



# Behaviour and rehabilitation of queen post timber trusses: A case study

M. del Senno<sup>1</sup> & M. Piazza<sup>2</sup>

<sup>1</sup> *TTI Trees and Timber Institute, CNR, S. Michele Adige (Trento), Italy.*

<sup>2</sup> *Dept. of Mechanical and Structural Engineering, University of Trento, Trento, Italy.*

## Abstract

The availability of a wooden queen post truss of the early nineteenth century has provided a unique opportunity to evaluate performances and need for restoration of these structures, and to assess the possible intervention procedures, resorting to both traditional and innovative (glued in steel bars) methods. The truss has been characterized geometrically and statically, and its material (*Abies alba* Mill) has been classified. The damages have been identified using non-destructive test methods, and the actual performance of each timber element of the truss has been assessed by means of full scale loading tests in the elastic field. The same type of tests has been carried out on those components that had undergone restoration. The data gathered in the diagnostic phase have provided the input values for numeric simulations aimed to propose a predictive model for the truss. These simulations have been validated against the results of load tests, carried out both under static and cyclic (low-frequency) loads. Results show, on the one hand, the opportunities for intervention on this kind of structures offered by innovative connections exploiting glued in bars, and, on the other hand that the static behaviour of this type of truss requires a rather sophisticated approach to be satisfactorily predicted.

## 1 Queen post trusses

Trusses have long been utilised in timber constructions, becoming the covering support element “par excellence”. Simple construction, possibility to efficiently exploit a rather abundant and easily processable raw material have made the truss to become the main feature of the decking systems.

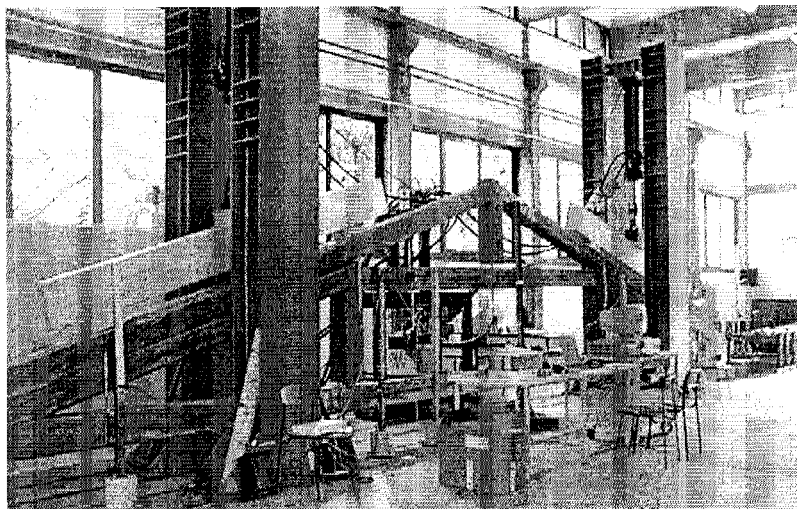


Figure 1: The investigated truss in testing configuration.

Andrea Palladio provided, if not the first, a systematic description of trusses at least the most significant up to his times, and an accurate approach to the interpretation of their behaviour. Paradoxically the so called “palladiana” or Queen post truss, that allowed to cover important spans by raising the top (where struts join) introducing a two chords system, does not behave as a truss, and its equilibrium is provided by geometric and load symmetry. This efficient configuration is very likely to be found in most ancient building of some size. The preservation of such buildings, provided that masonry be still able to support the covering frames, depends, obviously, on the presence of a sound deck, safely supported by trusses and beams.

Since static performances of truss depend, by and large, on its elements spatial organisation, rather than on their cross sections, alterations of the system geometry can give rise to danger situations. The system geometry is in turn connected to the correct performance of all the connections and structural joints. An accurate survey of the system geometry is a desirable investigation tool in order to define the performances to be expected, but, when dealing with century old buildings, properties poorer than expected are often found. Provided that support systems are working properly, one has to investigate the conditions of the timber structure, that is of its single elements.

