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Ground penetrating radar survey for structural assessment of a heritage sport building

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Abstract. In order to prevent seismic damage on building heritage built before Seismic Standards, constructions require to be assess to verify the structural response in the case of multi-level intensity seismic actions. This problem especially concerns those buildings with a social function as schools, hospitals, etc., or with historical and architectural value as that designed by important builders of the past.
This is the case of the bar-restaurant building of "Bellariva" Sport Centre, designed and built in Florence by the World-famous Italian engineer Pier Luigi Nervi in the Sixty years. Its structure is characterized by reinforced concrete frames and hosts the locker rooms of the swimming pool and a bar on the first floor, a restaurant on the second, where a long crack was observed. The presence of a large balcony with heavy perimeter planters near the cracked zone motivated the execution of on-site tests finalized to determine the steel bars connecting the restaurant floor to the balcony. A Ground Penetrating Radar survey was performed in order to determine the internal structure of the floor, dimensions and disposition of steel bars, and to gather information about the connections between perimeter beams and balcony at the level of the restaurant. The experimental campaign allowed to refine a computational Finite Element Model that was utilized for the performance analysis of the structure in the current state. The paper presents the main results of a preliminary seismic analysis carried out on the structure, on the basis of which some retrofit intervention are suggested.

1. Introduction

The high vulnerability of some constructions built in Italy before the issue of the Seismic Standards [1, 2], makes it necessary to plan investigation campaigns to assess the response of the same buildings subjected to seismic actions, especially when they have a social function like hospitals and schools, or have an historical and architectural value. Among this last type of constructions can be considered the bar-restaurant building of the "Bellariva" Sport Centre, built in Florence in the Sixty years of the last Century by the World-famous Italian engineer Pier Luigi Nervi and his son Antonio. This paper presents a summary of the phases of geometric reconstruction, carried out by means of the consultation of original design tables and the execution of on-site surveys, of modeling and final seismic analysis of the structure in current state conditions.

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2. The structural assessment of “Bellariva” bar-restaurant building

Seismic vulnerability analysis of a construction of historical-architectural interest requires a careful preliminary investigation aimed at redrawing the main structural details, starting from the original design documentation. With regard to the case study examined in this paper, a consistent part of the original design was found at the historical CSAC archive in Parma. In a next step of the study, the original design must be then compared to the actual structure, in order to define a computational model to utilize for seismic analysis of the construction. Regarding this, it can be observed that the actual strength properties of the materials constituting main structural members schematized in the same model should be confirmed by on-site testing campaign as those performed for the bar-restaurant “Bellariva” building. Next figures 1-3 show the plans of the ground floor, which houses the dressing rooms and the bar (figure 1), of the first floor where the restaurant is located (figure 2), and the roof (figure 3).

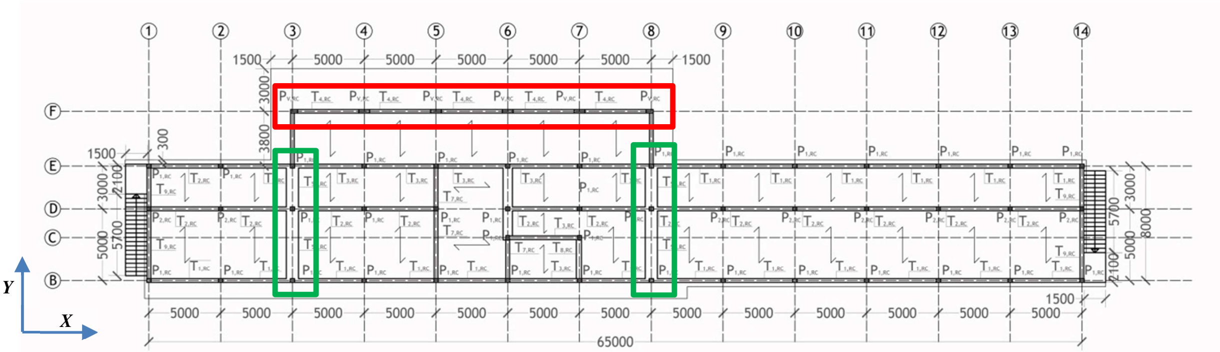

Figure 1. Ground floor plan of the building with numbering of beams and columns; localization of columns with variable section, squared in red, and of columns laterally placed with respect to the bar, squared in green.

