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Stone Architecture in Abruzzo: Seismic Risk Analysis of Bell Tower of the Church of San Lorenzo in San Buono

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ABSTRACT

The church of San Lorenzo in San Buono (Italy) is unique in the local landscape due to its technical, and architectural features. The building, whose construction dates from the 14th century to the mid-20th century, was realized by the most important designers in the region. The bell tower is an element characterized by refined neoclassicism and testifies to a remarkable episode of the permanence of the neoclassical style in Abruzzo until the first decades of the 20th century.The present study illustrates both a technological-constructive analysis of the building and an assessment of the seismic risk of the bell tower, to present an important point for the knowledge of the artefact with regard to possible conservative restoration works. The seismic risk analyses are carried out for two different structural configurations and two different typologies allowed by the Italian building codes for monumental masonry buildings: simplified linear and non-linear static. The study illustrates how the presence of the adjacent structure significantly alters the structural behavior of the masonry tower, both in terms of displacement and propagation of the crack pattern. The differences in the results of the non-linear calculation and the expeditious methods suggested by the standard are highlighted as well.

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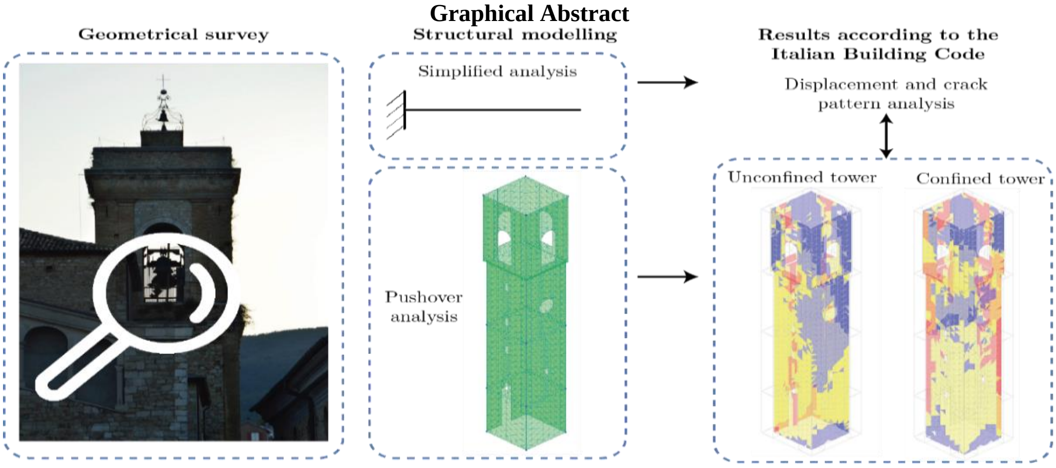
Graphical Abstract



1. INTRODUCTION

Neoclassicism refers to an eclectic movement that spanned the entire European cultural landscape and was based on both aesthetic considerations and matters of taste (1). In architecture, there was indeed a revival of Roman and Greek architectural principles. While searching for the architectural archetypes that had

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favoured the emergence and development of classical architecture, rationality, applied to previous architectural experiences, led to the definition of the concepts of typology and style (1, 2).

The neoclassic architecture, which origins in the 18th century, found a new home in the 19th century when it contributed to the "Empire" style (3). Although it is possible to identify certain historical and cultural-historical circumstances to locate its development, a precise spatial and temporal definition is not possible. Despite the fact that the European architectural scene at the beginning of the 20th century was oriented towards Art Nouveau and the new modern building systems that were becoming more and more prevalent, neoclassical orientations continued to shape the artistic culture of the 19th century. Moreover, it preserved an essential homogeneity of theoretical and speculative premises that formed the basis for mid-20th century monumentalism and the rationalist architecture that followed (4).

The present study illustrates both a technological-constructive analysis of the building and an assessment of the seismic risk of the bell tower, in order to present an important point for the knowledge of the artifact with regard to possible conservative restoration works.

