Seismic vulnerability of the 'Palazzo degli Uffici Statali' a public building realized in Italy at the beginning of last century

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SUMMARY:
This paper deals with the seismic vulnerability assessment of the 'Palazzo degli Uffici Statali', realized in Forlì, central Italy, at the beginning of last century, during the Fascist period. The investigated building is the corner body of a wider RC complex consisting of several structural unities. The structure consists of a basement and six storeys in elevation and presents marked irregularities both in plan and in height. Two types of analysis have been performed: a preliminary vulnerability analysis, following conventional procedures of the current Italian technical code, and a non-linear dynamic analysis, based on a set of accelerograms consistent with seismic hazard characteristics of the building site. Obtained results pointed out significant torsional modes as well as marked problems in brittle mechanisms due to shortage in transverse reinforcement.

Keywords: Existing buildings, RC structures, seismic vulnerability, inelastic response

1. INTRODUCTION

During the Fascist epoch, in a period when architecture was used as a tool for propagandizing and representing the central power, a great number of public buildings was realized in Italy. Since the 30s, architects were inspired by formal principles aimed to give the construction an exterior image of magnificence and firmness, consistent with the idea that political regime wanted to give about itself, so favouring the scenographic and monumental aspects. Many of these buildings were built in areas characterized by significant seismic risk and some of them have already experienced, with opposing results, earthquakes having medium-high intensity (Fig. 1). The assessment of seismic vulnerability of such a significant cultural heritage is therefore a matter of primary importance in Italy; in addition, due to the old age of this type of buildings and to construction techniques used in that period, it is right to ask ourselves whether an image of great solidity may hide problems of static nature, with particular reference to the earthquake-resistant capacity. This study is set in this context and presents the results of a numerical analysis carried out with reference to a building representative of the construction class described above: the 'Palazzo degli Uffici Statali', located in Forlì, a town in central Italy.

The investigated building, which is the corner body of a much larger structural complex, was built in the late 30s on design by well-known architect and engineer Cesare Bazzani, Academic of Italy, wanted at Forlì by Benito Mussolini in person to give prestige to the city whose province had given his birth. The construction was erected near Piazza Aurelio Saffi, one of the most important squares of the city, and underwent several design changes, as evidenced by the numerous drawings kept in both public and private archives (see sources in the References). However, the building in its initial configuration was short-lived: inaugurated in April 1938 (Fig. 2a), it was mined at the top and deprived of the turret by the retreating German army in November 1944; the crash damaged the roof and part of the floors of the fifth storey. Then the urgency and limited budget determined the choice of a partial reconstruction of the complex, which today stands on the square without the crowning turret (Fig. 2b). The passing of the years and the sequence of generations deleted from the collective memory the image of the original complex.

Figure 1. Public buildings of the 30s in L'Aquila (central Italy) affected by the earthquake of April 6, 2009.


a)

b)

Figure 2. The 'Palazzo degli Uffici Statali' after inauguration (a) (image taken from the Newspaper Library of Forlì, see References) and in a recent picture (b).

2. SURVEY OF THE BUILDING

2.1. Geometrical and structural characteristics

Acquisition of geometrical characteristics of the building, particularly of structural elements, was conducted through direct measurements taken in the edifice and, most of all, by consulting technical documents and drawings belonging to the personal archives of Cesare Bazzani and to the Office of Civil Engineering in Forlì (see reference section).

The complex of the 'Uffici Statali' consists of several buildings, all with RC frame structure, located along the perimeter of a large block of almost rectangular shape. The building under investigation, positioned at the corner facing Piazza Saffi, has a greater height with respect to the adjacent bodies, from which it is separated by means of technical joints having a dimension of about 50 mm. The structure consists of a basement and six storeys above ground, with an approximately square plan.

Consulted documents (contract specifications) showed that building foundations, originally planned as non-reinforced concrete beams, are actually made up of a RC webbed slab (Fig. 3) supported by timber piles having an average diameter of 0.21 m and a length of 6.5 m to reach the ground level with a sufficient mechanical strength (layer of clay and sand, safety load equal to 0.18 MPa). Before the insertion, piles were subjected to axial load tests which gave a safety load of 85 kN. They were placed at intervals of 0.7 m, along parallel joggled lines; the only corner body needed about a thousand piles.

