

# **Structural Safety Control of Masonry Buildings: Non-Linear Static Seismic Analysis with a Non-Linear Shear Strength Criterion**

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## **Abstract**

This paper has the purpose of comparing the shear strength obtained using the values proposed by the code for a level of knowledge LC3 [1,2] of a masonry structure with those experimentally obtained from the mechanical tests with flat jacks and the compression tests on stone elements. The shear strength criterion of Mohr-Coulomb has been utilised, applying for the cohesion and friction angle the values obtained by the non-linear criterion of Hoek-Brown for rock masses [3]. For the last criterion an analogy has been assumed between rock mass and masonry; it has been properly adapted to the specific case of the masonry structure examined in the paper.

**Keywords:** masonry structure, pushover analysis, Hoek-Brown criterion.

## **1 Introduction**

In Italy most of the existing building heritage consists of masonry buildings made with techniques and materials which are different for period and site of construction.

The mechanical characteristics of the materials forming the structure is due to the heterogeneity of the materials, of complex determination as they require specific experimental investigations for each individual case study [4].

For existing buildings it is therefore complex to assume FE models in a nonlinear analysis so that the returned results could describe the behavior of the real structure [5]. The difficulties of determining the experimental data required for FE modeling in nonlinear analyses, are overcome by the Italian code by assuming tabulated values (that include wide enough ranges of typologies) instead of a few acquired experimental data. This generalization, however, requires high safety factors.

This approach is very "large" and implies the risk of nullifying the results to be obtained with a dynamic finite element analysis, with obvious negative effects on the results obtained, also from an economic point of view.

This work aims to compare the results obtained using the values proposed in the code for a level of knowledge LC3, for a masonry structure, with the values experimentally obtained by mechanical tests with flat jacks and compressive tests on samples extracted undisturbed.

When the experimental results have been obtained it is possible, following the code, to reach the values of the shear strength of masonry  $f_{vk0}$  required for the static control to horizontal loads (seismic).

The novelty of this research is to apply an analogy between masonry and rock mass to determine the value of the shear strength. To this aim the Hoek-Brown criterion is applied: it turns the Mohr-Coulomb criterion in the non-linear field, following an approach proposed by the authors and already quite known and applied in the geotechnical literature.

The parameters for this approach are: compressive stress from undisturbed samples, percentage of elements extracted intact, presence of discontinuities, cracks in the masonry mass, level of humidity in the walls.

A push-over analysis was conducted on an existing building by comparing the results obtained from the values tabulated by the code, with those obtained by the Hoek-Brown criterion, at the same seismic intensity [6].

## 2 Static equivalent nonlinear analysis: Equivalent Frame model –Pushover Analysis

The building has been analyzed by a static equivalent nonlinear analysis (Pushover), modeling the structure with equivalent frames.

The capacity curves were plotted by acting increasing forces and excluding the plasticized or collapsed elements, updating the stiffness of the still reacting structural elements. In this way the redistribution of forces during the analysis at the level of the damage (Adaptive Pushover) has been taken into account.

From the capacity curves a comparison with the design seismic action (determined according to the code) has been performed [6].

### 2.1 Static control of existing buildings following the Italian Code NTC 08

The code, for the nonlinear analysis of the structure requires that the safety checks listed below are carried out.

It has been considered the prescriptions provided by the Italian code for the nonlinear analysis of the structure [1, 2]. The control has resulted positive for both modeling procedures of the structure.

#### 2.1.1 Evaluation of the ultimate bending moment

A nonlinear distribution of the compression stresses is assumed:

$$M_u = 0,5 \cdot \sigma_0 \cdot t \cdot l^2 \cdot \left[ 1 - \left( \frac{\sigma_0}{0,85 \cdot f_d} \right) \right] \quad (1)$$

where:

- $\sigma_0$  [equal to  $P/t \cdot l$ ] is the average normal stress on the section;
- $P$  is the vertical force normal to the considered section;
- $t$  and  $l$  are, respectively, the thickness and length of the wall;
- $f_d$  is the calculus compression strength (from the design calculus).

### 2.1.2 Evaluation of the ultimate shear force

The ultimate horizontal force that leads to the collapse for shear of the element in its plane is evaluated ( $V_t$ ):

$$V_t = \left( \frac{l \cdot t \cdot f_{vmo}}{\gamma_M \cdot FC} \right) + \left( \frac{0,4 \cdot l \cdot t \cdot \sigma_N}{\gamma_M \cdot FC} \right) \quad (2)$$

$V_t$  is then compared to the seismic shear force.

