Iscarsah
International Scientific Committee on the Analysis and Restoration of Structures of Architectural Heritage

12-13-14
newsletter
April-December
2016

ICOMOS
International Council on Monuments and Sites
Conseil International des Monuments et des Sites

Visiting Loboc Church in Bohol (Philippines).

ISCARSAH, ICORP and CIPA Joint Meeting in Istanbul (October 2016).

Message of the President Görün Arun

This year was very busy for ISCARSAH members in achieving ISCARSAH mission.

National Commission on Culture and the Arts of the Philippines (NCCA), Philippines National Museum, Bakás Pilipinas, Inc., ICOMOS Philippines and University of Santo Tomas organized a technical Symposium for an audience of Filipino structural engineers, architects and preservation professionals on *Seismic Retrofit of Unreinforced Masonry Heritage Structures in the Philippines*, on 14-16 January 2016 in Manila (Philippines).

Ten ISCARSAH Expert members were invited as lecturers to the Symposium.

Before the Symposium, on 12-14 January, the group was taken to island of Bohol where Mw 7.2 earthquake hit on October 15, 2013 to see, evaluate and exchange ideas on the damage (and some collapse) to several historic 18th and 19th century Spanish-era coral-stone-faced unreinforced masonry churches.

The ISCARSAH annual meeting was in Leuven, Belgium during SAHC2016 on 13 September.

ISCARSAH members joined this meeting. In-Souk CHO made a presentation about her proposal to make 2017 ISCARSAH meeting in Seoul and a workshop on the World Heritage Sites in Korea. This proposal was accepted unanimously. The group decided to continue on preparation of ISCARSAH Guidelines and Prof. Pere Roca undertook this duty.

The next ISCARSAH meeting took place on 19th of October during the 2016 ICOMOS Annual General Assembly that was on 15-20 October in Istanbul.

In this occasion, for the first time ISCARSAH, ICORP and CIPA made a joint meeting on 20th of October. In this joint meeting, after each Scientific Committee introduced themselves, the need and importance of shoring in response phase of the disaster was highlighted. Possibility of preparing a common document among SAR and Humanitarian groups was explained. The need of focusing on the missions and having a common topic as work on disaster area among three ISCs were reminded.

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Reunión del Comité ISCARSAH en Lovaina (Bélgica)

María Margarita Segarra Lagunes
Università degli Studi Roma Tre - Vicepresidente del ISCARSAH

En el marco de la 10ª Conferencia Internacional *Structural Analysis of Historical Constructions SAHC 2016*, celebrada en Lovanio (Bélgica) en los días 13-15 de septiembre de 2016, se llevó a cabo la reunión del Comité ISCARSAH, con la participación de numerosos miembros procedentes de los cinco continentes.
El evento fue albergado en el aula intitulada a Monseñor Óscar Arnulfo Romero del *Collegium Veteranorum* de la Universidad Católica de Lovaina. El aula, situada en la planta alta del antiguo edificio de ladrillo y piedra construido hacia 1610, conserva una magnífica armadura de madera, típica de la arquitectura de los Países Bajos.
En la reunión se discutieron varios temas, relacionados con la actividad del Comité. Fue analizada y aprobada la propuesta presentada por In-Souk Cho, representante de Corea, de organizar en 2017 un encuentro en Seúl. Durante la reunión, algunos miembros hicieron breves presentaciones sobre investigaciones en curso. En especial, Ramiro Sofronie habló sobre *Advances in seismic strengthening of brick and stone masonry buildings*; Arkadiusz Kwiecień, sobre *Present protection and planned reconstruction of heritage city center in Nepal after the 2015 earthquake* y Stephen Kelley, sobre *Damage and Emergency Shoring following the 2016 Kathmandu Valley Earthquake*.
Se retomó asimismo la idea de avanzar en la redacción de las *Líneas guía* ISCARSAH, confiando a Pere Roca Fabregat la coordinación del grupo de trabajo. Quien escribe presentó el número doble (10-11) de la ISCARSAH Newsletter (2015), invitando a todos los miembros a colaborar con artículos y noticias para publicar en las próximas ediciones. Se informó también sobre la actualización de la base de datos de expertos, cuya coordinación está a cargo de la Presidente Görün Arun. La reunión se concluyó por la noche en un agradable convivio en un restaurant típico de Lovaina.

Reunión del Comité ISCARSAH en Lovaina (13 Septiembre 2016).

Frequent errors of ancient timber structures and consequent failures

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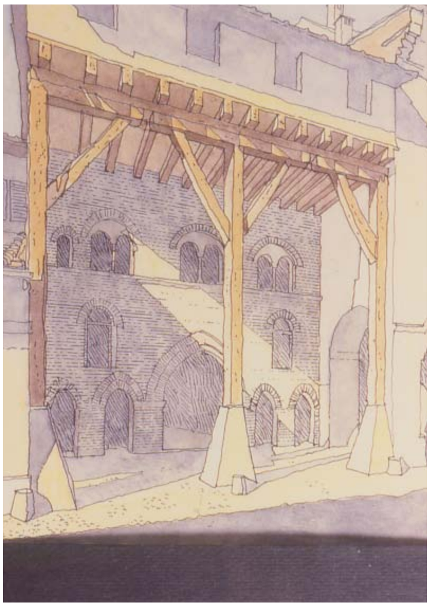
Invited Paper to IF CRASC 2017, IV Convegno di Ingegneria forense, VII Convegno di Affidabilità strutturale e consolidamento, Politecnico di Milano, 14–16 september 2017 (from the Proceedings with a few corrections and additions).

Summary
The most frequent errors of ancient timber structures are unsuitable evaluation of the ratio between the acting loads and the size of the members, lack of three-dimensional conception of the structure, inappropriate assembly, inadequacy of the configuration; in addition unawareness of the deformability of the timber members stressed by long term working loads.
The insufficient sizing of the members cannot be always considered an error when dealing with structures built when the possibilities of correct calculations where non existent or scarce. Some evident errors, anyway, should also be considered as an attempt to cut down the building costs for the structures. Buildings were not planned to last forever.
The effects of the errors are deformations, instability, cracks, failure, collapse.
Which conservation principles should be adopted? What seems to be advisable in general is not to correct the errors but control and govern their consequences.

1. Foreword
The paper is concerned with the errors currently made when planning, constructing and using the timber structural systems. This also demands to focus on practical and theoretical problems that technicians meet when a liability judgment and conservation interventions are requested for structures that are affected with errors.
The errors, of different kind as it will be specified, are examined here separately as pertaining to the single members or to the structural units (floors, trusses, frames), besides to the joints, finally to the whole system.

It ought to be noted, before any other consideration, that timber members are very flexible therefore, when stressed to bending, they can undergo immediate temporary considerable deformations, before ruptures occur. Deformations caused by even faint loads but working for a long time, occur slowly but they are permanent and can reach substantial values (viscoelastic creep of the wood).
The consequences of one or more members being considerably deformed are that a suitable geometry conferred to a structure can be distorted, i.e. the stresses eccentricity the structure could be hit by instability. There is the risk of failure and collapse of the building even before limit stresses in the members are reached and overtaken.
From the start of the iron construction, designed in imitation of the timber ones that had a very long tradition, a frequent error was to choose small size sections for the members relying on the high strength of the material, obviously much higher than that of the wood. Several cases of instability with consequent failure occurred, especially for the bridges where the loading regimen has important dynamic components.
Modern calculation rules of new timber structures, in fact, demand for limitation of both stresses and deformations. A similar approach is requested when considering or designing a joint between two or more timber members. No ancient timber structure has connections that work as fixed joint because of the deformability of the wood tissues under compression.
This characteristic, the ductility, is highly appreciated in seism prone areas.
The introduction of struts or braces connecting two members was the measure generally adopted when relative rotations of the members were not desirable; recurrent examples are the struts between pillars and beams, omnipresent in the joints of the timber building.
An industrial construction in Paramaribo, Suriname, shown in the picture, is an interesting example of this kind of arrangement.
With this paper the author also aims at showing the strict interconnection as well concurrence, when a timber structural system undergoes failure, of several errors and causes of disease.



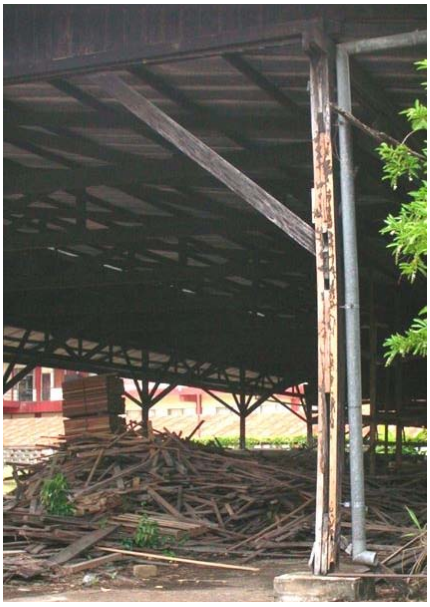


Figure 1. Timber porch of a Middle Ages masonry building in Bologna (from P. Donati, 1990).
Figure 2. Timber frame in Paramaribo showing, trough the deformations, the behaviour of a node of a frame stiffened by means of a strut.

Amongst these the decay of the wood caused by fungi and beetles occupies a primary place.

2. Errors

In spite of the recognized importance of the deformations, first of all one should quote a mistake generally made by technicians when surveying with traditional methods an existing building of any material, that is they do not pay much attention to the deformations and do not record them on the drawings. The drawings of this kind are useless.
The most frequent errors in the design of the ancient timber structures are the insufficient size of the members with respect to the duty (span, acting loads, constraints etc.), inappropriate assembly or placing, inadequacy of the structural scheme, absence of three-dimensional organization of the structure.
Furthermore, inappropriate use of the structures (overloading of a given structure designed for a less engaging use).
Not frequent is the error of placing the beams flat on their supports though a few cases are to be recorded.
No doubt the strength of a rectangular beam placed flat on its supports is lower than that of a beam correctly placed on edge but the not functional arrangement could be motivated by the practical need of keeping low the height of the member in order to not affect further the height of the room, sometimes already low.
Frequent error is locking up the end of a member or the node of a unit in the masonry support, as well lack of ventilation and protection against moisture, this way causing a condition that is favourable for attacks brought by beetles and fungi.
The consequent effect is the decay of the materials and the failure of the structures. Furthermore the lack of adjustment and maintenance over the time is amongst the most frequent errors.
A widespread error is the suspension of the false ceilings under the roofs to the tie beams of the trusses rather than to the rafters.
Unsuitable design of the relation with other structural systems, as the excessive pressure exerted by timber elements on the masonry supports, is a frequent error.
The specific effects of these errors, described in the paper, are deformations, instability, cracks, failure, collapse, generally evolving in a slow progression.

2.1. Insufficient size of the members

Structural calculation is not anterior to 19th C., with a very slow dissemination and application amongst the operators of the sector of timber structures though trials and assessments, also in planned series, to test materials and especially timber, were carried out since the second half of the 17th century to assess the specific strengths. Especially for the timber structures the size of the members was decided on the base of analogy with successful existing carpentries, practice rules, empiric tables, personal experience of the designer. Furthermore, by the availability of size in the stock. The budget was another item that would condition the choice.
An outstanding example of insufficient sizing of the members is offered by the timber roof carpentry, still standing, that covers one of the rooms of the Middle Ages Palazzo dei Vicari in Certaldo, Florence (Figures 3, 4). The structure of the roof is composed by the ridge and two side purlins, the biggest (the room has a trapezoidal shape) measuring more than 7 m; the three members have a variable section that can be assumed 17 x 17 cm on average. Since it is quite evident that they are undersized for such a large span one should presume that the builders were aware of that and made their choice for economic reasons or because they were operating in a hurry trying to use the materials available in the stock at that time.
All the members underwent a severe inflexion due to the deficient size, and creep but no ruptures, that usually accompanies the deformation of a roof, or leakage, caused by rainwater ponding, had occurred yet. Though no real structural damage had occurred, the situation was not acceptable with respect to the requirements of the present-day practice codes and structural safety standards.
Since inflections of this size are rare and, on the other hand, they show the phenomena with strong evidence of causes and effects that is very useful for further research, the strengthening measure adopted by the author (2013) was a permanent timber propping consistent with the original structure i.e. the addition of three supporting timber arches whose thrust is absorbed by steel cables placed horizontally. The deformations were not wiped out.
The tension regimen and the behaviour of each member of a simple or complicate truss, which is by itself a statically redundant structural unit with a high number of component members and internal constraints, and the behaviour of the same member were not (and aren't today) of easy understanding though the traditional terminology of some languages as the Italian one helps in using words that identify their role: for