Concerning the upper structure, obtained information are less detailed because of lack of some design drawings. From the available documents it was possible to achieve sizes of all the structural elements, while only in some cases it was possible to find both amount and position of reinforcements. Distribution of columns in plan is very inhomogeneous, with massive cross-sections along front sides and smaller sections elsewhere; in addition, columns in general reduce the cross-section from foundation to the top, except for elements belonging to sides adjacent to the two neighboring bodies; in particular, they present a smaller cross-section at storeys 1, 2 and 3 above ground with respect to storeys 4 and 5. Some columns placed near technical joints have a rather articulated cross-section geometry, forming a ‘grove and tongue’ coupling with corresponding elements of the adjacent buildings (see Fig. 3). Edge beams have L-shaped cross-sections, with the web emerging from extrados, while internal beams have rectangular cross-sections emerging from intrados. Floors consist of RC slabs with a thickness of 0.15 m, dimensioned for a service load of 3 kN/m^2.

Cladding of perimeter frames are made of brick masonry, arranged in such a way as to obtain different thicknesses between main fronts and back closures; the most common type is that constituted by two curtains of bricks with an interposed cavity. The internal partition walls are made of bricks placed according to the lesser dimension.

The whole of the above information outlines a structural framework characterized by marked irregularities, both in plan and elevation, due to non-uniformity in stiffness and strength distribution mainly due to variations in columns cross-sections.

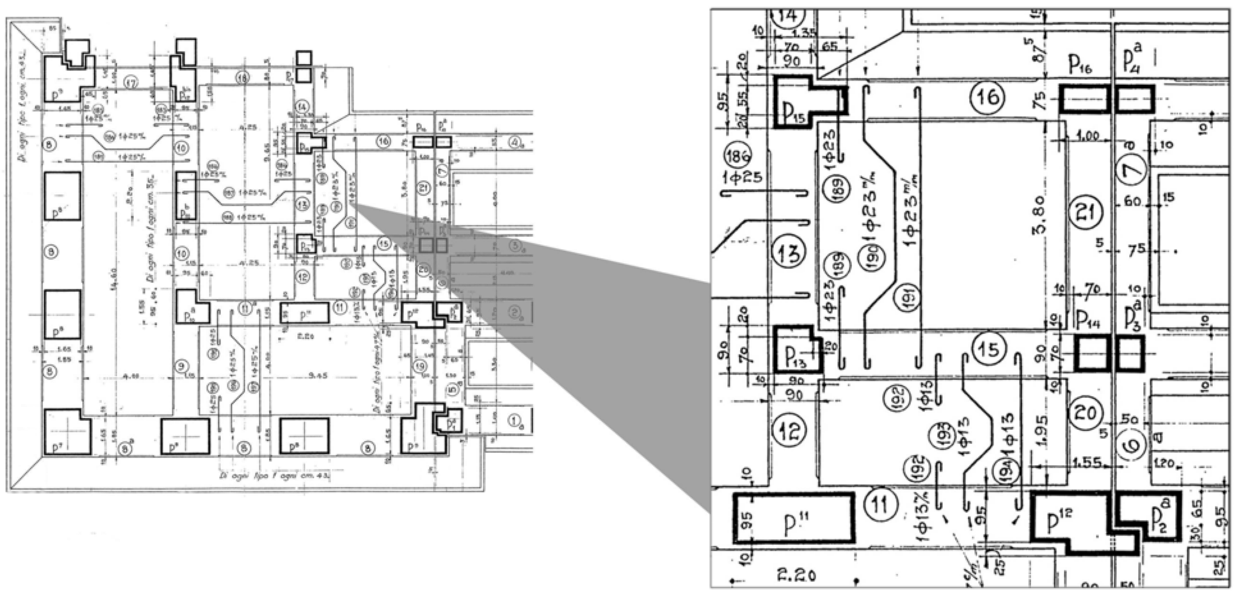

Figure 3. Original drawing of the foundation structure.

2.2. Measurements ‘in situ’ for mechanical characterization of materials

Concerning the upper structure, available documents only in rare cases provided explicit information about beams and columns reinforcements and materials strength. In order to obtain a sufficiently reliable estimate of not available data, both requirements of technical regulations in force in Italy at the time of building realization and non-destructive tests ‘in situ’ have been used.

