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The structural strengthening of early and mid 20th century reinforced concrete diaphragms

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ABSTRACT: During the 20th century it was very common in Italy to substitute masonry buildings' timber floors and roofs, which were considered inadequate, by reinforced concrete or reinforced clay diaphragms. Such interventions, especially if belonging to the early stage of reinforced concrete spreading, emphasize new conservation problems. Whereas on the one hand such structural elements are testimony of the construction practice of one era, a practice frequently not in use anymore, on the other they are also commonly inadequate to carry the loads currently specified by the building codes. Two such examples are to be found at Villa Borghese's Casino dell'Orologio, in Rome (Italy). One consists of reinforced clay unit beams, and the other of a thin ribbed reinforced concrete slab. The two have been strengthened in different ways as follows: 1) the first by replacing a few of the clay beams with reinforced concrete beams, 2) the second by adding an elastic support at the ribs' midspans. However, the intervention attitude is the same: preserving historical examples of construction techniques. Such attitude, as well as the structural calculation formulation, the construction details, and the approach when the knowledge of the existing structure is not complete are useful for other similar cases.

1 INTRODUCTION

Reinforced concrete floors were frequently used, during the 20th century in Italy, to substitute masonry buildings' timber floors and roofs, considered inadequate (Sorrentino et al 2007). Such interventions, particularly if belonging to the early rise of reinforced concrete, induce new conservation problems.

Such structural elements are witness of the building habit of one period, a habit frequently disappeared. Nonetheless, they are usually inadequate to carry the loads currently specified by the code of practice. Moreover, their complete substitution can put at risk the equilibrium condition of the masonry edifice, during the construction works.

Examples of such challenging problems are to be found in the 16th–18th centuries Villa Borghese's Casino dell'Orologio, in Rome (Italy). The precedent timber roof, probably not original, was removed at some stage around mid 20th century and replaced in one wing by a ribbed reinforced concrete slab and in an another one by a reinforced clay floor.

The first one, distinguished by very slender ribs, represents a solution currently rather unusual, since requires an expensive formwork. The second is a testimony of the strain to reduce the reinforcement while Italy, a country traditionally poor of iron, was subjected to an embargo due to the Fascist colonial wars.

Adopting the same intervention philosophy employed for masonry historical constructions, it is possible to avoid their complete removal, and to simply strengthen them. However, contrary to timber or steel elements, the retrofitting of reinforced concrete horizontal structural components is no straightforward task. Indeed it is not possible to merely reduce the span by adding some girder, since in such case inverted bending moments can be induced where no reinforcement is present. Therefore, two different strategies have been pursued.

The overall philosophy, the technical solutions, and the design procedure, although inspired by the particular features of the case at hand, are general and can constitute a useful reference for those facing the same kind of problems.

2 CONSTRUCTION HISTORY OF THE "CASINO DELL'OROLOGIO"

The original nucleus of the current "Casino dell'Orologio" (= clock lodge) was already existent when Cardinal Scipione Caffarelli Borghese bought a vineyard with a building in 1606 (Campitelli 2003a, 19). The building was initially known as the "house of the gardener" (*ibidem*, 43 and 195) or "house of the custodian" (*ibidem*, 244).

The Casino appears, already with an *L*-shaped plan and a double-pitched roof, in the 1676 plan of the Villa by Simone Felice Delino (*ibidem*, 13 and 193). However, the North-South wing seems to have two storeys, whereas the East-West one looks shorter.

According to Jacopo Manilli, who wrote in 1650 (Di Gaddo 1997, 135) the Casino had already two storeys and a courtyard.

Domenico Montelatici, 50 years later (Barberini 1966, 57), noted that the ground floor hall had a frescoed vault. Such frescoes were uncovered during recent maintenance works under a layer of white painting.

More accurate is the representation of the edifice made in 1786-91 by C. Perrier, a scholar at French Academy in Rome (Di Gaddo 1997, 142–143). The Casino has two floors above ground, the first one being very tall and with buttress-like walls, the plan is *L*-shaped, with its rectangular perimeter enclosed by a boundary wall, the roof is double-pitched.