Figure 3. Deflection of a member of a simple roof carpentry, Certaldo.
Figure 4. The deflection of the three timber members and of the joists.

instance, *puntone* (compressed member; rafter), *catena* (tended member; tie beam).
The struts of a truss (or of a joint) are simple members that may be considered connected by hinges at both ends and stressed by pure compression along the main axis and the grain. When they are too slender, there is the risk of deformations due to the critical Eulerian load overcoming. A defective sizing of this kind was rather usual in the 17th and 18th centuries when the understanding of the real function of these members had been wiped out and the problems of instability were not well known yet. The choice of the dimensions of the cross section was rather casual. Recurrent in this sense to find struts made by thick boards or planks rather than compact sections.
Speaking of struts, it ought to be noted that a process of instability can be triggered by the presence of the defect of deviation of the grain in one of the concurring members, eccentricities generated by imprecise geometry of the pieces, makeshift execution of the slots or mortise in the connection with the other members as well as irregularities in the shape of the same joint.
Similarly, the lack of symmetry of the pieces or of the hollows of the post in a truss (Fig. 11).
The deformation of the struts of a truss, whatever the cause, has the consequence of the rafters working only on two supports instead of three therefore generally incapable of carrying the permanent and temporary loads that weigh on them. The progressive inflection occurring in these cases becomes severe in the long period, followed by cracks of the member, failure of the truss and of the roof. It is exactly the case of the trusses of the roof of the Theatre of Sarteano, Siena (Tampone et al., 1987, cit.; Tampone, Atlas, 2016, cit.) (Fig. 4). The visible strut of one of the trusses shows clearly the deformation; the same for the rafter that is affected with very severe deformation and cracks. Both failures are caused by insufficiency of the size of the two members examined, combined with a questionable geometry (too long the part of the rafter, the lower, lacking intermediate supports).
The same strengthening principles cited before for the Certaldo's roof were adopted shoring the rafter and fitting it with a steel rod centring (Tampone, Franci, Campa, 1989, cit.). Belongs to the casuistry of this specific problem the excessive length of the bay left between the beams of a plane structural unit (a floor for instance). It is mainly due to search of economy and presumption of familiarity with structures. The effects are deformation of the beams and of all the other members, sinking of the whole unit (Tampone et al., 2005, on Palazzo Medici Riccardi, cit.). The purlins will meet with a similar condition when the bay of the trusses is excessive or the number of purlins too low with regard to the span of the roof (read

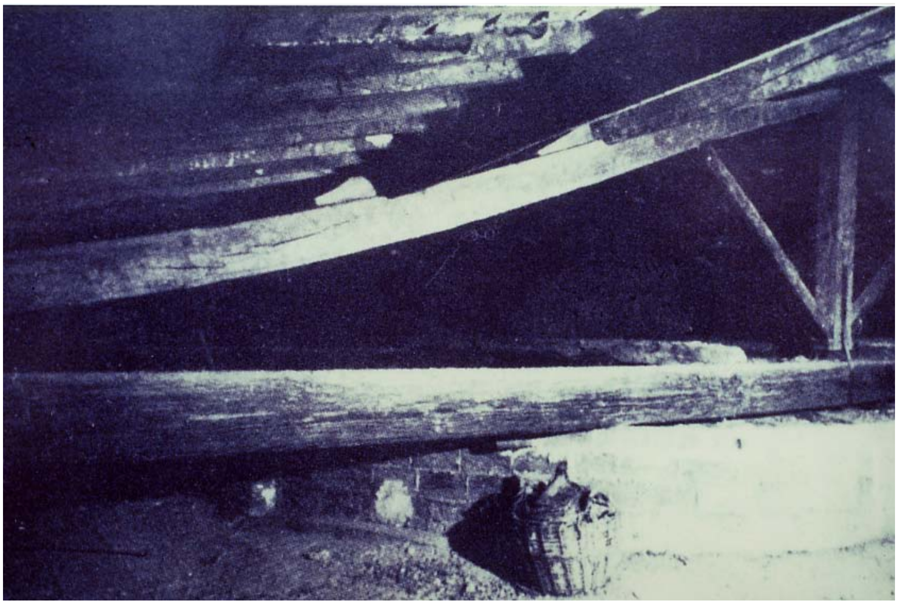


Figure 5. Very severe deflection of the rafter in the theatre of Sarteano, accompanied by cracks.

Figure 6. The last strengthening work, composed by an additional stiffening bracket, fixed with iron ties, and by a new strut, was not sufficient to prevent further deformation of the rafter and of the new strut of this truss (Tesi, Tampone, Martini, 1989, cit.). Note the cut made in the bracket to accommodate the end of the strut.

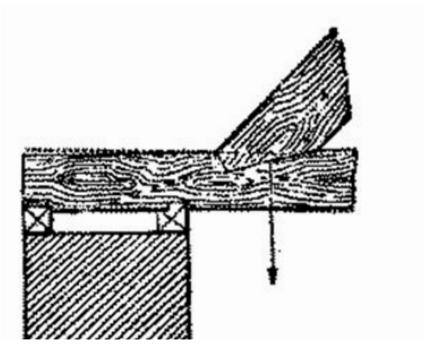


Figure 7. Short rafter, incorrectly designed node (error shown by Breymann, 1884, cit.).

Figure 8. Effects of combined situation of short rafter of a truss and fungal attack of the node. Note the deformation of the tie (Tampone, Atlas, 2016, cit.).

length of the rafter). The inflexion of the purlins is generally followed by the deflection of the joists and rotation of the next trusses as well, in the most severe cases, by piling up of the system of trusses.
The use of timber elements not long enough to realize the rafters of a truss is the cause of incorrect geometry of the truss and specifically of the node, that can turn into a series of behavioral problems and not rarely to the failure of the unit. The resultant of the actions exerted by the truss and by the over-structure happens to be not in line with the resultant of the reactions of the support.
Faint tensions are generated in the node and in the tie beam that, if the members are well proportioned, become significant only in case biologic attacks or other accidents reduce the strength of the material.
Main effects are local inflexion of the tie in sections downstream the node and widening of the angle between the members; cracks are inevitable (Tampone, Atlas, 2016, cit.).
Recent documentation on this erroneous arrangement and the possible consequences has been brought by the author (Tampone, Atlas, 2016, cit.) showing that the non-coincidence between the action of the truss, with the weights acting on it, on the support and the reaction of the same support causes a small bending moment.
Attempts to correct the imperfection were generally made placing a timber or stone bracket on the wall during construction.
The remedial measure is anyway really effective, within certain limits, only if an iron collar tie bounds and tightens all the members otherwise the end of the bracket loses contact with the tie beam, due to the rotation of the system and the deformation of the tie beam (Figures 8, 9) (Tampone, Atlas, 2016, cit.).

2.2. Inappropriate assembly

The assembly of the joint that connects the rafter to the tie beam was generally made, since the earliest examples known yet, as the very known early specimens (see over), with a mortise in the tie and a tenon at the end of the rafter, sometimes with the reinforcement of a collar tie that binds the two members. A different kind of joint is made, likewise the joint post-rafter, cutting the end of the rafter approximately along the bisectors of the angles of concurrence and making a cut in the whole thickness of the tie. In both cases the long-term efficiency of the joint relies on the length of the heel of the tie that is stressed by tangential tension. The incorrect design of the joints, as flimsiness or insufficient length of the heel of the tie beams of the trusses, is recurrent especially in the constructions as well as in the treatises and handbooks of the Renaissance dealing with the timber building techniques: the connections are represented with just a small tooth (rarely two) in the rafter and a shallow hollow in the tie beam. The biologic attacks consequent to

Figure 9. Node of the truss of the Certaldo Palace of the Vicars roof showing severe failure.

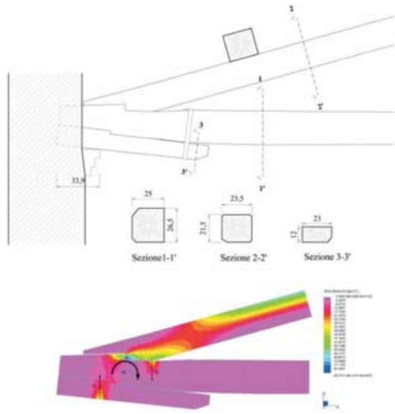


Figure 10. FEM analysis of the node with assessment of the punctual tensions and deformations.

Figure 11. Rupture of a rafter in the joint with the strut due to the hollow made in the member (from Tampone, Atlas, 2016, cit.).

Figure 12. Makeshift assembly, with considerable eccentricity, of a post to the rafter of a truss. The post is deformed (from Tampone, Atlas, 2016, cit.).

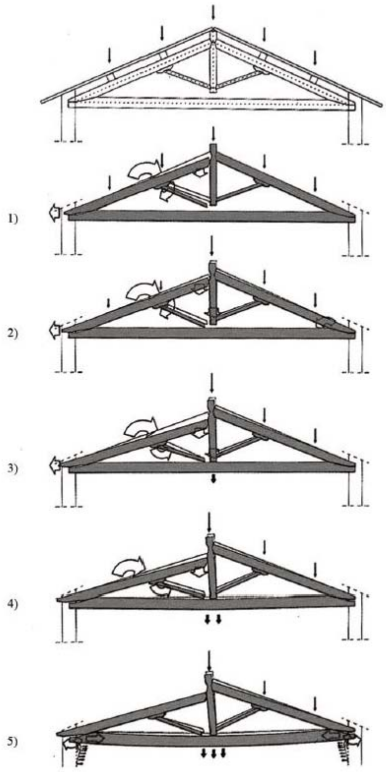


Figure 13. The scheme (Tampone, 2002, mod. 2009, cit.) reproduces the sequence of one of the most frequent behaviour of the trusses when the dissolution of a main node occurs.

high ratio humidity of the setting or of the walls, especially those brought by fungi that start a decay of the materials and reduce and almost nullify the strength of the wood, trigger a failure that is characterized by a solution of continuity of the heel along the fibres and by the sliding of the heel and of the rafter along the tie. The error of making hollows in the members in order to accomodate a concurring member was very diffused; typical in this sense those made at the lower edge of the rafters to house the end of the struts. The execution of the hollows cuts the compressed fibres and reduces the depth of the reacting section. Cracks in the section occur frequently (Fig. 9).

The correct design, according to a long tradition, provides instead for the fitting under the rafter with a timber bracket that accommodates the top of the strut that would diffuse the action exerted by this member in a large volume of wood so as to preserve anyway the integrity of the rafter and fix the strut. The application of cleats is another possibility of a technically adequate solution. In the practice it is rather frequent to meet into cases of connections made purely superposing one member on the other. This kind of defective assembly relies on the gravity and on the friction between the overlapped surfaces for the hindrance to sliding of the rafter along the tie (St Catharina's church roof in Borgo San Lorenzo, Florence, Renaissance period); sometimes (St Monica's church roof in Florence, late Middle Ages) the opposition to sliding is given by the thin masonry diaphragm that closes the housing of the truss node. In both cases the occurrence of failure is an easy prediction.

The severe consequences of these occurrences are a combination of rotation and translation of the rafter, disconnection of the strut, shirking of the rafter's duties, weighing of the whole loads of the roof on the other rafter with hard bending of the tie beam directly charged by the post, rotation of the joists. The described process is generally slow, also because when the problem starts the other members and units of the structure, provided that they are efficaciously connected, carry out actions of cooperation with the failing elements and joints.

At the end of the process the truss becomes unserviceable (Figures 8, 12, 13).

2.3. Lack of three-dimensional conception of the structure

An error rather frequently recurring in the timber constructions as well in the iron structures of the 18th, 19th C. is the absence of three-dimensional conception of the structure. It is often associated with slenderness of the members.

It occurs when a building is not sufficiently stiff in all directions; concurring cause of the consequent possible rotation is the incapability of the joints to withstand thrusts (weakness of the bracings) (Fig. 13). It is not rare still nowadays in constructions of any material.

In a roof this error and the consequent unsteadiness occur when the three-dimensional structure is simply conceived as an addition of beams or trusses that have no adequate connection along the main axis of the building, in other words when the members and the units that compose the structure are not organized in a system. Instability can anyway occur when the auxiliary members (purlins, top ridge, joists, boarding) of a covering, for instance, do not connect well the structural units (trusses, frames). The risk is buckling of a few units, piling up of the units.

In a hip-roof, for instance, the task of stability is automatically reached because the pitches along the short sides prevent any rotation of the trusses on their supports except if one of them fails.

2.4. Unsuitable structural scheme (configuration)

Specifically, it was a very general rule in Italy and elsewhere, at least until the 19th C., the practice of suspending the false ceilings of the roofs to simple beams that, in their turn, were placed on the tie beams of the trusses, rather than being suspended to the rafters.

