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CONSTRUCTIVE REHABILITATION OF THE FLOOR STRUCTURES OF ZAGREB'S DOWNTOWN BUILT IN THE FIRST HALF OF THE 20TH CENTURY

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ABSTRACT: The major damage to the buildings in Zagreb's Lower downtown caused by the earthquake is partly the result of the way the timber ceilings were built, which are mostly not connected to the masonry walls on which they rest. In this way, they do not have the properties of rigid diaphragms, which, due to the effect of an earthquake, leads to the loss of the integrity of the construction.

In the paper, we consider the possibility of strengthening timber ceilings with regard to the significant requirements of preserving cultural heritage, and present the results of selected types of reinforcement. The considered types of reinforcement are: reinforcement of the timber beams with a concrete compression slab, reinforcement of the timber beams with wooden elements - OSB boards, and reinforcement with steel elements.

Regarding the possibility of implementation, a conclusion was given on the chosen method that meets the given requirements in terms of horizontal stiffness and the preservation of the cultural and historical heritage of the downtown core.

KEYWORDS: Zagreb earthquake, timber ceilings, horizontal stiffness, seismic analysis

1 INTRODUCTION

Due to the earthquake in Zagreb on March 22, 2020, there was extensive damage to the downtown architecture. Damage is partly due to the way timber ceilings are built. Most of the damaged buildings were built in period 1900 and 1948. In that period, all buildings have deficiencies in terms of meeting the criteria of earthquake resistance, unconnected ceilings with walls, i.e., the inability to transmit horizontal forces, load-bearing walls made of unbounded masonry, and poor connections of basic assemblies and elements without the necessary load-bearing capacity and ductility [1].

Masonry buildings are structural systems of vertical and horizontal parts whose seismic response is the result of the interaction of wall with wall and wall with ceiling or roof. In order for the structure to meet the behaviour during an earthquake, it is necessary to realize the bearing capacity and rigidity of the walls in the plane and to prevent the collapse of the walls outside the plane [2].

In the paper we will analyse the possibility of strengthening the classic timber ceiling structures. These proposals for constructive rehabilitation will enable the strengthening of existing ceiling timber structures but also the preservation of cultural heritage, as damaged. buildings are individually or as an urban entity protected as Immovable Cultural Heritage.

2 CASE STUDY

The subject building is a residential building located in the downtown of the city of Zagreb, at the address Medulićeva street 16/3. The building was built in the first part of the 20th century (before 1920).

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Figure 1: The western façade (face) of the structure

The vertical bearing structure consists of brick walls, while the ceiling structure is made of timber beams. The residential building has a basement, ground floor and 3 floors [3].

Figure 2: The eastern façade (face) of the structure

In the floor plan, we see a deficiency of walls in the x direction, and we can immediately establish that this is the weaker direction of our building.

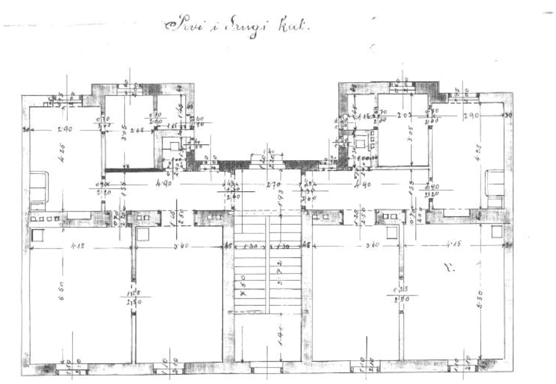

Figure 3: Archival floor plan of the building (first and second floor)

In this period [4], buildings are built that have predominantly timber ceiling structures, except for the basement ceilings, which are most often made in the structure of masonry vaults, arches, or Prussian ceilings. The extensive damage due to earthquakes is partly due to the way timber ceilings are built, which do not have the characteristics of rigid diaphragms, they cannot connect all the load-bearing elements into one whole. The consequence of this is the local failure of the walls.

Existing timber beams, meet the criteria of the final ultimate limit state of bearing capacity (ULS) while the limit states of serviceability are generally not met, mostly due to rheological phenomena (creeping). Furthermore, the connections of timber beams and walls are made by direct adhering the beams to the walls, as a rule, without anchoring. This type of design allows only reliable resistance to vertical actions, while resistance to horizontal actions is extremely low [3].

