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Diagnosis of Damage in a Wooden Vaulted Structure

Estimation des dommages dans une construction voûtée en bois

Schadensbeurteilung bei einer hölzernen Schalenkonstruktion

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SUMMARY

The methodology adopted in the diagnostic phase to understand the causes of the static anomalies in the wooden vaulted roof of the 'Palazzo della Loggia' in Brescia is illustrated. The paper underlines the usefulness for a correct diagnostic engineering interpretation of a mutual exchange of information between numerical and experimental findings. In particular, the realistic description of the constitutive law of the structural elements - needed in the numerical analysis - has been based upon particular experimental tests.

RÉSUMÉ

La méthode appliquée dans la phase d'estimation des causes des défauts statiques dans le plafond voûté en bois du 'Palazzo della Loggia' à Brescia est présentée. L'article montre l'utilité d'une estimation correcte et de l'interprétation du spécialiste pour un échange mutuel d'informations entre les résultats numériques et expérimentaux. En particulier, une description réaliste des lois constitutives des éléments structuraux qui sont nécessaires pour l'analyse numérique, a été réalisée sur la base d'essais expérimentaux particuliers.

ZUSAMMENFASSUNG

Die zum Ergründen der statischen Unregelmässigkeiten der hölzernen Gewölbeabdeckung im 'Palazzo della Loggia' in Brescia angewandte Methodologie wird vorgestellt. Der Artikel beschreibt die Notwendigkeit eines gegenseitigen Austausches der Werte aus numerischen und experimentellen Analysen, damit eine ingenieurtechnisch richtige Diagnose ermöglicht wird. Insbesondere die realistische Beschreibung der Werkstoffbeziehungen der tragenden Elemente - wie sie für die numerische Berechnung erforderlich sind - basiert auf eigens durchgeführten experimentellen Untersuchungen.

1. INTRODUCTION

Computer modelling provides a great tool in understanding the real behaviour of structures, either new or ancient. Neverthless, in particular for problems concerning old structures, the use of numerical methods are often unapplicable because of the lack of knowledge in the actual values of mechanical parameters. In situ tests or monitoring devices not allwais can be reliable in giving the required quantitative informations of the data usefull for an accurate analysis. In addition to these reasons, the structure under cosideration may present structural typology quite similar to many structures of the same historical period. In this respect the metodology for an adeguate understanding of the statical behaviour assumes a particular meaning, being the structure itself not only an historical monument, but also a prototype, the study of which may be usefull in many auther cases. With these premises the complete and correct metodology for the diagnostic interpretation needs a carefull and mutual exchange of informations between numerical and experimental findings.

The present work illustrates this kind of metodology used to understand the excessive deflection of the wooden vaulted roof of the Palazzo della Loggia in the city of Brescia. After a primal phase of inspection, monitoring and in situ non destructive tests able to give informations about the actual state of the materials and structure [1],[2], it has been necessary to make some ingegneristic interpretations, and to verity them or to search a different response by means of numerical models and experimental tests. In particular it was necessary to understand the efficency of the joints of the principal truss arches of the vault. Some prototypes of wooden joints have been constructed in the laboratory of the Departement of Civil Engineering to calibrate in a relistic way the parameters necessary to estimate the stiffnes of the connections for the numerical non-elastic analysis. As a matter of fact the - actual joints of the truss wooden arches are made by different series of - bolts. The efficency of which, as well as the behaviour of the connections arequite difficult to know beacause of the differential deformations among timbers, steel plates and bolts. The paper want to underline the importance of this metodology by describing the real structure and its damages, the diagnostic interpretation, the trials, by means of numericals modelling, to - adjust or confirm the ingegneristic hipotesys following the findings of the tests. Being the work still in progres, the present paper reports toughether with the numerical approach just a first part of the experimental research, which has provided nevertheless very significant results used in the numerical non-linear analysis of the structural behaviour.

2. THE WOODEN VAULTED ROOF OF THE LOGGIA PALACE

The figure 1 shows the principal facade (XVI th century) of the building. the actual roof is made by a vaulted wooden structure of important dimension - (fig.2), the hight of which reaches a maximum of 25 meters with a planar dimension shown in figure 3. The structural architecture of the vault consist of principal truss wooden arches and simple secondary arches (arch a and b); both are connected at the top by a truss made wooden beam (fig.4).