In 1791 Marcantonio IV Borghese decided to devote the building to museum of the pieces of art uncovered in his possessions in the ancient Gabii (not far from Rome). Initially, he thought to erect a new facility (Campitelli 1998 and 2003b), while later he opted to refurbish the existing lodge (Campitelli 2003a, 317–320). Architect Antonio Asprucci was responsible for the design and the following works, completed in 1793. Asprucci added the tower, gave to the masonry a rusticated ashlar finishing, and opened the great arches, necessary to adequately light the sculptures exposed (Figure 1). The clock in the tower is due to engineer-architect Nicola Fagioli, whereas the bells were casted by Giuseppe Valadier.

The life of the Casino as a museum was rather short. In 1807 Camillo, son of the late Marcantonio, sold his collection to Napoleon. The sculptures were partially replaced by other villa's statues and by the pieces uncovered during further archeological diggings in Sabina, a region at the east of Rome. Nonetheless, in 1832 the Casino was practically empty (*ibidem*, 326). Later the lodge hosted the exhibitions of the society of Roman acquerello painters (Barberini 1966, 58).

According to Barberini (1966, 57) between 1832 and 1834, the Casino was given in concession to a Stefano Giovannini who established a restaurant. However, Di Gaddo (1985, 143 and 208) states that Giovannini was given another building in the Villa.

A few views of the building appear in the first half of the 19th century. Both in G. Cipriani engraving of 1817 (Campitelli 2003a, 320), and in 1840s E. Landesio's and P. Rosa's engraving (*ibidem*, 474), as well as in I. Caffi's painting (Della Pergola 1962, p. 67 and fig. 8) the building has a terrace roof already.

In 1915 the big arcades of the ground floor are already partially closed, as shown in a land register plan (Luciano Cupelloni, personal communication).

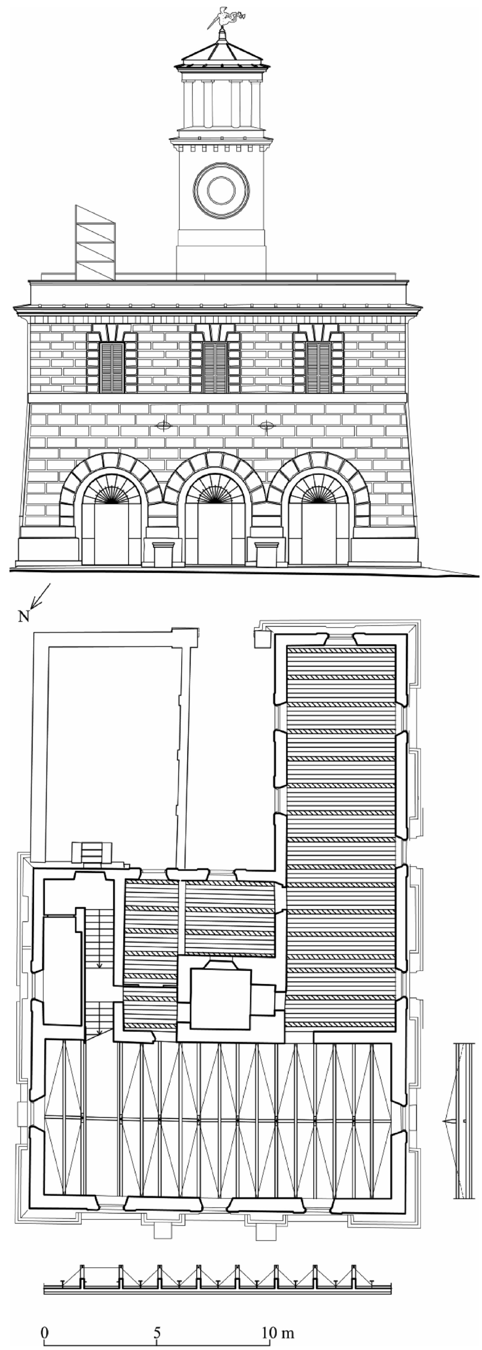

Figure 1. Plan of the first floor with projection of the roof structure (both existing and added) and north-west façade. Strengthened reinforced clay beams are dashed; in the bottom and right-lateral small sections is presented the three-dimensional bow-string retrofitting of the ribbed reinforced concrete roof.

