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Rehabilitation of an Historical Theatre in Italy

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ABSTRACT: Rehabilitation interventions on historical building with peculiar end-uses, such as, for example, theatres, raise problems non easy to be solved, especially when different architectural and structural functions must be ensured. The rehabilitation of the historical theatre R. Zandonai, built during the 18th century in Rovereto Italy, is a good example of how to cope with this kind of problems and to try to solve them. The will of the Rovereto municipality to rehabilitate the theatre, in order to suit the modern scenic requirements, involved the need to heighten the fly tower of about 6 meter. The deep structural modification and the reference to different design loads, led to verify the resulting structures, according to the rules and standards currently in force in Italy, among them the new Italian seismic law (O.P.C.M. 3431). The solution proposed and illustrated hereinafter, is conceived and designed with the purpose both to satisfy the severe requirements of the new Italian seismic code, and to be a reversible structure, therefore respectful to the original historical building.

1 INTRODUCTION

1.1 Historical information

The construction of the Council Theatre “Riccardo Zandonai” in Rovereto (the first theatre to be built in the region of Trentino), began in 1783, according to the design of the Italian architect Filippo Maccari. In 1870-1871 the body against the main avenue was built, endowed with a foyer, a salon and the beautiful façade, still maintained; in 1892-1893, thanks to the donation of an adjacent area made by a private citizen, the stage was enlarged up to the present dimension. After the damage caused by the 1st world war, the Theatre was newly restored; recently the Rovereto municipality deliberated to refurbish the building in order to have a "full flying height" for the satisfaction of the modern setting needs. For this reason the Authors have been required to evaluate the possibility to heighten the fly tower up to 6 meter.

1.2 The fly tower: the masonry walls

A classical theatre, such as “teatro Zandonai”, as well the modern theatres, is mainly composed by the *auditorium* (the part accommodating the audience during the performance), and the *stage*, on which performances happen. The part of the building where the stage is located, known as “fly tower”, is usually composed by four high masonry walls around the stage, whose extension allow to “fly up” the scenery until is out of sight of the audience (for this reason, it is no possible to place any horizontal diaphragms against the walls, except for the grid, i.e the support structure, close to the top of the fly tower, on which the pulleys of the flying system are supported). The wall between the stage and the audience has a big opening, the picture frame through which the audience sees the play (proscenium arch). The ideal fly tower should be more than twice the height of the proscenium arch: in this case, it is identified as a "full flying height" arch.

The structural typology of the fly tower is characterized by high walls, with some openings and without horizontal ties, with the exception of the wooden roofing with the support structure of the flying system (grid): from a structural point of view it can be seen like a “masonry tube”, in which a significant opening is provided by the proscenium arch. Therefore the seismic resistance of the fly tower can be a critical problem for the design.

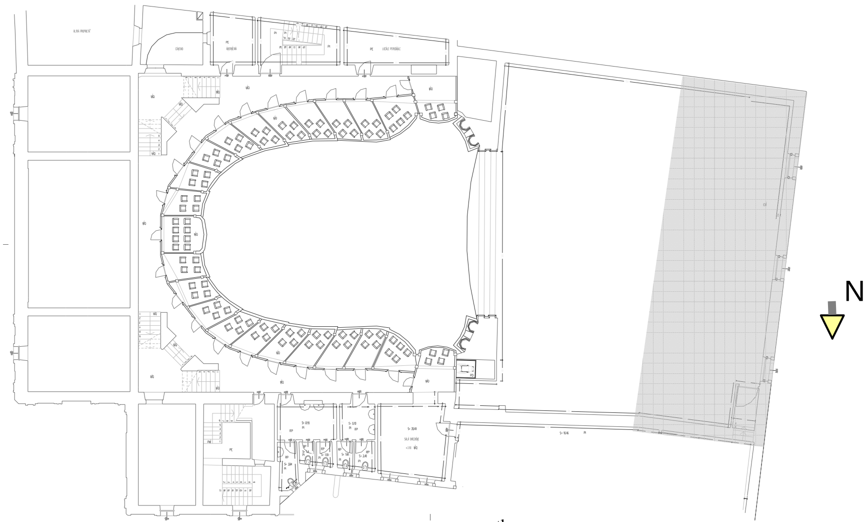


Figure 1 : The floor plan of “Teatro Zandonai”: the 18th century enlargement is highlighted.