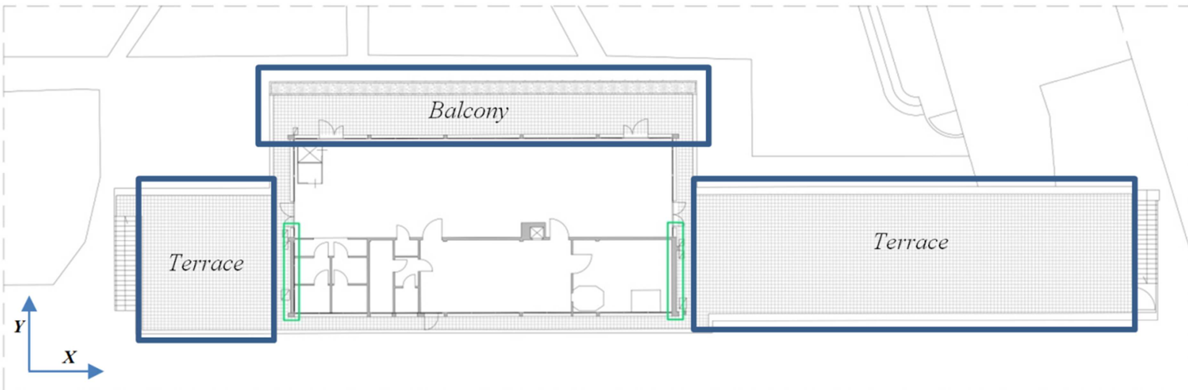

Figure 2. First floor plan with localization of lateral columns of the restaurant (squared in green), terraces and balcony (squared in blue).

2.1. *Description of the structure*

The building has a reinforced concrete structure with an irregular plan, characterized by maximum dimensions of 64.35×13.60 m×m on the ground floor, 25.05×11.40 m×m on the first storey. On this last level the restaurant has two large 8.5 m wide terraces on the sides, while in front there is a 1.5 m wide balcony (figure 2). In figure 4 are represented the main sections of central columns having dimensions of (260×300) mm×mm and (300×300) mm×mm in the ground floor, reduced to (200×250) mm×mm in the second storey. Perimeter columns squared in green on the plan in figures 1 and 2, have axes that are not aligned in the two interstoreys due to the presence of cavities passing through them.

As shown by the details in figure 4, they have C-type sections in the first storey (figure 4e), rectangular-type with dimensions of 420×250 mm×mm, in the upper level (figure 4f). The six columns squared in red in figure 1 have a section variable in height as detailed in figure 5 with reference to the central alignments, in figure 6 for the lateral positions. This particular shape is a characteristic also of other important structures designed by Nervi in the same time such as the Washington Bridge Bus Station in New York, and the "Palazzo del Lavoro" in Turin [3].

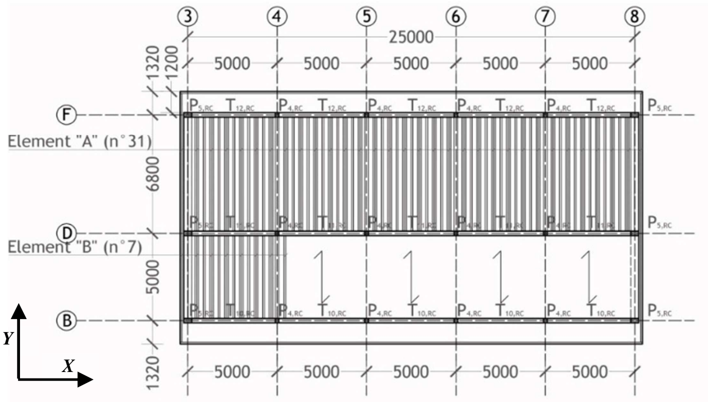

Figure 3. Roof plan with numbering of beams and columns.

