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# Multi-storey masonry buildings: evaluation of effectiveness of mechanical strengthening

# Wielopiętrowe budynki murowane – ocena skuteczności wzmocnienia mechanicznego

**Key words:** limit analysis, pushover, tie-rods effectiveness, seismic vulnerability, masonry walls

**Słowa kluczowe:** analiza graniczna, przewrócenie się, skuteczność stężeń, podatność na uszkodzenia sejsmiczne, ściany murowane

## 1. INTRODUCTION

Nowadays, the evaluation of seismic safety of historical buildings and the retrofitting design still deserve attention from the international scientific community, due to wide heritage of existing masonry buildings. In fact, the historical buildings were constructed according to ancient rules of art, without performing any explicit structural analysis. Furthermore, modeling and analyzing of masonry structures are complex tasks, due to the anisotropic and non-homogeneous material properties, as well as frequent modifications of the static scheme, occurring over the centuries as a consequence of elevations, openings of in the bearing walls, etc. In this regard, the Italian technical code [1] explicitly requires the evaluation of the structural safety, that has to be included in the structural report with the safety level achieved through the retrofitting and/or the possible limitations to be impose for the building use. For these assessments, the Italian technical codes [1] and [2] recommends of using linear or not linear kinematic analysis, and/or pushover analysis. In the light of the previous observations, in this paper both kinematic and pushover analyses are used for evaluating the horizontal capacity and the tie-rods effectiveness in multi-storey masonry building.

## 2. THE NEAPOLITAN SCHOOL AND THE MASONRY BUILDING

In the past years, Neapolitan buildings have been constructed in different structures typologies. However, if the analysis is restricted to ordinary masonry buildings, it is possible to identify common geometrical and typological characteristics, and making valid criteria of classification.

To this end, the approach proposed by Pagano [3] for the classification of masonry building is one of the most effective in the authors opinion. The basic idea classification is that the overall seismic behaviour of masonry buildings is strongly related to its construction technique, the masonry buildings have been classified in the following three classes:

– first class: masonry buildings with vaulted floor system;
– second class: masonry buildings with floors made by steel beams well fixed to the walls;
– third class: masonry buildings with walls interrupted at each level by reinforced concrete floors.

In order to approximately evaluate the seismic vulnerability of the masonry walls, Sparacio [4] proposed the following two geometrical parameters:

wall to opening ratio: $$R = B / L \quad (1)$$

pier aspect ratio: $$S_n = h / B \quad (2)$$

where $B$, $L$ and $h$ are the dimensions that define the geometry of the masonry wall (Figure 1).

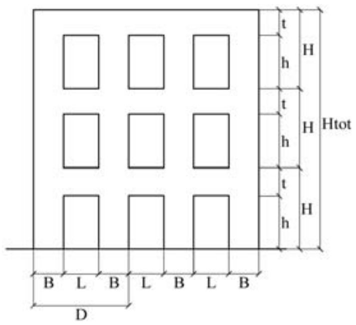


*Fig. 1. Geometrical dimensions of multi-storey masonry wall*

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R = 4 $S_n$ = 0.6 R = 3.15 $S_n$ = 0.76 R = 3.15 $S_n$ = 0.76 R = 2.3 $S_n$ = 1.04

R = 2.3 $S_n$ = 1.04 R = 2 $S_n$ = – R = 1.85 $S_n$ = 1.3 R = 1.5 $S_n$ = 1.6

*Fig. 2. Range of variability of geometrical parameters for Neapolitan buildings [4]*

In the Neapolitan building masonry construction (Figure 2) these two parameters vary considerably with the wealth of the owner and the time of construction [4].

Lenza et al. [5] proposed a valuable classification of the spandrel beams in the masonry walls, depending on their structural behavior. In detail, the spandrel beams are divided in following three classes depending on the constrain level that they are capable to explicate between two adjacent piers:

– weak spandrel beam, with no tensile-resistant element: the spandrel has no capacity of coupling between two adjacent piers (Figure 3a);
– spandrel beam with single tie, with one tension-resistant element, made of concrete beam or of steel tie-rods (Figure 3b);
– spandrel beam with double tension resistant element (Figure 3c).

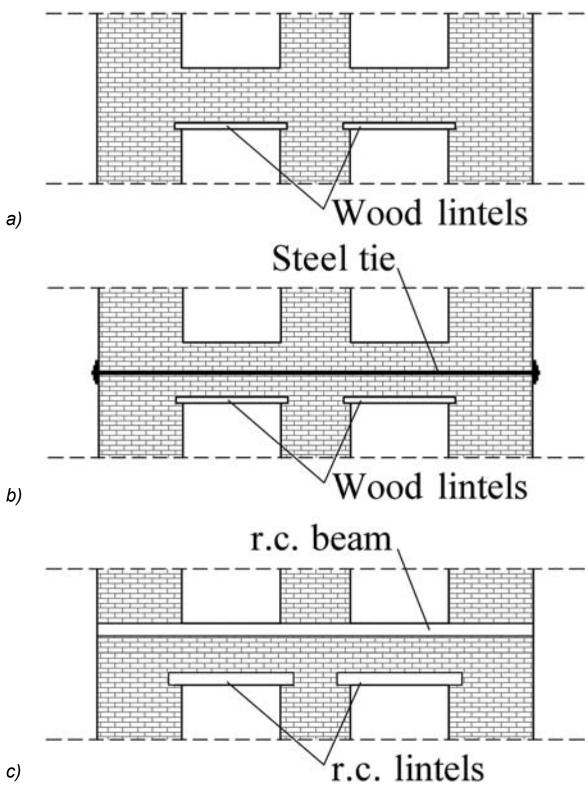



*Fig. 3. Spandrel beams classes: a) weak spandrel, b) spandrel with single tie, c) spandrel with double traction-resistant element*