Probably, this was due to the presence of municipality offices in the building. As a matter of fact the entire Villa became city property in 1903. Unfortunately the date of the reconstruction of the roof with reinforced concrete structural elements is not known. It might be suggested that it happened between 1939 (see in the following), and 1962, when the head office of the public garden services is already housed in the Casino (Della Pergola 1962, 79). Moreover, with the reinforced concrete standard of 1972 (DMLLPP 1972) such clay reinforced beams were practically banned since a minimum 5 mm concrete cover of rebars was required to prevent corrosion. As a matter of fact, corrosion problems in the reinforced clay beams have been reported in Michetti (2000, §C5-3b).

In 2004 the municipality decided to move the office for the city historical centre in the Casino, while transferring elsewhere the office for the garden services. General maintenance works, both structural and non structural, were carried out in that occasion. However, for the sake of brevity, such works are not reported here. The paper will focus on the roof structure strengthening only, made necessary by the renovation program. As a matter of fact, the plan assumed to change the terrace from a simple roof, subjected to a limited snow and maintenance load (around 1.0 kN/m^2), to a space for public exhibitions (live load in the order of 4.0 kN/m^2).

3 STRENGTHENING OF EARLY 20TH CENTURY REINFORCED CONCRETE FLOORS

3.1 *Reinforced clay beams floor*

The roof over the south-west wing and the central section of the building consists in a reinforced clay beams floor, called SAP. The acronym stays for "Solaio ad Alta Portata" (= high load-bearing capacity floor) according to Iori (2001, 172), or for "Struttura AutoPortante Senza Armatura Provvisoria" (= self-supporting structure without provisional falsework) according to Campioli & Laner (1992), who nonetheless do not quote any reference. The system was patented in 1932 by the RDB, Rizzi Donelli Breviglieri, company (Iori 2001, 173 and 188). It gained great success after an embargo was imposed to Italy in retaliation to the Fascist colonial wars in Africa. Such embargo caused a shortage for steel, and consequently encouraged lightly reinforced concrete structures (*ibidem*, 157–158). The SAP floor was probably the most successful among others in those years. In the late 1930s it was even recommended by the Italian Ministry of Public Works (*ibidem*, 174 and 188; Campioli 1992). This kind of floor, employing clay units, belongs to a tradition very common in Italy

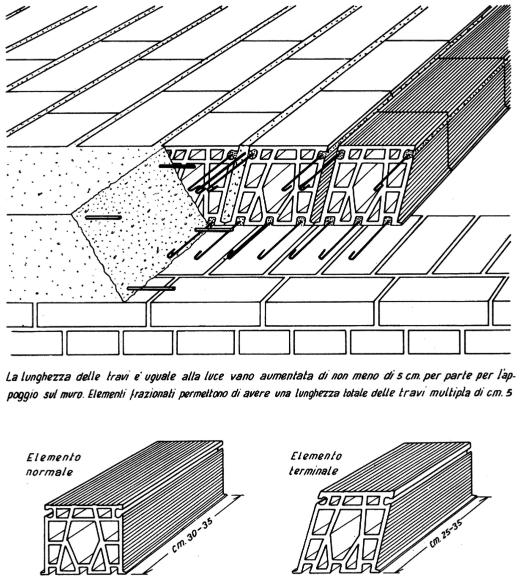

Figure 2. Reinforced clay floor in Colonnetti (1953, pl. E8g, detail).

and Spain, while rather unusual in the rest of Europe (Campioli 1992; Campioli & Laner 1992). Due to wide presence of clay in Italy, such elements were cheap from an economical point of view, and reduced the self-weight of the structure from an engineering point of view. In addition to this, this floor removed the need for a formwork, and enhanced both the thermal and the acoustic insulations.

