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ABSTRACT

On 28th October 2004 a rock failure occurred in a limestone open pit quarry in Tuscany (Italy). The phenomenon evolved slowly with the collapse of about 1000 cubic meters. The complex failure mechanism mainly involved plane sliding along stratifications with rotation and block toppling. The phenomenon continued to evolve slowly and on 3rd November failure extended to the neighbouring portion of rock with a total collapse of about 5000 cubic meters of rock. Failure analysis took advantage of an accurate characterization of geology, geometry and mechanical properties of the collapsed rock mass. Such analysis led to the conclusion that the failure has been caused by the concurrence of various factors, namely:

- unfavourable dip and dip directions of stratification which daylight in the slope face;
- possible reduction of the angle of shear resistance available along stratification especially as a consequence of intense rainfall and vibrations.

INTRODUCTION

The open pit quarry, under consideration, is located in the Lucca district (Tuscany - Italy). The exploitation of limestone shaped the quarry face as an amphitheatre, with height of about 190 m from the quarry crest to the quarry yard, and roughly developing E-W for a length of about 450 m along the Lodovica district road, which runs along the Serchio river and at the level of the quarry floor (Fig. 1). The rock slope failure occurred in the W sector of the quarry (Fig. 1), where a new access road was under construction, in two separate occasions (October/28/2004 & November/3-4/2004). Both episodes originated at the same elevation (170 m.s.l..m). The phenomenon evolved slowly with the collapse of about 1000 cubic meters on 28th October. The complex failure mechanism mainly involved plane sliding along a 20° dipping stratification with rotation and block fall/toppling. The phenomenon continued to evolve slowly and on 3rd November failure extended to the neighbouring portion of rock with a total collapse of about 5000 cubic meters of rock. The failed rock slid down and spread into two different directions to the quarry yard (elevation 60 m. s.l.m.), covering part of the quarry slope immediately below the failed rock face and obstructing a hairpin bend in the access road. The toe of the failure is located at an elevation of 150 m. s.l.m. in correspondence to the 20° dipping stratification while the crest of the scar is well marked, over a length of about 70 m, by a subvertical tension crack and by the collapse of the access road at 170 m elevation. Figures 2 & 3 show pictures of the same portion of quarry respectively before and after the failure. The characteristic features are identified in both pictures: the same sub - vertical joint indicated as JJ in both pictures, the same block indicated as B and the access road which in Fig. 3 is interrupted.

Fig. 1 View of the quarry site

The risk of rock fall interesting the Lodovica district road has been analysed by Casagli (2004) and will be not reported in this paper.

The rock slope failure had two main consequences: 1) mining exploitation was definitively stopped; and 2) a diversion road (Fig. 1) was constructed and kept in use until the west portion of the quarry was put under safe conditions by means of appropriate works not described in this paper which mainly concentrates on the analysis of triggering factors (rainfall and vibrations).

Such an analysis of the rock failure took advantage of a huge and accurate set of data. More specifically, the following data were available:

Fig. 2 Picture taken on 25th June 2003 of the collapsed rock mass before the failure: JJ sub-vertical joint, B Block, S 20° dipping stratification)

Fig. 3 Picture taken on 22nd November 2004 of the collapsed zone after the failure: JJ sub-vertical joint, B Block

- topography of the study area before and after failure, including pictures of the collapsed portion of rock before and after the failure;
- mechanical characterization of intact rock and of discontinuities;
- dip and dip direction of discontinuities for the whole open pit mine and for the local portion of rock mass subjected to collapse;
- quarry - blasting characteristics;
- accelerations measured in a nearby village (500 m far from production area) in occasion of quarry - blasting since 1997;
- accelerations and particle velocities measured at short distances from the blasting area during both production activities and in occasion of blasting for slope stabilization (reduction of slope inclination);
- historical series of rain intensity over the last 10 years;
- local geology, hydrologic conditions and geomorphology of the area;
- seismic site accelerations, predicted for the period August-November 2004 by means of appropriate attenuation laws from Magnitude and epicentre-site distance.