The mean value of the strength force has then been considered because it is an existing building ( $\gamma_M=1$  for nonlinear analyses), including the confidence factor  $FC$ .

### 2.1.3 Evaluation of the ultimate shear strength under a diagonal tensile force

Following the NTC 08 code the ultimate shear strength under a diagonal force is possible only for existing buildings. It has been determined when the principal tensile stress in the middle of the panel reaches the tensile strength of the masonry,  $f_{td}$  (diagonal tensile stress). The shear strength of the section is evaluated with the following expression:

$$V_t = l \cdot t \cdot \tau_{m,ult} = \left( \frac{l \cdot t \cdot f_{td}}{b} \right) \cdot \sqrt{\frac{1 + \sigma_N}{f_{td}}} \quad (3)$$

The tensile strength  $f_{td}$  can be assumed equal to  $1,5 \cdot f_{vmo}$ .

### 2.1.4 Evaluation of the out-of-plane behavior

The resistance of masonry is evaluated comparing the calculus bending force with the ultimate bending strength, determined to calculate the axial stress.

In this case the slenderness of the element is considered. The procedure evaluates the strength to the centered compression axial stress, using reduction coefficients of the compression strength, which take into account both the bending moment (due to the eccentricity) and the destabilizing effects of the second order (based on the slenderness).

### 3 Experimental Investigation

#### 3.1 Deformability parameters

A series of in situ experimental tests have been performed on the ex “Palazzina Comando” of the Rossani barrak in Bari to determine its deformation parameters [7].

For example, the stress/strain plots obtained from the tests with flat jacks on the first floor walls, are shown in Figure 1.

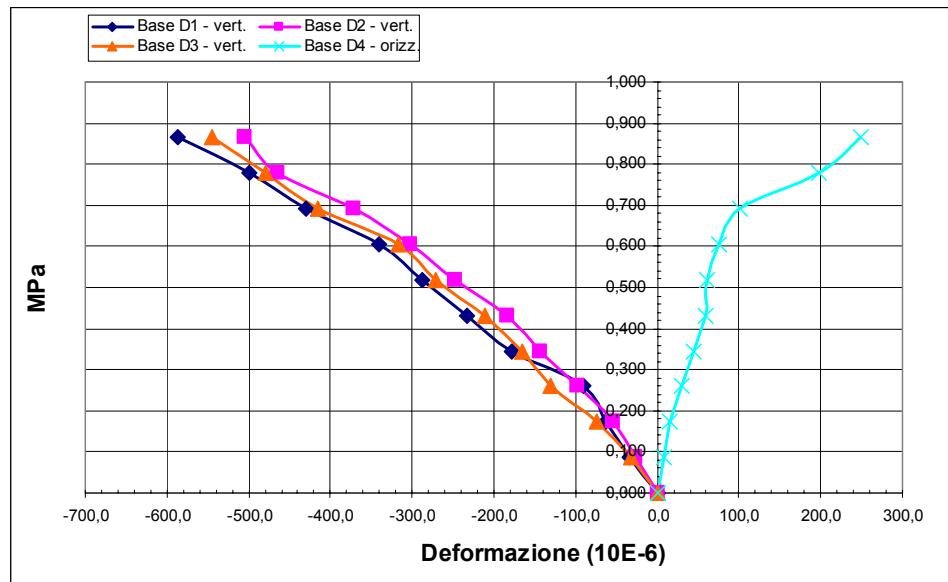


Figure 1: Stress/strain plots – first floor.

#### 3.2 Compressive strength of stones at the ground floor

To obtain the mean compressive strength of the limestone masonry at the ground floor, 11 cubic specimens of intact material have been extracted, with a side of about 70 mm.

The mean value of the compressive strength of the sampling is equal to  $\sigma'_{ci} = 157.63 \text{ N/mm}^2$ , as shown in Figure 2.

#### 3.3 Compressive fracture of the stones at the first floor

In a similar way it has been determined the average compressive strength of the masonry of the first floor (tuff) on 12 cubic samples of intact material. The mean value of the compressive strength on the samples is equal to  $\sigma'_{ci} = 3.46 \text{ N/mm}^2$ .

