Usual Soft Rock types.

<p>Sedimentary rocks: Clastic: mudstones, shales, silts, sandstones, conglomerates and beccias, marl. Evaporites: salt rock, carnalite, etc. Soluble: limestone, dolomite, gypsum. Coal.</p>
<p>Igneous rocks: Volcanic conglomerates, breccias and lahar, Piroclastic deposits, volcanic ash, tuff and ignimbrite. Basaltic breccia. Weathering products of crystalline rocks</p>
<p>Metamorphic rocks: Slate, Phyllite, Schists, Quartzite little cemented. Volcanic deposits.</p>

Some rock types usually referred as soft rock may be very hard according to their cementation degree, as for instance siltstones, sandstones and schists. Therefore it is not enough to mention the rock type, but also their cementation degree, among other characteristics.

Sedimentary rocks are affected by cementation and burial pressure along the time, and most usually older rocks are more resistant than younger ones, as shown by Bosio & Kanji (1998) in Figure 3 for sedimentary rocks from the Paraná Basin (Southern part of Brazil, East of Paraguay, Uruguay and North of Argentina), with rocks from Silurian-Ordovician age presenting higher strength than the ones from Jurassic, for example. It can be seen also that the porosity decreased with age, increasing UCS.

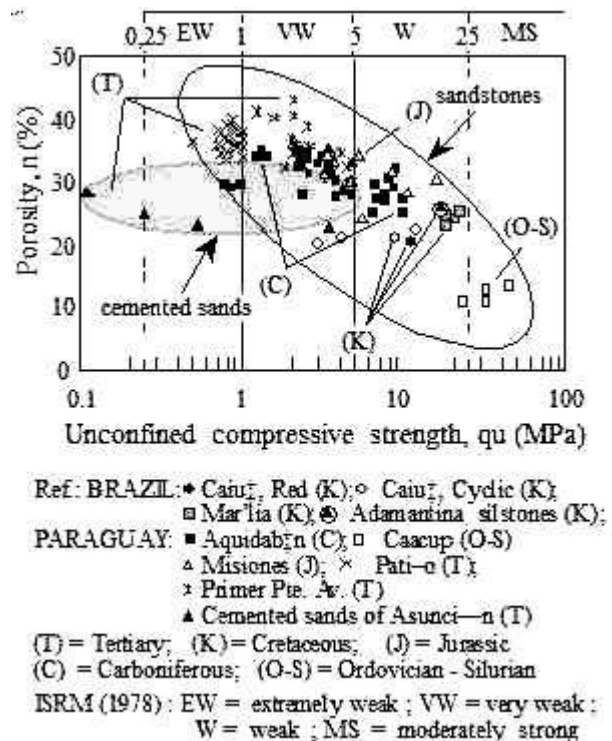


Figure 3. Graph of UCS vs Porosity for sedimentary rocks from Paraná Basin indicating their geologic age (Bosio & Kanji, 1998).

3 SOFT ROCK PROPERTIES

The characterization of soft rock properties has been made by several authors, with emphasis to Galván (1999), Kanji & Galván (1998) and Kanji (2011) who have correlated several physical and mechanical properties among them, showing sometimes very narrow correlations. As a result, it is suggested that some characteristics apparently are able to indicate preliminary the expected behavior of the rock, as the absorption, since it is closely correlated to the porosity, which in turn correlates to density and strength.

4 EXAMPLE OF DAM FOUNDATION PROBLEM OF SOFT ROCK.

The dam foundation must prevent problems related to stability, deformation and leakage, whatever is the type of dam. These requirements are even more important in the case of concrete

dams. The rock weakness affects mainly its sliding stability, depending on the rock strength and the position and orientation of the weaknesses.

In concrete gravity dams the most adverse condition is when horizontal or near horizontal are present in the foundation, impairing the safety with respect to sliding. The shallower the feature (weak layer or plane) the more important it is. This fact can be illustrated by the graph of Figure 4, relating the required friction angle for Safety Factor of 1, according to the foundation drainage condition (with data obtained from Cruz et al., 1978).