Therefore it is critical any loss of mechanical performance of these members, in which, moreover, stress concentrations are likely to arise in presence of the aforementioned geometry alteration, triggering a kind of avalanche effect. It must be moreover kept in mind that some elements and joints (namely the tie beam ends and the tie - rafter connections) are generally close to or sunk in masonry walls, that represent an easy access way for moisture, this latter being the most dangerous environmental threat for timber building elements.

## 2 Non-destructive tests

These considerations show the importance to provide an exhaustive picture of the physical and mechanical characteristics of the object for which the intervention is intended [1]. The determination or, more correctly, the estimate of residual strength parameters is certainly the most important and delicate phases, bearing in mind that data acquisition is affected by a general interpretation uncertainty that can be lessened only resorting to a comparison between results gathered with different test methods. Moreover the need to know the actual element condition cannot lead to its destruction, but must be satisfied by means of tests not liable to alter the physical and mechanical conditions of wood (Non Destructive Tests, NDT).

ND tests result uncertainty and the lack of tests of this kind standardised and generally agreed, lead to conclude that more tests must be carried on the same element in order to compare different results obtained from different methods [2].

The different test methods available and most frequently utilised for timber structures can be classified into two main groups:

- local non destructive tests, by means of which a limited timber volume is investigated and, generally, only near the surface: these tests do not provide quantitative information, but, when carried in many places over an element, they can provide a significant answer at least from the qualitative standpoint, of the conservation state;

- global non destructive tests: they are carried out on the whole of the element and are usually based on considerations referring to propagation speed of ultrasound waves or to vibrational element properties.

N.D. tests can also be classified on the basis of the measured physical entity [2]:

- ultrasound propagation;
- stress waves propagation;
- vibration frequency;
- indirect density measurements near the element surface layers (DIMS hardness tests [2] and Pilodyn®) or about the whole cross section (Resistograph®).

Global N.D.T. are the most interesting for those who will deal with intervention design, since they can directly provide information needed for structural analysis. Unfortunately, they demand precise definition of either boundary conditions (direct load tests) and, also, of the actual structural mass (vibrational test methods). Moreover, acceptable reliability of measurements of longitudinal wave transmission is often conflicting with the feasibility of in situ tests, the element ends being in many instances not accessible.

However, before any instrumental investigation, it is of paramount importance to carry out a visual inspection, that not only allows to define the main characteristics of the structure, but allows also to reveal defects, decay and damages, if present, of the single element. It must be remembered that, also at European standardisation level, rules have been set for the visual timber stress

## 806 *Structural Studies, Repairs and Maintenance of Heritage Architecture VIII*

grading. There are, however, difficulties and limits to apply such method in field operations. An instrumental testing campaign, anyway, can only be organised after a thorough visual investigation. Then a first instrumental analysis carried with quick acquisition techniques, can focus those portions of an element in which a defined parameter features relatively low values; in such portions it will be later appropriate to perform more refined investigations.

Pilodyn® (figure 2), for example, provides a rough and ready estimate of timber surface hardness on the basis of the penetration depth of a metallic point, therefore showing – in an indirect way - the timber state of conservation. A more exhaustive picture can be obtained by means of other methods, that allow to get information about the whole cross section of the element: e. g. the so called “penetrometers” measure the energy to drive a rotating cutter into the timber.

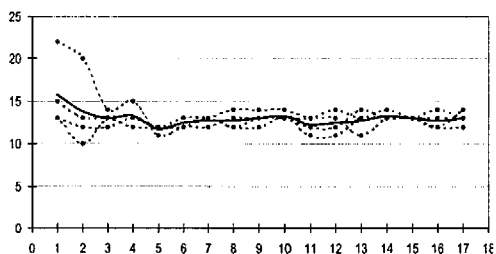


Figure 2: N.D. test with Pilodyn® 6J. The mean values obtained on each face of the 17 sections studied of the element show, evidently, a possible damaged zone on one beam end (sect. 1).

### 3 Case study

#### 3.1 Provenance of the investigated truss

The truss utilised for the experimental campaign came from a building located in Trento, owned by the municipal administration and which was built certainly before 1885, when a cadastral map was drafted, in which this building was already included.

The roof structure consisted of three main trusses (among which was the one studied in this case) supporting the ridge beam, and four smaller secondary trusses on which rested the hip rafters (see figure 3).