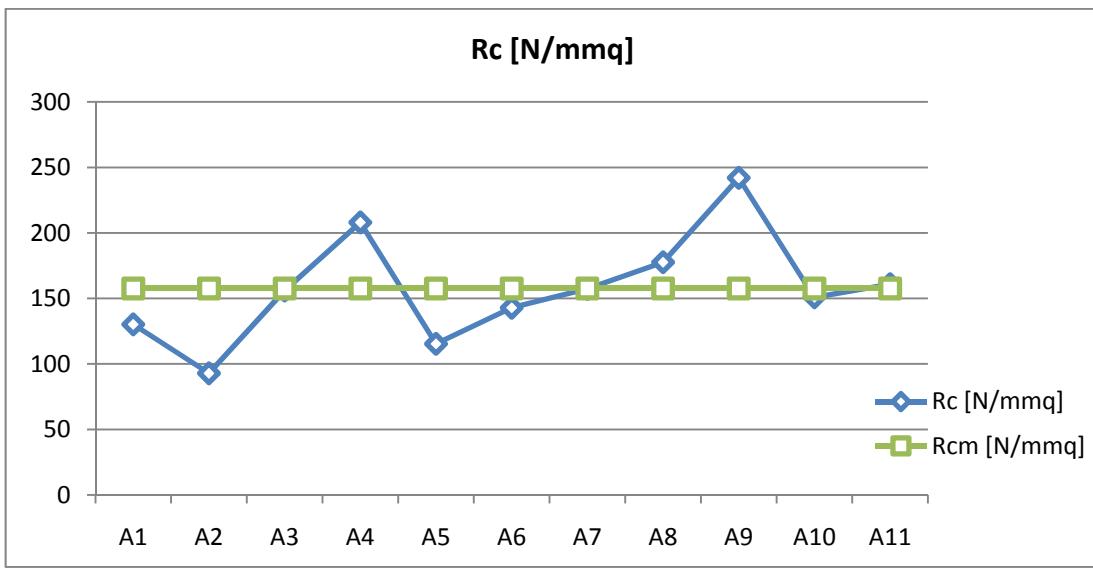


Figure 2: Values of the compressive strength for limestone elements at the ground floor.

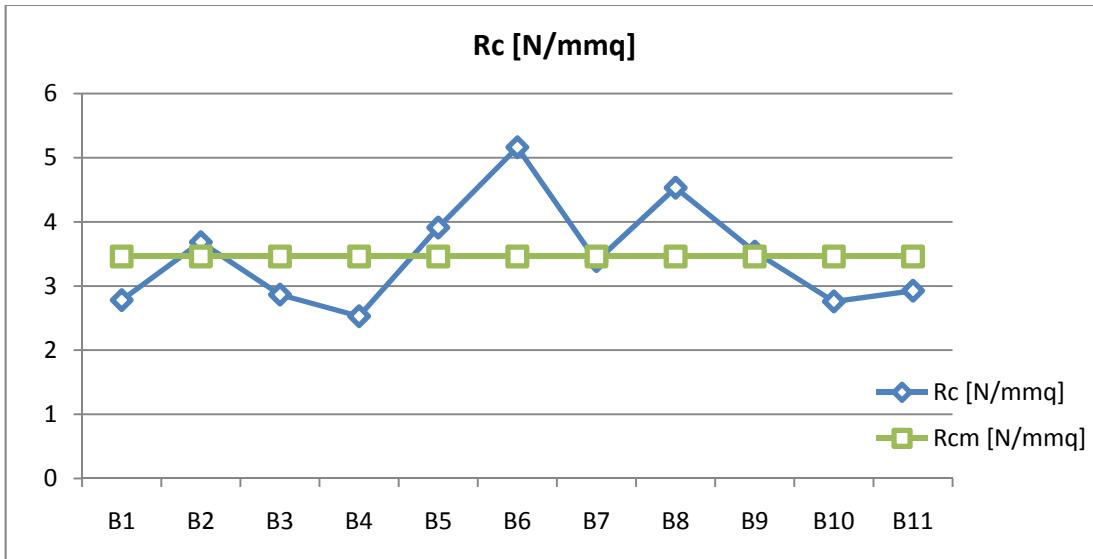


Figure 3: Values of the compressive strength for tuff elements at the first floor.

## 4 Mechanical characteristics of masonry

### 4.1 Shear strength following the NTC 08

To define the mechanical and elastic properties of a masonry it is very important the historical knowledge of the building, the readings of any crack patterns and the level of degradation, in order to highlight the structural diseases and suggest the correct structural intervention strategies [9,10].

