

EVALUATION OF COLLAPSE MECHANISMS IN MASONRY BUILDINGS: CRITICAL OVERVIEW AND CASE HISTORIES

VALUACIÓN DE LOS MECANISMOS DE COLAPSO EN LOS EDIFICIOS DE MAMPOSTERÍA: VISIÓN GENERAL CRÍTICA E HISTORIAS DE CASOS

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RESUMEN

En los últimos años el enfoque para el análisis de la vulnerabilidad sísmica de los edificios de mampostería considera crucial la evaluación de posibles mecanismos locales. Las líneas de "fractura" representadas en la pared (cuando es "compacta") pueden mostrar su estado lábil de equilibrio y señalar la cinemática potencial del movimiento relativo entre bloques. La evaluación analítica de la cinemática, lineal y no lineal, presenta algunos aspectos que pueden ser analizados con crítica.

Entre las técnicas para contrastar los mecanismos potenciales podrían considerarse algunas muy eficaces: aplicar placas o bandas de materiales compuestos (recubiertos por yeso) o el refuerzo adicional.

Se consideran en detalle los criterios básicos para aplicarlo y los procedimientos analíticos para determinar el nivel de mejora.

Este trabajo analizará algunos de estos aspectos con referencia a casos reales de estudio (teatro de Pavarotti y la Iglesia de San Barnaba en Módena, y campanario en Reno Centese, Ferrara, Italia).

ABSTRACT

In recent years the approach to the analysis of seismic vulnerability of masonry buildings considers as crucial the evaluation of potential local mechanisms. The "fracture" lines depicted on the wall (when it is "compact") can show its labile condition of equilibrium and point out the potential kinematic of relative motion among blocks. The analytical evaluation of kinematics, linear and non-linear, presents some aspects that could be analysed with criticism.

Among the techniques to contrast the potential mechanisms some very effective ones could be considered: plating with composites (covered by plaster) or reinforced re-pointing.

Basic criteria to apply it and analytical procedures to determine the level of upgrading are considered in detail.

This paper will analyse some of these aspects with reference to actual cases of study (Pavarotti theatre and San Barnaba Church in Modena, and campanile in Reno Centese, Ferrara, Italy).

PALABRAS CLAVE:

KEYWORDS: historic masonry, FRCM, capacity curve, collapse mechanism, composite material.

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INTRODUCTION

In this paper, we want to consider compact masonry buildings, then excluding “multiple leaves masonry” because the former presents its peculiar mechanisms of collapse during earthquake, different from the case of compact masonry. Here for “compact” we intend that masonry in which no lack of continuity is present within the thickness. Therefore, we can consider homogeneity at macro-level.

When an event (earthquake, tornado...) breaks the wall in different compact blocks, they stand in equilibrium until the mutual constraints reach (evolving the configuration under loading), the ipo-static (labile) condition of the structure. That is the internal and external constraints are not sufficient, in number or in position, to guarantee the equilibrium of the structure.

Some examples: 1) one constraint does not obstruct the movement allowed by all the other constraints, 2) the centres of rotation (absolute and relative) became aligned, as critical configuration.

The analysis is linear when is considered only the act of motion of blocks; this is a fundamental aspect for the application of principle of virtual work to evaluate the multiplier of loads.

When the analysis is devoted to the actual state of the structures, the Italian standard NTC-2008 [1, 5] allows to consider the introduction of a) factor of confidence, b) a safety factor in addition to the nominal compressive strength. These factors are important in the design of structures, but not for the investigation of the actual state of the structure under external actions.

In any case, two procedures can be followed: a) linear or b) non-linear kinematic analysis, but in both cases the first step is to evaluate the “multiplier of loads” (corresponding to the activation of the mechanism).

