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Numerical survey, analysis and assessment of past interventions on historical timber structures: the roof of Valentino Castle

Badania numeryczne, analiza i ocena wcześniejszych interwencji na zabytkowych konstrukcjach drewnianych: dach zamku Valentino

1. INTRODUCTION

1.1. Historical background

The Valentino Castle is located in the centre of the Italian city of Turin, facing the Po riverside. The origin of the building dated back in the 17th century and its the current structure is due to Princess Christine Marie of France (1606–1663), wife of Victor Amadeus I, who dwelt here from 1630 as she wanted a castle following in style the castles built in that period in France. Therefore the architecture of the castle is inspired by the French principle of the pavilion system, with four towers at each angle, and a wide inner court (figs. 1–2).

1.2. The timber roof structure

The timber roof structure of the towers has three-dimensional organization. More precisely, in the transversal direction, it is constituted by four great trusses and two small trussed at the ends. The pitch is strongly inclined. In the longitudinal direction of the timber structure is composed by the ridge, five series of purlins and by three orders of frames overlaid with stiffening functions (figs. 3–4). This structural complex is firmly secured to the covering planks that support the tiles of black stone [1, 2].

Fig. 1. Valentino Castle in Turin nowadays (photo T.M.)

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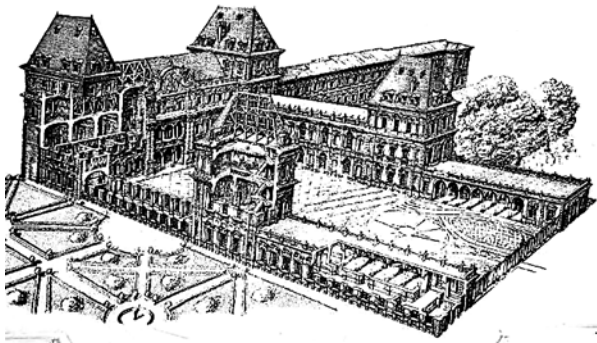
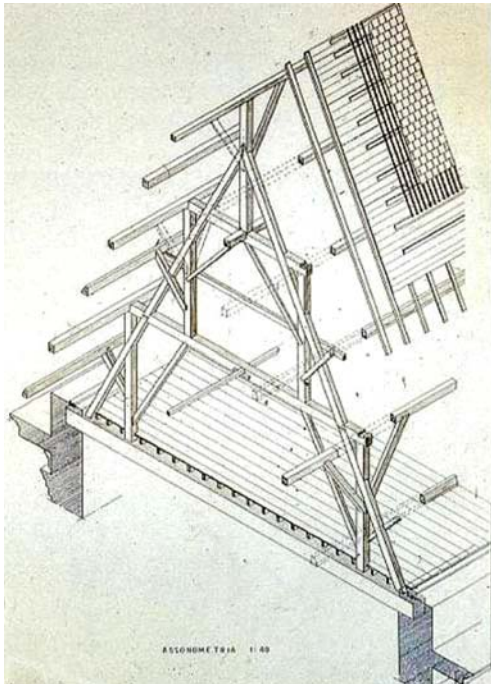Fig. 2. View of the Valentino Castle (drawing F. Corni, 2012)

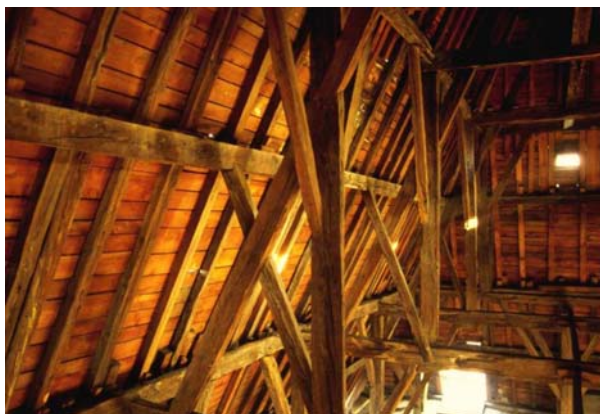Fig. 3. Scheme of a truss and its components (drawing C.B. 1986)

Fig. 4. Inner view of a tower (photo C.B. 1986)

2. MATERIALS AND METHODS

2.1. Past interventions (1980's)

In the late 1980s a conservative action was needed because a scarce maintenance through the years had caused damage which could have compromised the conservation of this timber structure.

During the restoration a particular caution was needed because of the historical relevance of the building as an UNESCO listed site and in order to preserve as much as possible the original carpentry.

2.1.1. Geometrical survey

The geometric survey was carried out carefully in order to understand the structure, the types of connections and each assembly step. There was a particular focus on some assembly marks dating back at the time of the building site which allowed the identification of the original components from those replaced later (figs. 5, 6, 7).

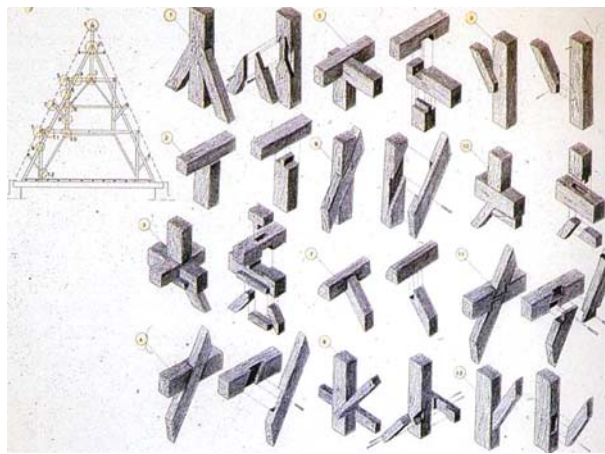Fig. 5. Abacus of timber joints (drawing C.B. 1986)

Fig. 6. Timber joints

Fig. 7. Assembly marks (photo C.B.)