The Italian code [3,4], in absence of experimental investigations, gives the possibility to determine the mechanical characteristics of masonry using prescribed parameters, which classify the mortar according to the average compressive strength (*NTC 08 -Table 11.10.III*), or to its composition (*NTC 08 -Table 11.10.IV*).

For natural elements of squared stone it is then possible to determine the characteristic compressive strength  $f_{bk}$ :

$$f_{bk} = 0,75 \cdot f_{bm} \quad (4)$$

where  $f_{bm}$  is the average compressive strength obtained from experimental tests.

Knowing the value of  $f_{bk}$  and the class of mortar utilized in the masonry, it is possible to define the compressive strength of the masonry from Table 1 (*NTC 08 - Tab. 11.10.VI*).

$f_{bk}$ (N/mm <sup>2</sup> )	Mortar			
	M 15	M 10	M 5	M 2,5
2,0	1,0	1,0	1,0	1,0
3,0	2,2	2,2	2,2	2,0
5,0	3,5	3,4	3,3	3,0
7,5	5,0	4,5	4,1	3,5
10,0	6,2	5,3	4,7	4,1
15,0	8,2	6,7	6,0	5,1
20,0	9,7	8,0	7,0	6,1
30,0	12,0	10,0	8,6	7,2
$\geq 40$	14,3	12,0	10,4	-

Table 1: Characteristic compressive strength for a masonry with natural elements in squared stones and 5-15 mm thick joints (*NTC 08-Tab. 11.10.VII*).

In absence of vertical loads (pure shear), the shear strength can be evaluated on masonry specimens, following the code instructions. The characteristic strength,  $f_{vk0}$ , is then obtained from the mean value,  $f_{vm}$ , determined from laboratory tests (shear tests under horizontal or vertical load):

$$f_{vk0} = 0,70 \cdot f_{vm} \quad (5)$$

Expression (5) is furnished by the code.

The value of the shear strength of a masonry, for natural elements or squared artificial elements in reinforced concrete, alternatively, can be obtained from the compressive strength of the elements and the class of mortar (see Table 2), (*NTC 08 - Tab. 11.10.VII*).

$f_{bk}$ (N/mm <sup>2</sup> )	Type of mortar	$f_{vk0}$ (N/mm <sup>2</sup> )
$f_{bk} > 15$	$M10 \leq M \leq M20$	0,20
$7,5 < f_{bk} \leq 15$	$M5 \leq M < M10$	0,15
$f_{bk} \leq 7,5$	$M2,5 \leq M < M5$	0,10

Table 2: Characteristic shear strength, in absence of a vertical load, for masonry with elements in natural stone.

The strength criterion usually adopted for masonry in presence of compressive and shear stresses is well expressed by the following:

$$f_{vk} = f_{vk0} + \mu \cdot \sigma_N \leq 1.5 \text{ N/mm}^2 \quad (6)$$

where  $f_{vk}$  is the characteristic value of the compressive strength on the plane of the wall, where  $f_{vk0}$  and  $\sigma_N$  are obtained from experimental tests.

In (6):

$f_{vk0}$  is the shear strength in absence of vertical loads,

$\mu$  is the coefficient of internal friction of masonry (usually assumed equal to 0.4),

$\sigma_N$  is the average normal stress acting on the plane of the section in consideration.

The values of the elastic characteristics of a masonry, that is the longitudinal elastic modulus  $E$  and the shear modulus  $G$ , are usually obtained from experimental tests. Alternatively, the following values can be assumed:

$$E = 1000 \cdot f_{bk} \quad G = 0.40 \cdot E \quad (7)$$

The Italian code provides the minimum and maximum values of the mechanical parameters for existing buildings, classifying the historical masonry according to the different morphological array. As an example, Table 3 shows the mechanical parameters related to two types of masonry, made of tender stone blocks and hard stone blocks (to be utilized in the case study) (*Table C8A.2.I*).

Masonry Typology	$f_m$ [N/mm <sup>2</sup> ]	$T_0$ [N/mm <sup>2</sup> ]	$E$ [N/mm <sup>2</sup> ]	$G$ [N/mm <sup>2</sup> ]	$w$ [kN/m <sup>3</sup> ]
	min-max	min-max	min-max	min-max	
tender stone	2,40	0,042	1260	420	16
hard stone	8,00	0,12	3200	940	22

Table 3: Reference values of the mechanical parameters provided by the code (*NTC 08-Tab. C8A.2.I*).

