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Analysis of Wooden Roofing Structures in Monumental Buildings

Anna Marzo, Beatrice Faggiano and Federico M. Mazzolani
University of Naples "Federico II", Department of Structural Analysis and Design, Naples, Italy

ABSTRACT: The paper deals with the analysis of the roofing structures of the Historical Apartment of the Royal Palace of Naples (XVIII century), which are representative of the past applications and technologies of wooden structures in monumental buildings. After an overview of the identified structural configurations, the attention is focused on the roofing structure of the Second Anteroom of the Throne Hall. A 3D FEM model has been set up on the basis of the geometrical and mechanical surveys. The results of the numerical analysis allowed to understand the behavior of each structural element in terms of strength and deformation capabilities, evidencing the weaknesses of the examined wooden structure. Consequently, on the basis of the acquired knowledge, the appropriate restoration intervention has been proposed.

1 INTRODUCTION

The refurbishment and enhancement of the existing building heritage held a very important rule in all the European Countries, where the belief that historical constructions should be preserved and restored, confirming their initial static function by means of appropriate interventions in harmony with the pre-existent construction, is deeply rooted. In the last decades, many research activities concerning the protection and retrofitting of historical buildings have been financially supported by public administrations and a great increment of the number of interventions on existing structures is underlined. This owes the natural ageing of constructional materials, the increasing environmental aggression, the lack of maintenance, the design and execution inaccuracy, the need to provide either the structural upgrading, because of changed boundary conditions, or the post-earthquake repairing. The problem is particularly felt in case of wooden structures, in fact wood is a material used since a very long time in both monumental constructions and historical buildings, particularly for realizing the floor slabs. Due to its nature wood presents several defects (nodes, shakes, shrinkage slits, shape imperfections, fibre deviations, etc.) and structural anomalies (cracking, deformations induced by long duration loads, connection degradation, etc.), moreover it suffers a biological deterioration, closely connected to the external environmental conditions. It is apparent as the analysis of ancient structures made of wood is very cumbersome for all the inherent difficulties to be faced at both material and structural behaviour characterization. Concerning the intervention techniques, evidently, they are based on the substitution of deteriorated existing elements as well as on the addition of new elements for improving strength, stiffness and ductility. The choice of the type of intervention can be made only after an accurate survey of the condition of the standing structures. Therefore the static behaviour of the whole structure and of each structural component, together with the state of conservation of single members and their complete characterization have to be identified. Consequently preliminary structural diagnostic inspections are needed to evidence any damage and decay situations, to find and eliminate their causes, and at the same time to characterize the structure as respect to the mechanical properties, for determining the strength, stiffness and ductility capacities.

In such framework in this paper an overview of the different typologies of beam floor and false ceiling structures representative of the past construction technologies is presented with reference to the roofing structures of the Historical Apartment of the Royal Palace of Naples. In particular the attention is focused on a single study case, which is the roofing structures of the IV Hall of the Historical Apartment, once the Second Anteroom of the Throne Hall. The detailed geometrical and mechanical survey have been illustrated. The evaluation of the deterioration degree has been achieved by means of in depth in situ investigations, leading to the complete structural identification. Once all the necessary knowledge has been acquired, a three-dimensional FEM model has been set-up, with the purpose to evaluate the bearing capacity of each structural component and its contribution to the behavior of the whole structure. The numerical analysis has evidenced the weaknesses of the structure in terms of strength and deformations and its residual capacity, needed for defining the appropriate restoration interventions.

2 WOODEN ROOFING STRUCTURES OF THE HISTORICAL APARTMENT OF THE ROYAL PALACE IN NAPLES

The roofing structures of the Historical Apartment of the Royal Palace of Naples are representative of the structural practice in ancient monumental buildings: they are a composition of two different wooden substructures, such as the beam floor and the vault. The latter was usually erected as false ceiling, at the intrados of which an internal coat of thin canes and a cover of stucco provided the base of the paint.

Three different static configurations have been evidenced: 1) the self-bearing vault, independent from the upper floor, it leaning on the perimeter masonry walls only; 2) the beam floor-vault composed structure, the vault being partially suspended to the upper beam floor; 3) the vault suspended to an ad-hoc wooden structure, independent from the beam floor.