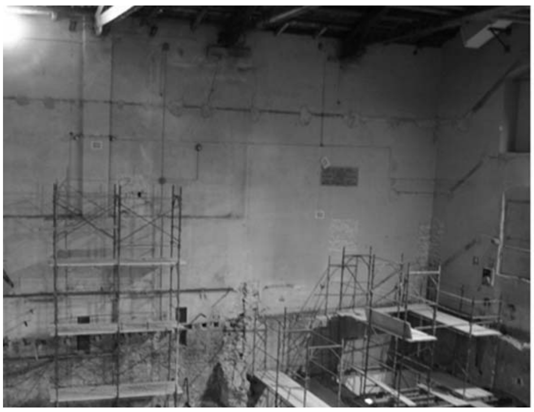


Figure 2 : The southern wall of the fly tower (note in the lower part free of plasterwork the discontinuity between the 18th century and 17th century walls); the western wall of the fly tower (note the openings in the wall, besides the three big arches not in view, in the lower part under the stage).

1.3 The fly tower: the roof and the grid system

The structural typology used for the roof is the queen-post timber truss illustrated in Fig. 3. In this typical configuration the compression top chord and two vertical posts form a central “square” to reduced the bending moment of the upper rafters (instead of the diagonal struts typical of the king-post truss). In this way it is possible to have more space under the roof for the scenery equipment. The secondary timber beams, which are supported by the lower chord of the truss, form the grid of the theatre.

Figure 3 : The queen-post timber truss.

1.4 The refurbishment requirement of the fly tower

In order to have a "full flying height" fly tower, the municipality of Rovereto required to increase the fly tower of 6 meters, preserving the existing masonry walls. For this reason, some preliminary tests on the masonry walls have been requested by the Authors, in order to obtain fundamental knowledge of the mechanical characteristics of existing structures.

1.5 The fly tower: preliminary survey of the masonry walls

A preliminary survey of masonry walls has been performed, in order to evaluate the capacity of existing structures to resist to an increase of vertical and horizontal loads (wind and seismic action). In different zones of the walls the plasterwork has been taken out, to permit a visual examination: in the northern and southern walls the lack of ties between the 17th century and the 18th century masonry has been revealed. The flat jack tests have clearly demonstrated the inadequate mechanical properties of the masonry, with low stiffness and strength values in the northern and southern walls, and a high stress level in the western wall, for the presence of three big arches at the ground level (Fig. 1 and 2).

In Table 1 the main results of the experimental tests performed on masonry are reported.

Table 1 : Mechanical properties of masonry of the fly tower (minimum and maximum values).

	Stress level σ MPa	Young's modulus E MPa	Elastic threshold σ_y MPa	Strength σ_R MPa
Northern wall	0,05÷0,09	309÷813	1÷1,2	1,6÷2,20
Western wall	0,09÷2,09	72÷8650	0,2÷2,2	0,80÷2,20
Southern wall	0,04÷0,63	122÷4000	0,6÷1,4	0,8÷1,2

Unfortunately the preliminary tests on masonry have shown the impossibility for the existing walls to support the lateral and vertical load due to the modern scenic requirements. In fact, the low values of mechanical properties, to be ascribed mainly to the insufficient quality of the mortar, determine the inadequacy of the original masonry to accept higher loads.

2 SEISMIC ADAPTATION OF THE FLY TOWER

2.1 Analysis of the global stability of the fly tower

A very important aspect for the structural rehabilitation of the theatre R. Zandonai is the analysis of the structural performance of the fly tower under lateral loads, like wind and seismic actions. In fact, in order to better understand the behaviour of the structure, it is not sufficient to analyze the strength of the masonry walls (*local stability*); the evaluation of the structural resistance under horizontal loads (*global stability*) is a very important and complicated aspect and it implies in the analysis of the building that should behave like a three-dimensional wall-box with very good connections between the different structural elements.

The structural typology of the fly tower results very sensitive to the effects of the seismic action, that depends on the distribution both of the masses and of the structural elements. The height and the width of the walls (15 meters in height and 19 meters in width) with the absence of any horizontal bracing, the big opening for the proscenium arch, the mass distribution concentrated only over the flying system (grid), the poor toothing-stone between perpendicular walls represent a critical problem for the seismic resistance of the fly tower. On this basis, only an innovative solution can solve the problem of the structural rehabilitation of the theatre, in which both structural, architectural and scenographichal aspects could converge.

2.2 A steel skeleton

In Fig. 4, the solution proposed by the authors is illustrated for: i) global reinforcement of the fly tower under horizontal action, and ii) support of the additional vertical loads of the new roof and timber - grid. The supporting structure, performing like a steel skeleton for the masonry, is reversible, taking into account the possibility to dismantle it and to return to the original configuration. The vertical steel columns are more as possible respectful of the architectural plan.