In Table 3:

$f_m$  is the mean value of the compressive strength;

$T_0$  is the mean value of the shear strength;

$E$ ,  $G$  are, respectively, the longitudinal and shear modules;  
 $w$  is the weight per unit volume.

## 4.2 Nonlinear shear strength – Hoek-Brown

The method to determine the non-linear shear strength proposed here becomes innovative and experimental by proposing an analogy between rock mass and wall mass in the development of the mechanical characterization of the masonry. Given that the structure under consideration consists of natural stone elements, from the data obtained from the tests performed on extracted samples of intact material, criteria of resistance valid for a rock mass are used. In this case, for example, the mortar joints are considered to be similar to the discontinuities found in a rock mass.

The first strength criterion used is the Mohr-Coulomb one; it establishes a linear relationship between the shear strength ( $\tau$ ) obtained on a sliding plane and the normal stress ( $\sigma$ ) acting on the plan according to the characteristics of the material. In terms of effective stresses the relationship can be written as:

$$\tau = c' + \sigma' \cdot \tan\varphi' \quad (8)$$

The parameters  $c'$  and  $\varphi'$ , respectively, cohesion and angle of internal friction, are determined referring to the nonlinear criterion of Hoek-Brown. The same authors define the methodology: it is assumed eight couples of values  $\sigma'_1-\sigma'_3$ , equidistant in the field  $0 < \sigma'_3 < 0,25 \cdot \sigma'_{ci}$ , that satisfy the equation of the criterion; these values are linearly interpolated and the parameters  $c'$  and  $\varphi'$  are determined.

Hoek-Brown criterion is expressed by the following relation:

$$\sigma'_1 = \sigma'_3 + \sigma'_{ci} \cdot \left[ m_b \cdot \left( \frac{\sigma'_3}{\sigma'_{ci}} \right) + s \right]^\alpha \quad (9)$$

where:

$\sigma'_1$  and  $\sigma'_3$  are the maximum and minimum effective principal stresses, respectively;  
 $\sigma'_{ci}$  is the uniaxial compressive strength of the masonry (average of the values obtained in laboratory);

$m_b$ ,  $s$ ,  $\alpha$  are parameters characteristic of the masonry wall and are defined in the following.

$m_b$  is obtained from:

$$m_b = m_i \cdot e^{\frac{\xi-100}{28}} \quad (10)$$

$m_i$  is a parameter defined by Hoek-Brown for a specific kind of intact material (for limestone at the ground floor  $m_i=12$ , for tuff at the first floor  $m_i=8$ ); the exponential coefficient  $\xi$  depends on the constructive typology and the humidity conditions of the masonry wall in situ;

$s$  is another parameter obtained in function of  $\xi$ :

$$s = e^{\frac{\xi-100}{9}} \quad (11)$$

The exponential  $\alpha$  in (5) is function of  $\xi$ :

$$\alpha = 0,65 - \frac{\xi}{200} \quad (12)$$

The complex behavior of the discontinuities in a rock mass is essentially considered in a masonry mass like a problem of contact between two surfaces in correspondence of the mortar joints. Therefore the parameter  $\xi$  has been determined assuming an analogy with the classification criteria of a rock mass:

$$\xi = \sum_i P_i - 5 = 52 \quad (13)$$

where the values  $P_i$  are obtained from [3] and for the present case are equal to:

$P_1 = 12$  if  $100 \leq \sigma'_{ci} \leq 200$  Mpa (uniaxial compression strength);

$P_2 = 20$  if the percentage of intact samples is higher than 90%;

$P_3 = 10$  if the distance of discontinuity of the mortar joints is included between 20 and 60 cm;

$P_4 = 0$  if the continuous openings are greater than 5 mm and the filling is greater than 5 mm;

$P_5 = 15$  if the rock mass is dry.