3 REHABILITATION METHODS

The possibility of rehabilitation, i.e., increasing horizontal and vertical stiffness is possible with concrete compression plate, OSB slabs or timber or wood-based elements and steel profiles. To achieve appropriate effect, it is necessary to connect the beams to the walls. In the case of reinforcement with the concrete compression plate [5], a pliable connection between concrete and wood is made.

Considering the weight of the rubble and the weight of the reinforced concrete slab, the own weight of the ceiling structure does not increase significantly. By placing the fasteners in the existing wooden beams, the structure is connected, and in this way, significantly greater load capacity and rigidity is achieved perpendicular to the plane, while inside the plane we get a rigid disc [1]. The pressure plate can be made of classic concrete C25/30 or light concrete EPS.

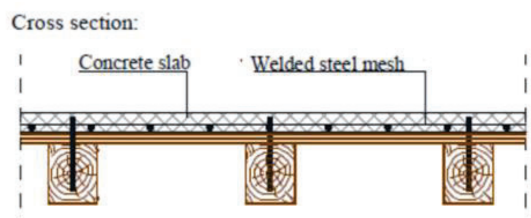

***Figure 4.** Reinforcement of the timber beams with a concrete compression slab*

***Figure 5.** Reinforcement of the timber beams with a concrete compression slab*

The disadvantages of this method of rehabilitation are changes in the stress state in the T-section and the action of two different materials of different stiffness and rheological characteristics [5].

Rehabilitation and reinforcement with OSB plates (possibly veneer plates or wooden boards) can be done

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from the bottom or top side of the timber beam. This way of stiffening is a partially rigid diaphragm.
By placing wooden boards on the existing ceiling structure, i.e., timber beams, a double reinforcement effect is achieved. Reinforcement to vertical load, but also to horizontal action, is achieved if the elements that form the diaphragm are properly connected to each other to take the shear load.
The ceiling structures must be connected to the walls so that they do not separate during the horizontal effects of the earthquake force, but also to enable the proper distribution of the seismic force on the individual walls. Wooden panels made of OSB boards 22 mm thick is placed in two layers, the first layer is placed at an angle of 45° in relation to the beam, and the second layer is placed at an angle of 90° in relation to the first. The connection to the beams is made with self-tapping screws. Each joint is made with a minimum of 2 wood screws [1].
With the diagonal arrangement of the additional layer, we get better pressure resistance and stiffness in both planes [2].
To achieve continuity, it is necessary to adequately secure the joints with the walls.

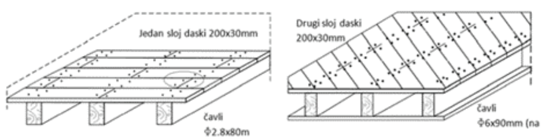

Figure 6. *Reinforcement with OSB boards on the top or bottom*

Rehabilitation and reinforcement with steel elements is usually carried out between the last two ceiling timber beams in such a way as to install steel sheets forming a lattice girder [4]. Horizontal stiffening of wooden ceiling structures with steel elements is carried out when no additional vertical bearing capacity of the structure is required, but only the absorption of horizontal forces [1].