When in the mature Renaissance and Mannerism in Italy the false ceilings started to be huge, that is very massive and overcharged with decorations, therefore very heavy, the usual scheme of the simple king post with two struts truss became inadequate to support roof and false ceiling of con-

Figure 14. Rigid rotation of one of the bodies of a timber building in Myanmar (from Tampone, Atlas, 2016, cit.).

Figure 15. Rigid rotation of the wall of the side nave (Tampone, Atlas, 2016, cit.).

siderable span; except in presence of very deep tie beams or stiff trusses, the tie beams and the same trusses sank and the ceilings underwent settlements. All the false ceilings of this kind up to the 19th c. are affected with this error with the cited consequences.

The full demonstration of what just said is offered by several cases such as the roof and false ceiling of the Sala Nova in Florence and the roof and the late false ceiling of the Saint Mark's church in Florence.

In the first case the construction of the whole complex, i.e. structure, over structure and false ceiling, was planned and directed by Giorgio Vasari from 1563 to 1565 (Tampone G., Derinaldis P.P., 2005, cit.). The trusses of classical type, i.e. with a king-post and two struts, the tie beam being

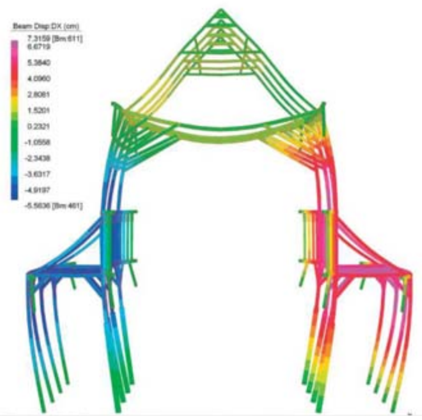


Figure 16. FEM general scheme of deformation of the Paramaribo's cathedral (Tampone, with Derinaldis, Atlas, 2016, cit.).

composed by two pieces, cover a span of almost 25 m, a record for the time. The trusses were designed to carry both the loads of the roof and of the false ceiling but the latter was suspended to the tie beams rather than to the rafters.

The trusses underwent sensible deformations, as is witnessed in the following centuries by numerous reports that describe a situation of uneven progressive deformation of the structural complex and especially of the tie beams of the trusses and of the false ceiling (Tampone, Derinaldis, 2005, cit.).

The problems, after a plethora of local measures and a few unsuccessful general attempts carried out over the time, were brought to satisfactory solution only in the half of the 19th C. when additional very stiff trusses, appointed to carry only the weight of the false ceiling, were built in the mid-bay of the original trusses. These were designed with the duty to bear only the weight of the roof.

It ought to be remembered that the previous false ceiling in the same room, made in 1495 by the prominent architect Simone del Pollaiolo detto il Cronaca, placed some 10 m lower than the current one, it too was suspended to the tie beams of the trusses but these were designed with three king-posts and equipped with considerable iron fittings therefore very stiff and suitable for the purpose.

No nuisance is recorded in the seventy years circa of its existence, before the dismantling of the whole roof operated by Vasari.

A similar situation is to be found in the Saint Mark's church in Florence but there the false ceiling was built only in the 18th C. and was suspended, as usual, to the tie beams of the original classical king post trusses. The latter, built at the beginning of the 14th C., support the roof and cover a span of 17,5 mt.

Here too several problems arose with the inflexion of the tie beams and deformations of the ceiling. They were brought to satisfactory solution only some twenty years ago. In consideration that the trusses were well dimensioned and suitable anyway to carry also the additional load of the false ceiling, the weight of the latter was trusted, following the author's suggestions, to the rafters by means of steel cables.

On the contrary it is not easy to find today ancient timber carpentries that present an unsuitable structural configuration because the erroneous specimens collapsed or were corrected or modified in order to allow them to stand up and perform the designed duty.

Intact on the contrary though widely propped (before strengthening) was the carpentry of the roof and false ceiling of the cathedral of Paramaribo, Suriname, Caribbean Region, built entirely in timber in 1878 by a Franciscan friar who was an experienced carpenter (Tampone, 2016, Atlas, cit.; Tampone, 2017, cit).

The roof includes a false ceiling constituted by very heavy timber cross vaults, built according to the "de l'Orme" system and made by boards. Two fundamental errors occurred in this case and a few minor ones.
First the thrusts of the vaults were expected to be absorbed by the pillars that divide the central nave by the side ones, the roofs of the side naves working as buttresses. But the pillars are too thin and the side naves are not wide and heavy enough to perform an effective shoring action. The trusses underwent deformations, the roof with the vaults sunk, the vaults widened.
Second, since the vaults are also suspended to the tie beams of the trusses of the roof, the severe deflection of the long and thin ties caused the inflexion of the main pillars that are loaded by struts that stiffen the joint between the two members (Fig. 14).
The facades of the side naves were affected with a visible considerable rotation that is more accentuated in the median regions of the walls (Fig. 15).
But a third error concurred to cause the general failure, the insufficient size of the timber columns (round sections of 30 cm at the base) made of an only piece and tall, from the ground to the roof, 14,60 m circa.
The considerable deformations were also fostered by the setting humidity ratio, very high in the whole region, and by severe attacks of termites.
The node tie beam-rafter of a generic truss performs at least two essential roles, to connect the cited members as components of the structural unit, the truss, that is deputed to carry the own weight of the structure with the external loads as well to transmit, with the symmetrical ends, all the loads on the external supports. In other words it plays the role of internal and external constraint. The design of such a node is mainly based on traditional patterns depending very little on the scheme of the truss.
The connection of the two members, generally of the type mortise (in the tie) and tenon as said, is already documented in the oldest trusses known so far, those of the roof of the Saint Catharina church in the Sinai peninsula, Egypt, ordered by the byzantine emperor Justinian in the half of the 6th C. The connection, very well designed, works perfectly (Katsibinis, 1983, cit.).

Figure 17. Sliding of the rafter due to rupture of the heel of the tie beam of a truss. The size of the sliding is evident in the picture (18 cm circa). The two collar ties are not correctly placed and do not work efficaciously (from Tampone, Atlas, 2016).

Several trusses built in the past show the presence of iron collars in order to avert this risk. The collars put in place by Bernardo di Monna Mattea for the trusses supporting roof and false ceiling of the roof designed by Giorgio Vasari for the Sala Nova in Palazzo Vecchio, cited before, are splendid ironmongery works not only provided with adjustment organs but also placed inclined in order to be active, i.e. ready to prevent sliding of the rafter along the tie just when the motion is starting, not later. This fundamental lesson was forgotten by many designers of the following times who placed the collars perpendicular to the upper edge of the rafter. The inefficacious placement is repeated in the handbooks of the 19th and 20th centuries. Sometimes the short nails put at both faces of the tie and rafter to pin the rafter up and prevent its motion were very short and not effective (Figures 7, 16).
Similarly in the half halved joints the strength of the two members, generally jointed by means of a wooden peg, is cut down to half of the original with an increased danger of instability for the compressed elements. Furthermore the peg can start a splitting crack in the weaker member. The consequences of malfunctioning of the main node of a truss may become severe as explained.
Frequent errors, for the risk of burns or fire, are the application of electric wires or lamps to the members of a timber carpentry and similar.

3. Conservation

Finding errors in ancient carpentries, especially design or construction, generally gives embarrassment to those who today express a judgment why it shows the limitedness of the notions as well their deficient dissemination at the time of the design or implementation or both, with regard to the current knowledge.
It ought to be noted that the adoption of insufficient sizing of the members included the cited shortness of the rafters of the trusses, is sometimes to be considered as a conscious decision for constructions of minor importance that were not planned to last over a long time or when money saving in the construction of a building was a must. Sometimes it was a fraud. Little consequent inconveniences were therefore expected and accepted.
A general problem of conservation is: When planning the conservation measures for timber carpentries that present peculiarities or errors like those exposed here, what conservation principles and strengthening measures should be adopted? Should a defective con-

Figure 18. The burn at the tie beam of a truss in the Fencing hall of Fortezza da basso, Florence (Tampone, 1989, cit.).

figuration be modified completely in order to recover an efficacious and, at the same time, safe structural behaviour?
In general, instead of correcting the errors, it is advisable to take measures capable to limit their effects and consequences.
The principle of the wise use of the buildings and their structures, if necessary with drastic limitations, and the theory of the permanent propping open several possibilities. In support of these assumptions one can quote the several applications of this principles satisfactorily carried out on ancient timber structures in the last decades, that were often inspired by the strengthening works made from the 19th C. on archaeological monuments for which the requirement of the conservation of the authenticity is more rigorous.
Instability can be controlled by means of buttresses, tension cables or other devices. Insufficient size can be corrected by means of additional members or devices (application of external fish plates, mainly of timber, insertion of internal steel plates etc.) (Tampone, 1996, cit.), paying attention that the effects of the operation should not spread in the other components of the units, affecting them, and not modify the behaviour of the same units and of the whole system. There is the risk, anyway, of adding new errors….

4. Conclusions

It ought to be considered, anyway, that a great deal of timber structures affected with errors stood the test of time, sometimes very extended, proving their vitality.
When analyzing an existing timber structure still standing, the presence of deformations and cracks, that are generally very evident, allows to form an opinion of its condition. The classification of the cracks according to type and extension may allow to make preliminary assessments on the main causes of disease. The mechanical actions impress the general features to the cracks.
The study of the deformations and stresses of the loaded timber structural systems and their members is fundamental to assess type and severity of the mechanical failures. As a consequence, Finite Element Method applications, that show quite well these elements under given loads, are very suitable for the design of the fixing works and prediction of the effects they would produce on the structural system; this will allow, on heuristic bases, to select the most appropriate technical solution.

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Numerical analysis of multi-drum columns: The case study of the two-storey colonnade of the ancient Forum in the archaeological site of Pompeii

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Background

High-intensity earthquake excitations have damaged several ancient monuments, such as classical columns and colonnades of great archaeological significance, which can be found in high seismicity areas in the Eastern Mediterranean such as Greece and Italy. Multi-drum columns are constructed of stones that are placed on top of each other, usually without connecting material between the individual blocks. The seismic behaviour of these structures exhibits rocking and sliding phenomena between the individual blocks, which very rarely appear in modern structures. Specifically, the drums may rock either individually or in groups resulting in several different shapes of oscillations.

The understanding of the seismic behaviour of ancient columns contributes to the rational assessment of potential proposals for their structural rehabilitation and strengthening, while it may also reveal some information about past earthquakes that had struck the respective region. As ancient monuments have been exposed to large numbers of strong seismic events, throughout the many centuries of their life spans, those that survived have withstood a natural seismic testing that extended several centuries. Therefore, it is very useful to understand the mechanisms that allowed them to avoid structural collapse and destruction during several strong earthquakes. Since analytical studies of such multi-block structures containing large number of drums and subjected to strong earthquake excitations is practically impossible, while laboratory tests are very difficult and costly to perform, numerical methods (Psycharis et al. 2003; Pompei et al. 1998; Makris and Zhang 2001; Maonos et al. 2001; Mitsipoulou and Paschalidis 2001) can be used to simulate their dynamic behaviour and seismic response.

Today, the most widely used approach to model the mechanical behaviour of masonry structures is the Finite Element Method. Although the Finite Element Methods (FEM) can be used for the analysis of problems with some discontinuities, the FEM are not suitable for the analysis of discontinuous systems that are characterized by continuous changes of the contact conditions among the constituent bodies. On the contrary, Discrete Element Methods (DEM) have been specifically developed for systems with distinct bodies that can move freely in space and interact with each other with contact forces, providing an automatic and efficient recognition of all contacts. Research efforts to use the DEM in simulations of ancient structures have already shown promising results, indicating a potential for further utilization of this method. Recent research work based on commercial DEM software applications (Papantonopoulos et al. 2002; Mouzoukis et al. 2002; Komodromos et al. 2007; Sarhosis et al 20015; Sarhosis et al 2016; Sarhosis 2012; Sarhosis and Sheng 2014), demonstrated that the DEM can effectively be used for the analysis of such structures.

This article presents the results from a numerical model developed by DEM to investigate the seismic vulnerability of a block-based frame of architectural heritage of the ancient city of Pompeii in Italy, aiming to gain a better understanding of its overall structural behaviour.

The archaeological site of Pompei

The archaeological site of Pompeii is a Roman town located near Naples in Italy. Pompeii was a busy commercial hub between Neapolis (Naples), Nola and Stabiae (Castellammare di Stabia); three main cities in southern Italy. The town was destroyed during a catastrophic eruption of the Vesuvius volcano in 79 A.D. As a result of the eruption, the city was mostly destroyed and buried under 4 to 6 m of ash pumice.