After the analysis of vulnerability, a strengthening intervention should be considered in all the cases that cannot be accepted regarding the expected loading conditions. In the field of Historic Constructions, among the possible interventions respecting the dominant cultural position we have to consider the use of appropriate technique of composite materials. Two effective techniques are nowadays on special attention in Italy [7]: a) plating and b) reinforced repointing [2, 3]. In both cases the material used is composite FRCM (Fabric Reinforced Cementitious Material) in which the matrix is inorganic, and reinforcement is a fabric made

with fibres (Basalt, Aramid, Carbon, PBO, galvanized steel wires,...). These techniques act to produce crack arrestors and then inhibitors to the potential mechanisms, reducing the vulnerability.

KINEMATIC PROCEDURES AND CRITICISMS

The evaluation of potential local mechanisms is one of the most commonly used approaches to study the vulnerability of masonry buildings. In accordance with Italian Codes [1, 5, 18], the local mechanisms can be studied by using different analysis: linear static analysis, non-linear static analysis, linear kinematic analysis, non-linear kinematic analysis. The aim of this paper is to focus on the kinematic approaches applied to compact masonry buildings.

The usual assumptions at the base of the kinematic analysis are [18]:

1. compressive strength of masonry is infinite;
2. tensile strength of masonry is close to zero;
3. no sliding between blocks can occur.

The kinematic approach and the so called capacity curve method [11,12, 14, 15], can be linear or non-linear, and is based on the determination of the horizontal action that the structure can sustain during the evolution of the collapse mechanism.

The main steps of the procedure are:

1. identification of a collapse mechanism;
2. definition of the position of the control point (CP);
3. evaluation of the multiplier of the horizontal loads α_0 that activates the mechanism by using the principle of virtual works (Eq. 1):

$$\alpha_0 \left(\sum P_i \delta_{x,i} + \sum P_j \delta_{x,j} \right) - \sum P_i \delta_{x,j} - \sum F_h \delta_h = L_{fi} \quad (1)$$

where α_0 is the multiplier of the horizontal loads, P are the quantities function of the weights, δ are the

virtual displacements;

4. curve of the multiplier of the loads vs. displacement of CP;
5. calculation of the spectral acceleration (Eq. 2) and consequently the spectral displacement (Eq. 3):

$$\alpha_0^* = \frac{\alpha_0 \sum P_i}{M^* F C} \quad (2)$$

$$d^* = d_K \frac{\sum P_{xi} \delta_{x,i}^2}{FC \delta_{x,k} \sum P_{xi} \delta_{x,i}} \quad (3)$$

where M^* is the participating mass during the mechanism, FC is the factor of confidence;

6. plot of the capacity curve (spectral acceleration vs. spectral displacement).

The control point CP should be representative of all the mechanisms analysed and, generally, is located [4] on the top floor of the structure or in the floor where there is a higher concentration of mass with relevant displacements.

The difference between linear and non-linear approach can have influence on the verification results: in the linear approach the verification is made in terms of spectral acceleration, while in the non-linear analysis is in terms of displacement.

In other words, the linear kinematic analysis is based on the determination of the resistance of the system with regard to the horizontal acceleration that activates the local mechanism. This approach is in terms of force and a behaviour factor, $q > 1$, is introduced [1] to decrease the demand in dependency to estimated ductility effects of the masonry structure. The coefficient q is recommended by standards but is dependent on the geometry and material consistency of structure.

The non-linear kinematic approach, is in terms of displacement and is based on a comparison between the ultimate displacement capacity of the structure, du^* , and the displacement demand of the earthquake, Δd [11, 14].

Starting from the dynamic characteristics of the equivalent S.D.O.F. system, the capacity curve is defined as the relation between the base shear vs. horizontal displacement of the control point CP, identified on the structure.

The spectral displacement d^* of the S.D.O.F. system during the mechanism is obtained multiplying at each step the displacement dk by a quantity function of the weights P_i and virtual displacements $\delta_{x,i}$, starting from the initial configuration [1, 5, 18]. The intersection between the capacity curve and the elastic spectrum (in terms of displacements and acceleration) identifies the so called performance point that provides information about the safety of the structure.

Many literature studies deal with the application of the non linear kinematic approach, and two crucial outcomes are [4]:

(1) the choice of CP doesn't affect so much the trend of the capacity curve; (2) the capacity curve can be approximated as a straight line.