In particular, with reference to eight surveyed halls of the Historical Apartment, the so-called Queen Passage Hall belongs to the first typology, the Diplomatic Hall to the second one and the other Halls belong to the third one.

The study of the Diplomatic Hall was presented in Mazzolani et al. (2004a), and the appropriate consolidation intervention was proposed in Mazzolani et al. (2005).

Hereafter the analysis of one hall, namely the Second Anteroom, corresponding to the third typology, is illustrated.

3 THE STUDY CASE: THE SECOND ANTEROOM

3.1 The structural identification

The Second Anteroom, is about 11.45x9.95m^2 large and it is composed by 3 main substructures (Fig. 1):

1. The beam floor. It is composed by two frames of wooden beams, perpendicular each other. The primary floor beams, which have circular variable cross-section (about 26cm mean diameter) are arranged along the minor span of the floor. They are stiffened by longitudinal inclined struts (about 15cm diameter) and strengthened by two rectangular wooden thick planks (5x30cm^2), placed at the ends of the beam, at both sides. The secondary floor beams have a rectangular cross-section, 15x13cm^2 size. All the elements are connected each other by nails. The floor slab is directly supported by the secondary beam frame and it is composed by planks with semi-circular cross section, about 4cm thick, over which a layer of lapilli and cement lime mortar is casted. The mean thickness of the layer is about 20cm.
2. The supporting structure of the vault. It is composed by 4 beams, with rectangular 36x45cm^2 cross-section, arranged along the major span of the floor, about 40cm below the primary beam intrados. The central beams directly support the vault crown, the lateral beams support a grid of inclined struts and horizontal joists orthogonal to them, realizing all together the bearing structure of the vault. The grid elements have circular cross-sections, 10cm and 6cm mean diameters for the struts and the joists, respectively. The connection be-

tween the vault and the supporting grid is realized by wooden links with a rectangular 4x4cm cross section. All the wood elements are connected by nails.

3. The vault. The structure is composed by a grid of ribs and splines. At the intrados a coat of thin canes, lime and plaster, as an overlay of the wooden structure, and the cover of stucco, as the base of the fresco, were realized. The vault is either suspended to the wooden grid and supported by the perimeter masonry.

All the structural elements of the beam floor, as well as of the vault supporting structure are made of chestnut, whereas the wooden structure of the vault and the links are made of poplar, according to the constructional practice of that time.