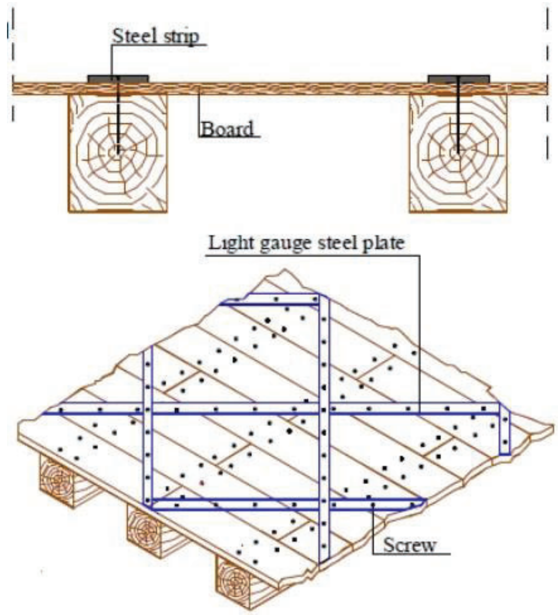

Figure 7: *Reinforcement with steel elements*

Figure 8: *Steel strip*

3.1 NUMERIC MODEL DESCRIPTION

For the purpose of the numerical modelling, an analysis of 3 cases was made – a timber beam without stiffening (existing condition) – model A, a timber beam with an 8 cm concrete plate - model B (composite timber-concrete), a timber beam with an OSB plate 24 mm – model C.
We describe the possibility of strengthening timber ceilings with steel elements, but we did not model it due to the complexity of creating such a model in the program for masonry structures.
Several models of the current state have been made, each of which has its own assumptions in modelling. A linear-elastic model was made. This model is acceptable for engineering practice. The following loads were considered:
LC1 - self-weight
LC2 – additional self-weight
LC3 – live load 2 kN/m2 in residential areas, 0.75 kN/m2 on an impassable roof and 3 kN/m2 on the staircase

LC4 - earthquake in X direction
LC5 - earthquake in the Y direction

A modal spectral analysis was made for the calculation, and the load was taken as if we wanted to strengthen the structure to meet level 2 of the seismic reconstruction. Among other data, soil type C, type 1 spectrum, soil acceleration coefficient 0.13 and behavioural factor 1.5 were assumed, which corresponds to the assumptions in Eurocode Standard for unbounded masonry. For each model, 30 different modes were considered [3].

In the model of the existing state, the beams are modeled so that they carry in the shorter direction, and in this way only the walls in the transverse direction take over the loads on the wall. (Model A). Due to the vertical/longitudinal direction of the earthquake in relation to the timber beam (in the direction of the timber beams), an orthotropic seismic response occurs, i.e., relative displacements of the beams and bending of the floor covering occur, while the load perpendicular to the direction of the timber beams causes bending of the beams and relative movements of the floor covering [2].
The second model is a model with a reinforced concrete pressure plate 8 cm anchored in the walls (model B).
The third model is stiffened using OSB panels on the upper side of the beams (model C). The thickness of the panels is assumed to be 24mm.

3.1.1 Seismic design in 3Muri

A 3D numerical model of the subject building was created using the 3Muri computer program. Seismic analysis of the observed structure was performed using a non-linear static calculation, i.e., a gradual pushover method with a constant gravity load and a monotonically increasing horizontal load.

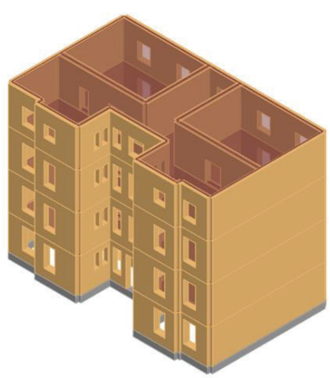

Figure 9: *3D building model - existing condition*

After the response of the structure to seismic excitation, i.e., the so-called capacity of the structure, which is independent of the seismic demand, is obtained, controls are carried out in accordance with the HRN EN 1998-3 standard according to the basic requirements related to the state of damage to the structure, which are defined by limit states.

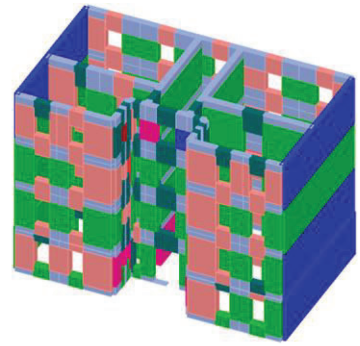

Figure 10: *3D Seismic model - existing condition (Model A)*

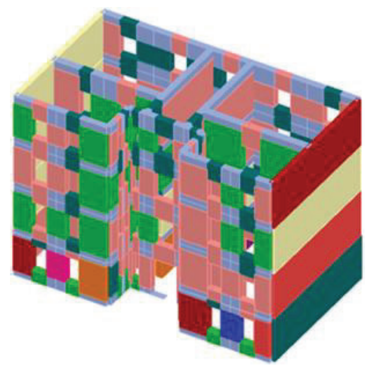

Figure 11: *3D Seismic model - reinforcement with a concrete compression slab (Model B)*

