# In situ fracturing mechanics stress measurements to improve underground quarry stability analyses

Anna M. Ferrero, Maria R. Migliazza, Andrea Segalini University of Parma, Italy

Gian P. Giani *University of Milan, Italy* 

ABSTRACT: Stability condition of underground excavations have to be assessed to guarantee safe operations within the quarrying working areas. This objective requires a reliable assessment of the rock mass natural state of stress and of the stress redistribution induced by the excavations. This goal is often complicated by difficult geological conditions induced by tectonic stresses, complex topographies and large and irregular excavation geometries. Consequently, the acting state of stress needs to be measured by specific in situ measurement devices. This work refers on an in situ measurements campaign carried out by CSIRO tests in an underground quarry located in the Carrara marble basin. The difficult path carried out for interpretation of the measurements is described in this paper. In particular, the numerical modeling performed using the Boundary Element Method (BEM) first, to study the influence of both surface topography and excavation geometry and then the Distinct Element Method (DEM) to evaluate the influence of rock discontinuities in the measurement zone are illustrated in this work. Parametrical analysis of the rock mass mechanical features is also described in order to calibrate the numerical models and analyze the experimental results.

The work has been carried out trough several phases, which are:

• characterization of the rock mass through geostructural surveys: this allowed us to classify the rock mass and determine the mechanical and physical parameters necessary for modeling;

• on-site measurement of stress levels through the application of overcoring techniques;

• numerical analyses for the simulation of excavation phases, using the Examine3D (BEM) engineering analysis code.

• comparison between the measured stress levels and those numerically calculated and subsequent calibration of the BEM model;

• Numerical model of the forecasted excavation based on the calibrated (DEM) model.

# 1 QUARRY SITES DESCRIPTION

This studied area is located in two adjacent underground quarries namely Ravaccione and Fantiscritti as shown in where the in situ tests indication is also reported. The figure shows the complex excavation geometry and in the BEM model is reported. The geomechanical survey, carried out in the chamber that borders the old access gallery on the west side, was realized following the ISRM recommendations (1978); several scanlines were surveyed by recording all the visible discontinuities on the exposed walls. The parameters recorded for each discontinuity are those suggested for the RMR or GSI (Bieniawski Z.T. 1989) classification of the rock mass, in particular: orientation, spacing, persistence, roughness, uniaxial compressive strength, filling and hydraulic conditions. From the survey results, the rock mass structure appeared constituted by different kind of rock masses: homogeneous and compact, foulted zones where two primary discontinuity systems are present that are already known for the whole Carrara basin.



Figure 1: Quarries plan with the localization of the stress measurements



Figure 2: Global view of the underground excavations



Figure 3: Stereogram of the surveyed poles and orientation of the main systems.

The RMR overall value obtained from the survey equals 61, by evidencing a rock mass classifiable between the third and the second class (between fair and good). The GSI value, derived from the RMR, equals 56. Using the Hoek & Brown criteria, the mechanical features of the rock mass were then determined considering the uniaxial compressive strength of the intact rock equal to 99 MPa and the value of  $m_i$  coefficient equal to 9, as suggested by the authors for the marble. The application of the criteria gave the results of. The Poisson coefficient was taken as 0.25.

Table 1 :Results obtained using the Hoek & Brown criteria within the surveyed rock mass.

Compressive strength	8.4 MPa
Tensile strength	0.4 MPa
Cohesion	5 MPa
Friction Angle	48°
Elastic Modulus	14055 MPa

#### 2 DESCRIPTION OF THE EQUIPMENT USED FOR THE CSIRO TESTS

The on site tests were carried out using standard CSIRO cells (Dunnicliff J. 1993). These instruments are triaxial cells equipped with a thermocouple for the correction of the measured strains with respect to temperature variations. In order to access the on site state of stress of the rock mass surrounding the quarries using the overcoring technique, five horizontal borehole where drilled and nine tests were carried out inside those boreholes at various depths.

Five of the nine tests were performed inside three pillars (all of them located inside the Fantiscritti quarry) while the remaining four tests were performed on the border walls of the excavation chambers.

In Table 2 is reported the summary of those tests, indicating their naming and depth of execution from the borehole head.

Hole – test	Depth (m)
1 01	4.27
1 02	7.06
2 01	6.90
2 02	9.65
3 01	4.57
4 01	3.60
4 02	7.45
5 01	5.00
5 02	5.50

Table 2: Summary of the performed CSIRO tests.