The structural complex was designed in the past 30s, under seismic regulations of national character represented by the Royal Decree (R.D.) of 1924 (first law not aimed at reconstruction), which introduced in Italy seismic zones of first and second category. From the contract specifications of the complex, it was also drawn that all the structures were built using homogeneous steel (soft steel) and a concrete with uniform volumetric proportions given by 3.5 kN of Portland cement, 0.4 m^3 of sand, 0.8 m^3 of gravel and 120 liters of water, in accordance with standards provided by the R.D.L. n. 1213 of 1933; in particular, this document prescribed for the reinforced concrete:

– Cast or homogeneous steel (soft steel) in bars with a tensile strength between 3800 and 5000 kg/cm^2 (380 and 500 MPa) and with an elongation at rupture of not less than 27% and 21% respectively.

– Normal concrete having a minimum strength of 300 kg/cm^2 (30 MPa) with the volumetric ratio of 300 kg (3 kN) of Portland cement, 0.4 m^3 of sand, 0.8 m^3 of gravel and 120 liters of water.

The above information were integrated with the results obtained from sclerometric tests performed on a sample of structural elements of the building. They provided uniform values with respect to the tested elements, with an average value of the cubic strength equal to about 35 MPa. Based on the obtained information, and considering the possible strength degradation due to aging, for the purposes of subsequent numerical investigation the following value of concrete strength (medium strength) f_{cm} = 25 MPa has been assumed. Concerning reinforcing steel, medium values of yield and ultimate strength equal respectively to f_{ym} = 270 MPa and f_{tm} = 420 MPa have been assumed, according to requirements of the R.D.L. of 1933.

To complete the survey, magnetometric measurements have been carried out on a significant number of structural elements; they allowed to estimate tie spacings (200 ÷ 400 mm for beams, 200 ÷ 300 mm for columns), while it was not possible to evaluate diameters and amounts of longitudinal reinforcement. To solve this problem, reinforcements have been dimensioned according to prescriptions of the Italian technical code in force at the time of construction (simplified calculations for beams, assumed as continuous beams subjected to increased vertical loads; columns reinforcement as a percentage of the concrete section) and on the basis of construction practice deducible from available documents.

In addition to information concerning geometrical and mechanical characteristics of the building, developed survey revealed a structure in good state, lacking in both significant cracking patterns and signs of material deterioration.

3. VULNERABILITY ANALYSIS

3.1. Conventional elastic analysis

In order to a preliminary assessment of the ability of the investigated structure to undergo actions consistent with the seismic hazard of the construction site, a vulnerability analysis was firstly performed according to procedures established by the technical code currently in force in Italy (NTC 2008). In particular, as a result of the building survey described in the previous section, an intermediate level of knowledge (LC2) was assumed; it allowed, in turn, to define the type of analysis (modal analysis with response spectrum) and the confidence factor (FC = 1.2). Assessment of safety levels was conducted in relation to the ‘life safety limit state’ (SLV).

A modal analysis has been then developed, applied to a three-dimensional model representative of the building, taking into account its characteristics of non-regularity mainly due to the non uniform distribution of lateral stiffness (Nudo & Acciai 2008). As regards the definition of the design spectrum, soil type E was assumed; corresponding PGA was equal to 0.204 g and the structure factor was set equal to 2.5.

According to code requirements, safety verifications were conducted with reference to both ductile (bending and axial force) and brittle (shear force) mechanisms. In particular, the first type of verification was performed in terms of deformation on the basis of the following condition:

$$\theta \leq \theta_{SD} = \frac{3}{4}\theta_u^{'} \tag{3.1}$$

where θ is the 'chord rotation' valued at each end sections of the considered element, and θ_{SD} the corresponding rotational capacity referred to the SLV limit state, given by:

$$\theta_u^{'} = 0.85\,\theta_u \tag{3.2}$$

$$\theta_u = \frac{1}{\gamma_{el}} 0.016 \times 0.3^{\nu} \left[\frac{\max(0.01;\omega^{'})}{\max(0.01;\omega)} f_c\right]^{0.225} \left(\frac{L_v}{h}\right)^{0.35} 25^{\left(\alpha\rho_{sx}\frac{f_{yw}}{f_c}\right)} \left(1.25^{100\rho_d}\right) \tag{3.3}$$

θ_u being the ultimate rotation at collapse. As can be seen from Eqn. 3.2, a reduced value of the ultimate rotation has been assumed, considering that the structural elements are not provided with appropriate details of seismic type in terms of transverse and longitudinal reinforcement. For the meaning of the quantities contained in Eqn. 3.3, see the paragraph C8A.6.1 in the Appendix to Ch. 8 of the Instructions for the NTC 2008 (2009); in particular, in the same expression the factor α has been set equal to zero, since it was not possible to ascertain whether tie anchorage provides or not a suitable confinement level.

