STRUCTURAL ASSESSMENT OF THE MASONRY VAULTS OF A PAVILION OF HOSPITAL DE SANT PAU IN BARCELONA

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ABSTRACT. The large complex of Hospital de la Santa Creu i de Sant Pau in Barcelona, built between 1901 and 1930 and designed (in their largest part) by architect Lluís Domènech i Montaner, constitute one of the major examples of the Catalan Modernism which flourished in Barcelona in the early 20th century. The historical part of the complex was inscribed as UNESCO World Heritage Site in 1997. In addition to their interesting architectural and aesthetic value, the buildings draw attention because of their genuine structural features. The structures combine a significant amount of steel members with masonry ones. In most of the pavilions of the complex, masonry is utilised for walls and single- and double-curvature Catalan vaults forming the floor slabs and ceilings, while steel members provide the main vertical structure sustaining the vaults as well as the ties necessary to retain their horizontal thrust. The combination of steel and thin tile vaults permits diaphanous and functional spaces, both wide and tall, with large windows allowing satisfactory levels of natural light. A large part of the complex is at present being subjected to detailed studies and restoration works for the rehabilitation and adaptation to new uses. The buildings have required a deep repair due to the corrosion of steel members. The paper comprises two parts. Firstly, the structure of one of the pavilions, Nra. Sra de la Mercè pavilion, is described with focus on the vault morphology and the combined use of steel and masonry members. Secondly, a tentative structural analysis is presented on two different selected vaults of the same pavilion. The analysis carried out, aimed to provide an estimation of the loading capacity, takes into account the combination of masonry and steel members that compose the vaulting system.
1 INTRODUCTION

In 1901, LLuis Domènech i Montaner was committed with the design of the large hospital complex that is today known as Hospital de la Santa Creu i de Sant Pau in Barcelona, Spain (Figure 1). His proposal for the buildings of the hospital was strongly based on two contemporary concepts related to the architectural hygiene of hospitals. Firstly, as a way to improve recovery rates and decrease mortality, patients were supposed to breathe pure air. Secondly, it was also important to group patients according to types of illness and to separate them from one another. Domènech i Muntaner attained the first requirement by providing sufficient volume and effective ventilation to the buildings. The latter was achieved by designing the hospital as a set of numerous different individual pavilions, part of which were designed and built by him until 1913. Domènech i Muntaner enjoyed an almost absolute freedom in the design, construction and decoration of the pavilions. Today the Hospital Sant Pau is regarded as one of the major examples of the Catalan Modernism which flourished in Barcelona in the early 20th century. The historical part of the complex was inscribed as UNESCO World Heritage Site in 1997. The buildings show genuine structural features, as will be described in the following section.

The complex has actually been used as a hospital until recent time. Due to the need for additional space and more modern facilities, the construction of a new hospital complex was decided in 1990. The moving of the hospital to the new premises allowed the restoration of the modernist buildings and their adaptation to new uses. At present, most of the buildings are intended to host offices of international organisations. The restoration of the modernist pavilions has motivated comprehensive studies on their structure and architecture oriented to respectful conservation and rehabilitation interventions. In most of the pavilions, additions implemented during the 20th c., such as intermediate stories and partition walls, have been removed in order to recover the original spaces and construction features designed by Domènech i Muntaner. More information on history of Hospital Sant Pau can be found in [1].

Figure 1: Original drawing showing the general plan and distribution of the pavilions envisaged by Domènech i Montaner.
The present paper includes, firstly, a description of the structure of Ntra. Sra. De la Mercè pavilion with specific attention towards the vaulting systems composed of masonry and steel profiles. Secondly, a tentative analysis of two selected vault types is presented. The vaults chosen for the analysis correspond to the ones used to form the floor slab and the ceiling of the central main space of the pavilion.

2 DESCRIPTION OF THE STRUCTURE

Ntra. Sra. De la Mercè is one of the eight pavilions located in the centre of the modernist complex. The exterior of the building is seen in Figure 2. The building is composed of four parts: entrance area (corresponding to zone 1 in Figure 3, a), dome area (zone 2), central area (zone 3) and rear area (zone 4). The entrance and rear area are both composed of basement, ground and first floor levels, all roofed with double curvature vaults. The dome area has an underground level, roofed with a circular vault, and ground floor roofed with a masonry dome. The central area includes an underground floor consisting of simple curvature vaults and a ground floor roofed with complex double curvature ones.

