INTERVENTION DECISION ON TURKISH TIMBER STRUCTURE

KURUSCU Ali Osman¹, ARUN Gorun²

¹ Yildiz Technical University, Istanbul, Turkey, aliosmankuruscu@yahoo.com
² Yildiz Technical University, Istanbul, Turkey, goruna@gmail.com

Keywords: Timber structure, Failures of timber, Intervention of timber frame

Abstract. Istanbul, which has been the capital city of different empires throughout history, houses numerous listed architectural and cultural historical buildings. There are diverse applications of timber structures all through Anatolia. The timber structures, still being used in Istanbul are from 19th century. These structures are used as public buildings or private dwellings. During their long life, these structures suffer lack of maintenance and many additions or changes according to the changing needs.

The timber structures in Istanbul are of diagonally braced timber frame construction without any infill. The frame is covered either by plaster over closely spaced laths nailed on the frame’s columns/studs (Baghdadi plaster) or the frame is veneered with sawn timber planks.

In this paper, intervention decisions on a 19th century timber building which is used as district governorate will be presented. This building of five floors had lots of problems with its load bearing beams, floor joists and floors. The densely decorated ceiling of the main hall of 6.0x13.0 m was highly deflected due to material loss and wrong interventions applied in the past. The distance to be spanned had to be 6.0 meters and because the walls also had paintings, the height of the beam, joists and the finishing shouldn’t exceed 26 cm. To accomplish these restrictions calculations of several construction materials were made. And it was decided to make a floor height steel truss in place of the deteriorated wall of the upper floor. Special measures are taken between steel and wood because steel and wood are incompatible materials.
1 INTRODUCTION

Istanbul, situated on the northern Anatolian fault line has been the capital city of different empires throughout history and houses numerous listed architectural and cultural historic buildings. Due to its location, structures in this area have suffered numerous intense and destructive earthquakes all through history.

As capital city of Ottoman period, construction of timber had high and low periods in Istanbul. As masonry houses have suffered numerous destructive earthquakes, wooden buildings gained importance to be safe especially among rich people. As several fires wiped out timber houses and the districts with timber dwellings, masonry buildings were made obligatory by law in the form of building regulations. Later, as repeated earthquakes caused great damages, construction of timber buildings was again allowed under the law. Today, historic timber dwellings with variety of materials and techniques are from 19th century. Most of these buildings suffer from lack of maintenance, abandonee, continuous changes in the structural layout, negligence and repair of the past works.

In this study, present situation and intervention decisions of a timber - masonry building which is used as district governorate in Nişantaşı, Şişli District of Istanbul, the very crowded place of the city, will be presented (Figure 1).

![Figure 1: Location of building in Pervititch [1], Satellite view of today[2]](image)

The timber structures in Istanbul are of diagonally braced timber frame construction without any infill. The frame is covered either by plaster over closely spaced laths nailed on the frame’s columns/studs (Baghdadi plaster) or the frame is veneered with sawn timber planks (Fig. 2).

![Figure 2: Diagonally braced frame without infill](image)
2 DESCRIPTION OF THE BUILDING

The building was constructed by Köse Raif Pasha as his residence in the second half of the 19th century and was called as “Köse Raif Pasha Mansion”. In the insurance maps of Istanbul prepared by Jacques Pervititch in 1925, the building is defined as basement and two floored mansion of "Bekir Bey". The basement with 2.45m height has five rooms and used as the kitchen. Exterior and interior walls are of stone masonry with jack arch floor. Entrance to the ground floor is from south, Rumeli Street, with eight steps. This floor of 4.10m height had four rooms with an exit to back garden facing Sair Nigar Street on north. The exterior walls of the entrance floor are of brick masonry, interior partition walls are timber frame filled with brick and both sides veneered by bagdadi plaster. These rooms are closed with timber floor. The first floor that was the last floor in 1925 had three rooms where the room facing Rumeli Street was arranged as a spacious room with 4.10 m height and 6.00m x 13.00m dimensions in plan (Fig. 3). The exterior and interior walls are same as the entrance floor.

Figure 3: The spacious room of the first floor

Today, the building that has rectangular plan with dimensions 14.80m x 13.00m has additional second and attic floor. The second floor with 3.90 m height has six rooms, dividing the space over the spacious room into three (Fig. 4).

Figure 4: First and second floor plan of building [3]
The exterior east and west walls of the building are of brick masonry. The exterior and the interior partition walls are timber frame without any infill. The south exterior wall and the inner walls are covered by bagdadi plaster where the exterior wall on north is veneered by sawn timber planks on outer face and bagdadi plaster on inner (Fig. 5). The attic floor placed at the center has three rooms where both exterior and interior walls are of bagdadi plastered timber frame without infill. The height of this floor is 2.20m. All the internal stairs are of timber. There are reinforced concrete buildings constructed on its east and west.