The site was covered for about 1,500 years until its initial rediscovery in 1599 and a broader rediscovery almost 150 years later by a Spanish engineer in 1748 (Maiuri 1942). The ancient city of Pompeii has been preserved throughout the centuries due to a lack of air and moisture and preserved from earthquakes, thus providing a complete picture of the city as well as of the daily life at that period. However, since its excavation, Pompeii is experiencing many events endangering its conservation. For instance, during the Second World War the Forum area was stroke by a bomb suffering severe damages. Nowadays, the ruins of the ancient town show many partially collapsed buildings.

Since 1997, the ancient city of Pompeii has been recognised as a world heritage site by UNESCO. Today, Pompeii is one of the most popular tourist attractions in Southern Italy, with millions of visitors every year. During its lifetime, the city suffered many earthquakes. Before the catastrophic eruption, in 62 A.D., a strong catastrophic earthquake hit the town. However, no reconstruction works concluded. After its rediscovery, in 1749, several other earthquakes hit Pompeii. The last earthquake to hit the city and to produce damages in the whole archaeological site was in 1980. Also, weathering, erosion, light exposure, water damage and poor methods of excavation and reconstruction have resulted towards the deterioration of the archaeological structures in Pompeii (Bergamasco 2012).

Building practice in ancient Pompeii

During its lifetime, the city underwent many earthquakes. "Innovative solutions" in the building practice at that time were conceived to improve the seismic performance of structures; which is evident from the execution speed and the overall economy in the reconstruction works after the earthquake of 62 A.D. Analysing the partially collapsed structures, constructive wisdom can be foreseen in the structural details conceived by ancient builders in Pompeii. For example, an analysis of the pillars of the Basilica in the Forum shows that mortar joints were aligned with an unprecedented staggered pattern able to increase the seismic performance of the structure (de Martino et al. 2006). Also, the dimensions and the proportions of the Basilica were large and the realization of the brick tall pillars of the nave has been interpreted as the necessity of rapid execution of works with particular attention to the overall economy of the construction. This technique, widely used in the roman architecture, has never found such a peculiar model for the brick elements, almost a unicum in which aesthetics and techniques are joined perfectly.

Description of the monument

The structure under investigation (Figure 1b) is a two-storey colonnade of the Forum in Pompeii (Figure 1a) made of white limestone. The colonnade was erected in the

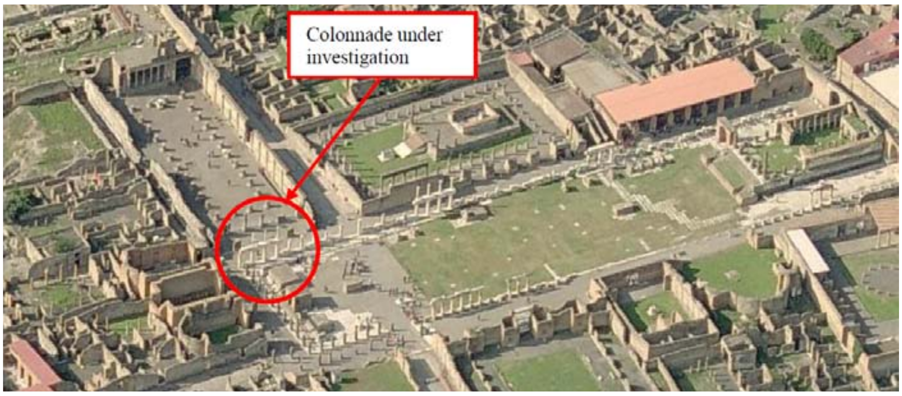


Figure 1. The forum of Pompeii: (a) Aerial view of the southern part of the Forum showing the colonnade under investigation inside the circle; (b) Two storeys colonnade with multi-drum columns and multi-blocks segmented trabeations.

main square of the ancient town where political, economic and religious events were taking place. To prevent the passage of the carts, the pavement was raised with respect to the height of the adjacent roads by means of two steps. It is believed that originally (VII-VI century B.C.) the Forum had an irregular shape. Today, the Forum has a rectangular shape with dimensions 143 m long by 38 m wide. Probably, at the end of the II century B.C., a double row colonnade was built which was made of tuff. Some years later in 79 A.D. the colonnade was reconstructed and the tuff was replaced with white limestone. The columns of the second storey followed the Ionic order while the columns of the lower storey followed the Doric order. In the second half of the 20th century, only a small part of the second storey was re-erected for educational and touristic purposes. In 1980, an earthquake produced damages in the colonnades of the Forum and it was decided to remove the beam over the second storey.

An "innovative solution" was adopted for the construction of the trabeation. To avoid long span beams over the columns, short segments were built up providing opposing inclined patterned edges "flat arch" (Figure 2a). This solution was conceived to simplify construction phases and prevent the lifting of long span heavy beams over the columns. Blocks mutually supporting each other over inclined surfaces (keystone) induced horizontal thrust in the horizontal structures to carry loads without any tensile strength. In a fully functioning structure, each keystone pushes over the two adjacent blocks and this load is counteracted. The static problem may arise at the corners of the structure, where there is no symmetric mutual interaction and could lead to the overturning of the column. To overcome the above hurdle, the builders avoided the reduced size blocks at the end of the colonnade. Instead, a long block (solid long beam in Figure 2b), supported by the first two extremity columns has been placed. In that way, the horizontal thrust, not counteracted by the contiguous blocks, missing, was counteracted by two columns, so halving the horizontal thrust. The examination of the methods employed by the ancient builders revealed the continuous research and evolution in design of structures against earthquakes (Adam 1989).

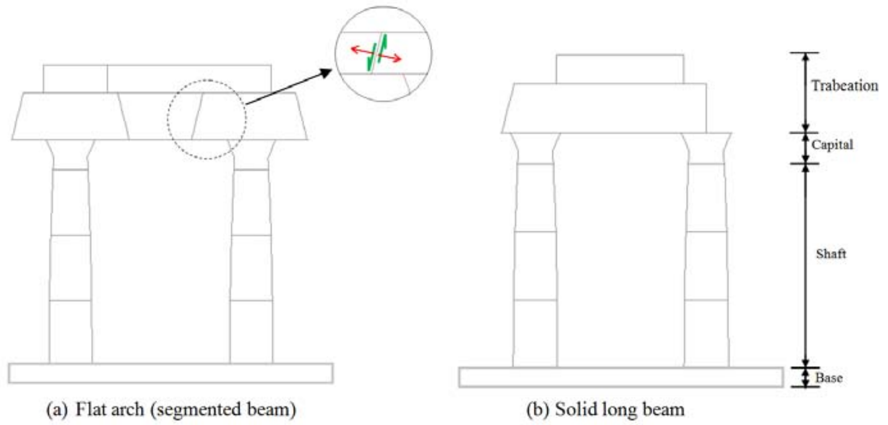


Figure 2. Different construction methods at the trabeation of the Forum in Pompeii.

Development of the computational model

Geometric models representing the two-storey colonnade of the Forum in Pompeii were created in the computational model (Figure 3). Each stone of the monument (i.e. drum of the column and stone block of the trabeation) was represented by a deformable block separated by zero thickness interfaces at each joint. The zero thickness interfaces between each block were modelled using elastic perfectly plastic Coulomb criterion defined by: the elastic normal stiffness (JKn); the shear stiffness (JKs); and the joint angle of friction (Jfric). The material

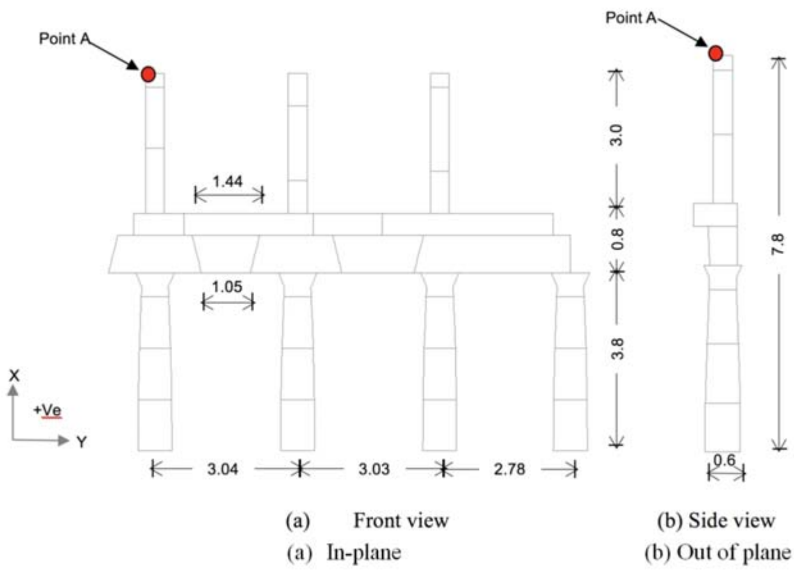


Figure 3. Geometry of the two-storey colonnade of the Forum of Pompeii,

Table 1. Properties of the limestone blocks

Unit Weight d [kg/m³]	Young Modulus E [GPa]	Shear Modulus G [GPa]	Bulk Modulus K [GPa]	Poisson's Ratio v [-]
2680	40	16	27	0.25

Table 2. Properties of the joint interfaces (Kastenmeier et al. 2010 and Angrisani et al. 2010)

Normal Stiffness JKn [GPa/mm]	Shear Stiffness JKs [GPa/mm]	Joint friction angle Jfric [degrees]	Joint tensile strength Jten [MPa]	Joint cohesive strength Jten [MPa]	Joint dilation angle Jdil [degrees]
4	2	14° to 36.8°	0	0	0

parameters used for the development of the computational model are shown in Tables 1 and 2. Also, a sensitivity analysis on the frictional performance of joints was undertaken. The joint friction angle varied from 14° to 36.8° degrees. This was to simulate potential joint degradation effects and/or possible water lubrication at the joint.
The bottom edges of the monument were fixed. Self-weight effects were assigned as gravitational load. At first, the model was brought into a state of equilibrium under its own weight (static gravity loads). Then, external loading has been applied to the structure by increasing horizontal accelerations (non-linear pushover analysis). Horizontal displacements at the upper part of the colonnade (ref. Figure 3, Point A) have been recorded.

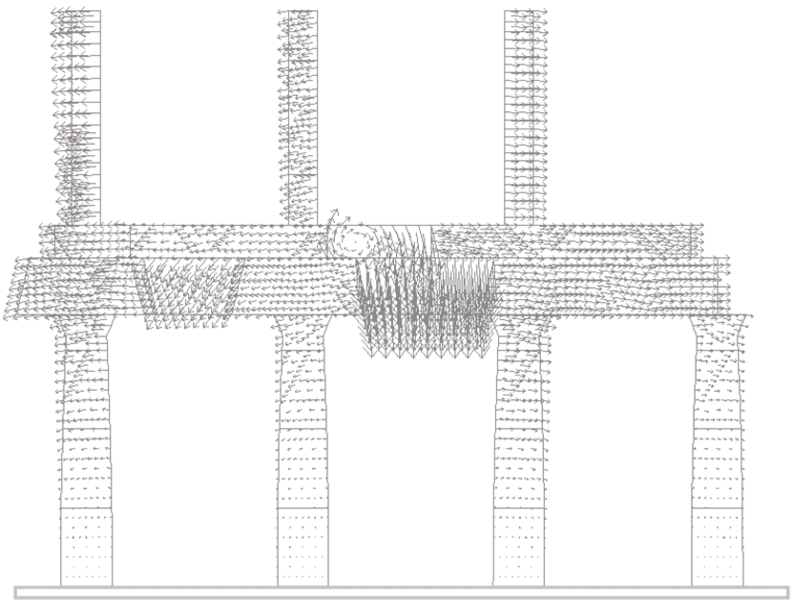
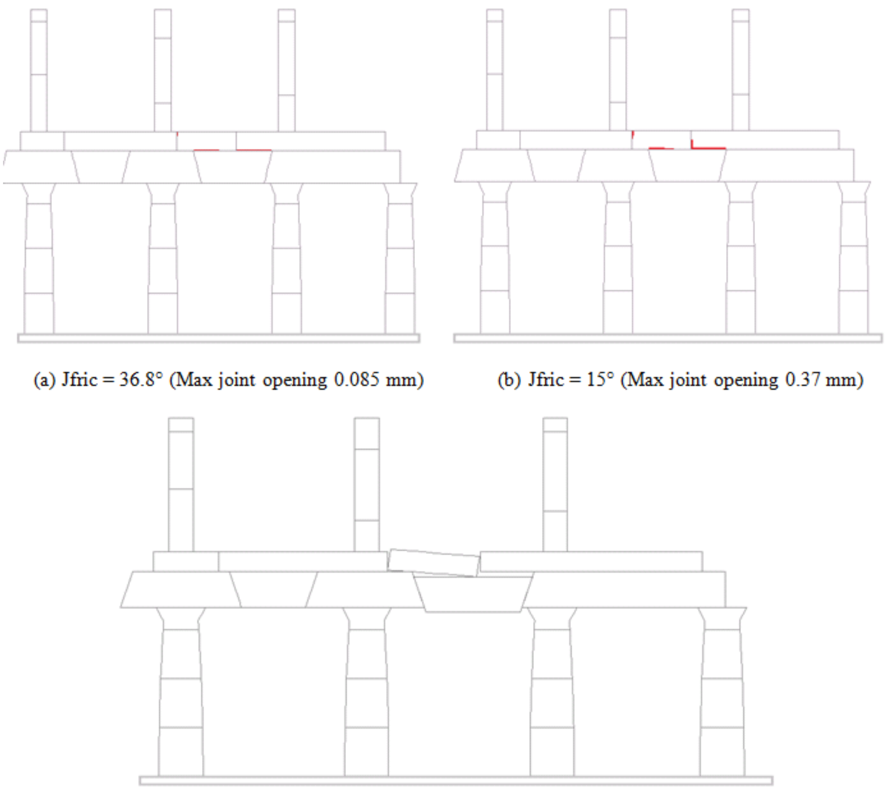


Figure 4. Displacement vectors (Jfric=15°).