In order to find a rationale explanation of the fact that the collapsed portion of slope remained stable for more than one year it is necessary to take into account not only the predisposing factors (mainly geostructure and mechanical characteristics) but also triggering factors and to perform appropriate analyses. As for the triggering factors only intense rainfall and blasting vibrations have been considered. Seismic activity, in the period of interest, has not induced important acceleration at the site. The highest PGA at the site was estimated on the order of 0.016g for two different events occurred on 27th August 2004 (Ms = 3.7, d = 37.4 km) and 25th October 2007 (Ms = 2.2, d = 4.16 km). An empirical correlation by Frisenda et al. (2005), developed for the specific area under consideration and for small Magnitude earthquakes, have been used. On the contrary, the average annual rainfall is about 1500 mm, concentrated in the September - May period. It can be observed that a relatively high rainfall level was recorded between the end of October and the beginning of November 2004 when the failure occurred (Fig. 4). Following quarry blasts (350-400 kg of explosives are charged in the blast-holes), accelerations of modest entity (1-12 mm/s^2) were measured on buildings in a village (Aquileia) located at a distance of ~500 m, while peak velocities of ~120 mm/s were recorded near the blasted benches. The repeated vibratory action due to the quarry blasts could have triggered the failure, or cyclically induced a decay of the joint strength. It is worthwhile to remember that from 30th September to 25th October 2004 (2 days before the collapse) 10 quarry blasts have been done at distances progressively closer to the failure zone (the shortest distance between blasting area and failure zone ranged from 85 to 15 m). Before October 2004, usually, only 2 to 3 blasts per month were performed.

The following analyses have been performed:

- stereonet projections to check the potential failure mechanisms (Markland's test, Goodman 1980, Hoek & Bray 1981, RocScience 2002);
- analysis of potential toppling failure (Hoek & Bray 1981, Goodman and Shi 1985);

- static and pseudo-static analyses by the limit equilibrium method (LEM);
- pseudo-dynamic (Newmark 1965) analyses;
- static and dynamic analyses performed by means of Distinct Element Method (DEM) using UDEC (Itasca 2000).

Part of the above indicated analyses (which were performed considering the observed variability of structural data and joint strength) have already been published by Cravero, Iabichino & Lo Presti (2007). This paper concentrates on pseudo – dynamic analyses (Newmark 1965) which have been performed using, as input, the accelerograms recorded at distances between about 30 and 70 m from the blasts. The above distances are the closest ones between blast area and 3D accelerometers installed for vibration monitoring during activities undertaken to re-profiling the slope after the collapse. Nonetheless the quantity of explosives charged in blast holes during production activities were about twice those employed for re-profiling purposes the effects observed in Aquileia were almost identical. Therefore the accelerograms, which have been recorded during re- profiling blasts, were considered representative of vibration history during normal production activities without any scaling procedure.

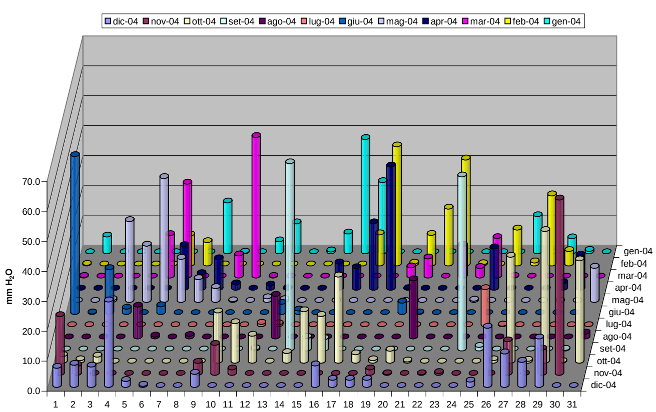

Fig. 4: Rainfall recorded at the Piaggione pluviometer, located near the quarry site (year 2004)

GEOLOGICAL AND GEOSTRUCTURAL SETTING

The rock formations of the low Serchio river valley belong to the non metamorphic Tuscany succession, which, at the quarry location, mainly consists of the "Maiolica" limestone and the "Scaglia Rossa" (i.e. varicolour claystone, with sporadic calcarenite and siliceous limestone beds) formations. The "Maiolica" limestone, which makes up the exploited ore body, derives from the diagenesis of very small calcite grains, it is hard and, when broken, gives rise to smooth fractures of a concoidal type. The limestone beds, clearly recognizable along the quarry face, vary from a few centimeters to 3-4m in thickness and locally show thin interbeds of marl, argillite or selciferous. When weathered, the softer interbeds change into clayey materials. Based on the joint structure and joint condition, the site was partitioned into three sectors that follow quarry face orientations: sector 1, covering the E and central quarry faces, where the beds dip into the slope face or are in a traverse slope condition; sector 2 (central to W quarry face) where the rock mass shows karstic features and a fold hinge abruptly changes the average attitude of the limestone beds to subvertical; sector 3, covering the W quarry face, where the beds dip out of the face and the rock mass appears jointed or karstified to a higher degree. Specific surveys and technical evaluations were successively made, but they were limited, to the quarry sector where the rock slope failure occurred.