![Figure 5: South and north facades of the building](image)

This building had lots of problems with its load bearing beams, wall studs, floor joists and floors. The two internal walls dividing the space over the spacious room into three on second floor caused deflection of the densely decorated ceiling of the main hall of 6.0m x 13.0 m. To construct these walls during addition of the upper floor, they had spanned 6.00 m of first floor with two iron 1160 beams suspended by iron sheets from its flanges to the 10cm x 40cm timber beam spanning 6.00m over the second floor. As the timber beam deflected, the iron hangers loosened from the flanges of the iron beams deflected the iron beams, causing deformations at the ceiling of the big hall (Fig. 6). The floor joists were also highly deflected due to material loss and wrong interventions applied in the past.

![Figure 6: Deflections of timber and iron beams](image)
3 STRUCTURAL EVALUATION

3.1 Damage and Deterioration

During the inspection of the building, no damage or deterioration was observed in the basement and entrance floors and on masonry walls. The only problem in the first floor was the deflection of the decorated ceiling of the main hall. The exterior and interior timber frame walls of the second floor had material deterioration due to physical, chemical and biological actions effecting structural distortions mainly caused by water penetrating from the roof due to blocked rainwater outlets, or from the damaged façade timbers (Fig. 7).

![Figure 7: Deteriorations](image)

The iron beams spanning 6.0m over the main hall were corroded and iron supports of iron beams were broken (Fig. 8).

![Figure 8: Corrosion on iron beams and supports on 1st floor ceiling](image)

3.2 Structural Evaluation of the Building

*Structural Analysis of Timber Elements*

The timber joists of the timber floors that have 6cm x 18cm section were placed at 25-35 cm distance. For the structural analysis of timber roof/floor elements spanning 5.0m, dead loads are taken as 110 N/m for roof rafters and 770 N/m for floor joists. Controls of the bending moments and deflections with the accepted limits show that they are at acceptable levels.
\[ \sigma = 6.875 \text{ MPa} < \sigma_{\text{allow}} = 10.0 \text{ MPa} \quad \text{acceptable.} \]

\[ \text{maxf} = 0.089\text{cm} > \frac{1}{300} = 1.67\text{cm} \text{ is acceptable}. \]

The structural analysis of the timber studs with 10cm x10cm cross-section and 4.10m high are placed at approximately 0.40m spacing. The buckling control of the studs is also at acceptable levels.

\[ \sigma = 3.1 \text{ MPa} < \sigma_{\text{allow}} = 8.5 \text{ MPa} \text{ is acceptable.} \]

**Structural Analysis of Iron Elements**

For structural evaluation of the iron elements, the stress from bending moments and deflection appeared from current loads, are calculated and compared with limit values. The control of the bending moments and deflections of the two iron I160 sections spanning 6.0m show that these beams have exceeding values compared to the allowable ones. So, they are not safe.

\[ \sigma = 166.6 \text{MPa} > \sigma_{\text{allow}} = 140 \text{ MPa} \]

\[ \text{maxf} = 2.76\text{cm} > \frac{550}{300} = 1.83\text{cm} \]

**Structural Analysis of Masonry Elements**

Due to uncertainties of masonry elements, structural analyses were not made with computational methods. Axial and shear forces were calculated according to the accepted criteria’s of Turkish earthquake code [4], Eurocode 8 [5].

Total plan area of the building is 183.0m² and weight of the building is 8257kN.

- Average compression stress on the basement masonry walls: \[ \sigma = W / A \leq 5 \text{ MPa} \] 
  \[ \sigma = 1.6N/mm² \text{ MPa} \]
  \( (1.6 \text{ MPa} < \sigma_{\text{max}} = 5 \text{ MPa}) \) as calculated, structure is safe in terms of axial forces.

- Base shear ratio, \( \tau_{\text{av}} = V_t / 0.5 * \sum A_w \leq 2.5 \text{ MPa} \)  (Table 1)

<table>
<thead>
<tr>
<th>Earthquake Zone 1</th>
<th>A_w=0.40</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ground Class Z3</td>
<td>T_\text{a}=0.15 , T_b= 0.60</td>
</tr>
<tr>
<td>Building Importance Factor I</td>
<td>1.4</td>
</tr>
<tr>
<td>Acceleration Spectrum S(T)</td>
<td>2.5</td>
</tr>
<tr>
<td>A(T)=A_0<em>1</em>1*S(T)</td>
<td>1.4</td>
</tr>
<tr>
<td>Structural behavior factor R</td>
<td>2.0</td>
</tr>
<tr>
<td>V_t=WA(T)/R</td>
<td>20229650N</td>
</tr>
</tbody>
</table>

\[ \tau_{\text{av}} = 2.32 \text{ MPa} \leq 2.5 \text{ MPa} \]

\( (2.32\text{MPa} < \tau_{\text{max}} = 2.5\text{MPa}) \) as calculated, structure is safe in terms of base shear.

- Total wall area in one direction has to be higher than \%10 of total area.
X direction $\gamma_{1,x} = 9.5 \% < 10 \%$ acceptable.

Y direction $\gamma_{1,y} = 18 \% > 10 \%$ acceptable.

- The ratio of total area of the wall to the weight of building has to be higher than 1.2 $m^2/MN$
  
  $$\gamma_2 = \frac{2.1}{6.11} > 1.2$$

- Shear strength capacity of building to equivalent earthquake load ratio have to be higher than 1 (Table 2).