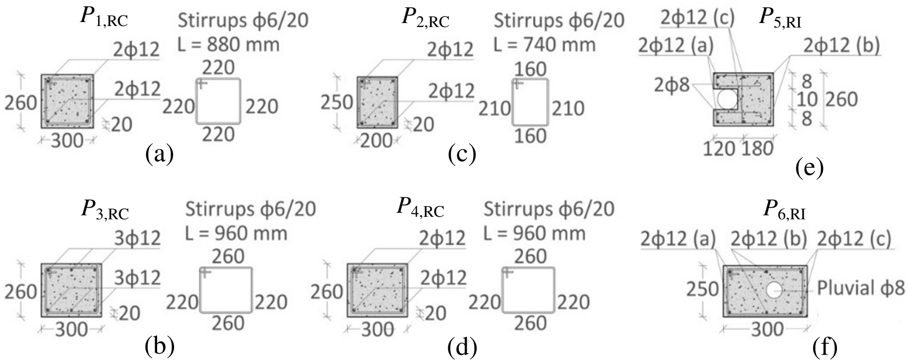

Figure 4. Structural details of columns with constant sections in the first (a, c, e) and second (b, d, f) interstoreys.

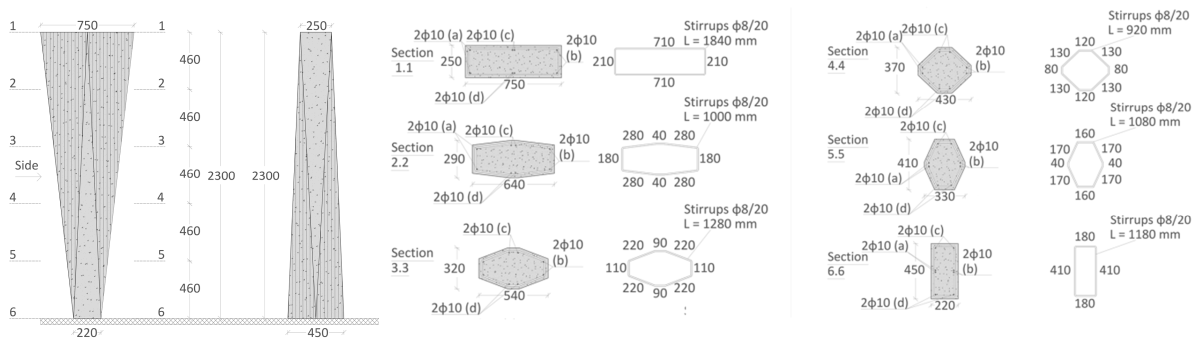

Figure 5. Front view and sections of an internal column of the first interstorey with variable dimensions and shape.

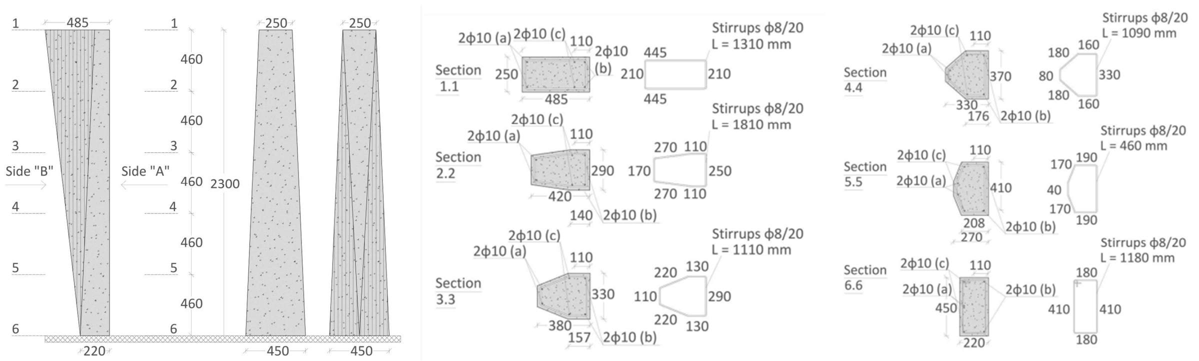

Figure 6. Front view and sections of a lateral column in the first interstorey with variable dimensions and shape.

R/C beams have section dimensions, steel bars and stirrups detailed in table 1, identified according to the nomenclature reported in figures 1-3.

Table 1. Sections, steel bars and stirrups of beam elements.