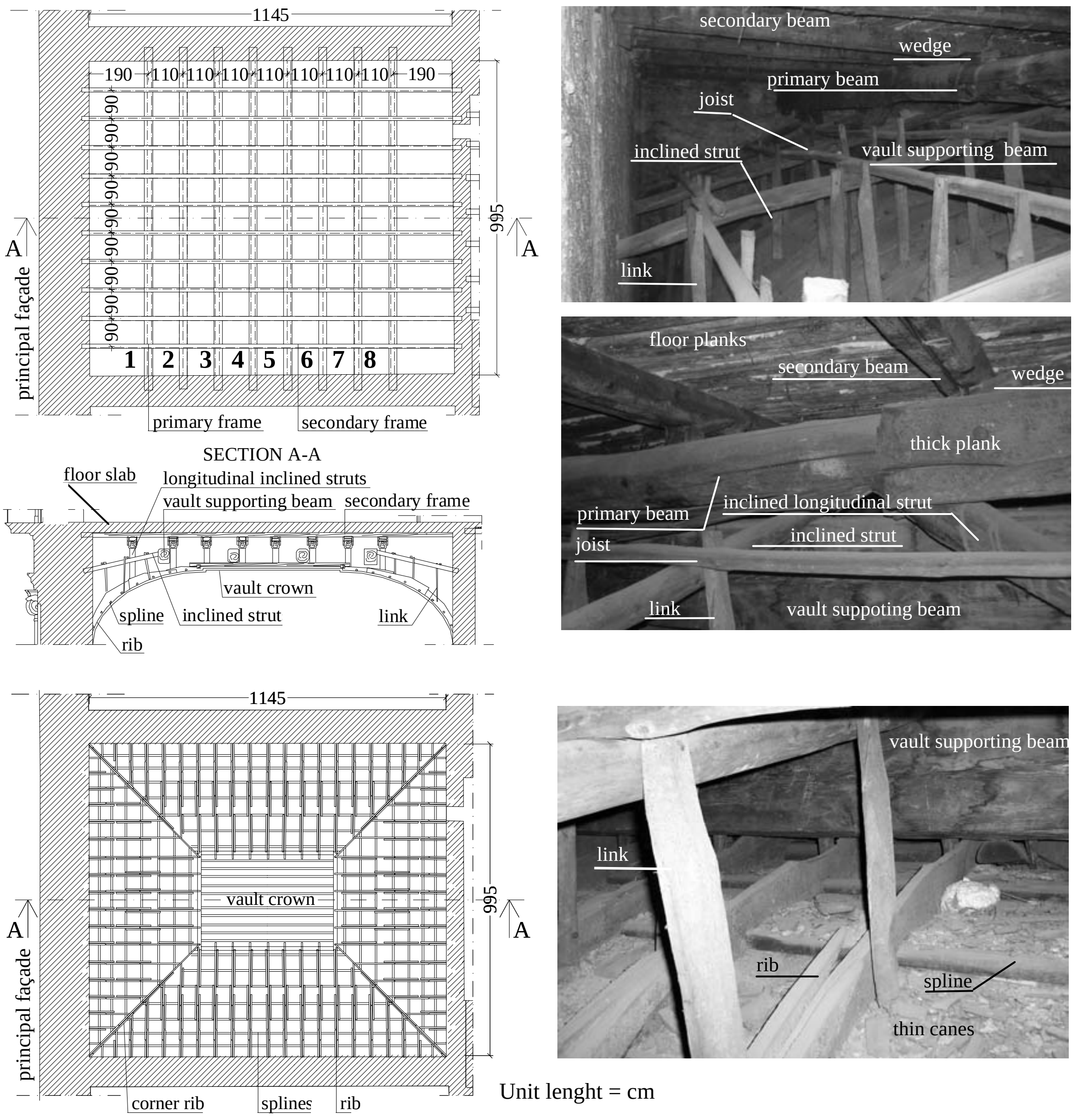

Figure 1 : The second Anteroom roofing structures.

During the survey it was evidenced that one beam, n. 1 in Fig. 1, collapsed for bending overstress and the related struts failed for compression at the joints. Both primary and secondary beams show important deformations, in fact in the Fig. 1 wedges between primary end secondary beams are apparent. All structural elements have not been attacked by fungi or insect.

3.2 The numerical model

A 3D FEM model of the whole structure was set up by means of the program for structural calculation SAP2000 v. 8.23 (Wilson 1998). Due to the high level of variability and irregularity, which is peculiar of the ancient wooden structures, the geometrical model is necessarily affected by some approximations, such as:

- Primary beams are modelled as equally spaced, with equivalent constant circular cross section; secondary beams are simply supported by the primary ones; the internal joints are internal hinges; the restraint conditions at the perimeter masonries are simple supports.
- Ribs and splines are modelled as single elements with an equivalent rectangular cross section, $4\times15cm^2$ and $7\times5cm^2$ respectively. The vault crown elements have a circular cross section, 12cm diameter, spanning 25-30cm.

3.3 The used materials

The current mechanical properties of the wooden beam have been measured by in situ investigations carried out on the roofing structures of the II Hall, namely the Diplomatic Hall, according to the Italian current standard rules (UNI 11118 and UNI 11119 2004), which give procedures and criteria to evaluate the preservation state and the residual strength of historical in situ structural elements (Bertolini et al. 1998). In particular, by means of in depth visual investigations and several ND tests, the identification of the species, the measurement of the moisture content, the survey of any defects and the degradation state due to aging were achieved and the joints effectiveness assessed (Mazzolani et al. 2004a). Consequently the grading of all structural in situ elements was carried out, obtaining the actual mechanical properties of the material given in Table 1 for both new and ancient wood. In addition the residual resistant cross-section of each structural element was measured.