The SAP floor consists of parallel beams made up by clay units, with a width of 20 cm and height between 8 and 20 cm depending on the span. The units were assembled on site on the ground upside-down, by placing the small diameter rebars in special chases, sealed with a fluid cement mortar. The beams were then laid one close to the other, without the need of a falsework. This was a great benefit, especially when the height of the room was not negligible, as in the Casino. Between the beams a very thin (3 cm) concrete rib was cast in place (Figure 2). The reinforcement consisted usually in three high strength (700–800 MPa), small diameter, steel rebars at the bottom and two at the top. The floor had frequently no top slab, substituted by a higher density of webs, as allowed by the 1932 Standard (RDL 1932, §23). This solution, based on extensive laboratory tests, ignited a vivacious debate (Iori 2001, 173–174). The 1939 Standard added the recommendation of a transversal rib in the span exceeded 5 m (RDL 1939, §25). (The request of a rib's 7 cm minimum width seems to have not been enforced).

Figure 3. Reinforced clay floor in the Casino.

Figure 4. Reinforced clay floor: one out of six clay beams is replaced by a reinforced concrete one.

There was no design of such floors. Instead the practitioner recurred to the tables supplied by the manufacturer and based on experiments (e.g. refer to Colonnetti 1946 and 1953, pl. E8g).

Usually a safety coefficient equal to 2.5 was assumed to get working bending moment from failure bending moment (Colonnetti 1962, 81). The very wide success of SAP floors is proven by its presence in handbooks until the end of the 1960s (Arosio 1969, 596), but not later (Arosio 1978, 658–660).

In the Casino the clay beams are 20 cm high, the bottom longitudinal reinforcement consists in three 5 mm diameter plain bars, and the top reinforcement in two 2 mm diameter plain bars.

The maximum clear span equals 4.8 m. Whenever the span exceeds 5 m, as in the central section of the Casino, a transversal beam is present. Hence, it is possible that the floor was built after the 1939 standard. The cross section of the clay unit (Figure 3) is very similar to the one in Figure 2, whose first representation is that in Corsetti (1939, 313). Previously the SAP floor had a different shape either without the central hexagon (Guerra 1938, 263; Colonnetti 1946, pl. E8g; Iori 2001, 172), or with only one rebar chase in the bottom face (Arosio 1941, 408). Moreover, the first time that a 20 cm height is represented is in Colonnetti (1953).

A 2 cm concrete finishing is present on top of the beams. The reinforcement is anchored in a lateral beam, reinforced with four plain 6 mm diameter bars. Overall state of conservation is rather good.

Based on the data available in the technical literature (Colonnetti 1953 and 1962) the extant floor is assessed capable of carrying a load per unit area of 4.4 kN/m^2. Since the dead load, without considering the floor structure self-weight, is estimated in 3.25 kN/m^2, and the live load imposed by the program is 4.0 kN/m^2 the reinforced clay floor is considered inadequate. Moreover, the structure lacks any capacity of laterally distributing whichever concentrated load. In order to avoid a complete removal of the existing structure, which would leave the walls unrestrained, would require substantial interventions on the masonry, and would destroy a testimony of a typical construction technique of the past, a different solution is devised.

Already after the 1980 Irpinia earthquake, guidelines have suggested to retrofit existing reinforced concrete floors, with clay units embedded, by demolishing some of the units and replace them by reinforced concrete elements (CMLLPP 1981, §4.3.4.5). Such a solution has been also graphically represented in handbooks (Furiozzi et al 1984, 457). Moreover, in a appraisal about a 1930s building in Central Italy, which has SAP floors, Professor Antonino Giuffrè suggested to remove some of the clay beams and replace them by reinforced concrete elements (Giuffrè 1994).

Therefore, it has been computed that it is sufficient to replace one out of six clay beams (Figure 4), with some occasional thickening in order to avoid supports over the openings (Figure 1, dashed beams). Furthermore, a 5 cm concrete slab reinforced by welded wire mesh is cast on top of the existing structure. This grants a load distribution capacity to the terrace and connects the new reinforced concrete beams.

In addition, the slab gives to the reinforced concrete beams a very convenient T cross section (Figure 5).

The use of reinforced concrete for the retrofitting has the advantage of employing the lateral clay beams as part of the formwork. In addition, it guarantees a good bond between new and existing structure. The adopted solution markedly reduced the need for formwork and temporary shoring, taking advantage of the load-bearing capacity of the preserved clay elements (Figure 6).