Beams	Dimensions (mm×mm)	Floor	Half-span section		End section		Stirrups
			superior bars	inferior bars	superior bars	inferior bars	
T1	260×500	1F	2ϕ8	6ϕ16	6ϕ16+2ϕ8	2ϕ16	ϕ8/20
T2	260×600	1F	2ϕ6	6ϕ12	6ϕ12+2ϕ6	2ϕ12	ϕ8/20
T3	260×550	1F	2ϕ6	6ϕ12	6ϕ12+2ϕ6	2ϕ12	ϕ6/20
T4	250×550–250×750	1F	2ϕ12	8ϕ16	3ϕ16+2ϕ12	2ϕ16	ϕ8/20
T5a	850×260	1F	4ϕ6	8ϕ16	6ϕ16	4ϕ16	ϕ6/20
T5b	850×250	1F	4ϕ16	4ϕ16	3ϕ6+3ϕ16	4ϕ16	ϕ6/20
T6	250×500–250×750	1F	2ϕ12	6ϕ16	7ϕ16/3ϕ16	4ϕ16/6ϕ16	ϕ8/20
T7a	260×500	1F	2ϕ6	6ϕ12/5ϕ12	5ϕ12	3ϕ12	ϕ8/20
T7b	260×500	1F	2ϕ6	4ϕ12/5ϕ12	2ϕ12/2ϕ6	2ϕ12	ϕ6/20
T7c	260×500	1F	2ϕ6/2ϕ6+2ϕ12	4ϕ12/5ϕ12	4ϕ16+2ϕ6	2ϕ12	ϕ6/20
T8	250×450×10×500	1F	1ϕ8+2ϕ10	4ϕ10	1ϕ8+2ϕ10	4ϕ10	ϕ6/20
T9	260×200	1F	2ϕ12	2ϕ12	2ϕ12	2ϕ12	ϕ6/20
T10	250×450	2F	2ϕ8	4ϕ16/5ϕ16	2ϕ16/4ϕ16	2ϕ16	ϕ8/20
T11	250×650	2F	2ϕ8	5ϕ16/6ϕ16	2ϕ16/4ϕ16+2ϕ8	2ϕ16	ϕ8/10-20
T12	250×370	2F	2ϕ8	6ϕ16/7ϕ16	3ϕ16/6ϕ16+2ϕ8	3ϕ16	ϕ8/10-20

The first floor structure is composed by prefabricated R/C joists, with length of 5.0 m, 3.0 m, and 3.8 m in three different zones. The second floor is instead partly constituted by two types of V-section prefabricated tiles, shown in figure 7. The remaining portion of the floor is of the joist-brick type, with slab thickness ranging from 30 mm to 50 mm. Two external side staircase with single ramp allow the vertical connection between the two floors; they are supported by knee beams contained within the infill walls.

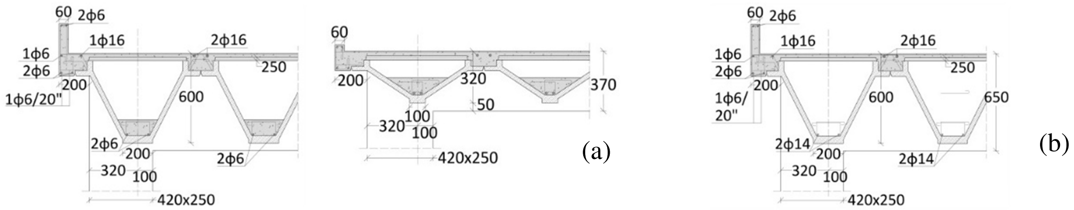

Figure 7. Prefab V-shaped beams of type A, with variable section (a), and of type B, with constant section (b).

2.2. *On-site experimental tests*

An extensive on-site testing campaign was carried out on the building to identify the mechanical characteristics of the constituting materials and draw the main structural details, starting from the original design documentation. The on-site testing programme consisted of: core drillings, pacometer tests, and extraction of steel reinforcement samples on the ground storey R/C members. The following main properties of the constituting materials resulted from the characterization tests: mean cubic compressive strength of concrete equal to 37.35 N/mm^2 (corresponding to a concrete material of class R/C 35/45) for beams and columns; to 45.65 N/mm^2 (class-type R/C 45/55) for the prefab beams; 21.25 N/mm^2 (class-type R/C 20/25) for the slab; yield stress of reinforcing steel equal to 304.11 N/mm^2, corresponding to a AQ 60 steel-type, according to the nomenclature adopted at the time the building was constructed. The information gained from the testing campaign made it possible to reach the highest "knowledge level" (named LC3) established by the current Italian Technical Standards [4, 5] in the structural assessment analysis of existing buildings. The corresponding value of the "confidence factor" FC is equal to 1.