The crucial points of the procedures can be evaluated on the case of study, where the applications give evidence of the peculiarities.

CASE STUDY

SAN BARNABA CHURCH IN MODENA: IN PLANE COLLAPSE OF FAÇADE

The façade of San Barnaba Church (figure. 1) is one of the most important macro-elements that can be identified and studied in the context of collapse mechanism on this building.

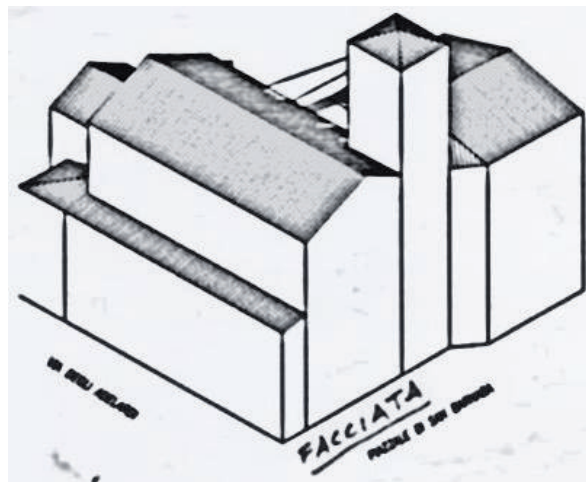


Figure 1. a) The façade of San Barnaba Church and b) its simplified geometrical model.

A dominant in plane mechanism of the façade (figure 2) is due to the formation of an inclined crack that starts from the top right corner of the window and goes to the top left corner of the main door.

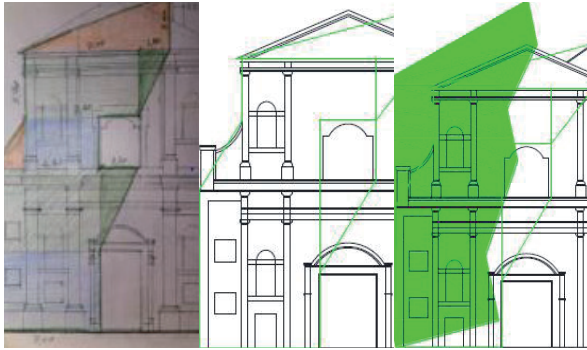


Figure 2. Collapse of the façade of San Barnaba Church: in plane rotation of left section.

Following the linear kinematic approach described in Section 2, the multiplier of the loads (Eq. 1) and the spectral acceleration that activates the mechanism (capacity of the system) (Eq. 2) are calculated. In order to have information about the safety of the structure with respect to the mechanism under consideration, a ratio R (Eq.4) between the capacity and demand in terms of acceleration is introduced.

$$R = \frac{a_o^*}{a_d^*} \geq 1 \quad (4)$$

If R is major than 1 the verification is considered satisfied, otherwise reinforcements are needed.

In this present case, taking into account a behaviour factor $q = 2.25$ for un-reinforced masonry structures, following suggestion in [5], the spectral acceleration (demand) is evaluated and the structure is in safety conditions against the collapse of the façade as local mechanism.

In table 1 the results of un- reinforced structure are collected.

TABLE 1. RESULTS OF LINEAR KINEMATIC ANALYSIS FOR NOT REINFORCED STRUCTURE

A_0	A_0^*	A_D^*	R
0.568	0.554g	0.322g	1.72

If we want to increase the stability, we can insert, for example, three GeoSteel600 strips Kerakoll (figure 3), that are FRCM with steel wire fabrics, under the plaster, to contrast potential Mode I opening.