The semiarches of the short side of the roof end on the diagonal arches, to which are connected (fig.5). The principal arches, made as the entire structure, of hoak wood, are trussed structure, as shown in figure 4, made of two longitudinal curved beams a1 and a2 (the intradoss and the extradoss of the arch) connected by diagonal and straigth elements . The two curved beams a1 and a2 converge to forme a unique monolitical element at the base, which ends at two different levels.

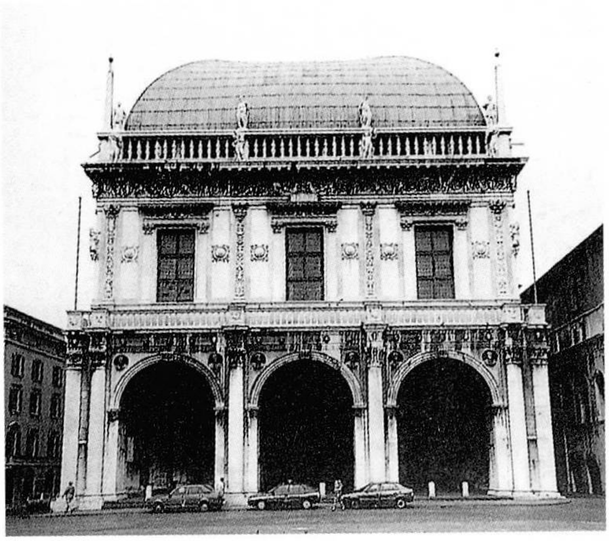


Fig.1 The main view of the Loggia

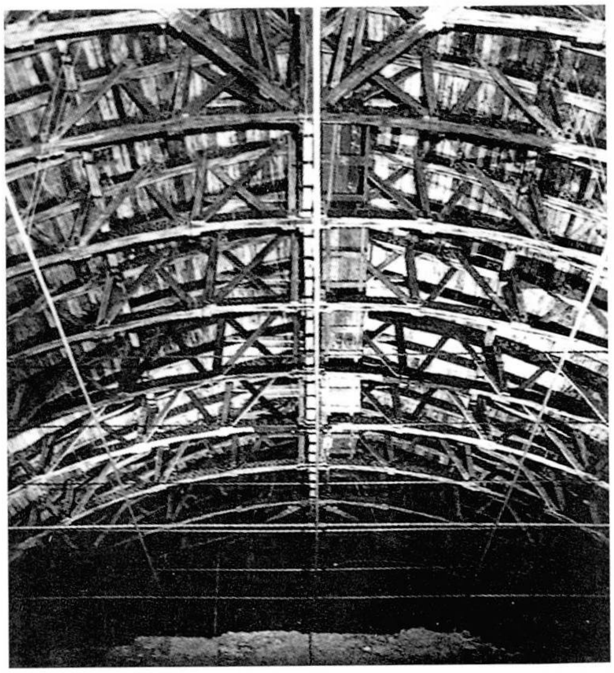


Fig.2 The wooden vaulted roof

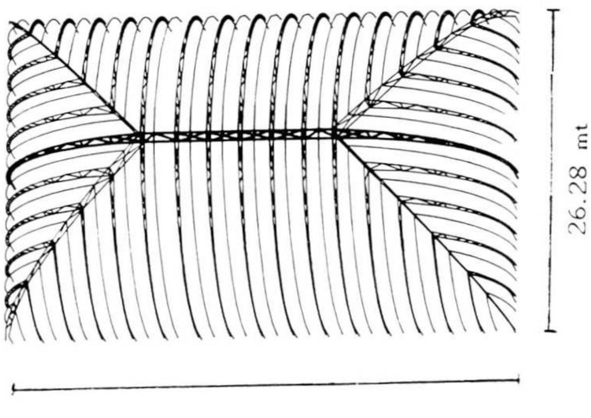


Fig.3 Planar drawing of the roof

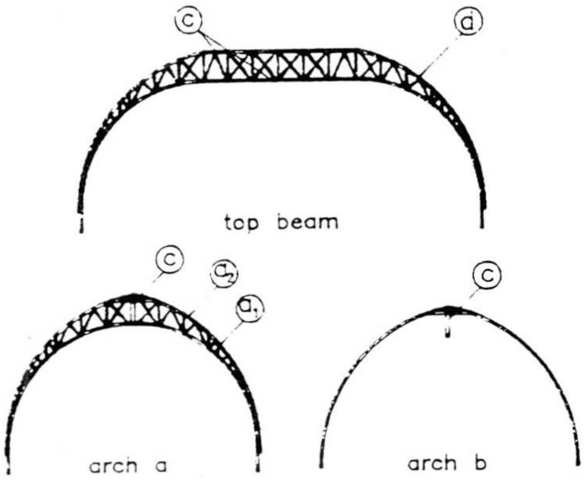


Fig.4 the principal structural element of the roof