2.1.2. Wood species

The test that were carried out (macroscopic and microscopic) on the main elements of the roof system have provided the following results:
– wooden elements constituting main timber structure (truss, beams, rafters, pillars, etc..) are in Larch (*Larix decidua* Mill.)

– planks put in place in 1960's are in European Spruce (*Picea excelsa* Link) [3].

2.1.3. Dendrochronological analysis

For the dendrochronological analysis samples were collected from the Laboratory Dendrodata of Verona and it was possible to determine the age of the trees from which they derived. Most of the members date back to 1620–1623, historical additions were realized in 1785 and 1960 [4].

2.1.4. In situ assessment

The diagnostic investigation carried out in the 1985 allowed a depth analysis of the state of conservation of wood material and different stage of bio deterioration were identified. There were structural disconnections due both to deformation and to shrinkage. It has been assessed biotic decay due fungi and insects that occurs with different levels depending on the wood species.

Figs. 8–10. Biotic decay due to due fungi and insects (photo C.B. 1986)

Moreover diffuse infiltrations of rainwater speeded up the attacks in some specific locations in the structure and in particular in the beams' head area. In the wood of some components restrained in the external wall, a high level of humidity was detected as well as a diffuse state of decay (figs. 8, 9, 10).

2.1.4. Structural investigation

The structural investigation carried out in 1980's was aimed at verifying the load-bearing capability and general deformation of the truss structure. The analysis was performed using a series of mathematic models (bi and tri-dimensional) capable of understanding the mixed reticular and frame structure, allocating to the joints and rods constraint and rigidity levels in accordance with the level of degradation observed [5].

In the structural analysis phase several hypotheses of schematization were considered in order to verify the static strength and deformation of the whole system which can be considered as a hybrid between a truss and frame. Assuming both vertical and horizontal loads, maximum stresses were identified to obtain an assessment of the safety of the structure.

The investigation was performed in 4 steps:
– analysis of the truss structure in the plane and assuming non bio deteriorated members
– (σ max = + 190–120 daN/cm^2);
– analysis of the truss structure in the plane with bio deteriorated elements
– (σ max = +260–270 daN/cm^2);
– analysis of the tri-dimensional structure (with some bio deteriorated members)
– (σ max = +180–190 daN/cm^2);

tri-dimensional analysis of the structure formed by listels and planks together with some members of the main frame (shell behaviour) (σ max = $\pm$83 daN/cm^2);

In the case of the spatial model 500 elements and 1040 joints were taken into account with a total amount of 1818 equations. That investigation showed that the structural components were not verified for the loads considered. Later the behaviour of the roof planking as a "shell" was assumed, allowing the structural verification (fig. 11, 12, 13).
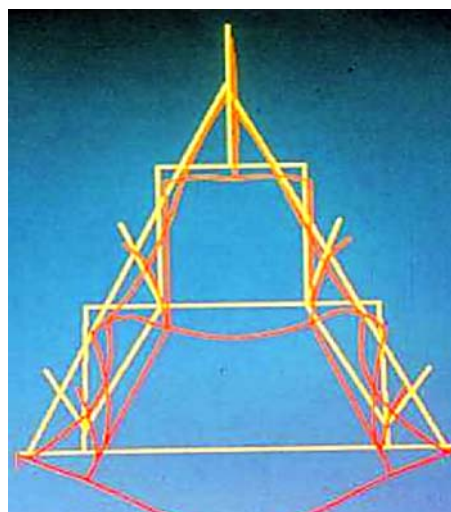
Figs. 11. Structural analysis: bi-dimensional model, deformations and stresses, tri-dimensional model (1986)
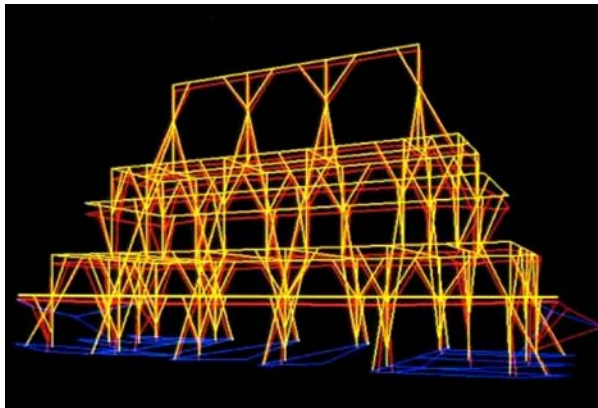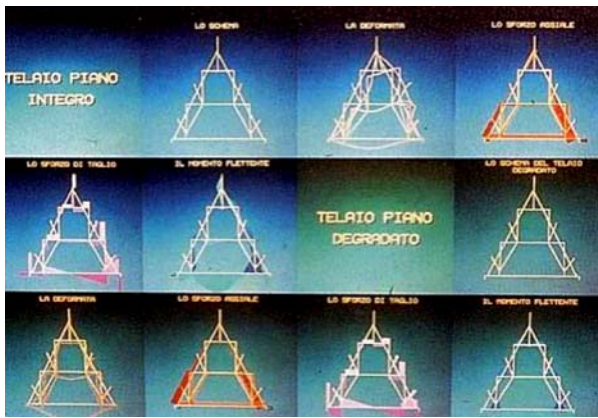
Figs. 12–13. Structural analysis: bi-dimensional model, deformations and stresses, tri-dimensional model (1986)

Figs. 15. Reinforcement intervention on tie beams' head-pieces using reinforced glue-laminated timber. Realization (1986)