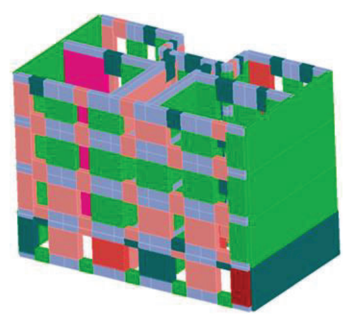

Figure 12: *3D Seismic model - reinforcement with wooden elements - OSB boards (Model C)*

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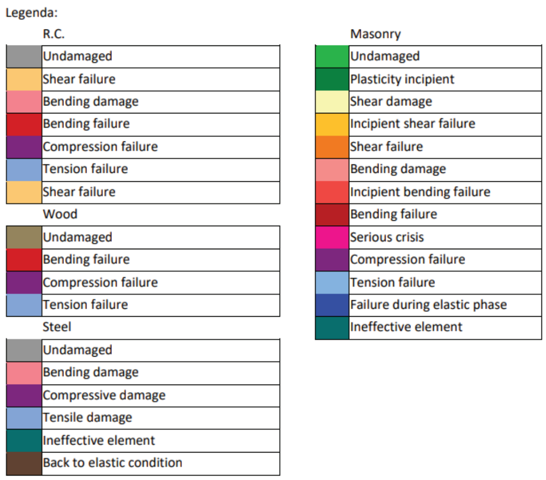

Figure 13: *Legend of structural damage*

The constructed 3D models represent the distribution of stiffness and masses, and thus all significant forms of deformation as well as inertial forces in the observed seismic action are taken into account. The analysis was performed according to Eurocode 8 [5,6].

For the method of gradual pushing, two forms of vertical distribution of lateral forces were used. These are a uniform distribution that is proportional to the mass of the structure on each floor, and a linearly increasing distribution in height that has the shape of an inverted triangle. The mentioned lateral forces act at the locations of the masses in the model. A random eccentricity of 5% was also considered, which takes into account possible uncertainties in the position of the masses. The result of the earthquake analysis is a capacity curve that gives the relationship between the transverse force at the footing and the control displacement.

4 RESULTS

The analysis for the existing condition shows the expected results - local failure of individual elements (walls are not horizontally connected). The largest displacements were achieved on the model of the existing state, where the horizontal stiffness of the timber ceiling is the least, and the smallest displacements are on the model of the reinforced concrete compression slab, which has the highest stiffness of the mentioned models.

Table 1: *Calculated construction shifts for individual models*

Model	Maximum displacement of the structure compared to model A [%]
Model A	100,00%
Model B	73,74%
Model C	65,42%

The tables below show the results of the pushover analysis for displacements under seismic loading for the X and Y directions:

Table 2: *Pushover analysis – displacement (Model A)*

Br.	Seismic load direction	Seismic load	dt SD [cm]	dm SD [cm]	SD Ver.	Sd DL [cm]	d*y DL [cm]	DL Ver.
1	+X	Uniform	10,16	7,46	No	3,74	1,47	No
2	+X	Modal distribution	14,78	0,87	No	5,44	0,85	Ne
3	-X	Uniform	10,38	8,20	No	3,82	1,32	No
4	-X	Modal distribution	15,79	1,01	No	5,82	0,98	No
5	+Y	Uniform	5,53	1,55	No	1,67	1,21	No
6	+Y	Modal distribution	6,23	1,26	No	1,94	1,02	No
7	-Y	Uniform	5,14	2,17	No	1,48	1,01	No
8	-Y	Modal distribution	5,74	1,56	No	1,75	1,15	No

Table 3: *Pushover analysis – displacement (Model B)*

Br.	Seismic load direction	Seismic load	dt SD [cm]	dm SD [cm]	SD Ver.	Sd DL [cm]	d*y DL [cm]	DL Ver.
1	+X	Uniform	10,61	8,03	No	3,96	1,65	No
2	+X	Modal distribution	10,90	3,65	No	4,07	1,49	No
3	-X	Uniform	10,68	8,76	No	3,99	1,46	No
4	-X	Modal distribution	9,80	4,54	No	3,66	1,58	No
5	+Y	Uniform	5,29	3,06	No	1,57	1,27	No
6	+Y	Modal distribution	4,95	2,71	No	1,43	1,21	No
7	-Y	Uniform	5,26	2,27	No	1,52	1,11	No
8	-Y	Modal distribution	4,92	2,15	No	1,38	1,04	No