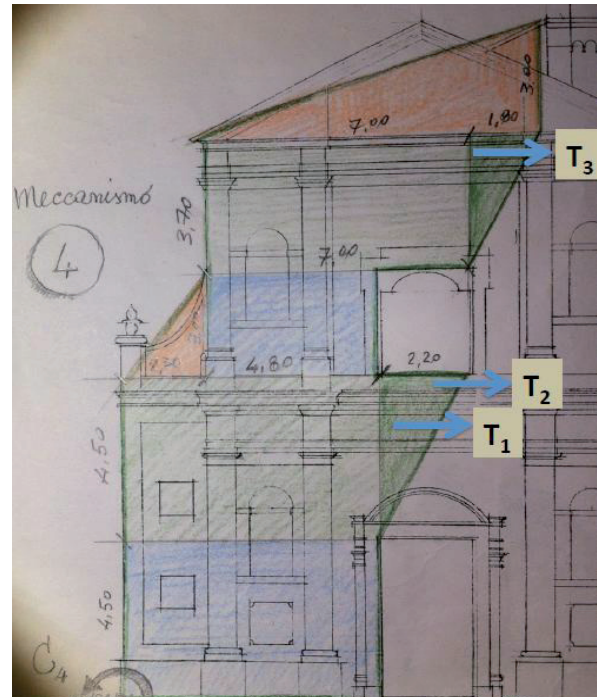


Figure 3. Three GeoSteel600 strips to contrast the in plane collapse of the façade.

In this case, considering reinforced masonry, the contribution of strips and a revised behaviour factor q equal to 2.70 as suggested in [5], should be added in the evaluation of the multiplier of loads, the capacity, the demand and, consequently, the ratio between capacity and demand are updated.

In table 2, the results for the reinforced structure are shown.

TABLE 2. RESULTS OF LINEAR KINEMATIC ANALYSIS FOR REINFORCED STRUCTURE

A_0	A_0^*	A_D^*	R
0.593	0.579g	0.268g	2.16

EVALUATION OF "CRACK CLOSING EFFECT" DUE TO COMPOSITE STRIP PLATING

One important aspect in the design of FRCM as strengthening materials is the evaluation of the detachment force of the fabric strip from masonry, taking into account mechanical connectors (Eq.5):

$$F_{\max} = \sqrt{N_0^2 + 2b_f^2 E_f t_f \Gamma} \quad (5)$$

where N_0 is the maximum force of the connector, b_f , E_f and t_f are the width, the elastic modulus and the thickness of the strip, respectively; Γ is the fracture energy of brick [9].

The maximum force of the connector can be calculated [9] by using Eq. 6:

$$N_0 = \frac{1}{c} A E_c \varepsilon_0 \quad (6)$$

with A is the effective area (generally is the 30% of the total area of the connector, it depends by the technology of application), E_c and ε_0 , are the elastic modulus and the ultimate strain of the connector, respectively; c is the reduction coefficient due to bending of the wires composing the connector.

With the aim to evaluate the effective adherence length of GeoSteel600, Eq. 7 can be used:

$$L_{eff} = \sqrt{\frac{E_f t_f \Gamma}{2 f_{ctm}}} \quad (7)$$

where f_{ctm} is the mean tensile strength of masonry, $(t_f \times \Gamma)$ is the transversal area of the fabric whose width is unitary. Considering t_f equal to 0.084mm, E_f equal to 190GPa, 1.5% as ε_0 , 1.5 as c and Γ equal to 0.024Nmm⁻¹, in table 3 the detachment force of the strip, the maximum force of the connectors and the effective length are shown.

TABLE 3. CHARACTERISTICS OF FABRIC GEOSTEEL600.

F_{MAX} [KN]	N_0 [KN]	L_{EFF} [MM]
11	7	160

All these elements can be utilized for a correct design of the proposed three reinforcing FRCM strips.

3.3 Pavarotti Theatre: collapse of rear wall (stage volume)

The Pavarotti Theatre (figures.4a and 4b) is located in Modena city centre, Italy, and was built in 1841 by the architect F. Vandelli.

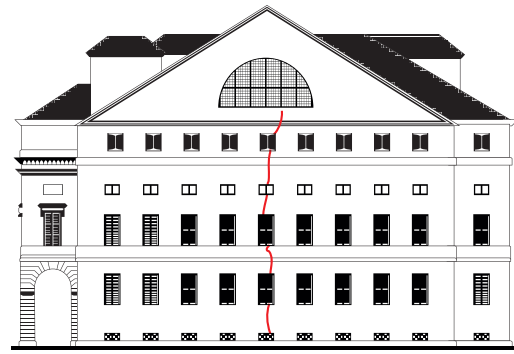


Figure 4. Pavarotti Theatre: a) main façade, b) rear façade (seismic cracks depicted).