Ozturk (5) studied the seismic behavior of two monumental masonry buildings in Cappadocia. The effect of structural walls on their seismic behavior was highlighted through dynamic analyses.

In the analysis of the damage, the decisive role played by the layout of the floors in the pattern of damage in the dome roofs was described. Guney et al. (6) studied the effect of seismic action on earthen buildings affected by the earthquake in Turkey in 2010 and 2011. The study has focused on the main parameters for describing the seismic behavior of buildings affected by the earthquake (i.e., foundations, type of masonry, geometry of the building, roofing).

Li (7) proposed an optimized macroseismic analysis, integrating the traditional macroseismic intensity scale and evaluation models of resilience and empirical vulnerability of regional buildings. In another work, Li (8) studied the seismic vulnerability of regional group structures, establishing empirical vulnerability matrices of six building groups in different seismic intensity zones. A series of zonal fragility prediction models of six typical regional structure groups is established. Li (9, 10) highlighted the importance of measuring seismic intensity to evaluate and predict seismic risk and vulnerability of structures, proposing an improved equation for instrumental intensity. The innovation introduced consists of an improved model compared to traditional techniques for assessing the seismic risk of regional structures.

The correct assessment of the seismic risk of an existing structure - important for economic and occupant safety considerations - cannot ignore the nonlinear structural behavior of the material. The most appropriate methods for such analyses are: nonlinear static analysis (pushover) and nonlinear time history analysis. The second method can determine the non-linear behavior of the structure in a more realistic way; however it is complex to solve. Therefore pushover analysis is preferable for its computational advantages.

2. DESCRIPTION

The church of San Lorenzo is an ancient building, mentioned in the tithe books from the 14th century. It was only rebuilt in the middle of the 20th century. It represents a valuable architectural element in the area and is an important example of late 19th century neoclassicism, which is only expressed in the interior and the bell tower, as the main façade was remodelled more recently.

In the transformation of the above mentioned building, many important designers of the area follow one another, from the architects Di Rienzo and Aloisio in the early 19th century to the engineer Castelli and the more recent Genio Civile of Chieti in the mid-20th century (11). The church in its present form is a fusion of various earlier buildings. It is the result of the Caracciolo-Pisquizi princes' desire for renewal. Figure 1 shows the main view of the building.

The building is - like the entire historic centre - a masonry building made of local river stone. It measures 10.70 m in width and 29 m in length up to the presbytery. The dome is decorated with stucco; from the ground floor a descent leads into the crypt where the body of the patron saint Buono is kept. The crypt, added in 1774, compensates for the difference in height between the floor of the church and the square behind it and also provides an entrance from there.

Figure 1. Main elevation

The exterior façades are simple and unadorned, with the exception of the main façade and the bell tower.

The bell tower which was restored in 1893 by the engineer Castelli, rises more than 40 metres above the main square. Its height is divided into four registers. The earth extension consists of a steep ashlar base: this is the only part in which this material is used. The first register consist of projecting brick retangular cornices that delimit the masonry. The third register is completely empty and houses the bell tower with arches on all four sides framed by composite-style brick pilasters.

The church has a simple floor plan, consisting of a single vaulted hall in the shape of a Latin cross, with a lowered domed roof in the choir area. In the floor plan, it is noticeable that the width of the space is determined by the diameter of the dome.

The elevation is divided into two registers: a lower one, consisting of the side altars framed by round arches supported by semi-columns of the giant Corinthian order, and an upper one, consisting of the claristorium and the barrel vault. To the right and left of the liturgical hall, Corinthian semi-columns frame three niches in which three side altars are located. The area enclosed by the dome is flanked by two other smaller altars, which are larger than the previous ones. In the wall opposite the entrance wall is the main altar, which is dedicated to the patron saint of the small center. Figure 2 shows the elevation view of the belltower.

3. DESIGN FEATURES

The construction of the church extended over a considerable period of time: therefore, there are overlaps in the technical and architectural solutions and languages.