The linear interpolation of the equation of Hoek-Brown in the plane  $\sigma'_1-\sigma'_3$  furnishes the values of  $c'$  and  $\varphi'$ :

$$\varphi' = \frac{\sin^{-1} \cdot [6 \cdot a \cdot m_b \cdot (s + m_b \cdot \sigma'_{3n})^{a-1}]}{[2 \cdot (1+a) \cdot (2+a) + 6 \cdot a \cdot m_b \cdot (s + m_b \cdot \sigma'_{3n})^{a-1}]} \quad (14)$$

$$c' = \frac{\sigma'_{ci} [(1+2 \cdot a) \cdot s + (1-a) \cdot m_b \cdot \sigma'_{3n}] \cdot (s + m_b \cdot \sigma'_{3n})^{a-1}}{\sqrt{[1 + (6 \cdot a \cdot m_b \cdot (s + m_b \cdot \sigma'_{3n})^{a-1}) \cdot (1+a) \cdot (2+a)]}} \quad (15)$$

where  $\sigma'_{3n} = \sigma'_{3max} / \sigma'_{ci}$  and  $\sigma'_{3max}$  is the highest value of the pairs  $\sigma'_1-\sigma'_3$ .

Table 4 for limestone at the ground floor and tuff at the first floor, shows the interpolation of the Hoek-Brown method, obtaining the following values:

- Tensile strength  $\sigma_t$ ;
- Uniaxial compressive strength  $\sigma_c$ ;
- Global compressive strength  $\sigma_{cm}$ ;
- Longitudinal elastic modulus  $E$ ;
- Friction angle  $\varphi'$ ;
- Cohesion  $c'$ .

<b>Limestone intact rock (ground floor)</b>		$m_i = 12$	$\xi = 52$	$\sigma'_{ci} = 157,63$
$m_b = 2,161$	$\sigma_t = -0,352 \text{ MPa}$		$E = 11220 \text{ MPa}$	
$s = 0,0048$	$\sigma_c = 0,665 \text{ MPa}$		$\Phi' = 32,67^\circ$	
$a = 0,505$	$\sigma_{cm} = 1,295 \text{ MPa}$		$c' = 8,555 \text{ MPa}$	
<b>Tuff intact rock (first floor)</b>		$m_i = 8$	$\xi = 52$	$\sigma'_{ci} = 3,46$
$m_b = 1,441$	$\sigma_t = -0,012 \text{ MPa}$		$E = 2087 \text{ MPa}$	
$s = 0,0048$	$\sigma_c = 0,234 \text{ MPa}$		$\Phi' = 29,22^\circ$	
$a = 0,505$	$\sigma_{cm} = 0,568 \text{ MPa}$		$c' = 0,167 \text{ MPa}$	

Table 4: Constants of Hoek-Brown and mechanical parameters of the masonry.

The values of  $c'$  and  $\varphi'$  are utilized in (8) to obtain the value of the shear strength  $\tau$  for the Pushover analysis.

## 5 Results

A Pushover analysis was conducted for a level of knowledge LC3 applying the prescriptions of the code and modeling the structure using the two methodologies previously described. As a result two cases have been compared:

CASE A: utilizes the values derived from the tables of the code for the masonry (*see Table 3*).

CASE B: utilizes the experimental values obtained from the tests and some parameters derived from the theories of Mohr-Coulomb and Hoek-Brown, as previously described.

Table 5 shows the values of the mechanical and elastic parameters for the two materials, limestone and tuff, obtained for CASE A and CASE B, respectively.

Limestone – ground floor			Tuff – first floor		
Parameters	CASE A	CASE B	Parameters	CASE A	CASE B
$E \text{ [MPa]}$	3200	11220	$E \text{ [MPa]}$	1260	2087
$G \text{ [MPa]}$	940	4488	$G \text{ [MPa]}$	420	2922
$\tau_0 \text{ [N/mm}^2\text{]}$	0.12	6.66	$\tau_0 \text{ [N/mm}^2\text{]}$	0.042	0.129
$\sigma \text{ [N/mm}^2\text{]}$	8.0	157.63	$\sigma \text{ [N/mm}^2\text{]}$	2.4	3.46

Table 5: Mechanical parameters utilized in the two modeling.

From Table 5 it is evident that the mechanical strength of the buildings proposed in CASE B is from 1,4 to 7 times higher respect to the mechanical characterization of CASE A, excluding from the comparison the peak values that exceed the probability distribution.

A Pushover analysis has been applied to the two models defined as CASE A and CASE B and 16 capacity curves have been obtained.

From the diagrams of the capacity curves the values of the ultimate displacements in terms of capacity (C) and demand (D) of the structure and the values of the maximum displacements for the Safety Life Limit State ( $SLV = C/D$ ) have been determined.

Table 6 shows the numerical results of the 16 curves obtained from both modeling, comparing CASE A and CASE B.

