Effects of the Boumerdes earthquake of May 21st, 2003 on the great mosque of Dellys (Algeria)

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Effects of the Boumerdes earthquake of May 21st, 2003 on the great mosque of *Dellys* (Algeria)

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Abstract. May 21st, 2003 a great earthquake which the magnitude was estimated at MW 6,8 shook all the *wilaya* of Boumerdes causing enormous damage to constructions and great human losses. The historical nucleus of *Dellys* classified as a safeguarded sector being located in the stricken area also suffered from important damage. Many houses were destroyed and a great number were seriously damaged. The headlight Ben Gut, which dominates the bay of *Dellys*, was not saved; important cracks were observed on its walls. The great mosque of *Dellys* "*Djamaa' al kabir*" re-built from 1844 to 1847 is the most important monument located on the higher part of the old nucleus. This classified national heritage underwent large damage such as the torsion of the minaret, the detachment of the wall of the *qibla*, various cracks, among them, those observed on the arcades' key of the room of prayer, those marking the main facade and the minaret. Besides the accumulated impact of this recent earthquake, other factors in the past have increased its vulnerability. This paper presents various pathologies related to these seismic disorders as well as the solutions of repair and consolidation which were undertaken from September 2010 until May of 2013 within the framework of the project of restoration supported by the Algerian Ministry of Culture.

Keywords: " Earthquake"; "Historical building"; " Seismic damage"; " Repair"; " Consolidation".

1 INTRODUCTION

1.1 The *Boumerdes* earthquake

The *Boumerdes* Earthquake Ms=6.8 (EMSC), happened 21/05/2003, hit a 250 Km radius Algerian area, has caused 2300 victims, more than 11000 injured and destroyed 20800 housing units. The epicenter has been located off-shore, 9 Km of depth from the surface. The maximum peak ground acceleration recorded in Algeria at 20 km from the epicenter reached 0.58 g (Boulaouad and all 2010, Harbi and all 2006). The main shock also caused geological related phenomena such as liquefaction of some areas along the close rivers *Isser* and *Sebaou*, a soft tsunami (1.5 mt waves) felt from the Algerian shores until Balearic Islands and an uplift of seafloor of 40cm (Allasset and all 2006). The response spectrum highlighted that the most excited frequencies are medium – high (between 8 and 30 Hz), correspondingly with the most frequent damages found on medium height and rigid buildings (Laouami and all 2003). The macroseismic intensity was estimated between IX and X at *Dellys* (Harbi and all 2006).

1.2 Earthquake effects

The Casbah of *Dellys* is located within the stricken area at about 170 km east of Algiers. *Dellys* live today following the Boumerdes earthquake its third major earthquake (42 BC and 1631)

(Abdessemed-Foufa 2007). The latest one has caused a massive destruction of the historical nucleus. Beside serious damage to the Great Mosque of *Dellys* were observed. Following an investigation procedure for the diagnosis of masonry buildings of historic masonry (Binda and all 2000) major damage and disease were identified i.e. the torsion of the minaret, the detachment from 11 cm of the main facade from the parapet located on the opposite side of the minaret, detachment of the stairs of the minaret from the load-bearing walls while the central pillar completely broke Fig 1 a, b and c. Many deep and superficial cracks have appeared. The high seismic vulnerability of this type of building is due both to mechanical properties of masonry materials, characterized by a very small tensile strength, and in particular configuration (slender walls, lack of effective connection among the structural elements, ect (Lagomarsino 1998). Therefore the study of masonry heritage buildings remains a challenging task as many recent studies confirm (Lourenço and Ordūna 2007)

Figure 1. a) Deep cracks due to the torsion of minaret- b) Detachment of the main façade- c) Damage to stairs

2 DESCRIPTION OF THE MOSQUE

2.1 History of building and past interventions

There remain today no traces of the first *Dellys* mosque built during the medieval period. Originally located in the lower Casbah, the mosque was destroyed following the military engineering alignments performed between 1845 and 1895. To avoid the wrath of the local population, military engineering rebuilt its reply a little higher in the city between 1845 and 1847. We can in no way say that it is a replica but the date of its construction is indicated on the cadastral plan of 1845 Fig 2.