A geological section in sector 3, is shown in Fig. 5. The local rock mass structure is characterized by the bedding joints S_t (107°/19°) and three principal joint systems: K_1 (348°/83°), K_2 (45°/82°), K_3 (74°/86°). Bedding joints(those joints developed on bedding surfaces) are highly persistent and represent the lowest angle set while the other three joint sets represent high angle or subvertical joints. All the joints subdivide the rock mass in almost straight dihedra prevailingly formed by the intersection of the K_2, and K_3 sets as the set S_t which makes up their base. The S_t surfaces appear smooth, slightly weathered and usually tight, sometimes showing silty or sandy infillings. According to the local bed thickness, their spacing is usually <0.5m, while the spacing of the other joints is generally ~1m. The surfaces of the K_2 set, which often shows a less than 1m linear size, are slightly rough and have wide separations. Their intersection with S_t makes the edge of the wedges align with the quarry slope. K_1, and K_3 sets are also non - persistent and show wide separations, however their surfaces appear oxidized and very weathered and karstic features can be seen. All these disjunctive features could have had an important role on the structural control of the rock mass and on shaping the scar of the rock slope failure.

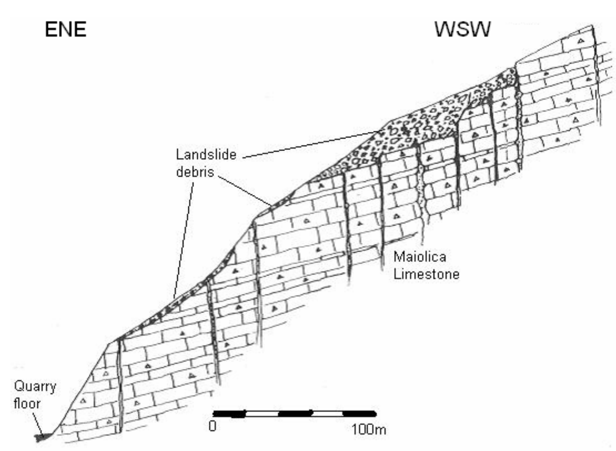

Fig. 5: Geological section in the sector 3, subject to the failure (Puccinelli, 2005)

GEOMECHANICAL CHARACTERIZATION OF THE ROCK AND ROCK JOINTS AND SHORT RESUME OF THE RESULTS OF PREVIOUS ANALYSES

A comprehensive laboratory experimental program has been carried out on both intact rock samples and natural discontinuities (13 prismatic specimens containing matched unfilled joints). In addition an in situ survey has been carried out in order to obtain geomechanical indexes of the stratification. Details of the in situ and laboratory experimental program are given by Cravero et al. (2007). Strength parameters according to Mohr Coulomb criterion and Barton (1976), Bandis et al (1981) criteria have been obtained from the experimental program test results (Cravero et al. 2007). In conclusion it was assumed, for the in situ bedding discontinuities a zero cohesion and an angle of shear resistance at peak of $\varphi_p = 37°$. In residual conditions an angle of $\varphi_r = 33°$ was assumed. Both values refer to a normal stress of about 1 MPa. However the experimentally determined strength of discontinuities (bedding) showed a very high variability. In order to take into account such a variability and possible degradation phenomena, due to the occurrence of joints with soft infilling permeated by water, Cravero et al. (2007) assumed an angle of shear resistance between 35 and 20°. The following type of analyses have been performed by Cravero et al. (2007): 1) kinematical and key block, 2) Limit Equilibrium Method (LEM) and 3) Distinct Element Method. (DEM)

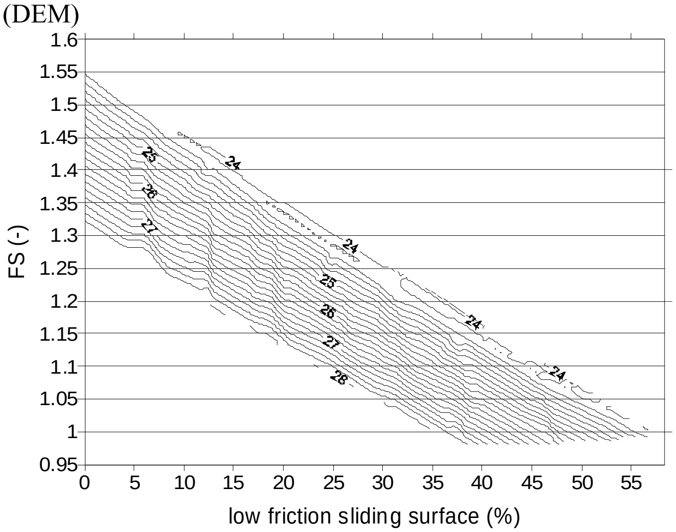

Fig. 6 Trend of FS versus percentage of the sliding surface having low φ_j for different α_{St}.