3.1. Reinforcement interventions on tie beams' head-pieces using reinforced glue-laminated timber

A phase of reconstruction of each damaged area of the tie-beam heads was provided using glue-laminated timber reinforced by fiber-glass rods. A good level of ventilation was guaranteed through the insertion of neoprene layer in between the wall and the glulam to get to a good durability of the intervention (figs. 14–15).
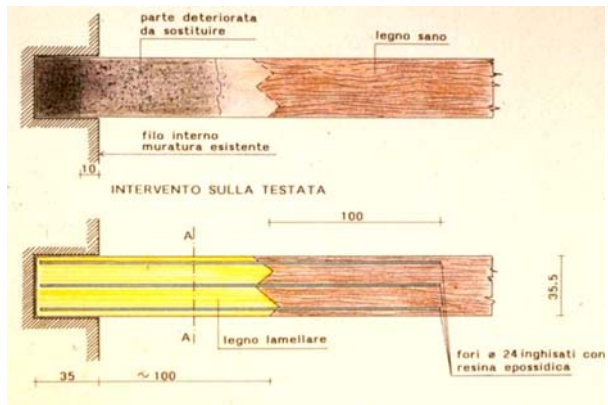
3. REINFORCEMENT INTERVENTIONS OF 1980'S

In the late 1980s a conservative intervention, mainly consisting as a reinforcement of the structural system, was carried out (coordinator Prof. Clara Bertolini, Politecnico di Torino).

In the organization of the building site two different phases have taken place, involving first the exterior of the tower and then all the interventions inside the roof.

Several reinforcement interventions were carried out. The main ones are described hereafter.

3.2. Reinforcement intervention of the masonry wall supporting the timber carpentry of the roof

Another phase of intervention involved the reinforcement of the area of support of the timber joists and planking at the top of the masonry walls (figs. 16–17).

Figs. 14. Reinforcement intervention on tie beams' head-pieces using reinforced glue-laminated timber. Realization (1986)

Figs. 16–17. Area of the masonry wall supporting the timber carpentry before (Fig. 16) and after (Fig. 17) the intervention (1986)

3.3. Connection between the planks and the tie-beams with steel pins

This intervention was conducted to connect the covering planks and to reinforce the "shell" behaviour (figs. 18–19).
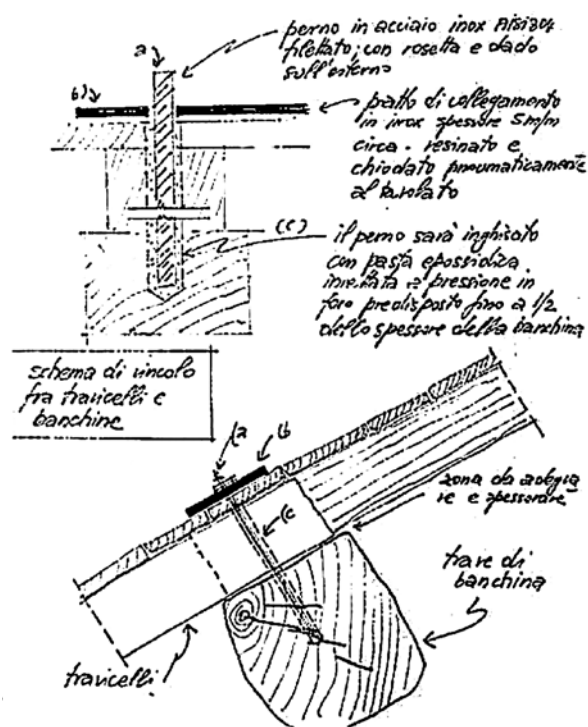
Figs. 18–19. Connection between rafters and girds via steel pins to get the continuity of roof pitch

4. LASER SURVEING

The terrestrial laser scanning (TLS) has becoming an accomplished method for data acquisition in the close range domain, especially in recording and mod-elling buildings. It is clearly recognized that the high quality of accuracy, the productivity and the suppleness of range are very effective in constructions and cultural heritage analyses and modelling.

Despite a substantial literature and a wide body of experiences are available for TLS applications on many different kind of architectural assets, in the cases of the timber roof or dome structures some procedural issues arise and some considerations should be given to some factors.

One of the main purposes for adopting 3D laser technologies is surely the advantage to record large amount of high resolution information in order to model very articulated and complex timber structures [6]. Other times the detection of shape anomalies such as subsidence or other pathologies are mandatory for supporting structural assessment [7, 8].

Nevertheless, historical timber roof structures are rough, dusty, and dark, so as the attics are half light. This last condition would be unfeasible for photogrammetric method that is often used together with TLS, but fortu-nately laser sensor is unaffected from semi darkness. On the contrary some tests have proved that the recorded laser intensity values show somewhat higher values for night-time measurements [9]. Accordingly to that, the point clouds acquired in the roof of the tower of the Valentino castle have been perfectly aligned thanks to the high accuracy in the target detection. The same study shows that different species of wood and differ-ent conditions of wetness have no significant effect on the range accuracy, while it confirms that dark colours have a great influence. In spite of this negative factor, the nearly short distances made the survey fully satisfying.
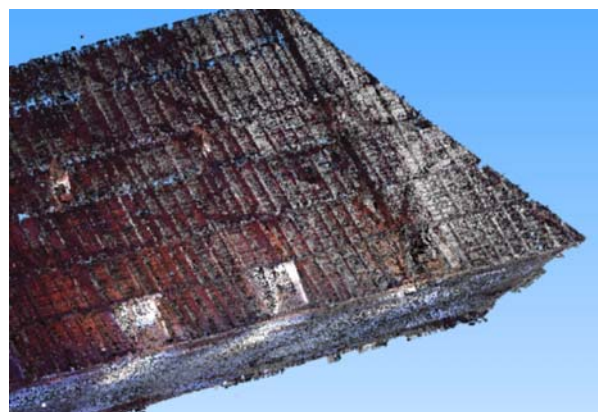