Figure 2. Mosque of *Dellys* past and present

2.2 Architectural and materials aspects

The mosque belongs to the Ifriqyen hypostyle style. The north-south length reaches 29.55 m while the width is 16.25 m. The minaret with a square base and a high of 22,96 m is located at the east corner. The mosque reaches the surface of 840 m^2. The prayer room occupies the entire ground floor. It is accessed through two large doors on either side of the *mihrab*. The hypostyle room has seven longitudinal spans perpendicular to the *qibla* wall and five cross spans. Piers are pointed arches supporting a steel floor made by metallic-I-beams and voutains of masonry brick. There is an intermediate floor that covers half of the prayer room called *sedda*, it is a gallery. Above the *mihrab* is built an octagonal dome rising over its square core. It is 3,20 m in diameter and 3,48 m high, and rests on an almost octagonal masonry chimney that takes its departure on the pillars at the start of the arch Fig 3a, b, c, d, e and f. Outside only the main facade and minaret which carry architectural elements related to the neo-Moorish architecture i.e. the openings are decorated with horseshoe arches called "*Arc Algérois*". On the façade of the minaret is also plated the "*Arc Algérois*" without opening Fig 4. From a structural point of view, the walls of the mosque are built with large dimension of unreinforced masonry (URM). The walls are built of stones bound with clay and lime mortar, reaching 0,80 m thick at the prayer room and 1,20 m at the minaret. The thickness of joint mortar is of 3cm approximately Fig 5. The pillars are of two dimensions overall 0,80x0,80 m and in the transverse side bays 0,80x0,90 m. The intermediate floor *sedda* is a reinforced concrete slab laid on the steel- I- beam. The roof terrace is a steel floor with brick masonry voutains.

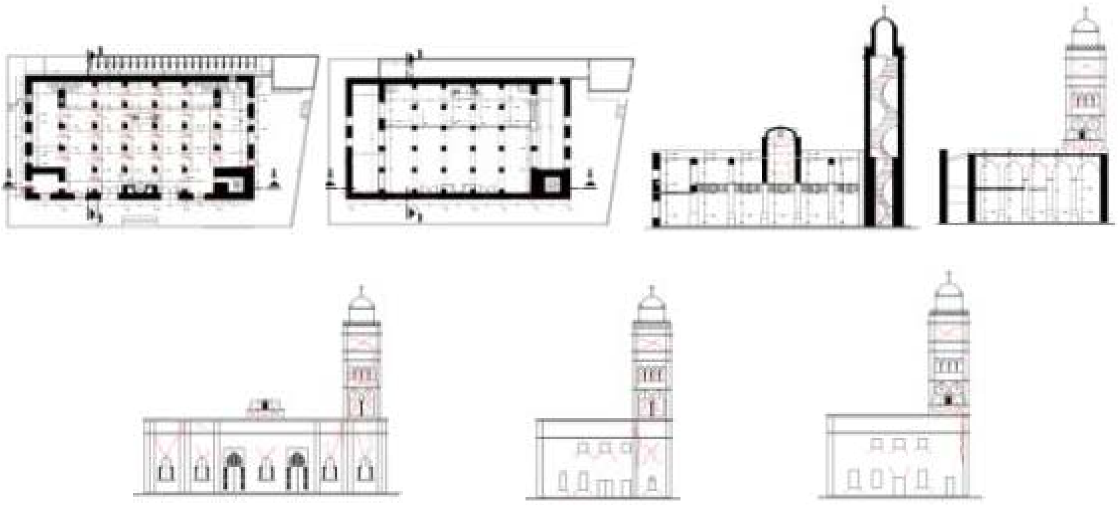

Figure 3. a**)** Plan of the ground floor- b**)** Plan op the gallery- c)) Longitudinal section A-A - d) Transversal section B-B- e) Main façade – f) East cross façade- g) West cross façade.