Besides, verifications for brittle mechanisms were conducted in terms of strength, based on the following condition:

$$V \leq V_R \tag{3.4}$$

V being the shear force provided by the analysis and V_R the corresponding shear capacity evaluated as specified in the next paragraph. In evaluating capacity of structural elements, the medium strength of materials (R_m) was reduced as follows:

$$\begin{aligned} &R_m / FC && \text{(for ductile mechanisms)} \\ &R_m / (FC \times \gamma_M) && \text{(for brittle mechanisms)} \end{aligned} \tag{3.5}$$

being FC the confidence factor (= 1.2) and γ_M the material partial factor.

Verification procedures, and in particular those related to the shear force, revealed that a significant number of structural elements does not meet code requirements.

3.2. Non-linear dynamic analysis

The elastic analysis described in the previous paragraph pointed out a marked vulnerability of the investigated building with respect to brittle mechanisms (shear force). For a more complete evaluation of the seismic performance of the structure, a non-linear dynamic analysis has been therefore developed, based on a set of accelerograms providing an average elastic spectrum consistent with the one stated by Italian code prescriptions for the limit state SLV and for soil type E.

Table 1 shows the overall data of the selected records used for definition of seismic input, inclusive of peak accelerations in the x, y and z directions (Iervolino et al. 2009); it is underlined that only some of these components (x or y) were used for the construction of response spectra, as shown in Figure 4; in the same figure a schematic plan of the building has been reported, together with the column numbering and the assumed direction for the seismic action.

Table 1. Characteristics of selected records.

Waveform ID	Station ID	Earthquake Name	Date	Mw	Epicentral Distance [km]	PGA_X [m/s^2]	PGA_Y [m/s^2]	PGA_Z [m/s^2]	EC8 Site class
791	AQK	L'Aquila Mainshock	06/04/2009	6.3	5.6501	3.4678	3.2373	3.5546	B
181	STR	IRPINIA EARTHQUAKE	23/11/1980	6.9	33.2615	2.2088	3.0972	2.1013	B
790	AQG	L'Aquila Mainshock	06/04/2009	6.3	4.3919	4.7927	4.3743	2.3462	B
014	TLM1	FRIULI EARTHQUAKE 1ST S	06/05/1976	6.4	21.7205	3.3904	3.0901	2.5873	B
171	CLT	IRPINIA EARTHQUAKE	23/11/1980	6.9	18.8553	1.55	1.7177	1.6275	B
181	STR	IRPINIA EARTHQUAKE	23/11/1980	6.9	33.2615	2.2088	3.0972	2.1013	B
091	GMN	FRIULI EARTHQUAKE 3RD S	15/09/1976	5.9	5.2292	3.1832	6.3642	4.7634	B