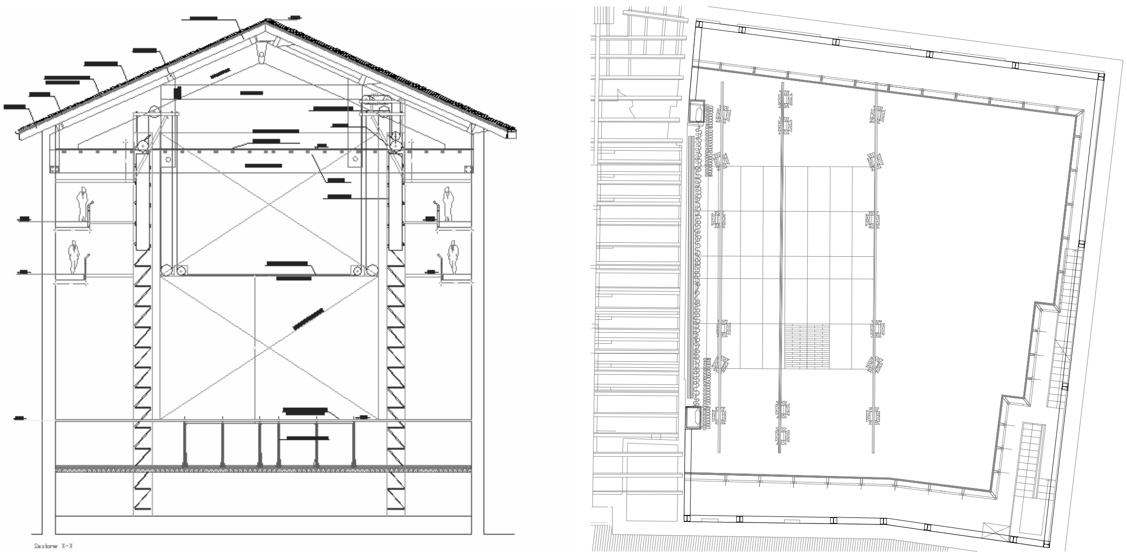


Figure 4 : Floor plan and section of the earthquake resistance steel structure for the fly tower.

In the structural conception a special attention has been dedicated in order to take advantage of all the architectural elements for structural purpose. The two levels of *small balcony* around the fly towers, which are functional for the backstage operations, allow to have an horizontal bracing of the steel structure, with regard to the torsional behaviour. For the same reason the *grid*, if adequately braced, can be considered as an horizontal rigid diaphragm for the fly tower. Even the steel vertical truss-structure supporting the *fireproof safety curtain* (which can be dropped to separate the audience from the stage in case of fire), is included in the skeleton designed as an earthquake resistant structure (Fig. 5).

2.3 Structural solutions for the roof and for the grid

The original project aimed to maintain the queen post truss geometry for the roof, following the conservative restoration principles. Unfortunately the high level of the loads and the particular geometry of the joint between the rafter and the tie beam (see Fig. 4) cause a state of stress inconsistent with the truss geometry. Therefore a solution has been proposed in order to separate the roof structure from the grid structures (see Fig. 5).

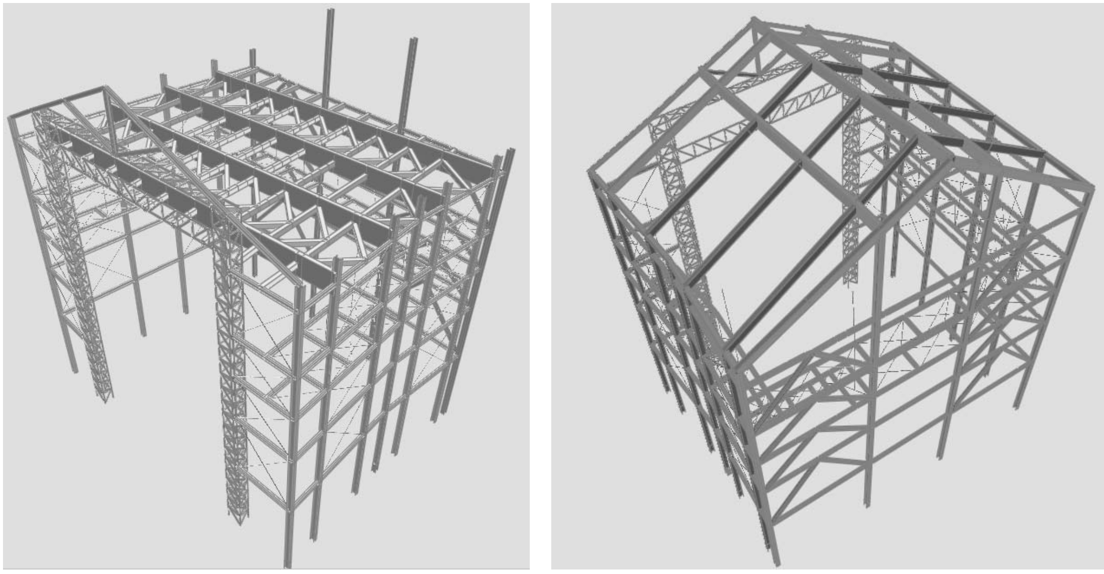


Figure 5: Three dimensional views of the earthquake resistant structure.