All the above mentioned types of analyses led to the conclusion that the rock failure becomes justified only assuming a degradation of the joint strength. It is especially instructive to see the safety factor FS obtained from LEM analyses as a function of bedding plane inclination (α_{ST}) and percentage of the bedding plane having low strength (Fig. 6). It is possible to notice that the safety factor becomes less than 1.0 when such a percentage is greater than 40 -55 % for α_{ST} = 24 – 28°

NEWMARK - ANALYSIS RESULTS

Three 3D SETAC (SEquoia Triaxial Acceleration Computer) accelerometers, located at different distances from the blast area have been used to measure acceleration time histories during the activities for the rock slope re- profiling. The X axis was oriented along the bedding plane. The maximum peak accelerations were recorded in the X direction (Table 1). Probably, the maximum amplitude is associated to surface waves. Figure 7 shows the time histories in the three different directions recorded by SETAC 1. Similar time histories have been observed for SETAC 2 and SETAC 3. The effective duration of time histories is in between 0.7-0.9 s, which appear white reasonable. Two well separated wave trains are observed in each wave trace. In some cases the second train contains the maximum amplitude. Frequency content of both trains is in the same band.

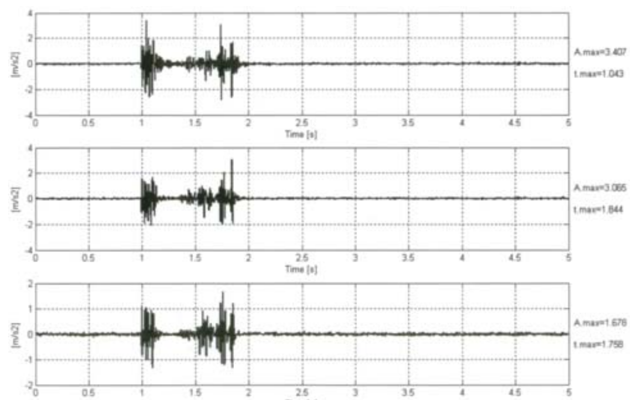

Fig. 7 Time histories (X, Y, Z) recorded by SETAC 1

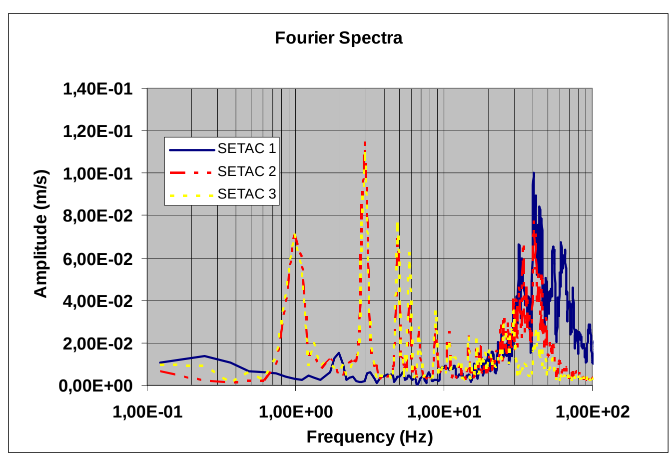

Fig. 8 Fourier Spectra X component of acceleration time histories recorded by SETAC 1, SETAC 2, SETAC 3.

Figure 8 shows the Fourier spectra of the X components, recorded at three different distances. It is possible to notice that for the accelerometer closest to the blast area (SETAC 1) the peak amplitude of Fourier spectrum is observed at around 50 Hz while such a peak moves towards lower frequencies for the other two accelerometers. The same result is obtained if the two wave trains, observed in the acceleration time histories, are analysed separately.

Table 1 summarizes the distances and observed peak accelerations at the three location in the three orthogonal directions (X, Y, Z). Table 2 shows the results of a parametric study.

Parametric analysis computed the accumulated displacement of an infinite slope according to the Newmark (1965) approach. The yield acceleration as been computed as

$$k_y = \tan(\varphi - \alpha_{ST}) \tag{1}$$

where:
α_{ST} = inclination of the bedding plane (20°); $\varphi =$ angle of shear resistance of the bedding discontinuities assumed variable between 35 and 21°.

