Nicola Ruggieri · Gennaro Tampone
Raffaele Zinno *Editors*

Historical Earthquake-Resistant Timber Frames in the Mediterranean Area

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Preface

This volume presents selected authors of the H.Ea.R.T. 2013 conference held in November 2013 in Cosenza, Italy, and organized by the SmartLab of University of Calabria, financially supported by Regione Calabria, with the scientific collaboration of Collegio degli Ingegneri della Toscana; Ivalsa, Trees and Timber Institute; Icomos International Wood Committee; Mimar Sinan Fine Arts University, Vocational School, Architectural Restoration Program of Istanbul; National Technical University of Athens; Institute for Sustainability and Innovation in Structural Engineering, School of Engineering, University of Minho. The International conference was held under the UNESCO patronage.

The international meeting provided a forum for engineers, architects, researchers, and educators in the field of technology history, constructive features, and seismic behavior of historical timber framed walls; a contribution to the knowledge of these constructive systems, a fundamental requisite to encourage the conservation of these important and precious documents of the history of the world technology.

The use of timber framing technology is known since antiquity, witnessed by six clay models dated to sixth century BC representing *Italian* huts and emphasizing a wooden constructive system constituted by timber frames as reported in Ruggieri. Then, after disastrous earthquakes, it has been recommended, with various connotations and regulations, by several codes, as a recognition of its validity and as a planned technical answer designed to withstand natural disasters, especially seismic events. Relevant to this purpose are the examples of the so-called Sistema Borbonico in Southern Italy, the theoretical peak of which is represented by Vivenzio's model (Ruggieri and Zinno) and the so-called Gaiola in Portugal that was described from a historical and mechanical point of view by Vasconcelos et al.

Other systems, Himis, Dhajji Dewari, and a type spread in Lefkas Island, outcome of a traditional knowledge, showed a proper response during recent telluric events (Langhenbach; Vasconcelos et al.; Makarios and Demosthenous).

Furthermore, the book deals with the interpretation of the behavior of timber framed walls under seismic actions by means of calculations and experimental tests.

Poletti and Vasconcelos described the results of several cyclic tests (ISO DIS 21581 protocol) performed on reinforced and unreinforced samples of Gaiola in the laboratories of Minho in Portugal; while Ruggieri and Zinno's manuscript devotes its attention to an experimental campaign (UNI 12512 protocol) carried out on the Borbone system full-scale specimens.

The answer of a timber framed building subjected to shaking table tests using full-scale model is presented by Hanazato et al. The contribution examines also the effectiveness of a strengthening technique using aramid fiber wires and, contemporarily, a new measure technology is presented to reveal both the dynamic deformation of the walls and safety limit of the displacement in dynamic phase. Dutu et al. present a research project data performed in the Center of Urban Earthquake Engineering of Tokyo Institute of Technology with the main aim to interpret, by means of reversed cyclic loading, relied on the Curee procedure, the efficiency of a retrofitting method using aramid fiber sheets.

Other contributions contained in the book deal with modeling analysis as in that of Kouris in which is presented a nonlinear empirical macro-model for nonlinear static analysis of Timber-Framed masonry structures and in Makarios and Demosthenous's paper that examines in particular the out-of-plane behavior of the panel by means of a 3D finite element numerical model. Ceccotti and Sandhaas propose to assess the seismic vulnerability of historical timber framed buildings, through the use of a commercial software, implemented with the "Florence pinching", namely introducing a spring model able to reproduce the behavior of semi-rigid joints under reversed cyclic loading.

Galassi et al. investigate and evaluate the seismic performance of the Borbone system through a comparison between experimental tests, carried out by means of cyclic tests on 1:1 scale models, and the results obtained by numerical modeling of the mechanical system that is able to interpret the contribution of the wooden structure, as well as that of the masonry, to the overall stiffness of the wall.

Moreover the textbook provides, based on case studies, a methodology relative to the diagnosis, strengthening, and seismic improvement, interventions compatible with the modern theoretical principles and criteria for conservation (Gattuso and Gattuso).

Nicola Ruggieri
Gennaro Tampone
Raffaele Zinno

Contents

Chapter 1
Timber Framing Wall in the Italic Civilization

Nicola Ruggieri

Abstract Several clay models, which date back to the 6th century B.C., were discovered in the enotrian site of Guardia Perticara (Pz). They represent Italic huts and emphasize a wooden constructive system constituted by timber frames. The research aims to propose a systematic reading of the represented load bearing system which embodies an important step in the historical development of timber structures. The clay models show a concordance with archeological evidences. In fact, some holes dug into the ground to hold timber post, with a rectangular development in plan, were discovered in this geographical area. These models reveal a complex hierarchic organization of the various members with the presence of bracings inside timber frame and along the roof structure. The wooden diagonals increase the constructive system stiffness and improve the timber frame response to horizontal forces and consequently to in-plane seismic actions. The paper also focuses on a representation of a joint failure with loss of equilibrium and members bending which characterize some load bearing elements. Such realistic description is coherent and compatible with the structural configuration, the loads system and the mechanical material characteristics.

Keywords Timber frames · Clay model · Italic culture

Introduction

The research has analysed five clay artefacts, identified in the paper as “A, B, C, D, E”, with a provenience from the Guardia Perticara (Pz) Enotrian site. They have been discovered by Salvatore Bianco in female burials to which it is added a sixth one (“F”) found out of the context and date back to the 6th B.C. The models point

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out similar geometric characteristics and they are house/temple representations with a wooden load bearing structure (Figs. 1.1 and 1.2).

The handmade models are characterized by a rectangular small case, a couple-close roof and a base rise by means of four quadrangular supports, as an alternative to a foot shape, excepted one specimen that is devoid of supports.

Fig. 1.1 Model "A", a clay artefact discovered in the Guardia Perticara Enotrian site. The timber framing representation emphasizes Saint Andrew crosses both in the roof plane and wall (reproduced with permission of "Ministero dei Beni e delle Attività Culturali e del Turismo Direzione Regionale per i Beni Culturali e Paesaggistici della Basilicata"—"Soprintendenza per i Beni Archeologici della Basilicata")

Fig. 1.2 Model "B", a clay artefact discovered in the Guardia Perticara Enotrian site. The roof carpentry is characterized by the presence of two extremity trusses connected by means of a ridge beam (reproduced with permission of "Ministero dei Beni e delle Attività Culturali e del Turismo Direzione Regionale per i Beni Culturali e Paesaggistici della Basilicata"—"Soprintendenza per i Beni Archeologici della Basilicata")

Fig. 1.3 Model "C", a clay artefact discovered in the Guardia Perticara Enotrian site. The "check" pattern reminds a wooden wattle and daub (reproduced with permission of "Ministero dei Beni e delle Attività Culturali e del Turismo Direzione Regionale per i Beni Culturali e Paesaggistici della Basilicata"—"Soprintendenza per i Beni Archeologici della Basilicata")

The dimensional features,[1] similar for all the artefacts, are about of 20 cm high, about 15 cm wide and the largest side is around 25 cm long. They can be assigned to the Sub Geometric period for their decoration motifs, in detail the models include Bi-chrome paintings, representing Saint Andrew crosses, swastikas, and grid pattern; in only one specimen is present a possible *apotropaica* form, a human figure with raised arms (Fig. 1.3).

The upper part of the models is characterized by relief elements, zoomorphic and bull-like representation, alternatively birds figures.

The Guardia Perticara artefacts depict a realistic reproduction of a wooden framed system, as well as an evidence of the typological and in plan distribution characteristics of a Proto-historic house. It is worth considering that they represent an important phase in the scientific knowledge evolution of timber structures during the Italic civilization.

The represented house shows a certain typology and constructive technique autonomy if compared to the Hellenic one of the same period. It is due to a peripheral constructions development which provides a limited culture influence by Greek colonizers, even though the discovery territory is localized near an important communication route as the Agri river. However it is ought to "read" the artefacts

[1] These dimensions are deduced on the base of a visual analyses.

considering the sources limitation because of a possible reality idealization by the model realizer, relate to holy symbolisms and to the intrinsic properties of the material model. In fact, in some cases, the artefact is not easy to adapt to the real structural and dimensional configuration due to physical and execution characteristics.

The Constructive Material

Wood is characterized by an easy procurement in nature, a simple processing and laying, as well as a low specific weight and easy carrying. It represents one of the available material in the Proto-historic age to carry out load bearing structures.

The whole constructive system depicted in the clay models shows a timber employment: the vertical structure, posts and transversal elements with possible wattle and daub infill and the roof structure, the king-post trusses, beam and joists. We can hypothesize the use of Oak, as wooden genus, quoted also in the *tavole di Eraclea*, discovered in the Metaponto archaeological site and as an alternative Chestnut. This is a conjecture legitimised by a spread of chestnut and oak tree in the plain and in the hill in Italy and in particularly in the Basilicata region since the Bronze Age, that persists during the Iron Age [1]. The various timber members manufacture appears in a rough way, from which is obtained a representation of a roundish structural element in the majority of the members, consequently probably they have been put in place with the bark or just debarking. Different features characterize the rafter and the ridge beam of the "B" model, where the cross section is quadrangular, even if it does not display a dimensional hierarchy of the sides. We have no data about the structure execution, whether the house skeleton depicted has been built on the ground and then hoisted in position or assembled directly in situ.

The Resistant Framing

The structural mechanism emphasized in the "A", "B", "D", "F", artefacts is constituted by timber frames with no specified infill. The latter could be composed by lathing, mud and daub, alternatively by stones,[2] according to the archaeological discoveries.

[2] Several cyclic tests have been performed by Professor Ceccotti et other on full scale models of timber framings with lathing plastered infill and alternatively brick or stone masonry. These experimental proofs have showed an excellent anti-seismic capacity and in general under horizontal actions of the tested constructive system, providing results relatively to its ductility and stiffness.

Fig. 1.4 Virtual reconstruction

The “E” model is devoid of decorations. This lack of constructive evidences could be due to the presence of a coating plaster.

The “check” pattern showed in the “B” and “C” artefacts remind a wooden wattle and daub. This is composed by posts and horizontal members closely arranged with a consequent improvement of the transversal stiffness of the wall (Fig. 1.4).

The pillars distribution, relative to the “A” and “F” specimens, divides the longitudinal wall in three spans with similar dimensions. Saint Andrew crosses are represented both in the longitudinal (“A” and “F”) and transversal wall (“B”). This device generates a limited frames deformability and a high resistance to actions deriving from wind and seism. Furthermore the bracings arrangement along the two main directions (“A” and “F”) allows a tri-dimensional behaviour to horizontal actions of the depicted construction. The research of an anti-seismic response of this kind of construction is a fascinating hypothesis, prelude to the more complex system “conceived” in the Calabria region in the 18th C [2].

Maybe an illation, even if the presence of not deformable frames and consequently of a major transversal stiffness by means of bracings is an incontrovertible truth.

Anyway the research of a frames stability is also motivated by the inaccurate execution of the load bearing elements that are characterized by many eccentricities.

The whole structural system of the models represented is based presumably on a wooden boarding. This is raised from the ground and discharges its weight in the four corners building by means of poles with different shape features.

This constructive device reveals a certain technological development and is realized probably to preserve the wooden structure by the humid environment of the ground and by xilophagous insects, for instance *carpenter ants* (*camponotus ligniperda*) and in general to avoid not many favourable conditions for timber conservation.

The Roof Structure

All the specimens, excluding “C” and “E” models, show a basic king post truss complete in all its elements, devoid only of struts. They display also wind bracings on the slope plan with a consequently deformability reduction under horizontal actions.

The roof carpentry is composed by three similar dimension spans and two basic king post trusses at the extremities, these are proto-trusses constituted by two rafters, a tie beam and a kingpost sitting directly above the same chord. These are common features for all the clay artefacts excluded the “B” one, where the representation doesn’t display an intermediate bearing. The two trusses are connected by means of the purlins at the pitch impost of the ridge beam. The joints concerning these various structural elements are composed by three members, relatively to the inferior connection purlin-rafter-tie beam and by four members for the junction king-post-rafter-rafter-beam; the inferior extremity of the king-post above the tie-beam is simply sat. It’s worth noting a hierarchy configuration, emphasized by the decorations and by the different dimensional features; in fact the purlins present a larger resistant section dimensions than the proto-truss members (Fig. 1.5).

The king-post is characterized by a thin cross section and is lightly backward if compared to the vertical plan of the structural unit. The artefacts called “B” presents, the unique among all the specimens, a secondary structure running parallel to the roof. This is emphasized by lines mat painted representing six joists spaced with span similar to the same member dimension. Relatively to the roof load bearing structure of the “A” model, the two joists are put near and sat on the ridge beam and the pitch impost. This kind of members arrangement reminds the Vitruvian recommendations for the roof structure of the Tuscanic order temple, the *trabes compactiles.* The thatched roofing ensures a low self weight and shows a steep pitch and a consequent high rafters slope, contrary to the following specimens of the same geographic area, with a low slope roof due to climatic reasons. The whole structural unit benefits of such arrangement considerably reducing the horizontal component of the force applied to the joint rafter- tie beam. In this way it is avoided a high thrust that could provoking an ineffective of the connection, consequently the tension stress parallel to the grain is low in the tie-beam. Moreover the two rafters trusses of steep pitch roof are characterized by a modest bending moment develop generated by the purlins and limited shear stress parallel to the grain at the heel of the tie-beam.

Geometrical motifs, rectangular figures, in the minor side of the “D” and “F” models, are represented on the tie-beam. This drawn reminds the empty and full

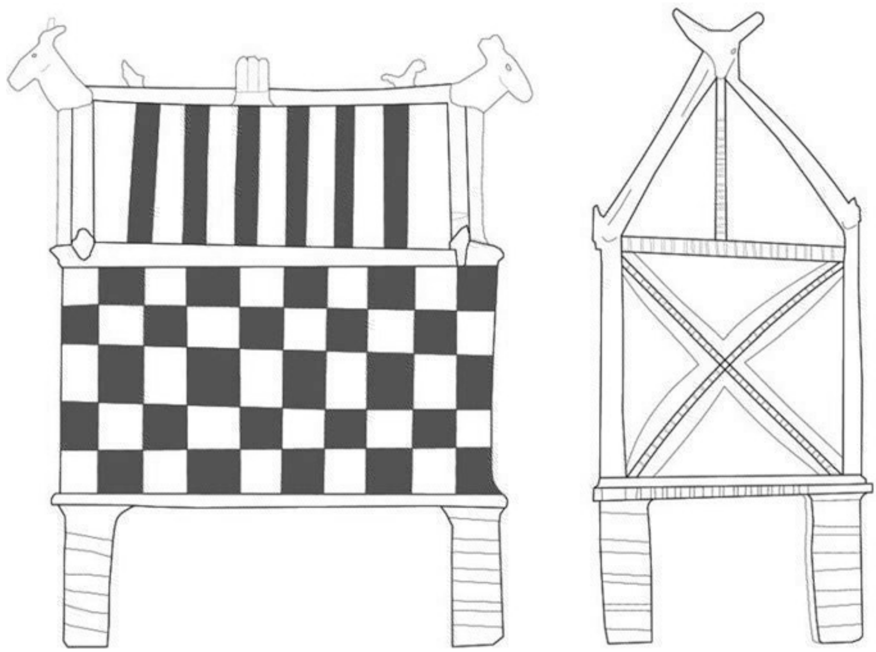


Fig. 1.5 Model "B"

parts alternation generated by the beams presence of an intermediate floor of a garret or of a ceiling.