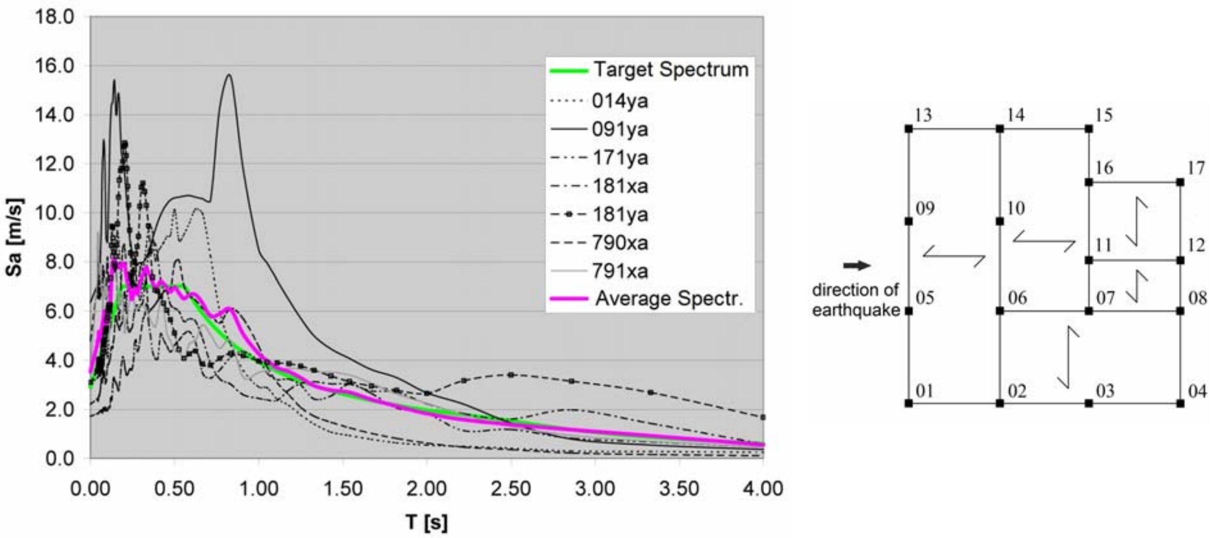

Figure 4. Elastic spectra of selected records and schematic plan of the building.

To define the response of the building generated by the selected seismic input, time-histories of parameters representing inelastic response of the investigated building were evaluated using the SeismoStruct computer code (2011); in particular, both chord rotation θ and shear force V in all the structural elements were calculated for verifications in terms of deformation (ductile mechanisms) and stress level (brittle mechanisms). Figures 5 and 6 show profiles of chord rotation and shear force in columns at different storeys; in particular they illustrate the average values of chord rotation and shear force to the corresponding capacities (θ/θ_{SD} and V/V_R).

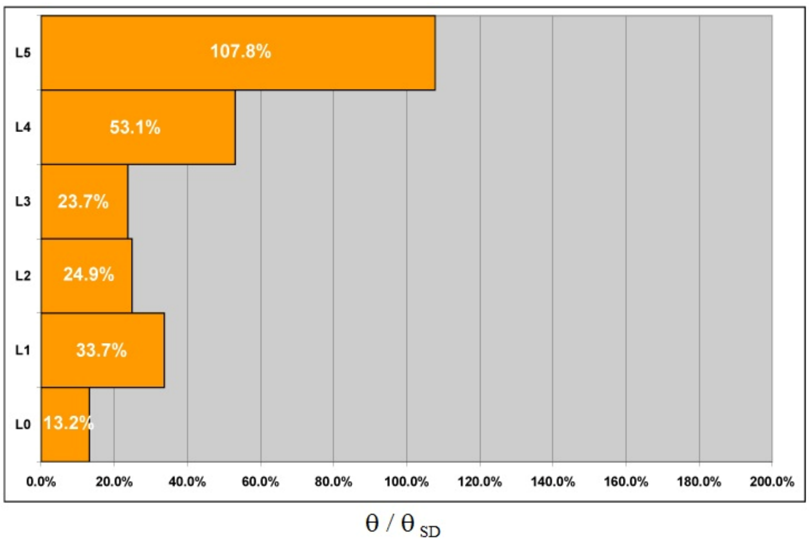

Figure 5. Average values of θ/θ_{SD} for columns at different storeys.

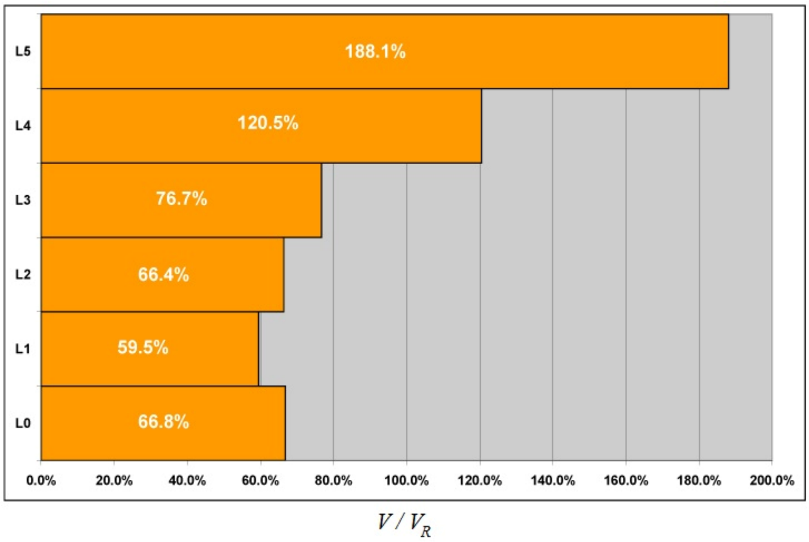

Figure 6. Average values of V/V_R for columns at different storeys.