Table 1 : Mechanical properties of wood.

Wood	γ	E_0	E_{90}	ν_0	ν_{90}	G_0	G_{90}
	kN/m^3	$kN/m^2\times10^{-3}$				$kN/m^2\times10^{-3}$	
New wood							
Chestnut	5.8	11380	544	0.37	0.46	750	185
Poplar	3.7	7850	376	0.38	0.47	490	128
Ancient wood							
Chestnut		8000	400			500	137
Poplar		7000	350			450	119

3.4 The loading model

Different load conditions and corresponding material mechanical models, which the structure is supposed to have undergone during its life, have been examined, starting from the erection stages up to the service condition, in order to depict the stress and strain state histories. In particular the material is modeled as new wood at the construction time and as ancient wood at the present time. Therefore the following numerical models have been set up accordingly:

Beam floor:

- Model F1. *Erection of the beam floor (new wood):* the structure consists of the beam floor only, corresponding dead loads are applied;
- Model F2. *Beam floor completion (new wood)*: the structure consists of the beam floor completed by non structural elements; corresponding dead loads are applied;
- Model F3. *Serviceability conditions (new wood)*: both dead and live loads are applied;
- Model F4. *Serviceability conditions (ancient wood)*: the mechanical properties are referred to the ancient wood, accounting for the degradation effects and the lack of the collapsed beam;

Vault:

- Model V1. *Erection of the vault and vault completion (new wood):* the structure consists of the whole vault completed by lathing and stucco;
- Model V2. *Connection vault-supporting structure (new wood)*: the structure consists of the vault and its supporting system; all permanent loads are applied;
- Model V3. *Serviceability conditions (ancient wood)*: the mechanical properties are referred to the ancient wood, taking account of the degradation effects; the wooden links, which con-

nect the vault to the upper supporting structure, were gradually removed in order to asses their effectiveness.

Concerning the applied loads, dead loads consider the contribution of all the completion elements, such as: on the floor, the planks and the lapilli-cement lime mortar (4cm and 20cm thick, respectively), over which the floor rough and the majolica tile are realized (both layers 2 cm thick) and the partition walls; on the vault, the stucco and plaster layer, 7 cm thick.

Live loads on the floor correspond to the destination of use as office.

For both vault and beam floor, vertical seismic motion has been considered according to the Italian seismic codification (D.M. LL.PP. 16/01/1996).

3.5 Evaluation of the stress state

In Fig. 2 the stress state in the floor beams is qualitatively described in terms of Moment (M), Shear (S) and Axial Force (A) diagrams for both primary and secondary beams, at the erection time (Model F3) and at present time (Model 4). The effects of the lack of the primary beam on the secondary beam is evidenced by comparing Models F3 and F4. It is worth noticed that axial forces in secondary beams are equal to zero.