Figure 2. Belltower: elevation view

An analysis of the historical sources shows that the search for building materials followed the classic criterion of maximum cost-and-time savings. The semi-finished stonework that characterises the church is shown in an arrangement that follows continuous rows, with a finished mortar joint that is much thinner compared to the rustic architecture that characterises the urban area, testifying to the richness of the construction.

More specifically, the masonry technique can be defined as being characterised by blocks of different sizes, some of which have been worked; discontinuous mortar layers with brick fragments (11). The lithological setting consists of irregular blocks of quarry limestone, pebbles and erratic material from river beds. The state of preservation of the masonry is good and no cracks or active kinematics can be observed.

All the walls consist of masonry with the features just described and a bag profile with inserted diatons. The thickness of the perimeter walls is between 1 and 2 metres. The masonry of the building consists of vaults and domes. In the liturgical area, a structure similar to that typical of the Latin development of the eastern basilicas can be observed. It consists of a series of repeating masonry structures that frame the barrel vaults and on which they rest. These arches relieve the weight of the roof and the vaults themselves on the inner buttresses, which are represented by the masonry areas with the semi-columns of the giant order.

4. MATERIAL CONDITION ANALYSIS

The building features technical solutions and material choices typical of traditional architecture in Abruzzo at the end of the 19th century, although there are no particular innovations at the technological level.

As there are no signs of foundation or structural cracks, it can be assumed that the deterioration of the materials is due to other causes. The main problems that the building suffers from concern the low resistance of the materials to weathering, which is due both to the typological characteristics and to the lack of experimentation with new materials, as reported in literature (12).

As far as typological characteristics are concerned, the domed roof which, despite its expert construction, shows a slight infiltration of rainwater. Other elements exposed to decay are the plaster cladding and the paintwork. The main causes are exposure to drive rain in areas that are not protected by waterproofing, the use of non-breathable paints or coatings and inadequate rainwater disposal solutions. On the outer wall, especially in the upper part of the bell tower, the decay affects the mortar. This is located in the most superficial area.

5. SIMPLIFIED SEISMIC ANALYSIS

Tower structures such as a bell tower can be compared to a system of distributed masses with an infinite number of degrees of freedom and consequently an infinite number of vibration modes. In reality, it is only the first modes of vibration that define the true seismic response of the system. In this section, the results of the simplified calculation model provided by Italian standard (13) are presented.

The equation of motion of a shelf with distributed mass embedded in the ground can be written as follows (12):

$$m^{*}q'' + K^{*}q = 0 \tag{1}$$

where:

$$m^{*} = \int \mu\phi^{2}(z)\, dz \tag{2}$$

$$K^{*} = \int EI(\phi^{2}(z))^{2}\, dz \tag{3}$$

where:

- $\phi(z)$ is the oscillation waveform;
- μ is the mass distributed over height;
- q'' is the second derivative of displacement as a function of time

Considering a quadratic waveform of the type:

$$\phi(z) = \frac{z^{2}}{H^{2}} \tag{4}$$

It is obtained from previous Equations:

$$m^{*} = \mu\frac{H}{5} \tag{5}$$

$$K^{*} = 4\frac{EI}{H^{3}} \tag{6}$$

$$c^{*} = \mu\frac{H}{3} \tag{7}$$

The expressions for the maximum displacement of the cantilever and the load distribution as a function of elevation can be derived.

$$y_{max} = \mu g H^{4}\frac{S_{e}}{g\, 12\, EJ} \tag{8}$$

$$p(z) = \frac{5}{3}\mu\frac{g}{H^{2}}\frac{S_{d}}{g}z^{2} \tag{9}$$

5. 1. Structural Modeling The modelling of the belfry was carried out by varying the boundary conditions (i.e., constraints), offered in this case by the presence of the adiacent bodies (e.g., presbytery). The effects of two calculation models, different in terms of constraint conditions, are thus considered:

i. Model A - isolated tower, with a height of 42 m;
ii. Model B - tower constrained in the portions adjacent to the church, which interact with the tower itself. In this case the free span of the church is lower and equal to 30 m.