CURVE	CASE A [mm]			CASE B [mm]		
	capacity	demand	SLV	capacity	demand	SLV
1	1.18	0.52	<b>2.27</b>	14.89	1.19	<b>12.47</b>
2	1.15	0.52	<b>2.23</b>	14.89	1.13	<b>13.22</b>
3	1.15	0.52	<b>2.23</b>	14.89	1.13	<b>13.22</b>
4	1.18	0.52	<b>2.27</b>	14.89	1.19	<b>12.47</b>
5	0.83	1.69	<b>0.49</b>	2.16	0.41	<b>5.32</b>
6	0.88	1.45	<b>0.61</b>	1.59	0.30	<b>5.28</b>
7	0.88	1.45	<b>0.61</b>	1.59	0.30	<b>5.28</b>
8	0.83	1.69	<b>0.49</b>	2.16	0.41	<b>5.32</b>
9	1.60	0.53	<b>2.99</b>	1492	1.15	<b>12.95</b>
10	1.62	0.53	<b>3.04</b>	14.93	1.03	<b>14.54</b>
11	1.62	0.53	<b>3.04</b>	14.93	1.03	<b>14.54</b>
12	1.60	0.53	<b>2.99</b>	14.92	1.15	<b>12.95</b>
13	1.08	1.04	<b>1.04</b>	14.81	5.12	<b>2.89</b>
14	1.57	1.16	<b>1.35</b>	0.97	0.33	<b>2.91</b>
15	1.57	1.16	<b>1.35</b>	0.97	0.33	<b>2.91</b>
16	1.08	1.04	<b>1.04</b>	14.81	5.12	<b>2.89</b>

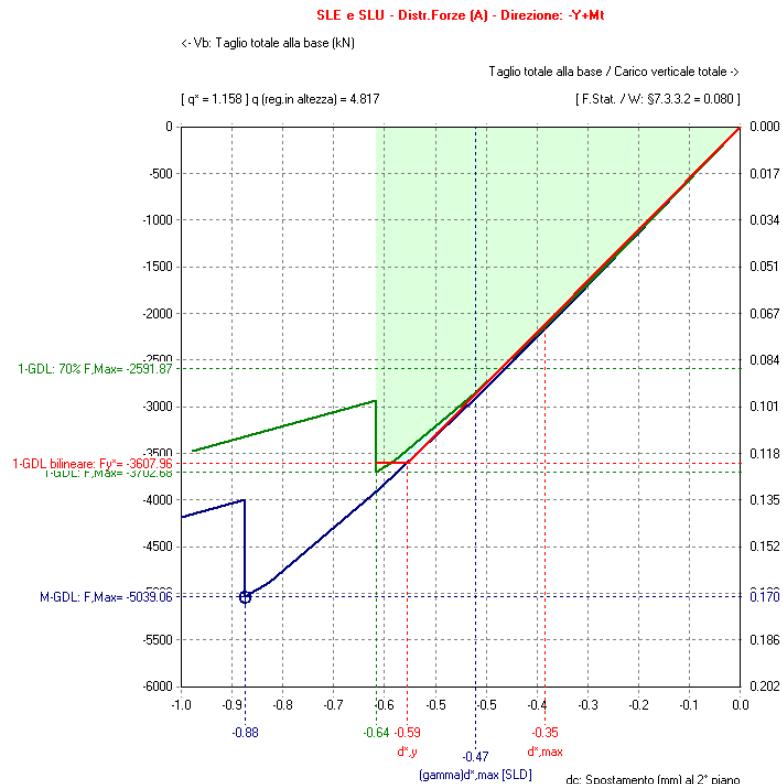
Table 6: Ultimate displacements obtained from the capacity curves for the two modeling.

The structural response provided by CASE B shows margins for improvement from 3 to 6 times higher than in CASE A; for some values results even 10 times higher have been found.

It is observed that in CASE A the capacity curves 5, 6, 7, 8 associated to the earthquake in the Y direction are not verified regarding to the seismic vulnerability, while the homologous curves for CASE B fully meet the criteria of resistance.