![Figure 2: Mercé Pavilion before the restoration: (a) west façade and (b) south lateral façade.](image)

This research focuses on the central area. It is composed of seven two-story similar vaulted bays (Figure 3, a). The dimension of the bay is of 3x9 m² in plan. The height of the story is of 4 m at the underground level and 7.5 m at the ground level (Figure 3, b). Each bay has two windows in the ground floor at both ends (Figure 4, b). The dimension of the upper window is 3x2 m² and that of the lower is 1.1x2.5 m². The ground and underground floors are composed of double- and single-curvature Catalan vaults respectively (Figure 4). In this paper, the single-curvature vault that roofs the underground level is named lower vault and the double-curvature vault that roofs the ground level is named upper vault in accordance with their location. In both the vaults and the walls of the building, the masonry is composed of clay bricks and lime mortar. Steel profiles, embedded in walls and vaults are abundantly used as part of the structure of the building. Before the restoration works, the steel profiles in both the lower and the upper vault were affected by corrosion at various locations (Figure 5).
Figure 3: Pavilion of Ntra Sra. de la Mercè in Hospital Sant Pau: (a) plan of ground floor, (b) section of the structure of a typical bay.

Figure 4: Pavilion of Ntra Sra. de la Mercè in Hospital Sant Pau: (a) upper and (b) lower vaults after the restoration.

Figure 5: Corrosion of steel profiles embedded in masonry vaults before the restoration works.
As aforementioned, the lower vault is a single curvature one (Figure 4, b). Its transverse span is 3 m and its maximum rise is 0.35 m, thus showing a rise/span ratio of 0.116. I-beam steel profiles (IPN 240) are placed longitudinally along the springing of the vaults to support them (Figure 6). These profiles are connected to a couple of vertical U-shaped steel profiles (UPN 200) embedded in the façade wall. The vault is composed of three layers of solid bricks bonded with lime mortar. The first layer (from the intrados of the vault) is 40 mm thick and the second and third layers are 20 mm thick. The thickness of the mortar beds is 5 mm. The total thickness of the vault is 90 mm.

The upper vault is a double-curvature one with a span of 9 m in the longitudinal direction of the vault and a span of 3 m in its transverse direction (Figure 4, a). The rise at the perimeter of the vault is 0.4 m (with rise/span ratio of 0.133) in the transversal direction and 0.8 m (with rise/span ratio of 0.0889) in the longitudinal direction. The maximum rise, at the centre of the vault, is 1.05 m. Like the lower one, the vault is composed of three layers of solid bricks bonded with lime mortar. As in the previous case, the first layer is 40 mm thick and the second and third layers are 20 mm thick. The thickness of the lime mortar beds is 5 mm. The total thickness of the vault is 95 mm since the intrados is covered with 5 mm-thick tiles as seen in Figure 5.

As in the lower one, the upper vault is supported on steel profiles. In the case of the upper vault, however, the supporting system is more complex and redundant, and involves not only the existing steel structure but also the upper masonry arches that shape the roof of the building. The vault is directly supported on two different steel members. On the one hand, the vault is supported, along its lateral sides, over curved T profiles shaped as an arch. On the other hand, the vault is also supported, at mid-span of its lateral sides, on two horizontal U-shaped steel profiles (UPN 200). These horizontal profiles are, in turn, supported on steel pillars embedded in the façade walls. The horizontal profiles are also suspended, at a certain distance from their connection to the pillars, from diagonal steel profiles (also UPN 200 ones) that hang from the upper masonry arches (Figure 7, b-c). The diagonal profiles are also connected to the arched T profiles on which the vault is partly supported. The masonry arches that support the roof and also sustain the vault through the diagonal profiles are made of a brick masonry hollow box showing a width of 60 cm and a variable depth measuring 24 cm at the bottom and 100 cm at the top (Figure 7, a).
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Figure 7: Upper vault: (a) masonry arches shaping the roof; (b) horizontal and diagonal UPN profiles in which the vault is partly supported; (c) details of the steel structure that supports the vault; (d) steel skeleton of one of the pavilions visible during the construction. ((c) and (d) from [2]).

Figure 7, d, corresponding to a construction stage, helps understand the important role of the steel skeleton of the structure and the only secondary role of the masonry façade walls. More information on the structure of some of the pavilions of the Hospital complex can be found in previous publications [2, 3].