Table 1 Peak acceleration components (X, Y, Z) measured at three different distances from the blast area

Accelerometer	**Distance (m) (*)**	**X (m/s^2)**	**Y (m/s^2)**	**Z (m/s^2)**
SETAC 1	29	3.407	3.065	1.678
SETAC 2	43	1.449	1.001	0.477
SETAC 3	69	0.555	0.531	0.458

(*) distance between accelerometer and the blast hole more NW oriented

Table 2 Parametric pseudo – dynamic analysis results (Newmark)

		SETAC 1	**SETAC 2**	**SETAC 3**	**(*)**
φ (°)	**k_y**	**Displ. (mm)**	**Displ. (mm)**	**Displ. (mm)**	**Displ. (mm)**
35	0.27	0.001	3.7	0.4	0.23 - 38
30	0.18	0.012	6.0	1.6	0.37 - 213
25	0.09	0.13	14	17	0.69 – 295
21	0.02	1.03	190.6	189.0	4.45 - 430

(*) Cravero et al. (2007) from DEM analysis

It is interesting to observe that, more importantly than the peak ground acceleration (PGA), the frequency content of the motion influences the accumulated displacement. More specifically, greater displacements have been computed for SETAC 2 and SETAC 3 accelerograms having smaller PGA and highest Fourier amplitudes in the low frequency band.

It is also interesting to notice that, for extremely low values of the angle of shear resistance, very large accumulated displacements are predicted as a consequence of a single blast.

Preta (2007) has re-analysed the direct shear test results on bedding discontinuities (Cravero et al. 2007). He observed that the passage from peak to residual strength occurs for displacements in between 3 and 10 mm. It has therefore been assumed that the passage from peak (φ_p = 37°) to residual (φ_r = 33°) occurs, in situ, when the accumulated displacement becomes greater than 6.0 mm (average value between 3 and 10 mm). The displacements have been computed using a MS worksheet developed by Salvadorini (2006) and Nardi (2006), which gives the possibility of reducing the shear strength parameters (i.e. reducing the yield acceleration) when the computed displacement becomes greater than e pre-fixed value.

Table 3 summarizes the computed displacements under the following hypotheses:

- inclination of the bedding plane α_{ST} = 31 – 20 °
- angle of shear resistance equal to $\varphi_p = \varphi_r$ = 37 – 33°
- angle of peak shear resistance φ_p = 37°, angle of residual shear resistance φ_r = 33°, the passage from peak to residual conditions occur when the displacement is greater than 6 mm.

It is possible to notice that with α_{ST} = 31 - 20° and strength degradation hypothesis, the accumulated displacement range from several mm to several cm for a single blast. It seems reasonable to believe that accumulation of displacements for repeated blasting (10 within 30 days before the rock failure) is one of the triggering factors. In fact, it is intuitive to understand that displacements of few cm in the rock mass under consideration, cause:

- tension cracks where water permeation becomes possible;
- degradation of the shear resistance (passage from peak to residual strength)

In conclusion, the Newmark - analysis results suggest that, more than one single blast, the accumulated effect of repeated blasts should be considered as one possible triggering factor. The rock mass which collapsed in autumn 2004 was apparently stable in June 2003 (Fig. 2). It is also worthwhile to remark that the first signs of rock instability (tension cracks in the access road) appeared in the afternoon of 27th October (the day before the first failure) in concomitance with intense rainfall (see Fig. 4).

Therefore the rock failure analysed in this paper has been probably triggered by repeated blast vibrations in concomitance with heavy rainfall.

Table 3 Pseudo – dynamic analysis results (Newmark) assuming constant and degrading strength parameters

		SETAC 1	**SETAC 2**	**SETAC 3**	
φ_p (°)	**φ_r (°)**	**Displ. (mm)**	**Displ. (mm)**	**Displ. (mm)**	α_{ST} (°)
37	33	25.4	78.1	62.7	31
37	37	15.1	10.3	8.9	31
33	33	29.8	119.8	122.5	31
37	33	7.6	3.0	0.2	20
37	37	7.3	3.0	0.2	20
33	33	9.2	4.6	0.7	20

CONCLUSIONS

The following conclusions can be drawn:

- the Newmark approach, even in its simplest form, is a very powerful tool to predict the order of magnitude of slope displacements due to a given acceleration time history;
- the knowledge of the acceleration time history is a key parameter in such type of analysis;
- as far as the acceleration time history is concerned, the frequency content appears in many circumstances more relevant than the peak acceleration itself;
- another key parameter is the yield acceleration k_y. This parameter includes both slope geometry and soil/rock strength. In the present study a simple geometry (infinite slope) has been considered but a possible degradation of joint strength has been introduced in a very simple way;
- Newmark analysis, as other types of analysis (kinematical, LEM, etc.), clearly show that the rock failure, documented in this paper, can be justified only considering a degradation of the joint strength;
- the most probable triggering factors appear to be blast vibrations and intense rainfall.

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