Response to static loading conditions

Since columns adjacent to the structure have collapsed and thus the continuity of the "flat arch" beam above the columns has been lost, mechanisms can develop between inclined faces of the blocks which provide thrust forces and hence sliding of the blocks and opening of the joints. To simulate joint degradation, a sensitivity study undertaken where values of the joint friction angle ranged from 14 to 36.9 degrees. From the results of the analysis, it was found that sliding of blocks and opening at the joints increases as the joint friction angle decreases. Also, stability of the historical structure can be granted for values of friction angle greater or equal to 15 degrees, hence such value was used from this point further.
Figure 4 shows the displacement vectors in the colonnade when the joint friction angle is 15 degrees. The displacement vectors clearly show the opening of the columns in the lower storey which is a result of the thrust of the central blocks of the segmented beam. The block at the central bay slides down and pushes the contiguous blocks apart. Also, due to the sliding, the upper block tends to rotate and further pushes down the former. The upper storey columns just follow the sliding of the blocks of the beam. From the above, stability in this case is mainly a geometrical problem since the loss of support is the reason of collapse. On the other hand, the stress level at the limestone blocks is negligible.
Figure 5 shows the location of the



(a) Jfric = 36.8° (Max joint opening 0.085 mm) (b) Jfric = 15° (Max joint opening 0.37 mm)

(c) Jfric = 14° (collapse state)

Figure 5. Joints opened in the structure.

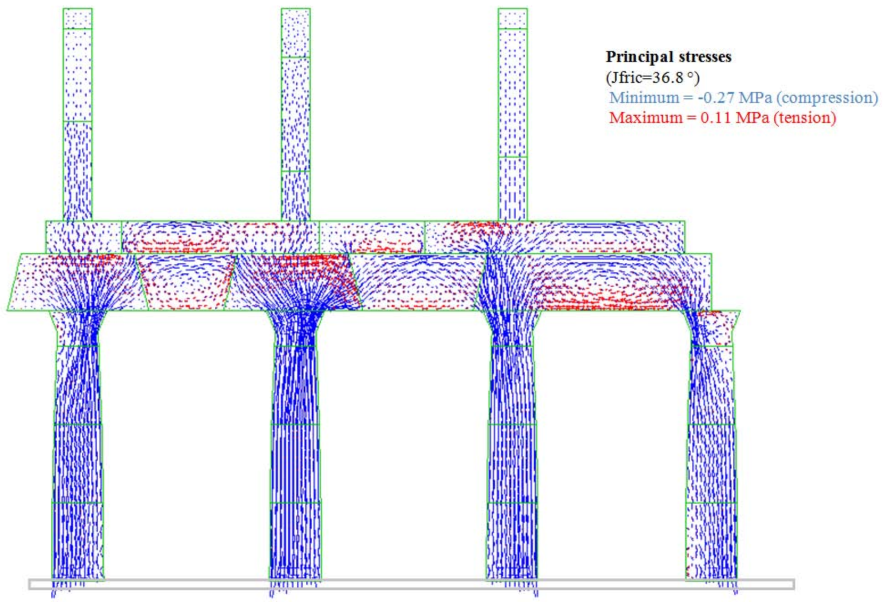


Figure 6a. Principal stress distribution for different values of joint friction (tensile: red, compression: blue).

joint opening for different values of friction angle. For a joint friction angle equal to 36.8 degrees, the maximum opening at the joints is in the order of 0.085 mm (Figure 5a). For a joint friction angle equal to 15 degrees, the maximum opening at the joints is 0.37 mm (Figure 5b).

When the joint friction angle is 14 degrees or below, equilibrium of the colonnade cannot be granted and failure occurred (Figure 7c). This is due to the fact that sliding of the short blocks over the two columns on the left is not able to hold in place the central block which is flowing down. It is worth mentioning that the angle of the inclined faces of the segmented blocks in the trabeation is also 14 degrees

Figures 6a and 6b show the principal stress distribution when joint friction angle is 36.8 and 15 degrees accordingly. From Figure 8, stress intensities are reducing as the joint friction angle increases. When the joint friction angle is 36.8 degrees, the maximum principal compressive stress in the structure is 0.27 MPa and the minimum principal tensile stress is 0.11 MPa. Similarly, when the joint friction angle is 15 degrees, the maximum principal compressive stress in the structure is 0.54 MPa and the minimum principal tensile stress is 0.24 MPa. Also, for the continuous beam (left part of the structure), high tensile stresses occurred at the bottom of the beam while compression occurred at the top of the overlapped elements. Conversely, the left layout provides a "flat-arch"; hence having almost negligible tension and widespread compression stresses. Since joint degradation implies a mechanical reduction of properties, this means that both joint opening and stresses tend to increase with this phenomenon; hence aggravating it.

Non-linear equivalent static (push over analysis)

In order to evaluate the seismic vulnerability of the structure, a non-linear static analysis has been performed. The analysis was undertaken under constant gravity loads and uniform monotonically increasing horizontal accelerations along the height of the structure until the structure no longer is in equilibrium (e.g. in-plane loss of stability due to sliding and/or rocking failure of the blocks). Based on the ITACA (Luzi 2008) seismic database, peak ground accelerations have been ex-

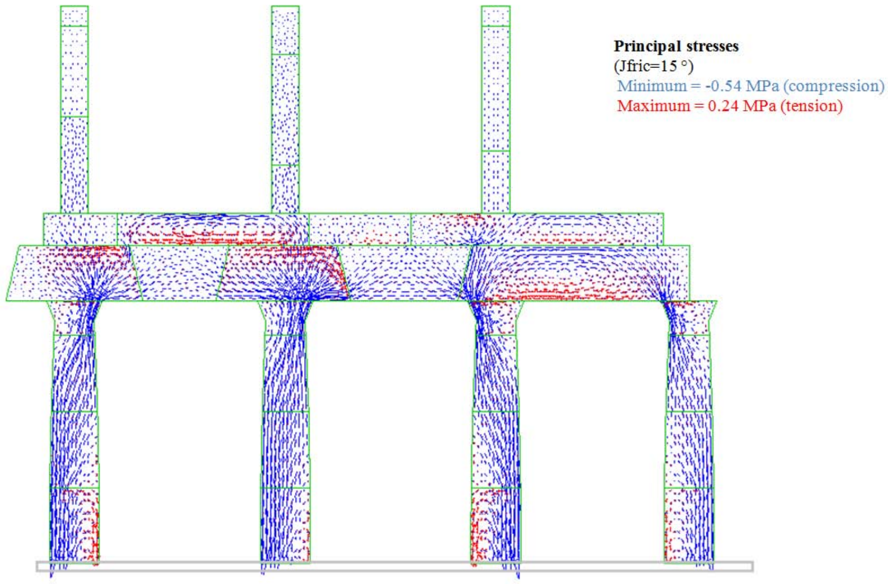


Figure 6b. Principal stress distribution for different values of joint friction (tensile: red, compression: blue).

Table 3. Values of frequencies and periods for the first six modes for the in-plane and out-of-plane analysis

Mode	In-plane		Out-of-plane	
	Frequency [Hz]	Period [s]	Frequency [Hz]	Period [s]
1	18.1	0.055	7.40	0.135
2	25.3	0.040	27.4	0.036
3	25.5	0.039	71.5	0.014
4	27.6	0.036	104	0.010
5	104	0.010	159	0.006
6	111	0.009	202	0.005

pressed in terms of probability of exceedance in a 50 year return period according to Eurocode 8. Preliminary modal analyses were undertaken and the dynamic response of the colonnade in its elastic phase was obtained by means of preliminary numerical investigations (see Lignola et al. 2014). The free vibration problem was analysed at the in-plane and out-of-plane direction and the higher modes involved almost negligible participating mass. The first three modes involved the translation in the longitudinal direction and the other three in the out of plane direction. Table 3 shows the natural frequencies and periods for the first six modes of the response of the colonnade in the in-plane and out-of-plane direction. Clearly the natural period is higher for the out-of-plane direction, where the structure is slenderer; hence prone to overturning. Conversely, the first natural period in the in-plane direction is much smaller, because the structure is much stiffer in that direction. Nevertheless, higher periods are almost comparable in both in-plane and out-of-plane directions. To improve the knowledge about the seismic vulnerability of the structure, it is crucial to include the nonlinear behaviour of the interfaces and especially their 'no tension' behaviour.

The seismic vulnerability was undertaken according to Eurocode 8 (part 3 §4.4.4.2) by means of a non-linear static (pushover) analysis under constant gravity loads and monotonically increasing horizontal loads at the location of the masses in the model. The relation between base-shear horizontal force and the displacement at a control point (ref. Figure 3, Point A) gives the "capacity curve" which provides the characteristic force against displacement relationship of the Multi-Degree-Of-Freedom (MDOF) system. Since the structure is not symmetric, both positive and negative lateral accelerations were considered. The structure was pushed (positive direction) and pulled (negative direction) up to failure i.e. when equilibrium was no more obtained. The equivalent Single-Degree-of-Freedom (SDOF) system was determined according to Eurocode 8 (Annex B) assuming a bilinearized curve providing the idealized elasto-perfectly plastic force against displacement relationship of the SDOF system. In the analyses, the initial stiffness of the idealized system is the same as that of the model while the yield force of the idealized system is determined in such a way that the areas under the actual and the idealized force against deformation curves are equal.

Figure 7 shows the curves of the SDOF systems in terms of acceleration and displacement. Each colour refers to a different friction angle. Also, thick lines represent the actual capacity curve while thin lines represent the corresponding idealized bilinear curve. Similarly, the seismic demand has been expressed in terms of an equivalent SDOF system. A spectral displacement demand represents a target displacement for the structure. For structures in the short-period (i.e. lower than upper

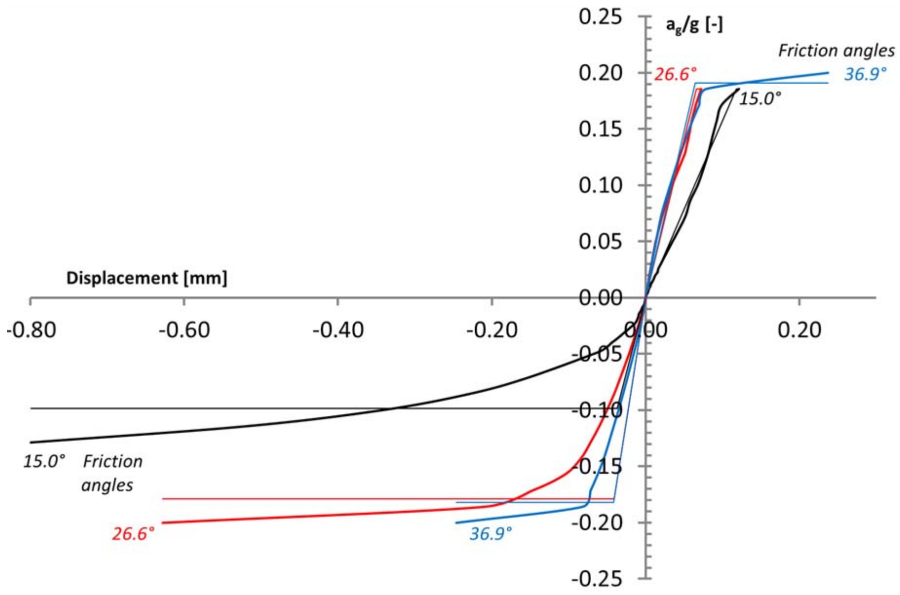


Figure 7. In-plane pushover analysis in positive and negative directions and bilinearization (thin line).

corner period of the constant acceleration region of the elastic spectrum) the elastic spectral demand is properly increased, while for structures in the medium- long-period the target displacement is the displacement of the elastic response spectrum (Eurocode 8). Finally, for the structure to be safe, the displacement capacity should exceed the displacement demand. Assuming different friction angles for the joints, the behaviour is clearly different and the variations are evident for each direction.