The local mechanism object of the study is the collapse of the rear façade wall (figure. 4 and 5a) due to the formation of three horizontal cylindrical hinges (no decks are present because there is located the volume of stage):

(1) C1: at the bottom level of lowest row of windows; (2) C2: at the top level of highest row of windows; (3) C3: its position is the unknown of the problem.

In figure. 5b the kinematic scheme of the local mechanism is shown.

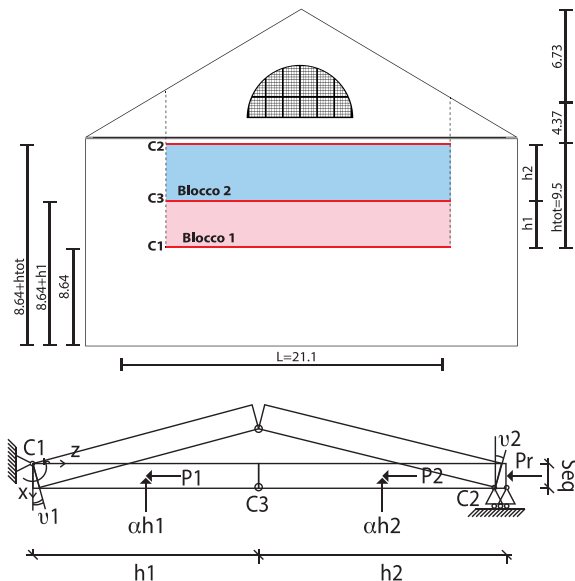


Figure 5. (a) Local mechanism analysed and (b) magnification of the portion of façade involved in the mechanism: the kinematic scheme.

TABLE 5. RESULTS OF LINEAR KINEMATIC ANALYSIS FOR REINFORCED STRUCTURE.

A_0	A_0^*	A_D^*	R
0.647	0.479g	0.322g	1.49

Even if the structure is safe against this mechanism, a design of reinforcement can be made to reduce the spectral acceleration (demand). One possibility is to insert 4 GeoSteel600 strips, 2 for each side of the wall, with connectors (figure. 6a and 6b), to contrast the opening of the wall near

the intermediate hinge. The connectors are placed at a distance of 25 cm one from each other all along the strips. The detachment force of the strip, the maximum force of the connectors and the effective length are the same as in previous example.

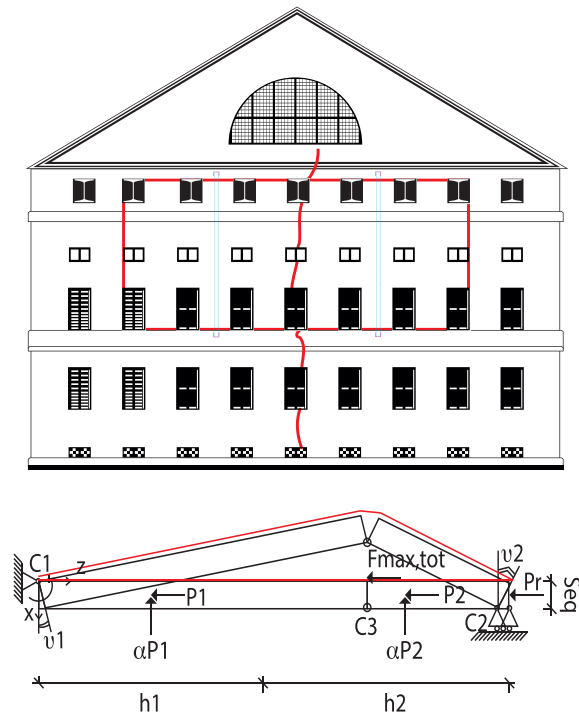


Figure 6. (a) Design of the reinforcement on the front wall and (b) magnification of the portion of façade involved in the mechanism: the static scheme with reinforcement.