The theoretical concept on which those tests are based is the measurement of the strain induced by the stress relaxation of the rock mass. In order to accomplish this objective, a portion of the rock is cut free from the whole mass, removing it from the active field of stress while measuring its response in terms of strains. Duncan et al. (1980) proposed a relationship between the recorder strains and the field of stress acting on an isotropic media; such relation, applied to the data obtained from the CSIRO cells, gives the results in terms of main stresses reported in Table 3. In this table are indicated, for each test, the main stress tensor components in terms of module (MPa), dip direction and dips (deg).

# 3 BEM MODELS

The purpose of 3D BEM modelling using the Examine code is to provide a preliminary assessment of the mechanical behaviour in the exploited rock mass due to the evolution of the excavation phases for the three experimental room and pillar panels. These analyses are based on the data gathered from classification schemes and assuming a natural stress state of a lithostatic type, with a  $K_0$  vale equal 0,33. The modelled geometry and exploitation of the experimental panel were determined by an accurate geometrical survey.

	σ1	dip		σ2	dip	dd	σ3	dip	dd
Nome	(MPa)	(°)	dd (°)	(MPa)	(°)	(°)	(MPa)	(°)	(°)
1 01	17.4	53	172	4.3	27	40	2.2	23	-63
1 02	10.8	46	170	5.2	41	16	3.7	13	-86
2 01	16.5	79	-118	1.3	10	86	0.5	4	-5
2 02	16.5	81	-67	2.2	9	117	0.6	1	27
03 01	13.4	62	-68	5.7	27	128	4.1	7	37
04 01	21.7	57	98	8.1	28	-117	4.5	16	-18
04 02	14.5	59	147	3.9	0	-56	1.8	31	34
05 01	15.08	66	40	2.6	24	-146	1.3	2.2	-55
05 02	16.82	63	45	3.8	25	-157	0.5	8.7	-63

Table 3: On site main stresses (module, dip direction, dips) for each CSIRO test.

The rock mass deformability and strength of the equivalent continuum were derived from RMR and GSI schemes and using the widely accepted empirical Hoek & Brown strength model (Hoek E. 1994). The parameters are summarised in Table 1. Figure 4 shows the model reconstruction of the topographic surface. The modelling results, in terms of stress distribution, in the natural supporting structures (pillar and walls) are shown in Figure 5. Experimental values of measured principal stresses are also reported in Figure 6. The numerical values seams to be in good agreement with the experimental values for the rock mass 5 m far from the rock wall.



Figure 4 : Modeling schemes of the whole underground quarry





Figure 5: Stress distribution in the natural supporting structures (pillar and walls).

FORO 3D 01 cond. Anisotropa piano orizzontale



Figure 6: Computed main stresses obtained from one of the tests.

## 4 DEM MODELS

The objective of 3D DEM (Cundall P.A.. 1971) numerical modelling, using 3DEC code Itasca Consulting Group, 1988), is to assess the mechanical behaviour of the experimental room and pillar panel, explicitly taking into account the blocky structure of the rock mass. The model of the stope shown in Figure 7 is constituted by deformable blocks. In order to obtain the highest reliability for the geometrical model, in the areas lacking of survey data were performed statistical analysis of discontinuity orientation and spacing.

Successively, using the statistical option of the Resoblok (Héliot D., 1988) code, a 3D model reproducing the whole rock mass involved in the excavation was generated. After the model was built, in order to verify its geometrical reliability, several section containing the surveyed scanlines were created. This allowed for a comparison between the numerical model geometry and the surveyed rock mass. The analysis of the discontinuous numerical model was performed by means of the numerical code 3DEC. The numerical model geometry created with the Resoblok code was directly used by 3DEC code and, therefore, the rock mass area affected by the quarry activity was accurately reproduced in terms of orientation, spacing and position of the discontinuities, subdividing the rock mass volume into blocks of rock of finite dimensions. The resulting geometrical model had the total dimension of 170 x 130 x 80 m.

The rock blocks in the numerical model were simulated as deformable; this hypothesis is the most appropriate considering the depth of the quarry (500 m on average) that implicates an induced state of stress able to produce an elasto-plastic behaviour in the rock matrix near to the rock walls. The constitutive model chosen for the rock matrix is isotropic and elasto-plastic, with a Mohr-Coulomb failure criteria. The numerical values of the physical and mechanical parameters for matrix and joints, as well as the load conditions of the model are reported in Table 4 and Table 5.