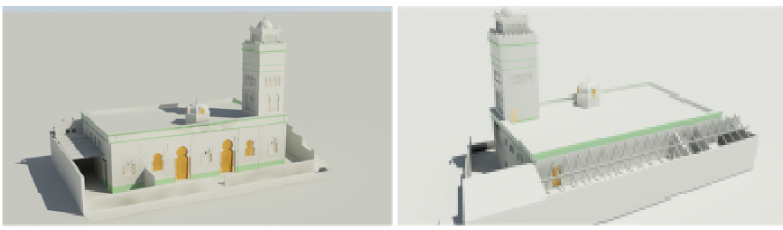

Figure 4. Architectural element "*Arc Algérois*" 3D restitution

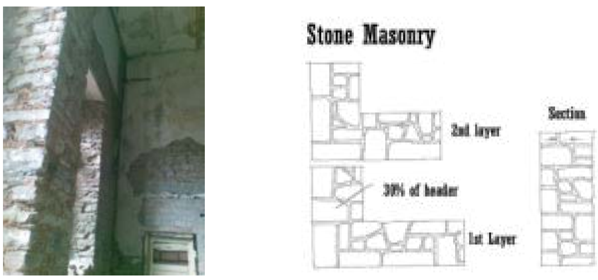

Figure 5. Stones masonry walls of the mosque

3 EARTHQUAKE DAMAGE

The Mosque suffered many damage which are mainly due to numerous conditions: eccentricity and difference of inertia of the main block and the minaret, the structural bending is due because the vertical stiffnesses are not symmetrical in the center of gravity, no connection of structural elements (orthogonal walls and floors), recurrent feature of traditional masonry construction, stiffness of the reinforced concrete *sedda* from the overall structure of stone masonry, damage due to seismic origins pathologies, such as x cracks on the facade walls and between the openings. However the damages were more extensive in the minaret and the main façade. Therefore we suggested different components: rotation and displacement in the outside direction of the minaret, shear stresses, in-plan and out-of-plan both for the main façade, shape of the activated kinematic chain of arcades and the deformation of the façade plan Fig 6 a, b, c and d.

Figure 6. Damage to (a) dome, (b) minaret and (c) arches of room prayer and (d) galery.

4 SRTUCTURAL ANALYSIS

Old masonry buildings cannot be classified as "mechanical controlled" (Giuffrè 1991); they have been conceived applying the traditional precepts of proportions. Historical buildings, especially monumental ones, often reveal uncertainties and execution anomalies due to their constructive histories that could imply a number of variables that are hard to quantify. Specifically, the particular shape of the structure of the Mosque (plan regularity but height irregularity) suggested the impossibility to perform an equivalent static analysis (non plausibility to transform the MDOF global system in an equivalent SDOF). In this case it is possible to perform a global verification with a Linear Dynamic Analysis with a FEM modeling that was done thanks to Straus7® software. In addition to the Spectral Analysis on the FEM model, in order to have a complete description of the structural behavior of the Mosque, both Linear and non-Linear Kinematic analyses were operated.

4.1 Linear and non-linear kinematic analysis for the out- of- plane collapse mechanism

Systematic analysis of the damage suffered by structural monuments during recent earthquakes has shown that the seismic behavior of this type of structures may be better interpreted through their decomposition into a number of architectural portions, i.e. macro-elements characterized by a structural response that can be considered independent from the global behavior of the building (i.e. partial collapse). If masonry shows good characteristics, local damage mechanisms develop as loss of equilibrium of portions capable of sliding and rotating. In the case of Dellys mosque, masonry generally behaved as a composition of rigid blocks. Local mechanisms of overturning are caused by out-of-plane actions in case of standing walls and by both out-of-plane and in-plane actions for arches systems. It is thus necessary to define different damage states: the mechanism activation (i.e. damage limit state and the corresponding acceleration threshold) and ultimate condition (i.e. the collapse limit state and its corresponding displacement capacity (Lagomarsino 2009).