However the resistant members arrangement is depicted parallel to the major side therefore a not optimized structural behaviour. It is interesting for the constructive history development to emphasize the presence of wind bracings along the roof pitch of the "A", "D", "F" specimens. This configuration acts a stiffness improvement of the roof under horizontal actions, in other words the regular frame polygon of the roof structure is made "not deformable" thank to the saint Andrew crosses presence.

The Joints

The connections represented remind to carpenter nodes, where loads are transmitted by means of pressure, also fastened by ropes. The latter, constituted by vegetable fibres, are put in evidence by a small prominence at the rafter-purlin-tie beam crossing, at the post to inferior transversal member, for the "F" artefact and they improve the joint stiffness also probably relative to the ridge beam-rafter-kingpost-rafter junction, even if in this case the ropes are hidden by zoomorphic *protome* presence.

It is no evident the presence of ropes or other device to improve the junction between the Saint Andrew diagonal and the wooden frame. However it is not strictly indispensable to connect the two elements by means of a rope thanks to the contrast performed by the frame constituted by post and transversal timber. Any clear data characterizes the foot of the king post representation, the latter seems simply bearing on the tie-beam. The joints, relatively to the ridge beam-rafter-kingpost-rafter and the tie beam to kingpost connections are, in a way, preserved from biotic decay with a correct aeration and contemporaneously with a rain water protection. On the contrary, the tie beam-rafter-purlin joint continues outward, beyond the crossing members, and consequently it is exposed to the inclemency of the weather. This arrangement generates favourable conditions for a biotic attacks, mycotic and entomatic, that can easily spread also to the internal surface and therefore interesting the whole junction in a short time (Fig. 1.6).

The represented connections are characterized by a certain ductility under seismic actions. The connection fastens with ropes, performs, even though limited, a tri-dimensional structural response, despite of modern and pre-modern trusses, where in general, the notches of the *gravity joint* provides a unilateral behaviour and response to the stresses with a preferential direction.

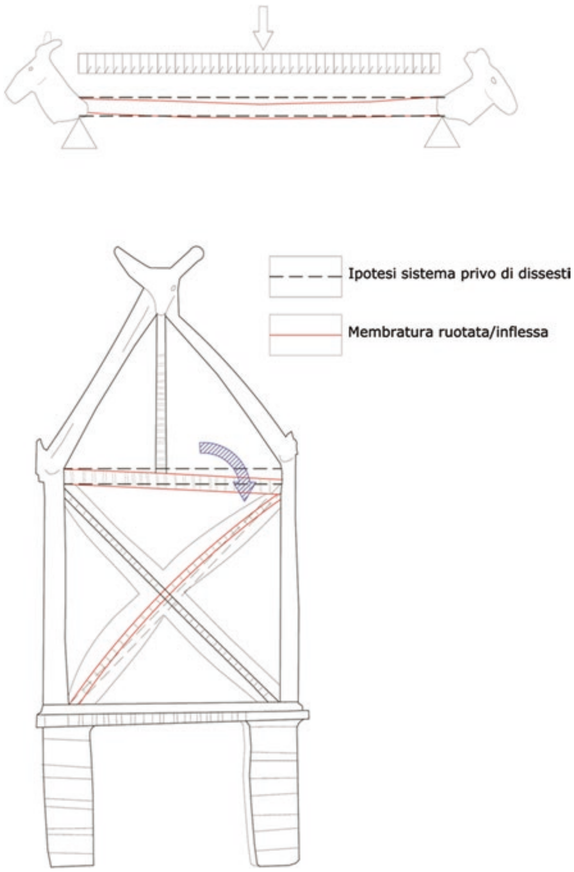
Fig. 1.6 Model "B", joints detail. The presence of ropes entails the increasing of the junction stiffness (reproduced with permission of "Ministero dei Beni e delle Attività Culturali e del Turismo Direzione Regionale per i Beni Culturali e Paesaggistici della Basilicata"—"Soprintendenza per i Beni Archeologici della Basilicata")

Some wind bracings of the “A”, “B” and “F” models are not perfectly converging in the node, but sit on the frame member span. This execution, due to a technical difficulty of connecting three members with ropes in the same restraint, generates secondary moment close to the connection even if it doesn’t lead to a stiffness reduction of the frame [3].

Deformations and Failures

The creator of the models is a sharp observer, in fact this artisan clearly shows, with extreme detail, both the constructive elements and simultaneously the structural failures occurred to the “construction”. In fact he makes note from the reality structural elements too minute, not within calculation requirements, engaged until their resistance limit. Furthermore the artisan records the description of the members deformations in a punctual way. Referring to the artefact called

Fig. 1.7 Model “B”. Failures representation of the ridge-beam and post-tie beam-diagonal joint



"B", relatively to one of the small side of the model, the representation displays the tie beam extremity subsiding. This displacement, thanks to the structural solidarity offered by the timber framing, provokes the bracing deflection, however it does not cause the whole structural system collapse (Fig. 1.7).

The deformation depiction above mentioned represents the pioneer antecedent in the instability analyses of wooden members, before faced by Leonardo, then by Musschenbroeck, therefore it was exhaustively settled by Eulero [4].

Moreover the reproduction of the ridge beam deformation, improperly designed, is compatible with the loads and restraints system represented. This confirms the artisan scrupulosity in rendering the model verisimilar to a real construction. The damage interests the "B" and "D" model member, where elastic and creep deflection are depicted in a proper way with maximum concave rotation in the mid point of the timber element.

Conclusion

The Guardia Perticara clay artefacts represent constructions, in fact, apart from the symbolic character possessed, many comparable elements are found with the coeval archaeological evidences; in addition other proofs are provided by the depicted structural elements wealth and constructive details precision. These are, in the complex, coherent and totally compatible with the described mechanical features of the material and technical/technologic features of the structural system. Such representations assume a not secondary role in the history of the timber structures. In fact they ought to be intended, even if with necessary caution, as real document, to which refer for a proper and precise definition of the dwelling evolution and in particular of its mechanical and constructive characteristics.

A description that shows a hierarchic complex organization of the wooden load bearing structure, in which are ensured the efficiency, stability and equilibrium of the structure elements.

Furthermore the wind bracings presence both in the vertical wall and along pitch roof, if it will be confirmed by ulterior researches, predicts a pioneer development of the earthquake-resistant timber structures.

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Chapter 2
Mechanical and Constructive Interpretation of the *Giovanni Vivenzio*'s Model

Nicola Ruggieri and Raffaele Zinno

Abstract The Borbone government promulgated several measures following the 1783 earthquake in the Calabria region, such as sending numerous engineers from Naples to organize the reconstruction of the areas struck by earthquake. Among these various experts there is Giovanni Vivenzio, doctor of the Royal Household, university teacher, erudite in science, in particularly seismology and vulcanology. The polyhydric intellectual describes buildings damages and victims number in a detailed report, included in *Istoria e teoria de' tremuoti in generale ed in particolare di quelli della Calabria e di Messina,* published in 1783. This treatise is accompanied by Tables with *Spiegazione* (explanations) which represent *Case formate di legno*, an earthquake resistant prototype constituted by masonry reinforced with a double timber framing. The research is based on the publications of the 18th century. concerning civil architecture, on realized examples and on the drawings and relative *Spiegazione delle Tavole.* The study provides data about the constructive interpretation of the anti-seismic model included in the Vivenzio's work and about its behavior in static field and especially in presence of dynamic actions due to the earthquake.

Keywords 1783 earthquake · Borbone anti-seismic system · Reinforced masonry with timber framing

Introduction

The earthquakes, in February and March 1783, struck overall the Calabria region, with disastrous consequences in Reggio Calabria and Monteleone (nowadays Vibo Valentia) provinces. Entire towns and villages were completely wrecked by

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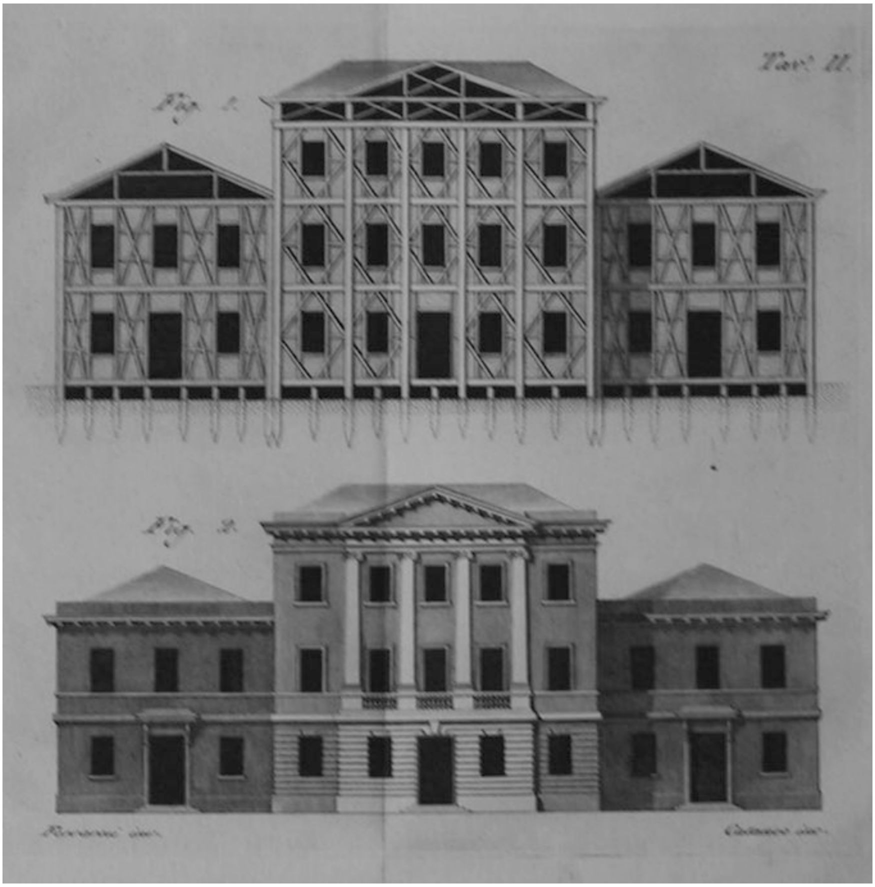

Fig. 2.1 Vivenzio's table. Cross section and facade of the anti-seismic prototype

earthquake involving approximately 30,000 victims, according to the valuation of General Vicar Principe Pignatelli.

For this purpose, Giovanni Vivenzio was sent in the Calabria region to relate to the Borbone government about the *"…effetti funesti che hanno prodotto…* the earthquakes *…nelle due sicilie…"*[1] [1]. After few months, the doctor of the Royal Household published a detailed report in *Istoria e teoria de' tremuoti in generale ed in particolare di quelli della Calabria e di Messina;* in fact the work printed in Naples in the *Stamperia Regale* is dated 8th September 1783, to which followed an updated and corrected version of the 1788.[2]

[1] *…ruinous effects which have produced … in the Due Sicilie… .*

[2] Vivenzio, G., 1788, *Istoria de' tremuoti avvenuti nella provincia della Calabria ulteriore e nella città de Messina nell'anno 1783 e di quanto nella Calabria fu fatto per lo suo risorgimento fino al 1787,* Stamperia Regale, Napoli. The Tables emphasizing a building characterized by inner timber framing are absent in this reprint.

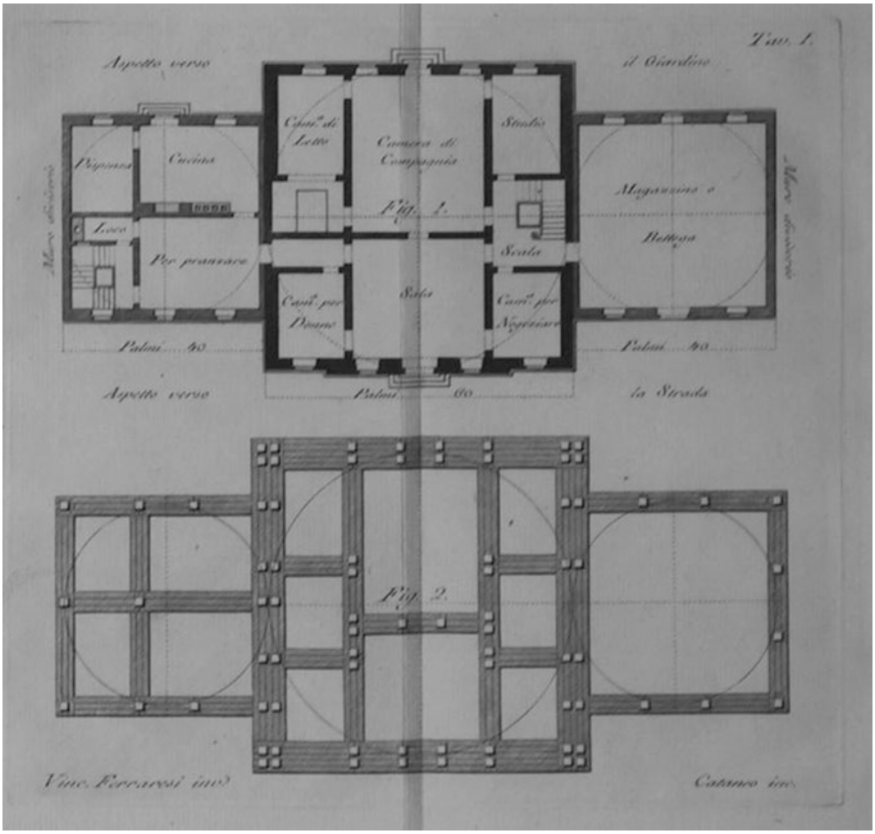

Fig. 2.2 Vivenzio‘s table. Plan view drawings of the anti-seismic prototype

Such treatise deals an interesting history of the telluric events occurred to overall world, to which follows a detailed representation of buildings damages and morphological variations of the landscape due to the earthquake.

Case formate di legno an earthquake resistant prototype constituted by masonry reinforced with timber frames is represented at the end of the book. These drawings depicts a main building constituted by three storeys and two laterals constructions both characterized by two floors. These buildings are described through the ground floor plan, the foundation, the building elevation and some constructive details of the corner posts and of the rafter to tie joint of the king-post roof truss (Figs. 2.1 and 2.2).

The Depicted Buildings and Their Seismic Behaviour

Vivenzio embodies the syntheses of scientific knowledge and in general of civil architecture rules of the 18th century. The correct building criteria stated in several treatises and the existing constructions general check outs in occasion of previous earthquakes which show a certain strength to dynamic actions [2], represent principles and examples to imitate in the Vivenzio model.

The *Case formate di legno* drawings emphasize a regular development in plan and elevation with a bi-axial symmetry [3]. This principle is strengthened by other directions concerning the street on which the buildings prospecting, *"…fossero diritte … e per quanto si potesse esigere fossero queste perpendicolari le une con le altre trasversali, affinché le fabbriche venissero formate ad angoli retti…"* [3] [1]. These are recommendations given with the aim to decrease torsional motions in case of earthquake. In fact the load bearing system opposes identical distribution of the structural elements and stiffness according to the two main directions, under seismic actions.