In the frame of a cooperation between the Department of Mechanical and Structural Engineering of the University of Trento and the *TTT* Trees and Timber Institute, CNR, S. Michele Adige (Trento), the structural elements, once dismantled, have been stored in the premises of the latter, where investigations and testing have been performed that, together with theoretical and numerical analyses, have allowed a complete definition of the trusses mechanical and physical properties and the design and execution of precisely focused rehabilitation interventions.

The considered element has been one of the three main trusses. After an accurate geometric survey and thorough diagnostic investigations (NDT methods have been coupled to bending tests on some elements, such as rafters), the properties of the material have been assessed, identifying the areas where damages and /or decay had taken place. The truss has been then reassembled, and a thorough test campaign has been performed, in both static and quasi static load conditions.

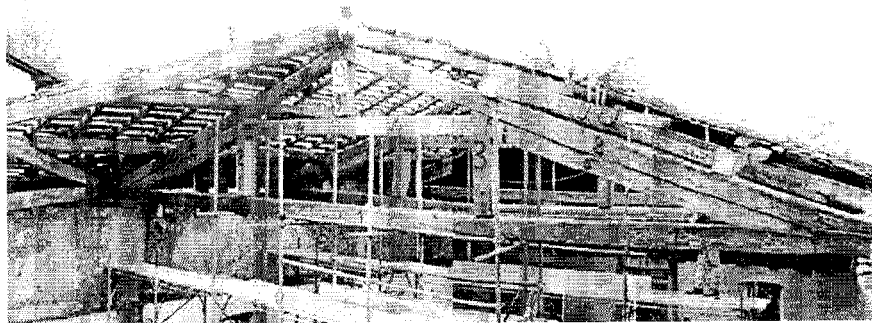


Figure 3: The timber trusses utilized for the research still on site.

Would the designer really know the behaviour of joint areas in either traditional “carpentry” or strengthened joints, by looking for the values of the joint rotational stiffness, both in the elastic and post-elastic field, a detailed analysis is undoubtedly needed that only an experimental research, supported by theoretical and numerical models, can warrant.

### **3.2 Areas with high uncertainty level**

Some areas of the investigated truss can be singled out whose mechanical behaviour is affected by a high uncertainty level. Classically these structures are analysed as constituted by members (both struts and ties) with constant (at most linearly variable) cross section, connected by perfect joints, generally hinges. Actually, real joints are known to behave in a semi-rigid way [3], with not negligible rotational stiffness causing a stress distribution significantly different from those derived from the classical assumptions [4]. Moreover when dealing with a restoration (and/or rehabilitation) project for a historically important building, the classical approach proves generally inadequate to quantify the safety level warranted by the truss. Or, again, the need can arise to verify the structure reliability when subjected to new loads, dictated by new end uses, or to lay down a seismic resistance analysis.

Resistance and, in some instances, stability, analysis of single members is rather easy, but stress distribution on the same members is strongly affected by

the semi-rigid joint behaviour. Joint areas are characterised by the greatest uncertainties, since there important phenomena of stress concentration and unforeseen stress components arise, and there reinforcing elements are placed.

In the following we attempt to outline these high uncertainty level areas of the structure, emphasising the main characteristics of some of them and trying to give an interpretation of their behaviour by means of a finite elements model.

Bi-dimensional models were set up, accurate enough to simulate the experimental test conditions (ABAQUS<sup>®</sup>5.8 F.E.). Some aspects, typical of modelling continuous structural problems involving wood and complex geometries, are worth reporting here. The simulations took first of all into account the anisotropic nature of wood as a continuous material [5]. The constitutive law was reproduced according to an orthotropic elasticity approximation, thus prescribing different elastic moduli in direction parallel and perpendicular to the grain [6]. Furthermore, the material response to compression perpendicular to fibres beyond the elastic limit was considered by means of a yielding model according to Hill's potential and an appropriate hardening rule, fitted on the plastic behaviour perpendicular to the grain.

Boundary conditions were also a major issue. The interaction based on friction between the various surfaces forced to contact (timber-to-timber and steel-to-timber) was reproduced through the "*finite sliding*" formulation [7] which allows for arbitrary detachments and surface penetrations perpendicular to the contact surface. Friction was implemented with reference to Coulomb's friction model [8].