In table 5, the results of the reinforced structure are shown.

TABLE 5. RESULTS OF LINEAR KINEMATIC ANALYSIS FOR REINFORCED STRUCTURE

A_0	A_0^*	A_D^*	R
0.651	0.482g	0.289g	1.67

RENO CENTESE BELL TOWER

The bell tower of Reno Centese is located in the municipality of Cento, province of Ferrara, Italy [13]. Starting from its base, the tower is made up of 1) a main prism-shaped corpus (square section shape) including the basement, with small circular openings (windows), 2) a belfry and 3) a spire (figure 7). The tower is 29.25m high with walls characterized by a constant thickness (in the trunk) about 0.50 m.

An earthquake of 5.9 magnitude occurred in the provinces of Ferrara, Modena and Bologna during the night of 20th May 2012,

causing severe damages to many masonry towers and bell towers [7]. Over the successive two months, further seismic events occurred with another six events of a magnitude greater than 5. At the time of the May 2012 earthquake, the tower of Reno Centese was undergoing a “cosmetic” restoration so scaffolding was in place. A severe inclined crack was due to an out of plane rotation of the upper part of the bell tower, which caused successive sliding, two dislocations after shock appeared and experts determined that the tower was in risk of collapse (figure 8). For this reason, a “red area” was delimited around the tower and all activities inside the area were suspended.



Figure 7. The bell tower of Reno Centese.

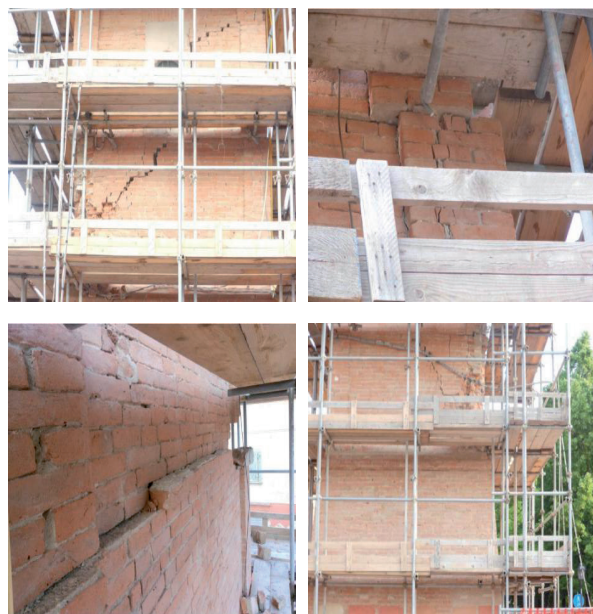


Figure 8. Main damage resulting from the May 2012 earthquake in Emilia: a) diagonal crack (west side), b) dislocation A (20cm), c) dislocation B (6cm), d) cracks (north side).



Figure 9. Propagation of crack due to bending.

After the propagation of the cracks due to bending, the bell tower is composed by two blocks, figure 8: the first one in the bottom part containing the basement and the first part of the trunk, the second block is made by the second part of the trunk, the belfry and the spire.

In order to study the capacity curve, an ideal 3D tower (figure 9), is used as simplified model.

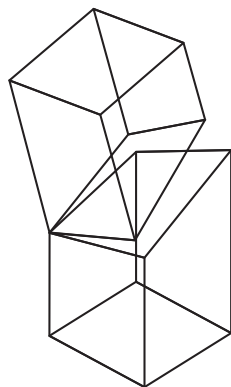


Figure 10. Ideal 3D of tower.

On the bell tower, the propagation of the cracks due to recent earthquake follows two different directions: on the north and west sides the angle of inclination is about $\theta=37^\circ$, while on the south and east sides the angle of inclination is about $\phi=3^\circ$. At this stage, the kinematic chain is defined so it is possible to evaluate the load multiplier that corresponds to the activation of the mechanism (Eq. 1).