In [6], the authors proposed closed form expressions of the horizontal collapse multiplier of masonry portal frames under different loading conditions. The provided results showed that the seismic capacity of masonry portal frames is strictly related to the geometrical parameters that completely define the portal frame geometry, i.e. (Figure 1): the global slenderness $\chi = H/D$, the pier slenderness $\zeta = B/D$, and the girder slenderness $\xi = t/H$.

## 3. LIMIT ANALYSIS: CLOSED FORMULATION OF COLLAPSE MULTIPLIER

For the evaluation of the seismic safety of masonry buildings, the Italian codes [1] and [2] suggest of using the linear or nonlinear kinematic analysis. In particular, the kinematic analysis consists in: (i) defining the possible collapse mechanisms; (ii) evaluating the seismic capacity, i.e. the value of horizontal force corresponding to the activation of the mechanism; (iii) comparing the seismic capacity with the seismic demand. With reference to section C8A.4.1. of CM'09 [2], the application of the linear kinematic analysis is based on the following assumptions [7]:

1. null tensile strength;
2. infinite compression strength;
3. sliding of a stone or of a part of the structure upon another cannot occur.

According to the above assumptions, the following collapse mechanisms for the generic masonry wall depicted in Figure 1, can be hypothesized (Figure 4):

a) global mechanism, characterized by the formation of hinges at the ends of the girders and at the base of the piers (Figure 4a);
b) floor mechanism, characterized by the formation of hinges at the base of piers of one specific floor, while in the remaining part of the building (floors above and below) the collapse does not occur (Figure 4b);
c) overturning mechanism of the building or of a portion of the building (in Figure 4c, the overturning mechanism of the entire façade is shown);
d) shear failure of the piers (Figure 4d).

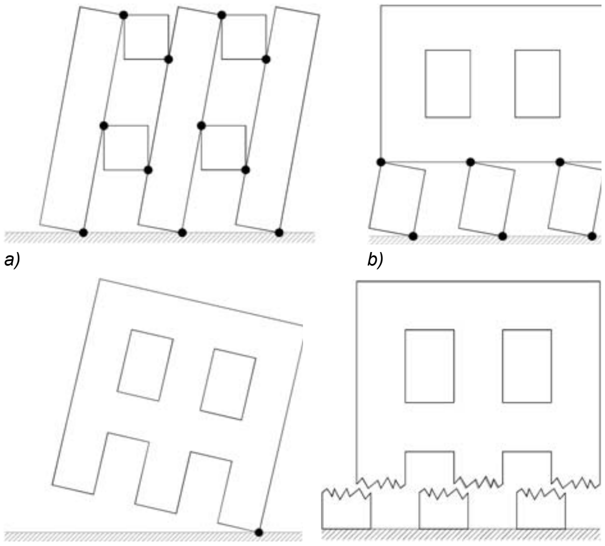



*Fig. 4. Considered collapse mechanisms: a) global mechanism, b) floor mechanism, c) overturning mechanism, d) shear failure mechanism*

According to the above hypotheses, the multiplier of the horizontal actions ($\lambda = F/W$), defined by the maximum horizontal force ($F$) – to – weight ($W$) ratio, that is associated with the four considered collapse mechanisms can be evaluated by means of closed form expressions, which are given in the following for the four cases.

a) global mechanism:

$$\lambda_1 = \frac{B}{2H} \cdot \left[ (1+n_c) \cdot \left( n_p + \frac{A_{tv} \cdot f_{yd}}{W_{TOT}} \right) + \right. \\ \left. + \sum_{i=1}^{n_p} \frac{2 \cdot M_u}{W_{tot}} \cdot n_c \left( \frac{1-\zeta}{1-2\zeta} \right) \right] / \left[ \left( \sum_{i=1}^{n_p} i \right) - \frac{n_p}{2} \cdot \xi \right] \tag{3}$$

where $n_c$ is the number of span, $n_p$ is the number of floors, $A_{tv}$ is the area of the vertical steel tie-rods, $f_{yd}$ is the yielding design stress of the steel of the tie-rods, $W_{tot}$ is the total weight, $M_u$ is the bending moment capacity of the girders, that according to the indications of the paragraph 7.8.2.2.4. of NTC'08 [1] can be calculated as:

$$M_u = min \begin{cases} A_{th} \cdot f_{yd} \cdot \frac{t}{2} \cdot \left[ 1 - \frac{A_{th} \cdot f_{yd}}{0{,}85 \cdot f_{hd} \cdot t \cdot s} \right] \\ 0{,}106 \cdot f_{hd} \cdot t^2 \cdot s \end{cases} \tag{4}$$

where $A_{th}$ is the area of the horizontal steel tie-rods, $s$ is the girder thickness, $f_{hd}$ is the compression design stress of masonry in the horizontal direction. In the above equations $B$, $H$ and $t$ are the parameters that define the wall geometry (Figure 1), while $\zeta$ and $\xi$ are the geometrical ratios defined as follows [6]:

– pier slenderness: $\zeta = B / D$ (5)

– girder slenderness: $\xi = t / H$ (6)

floor mechanism:

$$\lambda_2^j = \frac{B}{H} \cdot \frac{(1+n_c)}{(1-\xi)} \cdot \left[ 1 + \frac{A_{tv} f_{yd}}{W_{tot,j}} \right] \qquad j = 1, \ldots, n_p \tag{7}$$

where $Wtot{,}j$ is the total weight of the portion of façade upstairs of the considerd floor:

$$W_{tot,j} = \sum_{k=j}^{n_p} W_k \tag{8}$$

overturning mechanism:

$$\lambda_3^j = \frac{B}{H} \cdot \left[ \left( n_p + \frac{A_{tv} f_{yk}}{n_p W_{tot,j}} \right) \cdot \frac{1}{2}(1+n_c) + \right. \\ \left. + n_c \cdot \left( \frac{1-\zeta}{\zeta} \right) \right] / \left[ \sum_1^{n_p} i - n_p \xi \right] \qquad j = 1, \ldots, n_p \tag{9}$$

shear failure mechanism:

$$\lambda_4^j = \frac{1}{\gamma_M} \left[ 0{,}4 + f_{vk0} \cdot \frac{(1+n_c) \cdot B \cdot s}{W_{tot,j}} \right] \qquad j = 1, \ldots, n_p \tag{10}$$

where $f_{vk0}$ is the shear strength of the masonry and $\gamma_M$ is the masonry safety factor that, according to Italian technical code [1], can be assumed equal to 2.0.

## 4. PUSHOVER ANALYSIS

The non-linear static analysis (pushover) is nowadays a design tool diffused also in the professional practice for the assessment of masonry structures.

As suggested by the Italian code [1] the masonry walls can be schematized with one-dimensional beam elements through the so called "Equivalent Frame" modeling. The walls are divided in vertical panels (piers), horizontal panels (spandrel beams) and intersection panels between the piers and the spandrels (Figure 5). The piers and spandrels are modeled respectively as columns and beams of 2D frame, while the intersection panels are schematized as rigid links. This structural modeling allows of using lumped plasticity model with plastic hinges for bending and shear in pre-defined points of

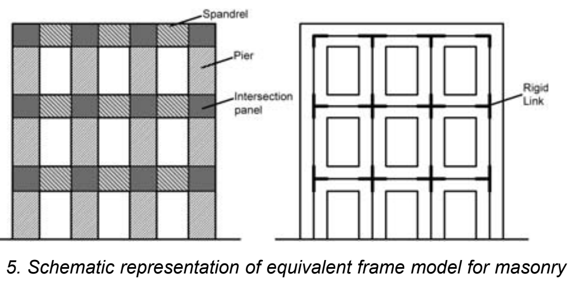



*Fig. 5. Schematic representation of equivalent frame model for masonry façade*

the structure (at bases and tops of the columns, and at ends of the beams). This structural modeling allows for performing a non-linear incremental collapse analysis of the masonry walls.

In the piers, the plastic hinges can be activated, for combined compression and bending action and for shear. In particular, the plastic hinges strength of the pier can be evaluated using the following formulations suggested by Italian code [1] for bending, ($Mu$), and shear, ($Vt$), respectively:

$$Mu = (sB^2 \sigma_0 / 2)(1 - \sigma_0 / 0.85 f_d) \tag{11}$$

$$Vt = Bs \, (f_{vk0} + 0{,}4 \, \sigma_0) / \gamma_M \tag{12}$$

where $\sigma_0$ is the average normal stress and $f_d$ is the design compressive strength of the masonry, $\gamma_M = 2$ is the material safety factor.

When the internal forces reach the ultimate values (*Mu* or *Vu*), the section has a plastic deformation until an ultimate limit ($\varphi_u$ or $\gamma_u$) beyond which the section loses all strength. Figure 6 shows a qualitative behavior of the plastic hinges for flexure and shear, respectively, where $\varphi$ is the section rotation, $\gamma$ is the shear deformation. The ultimate deformations of the plastic hinges can be checked by limitations of interstory limits, as the Italian code suggests. In particular for the combined compression and bending the interstory drift limitation is $\delta/h = 0.6\%$, while for shear is $\delta/h = 0.4\%$.