As for the discretization of the structure, models have been created using quadrilateral and triangular second order isoparametric elements with a "*plane stress*" formulation. Through a sensitivity analysis, the best size and shape of the elements has been chosen in order to obtain the most effective mesh arrangement at the lowest computational costs, especially in the high uncertainty areas (especially for the strut-tie joint, see figure 4).

Strut and tie joints feature double connections, involving both upper and lower element of the two-layer strut, obtained by indenting in the tie-beam a recess for the slanted ends of the compressed members.

A transversal steel reinforcement (a pair of bolts) is present, subjected to bending and axial force. Stress/strain conditions near the indented regions have been studied, together with the onset of stress components (local stresses orthogonal to the fibres partially originated by friction) and looking for a correlation between the member axial load and local stress levels in the joint. A general discussion about the possibility of a correct modelling of these kinds has been presented by Parisi [9].

### **3.2.1 Strut / tie-beam joints**

Strut / tie-beam joints feature double joints, involving both upper and lower element of the two-layer strut, obtained by milling in the tie-beam a recess for the slanted ends of the compressed members (figure 4).

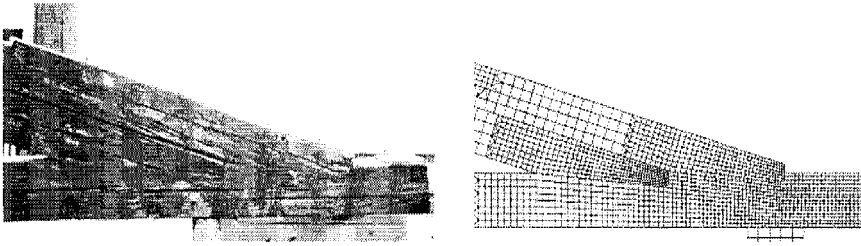


Figure 4 A strut / tie-beam joint and the correspondent mesh .

### 3.2.2 Indented scarfed joint in the tie-beam

The tie beam of the examined truss is characterised by an indented scarfed joint (figure 5). The tie beam is mainly subjected to tensile stresses, even if a weak bending component, due to the end moments (caused by the semi-rigid end joints) and to the dead weight.

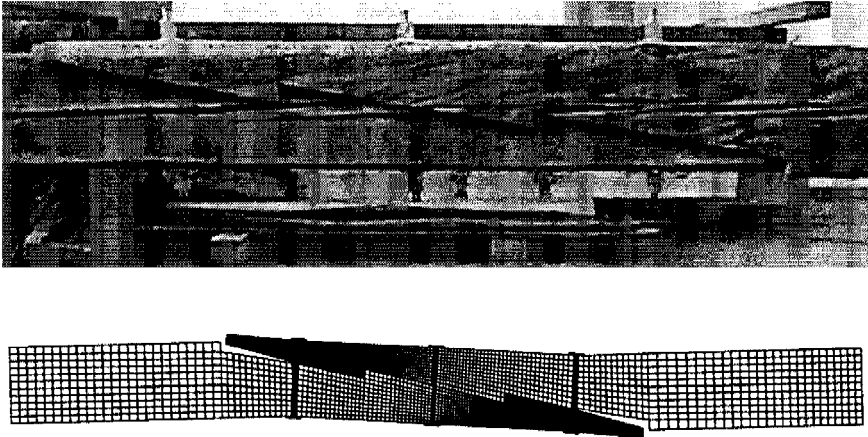


Figure 5: Indented scarfed joint and correspondent mesh at incipient failure

The tie beam of the examined truss is characterised by an indented scarfed joint. The tie beam is mainly subjected to tensile stresses, even if a weak bending component, due to the end moments (caused by the semi-rigid end joints) and to the dead weight. Earlier investigations, both numeric and experimental, carried out by the authors show this joint to exhibit significant stiffness and resistance losses with respect to the continuous corresponding timber element. In this case and according to the numeric model hereafter discussed, the joint has a tensile strength of about 20 % of that of the continuous element. Since this behaviour is far different from the one that should be exhibited by the original tie beam, it has been decided to accurately modelise the joint by means of an ABAQUS<sup>®</sup>

810 *Structural Studies, Repairs and Maintenance of Heritage Architecture VIII*

characterised by an extremely high detail level: through the well known sub-structuring technique, only the detailed numerical model of the joint has been studied. The results of this model has been successively implemented on the global model of the truss, utilizing a special set of elements, in which the mechanical properties of the joint (stiffness and strength) have been implemented. For the analysed substructure, the model has been conveniently extended beyond the joint area, on a distance equal to 5 times the greater size of the tie beam, starting from the position of the last indent.