The pacometric surveys carried out generally allowed verifying the positions and diameter dimensions of the most superficial reinforcements, but not the detail of anchoring longitudinal bars of the terrace to the floor of the restaurant, a section detail of which is shown in figure 8a, extracted from the original design project. The verification of this information was made necessary by the detection, on the floor of the restaurant, of a long crack parallel to the *X* direction, as shown on the plan in figure 8b, perhaps generated by the execution of an elevator, placed in the East corner of the building, several years after its construction. Aiming this, the limit of pacometer tests was overcame by means of more refined instruments such as Ground Penetrating Radar (GPR), commonly used for investigating concrete pavement, floor and for identifying steel bars placed in deeper alignments.

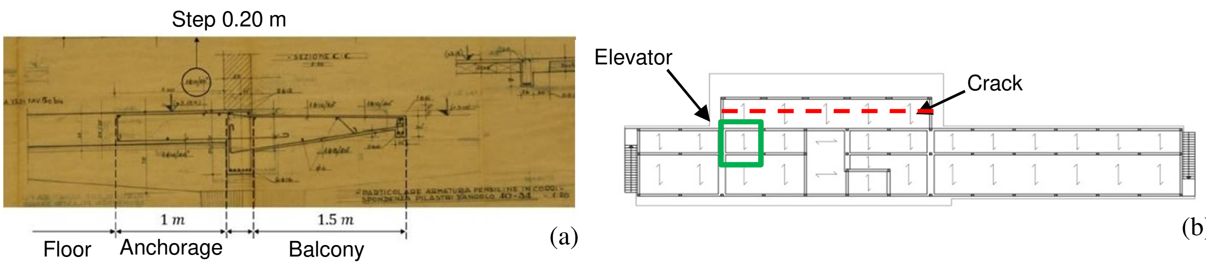

Figure 8. Original drawing of a detail of the balcony section (a), and representation in plan of the crack in the first floor.

The GPR survey allows to gather information about structures without any destructive test [6, 7]. The aim of this GPR survey is to investigate the structural characteristics of the damaged floor, to identify the spacing of steel bars and to gather information about the connections between the edge beams and the cantilever balcony. Indeed, the original drawings of the long side of the balcony have been lost and there are no evidences about the type of the beam - cantilever connection. A drawing of the short side is shown in figure 8a. In this side the balcony is 1.5 m large and the bars pass through the perimeter beam for 1 m. The bars are positioned with a step of 0.20 m. For analogy we presume to have the same disposition of bars on the long side of the balcony where the bars are passing through the beam for 2 m. In figure 9 is shown the GPR equipment used. This system, called ORFEUS, was designed in the frame of a European Project [6]. As shown in figure 10, 12 longitudinal scans (L-scans) and 42 transversal scans (T-scans) on the internal floor were acquired with the GPR. Seven L-scans and one T-scan were acquired on the balcony. The step between two parallel scans was 0.5 m. The GPR results of this survey were reported in [7]. For sake of brevity in the following section only the useful radar gram has been shown.

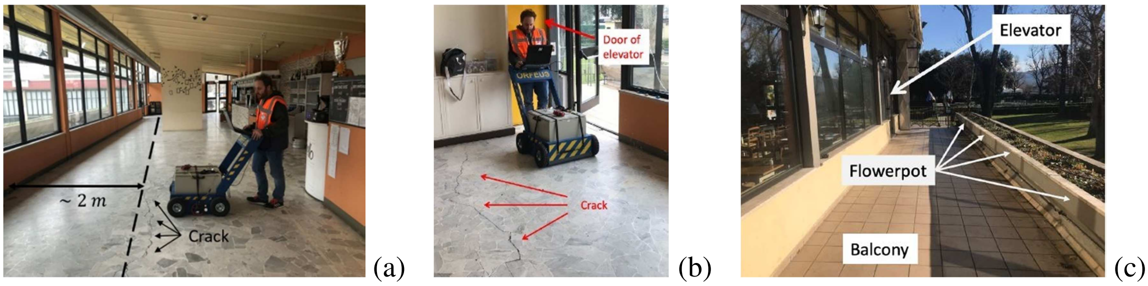

Figure 9. Detail of the crack on the first floor and of the ORFEO GPR radar, (a) and (b); detail of the balcony and flowerpot (c).