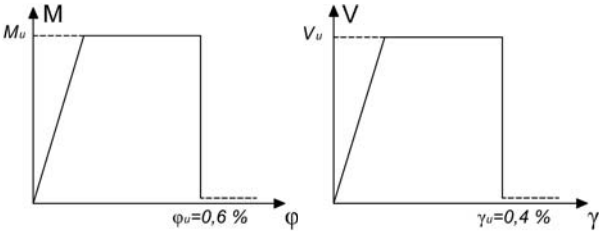



*Fig. 6. Plastic hinge behaviour for bending (a) and for shear (b)*

The mechanical modeling of spandrels is one of the most cumbersome phase of the structural modeling process. In fact, for weak spandrels with no-tension resistant elements (see §2), such as ties or well clamped lintels, the beams have not bending stiffness and strength, therefore the weak spandrel are not able to coupling the piers and they link the top of the piers as "simple pendulums". On the contrary, the confined spandrel with tension-resistant elements has a diagonal strut behavior, with shear resistant mechanisms [5]. Regarding the confined girder, the design bending moment strength ($M_u$) is given by Eq. (4), while the design shear strength associated with the combined compression and bending action ($V_p$) and the design shear strength ($V_t$) are given by the following formulations:

$$V_p = 2 \cdot M_u / L \qquad (13)$$

$$V_t = \mathrm{t} \cdot \mathrm{s} \cdot f_{vd0} \qquad (14)$$

where $f_{vd0}$ is the average value of the masonry shear strength without compression stress. The design shear strength is the minimum of the values of *Vp* and *Vt* given by Eqs. (13) and (14).

## 5. CASE STUDIES

The two methods of structural analysis described in the previous sections have been applied to two historical masonry buildings located in Naples, in the following appointed as building A and B, respectively (Figure 7). They have typical geometrical, typological and mechanical characteristics of Neapolitan building of the XVII–XVIII centuries.

### 5.1. Building A

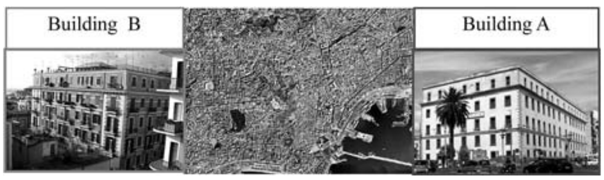
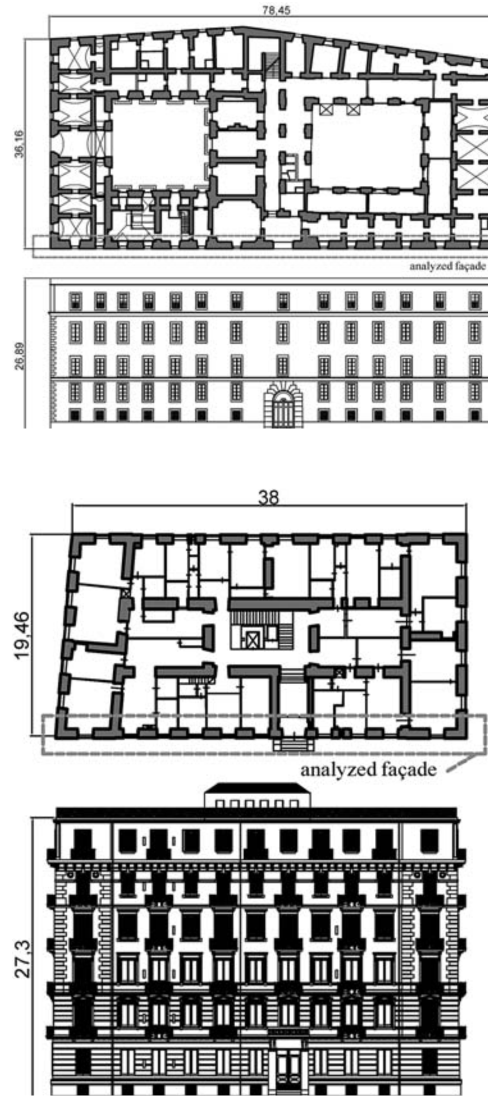



*Fig. 7. Neapolitan building, case studies*

The building A is located within the historical center of Naples and has a strong historical interest, in consideration of its different occupancy over time. Originally it was used as monastery; after a first massive transformation, at the end of 1700, it was converted into a hospital to serve the city prisons, assuming the nowadays volumetry and form. Later, in 1923, the building was further modified to allow the accommodation of judicial offices therein.

The building has five floors above ground and has a rectangular plan, with two courtyards; in plan the total size is about 78 × 36 m, while the two inner courts, almost square, are 16 × 17 and 15 × 19 m, respectively. The overall height of the building is approximately 27 m (Figure 8a).




a)

b)

*Fig. 8. a) Building A – ground floor plan and principal façade, b) Building B – ground floor plan and principal façade*

The structure is made of tuff masonry for the first four levels, while the top level, made in a subsequent time, is in clay brick and mortar walls; the material mechanical properties are reported in Table 1. The floor structure at the first four levels consists in tuff vaults, while the roof structure is composed of steel beams integrated in the lightweight mortar slab, cast-in-place between the adjacent steel beams. The average weight of the first four level vaults is equal to 12 kN/m$^2$, while the weight of the roof floor is 7 kN/m$^2$.

Table 1. Masonry mechanical properties

| Masonry | E [N/mm$^2$] | G [N/mm$^2$] | Unit Weight [kN/m$^3$] | Compression strength [N/mm$^2$] | Shear strength [N/mm$^2$] |
|---|---|---|---|---|---|
| Tuff | 1.080 | 360 | 16 | 2.0 | 0.035 |
| Clay bricks | 1.500 | 500 | 18 | 3.2 | 0.076 |

Neither the slabs of the building have any rigid behavior, nor the girders of the walls are capable to resist the flexure and shear; according to the Pagano classification [3], described in §2, the building A belongs to the first class, having all-brick walls and floor slabs with vaults.