In the specific case also reinforcement devices, namely 3 bolts  $\phi$  24 mm, 5.6 steel grade, have been inserted. The initial tests for the characterisation of the wood material have allowed to define a strength class C 30 (according to EN 338 [10]).

It must be noted that the interactions between timber surfaces involve two contact modes: one between surfaces with almost parallel grain, and one between surfaces with almost perpendicular grain. The main problem is to define the friction coefficient  $\mu$  to be attributed to each single mode, acknowledging that the friction coefficient for grains sliding crosswise to one another is far higher than the one for grains sliding in parallel direction. Utilising a constant ratio equal to 3 between these coefficients, different values of  $\mu_{\text{along\_grain}} = 0.1 \div 0.2$  have been introduced (as a consequence  $\mu_{\text{across\_grain}} = 0.3 \div 0.6$ ), in order to evacuate the effect of this coefficient on the model results. It must be observed that, luckily, this effect is practically negligible for the global stiffness and strength values.

On the basis of the performed analyses, the joint behaviour exhibits a significant post – elastic phase, induced by the beneficial effect of the metallic elements, working also (and mostly) after the shear failure has taken place in the wooden teeth along the horizontal surfaces near the indents.

Figure 6 shows the test set up, with the identifiers used for the LVDTs instruments, and the hydraulic jacks utilised for loading the structure, named SX and DX in table 1. The same table 1 summarizes the different tests (static and cyclic quasi-static) performed so far.

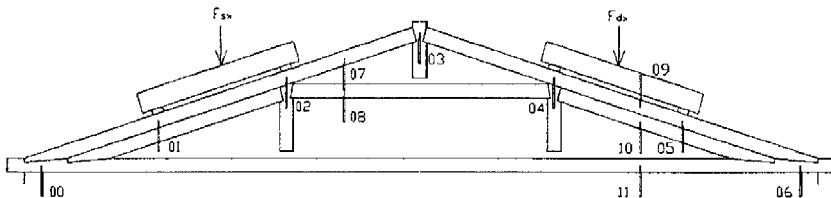


Figure 6: General testing lay out

Table 1. Schematic outline of the experimental tests.

Test	Max load SX (kN)	Load variation SX	Max load DX (kN)	Load variation DX
1	3	Constant	36	Linear
2	36	Linear	3	Constant
3	73	Linear	73	Linear
4	4	Constant	60	Linear
5	60	Linear	4	Constant
6	77	Linear	77	Linear
10	28	Sinusoidal	28	Sinusoidal
11	0	Sinusoidal	28	Sinusoidal
12	28	Sinusoidal	0	Sinusoidal
13	63	Sinusoidal	63	Sinusoidal
14	3	Sinusoidal	55	Sinusoidal
15	55	Sinusoidal	3	Sinusoidal

Tables 2 (static tests) and 3 (cyclic tests) give a synthetic comparison between experimental and numerical results (ABAQUS® code) with regard to the displacements in the relevant joints of the truss. In the same tables, the mean differences between experimental and numerical values are reported as percentage. The results reported in table 3 are the maximum displacements of the last of 20 quasi - static cycles.

Table 2. Displacements (mm): comparison between numerical values (ABAQUS®) and experimental data (static tests).