The reinforced concrete beams are designed in accordance with the Standards in force at the time of

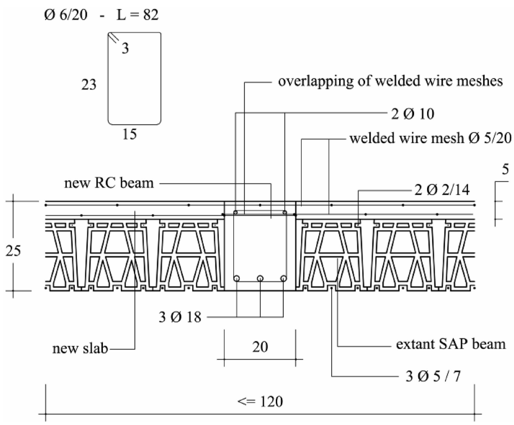

Figure 5. Reinforced clay floor: cross section of the intervention.

Figure 6. Intrados of the reinforced clay beams floor after the intervention.

the maintenance works (DMLLPP 1992 and 1996). In an ultimate limit state design, the loads are multiplied by the appropriate partial safety factors, whereas the capacity of the extant clay beams is assumed equal to its tabulated working load. This will allow for a safety coefficient to account for a possible material decay. The new beams are not continuous over the walls, since also the clay elements are not continuous.

The new reinforced concrete beam are designed also to act as tie for the façades in case of an earthquake.

3.2 *Ribbed reinforced concrete roof*

The ribbed reinforced concrete horizontal roof covers the north-west wing of the Casino, approximately a rectangle $15.15 \times 6.90\,m^2$ (Figure 1). The eight ribs span over the short side, exceeding the usual length of SAP elements, and have an approximate

Figure 7. Ribbed reinforced concrete floor before intervention.

spacing of 1.70 m, with a $12 \times 45\,cm^2$ cross section (Figure 7).

Before the intervention only limited investigation were possible on the geometry. Close to the support each rib has two 14 mm plain rebars as bottom reinforcement.

6 mm plain stirrups, whose spacing equals 33 cm, are used as lateral reinforcement.

The slab, only 5 cm thick, has 10 mm plain rebars, placed every 38 cm. No investigation was possible at midspan or at the extrados. The dead load was estimated in $4.25\,kN/m^2$. Preliminary calculations, considering such dead load only, strongly suggested the presence of additional reinforcement at midspan. Probably these rebars were bended close to the supports.

Moreover, no investigation at all was viable on the materials' mechanical characteristics. As a matter of fact, non destructive testing (ultrasonic, Schmidt hammer, etc.) were not considered fully reliable, since the concrete was already rather old, and carbonation was expected. Destructive tests, such as drilling cores, was at that moment not feasible due to the height of the roof (more than 5 m) and the occupancy of the building.

Therefore, uncertainties about geometry of the reinforcement and material mechanical properties are substantial.

Given the mentioned uncertainties, the design formulation is that of inducing in the structure after the interventions no internal loading higher than the one induced by the extant dead load. Such attitude assumes that the available capacity against live load is negligible. This is due both to the temerity of the structure and to the limited value of the snow load considered before modern codes (Breymann 1884, vol. 2, ch. 2, sec. 4c).

A first round analysis shows that the existing structure is not adequate to safely carry the loads required