The geometrical characteristics of the masonry walls can be defined and described by the parameters introduced in §2 and §3 (Eqs. (1), (2), (5) and (6)); in particular the analyzed wall (Figure 8a) is characterized by following values: $R = 1.89$ – $S_n=0.97$ – $\zeta = 0.40$ – $\xi = 0.39$.

### 5.2. Building B

The building was built in 1906, at Michelangelo Schipa Street – Naples (Italy), and it can be considered a typical example of the Neapolitan residential building, characterized by a nearly rectangular plan, approximately 20 m × 40 m with a central rectangular core 10 m × 5 m, which includes the staircase and elevator (Figure 8b); it consists of six storeys above the ground level and a basement below. The basement storey height is 3.85 m; for the ground floor and 1st level is 4.32 m; for the 2$^{nd}$, 3$^{rd}$ and 4$^{th}$ levels is 4.25 m; for the 5$^{th}$ and 6$^{th}$ storeys is 3.95 m (Figure 8b).

The walls are made of Neapolitan yellow tuff masonry, with the exception of the basement and ground floor, which are constructed by clay-brick; the mechanical parameters of masonry material assumed in the present paper for the structural analysis are summarized in Table 1. The wall thickness varies from 120 cm, at the basement, to 50 cm at the 5$^{th}$ level. The floor structure is composed of steel beams integrated in the lightweight mortar slab, cast-in-place between the adjacent steel beams and weakly reinforced by steel bars. The steel beams have I cross section, with depth ($d$) of 160 mm and flange width ($b_f$) of 74 mm; the beams are spaced at 90 cm centerline. Total floor structures thickness is 30 cm at the basement and ground floor, 32 cm at the 1st floor, 25 cm at the remaining floors. Additional information can be found in [8].

According to Pagano classification described in §2, the building B belongs to the second class, having continuous masonry walls and isostatic horizontal slabs, made of beams simply supported by the masonry walls. The geometrical characteristics of the masonry walls can be defined and described by parameters introduced in §2 and §3 (Eqs. (1), (2), (5) and (6)); in particular the analyzed façade (Figure 8b) is characterized by the following values: $R = 1.54$ $S_n=1.45$; $\zeta = 0.38$ and $\xi = 0.26$.

## 6. KINEMATIC ANALYSIS OF THE CASE STUDIES

### 6.1. Linear kinematic analysis of the "as is" façade walls

Thanks to regularity and repetitiveness of the openings, the main façades of the two case studies can be schematized with multi-storey frame.

Using the closed form equations (3), (7), (9) and (10), the lowest values of the collapse multipliers associated to the four mechanisms considered in the previous §3 have been calculated. For both the analyzed façades, the global mechanism (Figure 4a), is firstly activated with a collapse multipliers equal to $\lambda_A = 0.13$ and $\lambda_B = 0.07$, for the façade of the building A and B respectively.

A further simplified calculation, for an approximate evaluation of the horizontal collapse multiplier $\lambda$, can be made considering the rotational equilibrium of the single entire pier, with hinge at the base under two different distribution of the horizontal force, i.e. with the horizontal force ($F$) applied at the top (Figure 9a) and in the centroid (Figure 9b) of the entire pier. These two loading conditions have been chosen with the aim of providing a useful tool for checking the results obtained by means of the push-over analysis. In fact, in the non-linear static analysis of masonry structures, two different distributions of the lateral load are generally applied: the "modal" pattern and the "uniform" pattern, with lateral forces proportional to masses.

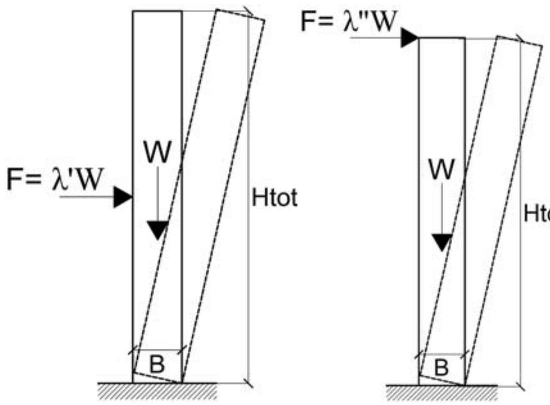



*Fig. 9. Simplified collapse mechanisms for a fast estimation of horizontal collapse multiplier*

The corresponding simplified expressions of the collapse multiplier are given by the following formulae:

$$\lambda' = F / W = B / H_{tot} \qquad (15)$$

$$\lambda'' = F / W = B / 2\, H_{tot} \qquad (15)$$

Applying the above equations (14) and (15) to the principle façade of building case studies, the following values of horizontal capacity can be obtained: $\lambda_A'=0.126$ e $\lambda_A''=0.063$ for the building A; $\lambda_B'=0.073$ e $\lambda_B''=0.036$ for the building B. It is interesting to note that the results related to the scheme with the horizontal force F applied to the pier centroid (i.e. $\lambda_A'$and $\lambda_B'$) are very close to the collapse multiplier values ($\lambda_A$ and $\lambda_B$) obtained with the kinematic analysis (Figure 9a).