The latest Algerian standards RPA03 (MUCH 2003) do not consider either verification and design of Unreinforced Masonry (URM) buildings and verification of existing and monumental buildings. So, it was decided to use the RPA03 for the construction of response spectrum and the modeling of load cases and verification criteria in global FEM analysis, together with Italian standards NTC08 (MIT 2008, MIT 2009, PCMD 2001) for calibration of interventions in relation with conservation needs. The Linear and non-Linear Kinematic Analysis here performed are approaches to the more general Limit Analysis theory applicable to masonries (Livesley 1978, Heyman 1966).

Considering a structure subjected to applied loads which are all multiplied by a load factor λ, the kinematic or upper bound theorem of plastic analysis gives the failure (cracking) load factor λ_p and the corresponding activating acceleration. The analysis is synthesized in the following steps, and the results are exposed in Table 1:

- For kinematic mechanism, main façade was subdivided in 3 macro-elements (B1; B2; B3) subjected to simple overturning around cylindrical hinges at the base of the building and two more blocks (CH4; CH5) under the minaret subjected to overturning in a 45°-direction respect to the façade-corner in accordance with the crack patterns Fig.7.
- Evaluating the kinematic multiplier α_0 in terms of displacements, and evaluation of the participating mass M* to the mechanism and the acceleration a_0* that activate the mechanism as follows:

$$\alpha_0\left(\sum_{i=1}^{n} P_i \cdot \delta_{xi}\right) = \sum_{i=1}^{n} P_i \cdot \delta_{yi} \qquad a_0^* = \frac{\alpha_0\left(\sum_{i=1}^{n} P_i \cdot \delta_{xi}\right)^2}{\left(\sum_{i=1}^{n} P_i \cdot \delta_{xi}^2\right)} \qquad M_0^* = \frac{\left(\sum_{i=1}^{n} P_i \cdot \delta_{xi}\right)^2}{g \cdot \left(\sum_{i=1}^{n} P_i \cdot \delta_{xi}^2\right)} \tag{1}$$

Considering that α_0: kinematic multiplier, P_i: i-th dead or live load, δ_{xi}: virtual horizontal displacement of the application point of the i-th load P_i (assuming positive the direction of the acceleration inducing the mechanism), δ_{yi} : virtual vertical displacement of the application point of the i-th load P_i (assuming positive the direction to the top), M_0^*: participating mass, a_0^*: activation acceleration, F_c: confidence factor – related to the knowledge level of the building

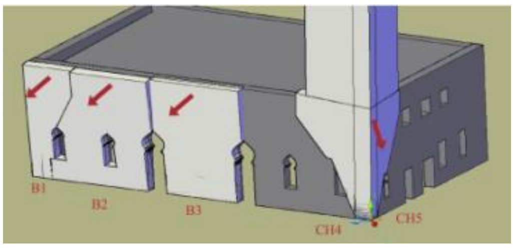

Figure 7. Macro-elements identification

The RPA03 locates the city of Dellys in the highest seismic hazard category (zone III); general monumental buildings are considered in group 1B and the resulting acceleration coefficient is A=0.3. The kind of soil of the area is classified as hard rock S1. The behavior coefficient, was assigned the lowest value D=2 (aiming higher security conditions), the quality factor 1.00≤Q≤1.35 has been chosen equal to 1.30 . T= CT $h^{3/4}$ = 0.05 • 8.23 $^{3/4}$ = 0.234s. The RPA03 response spectrum in continuous line highlights that the request acceleration is 0,488g = 4,78 ms^{-2}. Considering the absence, in the mentioned Standards, of any reference to Intervention Nominal Life and relative Return Period of the Seismic Action, it was adopted a series of principles that are illustrated in Italian Standards for Seismic Risk Mitigation on Cultural Heritage (PCMD 2011). It was decided to model an Intervention