5. 2. Seismic Parameters The geometric-mechanical parameters adopted in the calculation are summarised in Table 1.

The maximum displacement obtained for the unconfined tower model (i.e., Model A) with linear calculation assumptions is 9.12 cm.

The maximum displacement obtained for the confined tower model (i.e., Model B) with linear calculation assumptions is 2.37 cm.

The confinement offered by the adjacent structures of the church makes it possible to reduce the free span of the ideal cantilever that schematises the tower, thus considerably reducing the maximum displacements.

6. PUSHOVER ANALYSIS

This section describes the second evaluation level considered. The pushover analysis considers a non-linear numerical model of the tower, in which the external forces are increased in a linear manner until the structure collapses. The mechanical parameters of the materials, knowledge of the construction technology of the time and the state of preservation of the material play a fundamental role in this type of analysis.

6. 1. Mechanical Parameters of Masonry The mechanical parameters adopted in the calculation were taken from the guidance provided by Circular 2019 - NTC2018 (13). This regulation presents a collection of

TABLE 1. Geometrical-mechanical parameters for simplified calculation

Parameter	Value	Units
Tower height	42 – 30	m
Elastic modulus	12000	daN/cm^2
Base length	7.00	m
Base height	6.80	m
Wall thickness	1.30	m
p_s	2000	daN/m^3
$\tau_{vk,0}$	3.50	daN/cm^2
γ_M	2.40	-
FC	1.35	-
q	3	-
T*	0.451	s
C_C	1.37	-
T_D	2	s
ag	0.113	g
S	1.50	-
F_0	2.569	s

the most recurring mechanical parameters of masonry. For the model under consideration, the following typology was considered: “masonry with rough-hewn ashlars, with faces of uneven thickness”.

Table 2 shows the main mechanical parameters of this masonry type.

Figure 3 shows the constitutive law for the masonry material.

The nonlinear parameters are summarized in Table 3.

6. 2. Levels of Knowledge Considered According to the Italian regulations (NTC2018), the knowledge level (LC) influence the values of the mechanical parameters adopted. In fact, each knowledge level (i.e., LC) corresponds to a confidence factor (i.e., FC) that is used to reduce the values of the mechanical properties of the materials considered according to following equation:

TABLE 2. Mechanical properties of the material considered [15]

$f_{cm(k)}$ [MPa]	**$f_{vm(k)0}$ [MPa]**	**$f_{tm(k)}$ [MPa]**	**$f_{cm(k),0}$ [MPa]**
2	0.035	0.035	2
Γm [-]	**τ_0 [MPa]**	**M [-]**	**Λ [-]**
2.50	0.035	0.40	20

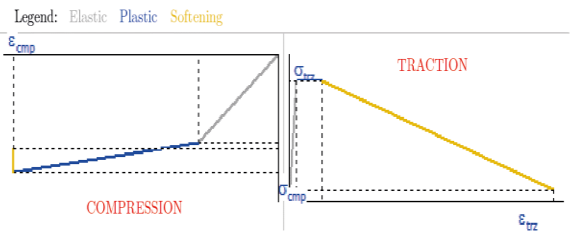


Figure 3. Constitutive law of masonry

TABLE 3. Nonlinear parameters of masonry

Phase	**Parameter**	**Unit**	**Value**
Elastic stage	Max compression stress	MPa	1.50
	Corr. deformation	%	1.22
	Max tensile stress	MPa	0.04
	Corr. elongation	%	0.03
Plastic stage	Max compression stress	MPa	2.00
	Corr. deformation	%	4.00
	Max tensile stress	MPa	0.04
	Corr. elongation	%	0.13
Softening stage	Max compression stress	MPa	4.00
	Corr. deformation	%	1.60
	Max tensile stress	MPa	1.00
	Corr. elongation	%	0.00

$$R_d = \frac{R}{FC} \tag{10}$$

As the level of knowledge increases, the FC factor will be lower. The Italian building regulations identify three knowledge levels.