Horizontal in-plane actions yield to dissimilar behaviours depending on the friction angle of the joint. In the positive direction (left to right), low values of joint friction angle (i.e. 15°) yield to a linear behaviour up to failure, while for high values of joint friction angle (i.e. 36.9°) a short nonlinear plastic behaviour is observed, being the strength almost the same (Figure 7). Yielding according to the ideal bi-linearized curve occurs at an acceleration slightly lower than 0.20g. In the negative direction (right to left), even though the strength is almost similar (when friction angle is 36.9°, the maximum acceleration is also close to about 0.20g); the nonlinear behaviour is remarkably different. A large amount of ductility is observed, being the maximum displacement almost three to five times higher than in the positive direction. Figure 11 shows the pushover curves when joint friction angle is 26.6°. According to the adopted performance evaluation, in each case, the structure is able to resist an expected earthquake if the displacement capacity exceeds the displacement demand.

Dotted vertical lines represent the SDOF target displacements for different levels of ground motions (expressed in terms of return period or the probability that an event will be met or exceeded during an interval of 50 years).

The colonnade is able to withstand a design earthquake with a return period of 200 years, or 22% in 50 years, both in the positive and negative directions, and this performance is provided being the structure still in its elastic branch (Figure 8). In this sense, the behaviour curves are overlapping in the two directions in the elastic branch. Conversely, in the negative direction, where the structure exhibits a ductile behaviour, the spectral displacement is properly amplified to account for the limited strength of the colonnade, hence accounting for its displacement capacity after yielding in the nonlinear field.

Figure 9 shows the highly magnified deformed shape; it clearly shows the development of the failure mechanism: central blocks of the beam lose their support and sliding occurs, yielding to the in-plane failure condition. The mutual rocking and sliding of the blocks is

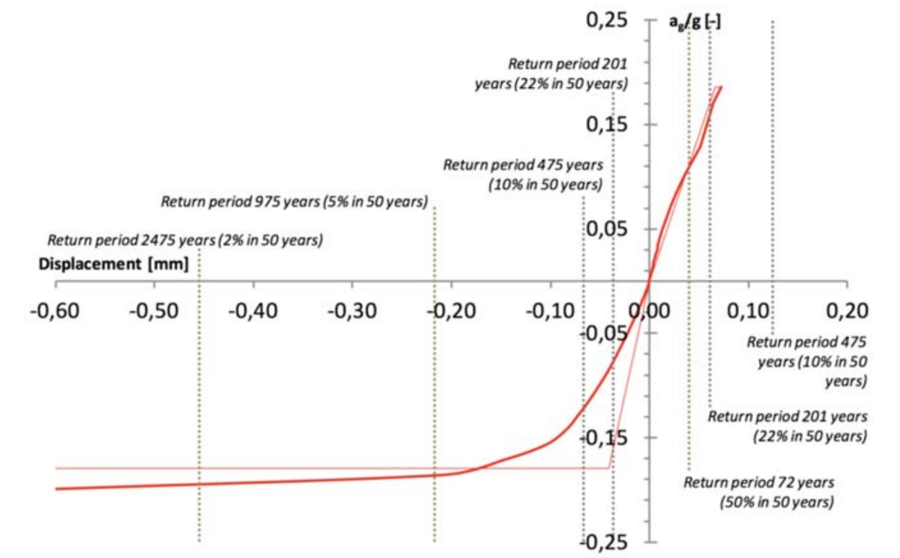


Figure 8. Comparison of the bilinearized curve (average friction angle 26.6°) with expected seismic demands (vertical dotted lines).

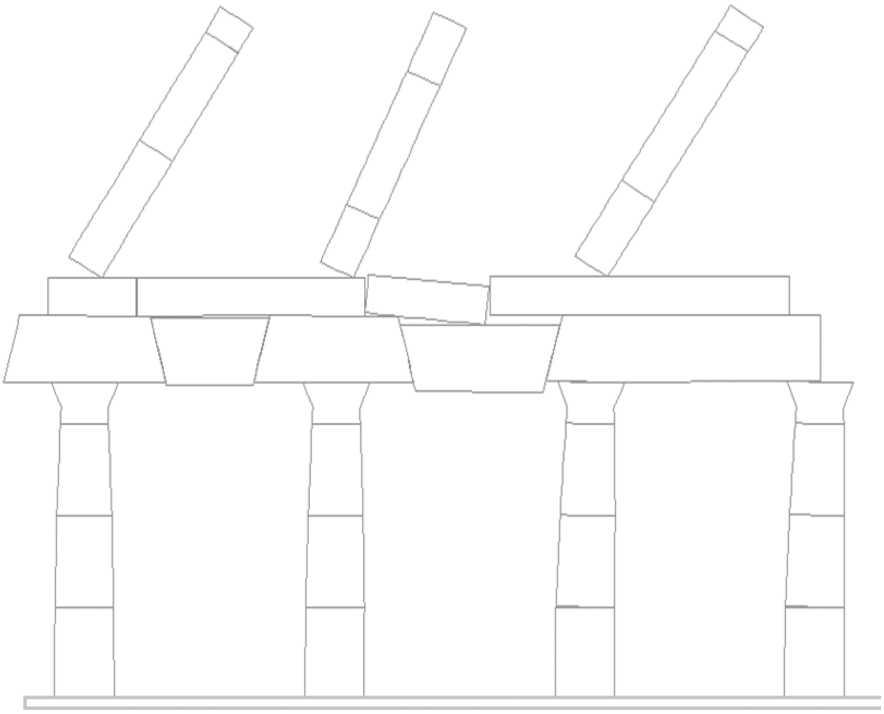


Figure 9. Magnified (x200) deformed shape, seismic in-plane condition (ag=0.15g).

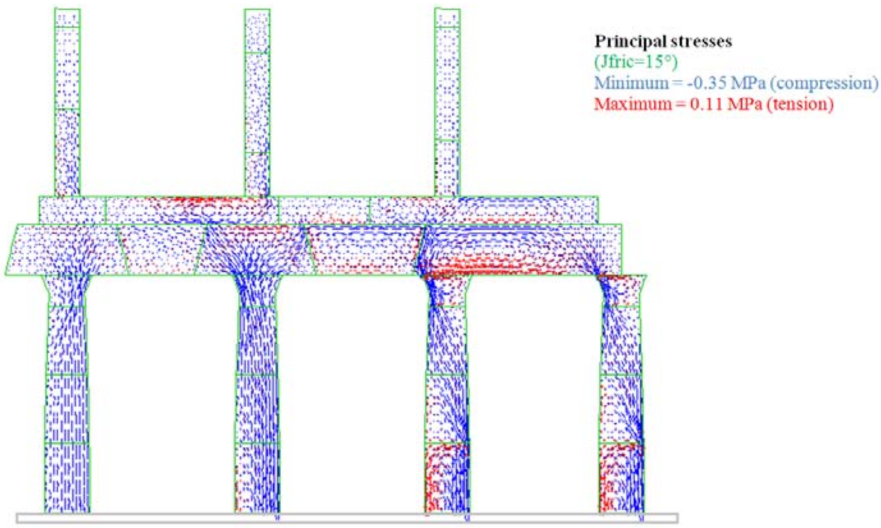


Figure 10. Contour plot of principal stress directions at peak load.

counteracted by the long beam lying on two columns in the positive direction; the long beam represents a stiffer restraint compared to the negative direction where all the small blocks are less constrained in their displacements because they are smaller and each of them lies on a single column, hence the articulated system is globally more deformable and horizontal thrust balance not symmetric.

The effect of seismic accelerations at the peak force is shown in the stress contour plots in Figure 10. From the point of view of stress levels, compression stresses are lower than 0.8 MPa (in all the analysed cases). Even though they exceed the stresses under static conditions, they are still lower than expected material strength, so not causing particular concern. Similarly tensile stresses are lower than 0.4 MPa.

This means that no internal block failure is expected, but sliding and rocking each other.

Conclusions

This article presented the results from a numerical model developed by DEM to investigate the seismic vulnerability of a block-based frame of architectural heritage of the ancient city of Pompeii in Italy, aiming to gain a better understanding of its overall structural behaviour.

The two-storey colonnade of the Forum in Pompeii was erected in the main square of the ancient town and was made of white limestone. An "innovative solution" was adopted for the construction of the trabeation. To avoid long span beams over the columns, short segments were built up providing opposing inclined patterned edges "flat arch". It is believed that this solution was conceived to simplify construction phases and prevent the lifting of long span heavy beams over the columns. The blocks mutually supporting each other over inclined surfaces induce horizontal thrust in the structure. Each block pushes over the other two contiguous blocks and this load is counteracted. However, at the corners of the colonnade, where there is no symmetry, stability becomes critical. Such thrust potentially could overturn a single extremity column. As a solution to this, the builders at that time used large blocks at the end of the colonnade to allow for the required horizontal thrust balance. Another point related to the static condition of the structures, relates to the potential swing of the blocks. In particular, the first block, supported over two columns, can be considered as fixed, then a portion of this first block overhangs and works as a suspended support for the contiguous block. The block in between two columns is simply supported. However, the block on the subsequent column is not fixed like its previous one, since it is freely supported by the column, so it can potentially swing. In order to prevent the potential swing of the blocks, a second row of smaller, but longer blocks was simply placed on the blocks with inclined sides

Joint openings that occur are considered to be detrimental for the structure. Joint openings may lead to water leakage and lubrication of the potentially sliding planes. From the results of the analysis, if the joint friction angle is 14 degrees, equilibrium of the structure cannot be granted and failure occurs. Also, the current condition of the partially collapsed colonnade presents higher vulnerability because the system of horizontal thrust is not perfectly balanced on one side, hence providing non-symmetric behaviour. From the results of the push over analyses and by examining the capacity of this multi-drum colonnade, it was found that the capacity of the colonnade is adequate to positively withstand medium earthquakes as those expected in the area where it is built. It is worth mentioning that the structure passed almost thirty years ago a natural test, withstanding, with negligible damages, an earthquake hitting the area in 1980, having an intensity comparable to a design earthquake with a return period of 72 years, or 50% of probability of occurrence in 50 years. A potential strengthening intervention should involve improving the in-plane horizontal behaviour by balancing the horizontal thrust at the first column on the left-hand side.

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Evaluation of the structural stability of an octagonal dome with meridional cracks

Case study: Temple of Santa Lucia, Ambalema (Colombia)

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Abstract

Objective: this work performed a comparative analysis between the construction process carried out when building the dome at the Temple of Santa Lucía, in Ambalema-Colombia and the typical process of an octagonal dome. Additionally, the structural stability is assessed of the dome of the case study against service and dynamic loads. To compare with the case study, known domes were taken as examples from structures in Italy and Spain. The analysis includes a study on the dome's geometry and the constructive errors found.

Methodology: The dome's stability was evaluated through structural analysis software for which the dome was simplified into a system of four articulated arches.

Conclusions: As a result, it was found that the dome of the temple of Santa Lucía does not have a system to counteract lateral thrusts, which permitted the appearance and widening of meridional cracks. These cracks propagate from the base to the crown, but do not compromise the structure's stability for service loads. The analysis for seismic loads indicates that the dome is at risk of collapse upon seismic events, even of moderate magnitudes.

Originality: The study is aimed at architects and engineers interested in the theme of restoration of historical structures.

Key words: Dome; Masonry; Arch; Traditional architecture in Colombia; Cultural Heritage

1. Introduction

Masonry structures (i.e., set of lime material with stone, brick, or adobe) emerge with the birth of civilization upon the desire for said constructions to withstand the passage of time. Constructions formerly consisted in supporting tree trunks on stone walls; however, the need to cover all the spaces with masonry led to the birth of the arch, which was invented in Mesopotamia or Egypt some 6,000 years ago, with it being a natural technique to bridge openings (Huerta, 2004). Arches are composed of equal, wedge-shaped keystones constructed over auxiliary scaffolding (Heyman, 1999, p. 39). It must be guaranteed that each keystone thrusts its adjacent keystones in such a way that thrusts counteract each other. Besides, the elements of the springers must be supported on abutments that resist the thrust transmitted by the keystones (Huerta, 2004). All the aforementioned with the purpose of each of the arch's keystones to work in compressive manner (Figure 2). From arches we get the creation of domes, which are differentiated by having lower thicknesses, supported on a drum in charge of restricting the displacements of each element. In addition, domes top off in lantern towers, in charge of generating stability to the structure, guaran-

Figure 1. Ambalema, Colombia. Temple of Santa Lucia, National Cultural Heritage from 1980 (Source: by the authors).