TEST	Transd. n° 1 Lower Strut SX		Transd. n° 2 Queen post SX		Transd. n° 3 Middle		Transd. n° 4 Queen post DX	
	Exp.	Num.	Exp.	Num.	Exp.	Num.	Exp.	Num.
01	0.28	0.31	0.19	0.19	4.16	3.82	22.08	23.88
02	14.67	14.34	21.40	23.07	5.50	5.04	0.47	0.43
03	27.21	28.45	40.24	42.47	18.44	17.16	41.82	45.12
04	0.61	0.68	0.55	0.51	7.63	6.86	38.84	43.26
05	25.75	26.89	37.61	39.48	9.73	9.51	0.76	0.90
06	28.82	30.25	42.62	39.98	19.46	19.07	44.15	46.98
Mean	5.3 %		0.9 %		-6.3 %		7.3 %	

Table 3. Displacements (mm): comparison between numerical values (ABAQUS®) and experimental data (cyclic tests).

TEST	Transd. n° 1 Lower Strut SX		Transd. n° 2 Queen post SX		Transd. n° 3 Middle		Transd. n° 4 Queen post DX	
	Exp.	Num.	Exp.	Num.	Exp.	Num.	Exp.	Num.
	10	16.45	16.82	13.86	12.95	26.50	25.32	7.21
11	0.24	0.26	0.86	0.78	24.98	23.43	2.98	3.11
12	n.a.	12.12	n.a.	14.11	0.96	0.82	4.33	3.84
13	33.24	33.34	26.26	25.14	60.60	56.16	14.93	16.13
14	n.a.	2.20	n.a.	3.33	53.78	52.65	6.91	6.02
15	30.24	34.07	22.93	23.95	2.50	2.29	7.95	6.65
Mean	6.0 %		-6.2 %		-7.1 %		-4.7 %	

n.a.: values not available due to data acquisition failure

These tables clearly show a good fit between the values worked out by means of ABAQUS® and the experimental results, the error mean keeping under 7 %. It must be noted that asymmetrical configurations (01, 02, 04, 05) induce slightly greater errors than the symmetrical ones (03, 06).

Experimental results for the king post vertical displacement are clearly underestimated by the numerical code: this should depend by the displacements of the struts meeting at the king post that have been shown, instead, to be well described by the numerical model. This inconsistency can only be explained by the faulty geometry of the contact surfaces between struts and king post.

It is of particular interest to note that the structure, even if approximately symmetrical and symmetrical loaded (test 03 and 06 for instance), does not behave symmetrically.

#### 4 Discussion of intervention criteria

A comparison has just been shown between experimental results and a high discretisation, non linear numerical model, stressing the decisive role of numerical modellisation in defining stress concentrations in particular structure areas.

We want now set a comparison with another, less sophisticated model, but more within the grasp of the professional designer. It is a model involving variable dimension beam elements, and non linear joints between the different elements (SAP 2000® NL code), assuming anyway a linear behaviour of the material.

Analyses according to the european recommendation ENV 1995 [11] on all elements and in joint areas (figures 7) allow to define a limit load for the structure, due to the compression – bending failure of the upper strut element.

Detailed analysis of the strut – tie area by means of ABAQUS® defines instead a failure, at about the same load level, in the heel area of tie – strut joint, because

of the simultaneous shear stresses along the grain and tensile stresses across the grain, this being in turn caused by the rotation of the strut end associated with the friction between strut end and joint heel.

This failure is shown by the symbol  $I_f$  (Tsai – Hill criterion). In this case such a criterion holds if associated to tensile stresses across the grain. Anelastic strains, significant but not such as to induce failure, can instead be localised in the compressed area. A correct definition of the actual joint behaviour should clearly depend on a numerical model taking into account crack triggering and propagation according to the methods of analysis of Linear Elastic Fracture Mechanics.