Nominal Life of 40 years instead of 50, and consequently to foresee a return period of the action to be overcome in 475 years instead of 975 years. The reduced acceleration considered for verification is 0,3904=3,82ms2 as shown in Fig 8. The NTC08 verification condition for existing buildings is the following:

$$a_0^* \geq \frac{a_g(P_{VR})S}{q} = \frac{3.82}{2} = 1.91ms^{-2} \tag{2}$$

considering:
$a_g(P_{VR})$: request acceleration depending on limit state considered, ULS
S=1 soil factor – referring to actual characteristics analyzed in NTC08 standards
q=2 : structure kind factor referring to actual characteristics analyzed in NTC08 standards
The activating acceleration of the mechanism was compared with the acceleration request (Table 1).

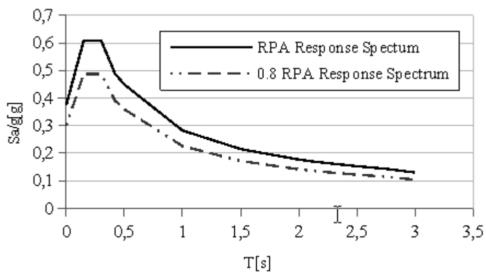

Figure 8. Spectral Acceleration Reduction

Table 1: Kinematic multiplier α0, Participating Mass M*, Mechanism Activation Acceleration a*0, Force Request Ti, Activation Acceleration for reinforced conditions a*r.. The index 1 or c refers to first floor slab and ceiling slab

ID block	*α_0*	*M**	*a^*_0 [ms^{--2}]*	*Ti [KN]*	*a^*_r [m/s^2]*
B1_1	0.061	36.63	0.52	50	2.09
B1_c	0.153	25.220	1.186	20	1.936
B2_1	0.079	72.39	0.73	70	2.016
B2_c	0.1714	44.433	1.370	25	1.98
B3	0.046	130.43	0.34	160	1.936
CH4	0.256	262.29	1.88	20	1.943
CH5	0.245	273.77	1.81	40	1.957

In terms of verification it is necessary to ensure that the value of the final displacement of the system would be higher than the seismic displacement demand (Doerthy and all 2002, Fajafar 2000). Several Out-of-plane mechanisms generally have a non-linear behavior, this fact is confirmed by the high displacement capacity that masonry have after first fractures and before collapsing (D'ayala and Sperenza 2003). A non linear analysis for the same mechanisms was also performed. Briefly, in KnLA the overturning mechanism is followed in its evolution Fig 9, the displacement-multiplier curve can be considered linear, as shown in Fig 10.

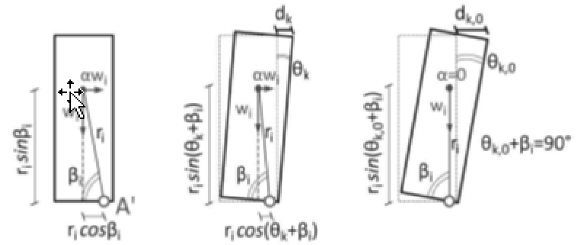

Figure 9. Collapsing steps

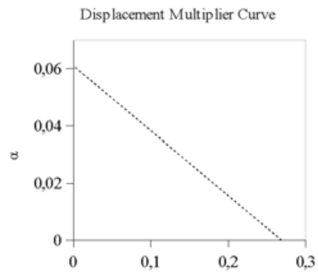

Figure 10. Displacement- Multiplier Curve

The real masonry system is thus transformed in an equivalent SDOF system subjected to spectral displacement (capacity) that has to be compared with displacement demand. The rotation $\theta_{k,0}$, that leads to collapse is given by the expression $M_s = P_i \cdot R_i \cdot Cos(\beta_i + \theta_k)$ of the stabilizing moment, is:

$$\theta_{k,0} = \frac{H_{cp}}{Sin(\theta_{k,0})} \tag{3}$$

from which follows:

$$d_0^* = d_{k,0} \frac{\sum_{i=1}^{n} W_i \cdot \delta^2{}_{k,i}}{\delta_{x,k} \sum_{i=1}^{n} W_i \cdot \delta_{k,i}} \tag{4}$$

where $\delta x_{,k}$ and $\delta_{x,I}$ are the horizontal virtual displacement of control point and i-th force respectively. The verification condition (Table 2) given is related to SLC Italian Standars, i.e. Collapse Limit State, as saying ULS:

$$d_u^* \geq \Delta d$$

$$d_u^* \geq \max\left\{ S_{De}(T_s); S_{De}(T_1) \cdot \frac{Z}{H} \cdot \frac{3N}{2N+1} \cdot \frac{(\frac{T_s}{T_1})^2}{\sqrt{(1-\frac{T_s}{T_1})^2 + 0.02 \cdot \frac{T_s}{T_1}}} \right\} \tag{5}$$

considering $T_s^* = 2\Pi \sqrt{\frac{d_s^*}{a_s^*}}$ $d_s^* = 0.4 d_u^*$ $a_s^* = a_0^* \cdot (1 - \frac{d_s^*}{d_0^*})$.

In Fig.11, the capacity curves for the considered local mechanisms are reported.

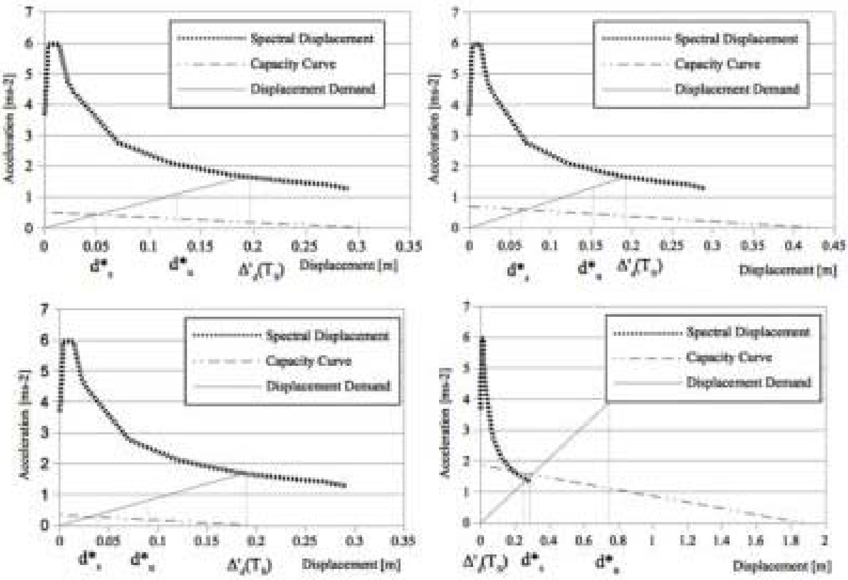

Figure 11. Macro-blocks Capacity Curves for blocks: (a) B1_1, (b) B2_1, (c) B3, (d) CH4-CH5

4.2 Steel Tie Rods Design and FRCM and FRP Strips Design

As regards for the steel reinforcement three conditions have to be verified: yielding of the rod (Ta), yielding of the slab (Tb) and failure of portion of the masonry (Tc) involved in the anchorage T=min{Ta; Tb; Tc} , where Ta= A f_{yd} , Tb= $f_{yd}[(2(a+t)+2(b+t)]t$ and Tc= $f_{cd}(a•b)$. In Fig 12(a), each green point represents an AISI304 steel rod ϕ22 mm and a slab for the anchorage 300x300x20 mm.