Numerical results showed that average values of parameter θ resulted, at every storey except for the fifth one, lower than the corresponding capacity; this has been conventionally evaluated by the Eqn. (3.1), (3.2) and (3.3) using both a constant value of the shear span ($L_V^{'} = L / 2$, with L element length) and an instantaneous value of the same ($L_V^{''} = M / V$). Different results were obtained in the case of shear forces, since they often resulted higher than the respective capacity, especially in columns of storeys 4 and 5 where the shear strength was reduced also due to the lower axial load. Concerning evaluation of shear strength, in the case of columns the inelastic relationship 'shear force-transverse displacement' has been carried out (De Stefano et al. 2006), using the static value of axial force provided by the analysis.

Time-history of chord-rotation at a column end is reported in Figure 7, referred to the corresponding element capacity.

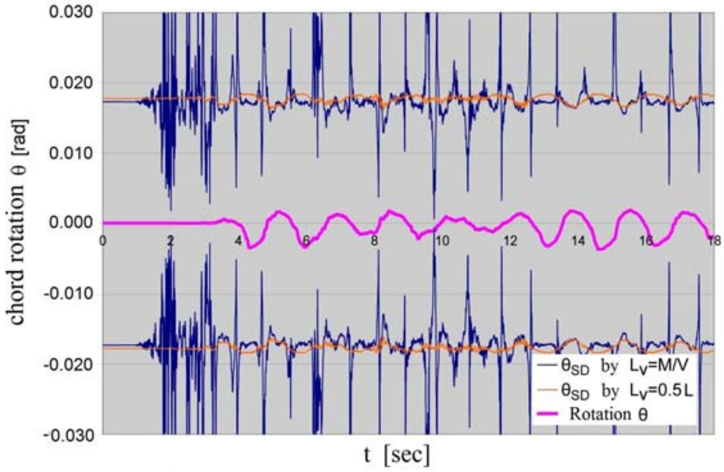

Figure 7. Time-history of chord rotation θ at the upper end of column n. 17, third storey (record 791xa).

The obtained results provide further evidence that the problem of shear is what most characterizes the seismic response of the building under investigation. The shear profile in Figure 6 reveals an inadequacy of shear strength at the upper storeys of the building due to the insufficient amount of transverse reinforcement. It is therefore confirmed the condition of seismic vulnerability already pointed out by conventional analysis developed in the elastic range.

4. BUILDING UPGRADE TO IMPROVE SEISMIC PERFORMANCE

The analysis of results provided by the model representing the building under present conditions pointed out activation of significant torsional modes due to the irregular distribution of lateral stiffness. For that reason a traditional upgrading intervention was hypothesized, based on the insertion of metal bracing (diagonal steel elements) positioned at the less stiff sides of the building, starting from the first storey (thus excluding the basement and ground floor). Through a suitable calibration of steel element sections it was possible to regulate lateral stiffness distribution. The reconstruction of the turret was also considered, assuming for it a steel structure. Figure 8 illustrates the geometric model with highlighting of the frame meshes interested by bracing elements; in the model, the lower ends of columns of the first level were assumed as fixed joints.

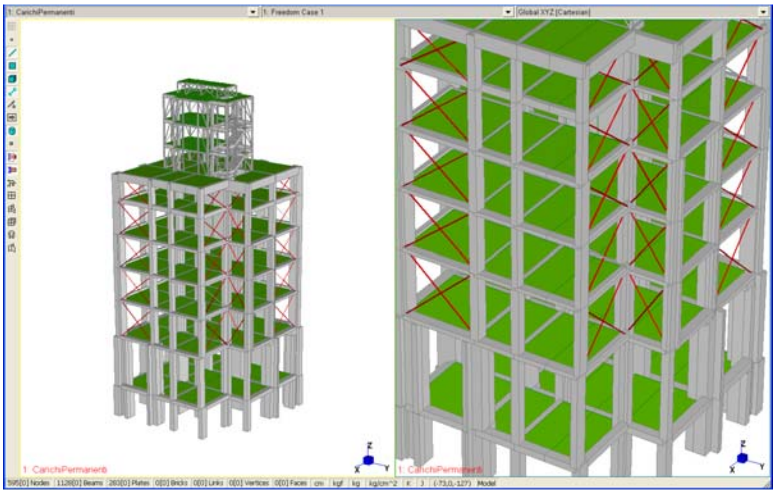

Figure 8. Building model with the turret and metal bracings.