Fig. 20. TLS point clouds adjusted in the same coordinate system

The acquisition phase has been featured by strength-ened procedures:
- all measurements have been processed in a unique, local coordinate system using a reference network of two topographical vertexes; these points have been the reference for measuring all the Control Points (CP) coordinates, through the positioning of targets.
- the LIDAR survey has been realized with the ter-restrial Laser Scanner Focus^{3D} – CAM 2. Six scans

have been executed on ground level of the attic; the scan positions have been chosen in order to optimize the complete data recording of the complex structure, placing them one for each space between trusses (figs. 20–21).

– the obtained clouds have been post-processed using the FARO software SCENE, for adjusting them in the same reference system, through reference targets, reaching a precision of about 1 cm.

Fig. 21. The Scanner Focus³ᴰ acquiring phase

4.1. The model optimization and profiles extraction

It is important to highlight that a point cloud is a 3D model wherein there is always a void between adjacent points and therefore there is no topological information, for this reason it is highly advisable to generate a 3D surface model (mesh) able to create afterwards some textured continuous models or section profiles. The model has been meshed using the software 3D Reshaper (Technodigit).

The mesh processing has been made in two steps: in a first step a rough mesh with regular triangles was created, in a second step a deviation error was entered in order to refine the mesh.

Before the mesh generation, a great attention has been paid to the points cloud segmentation, which is an essential phase enabling the identification of structure elements and joints.

Many editing procedures of the models, included the profiles extraction, is highly automated. As a consequence the dimensions of elements sections are easily achievable, while the beam axis scheme has been generated almost manually (figs. 22–25) [10].

5. NUMERICAL SIMULATIONS

The information acquired through the laser scanning survey can serve to obtain a detailed structural model. The conversion is not straightforward, since the cloud of acquired points can be interpolated with geometrical entities of different dimensionality (i.e. lines, surfaces and volumes), and with different resolution as we increase the discretization. In the present case, the natural choice is to obtain a wireframe model composed of lines, which will correspond to beam elements finite element discretization.
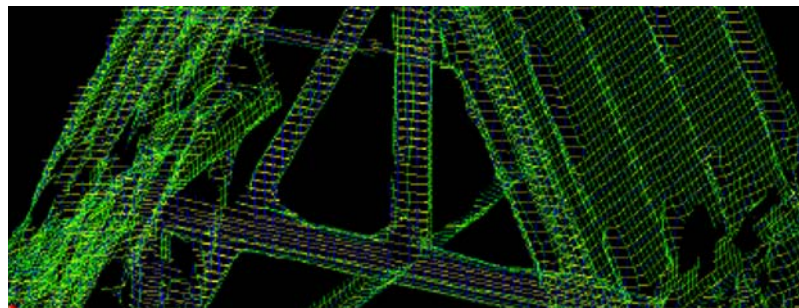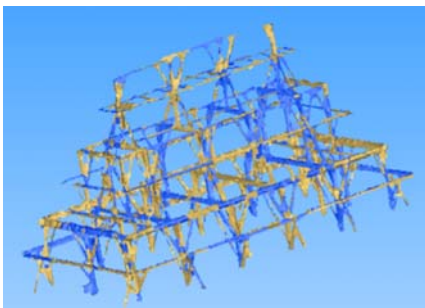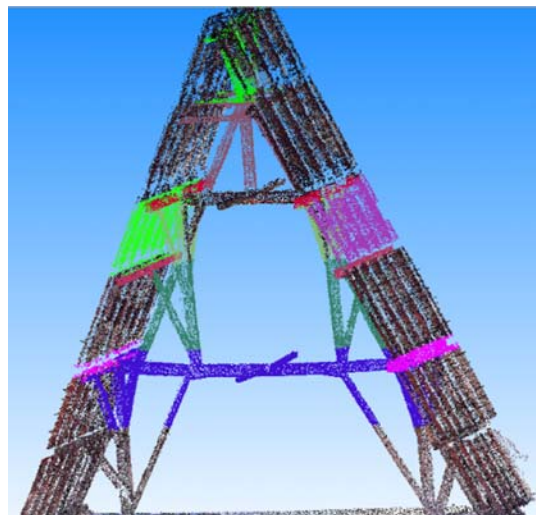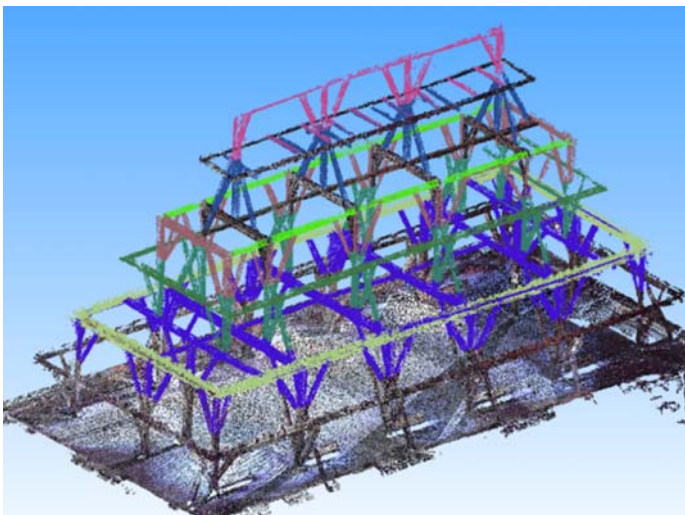
Figs. 22–25. (above) Whole roof structure segmentation, and the selection of a portion to be submitted to further analyses. (below) The complete mesh surface of roof structure and a zoom on extracted section profiles
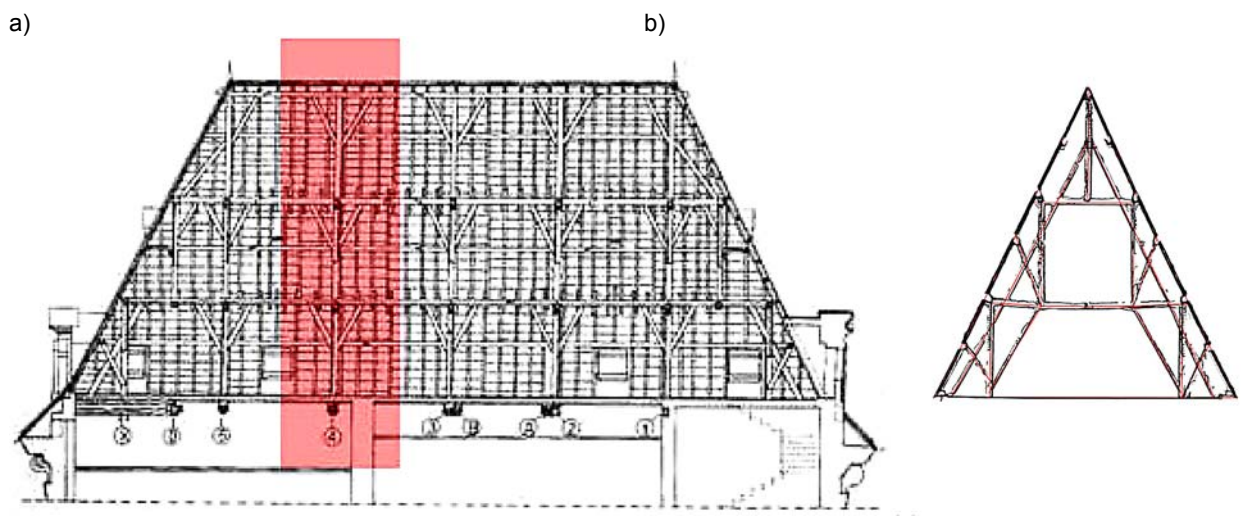
Fig. 26. Section of the roof structure and (in red) truss under consideration (a), beam axis line geometry (in red) obtained from the laser acquire points cloud (b)