2.4 Masonry reinforcement

Due to the poor quality of the masonry, the Authors suggested to execute masonry reinforcement interventions, in order to consolidate the existing walls. Some reinforcement techniques have been proposed, such as mortar injection for the lowest levels of the walls and polymers mesh reinforced plasterwork.

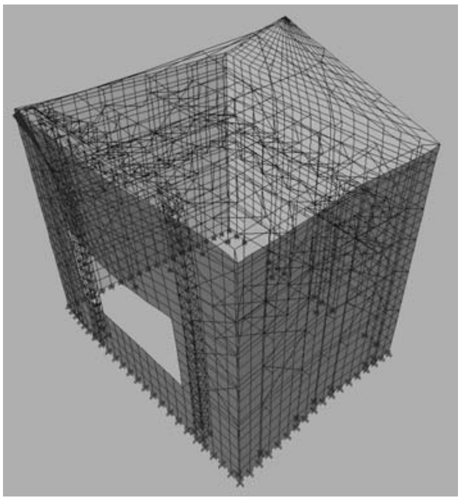
3 DESING OF THE STRUCTURE UNDER SEISMIC ACTIONS

3.1 The finite element model

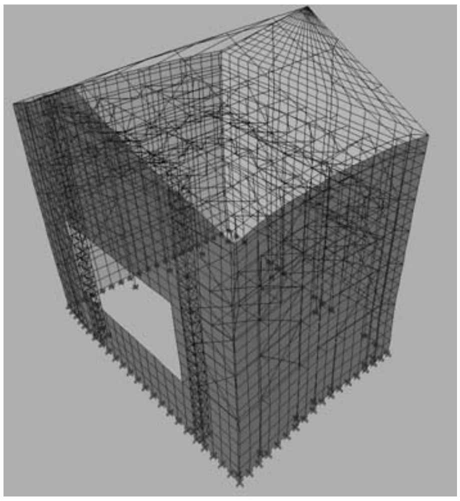
The design forces acting on the structural members under static and seismic load combinations have been obtained by means of a finite element (FE) three-dimensional (3D) model, developed with the non-linear SAP2000 code. All the structural elements have been implemented in the numerical model, considering the mechanical and geometrical properties of the steel skeleton-structure; in detail, the two levels of *small balcony* around the fly towers are considered as horizontal bracing of the steel structure. For the same reason, the *timber grid* is modelled as an horizontal rigid diaphragm. Even the steel vertical elements, supporting the *fireproof safety curtain*, are considered included in the steel structural skeleton.

The masonry walls have been implemented in the model using the strength and stiffness values obtained through the mechanical flat jack tests. In particular, the stiffness of the walls has been implemented in the model considering the no-tension behaviour of the masonry. Therefore the bending and shear stiffness of the walls have been set equal to 2/3 of the elastic value. The interaction between steel structure and the masonry walls has been simulated in the numerical model by means of special rigid constraints that impose the same displacements and rotations to the adjacent nodes of the steel part and of the masonry walls, respectively. This numerical constraint should be realized in the real structure by using insertion of special rebars anchored in the walls by means of chemical resin and reinforcing the interested portion of the masonry by means of electro-welded steel mesh.

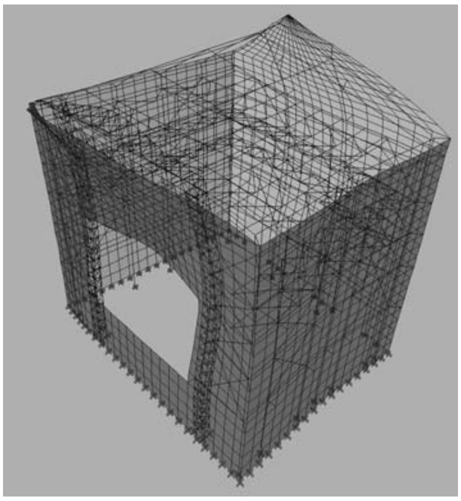
Moreover, the real distribution of the masses has been modelled, considering the different and worst position that the *fireproof safety curtain* can present, along the supporting truss-structure. In Fig. 6 the main modal shape configurations of the structure have been exported from the model. One can observe that the first and the second modal shapes depend on the bending stiffness of the fly-towers; on the contrary, the third and the fourth modal shapes depend on the torsional stiffness of the structure.