<table>
<thead>
<tr>
<th>Table 2: Calculations of shear strength capacity</th>
</tr>
</thead>
<tbody>
<tr>
<td>Total wall area through X direction</td>
</tr>
<tr>
<td>Shear strength capacity of building on X direction ($F_{Rd,i}$)</td>
</tr>
<tr>
<td>Equivalent earthquake load ($F_e$)</td>
</tr>
<tr>
<td>$\gamma_{3,x} = \frac{F_{Rd,i}}{F_e}$</td>
</tr>
<tr>
<td>Total wall area through Y direction</td>
</tr>
<tr>
<td>Shear strength capacity of building on Y direction ($F_{Rd,i}$)</td>
</tr>
<tr>
<td>Equivalent earthquake load ($F_e$)</td>
</tr>
<tr>
<td>$\gamma_{3,y} = \frac{F_{Rd,i}}{F_e}$</td>
</tr>
</tbody>
</table>

**Evaluation of the structural Analysis**

The calculations show that masonry walls have adequate stiffness. If any damage is to be observed when the plaster is uncovered, necessary repairs has to be made. Load bearing iron beams on the ceiling of the first floor were corroded and in some parts, especially on the supports, section had lost. Timber studs and joists also have adequate stiffness. But the elements that are deteriorated have to be repaired or replaced with suitable ones having the same cross-section.

Suspended 6.0m long iron 2I160 beams to upper timber beams during the addition of the second floor might be a good solution for that time. But when the iron hangers were loosened due to the deflection of the timber beam above the second floor, the 2I160 beams were not capable of carrying the floor load causing deformations of the decorated ceiling.

4 **PROPOSED INTERVENTION METHOD**

4.1 Structural Restrictions

For intervention decisions for the deflected ceiling of the main hall in the first floor, there were some restrictions on height of the load bearing floor. The distance to be spanned by the timber beam was 6.0 meters and because the walls also had paintings, the height of the beam, joists and the finishing shouldn’t exceed 26 cm. Besides, the iron supports of the I160 beams were almost lost due to corrosion.

To accomplish these restrictions analytical works for several construction materials were made. Using I beams to span 6.0m necessitated higher than the required height. Using composite wood might meet the need, but none of the firms gave more than 5 years guarantee. And the restoration architects pronounced that their service life depend upon the chemical’s service life. As a result, it was decided to make floor high two steel trusses spanning 6.0m in
place of the deteriorated walls of the second floor (Fig. 9). Steel columns are extended down to the ground floor on the masonry walls.

![Figure 9: Plan and of designed truss](image)

The calculated sections of the columns were IPN240. For the truss members; lower chord was IPN200 + 2L200.100.20 unequal angles, upper chord was IPN200, vertical elements were IPN200 and diagonals were IPN160. These dimensions made it possible to hide the steel truss in the timber wall of repeated studs without changing the dimensions of the original wall. The lower chord having 200mm height also let placing the 18cm joists with top and bottom covering in the required 26cm (Fig. 10). Timber joists are placed at 30cm intervals as they were in the original.

![Figure 10: Timber joists and studs](image)

The timber joists of the second and attic floor are designed to rest on bottom and lower chords of the steel truss (Fig. 11). Special insulation measures as laying neoprene have to be
taken between steel and wood to avoid the steel corroding because steel and wood are incompatible materials.

![Figure 11: Shape of lower beam of truss with timber beams](image)

During intervention, the exterior walls will be kept in their place and only deteriorated elements will be repaired or replaced. Because the openings are not large enough, most steel elements have to be welded on site. So, special care has to be taken to prevent fire during this process.

5 CONCLUSIONS

Choosing an appropriate treatment for a historic building necessitates variety of factors, including the property's historical significance, physical condition, proposed use, and intended interpretation. The decisions on interventions generally depend on the quality of the building materials, construction techniques, the location and amount of damage.

In the case of historic timber structure with heavy deterioration, material loss, high deformations and section loss it is not possible to keep the deteriorated elements.

The 19th century, five storey (basement + ground floor + 2 floors and an attic floor) timber building which is to be used as district governorate had lots of problems with its wall studs, load bearing beams, floor joists and floors due to lack of maintenance and wrong interventions applied in the past. The main problem was the deflection of the densely decorated ceiling of the main hall of 6.0x13.0 m at the first floor. The two internal walls dividing the space over the spacious room into three on the second floor had caused this deflection.

For intervention, the suspended 21160 beams spanning 6.0 meters at the ceiling of the first floor had to be replaced. The restriction was that the floor height of the second floor including the beam, joists and the finishing shouldn’t exceed 26 cm. To accomplish these restrictions calculations of several construction materials were made. And it was decided to make a floor high steel truss in place of the deteriorated wall of the second floor. During intervention process, special measures have to be taken between steel and wood. Also, special precautions have to be taken against fire risk especially during welding process of the steel profiles.

In this case study, the intervention decisions were undertaken demonstrating that they are indispensable. Bearing in mind safety and durability requirements, the least invasive technique was chosen as stated in ICOMOS ISCARSARH Principles [6].
REFERENCES