In the present study, knowledge level LC1 was chosen due to limited investigations (i.e. visual and documentary). The confidence factor is 1.35. For the resistances, the minimum values of the intervals indicated by Circular 2019 are considered; for the elastic moduli, the average values indicated by the same Circular are used.

6. 3. FE Modeling The modelling of the steeple was carried out using the finite element method (FEM), employing TESYS software (MS-DOS solver) [16]. In addition to the mechanical parameters outlined above, further determining factors for the result of the analysis are the degrees of constraint of the structure, offered in this case by the presence of the adjacent blacksmith's bodies (i.e., presbytery). The effects of two calculation models, differing in their constraint conditions, are thus studied:

i. Model A - isolated tower;
ii. Model B - tower constrained in the portions adjacent to the church, which interact with the tower itself.

The tower is modelled with two-dimensional linear triangular elements (CST - Constant Strain Triangle), and is considered constrained to the ground by non-yielding joints.

Table 4 summarizes the results of the pushover analysis.

It can be seen that the isolated (ideal) tower model is not able to sustain the expected displacement demand, in contrast to the real tower model (C/D Index always greater than one).

6. 4. Analysis of the Crack Pattern The evolution of the response of the structure analysed in

TABLE 4. Pushover analysis

Model	**Seismic direction**	**Seismic demand [d_{max} in cm]**	**Seismic capacity [d_e in cm]**	**C/D Index**
Isolated tower	+*x*	15.032	6.197	0.412
	-*x*	14.958	6.577	0.440
	+*y*	15.104	14.095	0.933
	-*y*	15.492	15.717	1.015
Confined tower	+*x*	18.403	21.199	1.152
	-*x*	8.587	8.671	1.010
	+*y*	9.787	46.681	4.770
	-*y*	9.166	37.063	4.044

terms of the crack framework is shown below. In particular, for a bell tower wall, the cracking frameworks for the main points of the capacity curve are shown.

Figure 4 shows the typical trend for a masonry capacity curve.

The points considered are as follows:

i. Point 1: Linear branch of the curve, where the material still exhibits elastic behaviour;
ii. Point 2: yielding of the structure; at some points, plasticisation of the material is observed and the first cracks begin to spread;
iii. Point 3: plastic stretch, where cracks increase and almost the entire structure is in the plastic phase;
iv. Point 4: collapse of the structure.

Figure 5 shows the evolution of the crack pattern for the unconfined tower, with force distribution proportional to the first mode of vibration (-*x* direction).

Figure 6 shows the evolution of the crack pattern for the confined tower, with force distribution proportional to the first mode of vibration (-*x* direction).

The results shown both refer to the wall adjacent to the square, for the analysis in the -x direction. For both models, the first cracks appear at the belfry due to the geometric singularities of the structure. The cracks then