### 6.2. Linear kinematic analysis of the strengthened façade walls

A parametric analysis have been carried out, varying the geometry (diameter/area) and the location of the steel tie-rods in order to understand their effectiveness and influence on the seismic capacity of the analyzed walls. To this aim, four different hypotheses of the retrofit intervention have been considered (Figure 10):

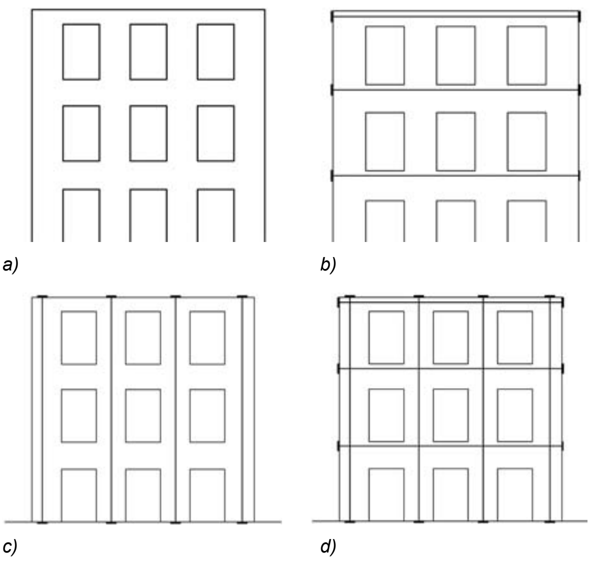



*Fig. 10. Retrofitting interventions with steel tie-rods: a) Case NT – no ties, b) Case HT – only horizontal ties, c) Case VT – only vertical ties, d) Case HVT – horizontal plus vertical ties*

- case NT (no tie), i.e. as is building façade (figure 10a);
- case HT (horizontal ties), i.e. with building façade strengthened through horizontal ties (figure 10b);
- case VT (vertical ties), i.e. with building façade strengthened through vertical ties (figure 10c);
- case HVT (horizontal and vertical ties), i.e. with building façade strengthened through horizontal and vertical ties (figure 10d).

For both the principal façade of the buildings case studies, the horizontal collapse multipliers ($\lambda$) associated with each type of strengthening scheme have been calculated using the closed form equations (3), (7), (9) and (10), by varying the area of the steel ties from 5 cm$^2$ to 40 cm$^2$.

Using ties with steel grade S235 ($f_y$=235 MPa) the variation of collapse multiplier $\lambda$ as a function of the ties area ($A_t$) has been plotted in Figure 11 for the façade of building B, for each considered case of strengthening interventions of Figure 10. It is worthy to note that in all cases the horizontal multiplier ($\lambda$) already increases with small area ties; the Figure 11 also shows that, for the case HT (only horizontal ties), $\lambda$ becomes constant for value of $A_t$ greater than 1000 mm$^2$ because for high values of $A_t$ the mechanism *b* or *d* (Figure 4) is achieved without any contribution of the horizontal ties. On the contrary, in cases VT and HVT, the value of $\lambda$ increases

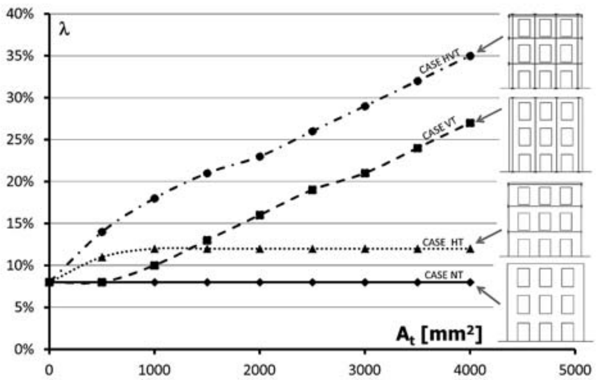



*Figure 11 Principle façade of the building B: λ value as a function of the ties area for the four hypotheses of considered retrofiting interventions*

with an approximately linear trend as the area of the steel ties ($A_t$) increases, showing greater effectiveness of the vertical than horizontal tie-rods.

The collapse multiplier $\lambda$, equal to 8% for the no retrofitted case (NT), becomes 12% for the case HT (only horizontal ties), i.e. it increases of 50%. In the case VT (only vertical ties), instead, the horizontal collapse multiplies ($\lambda$) increases up to 57% (when $A_t$ = 40 cm$^2$), or rather the horizontal capacity of the building increases to 2.4 times compared to NT case. Finally, considering both horizontal and vertical ties (case HVT) the maximum effectiveness of tie-rods has been observed, compared to case A with increasing of horizontal collapse multipier ($\lambda$), between 75% (when $A_t$ = 5 cm$^2$) and 348% (when $A_t$ = 40 cm$^2$).

## 7. NON-LINEAR STATIC ANALYSIS

### 7.1. Non-linear static analysis of the "as is" façade walls (case NT)

The principal façade of buildings case studies have been modeled as "equivalent 2D frame", according to what has been described in the previous §4.