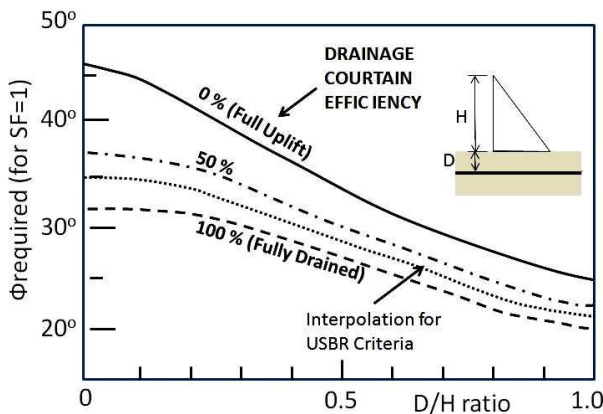


Figure 4. Necessary friction angles of a horizontal weak zone or plane at various depths within the foundation, for different drainage conditions

A typical example of weak layer affecting the foundation stability is that of the Castrovido Dam in Spain, mentioned by Alonso (2011), with siltstone intercalation in sandstones, as depicted in Figure 5, which obliged to changes in design due to the low strength of the siltstone.



Figure 5. Siltstone intercalations in sandstone, at the Castrovido dam foundation in Spain (Alonso, 2011)

The siltstone has IP of 10 to 12, and residual friction angles of about 15° to 18°. Coincidentally, this value coincides with the application of Kanji's (1974, 1998) expression

$$\phi_{res} = 46.6 / (IP)^{0.446} \quad (1)$$

Another impressive example of that of the Itiquira dam, Brazil, a low concrete gravity dam destined to hydroelectric generation by two power plants in a row. A few blocs at the right abutment have been built in advance. When excavating for the construction of blocks in the river bed a series of weathered siltstone weak layers were encountered, obliging to deepen the excavation, with extra cost and time delay.

Figure 6 show the highly adverse position of such layers with respect to the sliding stability of the dam, and Figure 7 depicts a detail of the weak bed within hard cemented sandstone.

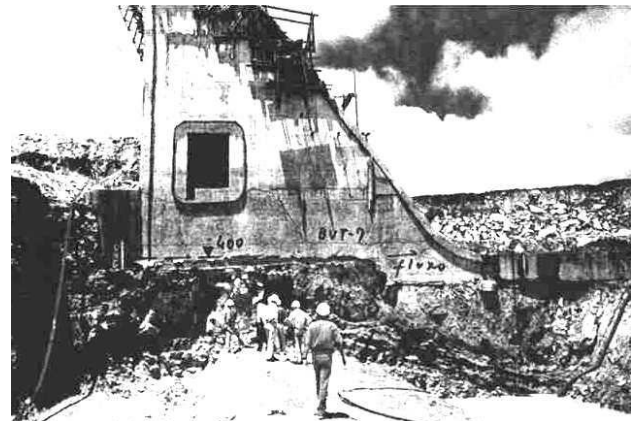


Figure 6. Soft siltstone layers in adverse position within cemented sandstone in the dam foundation.



Figure 7. Detail of the siltstone intercalations in the sandstone, corresponding to a weak layer.

It is worth mentioning that the geologic investigation made in the construction follow up detected correctly the weak layers and realized its horizontal continuity, as shown in the draft geologic cross section parallel to the dam axis presented in Figure 8, but this fact was overlooked by the contractor..

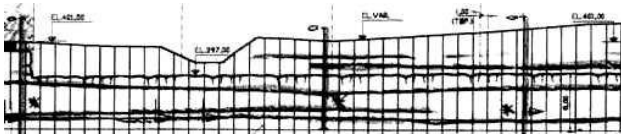


Figure 8. Geologic section along the dam axis, showing the continuous weak layers of siltstone.

5 EXAMPLE OF SLOPE FAILURE DUE TO SOFT ROCK AT THE BASE.

The South Panamerican Highway at Palpa, Peru, had a stretch with high declivity, and it was decided to build an alternate way more adequate to the traffic, with cuts up to 42m high. Since the rock is a medium cemented conglomerate, the cut was designed with inclinations of 3V:1H with benches each 20m (instead of the 7m interval established in the national code of practice). When the excavation was about 30 m deep a weak layer of volcanic tuff dipping gently was exposed as seen in Figure 9. Tension cracks were observed at the top and the slope failed due to the tuff weakness according to the sketch of Figure 10, deduced by stability analysis.