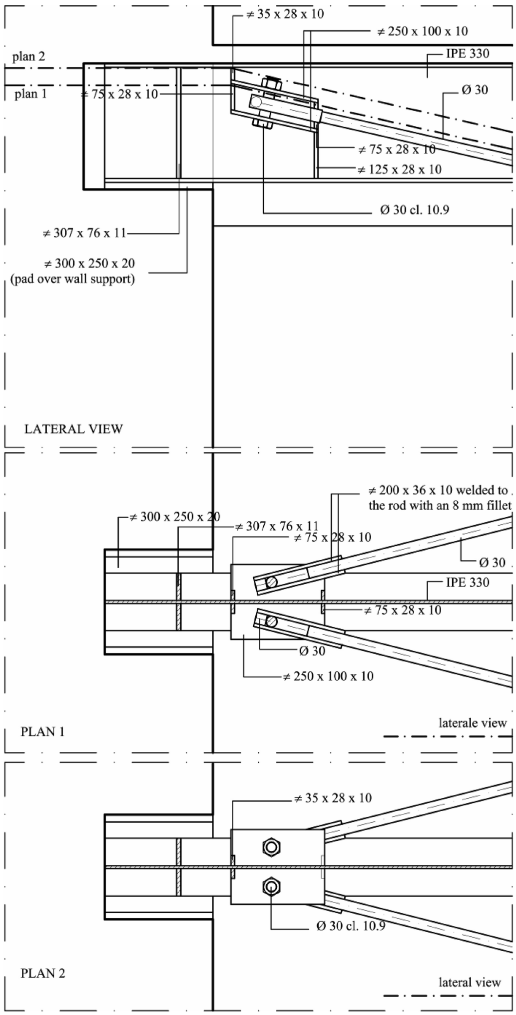

Figure 8. Detail of the intersection between the rods and the I beam.

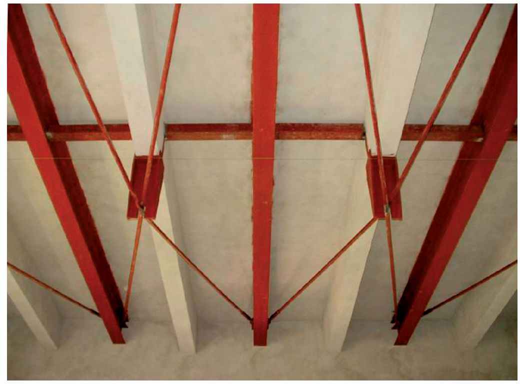

Figure 9. The ribs are supported at midspan by two adjacent bow-string beams.

Figure 10. The system of three-dimensional bow-string beams.

by the new use of the building. Hence, it is necessary to strengthen the structure.

Whereas timber and iron beams can be retrofitted simply by reducing their span, e.g. by means of intermediate additional supports, reinforced concrete ones usually cannot. In consequence, such supports generally will induce reversed bending moments, which the beam cannot carry due to lack of top reinforcement.

Nonetheless, if such supports are not fixed but instead act as an elastic support a more convenient bending moment diagram can be caused within the beam. Therefore, an elastic support at midspan is assumed as strengthening solution.

The retrofitting intervention consists of a system of steel three-dimensional bow-string beams, supporting each a reinforced concrete rib at midspan (Figures 1, and 8–11).

The top chord of the bow-string is made up by a steel *I* beam placed between two adjacent reinforced concrete ribs (Figure 8). In addition to the compressive component induced by the 30 mm diameter rods, the *I* beams contribute to halve the tributary width of each rib. The *I* beam section is selected in order to make the deflections of the existing reinforced concrete rib and of the steel added element compatible. Thus, a IPE 330 profile is necessary.

In order to make the slab effect similar both in the rib and in the *I* beam, the latter is bonded to the reinforced concrete deck through shear connectors.

Such connectors consist of screw bolts, fixed with both extrados and intrados nuts to the top flange of the *I* beam, having another nut at the opposite end. Since the original slab was so thin, an additional 3 cm lightweight concrete layer, reinforced by a welded wire

Figure 11. The exception in the system of bow-string beams were the stairs are going to be built.

mesh, is cast on top of the extant one. Having preserved the ribbed floor, no temporary formwork is necessary. The top nuts of the shear connectors are embedded in such additional concrete layer. Furthermore, these connectors prevent the buckling of the compressed I beams.

The serial system of bow-string beams has an exception where the stairs are located (Figure 1 and Figure 11). At the two ends of the wing the compression induced by the rods is transferred to an alignment of square box-section beams. These beams run along the longitudinal axis of the north-west wing (Figure 1 and Figure 9).

The entire intervention is designed to get an appropriate deflection at the rib's midspan, in order to obtain a convenient bending distribution.

Assuming a vertical deflection of the midspan support equal to δ, the bending moment M of a continuous beam at midspan amounts to:

$$M = \frac{p\, l_c^{\,2}}{32} - \frac{12\, E_c\, I_c}{l_c^{\,2}}\,\delta \tag{1}$$

with $p =$ design load per unit length, $l_c =$ rib design span, $E_c =$ reinforced concrete effective Young's modulus, $I_c =$ reinforced concrete cross section axial moment of inertia.