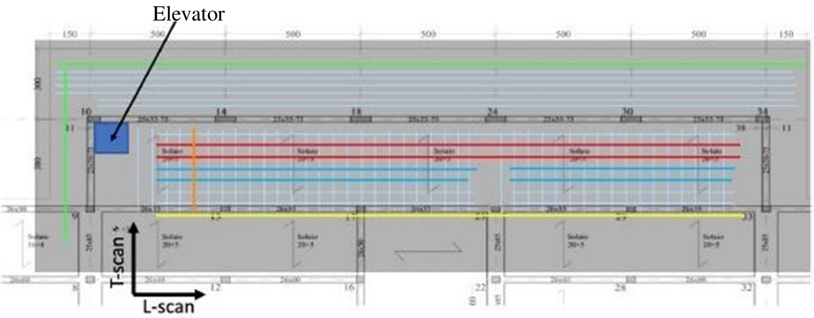

Figure 10. Scan grid of the Bellariva survey.

Figure 11 shows the results of the 3rd L-scan (in red in figure 10). This scan was taken at 1.5 m from the edge beam therefore the targets represent the bar connected to the edge beam. In the 3rd scan it is possible to count 47 targets in 10 m with an average step of 0.21 m. The same bars density was measured in the 2nd scan (in red in figure 10) These results confirm the analogy with short side of the balcony (figure 8a). In the 4th and 5th L-scan (in light blue in figure 10) the average step of targets was 0.52 m. The results of 4th scan is reported in figure 11b. The two consecutive L-scans (the 6th and 7th) gives an analogous result. Therefore, in the middle of the floor span the bars were positioned with a step of 0.5 m. In the 3rd L-scan (and also in the 2nd L-scan [7]) a echo is visible at 0.5 m depth. In the 4th (and also 5th L-scans [7]) this continuous echo is not visible. By comparing this result with the original drawing in figure 8a we can deduce that the steel bars from balcony pass through the perimeter beam for more than 1.5 m. The 8th L-scan (in yellow in figure 10) was performed close to a beam and it is able to survey three different floor-span [7]. From the starting of the scan to 7.25 m and from 12 m to the end we can measure 3.6 targets each meter with an average step between bars of 0.28 m. Indeed, in these areas the bars from different spans floor are overlapped on the beam and the step has to be about 0.25 m. In the central area (from 7.25 m to 12 m) of 8th L-scan the absence of reinforcement is probably due to a different orientation in that part floor. Anyway, due to the presence of the kitchen we are not able to scan this floor in transversal direction for confirming this hypothesis.

The results of T-scans of the internal floor are similar to each other. With the T-scan we measured the bars overlapping on the beam and it results to be about 1.2 m [7]. In whole T-scan of the internal floor we are able to see 6 targets that probably can associate to some longitudinal bars. Figure 11c shows the result of the 7th L-scan of the terrace (in green in figure 10). In this case we measured a bars density of 5.2 bars/m that correspond to an average step of 0.23 m.

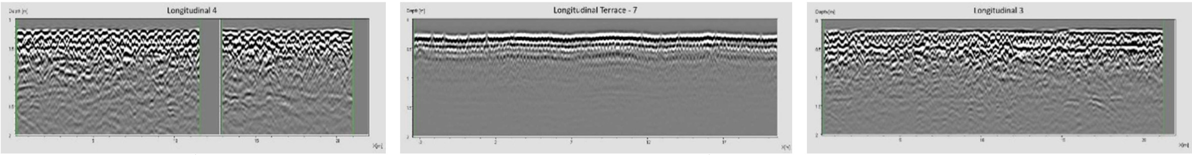

Figure 11. a) 3rd scan, 1.5 m from the perimeter beam; b) 4th scan, 2 m from the perimeter beam; c) 7th L-scan of the balcony taken close to the its final part

Finally, the T-Scan of the terrace (in green in figure 10) correspond to the original drawing (figure 8a).

In this scan it is possible to measure an average step between targets of 0.21 m that corresponds to the original drawing in figure 8a. The beam is also visible in this scan. In conclusion, the GPR survey confirmed what is reported in the original drawings, both for rebars density and for the design of the structure.

3. Assessment analysis in current conditions

The seismic response of the building in current conditions has been verified by utilizing the finite element model shown in figure 12, generated by SAP2000NL calculus program [8]. It is composed by frame elements reproducing beams and columns with rectangular sections and multi-frame elements for V-shaped beams and columns with variable section. Geometric and mechanical characteristics of the materials constituting the main structural elements correspond to those identified in the previous phase of the present study.