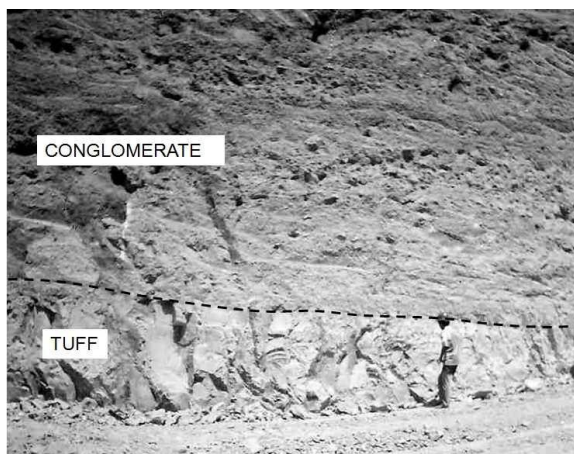


Figure 9. Picture of the failed slope showing the tuff at its foot and the conglomerate above.

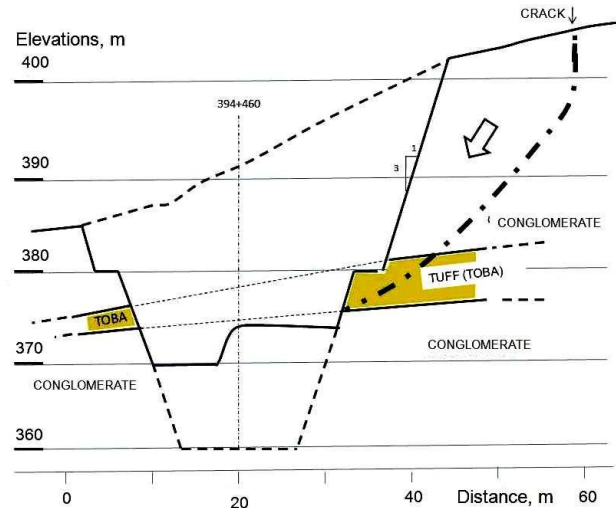


Figure 10. Sketch of the slope excavation, the tuff layer, the tension crack above and the failure surface obtained in the stability analysis.

Retro-analysis showed that the tuff has parameters equivalent to 20 kN/m^2 for cohesion and friction angle of 20° . The disaggregated tuff classifies as MH, having LL of 36% and IP of 16%. The fines percentage is of 20% to 30% with a maximum diameter of 3 to 4 cm.

The slope was not subjected previously to adequate investigation, and did not count with a rotary drilling for its entire depth. The design relied entirely on the conglomerate characteristics, and did not detect the tuff which caused the failure.

5 EXAMPLE OF THE COLLAPSE OF AN UNDERGROUND LARGE EXCAVATION.

The Pinheiros Station of Line 4 of the São Paulo Metro had an underground excavation of large size: 132 m long, 18 m wide, 15 m high, with an overburden of 20 m bellow surface, in an urban area. In the middle part it had a 40 m diameter shaft, from which the underground excavation was done in 3 levels: crown, middle bench and lower bench with invert.

The ground consisted of gneiss with vertical foliation practically parallel to the tunnel axis. The central part was in rock class III and both sides were in rock class IV according to the Bieniawski's (1989) classification. The

excavation of the tunnel crown was made with forepoling and steel lattice girders every 1 m and 0.5 m thick steel reinforced shotcrete. The case was studied jointly by Dr. David Hight and this Author of the Insurance companies.

The convergence of the North part of the tunnel was stabilized in the order of only 5mm during about two months. Just before Christmas of 2006, during the excavation of the middle bench, a slight increase of a few millimeters was noted, but the works was interrupted for the Christmas vacation, when the movements stopped, as the excavation was also stopped. When the excavation was resumed, the movements reactivated in one week with an increase of 5mm to 15mm in different stations.

Soon after the movement reactivation was noticed, the contractor called the designer, who determined the immediate reinforcement of the tunnel walls, with 3 lines of systematic 3m long rock bolts in both sides, totaling about 350 rock bolts. Figure 11 show the aspect of the North part of the tunnel, in total normality, without any sign of instability in the day of the designer inspection.