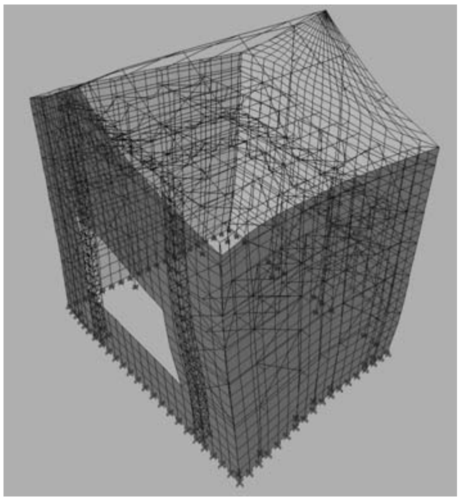
1st modal shape (T=1,41 sec)



2nd modal shape (T=1,21 sec)



3rd modal shape (T=0,96 sec)



4° modal shape (T=0,89 sec)

Figure 6 : the first modal shapes of the structure.

The resistance and energy-dissipation capacity assigned to the structure and related to its non-linear response is taken into account by means of the *behaviour factor q*. In this case, the structure has been considered as non-dissipative, neglecting any hysteretic energy dissipation (q factor equal to 1,5). A modal response spectrum analysis, using a linear-elastic model of the structure and the appropriate design spectrum has been carried out according to the national seismic code O.P.C.M 3431. This type of analysis has been applied to this structure, because it doesn't satisfy the conditions of "regularity" in plan and elevations. All vibration modes contributing significantly to the global response have been taken into account. Moreover, the horizontal components of the seismic action in x- and y-direction are supposed acting simultaneously, using both of the two following combinations:

(a) $E_{Edx} + 0{,}30E_{Edy}$ (b) $0{,}30E_{Edx} + E_{Edy}$

where "+" implies "to be combined with"; E_{Edx} represents the action effects due to the application of the earthquake acting along x axis; E_{Edy} represents the action effects due to the application of the earthquake acting along y axis.

3.2 Verifications

The structure has been designed and verified in such a way that the following requirements are met, with an adequate degree of reliability.

− *No-collapse requirement (ULS)*, i.e. the structure has been designed to withstand the design seismic action without local or global collapse, thus retaining its structural integrity and a residual load bearing capacity after the seismic event.

− *Damage limitation requirement (SLS)*, i.e. the structure has been designed to withstand a seismic action having a larger probability of occurrence than the design seismic action at ULS, without the occurrence of damage and the associated limitations of use, the costs of which would be disproportionately high in comparison with the costs of the structure itself.

The structure as a whole has been checked to ensure that it is stable under the design seismic action. Both overturning and sliding stability has been taken into account. Moreover, it has been verified that both the foundation elements and the foundation soil are able to resist the action effects resulting from the response of the superstructure without substantial permanent deformations. All the structural elements, including connections and the relevant non-structural elements, have been designed in order to satisfy the following relation:

$$E_d \leq R_d$$

where E_d is the design value of the action effect, due to the seismic design situation including second order effects; R_d is the corresponding design resistance of the element, calculated in accordance with the rules specific to the material used (in terms of the characteristic values of material properties f_k and partial factor γ_M) and in accordance with the mechanical models for the specific type of structural system, as given in Italian code (O.P.C.M. 3431, 2005) and Eurocode documents (EC3, 2005; EC5, 2004 and EC8; 2004).

4 CONCLUSIONS

In order to reach the request of Municipality of Rovereto, an earthquake resistant structure has been designed with the purpose to refurbish the fly tower of the historical theatre R. Zandonai, in order to suit it to modern scenic requirements. The structural typology of the fly tower, from a structural point of view, can be seen like a “masonry tube”, endowed with significant openings, and therefore unsuitable to adequately perform under seismic actions.

The preliminary tests on masonry walls demonstrated the poor mechanical properties of the material, and consequently the impossibility to support the lateral loads due to seismic actions and vertical loads due to modern scenic requirements. On this basis an innovative solution has been proposed, consisting in a steel skeleton that should provide a global reinforcement of the fly tower.

A finite element three-dimensional (3D) model, implemented through the non-linear SAP2000 code, has been developed, where all the structural elements have been considered, also taking into account the experimental evidences for the mechanical properties of the material. All the structural elements have been designed and verified in order to satisfy the requirements given in Italian and European documents.

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