The lateral buildings, real buttresses, *"…contribuendo alla solidità apporterrebbero meno ruina in caso di violentissimo, e sovversivo tremuoto…"*[4] [1], are characterized by an arrangement generating the center of mass position in the lower part of the edifice, providing a reduced acceleration to the constructions complex in presence of dynamic actions. Furthermore the self weight of the constructive system proposed by Vivenzio results less than an ordinary masonry wall, thanks to the consistent timber members presence, with consequently reduced seismic mass.

The floor buildings are represented higher than the soil grade, ..*"affine di garantire le abitazioni dall'umidità, e per maggiore conservazione dè legnami.."* [5] [1], with an apparent structure durability device. In fact, in this way timber elements are separated from the ground, potential source of moisture and consequently from favorable conditions for biotic attacks.

The Oak is the wooden genus suggested for the structural elements by Vivenzio, following the identical recommendation of the *Istruzioni sul metodo da tenersi nella riedificazione,* the Borbone anti-seismic code.

The Timber Framing

The perimetrical timber structure of the main building is represented with double parallel frames, whilst the inner walls and the whole skeleton of lateral constructions are characterized by a single wooden frame. The frames infill and the wooden *cassone*[6] are composed by regular ashlars bonded by mortar and metallic connectors or in alternative masonry arranged according to the *opus incertum* (Fig. 2.3).

[3] *" …they should be in straight mode … and how could require by them these should be perpendicular to the transversal one, in order for the buildings to arrange at right angle… ."*

[4] *"…contributing to the building solidity they should cause less ruin in case of violent and subversive earthquake…."*

[5] *« …to protect houses from the humid environment and for greater preservation of the timber… ».*

[6] It represents the inner empty, in plan, between the parallel timber frames.

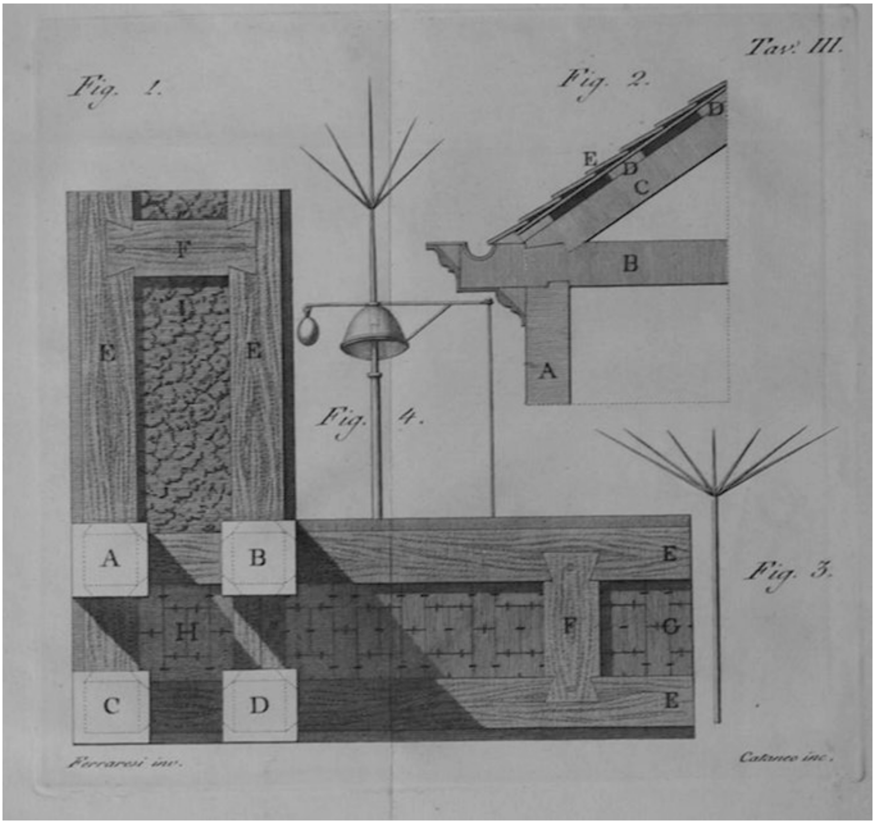

Fig. 2.3 Vivenzio's table (*Tavola III*)

The timber posts have a square resistant section and a length of about 13 m,[7] to which are added 3 m relative to soil fixed part.[8] They are probably constituted by several connected elements [4]. The deformability due to a high slenderness ratio is faced by the timber bracings, at least in one direction.

Every frames is fastened to the contiguous one by means of lap dovetail connection of diatonic elements. The number of the posts properly doubles in the corner with *quattro gran travi*, where the most seismic vulnerability is present; such timber members are connected through transversal and parallel wooden elements. They ensure the lateral stability of the posts, real intermediate restraints that reduce the effective length of the compression members. This device is described by Vivenzio: …*"affinchè dette travi non cedino da qualunque parte, s'incastrano*

[7] Deduced from the measures present in plan.

[8] Vivenzio is antithetic to the Milizia statements that recommended to isolate the structure from the soil, evident precursor of a-seismic engineering modern tendencies. These principles only in the second half of the 19th century will be applied in the constructions in a rudimental way.

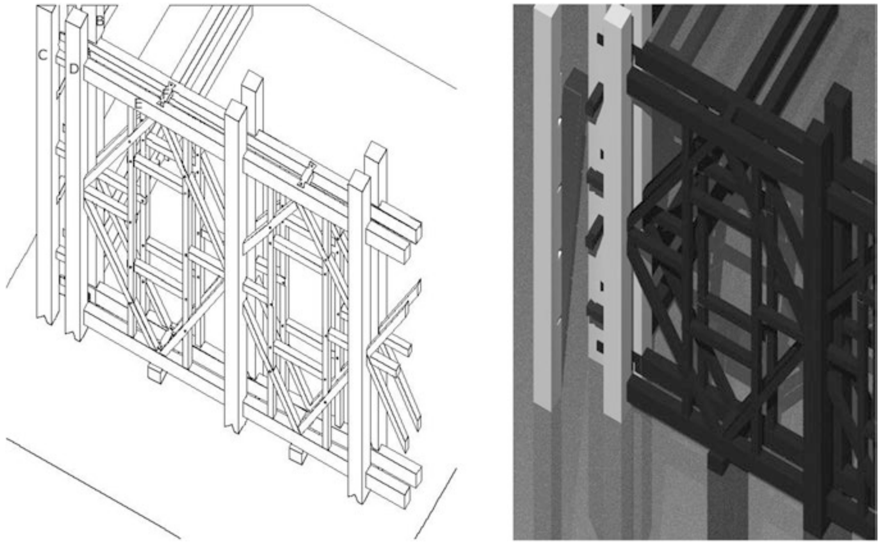

Fig. 2.4 Virtual reconstruction of the angle area of the Vivenzio's prototype

con le traverse AB, BC, CD, ecc intestate a coda di rondine, e replicate lungo la loro altezza…" [9] [1], that are *calastrelli* which connect the four structural elements generating a compound pillar. The lap dovetail joint ensures the transmission of flexural stress, shear and tension (Fig. 2.4).

The inclined members of the main building frames are arranged close to the framing corner, to provide the opening required for windows. They confer a timber frame stiffness improvement to in-plane seismic actions. Such bracings are continuous for the whole façade, including the attic storey.

The *Tavola III* depicts the angle bracket area characterized by a couple of framings, a constructive system recommended for *muraglie de pubblici edificj*. In this way Vivenzio anticipates the modern *Importance class and Importance factors*. In fact the actual practice codes recognize an "importance" of the buildings depending on the consequences of an eventual collapse for human life and for public safety.

The lateral buildings façade is characterized by pillars which are interrupted by the presence of the floor beam; whilst the horizontal ledgers are constituted by a continual wooden element.

Other two horizontal members divide the frame and improve its stiffness. Such elements are connected by means of half lap joints alternate on the front and on

[9] " …in order to the said beams don't collapse in every directions, they have to fasten with the transversal members AB, BC, CD, through lap dovetail joints, and they have to repeat along their height… ".

the back of the two extremities, another improvement to the timber elements connection, even if these notches reduce the resistant section.

The posts with a dimensional cross section larger than the other vertical members characterize the lateral building corner, to make up for the lack of a double pillars structure.

A continual timber ring runs at the upper part of the façade of both the main building and the lateral ones. This technical device contributes to counteract a possible wall overturning under out of plane seismic actions and to distribute concentrated loads on the bearing wall.

Although the queen-post truss of the steep slopes roof, at least for those depicted in the lateral buildings, represent a "closed" system, in which is ensured the absence of horizontal outward thrusts. It is worth considering that this roof carpentry arrangement is used to provide long spanning roofs and requires shorter lengths of wooden elements to cover the same span [5].

The two lateral buildings show a canonical typology of Saint Andrew crosses,[10] that are distributed in a continuous way in the whole façade.

The Joints

The drawings present lap dovetail joints, at least relatively to the connections among frames by means of diatonic elements and those characterizing also the post—*calastrello*—corner post[11] connection (Figs. 2.5 and 2.6).

The presence of nails in all the represented connections, in addition to the geometrical features of the junction, ensures the transmission of bending and shear, contemporarily the iron fasteners improve the stiffness of the joints. In fact these can dissipate seismic energy by means of the two contemporary mechanisms of the compressed and crushed wood grain with the formation of a cavity and of the iron inelastic deformation.

Moreover Vivenzio draws also the connection concerning king post truss tie to pillar, this shows notches that improve the joint.

The king post truss connection, represented in the *Tavola III,* is executed in such proper way to discharge the wall bearing the overall vertical component thrust of the rafter, without generating additional shear stress on the king post truss tie. This rafter—tie joint is stiffened by means of a double indentation.

The drawings and the relative explanations don't provide any evident indication about other structural elements. However we can hypothesize the presence of half

[10] The Borbone practice code does not detail about the stiffness system presence executed through Saint Andrew crosses. This lack of timber bracings is replaced by the masonry infill in the Bishop's building in Mileto (Vv).

[11] Leupold (1726), in *Teatrum pontificale*, shows a constructive detail of a wooden bridge with many similar features to the Borbone system, in particular a double framing and transversal diatonic elements characterized by a lap dovetail connection.

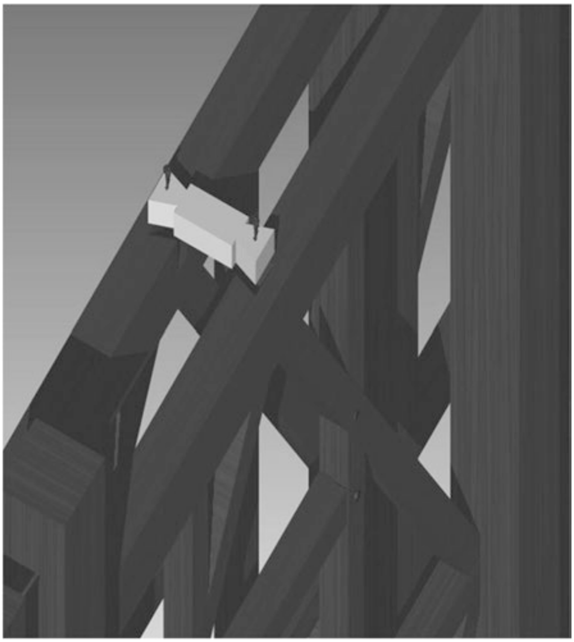

Fig. 2.5 Transversal element of connection between the two frames

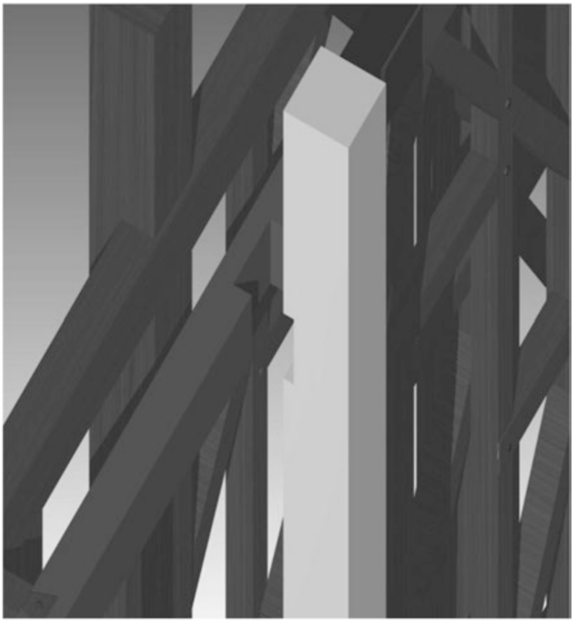

Fig. 2.6 Half lap joint (post-beam connection)

Fig. 2.7 Roof carpentry of the Town Hall of Mons, Belgium. The frame is stiffened by inclined timber member arranged in a similar Vivenzio's model mode

lap joints both for the connections pillar to beam and for the diagonal members of the bracings, characterized by a cut on both connecting elements. In this way the constrained members are contained in the same plane, even if the reduction of the cross section produces a mid point weakness under static and seismic actions. It is worth noting that the half lap joint which connects each other wooden elements, generates, contemporarily, a restraint and a consequent reduction of the effective length and increase of the buckling strength (Fig. 2.7).

Conclusion

We cannot know how this anti-seismic system is widely applied in carrying out building[12] and if it has generated wide effects on construction practice as well as. Masonry walls with a timber skeleton existed in Calabria region before the 1783 earthquake, although they were few specimens. Therefore the Vivenzio's work cannot intend a theoretical-practical *vademecum* for anti-seismic buildings, but it has to be considered a synthesis of the "scientific" knowledge of the 18th century in

[12] An example of a construction with a couple of timber framing internal to the masonry is in Seminara (Rc) in Fondatore Lauro street. My thanks to Architect Infantino for this kind suggestion.

the Borbone reign. In fact, the *doctor* of Ferdinando the IV includes the rising anti-seismic engineering principles of the Age of Enlightenment. The Borbone system and the Vivenzio's model are based on the reduction of the building seismic mass, on the in plan and elevation shape regularity and above all, they aim to reach a connection among the perpendicular masonry panels and to create a box through the use of the timber framings. These principles are conform to the most modern requirements to obtain the improvement of the building anti-seismic capacity.

The Vivenzio's *Case formate di legno* represents a constructive technique characterized by a high amount of timber elements and an extreme carpenter difficulty in connections execution. These features inevitably involve a spread of building structure with a single timber frame and as related by Baratta [6] of the "reduced Borbone system", ground storey constituted entirely by masonry wall and the only second floor characterized by a timber framing load bearing, with the aware aim to decrease the vulnerability under dynamic actions by means of limiting the self weights and consequently the seismic masses, according to the building height.

The Borbone system showed a ductile behaviour and therefore a seismic resistance even in its different versions. In fact, the 1905 and 1908 ruinous telluric events that again struck the Calabria region have pointed out in buildings with a wooden skeleton slight damages or even completely unharmed framing [2].

This constructive system characterized by an indubitable anti-seismic capacity, with an intrinsic material eco-compatibility, therefore can open new research sceneries to use masonry reinforced with timber framing also for modern buildings.