Figure 11. Aspect of the Northern part of the tunnel station the day before its collapse, with no fissures or any sign of instability problem.

The rock bolts installation started the same day and the next day about 150 ones had been already installed, when the works in the tunnel continued normally without any sign of problem. However, right after noon, the workers noticed a fissure in the shotcrete of the roof, followed some 2 minutes later by the fall of a small shotcrete slab. Office at the site was informed by

radio. The workers in the tunnel started to climb up the exit by the shaft. Some 2 minutes later cracks at the surface 20m above were seen, and some truck drivers pushed away their parked vehicles out of the area, and one worker run to the neighbor highway to stop the car traffic. A few instants later the whole area collapsed affecting half the shaft and half the tunnel at the North side of the shaft.

Figure 12 is a time table minute by minute showing the series of events, showing that since the first crack to the collapse, only some 6 minutes have elapsed. In the collapse unfortunately a microbus with passengers was dragged down causing 6 deaths. The view of the collapse is presented in Figure 13.

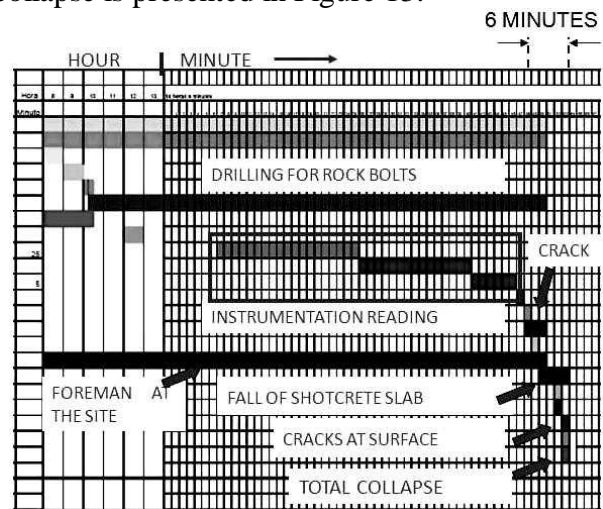


Figure 12. Chronology of the sequence of events leading to the collapse, happened in only 6 minutes (CVA, 2008, modified).



Figure 13. Air view of the collapsed tunnel and shaft.

It is impressive how so little deformation has happened prior to the collapse. The convergence

in the 18m diameter tunnel was limited to some 3cm, as can be seen in the graphs of Figure 14 (deformed slightly to fit the time scale). The settlement on surface above the tunnel was also limited to about 3,5 cm, which is striking for a tunnel with a 20m overburden, as shown in Figure 15.

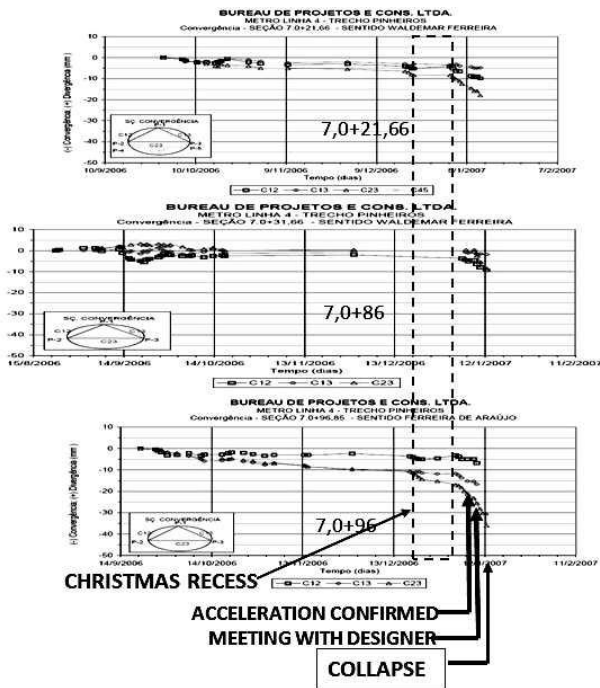


Figure 14. Convergence measurements of the tunnel walls since beginning of excavation up to the collapse.