Table 4: *Pushover analysis – displacement (Model C)*

Br.	Seismic load direction	Seismic load	dt SD [cm]	dm SD [cm]	SD Ver.	Sd DL [cm]	d*y DL [cm]	DL Ver.
1	+X	Uniform	9,67	8,87	No	3,57	1,44	No
2	+X	Modal distribution	9,40	8,87	No	3,47	1,40	No
3	-X	Uniform	9,70	11,03	No	3,58	1,29	No

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4	-X	Modal distribution	10,27	10,79	No	3,79	1,58	No
5	+Y	Uniform	4,73	3,16	No	1,33	1,12	No
6	+Y	Modal distribution	5,40	4,01	No	1,57	1,13	No
7	-Y	Uniform	4,77	1,85	No	1,29	0,94	No
8	-Y	Modal distribution	5,41	3,00	No	1,53	0,95	No

The tables below show the results of the pushover analysis for the limit states of significant and limited damage under seismic loading for the X and Y directions:

Table 5: *Pushover analysis – limit states (Model A)*

Br.	Seismic load direction	Seismic load	α SD	α DL
1	+X	Uniform	0,735	0,394
2	+X	Modal distribution	0,059	0,155
3	-X	Uniform	0,790	0,345
4	-X	Modal distribution	0,064	0,169
5	+Y	Uniform	0,354	0,722
6	+Y	Modal distribution	0,249	0,525
7	-Y	Uniform	0,489	0,683
8	-Y	Modal distribution	0,339	0,657

Table 6: *Pushover analysis – limit states (Model B)*

Br.	Seismic load direction	Seismic load	α SD	α DL
1	+X	Uniform	0,758	0,416
2	+X	Modal distribution	0,335	0,367
3	-X	Uniform	0,820	0,366
4	-X	Modal distribution	0,463	0,433
5	+Y	Uniform	0,632	0,811
6	+Y	Modal distribution	0,614	0,847
7	-Y	Uniform	0,500	0,727
8	-Y	Modal distribution	0,514	0,754

Table 7: *Pushover analysis – limit states (Model C)*

Br.	Seismic load direction	Seismic load	α SD	α DL
1	+X	Uniform	0,917	0,403
2	+X	Modal distribution	0,944	0,404
3	-X	Uniform	1,137	0,361
4	-X	Modal distribution	1,050	0,417
5	+Y	Uniform	0,720	0,841
6	+Y	Modal distribution	0,772	0,719
7	-Y	Uniform	0,472	0,726
8	-Y	Modal distribution	0,599	0,624

According to the results of the calculation, the building model without reinforcement (model A) does not satisfy the deformability conditions in all analyses for the limit state of limited damage (DL) and for the limit state of significant damage (SD), with the most unfavourable analysis being 2 in the direction x for which the structure had the ability to withstand 5.9% of the designed peak ground acceleration for a return period of 475 years.
The result of the calculation of model B - the beam reinforced with a concrete compression slab does not satisfy the deformability conditions for any of the limit states, and in the most unfavourable analysis 2 in the x direction, the structure had the ability to withstand 33.5% of the designed peak ground acceleration for a return period of 475 years.

Results for the calculation model C - strengthening with OSB plates, shows that the building does not satisfy the deformability conditions for both limit states of damage, with the most unfavourable analysis being 7 in the y direction, for which the structure had the ability to withstand 47.2% of the designed peak ground acceleration for the return period from 475 years.