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Chapter 3
Timber Frames and Solid Walls: Earthquake Resilient Construction from Roman Times to the Origins of the Modern Skyscraper

Randolph Langenbach

Abstract This paper explores what can be learned from the earthquake performance of simple, unsophisticated, non-engineered timber and masonry historical construction that resists earthquakes compared to that of modern reinforced concrete frame buildings of varying construction quality that are common in much of the world's seismically active areas. The paper includes an analysis of the observations in the 1930s by American seismic engineer, John Ripley Freeman, about the 1908 Messina-Reggio earthquake and the comparative performance of the 18th century *baraccata* construction mandated by the Bourbon government after the 1783 Calabria earthquake.

Keywords Traditional construction · Timber · Masonry · Reinforced concrete · Earthquakes · Seismic · Earthquake engineering · *Hımış* · Dhajji dewari

The 1999 Earthquakes in Turkey

In November 2000, 1 year after two devastating earthquakes struck near the Sea of Marmara in Turkey, a conference was convened by UNESCO, ICOMOS and the Turkish Government in Istanbul, called "Earthquake-Safe, Lessons to be Learned from Traditional Construction" [1]. The 1999 earthquakes in Turkey, as well as more recent earthquakes worldwide, have demonstrated that in spite of all of the knowledge of seismology and earthquake engineering gained over the last century in the science and practice, the death toll in such events has continued to rise. It has become apparent that steel and concrete, even when built according to modern codes, have not been able to guarantee seismic safety (Fig. 3.1).

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Fig. 3.1 A traditional *hımış* construction house still standing next to a row as far as one can see of reinforced concrete apartment buildings in Adapazari, Turkey after the 1999 Kocaeli earthquake. Photograph by © Randolph Langenbach

At the time of the conference, few would have thought that traditional construction could provide any meaningful lessons to address the dilemma of death and destruction in modern buildings of reinforced concrete (RC). Yet conspicuous among the ruins of the RC buildings were many traditional buildings (known in Turkish as *hımış* construction) with timber frames and infill masonry that had remained standing.

At the 13th World Conference on Earthquake Engineering in August 2004, Fouad Bendimerad, Director of the Earthquakes and Megacities Initiative, reported that "*approximately* 80 % *of the people at risk of death or injury in earthquakes in the world today are the occupants of reinforced concrete frame infill-masonry buildings.*" In fact, a new term has emerged in recent years to describe the problem with reinforced concrete buildings: "*pancake collapse*."

It is reasonable to ask: how can a technology of building construction based on the new strong materials of steel and reinforced concrete be linked to such deadly catastrophes? At the beginning of the last century both steel and reinforced concrete held great promise for earthquake-safe buildings, yet, 100 years later, in Turkey in 1999, in India and Pakistan Kashmir in 2005, and in Haiti in 2010, the pre-modern buildings of timber and masonry remained standing surrounded by collapsed concrete buildings. Clearly the original promise of these new materials has not been fully realized.

After the 1999 earthquakes in Turkey, the world's scholars and engineers descended on the ruins of the buildings that took the lives of 30,000 people. They made frequent pronouncements that the collapses were caused by bad design and poor construction, pointing to those that did not suffer major damage or collapse to prove their point. Inspection, quality control, better training was what was said to be needed. Some experts even asserted that nothing new can be learned because the myriad observed faults had previously been well documented, and that some well-engineered and constructed buildings, of which there were many in Turkey, had survived. From their perspective it may seem that justice had been served, and that bad construction met its rightful fate. Contractors were arrested and developers chased out of town, in the hope that in the future people could be taught to pay attention to building codes, and graft and corruption would cease. “Only then,” they say, “can we expect that earthquakes will not result in such massive mortality and the promise of earthquake resistance of RC construction be fully realized.”

Kashmir

Srinagar has been, and continues to be, a city largely unknown in the rest of the world by decades of regional civil strife. When first viewed by the author in the 1980s, it appeared as a magical world—a city beside a mountain lake with a way of life that seemed unchanged for a 1000 years. The construction practices used for the traditional houses in Srinagar, which stand in contrast to today’s codes, include (1) the use of mortar of negligible strength, (2) the lack of any bonding between the infill walls and the piers, (3) the weakness of the bond between the wythes of the masonry in the walls and (4) the traditional use of heavy sod roofs (now mostly replaced with corrugated steel sheets).

These buildings were observed almost a century earlier by Arthur Neve, a British visitor to Kashmir, when he witnessed the 1885 Kashmir earthquake: “*Part of the Palace and some other massive old buildings collapsed … [but] it was remarkable how few houses fell …. The general construction in the city of Srinagar is suitable for an earthquake country; wood is freely used, and well jointed; clay is employed instead of mortar, and gives a somewhat elastic bonding to the bricks, which are often arranged in thick square pillars, with thinner filling in. If well built in this style the whole house, even if three or four stories high, sways together, whereas more heavy rigid buildings would split and fall*” [2] (Figs. 3.2, 3.3 and 3.4).

There are two basic types of traditional construction with earthquake resistance capabilities found in Kashmir. One, of solid bearing-wall masonry with timber lacing, is known as *taq* and the other, a brick-nogged timber frame construction, is known as *dhajji-dewari* from the Persian words for “patch quilt wall” that is similar to the *hımış* found in Turkey [3]. The October 2005 Kashmir earthquake was centered on the Pakistan portion of Kashmir where 80,000 people died in both reinforced concrete buildings (including one modern highrise residential complex in Islamabad) and traditional unreinforced stone masonry buildings. It also affected India across the Line of Control, with approximately 2,000 fatalities. According to the structural engineering professors Durgesh Rai and Challa Murty of the Indian Institute of Technology-Kanpur: *“In Kashmir traditional timber-brick masonry (dhajji-dewari) construction consists of burnt clay bricks filling in a framework of timber to create a patchwork of masonry, which is confined in small panels by the surrounding timber elements. The resulting masonry is quite*

Fig. 3.2 Four story building of *taq* construction in Srinagar, Kashmir, India. The timbers are on the inside and outside face of the wall. Photograph by © Randolph Langenbach

Fig. 3.3 Buildings in Srinagar, Kashmir. The building on the left is of *taq* construction, and on the right is of *dhajji dewari* construction. Photograph by © Randolph Langenbach

Fig. 3.4 A partially demolished *dhajji dewari* building showing timber frame with typical single layer of infill masonry construction. Photograph by © Randolph Langenbach

different from typical brick masonry and its performance in this earthquake has once again been shown to be superior with no or very little damage.”

They cited the fact that *“the timber studs … resist progressive destruction of the … wall … and prevent propagation of diagonal shear cracks … and out-of-plane failure.”* They went on to suggest that: *“there is an urgent need to revive these traditional masonry practices which have proven their ability to resist earthquake loads”* [4].

Timber-Laced Construction in History

The origin of both types of timber-laced masonry systems dates as far back as the ancient world. The palaces at Knossos have been identified as having possessed timber lacing of both the horizontal and the infill frame variety [5]. This suggests that timber-laced masonry construction dates back to as early as 1500 to 2000 B.C. Evidence of infill-frame construction in ancient Rome emerged when archaeologists dug up the port town of Herculaneum that had been buried in a hot pyroclastic flow from Mount Vesuvius in 79 A.D. They found an entire two story half-timber house and interior walls in other houses believed by the archaeologists to be examples of what Vitruvius has called *Opus Craticium*. After the fall of Rome, infill-frame construction became widespread throughout Europe. Timber-with-brick-infill vernacular construction is documented to have first appeared in Turkey as early as the eighth century [6] (Figs. 3.5 and 3.6).

The question of whether timber-laced masonry construction evolved in response to the earthquake risk is an interesting one, but earthquakes are infrequent, and there were other compelling economic and cultural reasons for the evolution of these systems. For example, many variations of timber frame with masonry infill construction exist in areas well outside of the earthquake regions of the world, including Europe where in Britain it is called “half-timber,” in France *colombage*, and in Germany *Fachwerk*.

Other systems from around the world, including *hımış* and *dhajji dewari,* are described at length in my earlier papers that can be found at www.conservationtech.com, but of all of them, I have only found two historical examples of the frame and infill masonry typology which were invented specifically in response to earthquakes: Portuguese *Gaiola* and Italian *casa baraccata*. The *Gaiola* was developed in Portugal after the 1755 Lisbon earthquake under the direction of the Marquis de Pombal (which is why it is also called *Pombalino* construction). The *baraccata* or *la casa baraccata* was developed in Italy after the Calabria earthquake of 1783, and later was even registered for a patent as an invention [7].

While I have discussed the *gaiola* at length in several papers also to be found on www.conservationtech.com, I have not, prior to the HEaRT 2013 conference, had the opportunity to see on site examples of *casa baraccata*. Much work on *casa baraccata* has been done by other scholars, including our conference host, Nicola Ruggieri [8], and also another Italian Scholar, Alessia Bianco [9], and in

Fig. 3.5 A building in the ancient Roman town of Herculaneum uncovered after having been buried in the eruption of Vesuvius showing an example of timber frame with masonry infill that is thought to have been called Opus Craticium. Photograph by © Randolph Langenbach

addition by my own former academic colleague, Stephen Tobriner [10]. Rather than commenting on or attempting to add to this considerable work, I wish here to focus on a particular document written more than 80 years ago in English by an American engineer, John Ripley Freeman, who undertook a remarkably comprehensive study of earthquake damage and earthquake resistant construction around the world [11]. His section on Italian earthquakes makes up 75 pages of his 900-page book, and early on in his treatise on Italy, he states "Italy, more than any other country, was the early home of scientific research of many kinds, and in its great engineering schools of today, offspring of its ancient universities, the sacred flame of learning still burns brightly" [11, p. 564] (Figs. 3.7 and 3.8).

What is remarkable about Freeman's treatise is that, rather than simply documenting the damage from the earthquakes, he focused on the post-earthquake reports by Italian engineers who had developed post-earthquake building ordinances after arranging for the translation of many Italian documents. His work provides documentation of the evolution of the field of earthquake engineering from its empirical origins to a discipline based on the rigorous mathematics, a process that also traces the shift from traditional Italian stone masonry construction to the increasing use of reinforced concrete frames with masonry infill.

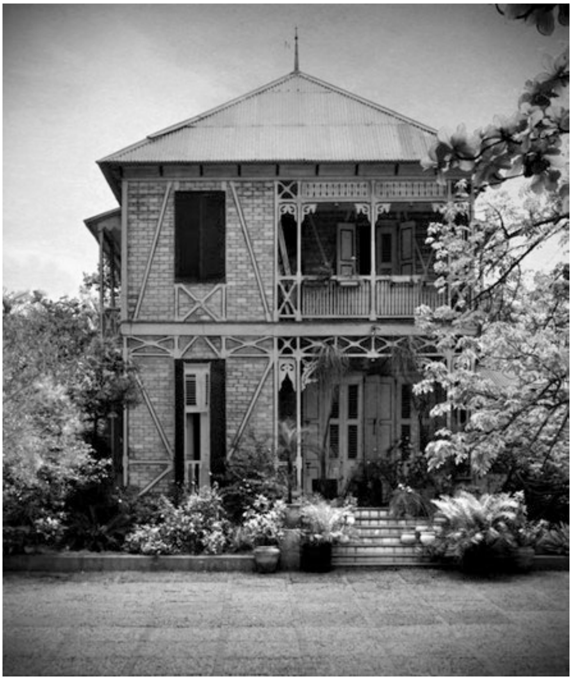

Fig. 3.6 A single family middle class house of colombage timber frame with single layer infill masonry construction near downtown Port-au-Prince photographed after the 2010 earthquake. This house was in the heart of the damage district with collapsed reinforced concrete buildings nearby. Photograph by © Randolph Langenbach

Beginning with the 1930 earthquake near to Naples, he then turns back to the December 28, 1908 Messina-Reggio earthquake, which he cites as *one of the greatest catastrophes since the dawn of history.* It was the engineering reports following this earthquake that has enabled him to cite the post-1783 earthquake provisions enacted by the Bourbon Government that then controlled this part of the Italian peninsula and Sicily—the provisions which established the *casa baraccata* form of construction as a requirement for reconstruction of buildings throughout 18th and 19th century Calabria. These provisions were based on guidelines written by Francesco La Vega, an engineer described by Tobriner as "*engaged in evaluating seismic damage to the sites of cities throughout southern Calabria*" [10, p. 134].

Freeman cites that the Report of the Italian Royal Committee of 1909 on the Messina-Reggio Earthquake [11, p. 564] which said that "*the Bourbon Government issued provisions on March 20, 1786, which even today … still show great sagacity, and the Committee deplores that in the space of a few decades these excellent provisions and suggestions should have been allowed to fall into oblivion, when their strict observance … would have saved to Italy the tremendous losses of 1894, 1905, and of 1908. The Committee stated that it knows of houses built under these Bourbon rules that had resisted all of the successive earthquakes. The system of wood-framed dwellings known as Baraccata, originated from these ordinances issued by the Bourbon Government, and … proved very successful so that the system is even today under certain circumstances highly commendable*" [11, p. 569].

Fig. 3.7 An early 20th century building in Mileto, Calabria, Italy constructed with a baraccata timber frame embedded on the inside face of the wall, with a photo of it taken during its construction. *Photo* courtesy of the owner

The Sub-Committee reports on the facts of 1908 earthquake and an earlier earthquake in 1905 documented that "*the experience in the Messina quake proved that the system of wood-framed houses known as "Baraccata," built according to the ordinances of the Bourbon Government, established immediately after the earthquake of 1783, is a system that may be considered good and advisable for all those cases where materials were lacking for a more nearly perfect system of construction, but where it is possible to obtain, cheaply, good quality of timber of proper dimensions for the framework, and bricks, or regularly shaped stones with at least two planes of repose, and good lime and sand*." [11, p. 568].

Noting the report's reference to "*cases where materials were lacking for a more nearly perfect system of construction*", it is interesting to speculate on what, in 1909, the committee members may have been referring to. The report on the 1908 earthquake stated that "*the greatest resistance was presented by houses of one or two stories in height, constructed of the best brick masonry and resting on rock or firm soil,*" but the sub-committee on the 1905 earthquake made an interesting observation about a new form of construction—reinforced concrete stating that "*... four structures of reinforced concrete in Messina, which remained wholly unharmed*" which "*the sub-committee cited as examples of the capabilities of this material when properly used*" [11, p. 568] (Figs. 3.9, 3.10 and 3.11).