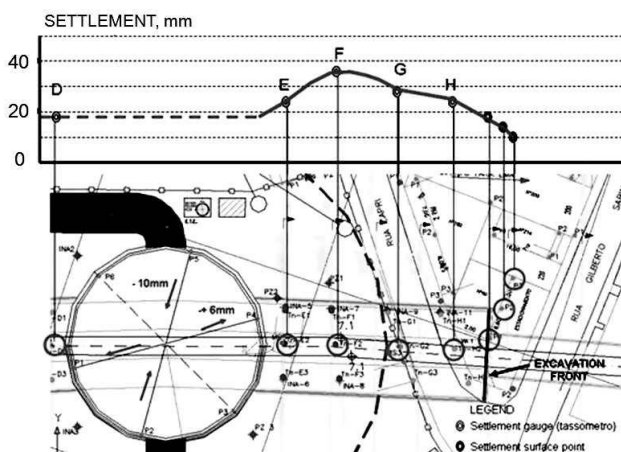


Figure 15. Surface settlements above tunnel just before the collapse, limited to about 3.5cm.

The cause of collapse was understood after the careful removal of the muck was made,

exposing the rock and the rock floor. The gneiss had vertical layers of weathered biotite schist very weak, in thin bands of one or two decimeters thick, as the example presented in Figure 16. One of these bands was almost parallel to the left wall and as the excavation proceeded the layer became closer and closer to the excavation wall, as shown by the geologic mapping made by the contractor (CVA, 2008) shown in Figure 17.



Figure 16. Vertical layer of weathered biotite schist (in the middle of the photo) within crystalline gneiss.

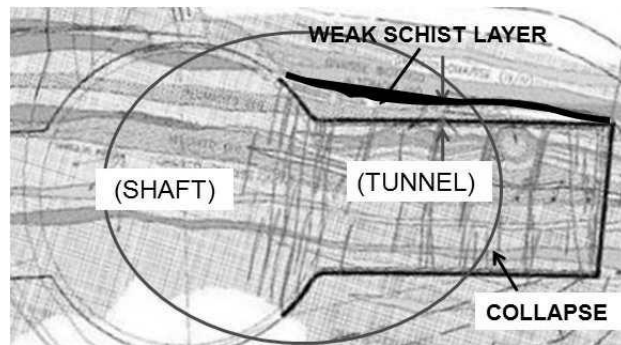


Figure 17. Geologic mapping of the excavation floor after muck removal, showing the vertical weak layer at short distance of the left wall. (CVA, 2008, modified)

The vertical weak layer prevented stress transfer beyond it, and the thin rock wall (of about 1m or so) received all the vertical load of the arching of the roof, overcoming the thin rock wall strength. The tunnel floor had a weathered meta basic rock which helped the failure, bringing down all the overburden, as shown in the sketch prepared by the contractor consultants in Figure 18. Additionally a somewhat weathered

transverse continuous joint at the end limit of the collapse was exposed after the event, showing that is acted as a sliding plane.

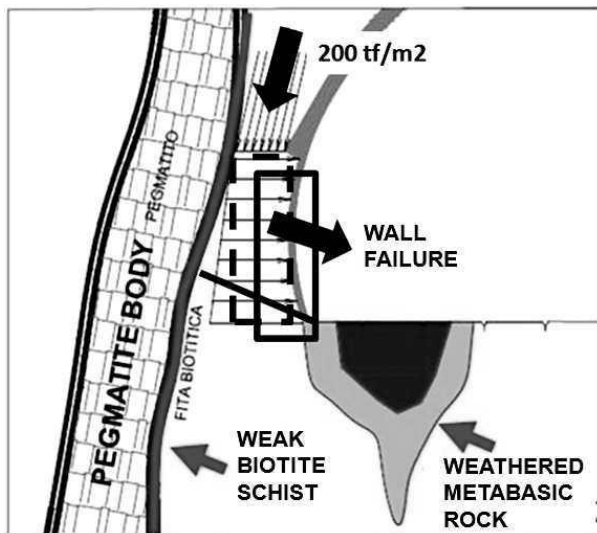


Figure 18. Interpreted mechanism of failure leading to the collapse of the station (CVA, 2008, modified)

The existence of the weak layer at the wall and of the crossing rock discontinuity at the end of the collapsed mass, besides the little displacements registered in the monitoring lead to the conclusion that the event was mainly controlled by discontinuities.

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