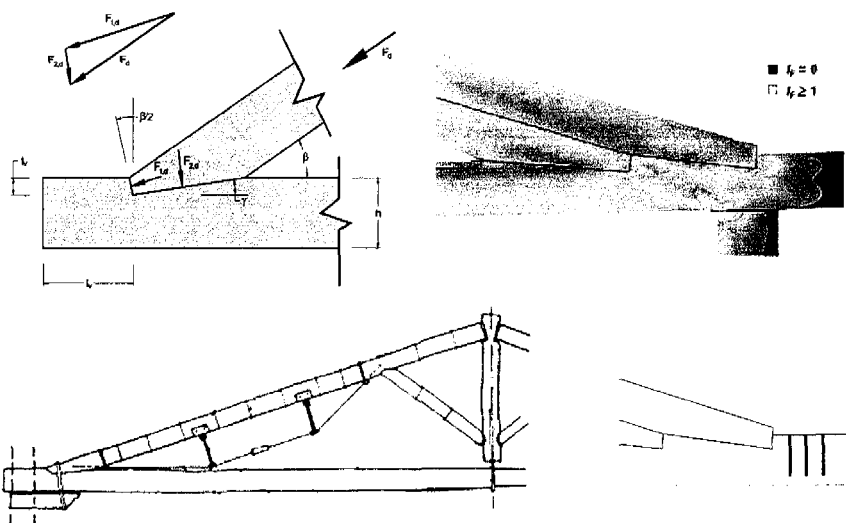


Figure 7: Results of numerical analyses for the strut/tie-beam joint and different proposals for strengthening.

Correct identification of failure mode is essential to correctly define the rehabilitation intervention [12], should one be proposed. In this case the significant discrepancies seen in the incipient failure modes, lead to two different proposals for such a rehabilitation (figure 7).

Moreover Eurocode 8 (Annex J of Part 4, devoted to "Strengthening and repair of buildings" [13]) does not supply specific design or analysis tools, but gives general directions for structural strengthening. It is interesting to note that Annex F, dealing with strengthening of monumental buildings, recommends that intervention should always preserve the architectural and constructional identity of the building: this recommendation extends to the original structural system that is part of the cultural heritage. Indeed, "only when the original structure is clearly insufficient against earthquakes should the addition of structural elements contributing to seismic resistance be considered. Nevertheless, such additional



elements should fulfil the requirement of compatibility with the original architectural and mechanical features of the construction".

## References

- [1] Larsen, K. E., Marstein, N., *Conservation of historic timber structures*, Butterworth-Heinemann, Oxford, 2000.
- [2] Piazza M., Baldassino N., Zanon P., In situ evaluation of the mechanical properties of timber structural elements, *Proceedings of the 10<sup>th</sup> Int. Symposium on Non-destructive Testing of Wood*, Lausanne, Switzerland, pp. 369-377, 1996.
- [3] Candelpergher L., Piazza M., Mechanics of traditional connections with metal devices in timber roof structures, *Proceedings of the 7<sup>th</sup> International Conference STREMAH 2001*, Bologna, pp. 415-424, 2001.
- [4] Ehlbeck J., Kromer M., Carpentry joints, Lecture C12 *Timber Engineering STEP I*, Centrum Hout, Almere, pp. C12/1-C12/7, 1995.
- [5] Bodig J., Jayne B.A., *Mechanics of Wood and Wood Composites*, Van Nostrand, New York, 1982.
- [6] Kollmann F.P., Cote' W.A., *Principles of wood science and technology*, Springer Verlag, Berlin, 1984.
- [7] Oden, J.T., Pires, E.B., Nonlocal and nonlinear friction laws and variational principles for contact problems in elasticity, *Journal of Applied Mechanics*, **50**, pp. 67-73, 1983.
- [8] Mckenzie W.M., Karpovic H., The frictional behaviour of wood, *Wood Science and Technology*, **2**, 1968.
- [9] Parisi M.A., Piazza M., Mechanics of plain and retrofitted traditional timber structures, *J. Struct.Engnrg., ASCE*, **126 (12)**, pp. 1395-1403, 2000.
- [10] *EN338, Structural Timber classes*, CEN/TC124, Brussels, Belgium, 1995.
- [11] *prEN 1995-1-1: Design of timber structures; Part 1-1: General rules and rules for buildings*, European Committee for Standardisation, Final Draft, April 2001.
- [12] Del Senno M., Piazza M., Proposals and criteria for the preliminary evaluation, the design and the execution of works on ancient load bearing timber structures, *Wooden Handwork/Wooden Carpentry: European Restoration Sites*, Elsevier, Paris, pp. 263-277, 2001.
- [13] *Eurocode 8. Design provisions for earthquake resistance of structures - Part 1-4: General rules - Strengthening and repair of buildings*, European Committee for Standardisation, January 1996.