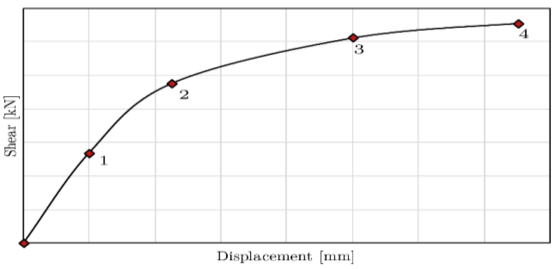
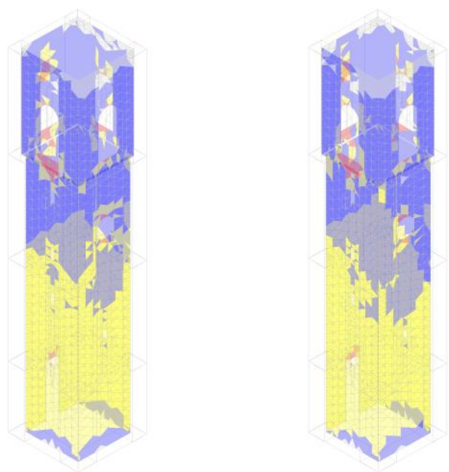
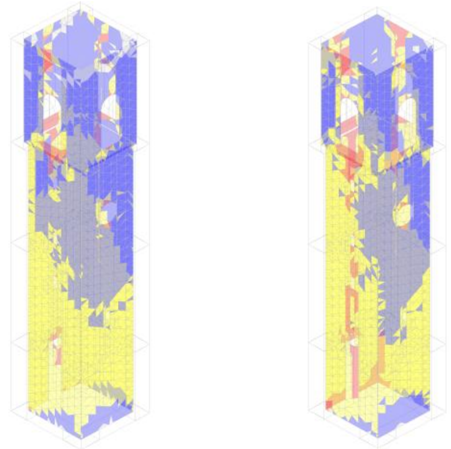


Figure 4. Capacity curve (qualitative)

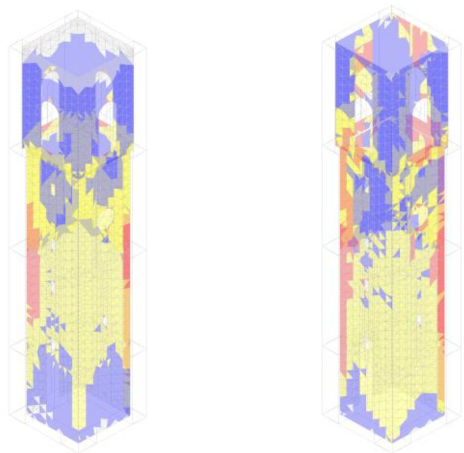


Point 1 Point 2

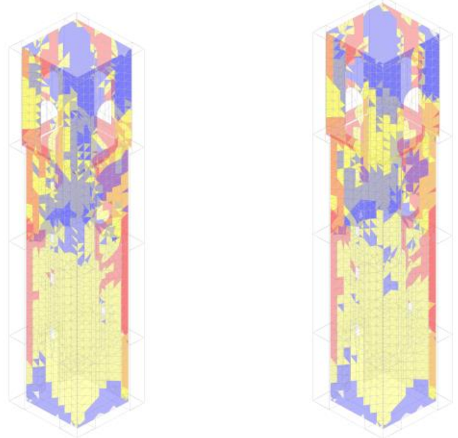


Point 3 Point 4

Figure 5. Damage evolution: unconfined tower with distribution proportional to the I mode of vibration and earthquake direction -*x*



Point 1 Point 2



Point 3 Point 4

Figure 6. Damage evolution: confined tower with distribution proportional to the I mode of vibration and earthquake direction -*x*

propagate from intermediate structural weaknesses (i.e. internal staircase lighting slits), until they reach collapse (Point 4), with cracks spreading along almost the entire shaft of the tower.

It can be seen how the influence of the degree of constraint is fundamental for the structure's crisis fashion: neglecting the interaction of the adjacent building (i.e., isolated tower) results in a different crack pattern at collapse. In fact, in the first case a brittle crisis mode can be considered, while in the second case there is a more diffuse plasticity behaviour, with cracks localized at the top of the structure instead of at the base (generalised collapse).

To validate the crack evolution model, it is possible to observe the two real cases that occurred in Italy (Figure 7 and 8).

The crack patterns observed in the real cases reflects the patterns obtained by the nonlinear static analysis. Figure 9 shows the Capacity Basket for the C/D index (i.e., capacity/demand) of the tower displacement.