Figure 26a shows the truss that was chosen for the analysis, while in figure 26b it is possible to appreciate the relation between the points of the cloud and the idealized beam axis of each element.

(a)

(b)

Fig. 27. Scheme of uniform section elements (reported in table 1) (a), Section of the ribbed planking (b)

Figure 27a represents the scheme of the different structural element sections. In figure 27b, the scheme of the ribbed planking is shown. In the present calculation, the ribbed planking has been accounted for by means of an equivalent flat shell of the same stiffness, with equivalent thickness equal to 7 cm. Table 1 lists the sections adopted in the model for the different structural elements (the same section is assigned to structural elements with the same color).
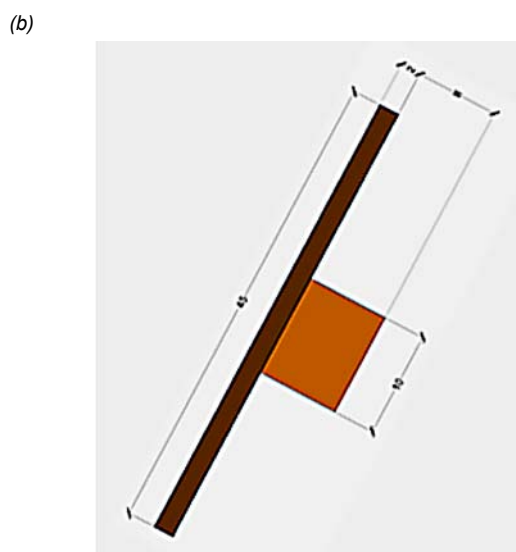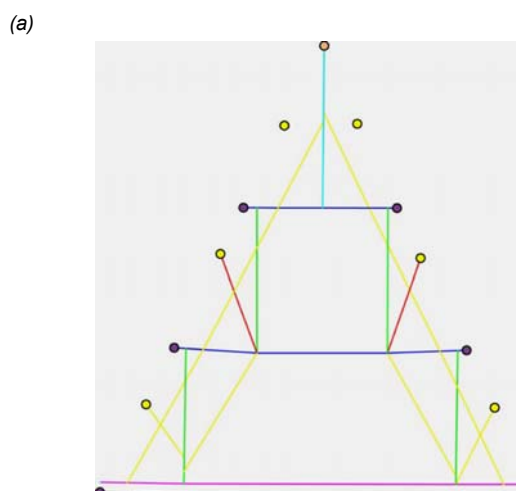
Table 1. Sections adopted in the model for the different structural elements (the same section is assigned to structural elements with the same color)

Structural element	Base [m]	Height [m]	Young modulus [Pa]	Tangential modulus [Pa]	material
Green	0.20	0.14	8e+9	4e+9	Larch
Magenta	0.25	0.35	8e+9	4e+9	Larch
Blue	0.14	0.20	8e+9	4e+9	Larch
Yellow	0.14	0.14	8e+9	4e+9	Larch
Violet	0.18	0.18	8e+9	4e+9	Larch
Red	0.12	0.08	8e+9	4e+9	Larch
Orange	0.10	0.10	8e+9	4e+9	Larch