The two curved beams a1 and a2 of the principal arches are made by the union of three lumbers of section 10x30 cm and length of about 2 meters, as shown in figure 6. The monolitical behaviour is provided by numerous steelplates and bolts, inserted probably in non calibrated holes. Originaly the transversal continuity of the element was assured by the friction of the lumbers, achieved by the tightening of the bolts. Essentially because of deformations of the holes and the time dipendent behaviour of the wood the bolts lost their tightening. As consequence, slips and mutual displacements either among the lumbers and among the elements of the truss joints has been permitted. As a - matter of fact signs of successive tightening of the bolts are visible.

Fig.5 The view of a diagonal arch

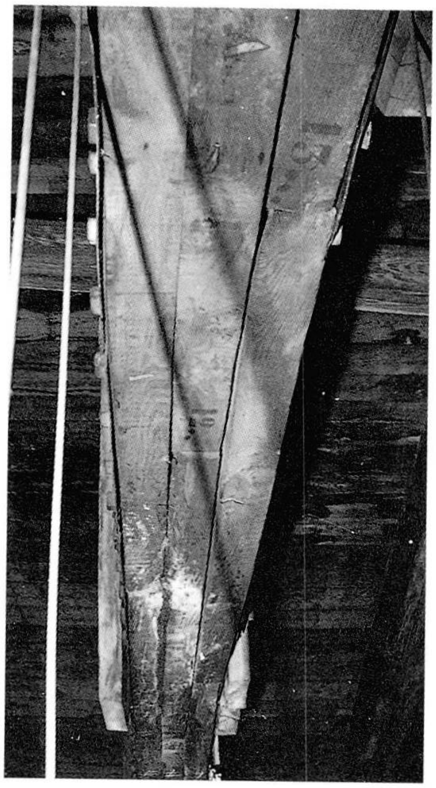


Fig.6 The lumbers and the steel-plates which form the extradoss and the the intradoss of the arches