The vertical deflection of the rods at midpsan is equal to:

$$\delta = \frac{R_M\, l_s}{4 E_s A_s \sin^2 \alpha} \tag{2}$$

with $R_M =$ reaction transferred by the reinforced concrete rib at midspan, $l_s =$ length of each rod, $E_s =$ steel Young's modulus, $A_s =$ rod cross section area, $\alpha =$ angle between the rod axis and the horizontal axis.

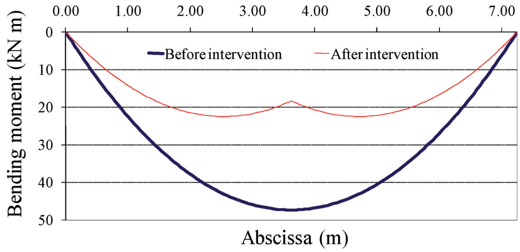

Figure 12. Bending moment diagram before and after intervention.

However, the reactions at support, R_A, and at midspan in equilibrium and compatible with the bending moment of Equation (1) are:

$$R_A = \frac{p_c l_c}{4} - \frac{2M}{l_c^{\,2}}; \qquad R_M = 2\left(\frac{p_c l_c}{4} + \frac{2M}{l_c^{\,2}}\right) \tag{3}$$

R_M is estimated by means of an iterative analysis. The bending moment diagram, after the convergence, is presented in Figure 12. Everywhere the internal loading is lower than the one induced by the dead load acting before the intervention. Therefore, any uncertainty about geometry and mechanical properties are experimentally overcome. Moreover the elastic support avoids any bending moment reversal.

The previous analysis has checked only the bending moment. On the other hand, it is necessary to take into account also the shear close to the elastic support, where it certainly exceeds the one before the intervention. This is accomplished assuming a very low strength of the concrete. Then, the shear capacity of the concrete section only, i.e. neglecting any lateral reinforcement, is computed. This is higher than the shear induced by the additional support. Thus, the proposed intervention is feasible.

4 CONCLUSIONS

Substitution of extant timber floors and roofs by reinforced concrete structures has been rather frequent, although not always justified, in Italy in the past. Nonetheless, especially in the first decades of the widespread use of reinforced concrete interesting solutions were employed. Two examples worthy of note are present in the Casino dell'Orologio of Villa Borghese, in Rome. They were uncovered when the municipality

decided to change the use of the building and carry out a general maintenance program.

A reinforced clay beams floor was observed over one wing. Although in a reasonable conservation state, the structure is not adequate for the new destination of the building. A complete removal would destroy a testimony of a past typical way to build. Moreover, it would call for temporary shoring works of the unrestrained masonry façades. Thus, a different solution is preferred. One out of six clay beams is replaced by a reinforced concrete beam, cast together with a slab.

Thus, the existing floor is preserved both in its material substance and in its structural function, since the deck continues to carry part of the roof load. As a matter of fact, the study of the technical literature of the time allows not only for the recognition of the technique, but also for the estimation of the load that can still be carried. However, this is reduced to take into account a possible material decay.

On another wing of the building a temerarious ribbed slab was uncovered. Very limited investigation was possible on the structure, but the lack of information is compensated by means of a careful design.

A system of three-dimensional steel bow-string beams is conceived, in order to add an elastic support at midspan. Such elastic support reduces the bending moment under the new live loads. Therefore, every section of the rib undergoes a bending moment smaller than the one present before the strengthening, although the load is increased. Thus, any uncertainty in rebars detailing and material mechanical properties is overcome by the *in situ* experiment represented by the structure under dead load before intervention. Moreover, since the support is not fixed, no negative bending moment is induced. Hence, no top reinforcement is needed.

The two examples given here are rather different in some way. The first is quite discreet, while the second has a high visual impact. Even though, they share the same overall intervention philosophy: strengthening the existing structures, while preserving them as a testimony of a widespread construction practice.

Such attitude, as well as design formulation, construction details, and approach when the knowledge of structure is not complete, are useful for other similar cases.

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