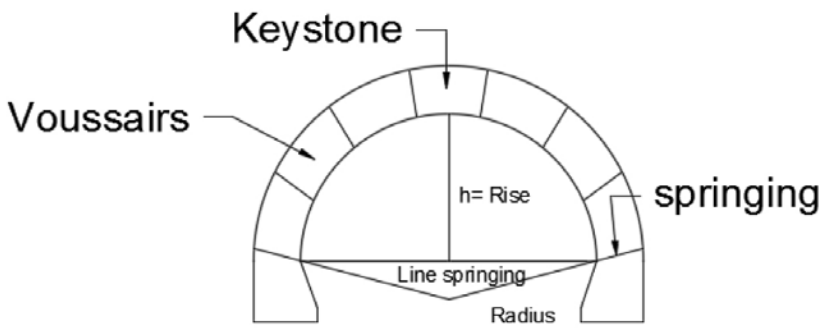

Figure 2. Parts of an arch (Source: by the authors).

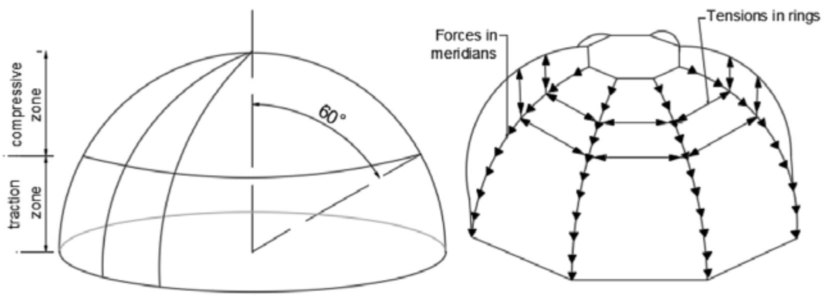

Figure 3. Forces and stress upon the domes (Source: by the authors).

teeing work under compressive stress.

Domes are vaults, which when divided into meridians behave as funicular arches, hence, resistant to load systems without developing bending tensions under certain levels of magnitudes. The domes have parallels that restrict their lateral displacement, developing tensions in rings that permit membrane behavior (Ignacio Requena Ruiz, n.d.). The forces on rings that restrict movement of the wedges out of the plane are compressive in the upper zone and tension in the lower zone. The transition from the compression zone to the tension zone occurs between 45 and 60 degrees with respect to the dome's vertical axis (Pavlovic, Reccia, & Cecchi, n.d.) (Figure 3). Tensions in the lower part are not present in atypical cases of domes with flat crown and heavy lantern tower and the effect of the ring forces is more pronounced in a dome's upper region, where the circular compression rings tend to maintain equilibrium (Hall, Lindsay, & Krayenhoff, 2012).

Investigations of different ancient contructions around the world evidence typical components of octagonal and circular domes. In Spain, the Ermitorio de *San Marcos* and the Church *Arciprestal San Jaime* show important characteristics to highlight like, for example, the elevation of the central dome over a drum that counteracts thrusts in both constructions. The outer plant in both domes is octagonal and the outer sheet is consistent with a cambered geometry of two partitioned leafs. The *Ermitorio de San Marcos* has an eight-arch supplement of variable width that converge by forming an upper crown, unlike the Church *Arciprestal San Jaime*, which has eight ribs formed by aristas coinciding with the diagonals from the drum (Soler-Verdú & Soler-Estrela, 2015). In Italy, the santuary *Basílica de la Virgen de la Humanidad* and the cathedral of Florence have octagonal masonry domes with ring and rib system, light lantern towers, and large drums (Foraboschi, 2014). Regarding the circular domes, the domes of the 35 temples of the province of Alicant (Spain) are characterized for being 97.37% of circular plant because they are easier to construct and geometrically the center of the supports coincides with that of the springer circumference of the domes, all are supported on drums or rings that comply with the minimum height and thickness parameters proposed by García (Pérez-Sánchez & Pie de Causa-García, 2015). These drums are in charge of counteracting the thrust produced by the domes (Gema López Manzanares, n.d.). The way to counteract thrusts in the absence of a drum is through the construction of adjacent vaults (López Mozo, 2013).

The domes of the ancient structures referenced all have a drum and lantern tower, indicating that these elements play a fundamental role in contributing to stability. The structural labor of the drum, besides counteracting thrusts, is to keep the dome from presenting a rotation mechanism. Drums require adequate thickness to fulfill their structural function. Mostly, these must be of considerable thickness and if not so, they need circumferential reinforcement elements. The lantern tower, on its part, is a joint mechanism that belongs to the load resistance system. The heavier the lantern tower is, greater force will be exerted by the drum. Hence, to prevent its aperture the drum must be thick (Foraboschi, 2014).

The shape of the domes determines the conditions of equilibrium. These conditions stem from the hypothesis of absence of friction between the keystones to guarantee that the trajectory of forces is perpendicular to the contact surface (Gómez de Cózar, 2001). In a study conducted in 2000, German López Manzanares concluded that "concave and conical shapes are always stable due to the dome's capacity to develop compressive stress, the opposite occurs with convex shapes that have defects in maintaining a constant thickness".

Construction processes used for ancient domes, arches, and walls are based on empirical techniques and knowledge of the nature of masonry structures, that is, these kept in mind laws of proportion more than resistance criteria, looking for geometry to provide an adequate transmission of stress in the material. A design was created in which all the elements would work under compression, without admission of another type of stress (Hurtado Valdez, n.d.).

The stability of an arch depends on the possibility of drawing a line of thrust within the central third of its thickness (Rankine 1858, Huerta Fernández, 2005). Thus, is how the theorem establishes: "if it is possible to find a system of internal stress in equilibrium with loads that comply with the limit condition, that is, that the materials are working at compressive stress, the arch is considered stable" (Heyman, 1998, Huerta Fernández, 2005). However, stability may also be determined through structural analysis programs. This last method evaluated the dome's stability of the ancient temple of Santa Lucía in Ambalema.

The ancient temple of Santa Lucía located in Ambalema – a municipality in the department of Tolima, Colombia, is a relatively new structure, given that anamnesis concluded that the current structure dates from early to middle 20th century: this structure is part of the historical center of Ambalema (site declared historical heritage of humanity), the temple is mostly constructed in masonry; however, wooden and reinforced concrete elements are found. This structure has an octagonal dome in masonry that presents meridional cracks; due to this, a study was conducted on stability against service loads and seismic loads through a spectral modal analysis.

2. Methodology

To conduct this study, the geometric characterization of the temple was performed; to determine the dome's geometry, photogrammetric estimates were made. The photographs were taken from the inside and outside (Figures 4, 5 and 6).

Samples of the materials used in the construction were extracted to obtain the mechanical behavior of each. Characterization of these materials was done through laboratory tests and correlations suggested in NSR-10 (Colombian Association of Seismic Engineering AIS, 2012) and Eurocode 6 (Comité Européen de Normalisation CEN, 2005).

The software SAP 2000 was used for the dome's structural stability analysis, The analysis included mechanical characteristics of the materials. It was made through macro-modelling, taking the mortar-brick set to obtain a single composite material. The model's geometry initially consisted in defining the elements via a distribution through pre-meshing (Muñoz, 2000). Due to the meridional cracks present between the caskets of the dome, membrane tensions become compressive stress and the structure starts to behave as a series of concentric arches along the meridians (Pavlovic, Reccia, & Cecchi, n.d.).

Figure 4. Exterior view of the dome of the temple of Santa Lucía (Source: by the authors).

Figure 5. Exterior view of the dome of the temple of Santa Lucía (Source: by the authors).

Thereby, four arches were considered for analysis, taking as an arch, the figured formed by two caskets one in front of the other. found; this can be done because the vertical cracking transforms the structure from bi-dimensional to unidimensional (Foraboschi, 2014). The analysis encompasses the behavior against service loads, as well as a spectral modal analysis.

Beam-type and wall-type elements that support the dome were considered. To include the lantern tower in the analysis, it was assigned as a point load representing its own weight over the crown of each arch.

A modal analysis was preformed, for which the temple's frontal structure was modeled. This analysis was carried out to evaluate the

Figure 6. Inner view of the dome of the temple of Santa Lucía (Source: by the authors).

dome's stability against seismic loads, which was conducted by determining the vibration periods belonging to the modal forms, followed by the elastic acceleration spectrum for damping at 5% established by NSR-10. Lastly, occurrence of seismic forces was defined in two directions (X and Y), bearing in mind the load combinations (Table 1).

On site data, an acceleration coefficient (Aa) of 0.25 and an effective peak acceleration (Av) of 0.2 were taken. Regarding soil data, a short-period amplification coefficient, Fa = 1.4, was used, along with an intermediate-period amplification coefficient, Fv = 2.0.

Table 1. Load combinations used in the dynamic analysis

COMBINATION	PERCENTAGE OF PARTICIPATION
C1	100% **X** + 30% **Y**
C3	100% **Y** + 30% **X**

Source: by the authors

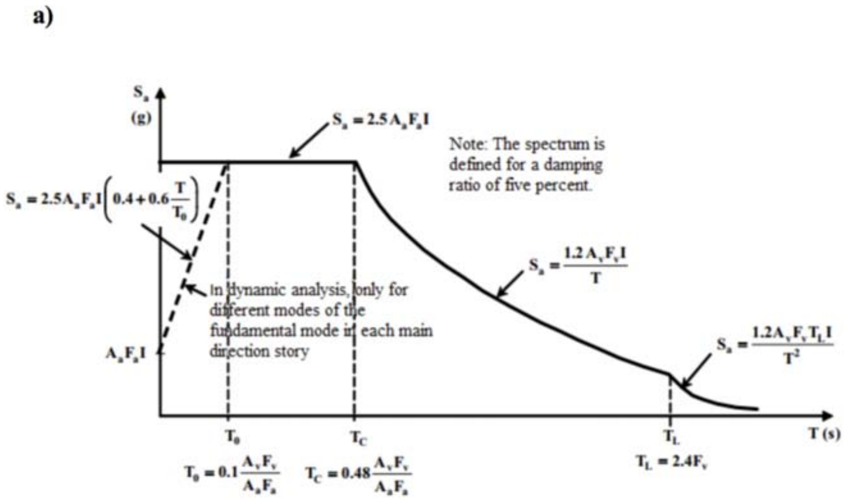

Figure 7. Elastic spectra of analysis: a). Elastic acceleration spectrum design (NSR-10) (Source: Colombian Association of Seismic Engineering, AIS, 2012).

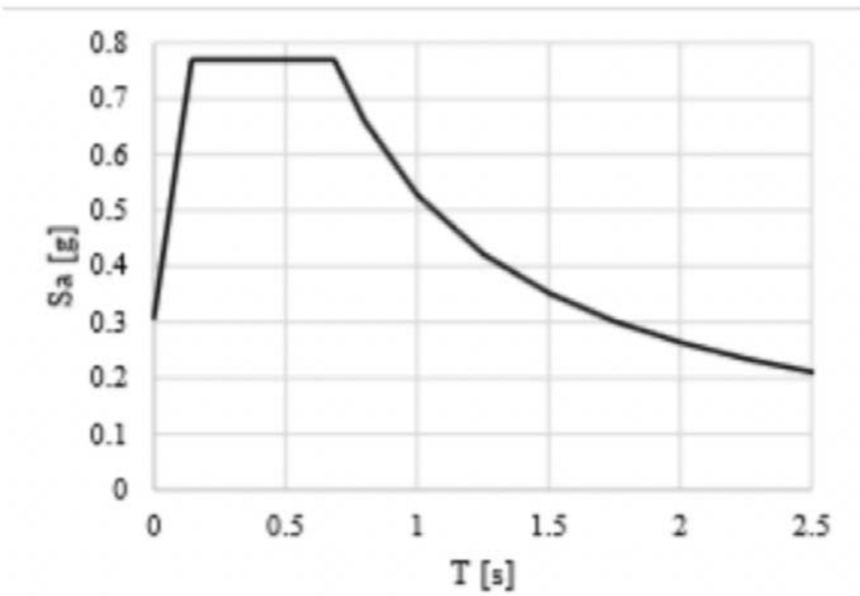

Figure 8. Elastic spectra of analysis: b). Elastic acceleration spectrum of determined design (Source: by the authors).