In addition to the dead load, due to the wood structure and to the black stone tiles (about 1750 Pa), the live load can be provided, in principle, by the snow and the wind pressure. The Italian standards, adopted for the calculation, states that the snow load can be disregarded for the present structure, since the slope of the roof is equal or higher than 60 degree. On the other hand, the wind thrust, must be calculated according to the location and topographic configuration of the structure, and must be arranged according to three different load configurations. The wind pressure ranges between 674 Pa (compression) and 1123 Pa (depression).
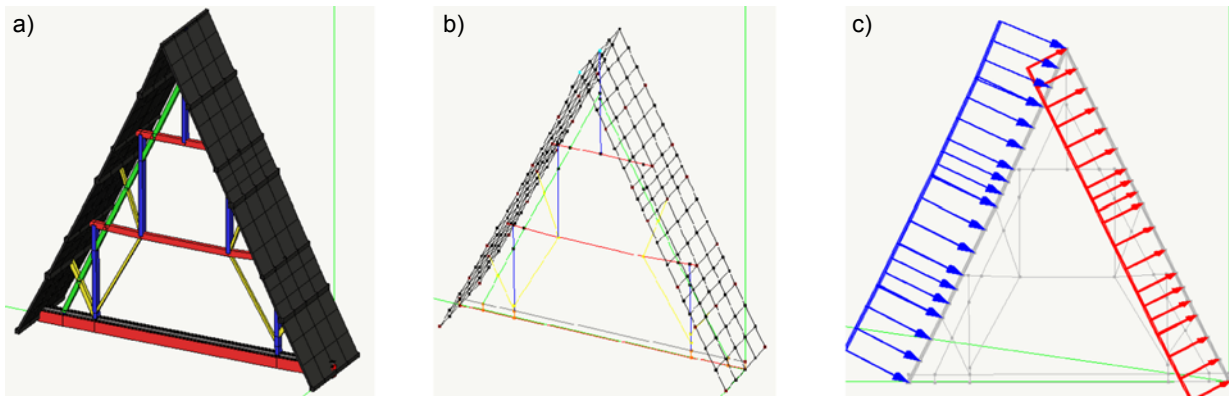
Fig. 28. Solid model of the structural subsystem (a), wireframe structure (b), wind load (c)
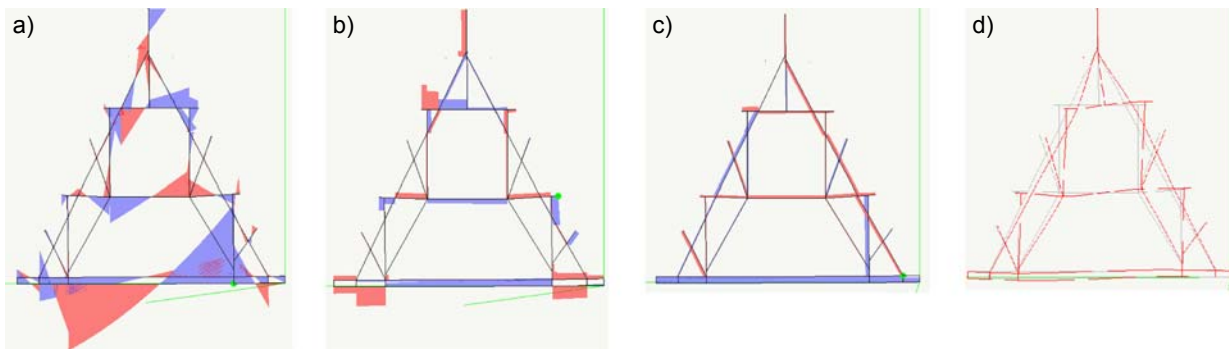

Fig. 29. Bending moment (a), shear force (b), axial force (c), and elastic deformed shape (d)

In order to account for different efficiency of the structural connection, two limit cases have been considered. The first model assumes that the planking system has no stiffness, while the second model account for the planking, and the three-dimensional constrain is modelled with proper periodic boundary conditions. The commercial finite element code Nòlian® (Softing srl) has been used to perform the calculation.

5.1. Truss frame with collaborating planking

In this structural model the effect of the stiffening of the planking structure is directly accounted for, by means of an equivalent flat shell. The truss structure is assumed as periodical, with a constant span between each frame. This is simulated providing the appropriate periodic boundary condition to the shells. Figure 28a shows the solid model of the structural subsystem, while figure 28b shows the wireframe structure. Figure 28c shows one of the load combinations for the wind action.

At this stage of the study, we performed a linear static analysis, and the dynamic action of the wind was accounted for by means of equivalent static actions as allowed by the Italian standards.

The envelope of the beam axial or shear forces and bending moment, together with the elastic deformed shape, are shown in figure 29.

The structure reacts to the action of wind without excessive deformation and without particular stress concentration in any structural element.

In order to better appreciate the levels of stress in the structure, the principal tensile and compression stress contour plot can be drawn. If the contour plot is limited to the planking elements (fig. 30c, d), it emerges that the planking in the lower position of the roof is the most beared.