As regards for the alternative composite reinforcement strips design, two different materials were considered: CFRCM, carbon fiber reinforcement cement matrix and CFRP, carbon fiber reinforcement polymer. For CFRP strips the design criteria are based on the assumption that the collapse modality happens for delaminating of the interface (i.e. detachment of fibers and removal of thin layer of masonry). As regards to the design values for FRCM strips are based on experimental test carried on at Florence University Official Test Laboratory. Position of strips are suggested in Fig 12(b) and the strips heights for each blocks are (in mm) for CFRP and FRP respectively: 879 and 979 (B1_1), 351 and 391 (B1_c), 1230 and 1370 (B2_1), 439 and 489 (B2_c), 351 and 391 (CH4), 703 and 783 (CH5).

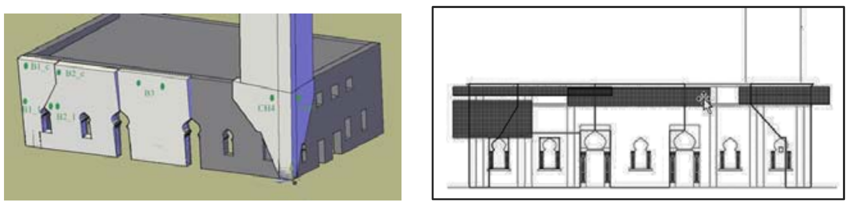

Figure 12. (a) Steel tie rods position. (b) Disposition of FRCM or FRP strips on the façade

4.3 Linear kinematic analysis for the in- plane collapse mechanisms

The five span arcade was transformed into a mechanism composed of 11 macro-elements (33 Degree Of Freedom) and 16 hinges (32 Degree Of Constraint) similar to the one that the earthquake activated. It was decided to reinforce masonry modifying the global behavior of the arcade with an intrados continuous FRCM strip Fig 13, where in grey are macro-elements and position of gravity centers, in green position of external and internal hinges and in red the hinges blocked thanks to the reinforcement Fig 14.

Table 2: Arcade Analysis Results

ID block	α_0	*M**	a^*_0 $[ms^{-2}]$	*S/D layer*	f_{tb} *[MPa]*	*Lstrip FRCM[mm]*	a^*_r $[m/s^2]$
AR	0.296	319.31	2.321	S	543.29	**600**	< 9.81

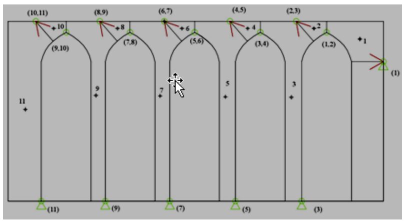

Figure 13. Arcade Mechanism

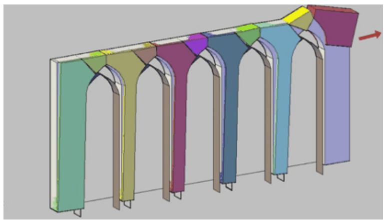

Figure 14.Arcade mechanism and position of FRCM strips

4.4 Global FEM analysis

The whole geometry of the building and its specificities was modeled in the Straus7 (HSH) environment. A static linear analysis were first performed for vertical loads and only after natural frequencies analysis it was possible to set RPA03 Response spectrum and corresponding direction of action (Table 3). Then all load cases was combined to read stress and displacements (Table 4). In order to conceive RPA03 prescriptions, 50 vibration modes were considered, even if only the first 36 converged to a result, it was reached 83.889% and 84.721% participating mass in x and y direction respectively, as synthesized in Table 5, were it is possible to see that modes 7th and 10th are the ones that excited the major part of the mass. These two ways of vibration mainly involve the oscillation of the Minaret that played the most important role in the building response Fig.17 a,b. The attention was focused on the distribution of stresses in comparison with reported damages suffered by the building.

As regards for stress, in Fig.15 c and d, the areas of maximum tensile strength for S1 and S4 combination respectively are highlighted in green. It is possible to see how the qualitative distribution of stresses perfectly overlaps the crack pattern suffered by the two façades, especially at the connection between the minaret and the main block and in correspondence of the arches keystones of the openings. The average tensile stress in these areas varies from 0.9 to 1.55 MPa, that confirm the cracks again.