In the "as is" building (case NT), the girders have no tension-resistant elements; for this reason, they are not able to couple the masonry piers and therefore they can be modeled with hinged ends. If girder with hinged ends is used in the structural model however, the computer code is not able to evaluate the rotation at the beam ends; so, it is not possible to verify if there are excessive rotations demand, or rather a local collapse of the girders. For this reason, the girders have been modeled with beam elements with flexural plastic hinges at the ends with low plastic moment and limited rotational capacity ($\varphi_u$ = 0.4%). For the principal façade of the analyzed buildings, the capacity curves are plotted in Figure 12a and Figure 12b, respectively. The maximum base shear force is equal to $F_A$ = 2.740 kN for building A, $F_B$ = 675 kN for building B. The corresponding values of the collapse multiplier are equal to $\lambda_A$=$F/W$=0.09 for building A, $\lambda_B$=$F/W$=0.055 for building B. The maximum displacement of the control point, located at the top of the wall, is equal to 25.2 cm and 55.2 cm for buildings A and B, respectively.

For both the analyzed walls the activation of the flexural plastic hinges at the girders ends is immediate; in both the models the collapse occur due to activation of the plastic hinges at the basis of the piers for combined compression and bending action (Figure 12c and Figure 12d). In fact, in the no-retrofitted configurations (case NT), the girders with no tension-resistance elements are not able to couple the piers, so the piers have a cantilever behavior with high bending moments at the base. In the building B, the first two levels are made of clay bricks masonry, i.e. with more strength material; this leads to the activation of plastic hinges at the basis of the third level piers.

### 7.2. Non-linear static analysis of the strengthened façade walls (case HT)

The model with a retrofitting intervention has been considered, consisting in horizontal tie-rods in the girders, that consequently have considerable flexural resistance and stiffness, and are able to couple the piers. Because of the used commercial software is not able to model vertical tie-rods

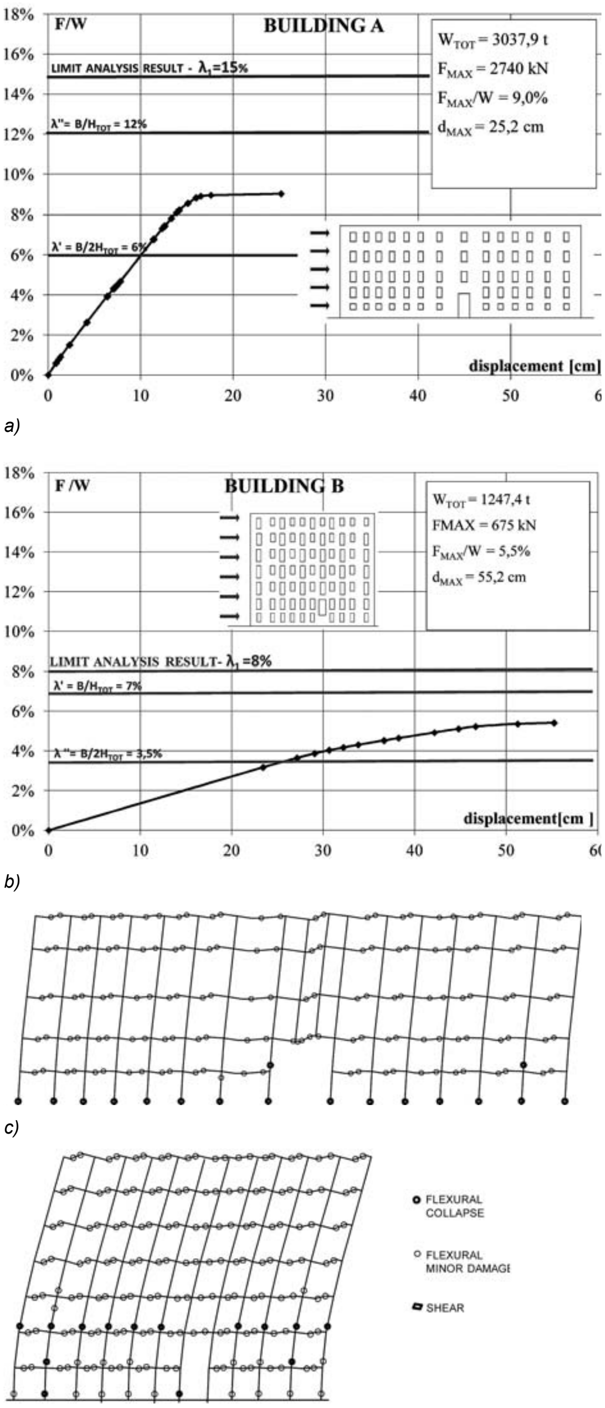



*Figure 12 No retrofitted models – Dimensionless Capacity Curves a) building A b) building B. Kinematic deformed shape a) Building A, b) Building B*

in piers, the retrofitting intervention with vertical ties (case VT) and horizontal and vertical tie (case HVT) have not been considered.

The pushover analysis performed on the principal façade, retrofitted by horizontal ties shows not encouraging results, due to limits of the modeling suggested by the Italian code [1] and [2] for the non linear static analysis of masonry construction. In fact, the capacity curves of Figure 13 are not regular, with a great increase of the horizontal stiffness; furthermore, they do not show increasing of the horizontal resistance for both the analyzed walls. These results are not consistent with the ones obtained with the limit analysis and, above all, are not consistent with what was observed in masonry buildings with horizontal ties, after severe earthquakes.

Finally, the comparison between the limit analysis results (horizontal lines in Figure 13) and the pushover curves of Figure 13 shows that the seismic capacities assessed using the non-linear analysis are always lower than those measured with the closed-form equations proposed in this paper, while they are always included in the range defined by the two multipliers λ' and λ'' evaluated with the simplified formulae (15) and (16).