The step sequence that was followed to set up and run DEM analyses for each site consisted of: • making the blocky rock mass structure using DEM input stream generated by RESOB-

LOCK (joint appearance along drift faces was used to constrain geometrical modelling of the joint network);

- tailoring the layout of the experimental panel geometry to the blocky model;
- mechanical parameters and in situ state of stress (of lithostatic type) assignment of the jointed rock mass taken by the BEM model above reported.



Figure 7: 3DEC model of the quarry portion

Tuble 1: Rock muss fourties uppried in the numerical modelning									
Unit	Compression	Et	Es	$\nu_t$	$\nu_{s}$	с	φ		
Weight	Strength								
kN/m3	Co [MPa]	MPa	MPa	-	-	MPa	[°]		
26.50	99.65	61140	39037	0.25	0.136	28	32		

Table 4: Rock mass	features ar	plied in the	numerical	modelling
				0

······································									
Joints Characterization							In S	itu Stress	
JKN	JKS	JRC	ф <sub>b</sub>	¢	$\sigma_{ m cf}$	xx-stress	уу-	ZZ-	G
							stress	stress	
MPa	MPa		[°]	[°]	MPa	MPa	MPa	MPa	m/s2
40'000	19'000	12.6	32.30	45	11.2*	-4.3	-11.7	-4.3	9.81

Table 5	Ioint	features	and	applied	stress
1 4010 5.	Joint	reatures	unu	applied	50,055

The applied lithostatic load (11.69 MPa) was distributed over the upper border of the model. On the lateral faces of the model there were no applied loads but constrain conditions (8 in total) were applied in order to give total stiffness to the four sides of the model with respect to horizontal movements and complete immobility to the lower boundary.

Examining these results it can be observed (Figure 8 and

Figure 9) the increase of the deviatoric stress around the excavation walls; the doubling of the vertical stress in the pillar; the rise of tensile stress on the excavation roof, due to the anisotropy of the initial state of stress; small block sliding along the joints within an area of 10 m from the excavation walls; an undisturbed stress condition at distances larger than 20 m from the void sides; the return of the principal state of stress directions at the condition they had before the excavation (undisturbed model) at a distance of 20 m from the excavation walls. Sudden increasing of radial stresses can be observed in correspondence of discontinuity presence as shown Figure 8.



Figure 8: Pricipal stress distribution computed by DEM model

# 5 CONCLUSIONS

In recent years underground mining of ornamental stone has become fairly widespread, giving rise to a series of problems related to mining techniques. It must be kept in mind that the principal aim of underground mining is the extraction of large volumes of material, while maintaining safety conditions. Such demands have led to various studies, aimed at solving the problem of sustaining underground extraction chambers.



Figure 9: Principal stress in the stope wall

Our testing efforts have afforded us the opportunity to appreciate the enormous potential of the latest numerical modelling techniques applicable to the sector described underling, however the necessity to calibrate the model with in situ measurements.

The creation of a three dimensional model of the quarry, allowed us to simulate stress levels with an assessment of parameters involved, which led us to our first conclusions regarding the stability of the excavation tunnel walls. This calibration procedure is possible following a back analysis based on on-site stress level measurements through the application of CSIRO testing method. The distance between the stress values measured on-site and those calculated numerically during an initial phase of simulation showed the fundamental importance of the "calibration" phase by including also the  $k_0$  assessment. The model thus obtained was a fair approximation of the real situation and laid the basis for the simulation of further phases of excavation.

# 6 REFERENCES

- BRADY B.H.G. AND E.T. BROWN. 1985. *Rock Mechanics for Underground Mining*. 1<sup>st</sup> ed. London: George Allen & Unwin.
- BIENIAWSKI Z.T. 1989. Engineering rock mass classification, Ed. John Wiley & Sons, New York. 251pp.
- CUNDALL P.A. 1971. The measurement and analysis of accelerations in rock slopes. Ph.D. Thesis, Imperial College of Science and Technology. 182 pp.

HOEK E. AND E.T. BROWN. 1982. Underground Excavations in Rock. Ed. London: The Institution of Mining and Metallurgy.

- DUNNICLIFF J. 1993. *Geotechnical instrumentation for monitoring field performance*. Ed: Wiley & Sons.
- *HÉLIOT D.*. 1988. Generating a blocky rock mass. *International Journal of Rock Mechanics* and Mining Science Vol. 25, n° 3. 127-138.

HOEK E. 1994. Strength of rock and rock masses. ISRM News Journal 2(2), 4-16.

ITASCA Consulting Group, Minnepolis, Minnesota, 3DEC, ver. 2, 1998