5.2. Truss frame with non-collaborating planking

In the second model, the constraining effect of planking has been disregarded. In this case, only the
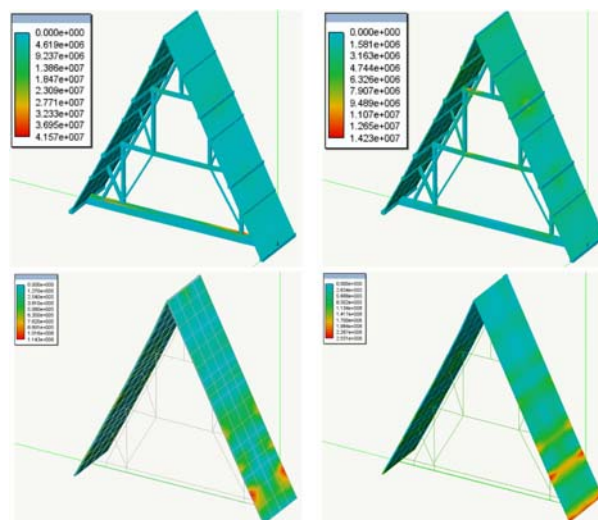

Fig. 30. Principal stress contours in the whole structure: tension (a) and compression (b); Principal stress contours in the planking: tension (c) and compression (d)

truss frame is expected to carry the load (fig. 31a), and the wind and non-structural dead load can be applied to the structure by means of specialized fictitious non-bearing load elements (fig. 31b).
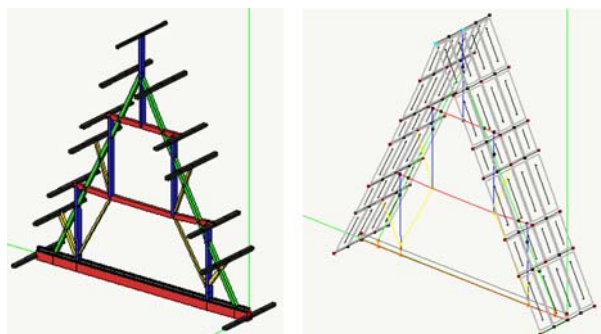

Fig. 31. Load bearing frame truss (a), specialized fictitious non-bearing load elements (b)

The results of the linear analysis are reported in figure 32. It is evident that, is the constraining of the planking is disregarded, overestimated bending moment and consequent deformations are obtained in several structural elements (especially in the median-upper part of the roof, where the previous analysis provided low-intermediate levels of stress). As a consequence, the calculated levels of stress are not admissible for timber.
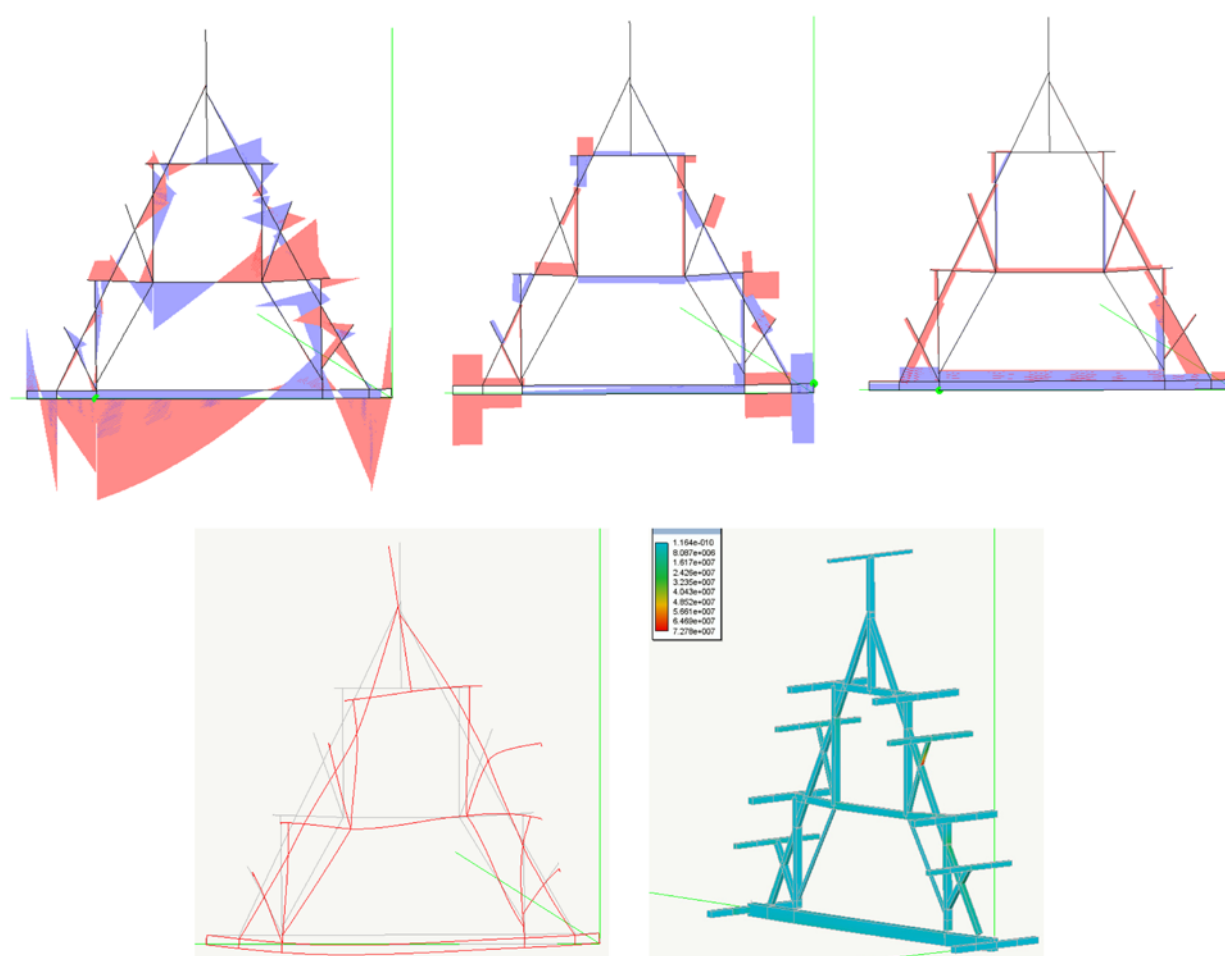
6. RESULTS AND DISCUSSION

From the analyses, it is clear that a proper model must represent precisely the structure and must include the constraining effect of the planking system. The calculated maximum stress (equal to 8 MPa), obtained from the first model, are lower than the valued obtained by previous analyses (1986), and lower than the admissible value for the present timber (10 MPa, being 30–50 the estimated failure stress).