From the 16 capacity curves obtained for both modeling, the most representative homologous curves, curves n° 7, are reported (Figure 4).

a)



b)

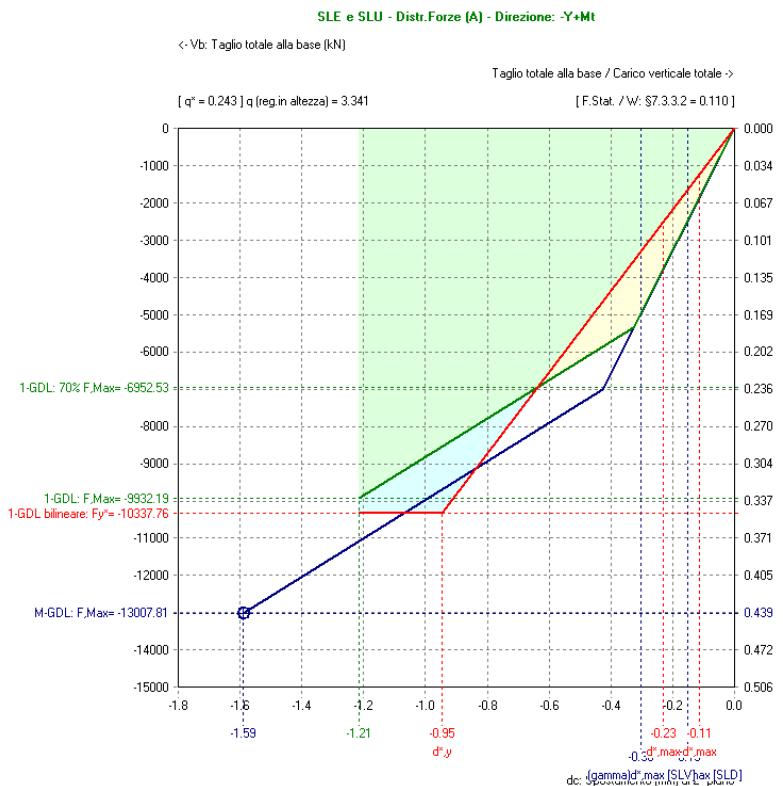


Figure 4: Homologous curves n°7 – a) case A; b) case B.

The plots reports on the horizontal axis the displacement (in mm) of the control point due to an earthquake in the Y direction; on the vertical axis the horizontal force (in kN) transmitted by the earthquake to the foot of the structure is reported.

The blue curve represents the behavior for multi-degrees-of-freedom (MDOF) system.

The green curve is representative of the behavior of a one degree-of-freedom (DOF) system.

The red curve represents the equivalent bilinear behavior.

Some of the capacity curves referred to CASE A are lower than the respective demand curves, therefore, it fails to comply with the control at the Limit State of Life (SLV). In CASE B the capacity curves are always higher than those of the demand, and the controls at the SLV are always satisfied.

## 6 Conclusions

In this paper a pushover analysis on the “ex Palazzina Comando” of Rossani barrak in Bari has been performed. For the analysis two approaches have been considered: the first by applying to the mechanical parameters the values provided by the code; the second by referring to the values experimentally determined and assuming the masonry similar to a rock mass. In the case of a rock mass the formulas in the literature are known.

In both cases, results of the same order of magnitude were recorded, making the proposed methodology acceptable. The safe controls were performed in the second modeling CASE B (experimental proposal). This result was expected because the values utilized have been determined directly using the experimental tests and they do not present a high uncertainty, so that it was possible to assume that the safety coefficients were not excessively precautionary.

Finally, we can attest that a proper, thorough and timely investigation of the structure, from a geometrical and material point of view, could help to overcome the values imposed by the code that, in fact, being generic and tabulated, can sometimes be too precautionary.

In fact we observed that the values given in the table of the code, for any masonry building and in any condition, are included in a range of values assumed after having simply identified “*a priori*” the type of masonry, in which the maximum value is about twice the minimum value.

To consider, instead, the masonry building with its unique mechanical properties, can lead to analyses that better correspond to the actual state, so that it would be possible to introduce less burdensome safety factors.

Future studies could be directed to find out the mechanical and elastic characteristics of masonry through experimental investigations in situ and in the laboratory to get, globally, the nonlinearity of the materials and the masonry rock. As a result of the quality of the new technologies, the goal must be the collection of experimental data on a large-scale, aiming at a more sophisticated analysis of existing masonry buildings.

As a result of the quality of the new technologies, the goal must be the retrieval of large-scale experimental data, where the collection will provide a more sophisticated analysis of existing masonry buildings, with reliable results and describe more accurately the real behaviour of the structure to be studied.

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