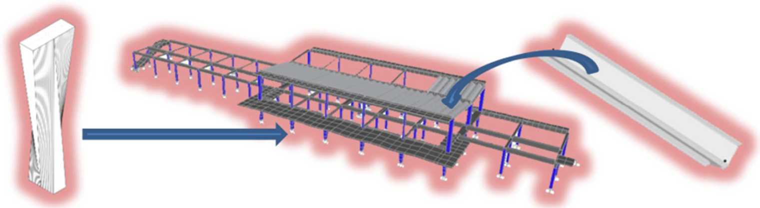

Figure 12. Finite Element model defined for the structure with details of V-shaped beams and variable section columns.

The verification enquiry in current conditions is articulated in a modal analysis to calculate the vibration periods and associated modal masses, and in a time-history analysis to assess the seismic performance in terms of stress states and displacements. The modal analysis carried out by the model shows a first horizontal roto-translational mode along Y with period of 0.62 s, a second horizontal mode mainly translational along X for the first interstorey and roto-translational for the second storey, with period of 0.35 s; a third rotational mode around Z with period of 0.31 s. The sum of modal masses exceeds the 85% after the tenth mode along X and Y, resulting of 98.4% (X) and 85.4% (Y).

The performance evaluation enquiry was carried out for the four reference seismic levels fixed in the Italian Standards [4], that is, Frequent Design Earthquake (FDE, with 81% probability of being exceeded over the reference time period V_R); Serviceability Design Earthquake (SDE, with 50%/V_R probability); Basic Design Earthquake (BDE, with 10%/V_R probability); and Maximum Considered Earthquake (MCE, with 5%/V_R probability). The V_R period is fixed at 75 years, which is obtained by multiplying the nominal structural life V_N of 50 years by a coefficient of use C_u equal to 1.5, imposed to structures whose seismic resistance is of importance in view of the consequences associated with their possible collapse, like the case-study school gym building. By referring to topographic category T1 (flat surface) and C-type soil, the resulting peak ground accelerations for the four seismic levels referred to the city of Florence are as follows: 0.082 g (FDE), 0.098 g (SDE), 0.223 g (BDE), and

0.270 g (MCE). Time-history analyses were developed by assuming artificial ground motions as inputs, generated in families of seven by the SIMQKE-II software [9] from the spectra above. In each time-history analysis the accelerograms were assumed in groups of two simultaneous horizontal components, with the first one selected from the first generated family of seven motions, and the second one selected from the second family. The results of the analyses carried out at the FDE and SDE are evaluated in terms of interlevel drift ratio (i.e., the ratio of the interlevel drift to the interlevel height of the columns) ILD_r. With reference to the SDE level, the maximum ILD_r values induced by the most severe among the seven groups of input motions, $ILD_{r,max}$, are as follows: 0.22% (SDE) in X, and 0.10% (SDE) in Y, on the first level; 0.098% (SDE) in X, 0.099% (SDE) in Y, on the second level. The drift ratios in X and Y are then far below the 0.33% limitation adopted by [4] at the Operational (OP) performance level for frame structures interacting with drift-sensitive non-structural elements.

The BDE- and MCE-related response was assessed also in terms of stress levels. The shear-related checks are met in both directions for both levels, up to the MCE. On the other hand, for some beams of first floor ($T_{2,RC}$ and $T_{4,RC}$) and roof (the end beams $T_{10,RC}$ and $T_{11,RC}$ on the roof), the stress state checks evidenced maximum stresses overcoming the corresponding resistance values of about 20% already at the BDE. With reference to this columns, the combined axial-force and biaxial-bending-moment stress state checks are met only for the internal columns with constant sections, and for those with variable sections on the first level at the BDE. The response of the corner columns ($P_{1,RC}$, $P_{2,RC}$, $P_{V,RC}$) at this level, as well as of all columns in the corners on the second level ($P_{5,RC}$) is unsafe starting from the BDE.

4. Conclusions

The paper presents a summary of the seismic vulnerability analysis carried out on the bar-restaurant of the "Bellariva" Sport Centre in Florence. This study provides an example of how to conduct the structural analysis for the seismic assessment of heritage buildings like that herein examined. It is also demonstrated how the execution of GPR non-destructive surveys can be useful in the on-site testing phase, allowing some structural details to be identified without the need for other destructive tests.

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