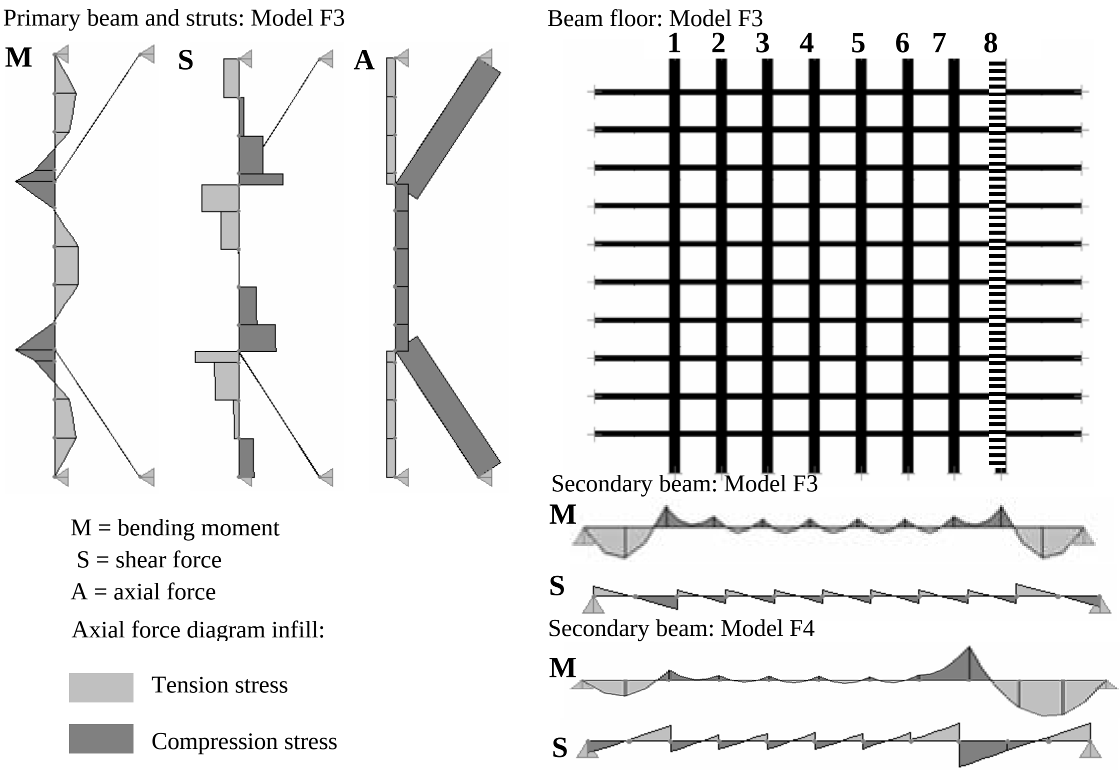

Figure 2 : Internal forces distribution in the floor beams

In Fig. 3 the stress state in the vault elements is depicted, with reference to both the case of self-bearing vault (Model V1) and suspended vault (Model V3). The beneficial effect of the connection to the supporting structure, in particular in the vault crown elements is apparent from the comparison between Model V1 and Model V3 stress diagrams.

According to the German code DIN 1052-1/A1 (1996) and with reference to the design strength given in the Italian Code UNI 11119 (2004) for the relevant stress conditions and wood species, safety checks have evidenced that:

- the primary floor beam n. 7 and the secondary beams, in ancient wood (Model F4), do not accomplish the strength requirements, whereas all the struts of the floor beam stiffening system do not satisfy the stability check;
- the crown elements of the self bearing vault do not accomplish the strength requirements, therefore the vault suspension to the supporting structure appears to be necessary. However

the actual link location is not effective. In fact some links outside the vault crown area work as struts (see detail in Fig. 3b) and the stress state of the links located at the perimeter supports of the vault crown evidences that only links located within the vault crown are needed.