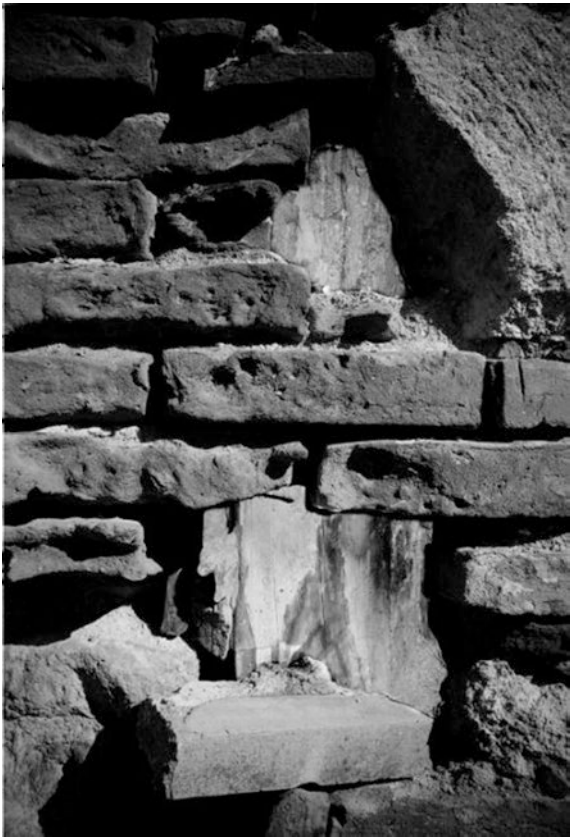

Fig. 3.8 A vertical column of the timber frame in the building in Fig. 3.7 showing through holes in the brickwork. Photograph by © Randolph Langenbach

This observation, however, followed a longer description of other reinforced concrete buildings that "*were nearly all tumbled down, in spite of the fact that … the shocks were less violent than elsewhere.*" They said that these buildings were "*improperly classed as reinforced concrete … because of the poor quality of material used, and lack of proper joints or connections between various members*."

This quote not only documents what was seen in this earthquake, but stands as a prescient observation of the extreme dialectic between good and bad performance that continues to bedevil reinforced concrete frame construction in 21st century earthquakes. The sub-committee's insistence that the failed structures were "*improperly classed as reinforced concrete* …" provides a record of how reinforced concrete was already by 1909 viewed as a modern system capable of great strength and resilience, but which if done badly, not only will be vulnerable to collapse, but should not even be identified as "*reinforced concrete.*" The seemingly odd categorization only would make sense if one would similarly insist that a brick building found to be collapsed in an earthquake because of poor construction should not be entitled to be classed as 'masonry construction', which of course is ridiculous.

Fig. 3.9 Side view of exterior wall showing the thickness of masonry in the Bishop's House in Mileto, Calabria, Italy constructed after the 1783 earthquake showing *casa baraccata* framework embedded in the inside face of the masonry wall. The house has been abandoned for many years. Photograph by © Randolph Langenbach

Fig. 3.10 Interior elevation view of the same wall as shown in Fig. 3.9 of the embedded *casa baraccata* framework in the Bishop's House in Mileto, Calabria, Italy. Photograph by © Randolph Langenbach

Fig. 3.11 This view of Messina as rebuilt after 1908 was identified by Freeman as "*built under the new building laws.*" In the caption to the image, Freeman states "*In some [buildings] the framework is concealed, and in others [such as these shown] it is revealed and made an architectural feature*" [11, p. 562] Photograph by © Randolph Langenbach

From Walls to Frames

Structural engineering has gone through a revolution over the past century. The 19th century was an era of enormous ferment, producing engineering giants like Brunel and Eiffel, along with Jenny and the other engineers of the first skyscrapers. In the first decades of the 20th century buildings went from a height of 10–20 stories to over 100 stories. This achievement required a shift in engineering practice from a largely empirical process to one of rigorous mathematics.

The teaching and practice of the structural engineering of buildings moved away from the design of solid wall structures with post and beam interiors to the analysis and design of frames. To fully understand the implications of this change, we must first isolate what is meant by the term "frame" in structural engineering in order to distinguish between a framework of columns with simply supported beams and a "moment frame" where the beams and columns are interconnected sufficiently to allow the structure as a whole to resist lateral forces, as well as to carry loads to the ground. Until the nineteenth century frame structures and the internal framework of buildings were most often made of timber, with the lateral forces resisted either by masonry walls or by braces within the heavy timber framework.

The advent of steel and steel reinforced concrete has allowed for the creation of moment frames which no longer need to rely on braces or masonry shear walls. In terms of engineering practice, the linear-elastic "portal frame" analysis of such structures has come to define most of the day-to-day professional work. While most of the historical focus is on the transition to the use of frames for taller buildings, the watershed event in this transition is the "invention" of a way of doing a portal frame analysis using the contraflexure methodology for isolating moments. This method allowed the calculation of the bending stresses on multi-story frames by mathematically separating the frame into parts at each neutral point of bending reversal of the columns and beams. This allows the forces to be calculated using the three equations of equilibrium. Modern skyscrapers in every practical sense date their origin to this change in engineering analysis methodology [12, 13].

Moment frames provide lateral resistance by both shear and flexure of the framing members. Their lateral capacity is primarily determined by the strength and ductility of the joints between the beams and the columns. The enclosure and partition walls that turn this open framework into a useable building are routinely ignored in the structural calculations, except as dead weight. The advantage of this approach is that it has allowed for a coherent mathematically-based engineering approach to building design by separating the infinite complexity of a finished building with all of its parts from that of the primary structural system—the frame.

An interesting fact about the historical development of the modern skeleton frame construction and portal frame analysis at the late 19th and early 20th centuries is that thick masonry infill and cladding was very much an accepted part of early steel and RC building construction, even though it was then, as now, not considered in the engineering calculations for lateral resistance. This is made clear by the author of one of the first textbooks on the subject of skeleton frame construction, Joseph Freitag when in both the 1895 and 1901 editions, he writes that "*'Skeleton Construction' … suggests a skeleton or simple framework of beams and columns, dependent largely for its efficiency upon the exterior and interior [masonry] walls and partitions which serve to brace the structure, and which render the skeleton efficient, much as the muscles and covering of the human skeleton … make possible the effective service of the component bones …. While the steel frame is more or less reinforced by the weight and stiffening effects of the [masonry infill], still no definite or even approximate values can be given to such items, except their purely static resistance or weight*" [13] (Figs. 3.12 and 3.13).

It was only a little over two decades after the construction of the first skeleton frame "skyscraper" in Chicago, the 10 story Home Insurance Co. Building by William Le Baron Jenney, that the 1906 earthquake in San Francisco put skeleton frame buildings—even some done by the same architects as those in Chicago—to the test. As it turned out, they passed that test remarkably well. Indeed, one must ask why these first generation steel skeleton skyscrapers in San Francisco remained standing with undamaged frames and repairable damage to the masonry walls, when so many frames with infill masonry buildings have been collapsed by earthquakes a century later. As clearly stated by Freitag, it was not the frame alone, but the masonry in partnership with the frame that was responsible. Many of these buildings are now more than 100 years old, and despite having been burnt out by the subsequent fire, they were repaired and continue in service today.

In spite of the noteworthy evidence from this famous earthquake, it was after the 'invention' of portal frame analysis based on contraflexure methodology when the essential incompatibility between the masonry infill and cladding and the engineering of the underlying frames came into both theoretical and practical conflict. And so, over the years to follow, the masonry walls were made thinner and weaker. Standard portal frame analysis is predicated on the existence of "frame action." In other words, the building design is based on the assumption that the frame will deform in a geometrically coherent way, so that all of the beams and columns can share the loads.

Fig. 3.12 Flatiron Building in New York City under construction in 1902 showing the heavy stone masonry façade resting on the steel frame. The upper walls were constructed separately probably to ensure the weight is bearing on the frame rather than the lower masonry walls. U.S.Library of Congress Photograph

In most parts of the world today, the enclosure and partition walls are most often of weak, but stiff brittle masonry. This has not been considered a problem for wind, as the system together with the masonry was designed to remain elastic, and the addition of the masonry simply made it stiffer and thus likely to be more resistant. Design-level earthquakes though were another matter because a building's structural system is expected to deflect into the nonlinear range. In other words, structural damage is expected to occur. Because these walls are considered by the design engineers to be "non-structural," these infill masonry walls are often not themselves designed to resist the lateral forces of an earthquake. Thus, their impact on the overall deformation of the building is not properly considered, especially after the point where some infill walls in the lower stories of the frame have broken, while others above continue to resist the deformation of the frame (Fig. 3.14).

From Solid Walls to Frames

Many historians of the early skyscraper era viewed the evolution of skeleton frame building design like a genie waiting to come out of the bottle—true transformation could only come when this traditional masonry envelope was shed, with the open

Fig. 3.13 View of San Francisco after the 1906 earthquake and fire. The three tall buildings visible in this view were burned out by the fire that followed the earthquake, but all three were in good enough condition despite this to be repaired, and they are still extant today. Photograph courtesy Bancroft Library, U.C.Berkeley

Fig. 3.14 "Frame action" of a moment frame shown here with the uniform elastic bending of the framing members unimpeded by the effects of infill walls

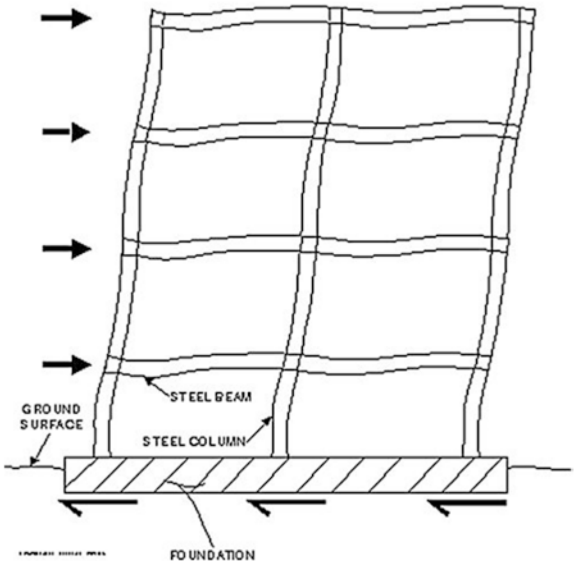

Fig. 3.15 The interior of the historic structure of this Middle Eastern house has been demolished and is being replaced in RC, with the stone façade to be reconstructed around the new frame. This photo provides a graphic image of the difference between the open self-supporting framework of a reinforced concrete moment frame structure and a masonry solid wall structure. Photograph by © Randolph Langenbach

frame itself made the basis for the architectural expression with flexible systems of open spaces and moveable walls [12]. The architectural precursor for the liberation of the skeleton frame 'genie' is often identified as Swiss architect Le Corbusier's 1915 drawing of the prototype bare concrete skeleton for multi-story residences known as the Dom-Ino house. The revolutionary change to building construction traditions that the adoption of RC moment frames for ordinary building construction represents can be seen particularly well in Fig. 3.15 of a Middle Eastern 3 story house where the remnant of the historic load-bearing stone masonry is relegated to be just a fragment while the self-supporting interior frame stands behind no longer dependent on it for support. The Dom-Ino house structural system became the icon of what he called the 'New Architecture'. As described by Le Corbusier's contemporary, Sigfried Giedion: "*Corbusier created … a single, indivisible space. The shells fall away between interior and exterior. … There arises … that dematerialization of solid demarcation … that gradually produces the feeling of walking in clouds*" [14].

From the Dom-Ino prototype, the reinforced concrete moment frame spread throughout Europe and the rest of the world including earthquake hazard areas. However, the 'dematerialisation' of the walls clashed directly with the usual enclosure requirements of completed buildings. As a result, although still not included in the engineering designs except as dead weight, masonry, the increasingly attenuated and weakened masonry walls did not disappear. This resulted in the problems specific to earthquakes enumerated above. Compounding this problem was the frequent use of open 'piloti' on the ground floor, as advocated by Le Corbusier. In earthquake engineering parlance, this became known as a soft or weak storey, which has become perhaps the single greatest threat to the safety of these buildings.

This transition to the nearly ubiquitous use of reinforced concrete frame construction with evidence of increasing numbers of failures of such buildings in each successive earthquake has presented a dilemma—a dilemma which only becomes evident when, as described above, certain disrespected and forgotten examples of traditional timber and masonry construction are found to have a better statistical record of collapse prevention in certain large earthquakes than the modern frame structures [6]. While timber lacing and timber frames with infill masonry intuitively are better than plain unreinforced masonry, *how can these, nevertheless, be better than reinforced concrete*? In other words, how can the failure of what is often determined after an earthquake to be flawed design and/or construction be considered to be an indictment against the RC frame systems themselves, when better designed and built RC buildings can be shown to have survived with little or no damage?

The issue can perhaps be seen as a basic dialectic that exists between the engineering of frames and walls. Engineering tools and methodologies—and code provisions—are used for both frames and walls, but the integration of the two systems in a single structure presents complexities. This is especially true when designing for design-level earthquakes for which inelastic yielding is not only probable, but fundamentally expected to occur. This behavior is accounted for in the building codes by ductility factors—factors which have been derived from extensive physical testing of what are now common modern materials and systems such as steel and RC, but not for archaic systems with combinations of timber and masonry that have better behavior than unreinforced masonry alone.

The most dramatic recent example of this is perhaps the 2010 earthquake in Haiti—where recent research shows that both late 19th century traditional timber and masonry houses of *colombage* construction and even more remarkably, the multi-story cement-block and concrete floor slab houses constructed in the informal settlements by itinerant owner-builders, out-performed the formal, contractor-built and often engineered buildings in the city center [15].

The performance of *dhajji* and *hımış* actually may provide insight into the better performance of the Haiti 19th century middle-class houses as well as large areas of the Haiti slum settlement houses. In my observations of the behavior of these systems in the Turkey earthquakes and the Kashmir and Gujarat earthquakes, the use of weak rather than strong mortars in combination with the timber framing allows the masonry to shift and slide early in the onset of earthquake shaking, rather than crack through the masonry units and fall out of the framework. The combination of the framework with the masonry thus is interactive, rather than one working against the other. "Frame action," the independent working of the frame as a structural system, is neither what exists nor what is important. Although a framework of timbers is constructed, it is imbedded in the masonry wall and "works" in the engineering sense of the term together with the masonry in the wall (Figs. 3.16 and 3.17).

This is exactly what John Ripley Freeman found when in 1932 he translated the 1909 Italian engineers Committee Report which includes a description of "*houses in Favellioni Piemonte [damaged in the 1905 earthquake were of] reinforced concrete framework … having walls of small hollow concrete blocks … while the framework remained intact the walls were damaged … in places detached from frame and*

Fig. 3.16 Two RC frame buildings in Bhuj, India after the 2001 Gujarat earthquake. The building on the left had been completed, while the frame only of an identical building had been completed on the right. The earthquake collapsed the one where the frame was restrained by the masonry infill walls, while the bare frame survived. Photograph by © Randolph Langenbach

Fig. 3.17 A collapsed RC building next to a surviving *hımış* building in Turkey after the 1999 earthquakes showing the resilience of the timber frame with masonry infill compared to that of a new RC frame building of modest height next door. It is interesting to compare this view with Fig. 3.16, where the infill probably caused the collapse, while here the infill and timber frame worked well together. Photograph courtesy of Adem Doğangün

collapsed," next to the report that "*wood-framed houses known as "Baraccata," built according to the ordinances of the Bourbon Government, … that had resisted all of the successive earthquakes … proved very successful*" [11, p. 568].

Thus, many buildings that we call "frame" structures are not frames in the engineering meaning of the term. More importantly, for earthquake resilience, it is important that they be understood not as frames, but as composite systems working more as solid walls that yield inelastically without collapse. The significance of this is that even if the framework is weak, the overall system can prove—as examples have already proved—to be resilient in large earthquakes.