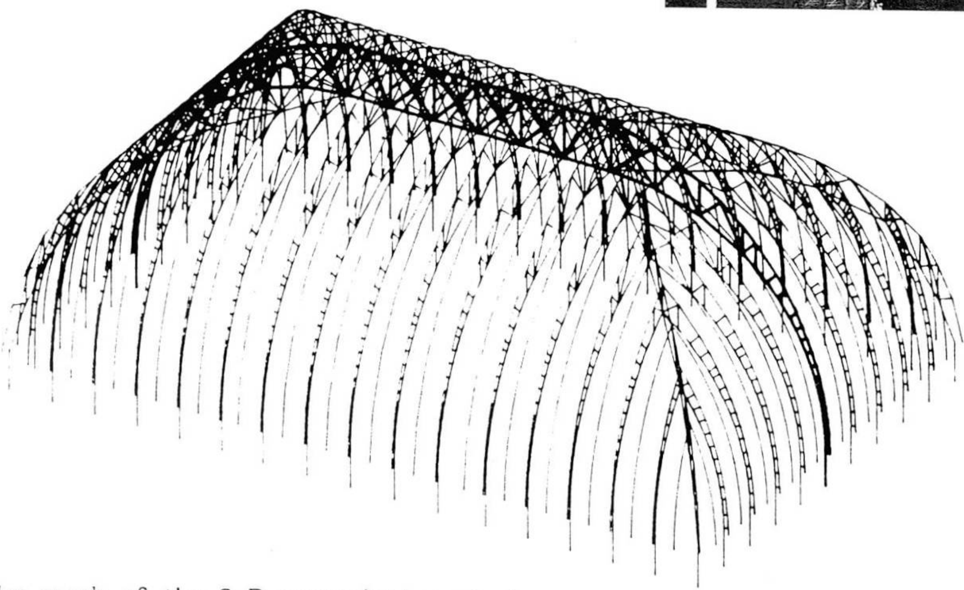


Fig.7 The mesh of the 3-D numerical analysis

3. SURVEYS AND STATIC INTERPRETATION OF THE DAMAGES

The structural damage of the vault is evidenced essentially by an important deflection of the longitudinal top beam and the connected arches. The diagnostic phase has required, as in most cases, the following different approaches:

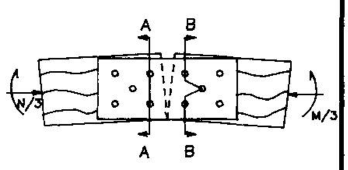
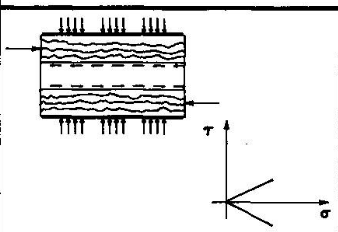
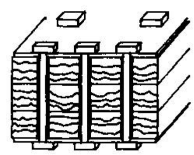
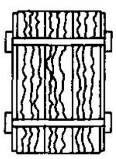
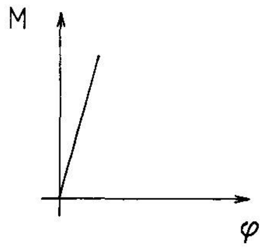
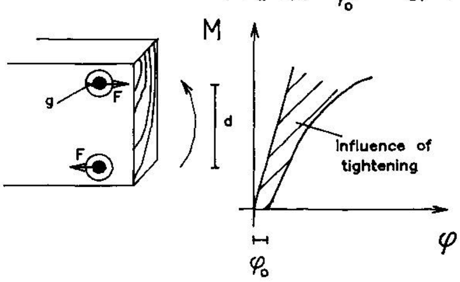
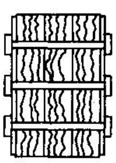
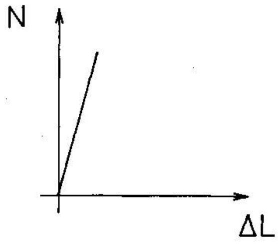
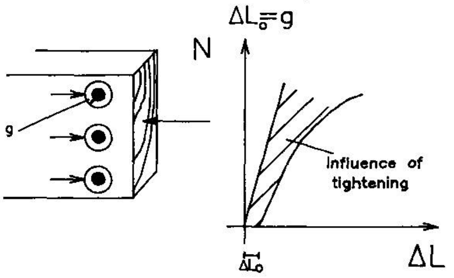
i) measures of the effctive out-of-plane of the lateral walls

ii) the set-up of a monitoring system able to give informations about the possibility of progressive deformations and the grade of sensitivity at the umidity and change of temperature

iii) the check up of the grade of the deterioration of wood.

Desription of these procedures and their related tecniques and results are given in details in [1],[2],[3]. In parallel of these in situ methods of inspections, an analitical interpretation of the overal static of the structure has been performed assuming, by hipotesis, rigid joints, elastic behaviour and values of mechanical parameters known by the previous mentioned in situ test. As it was possible to guess, the elastic analysis in this way performed has been not able to catch the real deformations of the structure (fig.7). It has been confirmed that the actual deflection is due to non-elastic behaviour of the structure, and in particular of the joints.
To be able to perform a realistic numerical analysis by which obtain results in term of deformations quite similar to the actual, it has been necessary to know experimentally the real constitutive law of the joints. The non linear behaviour of the connections may be essentially due to the following points:
i)local damages of the wood by non statical causes (insects,fungi,decay)
ii)climate and time dependent deformations
iii)localized deformations of some bolts
iv)instant deformability of the wood in the head joint
v)non-efficient tightening of the bolts
vi)the procedures and successive phases of construction.
Probably all these causes occur, the combination of which determines the actual flexural and axial rigidity of the joints and is responsible of the weakness of the structure. In table I the influence on the constitutive law of the tightening action, toughether with the probably non calibrated size of the

TABLE I

Structural head joint	Force of tightening	Influence of deformed holes of the bolts
		
Section A–A 		
Section B–B 		

holes is summarized for a typical head joint. More discussion and analytical consideration about the influence of each cause on the local behaviour of the joint may be found in [4].