## 8. CONCLUSIONS

The critical exam of the suggestions provided by Neapolitan researchers for masonry buildings has allowed of making a typological, geometrical and mechanical classification of the analyzed buildings.

The horizontal in-plane capacity of main façade of the analyzed building has been calculated, both with and without the vertical and/or horizontal tie-rods. With a parametric analysis, it has been evaluated the influence of the geometry (diameter/area) and of the location (vertical, horizontal, vertical plus horizontal) of the steel tie-rods.

The linear kinematic analysis, applied according to suggestions and provisions of the Italian codes [1] and [2], has allowed to propose closed form equations for the horizontal collapse multiplier, extension of the equations proposed in [6] for the masonry portal frames. The results of the application of these equations to the analyzed walls allow to observe that:

– the vertical ties (case VT) are more effective than the horizontal ones (case HT);

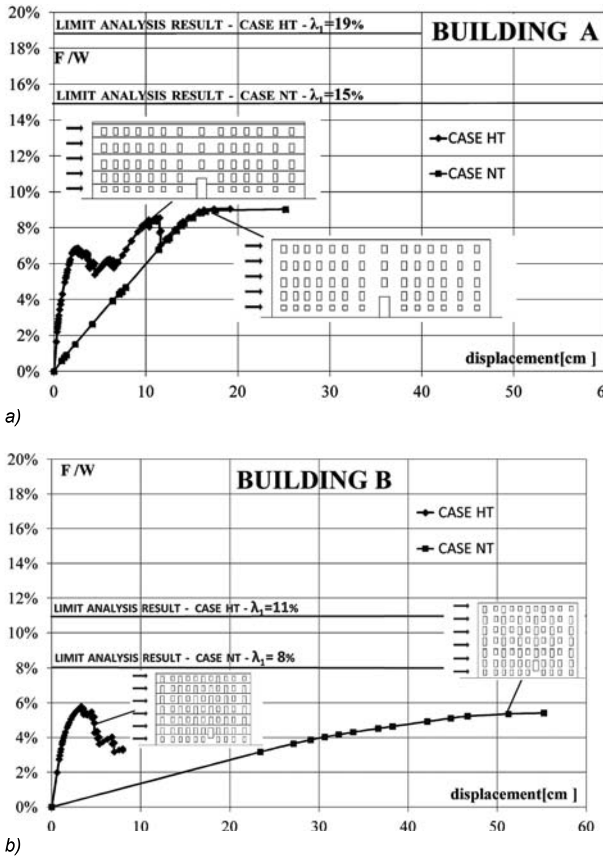



*Figure 13 Comparison between the capacity curves obtained with limit analysis and pushover: a) Building A, b) Building B*

– the effectiveness of the vertical ties linearly increases, when the tie-rods area increases;
– the maximum effectiveness of the horizontal ties can be obtained just for low values of rods area;

The pushover analysis performed, according to Italian Codes [1] and [2], by using a commercial computer code very common in Italy, allows to observe that:

– it is not possible to model the vertical tie-rods;
– the capacity curves of the models with horizontal ties (case HT) are irregular, while the horizontal stiffness that increases;
– the horizontal ties do not increase the seismic capacity of the masonry walls; this result is not consistent with what was observed in masonry buildings, after severe earthquakes.

In conclusion, it is worth to note that the Neapolitan buildings have many levels and very slender masonry walls, often in contrast with ancient builder's rules of thumb, given by tradition and experience; these factors increase their seismic vulnerability as has been highlighted by the results of the performed analyzes.

The authors emphasize the usefulness of the limit analysis, as simple tool both for checking the results of more complex analysis, often less manageable (pushover), and for evaluating the effectiveness of mechanical retrofitting interventions, not always appreciated by using commercial software for structural analysis.

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# Abstract

The paper deals with the problem of evaluating the effectiveness of mechanical retrofitting interventions and of assessing in-plane seismic capacity of unreinforced masonry structures. With reference to two case studies of typical Italian multi-storey masonry buildings, the above evalua1tions have been obtained by means of two different approaches: limit analysis and non-linear static analysis (Pushover). The aim of this study is to verify the pros and cons of examined analyses methods. Moreover, concerning the limit analysis approach, "closed form" expressions of horizontal collapse multipliers – which also account for the mechanical strengthening contribution – have been proposed for the typical in-plane seismic collapse mechanisms of unreinforced masonry walls.

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# Streszczenie

Artykuł dotyczy problemu oceny skuteczności mechanicznych interwencji modernizacyjnych oraz oceny wytrzymałości w płaszczyźnie na wstrząsy sejsmiczne niezbrojonych obiektów murowanych. Ocen dokonano dla dwóch przypadków typowych włoskich wielopiętrowych budynków murowanych stosując dwa różne podejścia: analizę graniczną i nieliniową analizę statyczną (przewrócenia się). Celem pracy było zweryfikowanie zalet i wad powyższych metod analizy. W przypadku analizy granicznej zaproponowano domknięte wyrażenia na mnożniki zawalenia poziomego, uwzględniające również wkład wzmocnienia mechanicznego, dla typowych mechanizmów zawalenia się w płaszczyźnie niezbrojonych ścian murowanych.

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