Figure 7. Belltower in Castelsantagelo, damaged by the earthquake in 2016 (confined tower)

Figure 8. Belltower in Amatrice, damaged by the earthquake in 2016 (unconfined tower)

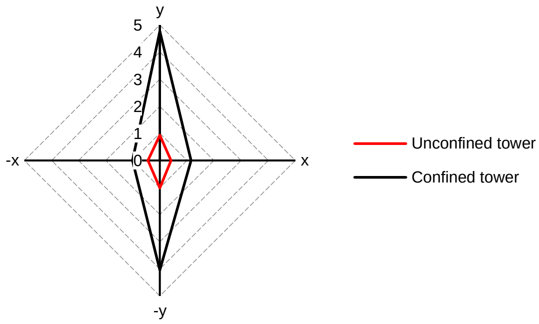


Figure 9. Capacity basket for the Capacity/Demand index with proportional distribution at I mode of vibration and earthquake direction *-x*

It can be observed that in the x-direction the confinement offered by the surrounding walls has the positive effect of doubling the ratio considered and increasing it significantly (+480%) in the y-direction, allowing the structure to meet the displacement requirements for both directions in the confined configuration.

Figure 10 shows the comparison between the maximum displacement results for the 4 nodes at the top of the tower according to the simplified linear model and the pushover analysis.

It can be observed that the linear model allows the displacement of the free cantilever to be predicted with good approximation (i.e., Model-A), while it significantly underestimates the maximum displacement in the case of the confined tower (i.e., Model-B). The effect of the confinement offered by the surrounding masonry is reflected more in the stress distribution in the masonry than in the maximum displacements. In fact, the greatest influence is observed in the mode of propagation of the crack pattern.

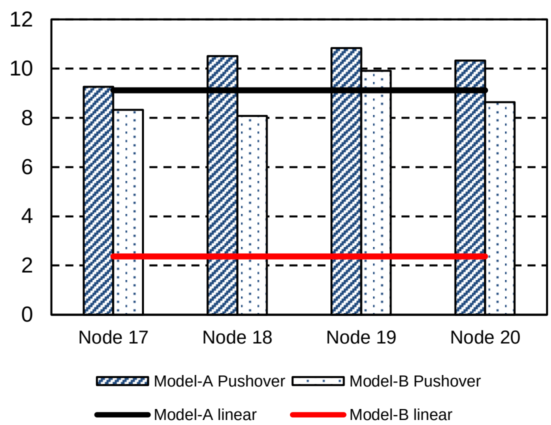


Figure 10. Comparison of the displacement (cm) from simplified linear and pushover analyses

7. CONCLUSIONS

The present study illustrates and compares two different seismic analyses allowed by Italian code on existing masonry monumental buildings (13).

In particular, the effect of the different degrees of constraint on the structure of the bell tower of the church of San Lorenzo in San Buono (Chieti, Italy) has been analyzed. It is chosen as a case study because of the peculiar characteristics of the area on which it stands.

The numerical analyses take into account the uncertainty of the type of material used, considering the various semi-probabilistic factors offered by the Italian technical regulations on existing buildings. The numerical analyses illustrated the difference between the different types of constraint (isolated tower and constrained tower), showing how this factor can significantly influence both the building's failure mode and the possibility of predicting possible hazard scenarios.

The main results are:

- The linear analysis closely approximates the behaviour of the tower structure only in the case of an unconfined structure (i.e., Model-A). The displacement values obtained are comparable in this case;
- The non-linear static analysis (pushover) allows the evolution of the structure's crack pattern to be highlighted;
- The confined tower configuration (Model-B) satisfies the requirements of the structure's seismic demand in all directions and highlights the beneficial effect of confinement offered by adjacent structures;
- The free tower (Model-A) or corbelled configuration is overly cautious.