4. NUMERICAL ANALYSIS AND EXPERIMENTAL TEST

As already mentioned, a linear numerical 3-D analysis on the entire structure, has been performed, as a first stage. A mesh using beam elements for the principal truss-arches has been prepared for a non-elastic analysis (whit ABAQUS v.4.9). The prototype constructed and tested in the laboratory of the University has the same dimensions and characteristics of the real arch-element, as shown in the photo of fig 8.
The Figure shows also the loading device able to simulate the load conditions of the element in the real structure, known by the results of the 3-D elastic analysis previous mentioned. In figure 9 some particulars of the instrumentation are shown.

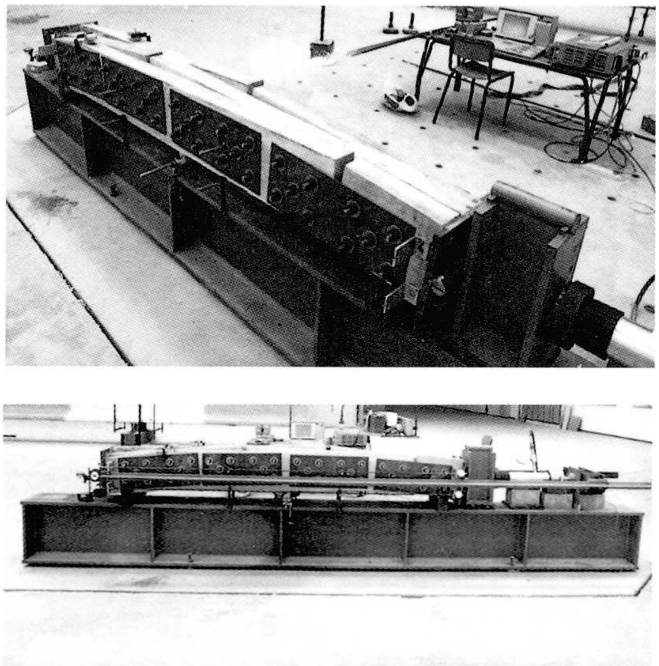


Fig.8 The model and the loading device of the test

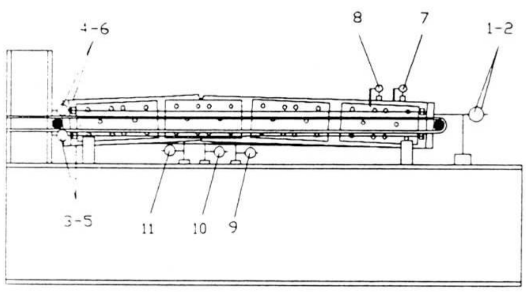


Fig.9 The model and the position of the instruments

The intensity of the actual tightening action of the bolts being unknown, it has been calibrated into four different steps, in each of them a different behaviour of the joints (and the entire element) at constant axial force has been observed and monitored during the test.
The maximum tightening force has been calculated such as the bolts on each head joint are able to transfer the axial force N by friction. According with Coulomb friction law and assuming the value of 0.35 for the friction angle, the maximum tightening force, in the case of an axial force N of about 7,000 Kg, must be equal to 700 Kg on each bolts (N/3 on the single lumber, see table I).
As mentioned the prototype as been tested giving four different intensity of tightening action, from the maximum to zero. The first test here reported concernes a prototype caracterized by holes of the same dimensions of the

bolts (g=0 in table I). The figure 10 shows the behaviour of the element loaded by an incresasing axial load centered at the two ands of the element (as shown in fig.8). Each curve represents the P-ΔL behaviour for a the definite tightening action. The diagram for the highest value of tighteting is linear elastic; in this case the maximum axial deformation of the element is of the same order of an equal monolytical wood element (ΔL_{exp}=0.3 mm, $\Delta L_{mon.}$=0.24 mm). In the other cases of less tightening forces, it has possible to read from the shape of the diagramms an initial minor rigidity due, probably, to local deformation of the wood subject to the pressure of the bolts.