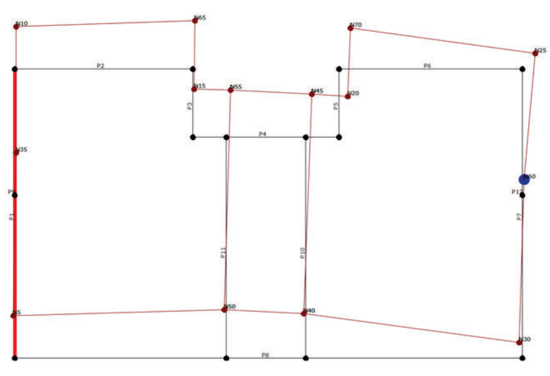

Figure 14: *Seismic analysis no. 2 in the X direction (critical direction), plan view of deformation, (Model A)*

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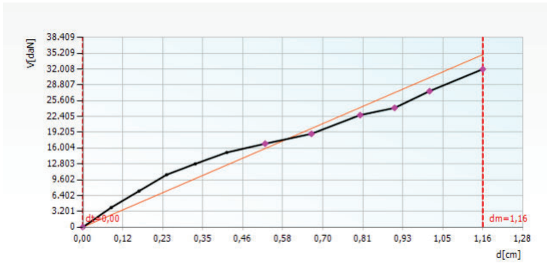

Figure 15: *Pushover curve (analysis n. 2) (Model A)*

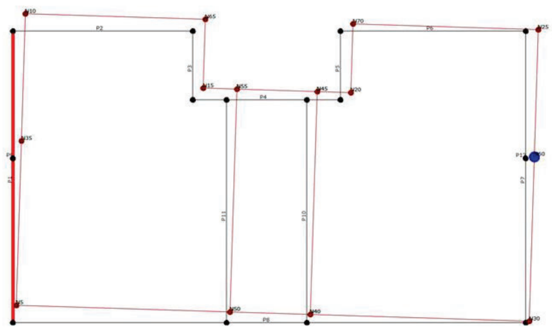

Figure 16: *Seismic analysis no. 2 in the X direction (critical direction), plan view of deformation, (Model B)*

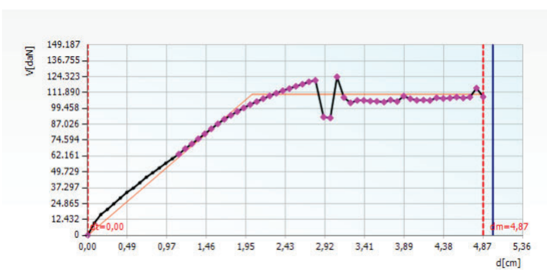

Figure 17: *Pushover curve (analysis n. 2) (Model B)*

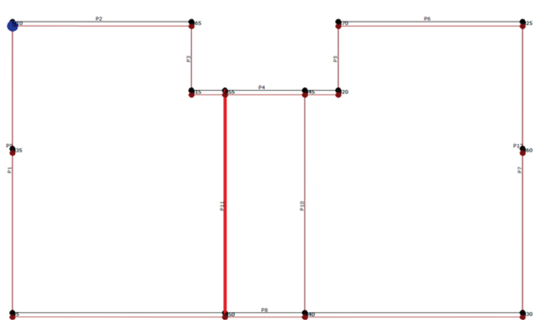

Figure 18: *Seismic analysis no. 7 in the Y direction (critical direction), plan view of deformation, (Model C)*

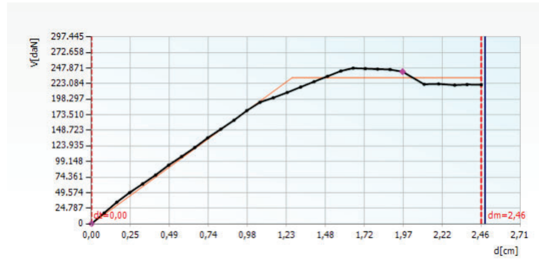

Figure 19: *Pushover curve (analysis n. 7) (Model C)*

5 CONCLUSIONS

After conducting an analysis of various preliminary models of this structure, the best result in terms of increasing the horizontal stiffness of the structure is the use of concrete plate. But there is a problem with the influence of the stiffness of the compression plate on other structural elements, i.e., masonry, and the invasiveness of such a procedure.

However, the analysis showed that despite the high local stiffness of the slab, the calculated displacements of model B are about 11% higher than those of model C. This is explained by the relatively larger mass acting on the walls.

The advantages of strengthening with wooden elements are:

- execution speed,
- low self-weight,
- non-invasiveness
- stiffening in both planes
- can be used for both ceiling and roof systems [2].

Reversible strengthening techniques are preferred when strengthening historical heritage buildings. In this case model B represents an irreversible technique while model C is reversible therefore, option C is presented as the optimal solution.

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