8. REFERENCES

1. Biraghi MS. Storia dell'architettura contemporanea I 1750-1945: Einaudi; 2008.
2. Curl JS. Lawrence Weaver (Book Review). RSA Journal. 1989;138(5401):63. https://doi.org/10.1080/13556207.1996.10785173
3. Raycheva R. An Empire Style Furniture Set at Nedkovich House, Old Town Plovdiv. Architecture and Urban Planning. 2015;10(1):49-54. https://doi.org/10.1515/aup-2015-0007
4. Zevi B, Zevi B, Zevi B, Architect I, Zevi B. Storia dell'architettura moderna: Einaudi Turin; 1953.
5. Ozturk B. Seismic behavior of two monumental buildings in historical Cappadocia region of Turkey. Bulletin of Earthquake Engineering. 2017;15:3103-23. https://doi.org/10.1007/s10518-016-0082-6
6. Güney D, Aydin E, Öztürk B, editors. The evaluation of damage mechanism of unreinforced masonry buildings after Van (2011) and Elazig (2010) Earthquakes. Journal of Physics: Conference Series; 2015: IOP Publishing.
7. Li S-Q. Empirical resilience and vulnerability model of regional group structure considering optimized macroseismic intensity measure. Soil Dynamics and Earthquake Engineering. 2023;164:107630. https://doi.org/10.1016/j.soildyn.2022.107630
8. Li S-Q, Chen Y-S, Liu H-B, Del Gaudio C. Empirical seismic vulnerability assessment model of typical urban buildings. Bulletin of Earthquake Engineering. 2023;21(4):2217-57. https://doi.org/10.1007/s10518-022-01585-8
9. Li S-Q. Improved seismic intensity measures and regional structural risk estimation models. Soil Dynamics and Earthquake Engineering. 2024;176:108256. https://doi.org/10.1016/j.soildyn.2023.108256
10. Li S-Q, Gardoni P. Empirical seismic vulnerability models for building clusters considering hybrid intensity measures. Journal of Building Engineering. 2023;68:106130. https://doi.org/10.1016/j.jobe.2023.106130
11. Varagnoli C. La costruzione tradizionale in Abruzzo. Fonti materiali e tecniche costruttive dalla fine del Medioevo all'Ottocento, Roma. 2008. https://books.google.it/books?id=ks-C2XRFKH4C
12. Rainieri C, Fabbrocino G, editors. Il periodo elastico delle torri in muratura: correlazioni empiriche per la previsione. XIV Congresso Nazionale L'Ingegneria Sismica in Italia, Bari; 2011.
13. Le Costruzioni NNTP. NTC: Rome. Italy; 2018.

Persian Abstract

چکیده

کلیسای سن لورنزو در سان بونو (ایتالیا) به دلیل ویژگی‌های فنی و معماری خود در منظر محلی منحصر به فرد است. این بنا که تاریخ ساخت آن از قرن چهاردهم تا اواسط قرن بیستم است، توسط مهم ترین طراحان منطقه محقق شد. برج ناقوس عنصری است که با نئوکلاسیسم پالایش شده مشخص می‌شود و گواه بخش قابل‌توجهی از ماندگاری سبک نئوکلاسیک در آبروزو تا دهه‌های اول قرن بیستم است. مطالعه حاضر هم تحلیلی فن‌آوری–سازنده از ساختمان و هم ارزیابی را نشان می‌دهد. خطر لرزه ای برج ناقوس، به منظور ارائه یک نکته مهم برای شناخت این اثر با توجه به کارهای مرمت محافظه کارانه احتمالی. تجزیه و تحلیل خطر لرزه ای برای دو پیکربندی ساختاری مختلف و دو نوع شناسی مختلف مجاز توسط کدهای ساختمان ایتالیایی برای ساختمان های بنایی بنای تاریخی انجام می شود: استاتیک خطی ساده و غیر خطی. این مطالعه نشان می دهد که چگونه حضور سازه مجاور به طور قابل توجهی رفتار ساختاری برج بنایی را هم از نظر جابجایی و هم از نظر انتشار الگوی ترک تغییر می دهد. تفاوت در نتایج محاسبه غیر خطی و روش های سریع پیشنهاد شده توسط استاندارد نیز برجسته شده است.