As mentioned we refer here just on a first series of results, as a part of a more comprehensive investigation. In any case on the basis of these experimental results it is possible to make a first quantitative hipotesis on the influence of the tightening action on the deformability of the basic element of the arch. As a constitutive non linear elastic law of the element, such results have been implemented in the numerical analysis of the principal arch. The figure 11 shows the deformed shape of the arch in the case of Tf=700 and Tf=0 (referred to the moment of the torquemeter Tf=25 Nm and Tf=0 respectively).

A second test is in progress with the aime of quantifying the influence of the tightening force on an element with non calibrated holes.

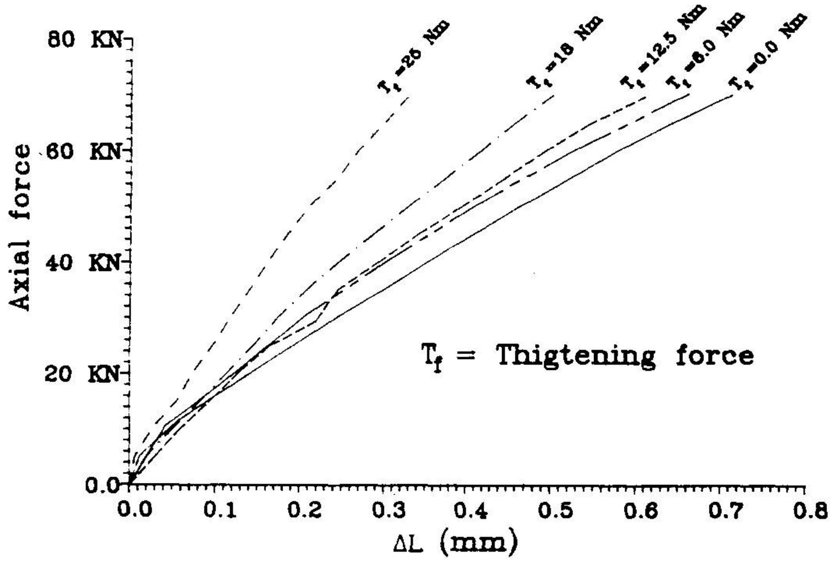


Fig.10 P-ΔL experimental behaviour for different tightening force

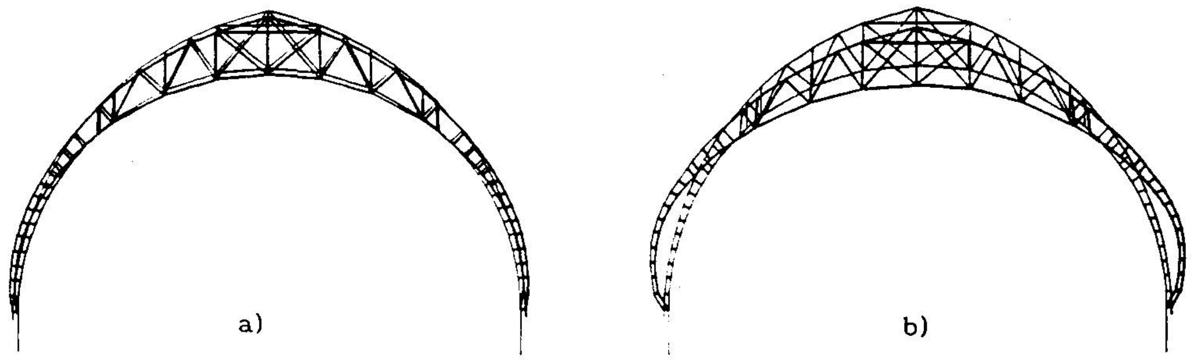


Fig.11 a)the deformed shape in the case of linear elastic analysis (Tf=700)
b)the arch deflection in the case of non elastic behaviour (Tf=0)

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