3 STRUCTURAL ANALYSIS

3.1 Material tests

A penetrometer test was carried out to examine the compressive strength of mortars in September, 2011. The test was carried out by means of a Windsor pin penetrometer according to the ASTM code C803/C803M-97 [4]. Tests were undertaken on the masonry of the walls of the façade, on interior load bearing walls and at the extrados of the lower and upper vault. The test allowed the measurement of a very low strength in the mortar beds of the façade walls, confirming their non-structural role. Conversely, the mortar of the masonry of the interior load bearing walls showed a higher compressive strength equal to 2.2 MPa in average. The compressive strength estimated for the lower vault masonry was 2.3 MPa. The value estimated for the mortar at the
extrados of the upper vault was surprisingly high (15.8 MPa). It is suspected that its high compressive strength is due to an original finishing or later repair with a kind of mortar different to lime one. The strength of bricks was examined by compression test on single units at the Laboratory of Materials and of Quality Control of the Technical University of Catalonia in September, 2011. An average compressive strength of 20 MPa was obtained.

3.2 Description of the model

The structural performance of the lower and upper vaults has been studied individually by means of FEM analysis. The analyses have been carried out considering material nonlinearity. The material properties of brick masonry and steel are assumed as indicated in Table 1. The compressive strength of masonry is assumed considering the values of compressive strength of bricks and mortar discussed in Section 3.1 taking into account different empirical correlations (Eurocode 6 [5] and PIET70 [6]) and also based on previous experience with similar brick masonries. Tensile strength is taken as 5 % of compressive strength and the Young’s modulus is estimated as 500 times the compressive strength. For the tensile fracture energy, a value of 50 N/m is assumed. The mechanical parameters of steel are determined based on information available on documents of the time of the construction. The recommendations on steel mechanical properties for historical steelwork by the British constructional steelwork association [7] have been also taken into consideration.

The Rankine’s criterion in tension and the Drucker-Prager’s criterion in compression are adopted as failure criteria for brick masonry. For the steel members, the Von Mises yield criterion is assumed as failure criterion. In this study, particular attention has been allocated to the modelling of the frictional behaviour at the contact between the steel profiles and the brick masonry of the vaults. To model this behaviour, a Coulomb friction model is considered [8]. Since information on the frictional behaviour in a masonry-steel contact is very limited or almost inexistent, values conventionally used to model friction between concrete and steel have been considered. A friction angle at the masonry-steel contact of 26.5° (tanϕ = 0.5) has been adopted [9]. For the value of cohesion, a very small value, of only 0.1 MPa, is adopted to take into account almost null cohesion between both materials. The masonry and steel contact is modelled by means of interface elements requiring also as input data values of the normal and shear linear stiffness. The values adopted these parameters have been 200 MPa/mm for the normal linear stiffness and 100 MPa/mm for the shear one.

3.3 Analysis of the lower vault

Two different models have been considered for the analysis of the lower vault. The first model represents a single-curvature vault supported on the IPN 240 profiles. In this model, no structural role is attributed to the masonry members existing over the vault (wallets and slab), of which only the weight is considered. The vault is discretised with 8-node quadrilateral curved shell elements (Figure 8). The steel profiles (IPN 240) are modelled with 3-node beam elements. The number of elements is 6,528 and the number of nodes is 9,473. The number of integration points of shell elements in thickness is 11. The model is restrained by rotationally fixed supports at the end of the steel profiles. The values adopted these parameters have been 200 MPa/mm for the normal linear stiffness and 100 MPa/mm for the shear one.
massive masonry pilasters as seen in Figure 3, b and Figure 6, b. Since in the real structure there are adjacent vaults on both sides, the transversal movement of the vault is restrained. Interface elements are adopted along the connection between the vault and the steel profiles.

The second model includes the vault, the steel profiles and the additional structural elements existing over the vault. These additional members are an upper masonry slab, the masonry transverse wallets on which it is supported and the longitudinal masonry walls that limit the vault laterally (Figure 9). The thickness of the slab is 10 cm and that of the wallets is 2.5 cm. These wallets are located at every 37.5 cm. The thickness of the longitudinal walls is 35 cm. The model prepared includes 32,351 nodes and 13,580 elements. The capacity of the vault has been analysed using the two models presented. A uniform live-load has been applied on the entire surface of the slab and increased gradually up to failure.

Table 1: Material properties

<table>
<thead>
<tr>
<th>Property</th>
<th>Brick masonry</th>
<th>Steel</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tensile strength (MPa)</td>
<td>0.2</td>
<td>250</td>
</tr>
<tr>
<td>Compressive strength (MPa)</td>
<td>4</td>
<td>250</td>
</tr>
<tr>
<td>Young’s modulus (MPa)</td>
<td>2000</td>
<td>200000</td>
</tr>
<tr>
<td>Unit weight (kg/m³)</td>
<td>1800</td>
<td>7850</td>
</tr>
</tbody>
</table>

Figure 8: FE model of the vault (first model).