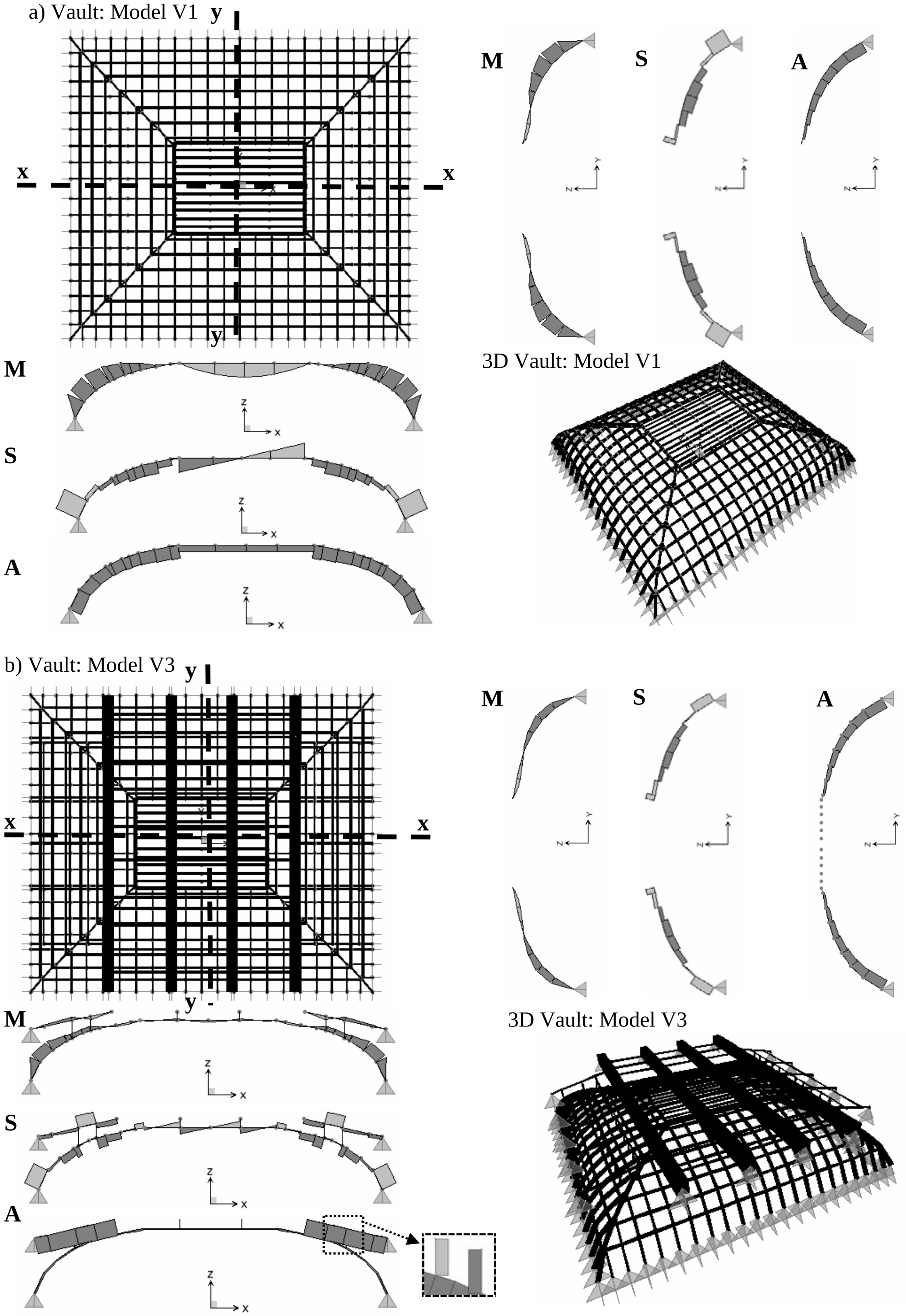

Figure 3 : Internal forces distribution in the vault elements.

3.6 Evaluation of the deflection state

In Fig. 4, with reference to the floor beam n. 2, vertical displacements (u) are represented for each of the examined load situations. In particular the increment of deformation, during the service life of the structure, due to creep, has been taken into account by amplifying the instantaneous deformation (u_{ist}) by using the following expression (DIN 1052-1/A1, 1996):

$$\mathrm{u}_{\mathrm{fin}} = \mathrm{u}_{\mathrm{ist}}(1+\varphi) \quad \text{with} \quad \varphi = (1/\eta_k) - 1 \tag{1}$$

The coefficient η_k is dependent on the moisture content and on the ratio between total (q) and dead (g) loads. For moisture content less than 18%, as it is the case of the study structure, η_k is equal to 0.78, therefore u_{fin} is 1.28 times u_{ist}.

According to DIN 1052-1/A1 (1996), the instantaneous vertical displacement due to the total loads (u_{tot}) should be less then L/300. In particular, for L=10m, u_{tot}=1000/300=3.33cm, which is larger than the maximum calculated value (u_{max}=1.20cm) with reference to the Model F4. These deformations must be added to the permanent deformation of the beam due to the irregularity, as testified by the presence of wedges (Fig. 1). It is important to notice that the deflection has been calculated considering the effectiveness of the struts of the floor beam stiffening system, (at y=±200cm in Fig. 4). On the contrary struts do not satisfy the stability check, therefore, in case the lack of the inclined struts is taken into account, the induced increased deflections are beyond the limit value (about 50cm in the middle span).