Conclusion

The importance of *dhajji dewari* itself as an earthquake-safe form of construction has already been proven, at least in 2005 earthquake epicentral area in Pakistan. Following the earthquake, the Government of Pakistan's post-earthquake emergency management agency ERRA mandated reinforced concrete or concrete block construction for anyone receiving financial assistance for reconstruction of their houses. Despite this limitation, many local people—especially those in remote settlements where access to concrete blocks and steel was difficult—proceeded to build *dhajji* houses despite the lack of government assistance (Figs. 3.18 and 3.19). After a little over a year, through the recommendations of international relief workers in the field, the government approved *dhajji dewari*, and a year later approved the masonry bearing wall construction which in Pakistan is known as *bhatar*. In the half-decade that has followed these approvals, UN-HABITAT reports that 150,000 to 250,000 new houses in these traditional systems have been constructed in northern Pakistan throughout what are mainly rural areas.

Why is this better than to continue to require reinforced concrete? First, *dhajji* and *bhatar* are more economical and affordable—largely because they

Fig. 3.18 A rural general store building near Thub, Kashmir, Pakistan in the damage district after the 2005 earthquake. Photograph by © Randolph Langenbach

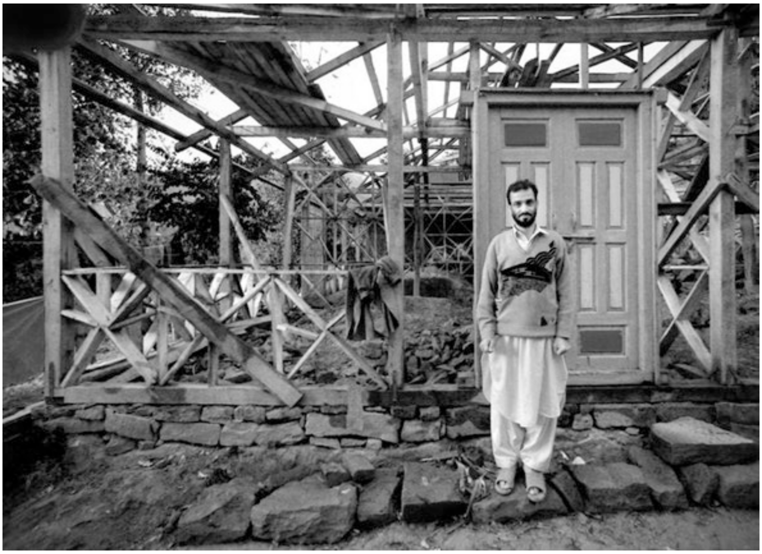

Fig. 3.19 The owner standing in front of his house under construction showing the light timber frame of *dhajji* construction before the rubble stone masonry with mud mortar infilling. His unreinforced rubble stone house had collapsed in the earthquake and he and his neighbors proceeded to rebuild with *dhajji* when they saw that the only house to survive in the village was the only one of *dhajji* construction and that the one building of RC construction had also collapsed. Photograph by © Randolph Langenbach

are primarily constructed of locally available materials: mud, stone and timber. Second, they are safer on average than RC houses for a simple reason: the safe construction of a multi-story RC moment frame requires a level of training and expertise that is not to be found even in most urban areas of Pakistan, and certainly not in the rural regions.

Not even the 1860 provision that "*imposed corporal punishment upon workmen who assisted in building structures which violated the provisions of the ordinance*" enacted in the Italian town of Norcia by the Pontifical government that controlled that part of Italy at that time—cited by Freeman—would be sufficient to assure the safety of all such buildings once RC construction became common even if re-enacted to cover such construction [11, p. 569]. With this, one can only imagine the scene of workmen, with their overalls dropped to their knees, being spanked for bad workmanship, and how often after earthquakes around the world where we may wish this had been done. Freeman did not include any further information on whether this was ever enforced in the Umbrian town of Norcia, which is located not far from the damage district of the devastating 2009 L'Aquila earthquake. More seriously, he did repeatedly emphasize the importance of the need for earthquake resistant construction that provides "*safety to human life, with greatest practicable economy … sought through use of local materials with which the people were familiar*" [11, p. 562].

Finally, although timber, which has traditionally been over-harvested in Pakistan, is used in these houses, most of it could be salvaged from the damaged and collapsed houses, and the amount needed for the rest meets sustainability quotas on the local forests. As the world moves from an era of profligate energy use to one where fossil fuels are gradually depleted, sustainability and green have become the catchwords in building design and construction. Wood is nature's most versatile renewable building material. Stone and unfired earth, together with wood, represent the most energy efficient materials that can be used for building construction. In addition, fired brick and lime mortar are materials which require far less energy to manufacture than Portland cement. Thus, identifying traditional vernacular construction practices that have performed well against one of the strongest forces that nature can throw at structures also can serve to provide a lens through which to see that the preservation of vernacular buildings represents far more than the saving of frozen artifacts. It is an opportunity for cultural regeneration—a reconnection with a way of building by people who, with a minimum of formal education, traditionally had learned how to build successfully for themselves with materials readily at hand.

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Chapter 4
Behaviour of the Borbone Constructive System Under Cyclic Loading: Preliminary Report

Nicola Ruggieri and Raffaele Zinno

Abstract The Borbone constructive system, constituted by masonry reinforced with timber frames, was conceived in the Calabria region after the earthquake of 1783. In fact, subsequently to this catastrophic event Ferdinando of Borbone the IV immediately drew up an anti-seismic code in which prescribed, among the other rules, the buildings reconstruction to be carried out with the reinforcement of a skeleton of timber elements. Also the residence of the Bishop of Diocese of Mileto-Nicotera-Tropea was built in Mileto (Vv), at the end of 18th c., adopting the Borbone anti-seismic system. This "*baracca*" of the Bishop was investigated with an accurate dimensional and structural survey, supported by petrographic and chemical analysis of mortars and stones and by assessment of the wood species of the inner members of the load bearing system. All these data allowed the designers to realize an "imperfectly" identical copy of the bishop's building wall. The paper will present the results of quasi-static cyclic tests according to UNI EN 12512:2003, on the full scale specimen of the Mileto masonry reinforced with timber framing. Furthermore, the hysteretic behaviour (ductility, energy dissipation, equivalent viscous damping ratio, strength impairment) of the Borbone constructive system will be discussed in the document, with a special focus on the single anti-seismic contribution of masonry and timber frame to the overall cyclic stiffness.

Keywords Reinforced masonry with timber framing · Cyclic test · Seismic behaviour

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Introduction

The Borbone anti-seismic system, so called *casa baraccata* in the 19th c., synthesizes an ancient constructive knowledge in the Calabria region and in particular realizes the instructions contained in the anti-seismic code. The latter were drawn up by Ferdinando the IVth of Borbone, immediately after the catastrophic earthquake that struck the Calabria region in the 1783.

Among the reconstruction buildings is also included the bishop's *baracca* of Mileto (Vv). It is characterized by the presence of a timber framing placed in the masonry internal part with in addition other wooden members in a continuous line in the horizontal direction and superposed at the corners.

A wall executed with such kind of constructive technique was reproduced in laboratory with the aim of subjecting it to cycling actions from which decode the wall seismic behaviour. In this specific case experimental tests have been carried out, at CNR Ivalsa in San Michele all'Adige (Tn), on a specimen made up of masonry and timber elements and a second one constituted by wooden framing with empty infill.

This document presents a preliminary analysis of the outcomes of the experimental campaign.

In literature, few cyclic tests were performed on historic timber frames; in fact the research is concentrated on new wooden panels with a completely lack of expertise on the dynamic behaviour of the Borbone timber framing.

Cyclic tests were performed on walls realized with a similar constructive techniques to the *casa baraccata* ones, the so called Gaiola, on several specimens characterized by different frame infill by Instituto Superior Técnico of Lisbon [1]. Other similar experimental proofs were conducted, on a real wall specimen extracted from an existing building, by Coias [2]. The University of Minho has tested some Pombaline timber frame units comparing them with other types retrofitted and strengthened by means of steel plates or bolts in the joints [3]. Experimental analysis of real scale models were conducted by Professor Ceccotti [4]. The tests on a wall reinforced with timber frames typical of Dolomites rural architecture have been carried out by means of two pairs of hydraulic pistons, fixed with chains at the four corners of the frame.

Description of the Model

A materials composition analyses was carried out on the Bishop's building wall, besides the dimensional survey. In particular analyses relative to mortar and stones of the panel were carried out at the Earth Science Department of University of Calabria, whilst the assessment of the wood species was carried out at the CNR Ivalsa in Trento. The mortar was predominantly composed by granite and carbonate rocks chips with average dimension about 2 mm; the binder level was about 55 % compared to the overall mortar and the inert presence was approximately the 40 %.

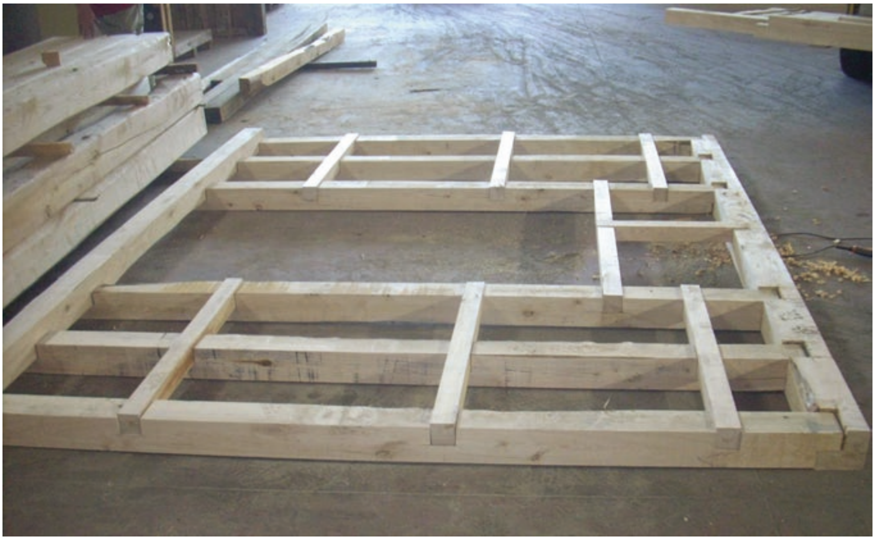

Fig. 4.1 The calabrian chestnut framing

The analyses of petrographic thin sections classified the rock as calcilitute.

The identified wood specie was the Calabrian Chestnut (Fig. 4.1).[1]

These results had provided data for carrying out an "imperfectly identical" copy of the Mileto wall in full scale to subject to cycling tests in the CNR Ivalsa laboratory in San Michele (Tn), Italy.

Therefore the specimens, basing on above results, has been executed with Calabrian chestnut (Castana sativa Mill.) frames devoid of wooden bracings, in which the stiffness under in plane horizontal actions is given by the infill masonry. The latter is made of different size voussoirs of Trentino Calcare, a kind of stone, that even more ancient than the Mileto one, it presents similar mineralogical and mechanical features to the stone detected by the petrographic analyses.

The mortar, characterized by an hydraulic binder, was composed by an aggregate-binder ratio of 1 to 2 and is constituted by a quartzose-granitic sand (grading 2–5 mm).

The mortar dried in about 1 month (August 2013).

The wooden members are constituted by a head beam with a cross section of 13×14 cm, the base member and all the posts of the frames of 12×12 cm, the horizontal elements are characterized by a measure of 7×7 cm. In plan the entire wall section is 41 cm, the length 339 cm, the height is 295 cm.

[1]Relative to this Calabrian wood specie, an experimental campaign performed at the CNR Ivalsa in Florence showed better mechanical features if compared to chestnut characterized by other European countries provenience. See Brunetti [5].

Fig. 4.2 Pyramidal nail and quartzose-granitic sand

All the nodes, half lap joints, are connected by pyramidal nail with maximum cross section dimension of 10 mm. These nails are clenched in the back side of the skeleton. Others wooden elements are placed in the masonry in addition to the timber framing, as in the original panel of Mileto (Fig. 4.2).

Test Set up and Protocol Procedure

The test set up was constituted by an hydraulic actuator with about 500 kN capacity that imposed an horizontal load at the top of the timber structure (Fig. 4.3).

The wall was instrumented by means of transducers to measure displacements in many points and have a complete and exhaustive overall structural behaviour.

In particular the LVDT (LVF047) (LPM052) measured the diagonal deformation; the displacement transducer (LPM009) was placed at the mid point of the upper part of the specimen and measured the relative horizontal displacement between the steel beam and the top of the wall; three transducers (LPM010; LPM011; LPM013) were arranged near the base of the system to measure horizontal and vertical displacements.

A computer processed the data provided by the transducers in real time.

The timber framing was fixed at the bottom using inclined steel nails to reproduce a too stiffness restraint and to counteract rocking movement. The upper part of the testing apparatus was characterized by lateral rollers that kept the top wooden beam of the wall and prevented its buckling. A uniformly distributed load (18.7 kN/m) was applied to the panel to replace the self weight of the timber post king truss of the Mileto building.

During the experimental campaign the laboratory had an air humidity of (65 ± 5) % and temperature of (20 ± 2) °C.

The cyclic test has adopted the procedures contained in UNI EN 12512:2003 (Timber structures—Test methods—Cycling testing of joints made with mechanical fasteners).

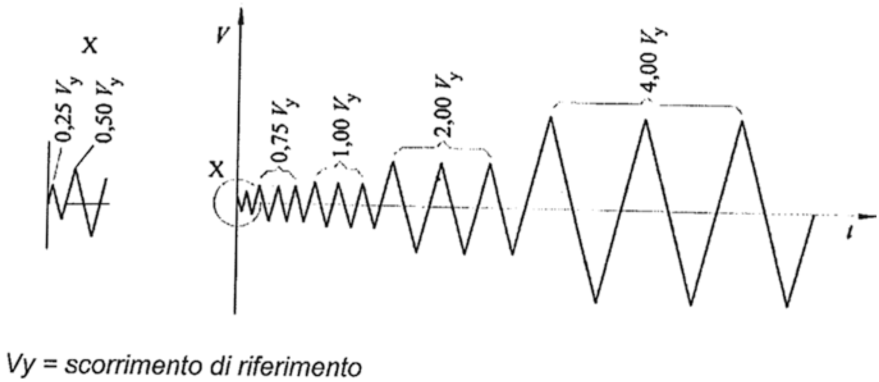

Fig. 4.3 The test loading procedure (UNI EN 12512:2003)

Fig. 4.4 The specimen before the cycling test

A number of cycle with increasing amplitude were imposed. In particular the horizontal load of the first cycle was applied until the reaching of a negative and positive displacement of 25 % the yielding displacement; until the 50 % yielding displacement value for the second cycle; the third, fourth, fifth and sixth cycle until, respectively, 75 %, same value of the V_y, twice value, the quadruple of the yielding displacement value and six times the V_y value.

The tests were conducted at a rate for all displacements of 0.2 mm/s. The models were subjected to a maximum displacement of 80 mm (Fig. 4.4).

Cycling Tests Results

Cyclic tests can simulate in a simple way the seismic action and provides data on global behaviour and shear resistance of the Borbone constructive system.

The experimental analysis, performed at CNR Ivalsa in Trento (Italy), included two cyclic tests, one on the timber frames with masonry infill and a second one on a specimen constituted by empty timber frames. The latter was tested to assess the anti-seismic contribute of the simple wooden framing to the global constructive system.