3. Results

The height from the base to the dome's crown was measured at 18.9 m. It is an octagonal dome of internal diameter and external diameter equal to 4.12 and 4.3 m, respectively. The thickness of the whole dome is constant at 0.09 m. Its total height is 2.03 m and it is hemispherical flattened at the crown, where a concrete lantern tower rests with a diameter of 0.6 m (Figure 9).

For the case study, the dome is supported on six reinforced concrete beams: two are parallel to the tower walls and the rest form a 45-degree angle with respect to the lateral walls; all the beams are of rectangular transversal section with 0.4 m in height and 0.3 m width and the tower walls have equal thickness at 0.24 m (Figure 10). The two other supports correspond to brick walls, which go to the foundation. The span between the lateral walls and the dome was covered by low-resistance concrete slabs (between 10 and 12 MPa) on wood falsework still evident from a lower view.

The dome of the temple of Santa Lucía is hemispherical single leaf with eight caskets constructed in solid brick and mortar. The caskets show cracks of great consideration between them (some reaching 3 cm), indicating the independent work each one undertakes and the tension stress present. However, the dome is stable against service loads due to the union provided by the lantern tower topping it off.

Analysis of the dome's constructive system permits determining its behavior upon loads and its current state. The meridional cracking between caskets presented by the temple's dome is a phenomenon that does not directly compromise the dome's structural behavior. In typical cases, the cracks commonly start in the springer and expand toward the crown without reaching it and descend through the drum; however, the lack of a drum present in the dome of the temple made it easier for total cracking from the springers to the crown.

Construction of the dome consisted, first, in the disposition of the elements to support it, which are of different transversal section

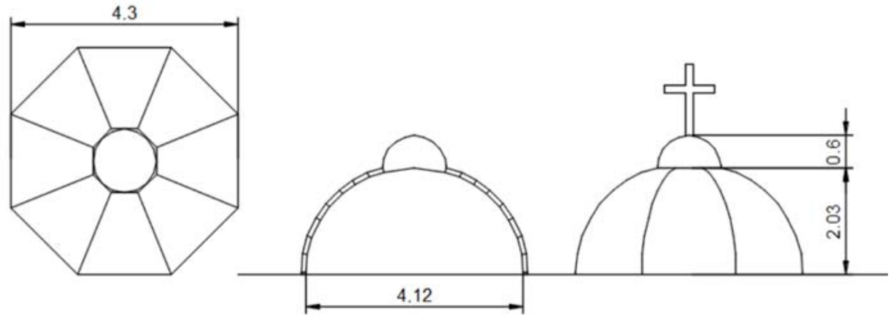

Figure 9. Dome dimensions in meters (Source: by the authors).

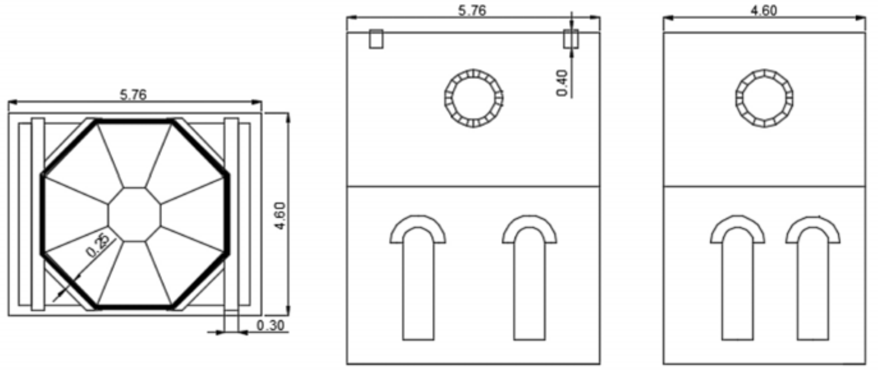

Figure 10. Dimensions in meters of the dome's supports (Source: by the authors).

and of different materials, specifically, two of the caskets rest on the tower walls and the other six on reinforced concrete beams; that is, they do not have support elements of equal rigidity.
The caskets were constructed independently; it was possible to determine through photographic inspection that the bricks were not interlocked continuously around the whole perimeter of the dome – which was a common practice in the construction of these types of domes; the rows were erected independently for each casket and these were then joined with mortar. Construction of the first rows, seen from inside the dome, shows that the beddings are completely horizontal during the first six rows for which falsework was not needed. Row number seven is the high row, from there an inclination appears, generating a simple radial form to the dome. In addition, the caskets have rope interlocking in which the bricks were joined with mortar of variable thickness (between 1 and 2.5 cm), which indicates that the thrusts among bricks are not transmitted homogeneously; the opposite occurs between the union of the rows, showing more or less constant mortar thickness among all.
Construction of octagonal domes may be done without falsework (Santiago Huerta Fernández, n.d.); this occurs when the interlocking of the bricks is conducted surrounding the perimeter of the support holding it. The dome of the temple of Santa Lucía was not constructed in this manner; there was need for falsework as of the high row, a consequence of the independent interlocking for each casket. The dome's thickness complies with the minimum thickness of 1/100 of the curvature radius proposed by Huerta (Huerta, 2004); this has a value of 1/22.55 of the curvature radius. Additionally, it is not supported on a drum in spite of presenting horizontal rows. These rows need to have a minimum thickness of 1/7.14 and a height of 1/3.34 of the dome's diameter (García Jara, 2008) to be considered drum.
The stability analysis for the service loads shows that the four arches present similar stress distribution. The presence of tensile stress is evident in all the arches. These stress values are noted in keystones 2, 3, and 8 (Figure 9), the maximum values of tension stress the arches are subjected to are 150 and 340 kPa for diagonal

Figure 11. Fissures on caskets of the dome of the temple of Santa Lucía (Source: by the authors).

and normal arches, respectively. The zones subjected to compression presented maximum stress of 270 and 320 kPa.
Upon modal analysis, the structure presents a fundamental period of 0.2184 seconds. The compressive and tensile stress generated after the dynamic analysis exceed the masonry's maximum resistance stress (Table 2).

4. Discussion of results

The results obtained from comparing among the different buildings and the dome of the ancient Temple of Santa Lucía permits attributing the fissures among the caskets to the lack of uniformity in supports and to the absence of a

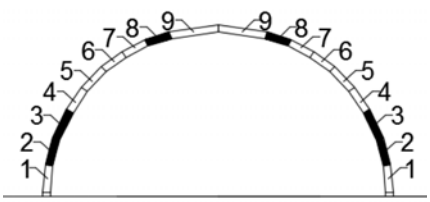

Figure 12. Numbering of the keystones (Source: by the authors).

drum. The fissures stem from the dome itself, that is, no records exist that the elements that support it present structural problems, and hence, fissures are attributed to the absence of a drum. The presence of a drum is not indispensable in the construction of domes, but it provides stability by counteracting the thrusts produced by its own weight and the service loads, avoiding the presence of tension stress. Lastly, the absence of interlocking between contiguous caskets did not permit guaranteeing transmission of monolithic forces, which facilitated the fissures being present in the brick-mortar union.
The tensile stress on the four arches occurs as of 60 degrees

Table 2. Maximum stress on arches, according to types of loads.

Arches	Stress (MPa)	Service	c1	c3
Normal	Compressive	0.32	10.95	8.43
	Tensile	0.34	10.95	8.43
Diagonal	Compressive	0.27	13.06	7.18
	Tensile	0.15	13.06	7.18

Source: by the authors

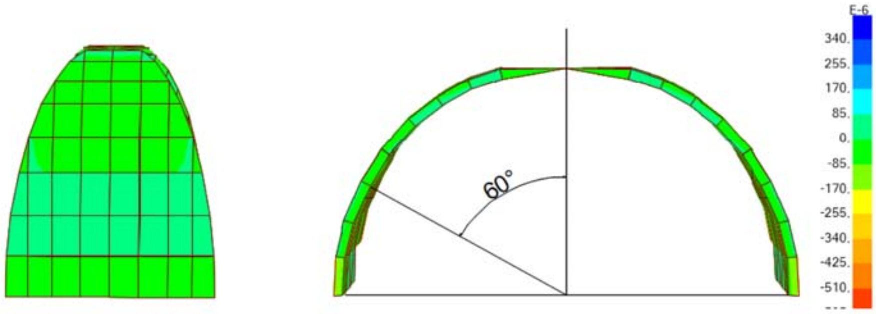

Figure 13. Stress, in kPa, on arch 1 (Source: by the authors).

with respect to the vertical axis from the center of the dome, thus, complying with that proposed by (Pavlovic, Reccia, & Cecchi, n.d.) (Figure 13), reiterating the dome's stability against service loads.

Regarding the analysis for service loads, the stress obtained do not compromise dome stability, given that the mechanical characteristics of the brick-mortar set has a maximum average resistance of 570 kPa for tension and 5700 kPa in compression. With respect to seismic loads, the arches present more critical stress with the combination involving 100% of the earthquake in X direction and 30% of the earthquake in Y direction; additionally, the maximum concentration of stress from combinations (C1 and C3) is generated in the springers of each casket. The compressive and tensile stress endured by the four arches are in a higher range than that of the resistance stress.

5. Conclusions

The comparative analysis performed permitted determining that the dome's construction process is not very different from those used in typical structures. From the support elements, independent caskets were constructed with falsework as of the seventh row. Although the dome has adequate geometry, it lacks an element to counteract lateral thrusts: the drum; which made the cracks between the caskets to propagate from the support to the lantern tower perimeter. Even though the concrete beams partially surround the dome, they are a failed drum attempt, given that they do not comply with the minimum heights and thicknesses required to be considered as such. The lantern tower is the only element keeping together the joints between the caskets because it is a solid lantern tower; if this were not so, the dome would have collapsed due to lack of internal equilibrium.

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A Review of the book
Atlante dei dissesti delle strutture lignee
Atlas of the failures of timber structures
author: Gennaro Tampone, Nardini ed., Firenze 2016.

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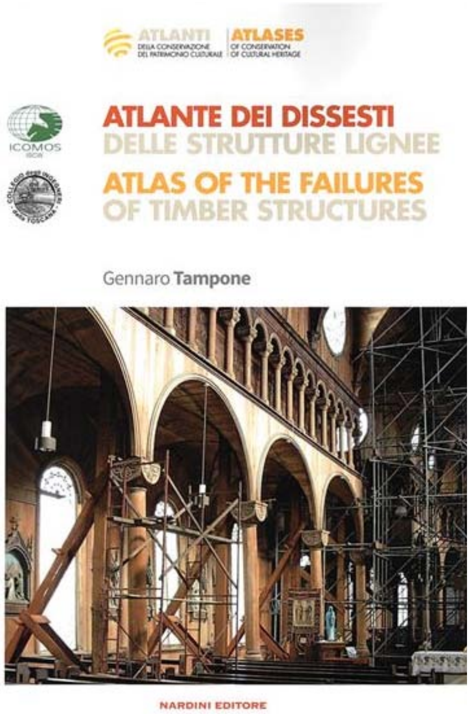
The book "Atlas of the failures of timber structures", by Gennaro Tampone, a bilingual text in Italian and English, may be intended as a guide for interpreting the behavior of structures that had evolved into a case of failure. The intent is to investigate and document systematically the various modes of failure of a member or a complex structure made of timber, in order to devise correct forms of strengthening interventions that should satisfy both the requirements of structural safety and of heritage conservation. This is a very welcome and useful help in a field that is rather disregarded and lacking specific educational supports.



The text is organized in two main sections.
The first, devoted to the effects of mechanical actions on timber members and assemblies, guides the reader in the world of traditional timber construction, starting from basics on material properties and their dependency from the physical environment, through the distinctive mechanical properties of timber members, their composition into structures that are typical of and suitable for this construction material, the common failure patterns deriving from mechanical action, often from an incorrect exploitation of the material characteristics, and finally arriving to outline the approach for examining and analyzing an existing timber structure.
The second section, which is the real atlas, contains the tables and relevant explanations describing the characteristics and the failure mode of an extraordinary number of structures, drawn from the deep and wide experience of the Author. Most of them are well known as belonging to the International architectural heritage. The rich collection of case studies is endowed with drawings and with numerous pictures of each structure and of its details.
The text may be, again, intended as a surefooted guide for professional practice, in line with the manuals of the tradition. Yet it is much more, as it drives the readers capturing them into the engaging world of timber structures. Reading is, thus, enticing for the novice and for the experienced professional. Both will find occasion for learning and especially for enjoying.
The book is published under the scientific patronage of the ICOMOS, International Scientific Wood Committee, and of Collegio degli Ingegneri della Toscana, Florence. ISBN: 9788840443751

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