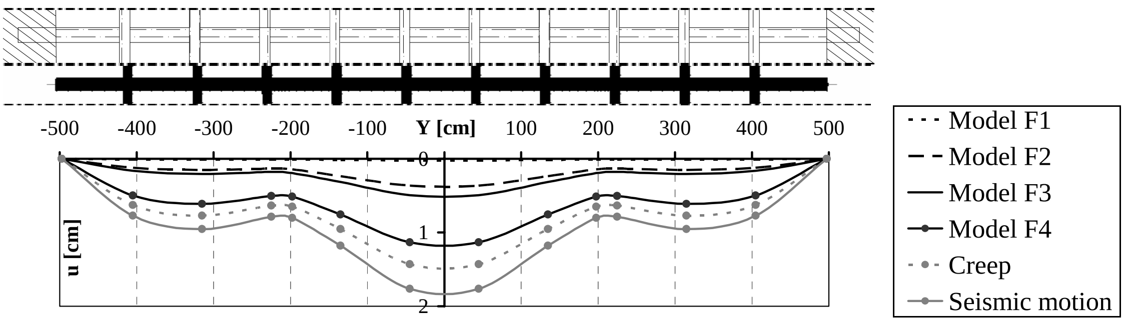

Figure 4 : Vertical displacements of the floor beam n. 7 for each examined structural model.

The deformed shapes of a typical vault section are analyzed for each of the examined situations. The amplification coefficient φ for creep is equal to 1, being the *g/q* ratio equal to 1. Therefore u_{fin} is 2 times u_{ist}.

The vault undergoes a generalized flattening, which consists of a sagging at the centre of the vault, with a maximum displacement larger than 6cm in the self bearing vault configuration (Model V1) and equal to about 1cm in the suspended vault (Model V2).

4 THE PROPOSED RETROFITTING INTERVENTIONS

The following retrofitting interventions are proposed according to the safety requirements of each simple substructure:

1. Substitution of the collapsed primary beam with a new one made of chestnut: the substitution of the collapsed beam induces an important decrease of stress in the adjacent primary beam and in the rafter, thus restoring the admissible stress state.
2. Reinforcement of the unstable longitudinal struts, by means of both horizontal and vertical stiffening steel elements (Mazzolani et al. 2005);
3. Restoration of the strengthening function of the two rectangular wooden thick planks placed at the ends of the beam at both sides (Fig. 1): the connection among the strengthening elements and the beams should be restored by means of bolts, to be located in the holes of the existing nails, suitably tightened. Short wooden plated stocks, which are not able to assure the needed strengthening of the beam at the strut connection, where the shear assumes the maximum value, should be substituted by opportunely longer elements, made of ancient chestnut.

4 Restoration of the efficiency of all the connections. In particular for the vault, the retrofitting intervention consist in restoring the vault-supporting structure connection at the vault crown only, by means of new steel nails.

5 CONCLUSIONS

The knowledge of the construction typologies and technologies used in the past for the realization of wooden beam floors is an important starting point for the structural identification, aimed at the evaluation of the strength and stiffness capability and to the definition of appropriate retrofitting or upgrading interventions. In fact it is necessary to know the static function of each element, how they are connected each other, in order to assess their contribution to the behaviour of the whole structure.

The study of the historical wooden structures is generally made difficult because of few information related to the past construction practice and of the inaccessibility of structures in situ for carrying out in depth surveys or investigations. Therefore corresponding structural models for analysis of the behaviour are affected by any approximations, which have to be made always in the safe side.

An appropriate analysis methodology has been applied with reference to a study case, which is the roof of the Second Anteroom of the Royal Palace of Naples (Italy). It is a composition of a wooden beam floor and a vault. The geometric and mechanical surveys, together with the results of the in situ investigations, have allowed the set up of a three-dimensional FEM models of the structure. The behaviour of the complex wooden structure has been evaluated from the construction up to today, taking into account the variation of the mechanical properties due to the degradation occurred during time. The structural analysis has evidenced the weaknesses of the structure in terms of strength and stability, with reference to some floor beams and inclined struts, respectively, and in terms of deformation, mainly related for the vault to the inefficiency of connections with the supporting structure and for the floor beams to instability of some stiffening struts. Consequently, a simple appropriate restoration intervention has been proposed.

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