Figure 9: FE model of the vault combined with its upper structural members (second model); (a) vault with upper slab and (b) interior wallets and longitudinal beams.
As for the first model, the maximum obtained live load capacity is equal to 10.4 kN/m². The corresponding deflection at the centre of the vault is 113.2 mm (Figure 10). Damage starts and propagates from the connections between the steel profiles and the vault (Figure 11). A second analysis has been carried out with applied conventional safety factors over the dead load (1.35) and over the live load (1.5) and also with reduced values of the material properties (with factors of 2.5 and 1.15 applied respectively over masonry and steel). In this second case, the resulting acceptable load capacity is 4.3 kN/m².

As for the second model, sliding starts under self-weight. Correspondingly, transversal damage across the middle of the intrados of the vault also appears. At the ultimate state, damage also appears in the middle of the longitudinal masonry walls (Figure 12). The maximum load is 13.2 kN/m² and the corresponding displacement is 82.5 mm (Figure 10). Significant damage is seen at the connection between the slab and longitudinal beams (Figure 12). Noticeable damage appears transversally around the middle of the intrados of the vault. When the safety factors over the loads and material properties are considered, the maximum acceptable live load is equal to 5.6 kN/m². In all the cases, the steel yield limit is reached at the end and mid-span sections of the steel profiles.

It must be noted that this load has been obtained assuming that the vaults and the steel beams can work together, according to the frictional laws adopted. However, since there is no experimental evidence on the combined action of steel profiles and vaults, from an engineering point of view it may be preferable to ignore this combined work and assume conservatively that all the load is resisted by the steel profiles. Under this assumption, the resulting capacity is significantly smaller than that predicted by the FEM analyses. Accepting this conservative approach requires an appropriate strengthening solution to grant the viability of the new uses foreseen for the building.

![Figure 10: Load-deflection curves as a relationship between deflection at the centre of the vault and applied uniform live load.](image-url)
3.4 Analysis of the upper vault

The model prepared for the upper vault includes the vault with the steel profiles and the masonry arch (Figure 13). The steel framework and the masonry arches are modelled with 3-node beam elements. The number of nodes is 12,697 and that of elements is 5,044. The same material and interface properties described in Section 3.2 are assumed. As in the previous case, the Coulomb friction model is adopted to model the contact between the T-steel profiles and the vault. The model is restrained by pin supports at the ends of the T-profiles. Also as in the previous case, a distributed uniform live load is applied all over the vault and increased until failure.

Damage in the vault starts to appear longitudinally both in the intrados and the extrados at a load of 4.5 kN/m². In the extrados, no other damaged regions are observed (Figure 14). The end of the analysis is reached for a maximum load of 9.4 kN/m². The displacement at the centre of the vault at ultimate load is 8.7 mm. When the safety factors over the loads and materials are considered, the resulting maximum live load is equal to 2.5 kN/m².
Figure 13: FEM model of the upper vault.

Figure 14: Principal tensile strain distribution close to the ultimate condition. The upper vault, vault extrados (a) and intrados (b).

It must be remarked that this estimation of the maximum capacity, for both the lower and upper vaults, is only based on the strength and does not consider other problems linked to the deformation of the vaults, such as the possible detachment of the decoration tiles that are bonded to their intrados. This detachment might in fact occur for a significantly lower value of applied live load. The maximum capacity can be also limited by the local strength of the connections between the steel members, which have not been modelled into detail in the analyses.

4 CONCLUSIONS

The Pavilion of Ntra. Sra. De la Mercé of the Hospital of Sant Pau in Barcelona, having a similar structural design to that of other pavilions of the Hospital, was designed with a complex structure combining masonry walls and vaults and steel members. In particular, most of the vaults of the structure, built according to the Catalan vaulting system, are both supported and laterally confined by steel profiles placed both horizontally and vertically around them. The study presented has focused on two typical vaults used to produce the floor slab and the roof of the central space of the pavilion. Remarkably, the upper slab is supported on a very complex and redundant system of steel profiles partly suspended from the masonry arches that shape the roof of the building.
Both the floor slab (lower) and roof (upper) vaults have been modelled and analysed in an attempt to obtain an estimation of their loading capacity. The FE models prepared include both the masonry and steel members composing the vault systems and have been provided with frictional joint elements to model the masonry-steel contact. The analysis has allowed the study of the initiation and propagation of damage up to failure with the increase of applied load.

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