The tests on the real-scale models were carried out by means of load increments according to the UNI EN 12512:2003 procedure up to a 80 mm displacement. The resulting data and envelope curve of the force-displacement diagram provides information to determine ductility, energy dissipation and strength impairment, according to the definitions adopted.[2]

The ultimate limit state of the two tested models occurred for excessive deformation. In particular the specimen characterized by masonry reinforced with timber framing showed a lateral resistance until 103.64 kN in "tension" (corresponds to 59.18 mm of displacement, drift 2 %) and—101.62 kN in "compression" (displacement 79.02 mm for a 2.6 % drift), with an impairment of the strength, calculated between the first and the third cycle, for each ductility level, variable between a peak of about 13 % in "tension" charge to a maximum value of about 15 % relatively to a displacement of −40 mm.

The first cycles were characterized by a linear behaviour: for low accelerations the deformability of the wooden elements can bring back in the original position or near to it the masonry infill, simultaneously the masonry realizes a confinement of the timber frame (Tables 4.1, 4.2 and 4.3).

[2] Ductility is the capacity to undergo large displacement in the plastic field without a resistance impairment. It can be evaluated by means of the ratio between the ultimate slip and the yield slip. The energy dissipation is given by the area of the force-displacement loop for half cycle due to inelastic deformation (hysteresis). The hysteresis equivalent damping ratio is the ratio between the dissipated energy Ed and the in-put energy Ep, ($V_{eq} = Ed/2\pi\ Ep$) measured in this case for the 3rd cycle of each ductility level. The resistance impairment is the load decreasing due to the reaching of an identical displacement between the first and the third cycle See UNI EN 12512:2003, Timber structures—Test methods—Cycling testing of joints made with mechanical fasteners.

Table 4.1 Impairment of the strength (LPM021, top of the wall displacement)

Displacement (mm)		Force (kN)	Strength impairment (kN)	Strength impairment (%)
80	1st cycle	99.3	–	–
	2nd cycle	91.8	7.5	7.5
	3rd cycle	–	–	–
60	1st cycle	101.7	–	–
	2nd cycle	91.6	8.9	9.9
	3rd cycle	87.3	14.4	14.1
40	1st cycle	90.7	–	–
	2nd cycle	83.8	6.9	7.6
	3rd cycle	78.8	11.9	13.1
−80	1st cycle	−98.9	–	–
	2nd cycle	−87.5	11.4	11.5
	3rd cycle	−85.8	13.1	13.2
−60	1st cycle	−88.3	–	–
	2nd cycle	−79.1	9.2	10.4
	3rd cycle	−75.5	12.8	14.4
−40	1st cycle	−87.3	–	–
	2nd cycle	−76.6	10.7	12.2
	3rd cycle	−74.1	13.2	15.1
Average (1st−3rd cycle)			13.0	13.9

Table 4.2 Energy dissipation and hysteresis equivalent damping, half 1st cycle (LPM021, top of the wall displacement)

Displacement (mm)	E^{+}input (kN mm)	E^{+}d (kN mm)	V_{eq} (%)
80	3,982	1,557	6.2
60	3,066	1,194	6.2
40	1,824	810	7.0
20	699	323	7.3
−80	4,014	1,579	6.2
−60	2,707	1,168	6.8
−40	1,741	829	7.5
−20	639	358	8.9
Average			7.0

During the tests, the increase of the displacements generated inelastic deformation in the masonry with cracks in the mortar and an increment of the internal frictions, between masonry elements and at the interface infill masonry—wooden frame, in addition with slips and small expulsion of stones. Hence the energy dissipated, measured relatively to the 1st half cycle for each ductility level, was a value variable between 1,579 kN mm corresponding

Table 4.3 The table shows, relative to the pulling direction of the diagram, the values of maximum load, ultimate load, yielding load and displacement, initial stiffness and ductility

F [max][a] (kN)	103.6
V_u^a (mm)	79.1
F_u^a (kN)	100.6
F_y^a (kN)	66.1
V_y^a (mm)	10.5
α^a (kN/mm)	6.31
$\mu^a = V_u/V_y$	7.6

[a] Relative to positive direction

Fig. 4.5 The rocking mechanism of the wall

to 80 mm of "tension" displacement and 323 kN mm related to 20 mm of "compression" displacement (Fig. 4.5).

This provided data about the hysteresis equivalent damping ratio (V_{eq}) that was characterized by constant values between 6 and 7 % for each displacement analyzed; even if a peak of 8.9 % was recorded relatively to a negative displacement of 20 mm (1st cycle) (Fig. 4.6).

The maximum ductility value ($\mu = V_u/V_y$) reached by the tested model was 7.6. Namely the specimen has emphasized a ductility response.

The panel was characterized by a predominant flexural behaviour with a limited rocking mechanism and a posts uplift, which are related to the last cycles, as much

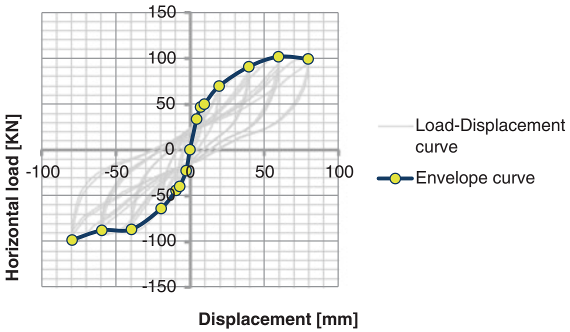

Fig. 4.6 Load-displacement envelope curve

as about 30 mm in tension and about 20 mm in compression and a consequent small wall rotation. A shear contribution was also present.

The wooden skeleton did not show evident cracks in the members and in the joints, they remained in elastic field at least relatively to the first cycles.

The tested model constituted by only wooden elements behaved in a weak manner, high deformation, hence showing the importance of the infill frame under seismic excitation. The specimen, characterized by a shear behaviour, did not show an evident rocking mechanism, with a maximum up lift in compression of 2.9 mm.

Conclusion

The experimental campaign demonstrates a proper behaviour of masonry wall reinforced by timber frames under horizontal force. The tested model (frames with masonry infill) pointed out a proper response under horizontal force with a high ductility, a no significant impairment of the strength and constant values, relatively to various ductility levels, of the hysteresis equivalent damping ratio.

The empty frames, tested to assess the infill framing properties, showed a poor mechanical capacity, with high deformation even in correspondence of low lateral forces.

The tests will provide data for numeric modelling in order to generalize the obtained outcomes to buildings characterized by load bearing structure realized in accordance with Borbone constructive system, but with different dimensional and infill arrangement features.

Furthermore the research represents an important step in the mechanical knowledge of the Borbone system and an acknowledgment of the cultural values the latter carries, a primary measure of safeguarding and a mood to encourage the integral conservation, original material, structural configuration and technique, of a fundamental element of the history of the anti-seismic science.

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Chapter 5
Seismic Performance of Traditional Half-Timbered Walls: Experimental Results

Elisa Poletti and Graça Vasconcelos

Abstract Half-timbered structures have been adopted as a typical seismic-resistant construction in many countries and they represent an important cultural heritage worth preserving. Little information is available on the performance of such structures when subjected to seismic actions; to overcome this lack of information, in-plane cyclic tests were carried out on real-scale half-timbered and timber frame walls, taking into consideration parameters such as vertical pre-compression and type of infill. Additionally, considering the great rehabilitation effort that has been taking place in the last decade, retrofitting solutions were proposed and tested on the already tested walls. Analysing the typical seismic parameters (e.g. ductility, energy dissipation, viscous damping), the influence of the type of infill and of the different retrofitting techniques were verified.

Keywords Half-timbered · Infill · Retrofitting · Steel plates · NSM · Energy dissipation

Introduction

Half-timbered structures constitute an important cultural heritage of many countries, since they represent a typical anti-seismic construction adopted worldwide [1, 2]; therefore, their preservation is of the utmost importance. Although recent earthquakes have pointed out the good seismic behaviour of this kind of structures, few experimental studies are available on the performance of traditional half-timbered walls and their retrofitting solutions and mainly only qualitative information is available of the seismic response of such structures. Due to this lack of information, an extensive experimental campaign was carried out, performing in-plane cyclic tests on real scale half-timbered

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and timber frame walls, adopting connections and dimensions encountered in real structures and considering different infill types (brick masonry and lath and plaster). Moreover, keeping in mind the great rehabilitation effort that has been carried out in recent years on these buildings, such in the case of the Portuguese Pombalino buildings in Lisbon, it is important to study the effect of strengthening on traditional timber-frame walls. To do this, retrofitting solutions have been applied to traditional walls and tested under cyclic loading. Both traditional and innovative techniques have been considered, namely bolts, steel plates and NSM steel flat bars. Results on the behaviour of both unreinforced and retrofitted walls are analysed and their seismic behaviour is discussed.

Experimental Campaign

To study the behaviour of traditional timber frame walls an extensive experimental campaign has been carried out taking into consideration parameters such as vertical pre-compression level, type of infill and type of strengthening.

Wall Specimens

The wall specimens chosen were built by specialized carpenters. All the connections between the vertical posts and the beams are half-lap joints, as well as the connections between the two diagonals of the St. Andrew's crosses, whilst the connections between the diagonal and the main frame are simple contact ones (see Fig. 5.1a). The walls were built in real scale, with realistic cross sections for all the elements (see Fig. 5.1b).

After the completion of the timber frame, part of the walls was filled with distinct types of infill to obtain the traditional half-timbered walls which characterize many cities in the world. Thus, besides the walls without any infill material, two additional groups of walls were considered, namely (1) timber frame walls with brick masonry infill and (2) lath and plaster walls. The use of different types of infill aimed also to assess its influence on the cyclic behaviour of timber frame walls.

For masonry infill, the masonry pattern consists of double leaf masonry with a transversal series of bricks every two rows of horizontal double leaf masonry, as detailed in Fig. 5.1c. It was decided to use modern materials available on the market mainly to represent what it is being done nowadays in rehabilitation works, in order to reduce rehabilitation costs.

Test Setup and Procedure

The cyclic tests were carried out using the setup illustrated in Fig. 5.2a. The application of the vertical load was done by means of vertical hydraulic actuators applied directly on the three posts of the walls and connected to the bottom beam through steel rods which connected the actuators to a hinge welded to the bottom

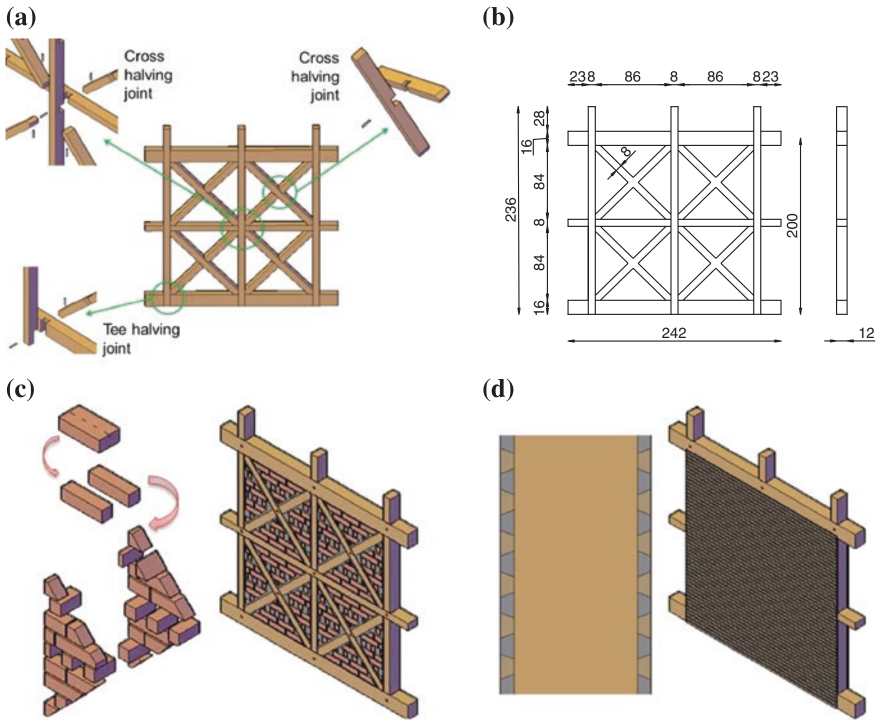

Fig. 5.1 Wall specimens: **a** connections used; **b** dimensions of elements in cm; **c** masonry infill; **d** lath and plaster

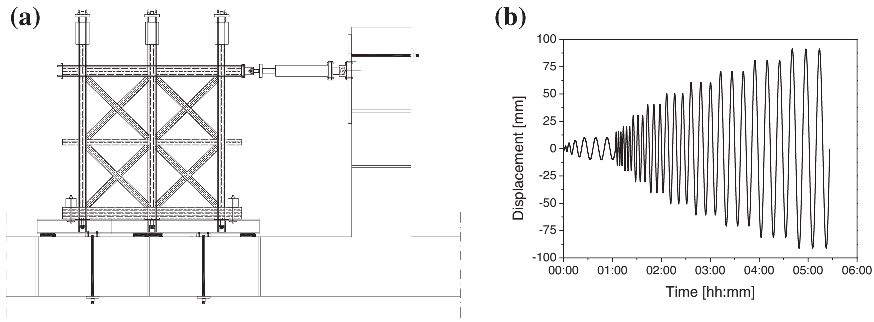

Fig. 5.2 **a** Test setup used in the experimental campaign; **b** test procedure

beam, so that the actuators were able to follow the horizontal movement of the wall. The horizontal displacement was applied to the top timber beam through a hydraulic servo-actuator, which was connected by means of a 3-D hinge to the reaction wall and a two-dimensional hinge was connecting it to the wall specimen.

The cyclic procedure adopted during the tests was based on standard ISO 21581 [3]. In order to better capture the highly non-linear behaviour of the walls, additional steps were added in the procedure, considering an increment in the applied displacement of 10 % (see Fig. 5.2b). Two different test speeds were adopted: one for displacement up to 10 % of the maximum displacement (namely 0.05 mm/s) and one for the remaining displacement levels (namely 0.35 mm/s).

Moreover, each wall type was subjected to two vertical pre-compression levels: (1) 25 kN/post, for a total of 75 kN, corresponding to the vertical load expected for these walls, considering their self-weight, the weight of the floors and live loads and (2) 50 kN/post, for a total of 150 kN, in order to take into account a possible change of use of the structure.

Type of Strengthening

The tested walls were repaired and retrofitted. The main repairs performed consisted of repairing the masonry blocks applying a fast setting natural cement, repairing damaged timber elements with a timber prosthesis or substituting a timber element when the damage was too extensive. Analysing the results of unreinforced timber frame walls, it was decided to strengthen the walls using three techniques, two traditional and one innovative. The first traditional technique adopted was to apply bolts to each half-lap joint of the main frame (Fig. 5.3a). This technique is only feasible for infill walls, since the infill confines the frame, guaranteeing a higher strength. The bolts used had a diameter of 10 mm and a total length of 160 mm and were a class 8.8 steel fasteners. They were inserted in pre-drilled holes. For this type of intervention, low tech equipment and workmanship are required; moreover the intervention is removable as well as economical.