Figure 9 illustrates deformation modes characterized by the maximum torsional components, before and after the intervention in the hypothesis; from comparison, a significant reduction in rotational components, in the case including structural rehabilitation, can be observed.

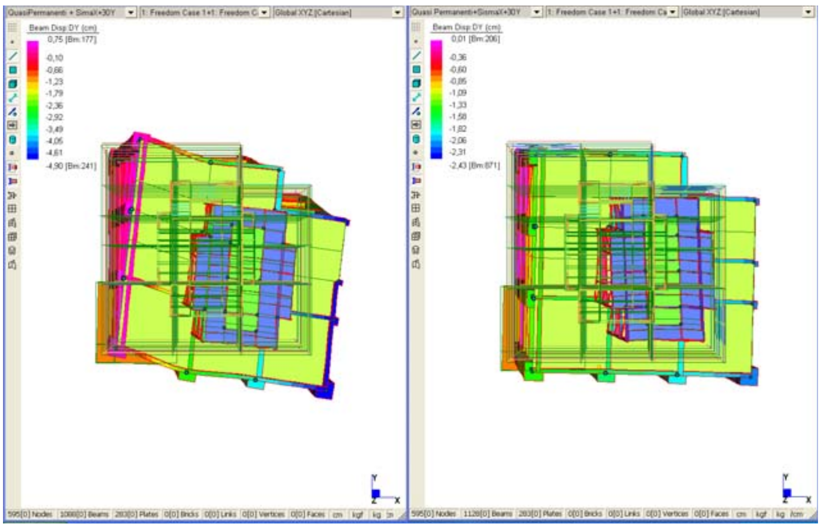

Figure 9. Torsional modes before (left) and after (right) insertion of metal bracings.

The results of the analysis therefore indicate the effectiveness of the suggested regularization in terms of translational stiffness, resulting in a reduction of both deformation demand and stress level. As regards conventional verifications, the intervention allowed to verify ductile mechanisms, while it was not possible to completely solve problems connected to the excess of shear force.

For this reason it may be suitable to consider alternative strategies to improve seismic performance of the investigated buildings. Among different innovative methods for seismic protection, passive energy dissipating devices, such as friction or viscous dampers, are currently being used worldwide (Levy et al. 2010).

5. CONCLUSIONS

A seismic vulnerability analysis was developed for the ‘Palazzo degli Uffici Statali’, a building representative of a broad building category on Italian territory, that is public buildings realized during the Fascist period. The investigated building, in particular, is characterized by an irregular distribution of stiffness and strength both in plan and elevation, mostly due to the presence of columns with significant variations in cross-sections and reinforcement. These morphological and mechanical characteristics, as is known, can expose the structure, in case of a seismic event, to a concentration of plastic demand in limited regions, with a consequent possibility of activating crisis mechanisms of local type (brittle mechanisms).

To define the seismic response of the above construction a vulnerability analysis was initially performed, according to procedures provided by Italian technical codes (NTC 2008) for existing buildings. For this purpose, authors made use of the large wealth of information gained through consultation of technical documents and execution of non-destructive investigations as well. Numerical results, obtained in this phase of the study, showed a critical issue of the structure in relation to shear forces. In order to improve the structural response an upgrading intervention was hypothesized to settle the distribution of stiffness through the insertion of metal braces in the meshes of the perimeter frames characterized by lower lateral stiffness. This measure has in fact resulted in a significant reduction of stress and deformation levels; nevertheless it was not possible to completely solve the problems associated with lack of shear reinforcement.

For a more complete investigation, a non-linear dynamic analysis was also performed, based on a seismic input consistent with the characteristics of the building site. As inelastic parameters, representing building response, chord rotation and shear force at column ends were evaluated. Also in this case numerical results pointed out an excess of shear stress in several structural elements, mostly located at the upper storeys of the structure; on the contrary, deformation parameters were almost compatible with the respective capacities.

It was hypothesized an upgrading intervention based on traditional techniques (insertion of metal bracings) that did not allow to completely satisfy safety verifications. In this case it is therefore necessary to consider a rehabilitation intervention based on more effective methods for seismic protection such as those based on passive energy dissipation.

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