It is important noting at each step, corresponding to an increment of rotation equal to 1 degree with respect to the previous step; the new position of the centroid is calculated.

In figure 10, the collapse multiplier function of the new position of the centroid is reported.

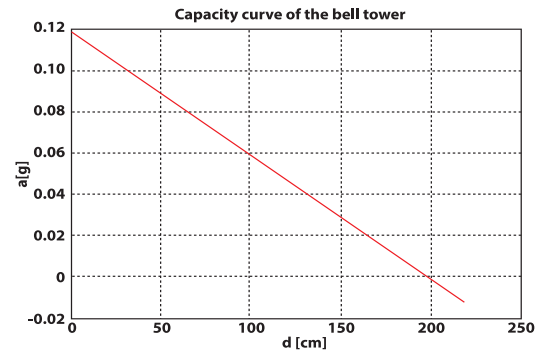


Figure 11. Tower Reno Centese. Collapse multiplier.

By using Eqs. 2 and 3, it is possible to calculate the spectral acceleration and the spectral displacement and, in the end, the capacity curve, figure 11.

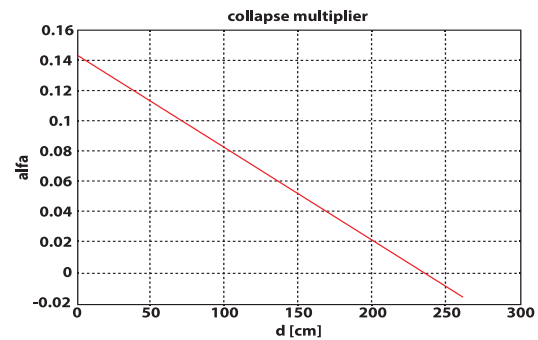


Figure 12. Capacity curve of Reno Centese Tower.

By using the linear kinematic approach, the tower is satisfied against near collapse performance state but not for the life safety performance state. Considering non-linear kinematic approach the bell tower is safe against collapse for life safety performance state

$$(d_u^* \geq \Delta d(T_s), 82.44 \text{ cm} \geq 15.1463 \text{ cm})$$

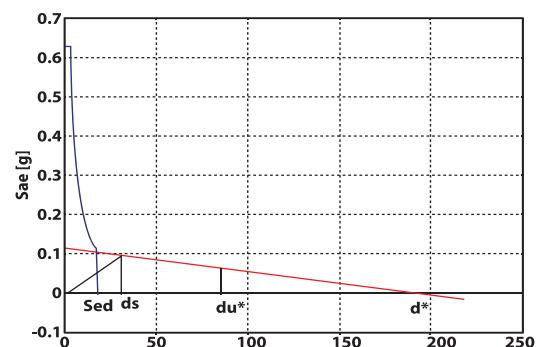


Figure 13. Graphical representation of the non-linear kinematic verification (spectral acceleration vs. spectral displacements).

CONCLUSIONS

In the present study some problems in the application of different procedures to evaluate

the vulnerability of existing monuments are discussed. The different procedures, also recommended by national and international standards, sometimes give different responses, then the engineer involved in the judgement has to utilize also personal convictions. In effect, the use of several not objective coefficients in the procedure drives to a not unique answer in terms of safety evaluation. The level of protection can be greater than minimum required by national standards covering the uncertainty of coefficients (FC: Confidence Factor, q: behaviour factor, etc...) choice. Regarding the procedure via local mechanisms, some aspects merit consideration: the interaction of blocks with surroundings structures that are difficult

to evaluate (see: the roof for upper blocks,...). For the "analytical" evaluation of the mechanisms sometimes the lability and its evolution is not clear. On the side of efficacy of strengthening with composites strips FRCM, when two lips of masonry have relative movements in Mode II (FM, sliding, see [8]) the contrast can be done essentially by strip with connections (mainly mechanical connectors, see [9]) and the evaluation of their mode of operation and analytical modelling is not covered by standard formulation. Among the mechanisms regarding towers we should include that involving soil [6]; here the analysis is not complete for each structure but only one mechanism has been taken into account.

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