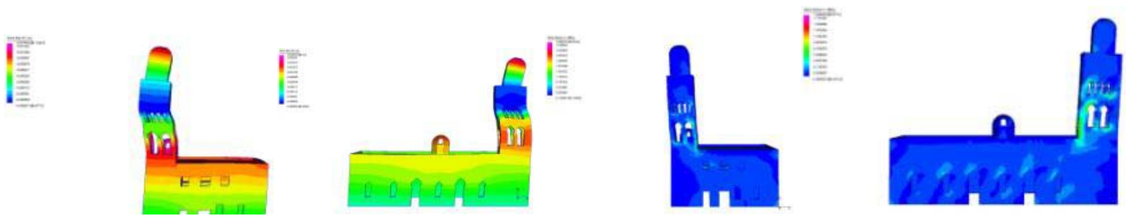

Figure 15. : a) vibration Mode 7 – Y dir. Displacements; b) vibration Mode 10 – X dir. Displacements; c) S1 combination – 11 dir. Stresses [MPa]; d) S4 combination – 11 dir. Stresses [MPa]

Table 3: Spectral Cases

Cases	X Dir.	Y Dir.	Z Dir.
Spectral Case 1	9.81E+00	2.97E+00	0.00E+00
Spectral Case 2	2.97E+00	9.81E+00	0.00E+00

Table 4: Load Combinations

Load Combos	S1	S2	S3	S4	C1	C2	C3
Dead	1.00E+00	1.00E+00	1.00E+00	1.00E+00	1.30E+00	1.30E+00	1.30E+00
Celing	6.00E-01	6.00E-01	6.00E-01	6.00E-01	1.50E+00	0.00E+00	1.50E+00
1st floor	6.00E-01	6.00E-01	6.00E-01	6.00E-01	0.00E+00	1.50E+00	1.50E+00
Spectral Case 1	1.00E+00	-1.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00
Spectral Case 2	0.00E+00	0.00E+00	1.00E+00	-1.00E+00	0.00E+00	0.00E+00	0.00E+00

Table 5: Natural Frequencies Analysis Results: the first 10 vibration modes

Mode	*Frequency(Hz)*	*Modal Mass*	*PF-X(%)*	*PF-Y(%)*	*PF-Z(%)*
1	1.27E+00	5.58E+04	0.002	6.293	0
2	1.64E+00	4.92E+04	6.074	0	0
3	2.73E+00	7.23E+04	0.001	0.001	0
4	4.79E+00	2.79E+04	0	1.155	0
5	6.00E+00	2.36E+04	1.687	0.008	0
6	8.00E+00	8.37E+04	0.065	1.93	8.105
7	***8.61E+00***	***5.06E+05***	***0.13***	***68.612***	***0.152***
8	9.17E+00	3.12E+04	0	0.034	0.001
9	9.89E+00	3.09E+04	0.369	1.884	0.002
10	***1.07E+01***	***1.71E+05***	***60.95***	***0.443***	***0.005***

5 CONCLUSION

The behavior under earthquake of the Delly's antique mosque, damaged in 2003 by the Boumerdes Earthquake (Ms=6.8), was studied with the aim to design consolidation interventions. Trying to describe the structural behavior of historical masonry is a complex theme and it is still a research issue to investigate on. In order to grasp the core of the problem and model the entity of interventions, the linear and non-linear kinematic analyses were used on the basis of local mechanisms of damage. This simple tool well represent the intrinsic behavior of masonry (Lagomarsino 2006, 2009) and allowed to design reinforcements systems of steel tie rods or in composite materials (FRP and FRCM). In addition, a FEM investigation also was made by Straus7. The linear dynamic analysis carried out on the mosque allowed to highlight the global response of the structure through the precise description of the geometry.

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