It is expected that taking into account the complete three-dimensional structure, the effect of improved constraining of the vessel shaped planking could provide even more optimistic judgments for the structural assessment.

During 2015 on-site inspections to the structures (after almost thirty years from the reinforcement interventions) allowed an assessment of the state of conservation of both the structures and the reinforcement interventions.

The methodology adopted during the on-site inspection for the grading according to the resistance, are the one foreseen by the standard UNI 11119 (Cultural Heritage – Wooden Artefacts – Load-bearing structures – On site inspections for the diagnosis of timber members).

From this retrofitting it is confirmed that performed interventions, on a very accurate diagnostic base, allow

Fig. 32. Bending moment (a), shear force (b), Axial force (c), elastic deformed shape (d), principal compression contours (e)

on one side punctual interventions respectful of the original structure, on the other side the durability of the interventions.

7. CONCLUSIONS

In conclusion, the current abundance of technical and technological solutions of reinforcement interventions on timber structures requires ex-post evaluations to assess the effectiveness of interventions on historical structures.

The positive assessment of the new accurate structural analysis improves and optimizes the results obtained at the time of the intervention carried out 30 years ago. This confirms the durability of an intervention performed with minimally invasive techniques due to the tri-dimensional behaviour of the structure. Furthermore, the present research allows to start a localised monitoring of this important timber roof structures dating back to four centuries ago (fig. 33).

This experience highlights that an interdisciplinary approach is very profitable in this kind of studies. There is not only a comparison and integration of results, the

Fig. 33. The timber roof structure of the noth-east tower nowadays (photo C.B.)

application has the role to evaluate the opportunity to exploit the interaction of methods and instruments to manage architectural heritage in an interdisciplinary perspective. These approaches are able to save resources while improving the assessment, in particular considering the different specialized investigations that are needed in any knowledge phase.

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Abstract

The paper analyzes a series of reinforcement interventions performed on the historical timber roof structure of the Valentino Castle in Torino (Italy) some thirty years ago. It is not very common to be able to assess the durability of interventions without relying to accelerated ageing test. In this case a real structure is considered, which was consolidated by one of the authors (C. Bertolini) in the past.

Recently the laser scanning survey has strengthened a relevant interest in sectors as monitoring and static assessment of building structures. The high detailed models which is possible to reach, and the chance to foresee the accuracy and the resolution of surface models, make them particularly adaptable for studies concerning conservation and maintenance of cultural heritage. The laser survey models is compared with the documentation of the intervention fulfilled three decades ago, in order to evidence the general safety level of whole complex. The fully three-dimensional geometrical information is input in the finite element code, and a structural model is presented which is able to account for the positive contribution of the roof planking connected above the main truss frame. Both the original planking and the reinforcing planking, put in place some thirty years ago, have been accounted for, considering different degrees of the connection stiffness.

The paper, starting from the original design, presents an assessment of the durability of the adopted techniques according to the present situation. The effectiveness of the past interventions is proved by nowadays survey, NDT investigations and numerical simulations.

Streszczenie

Artykuł analizuje serię wcześniejszych interwencji wzmacniających na zabytkowej konstrukcji drewnianej dachu w zamku Valentino w Turynie (Włochy) przeprowadzonych około 30 lat temu. Rzadko trafia się możliwość oceny trwałości zastosowanej interwencji bez konieczności polegania na teście przyspieszonego starzenia. W tym przypadku badano prawdziwą konstrukcję, której wzmocnienia dokonała w przeszłości jedna z autorek (C. Bertolini). Rozwój metod badawczych wykorzystujących skaner laserowy wpłynął ostatnio na wzrost zainteresowania takimi obszarami, jak monitorowanie i ocena statyczna konstrukcji budynku. Możliwość uzyskania bardzo szczegółowych modeli oraz możliwość prognozowania dokładności i rozdzielczości modeli powierzchniowych sprawiają, że metoda ta może z powodzeniem być wykorzystywana w badaniach dotyczących konserwacji i zachowania obiektów zabytkowych. Model obiektu uzyskany w wyniku skanu laserowego został porównany z dokumentacją interwencji przeprowadzonej trzydzieści lat wcześniej, w celu sprawdzenia ogólnego bezpieczeństwa całego obiektu. Trójwymiarowe dane geometryczne zostały wprowadzone do kodu elementów skończonych w celu wygenerowania modelu konstrukcji. Model ten uwzględniał pozytywne oddziaływanie deskowania dachu połączonego z konstrukcją wiązarów. Zarówno oryginalne deskowanie, jak też deskowanie wzmacniające, zastosowane ok. 30 lat temu, zostały uwzględnione w analizie, z odniesieniem do różnego stopnia sztywności połączeń.

Artykuł, wychodząc od oryginalnego projektu, prezentuje ocenę trwałości zastosowanych rozwiązań w świetle sytuacji dzisiejszej. Skuteczność podjętych w przeszłości interwencji została potwierdzona przez badania przeprowadzone obecnie, analizy metodami nieniszczącymi oraz symulacje numeryczne.