The second traditional technique consists of applying steel plates to the main connections of the frame, as shown in Fig. 5.3b, c. For infill walls, custom steel plates made in zinc-galvanized steel and having a thickness of 3 mm were used, with a star-shape (see Fig. 5.3b) to better fit the frame. They were secured to the timber frame with bolts. For timber frame walls without infill, commercial steel plates were used (Fig. 5.3c). The steel plates were secured with bolts and screws, to better distribute the stresses in the plates. Two different configurations have been tested for timber frame walls, one linking the main elements of the connection, post and beam, to the diagonals and one linking only the main members, leaving the diagonals free of strengthening. Perforated plates (Rothoblaas plates PF703085 (140×400 mm^2)) made of steel S250GD and having a thickness of 2 mm were used.

The innovative solution adopted consisted of applying steel flat bars in the connections inserted with the near surface mounted technique (NSM). This technique could not be applied to the bottom connections, since the anchorage length was insufficient. Therefore, steel plates were applied, as done in the previous retrofitting solutions. To perform the retrofitting, cuts were opened in the elements with a plunge

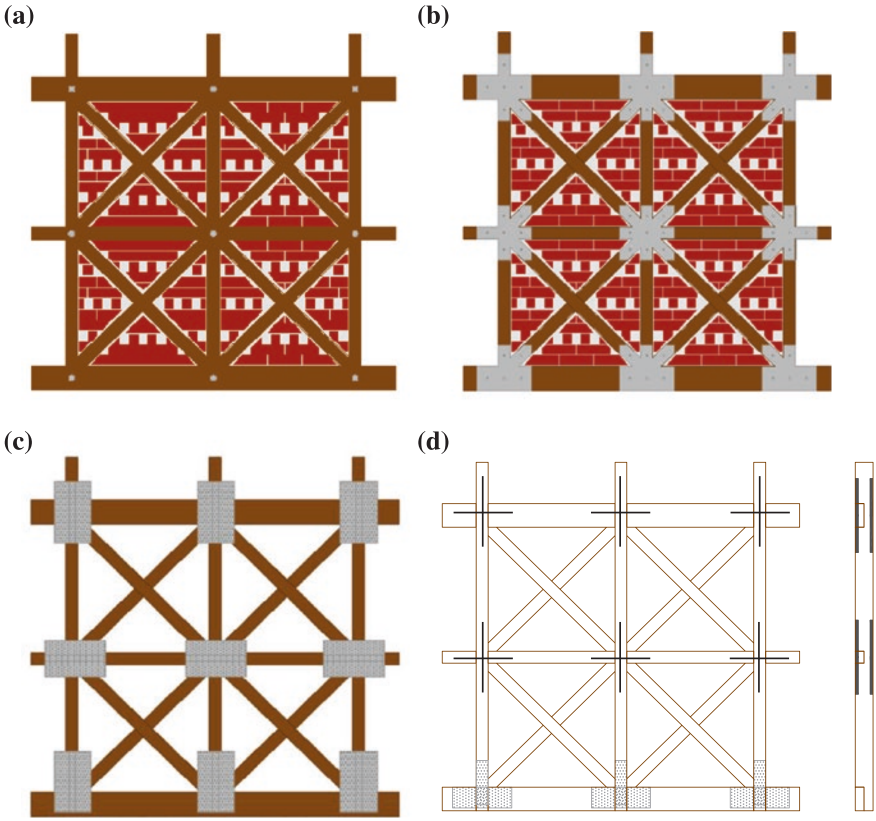

Fig. 5.3 Strengthening solutions adopted: **a** bolts; **b** custom steel plates; **c** commercial steel plates; **d** steel flat bars inserted with NSM technique

router, having a width of 12 mm and a depth of 23 mm to accommodate the flat bars with a section of 8 × 20 mm^2. The cuts were filled with structural timber glue, then the steel flat bars were inserted and additional glue was added if necessary to completely fill the cuts. This intervention is potentially invisible, but not removable. The flat bars were inserted in each connection (Fig. 5.3d) and had a cross-shape, being welded together with a notched connection in the middle.

Results on Unreinforced Walls

Cyclic test results performed on both infill and timber frame walls are here presented and a discussion of their general behaviour is reported.

All unreinforced half-timbered walls subjected to the same vertical pre-compression level present a similar behaviour. The walls tested with the lower vertical load

(UIW25) present a predominant rocking behaviour (see Fig. 5.4a), characterized by the S-shape of the force-displacement diagrams, with a significant vertical uplift of the posts. In Fig. 5.4, complementary to the hysteresis diagrams, the evolution of the vertical displacements measured by LVDTs placed at the bottom connections of the walls versus the top lateral displacement of the walls is also shown. It is seen that the lateral posts were uplifting as much as 50 mm, pointing out the important rotation experienced by the walls.

The rocking mechanism was evident even in masonry infill walls submitted to the higher level of vertical pre-compression load (UIW50), even if in this case a shear component was clearly present (Fig. 5.4b). The uplift of the vertical posts occurred during these tests too, in a lower amount, but it still conditioned the shape of the hysteretic loops. The walls exhibit a progressive loss of stiffness, even though the ultimate load does not differ greatly from the maximum one.

If the form of the hysteretic loops is compared with the uplift of the vertical posts for the same horizontal displacement, one can notice that: (1) the change in stiffness in the loading branch starts when the lateral posts start uplifting; (2) the plateaux that occur in the unloading branch of each cycle occur when the bottom connections start closing.

The increase of vertical pre-compression led to an increase of initial stiffness of 17 % and of load capacity of 65 %.

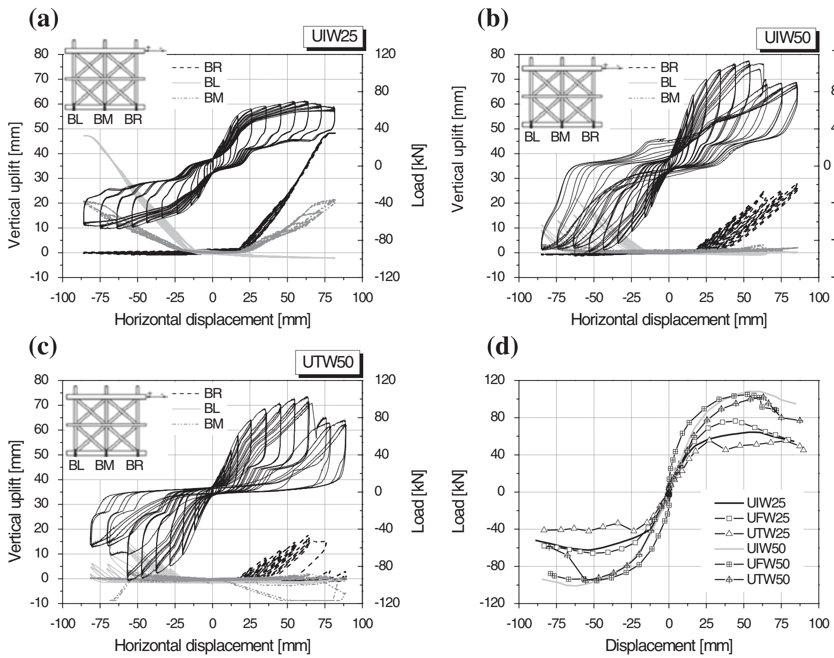

Fig. 5.4 Results on unreinforced walls: **a** UIW25; **b** UIW50; **c** UTW50; **d** envelope curves

The hysteresis diagrams found for lath and plaster walls (UFW) for both vertical load levels are very similar to those observed for half-timbered walls and will not be presented here. The confining effect given by the timber strips assured an important lateral stiffness, which resulted to be higher than that of masonry infill timber frame walls, as can be seen from the envelopes curves in Fig. 5.4d.

Timber frame walls (UTW walls) exhibited a different behaviour in relation to infill walls. The shear resisting mechanism predominated in the lateral response over the minor flexural component. But, as it can be noticed from the hysteretic diagrams (Fig. 5.4c), the walls experienced severe pinching. This appears to indicate that pinching can in a certain extent be avoided by the infill, both brick masonry and lath and plaster. Moreover, the unloading branch of the various loops is more regular, even if the plateau characterizing the post uplifting is still present.

As it can be deduced from the analysis done until now, the presence of infill greatly changes the response of timber frame walls to cyclic actions. The type of infill, though, does not appear to overly influence their behaviour.

Moreover, the amount of vertical pre-compression applied to the walls greatly influences their behaviour. It changes its response to cyclic actions, since a higher pre-compression leads to a stiffening of the wall and to a greater load capacity. UIW walls with brick masonry infill gained 64.7 % in terms of maximum load, while only losing 2.8 % in terms of ultimate displacement. UFW walls gained 29 % in terms of maximum load, but their ultimate displacement decreased by 2.8 %. In case of UTW walls an increase of vertical pre-compression resulted in an increase on the lateral resistance of 104.6 %, with a loss in terms of ultimate displacement of only 2.7 %.

Typical Damages

Typical damages for all walls were concentrated in the connections, pointing out the key role that they play in the overall response of the wall. Typical damages consisted in uplifting of the bottom connections (Fig. 5.5a), out-of-plane

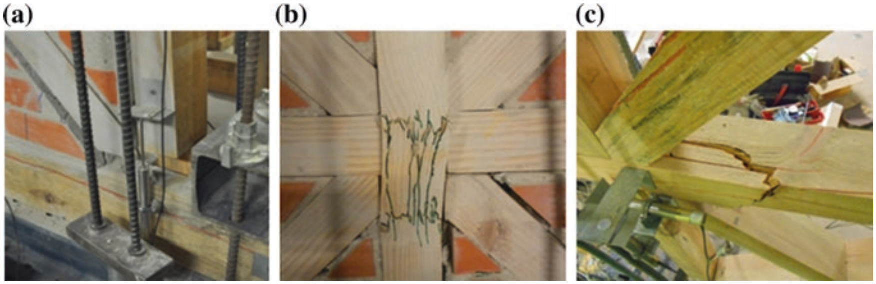

Fig. 5.5 Typical damages in walls: **a** uplifting of bottom connections in infill wall; **b** crushing of central connection in infill wall; **c** tearing off of half-lap joint in timber frame

opening of the connections and nail pull-out, cracks in masonry blocks, detachment between masonry and timber frame, crushing of the connection (Fig. 5.5b) (particularly for the higher vertical load level), tearing off of the connections (Fig. 5.5c).

Notice that heavier damages were experienced in timber frame walls, since the confining effect of the infill was not present and the timber frame suffered greater deformations. The timber elements experienced severe damages due to the shear effect of the diagonals, which would cut into the elements, tearing them off. On the other hand, the absence of infill led to minimal uplifting in timber frame walls, preventing the rotation of the wall.

Results on Retrofitted Walls

All retrofitted walls showed an improvement in comparison to unreinforced walls, being it in terms of strength and stiffness or in terms of general behaviour of the wall.

Comparing the hysteretic behaviour of the strengthened walls with bolts at connections and the corresponding unreinforced half-timbered walls, it is observed that there is no great gain in terms of ultimate capacity and stiffness. In fact, for the lower vertical load level, the gain in terms of maximum load was of 23.7 %, while for the higher vertical load level it lost 5 %. In terms of ultimate displacement, the walls gained 5.7 and 0.2 % respectively (Fig. 5.6a).

However, the general behaviour of the walls changes; the plateau caused by the uplifting of the vertical post from the base beam is still clearly present, but it is less pronounced and the unloading branch of the cycles is smoother. In fact, vertical uplifting in the posts decreased of approximately 40 % for both load cases. The bolts contribute to the resistance to tensile forces present in the bottom half-lap connections, ensuring a degree of continuity to the bottom connections.

For infill walls retrofitted with steel plates (Fig. 5.6b) the gain in terms of maximum load is considerable, reaching an increment of 60.4 %. Moreover, it is observed that the initial stiffness increased by 14 % and the ultimate displacement is of the same size order. However, the displacement imposed to the walls does not correspond to the maximum displacement capacity, but due to limitations of the equipment it was not possible to continue the test. A similar behaviour was observed for both vertical load levels. It should be pointed out that the use of star shape steel plates, linking the main elements of the connection (post and beam) to the diagonals gives a significant additional strength and stiffness to the wall. In fact, for this type of strengthening, the values of initial lateral stiffness are comparable for the two vertical load levels, meaning that for such a strong retrofitting technique, the effect on the amount of vertical load becomes secondary.

For timber frame walls it was observed how linking the diagonals to the main frame greatly stiffened the walls inducing out-of-plane movements [4], thus a second strengthening was done linking only the main elements of the frame

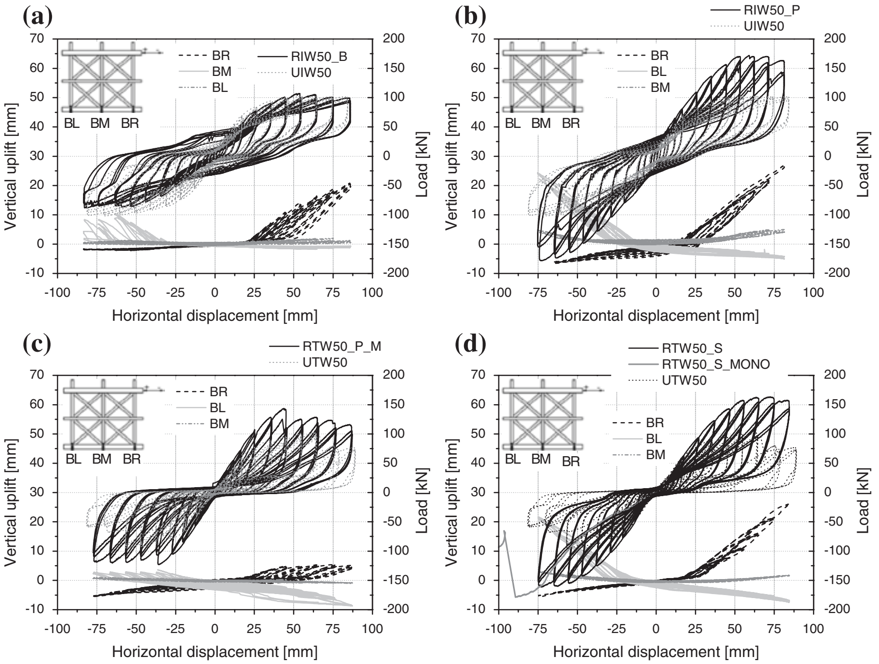

Fig. 5.6 Results on retrofitted walls: **a** bolts strengthening; **b** steel plates strengthening, infill wall; **c** steel plates strengthening, timber frame wall; **d** NSM strengthening, timber frame wall

(post and beam) with the steel plates. This solution allowed the walls to gain significantly both in terms of stiffness and load capacity, without compromising the displacement capacity (see Fig. 5.6c). In fact, in terms of maximum load, the walls gained 183 and 35 % for the lower and higher pre-compression load respectively, while lost 5 and 3.5 % in terms of ultimate displacement respectively. On the other hand, this retrofitting solution led to severe pinching in timber walls. Similarly to retrofitting with custom plates, also in this case, the vertical load has only marginal influence in terms of maximum load, even if it influences the initial stiffness, being higher for the higher vertical pre-compression.

Concerning the NSM steel flat bars retrofitting, it was observed that it was more appropriate for timber frame walls, since the confining effect of infill did not allow to take full advantage of the deformation capacity of the timber elements with embedded bars. Low values of strain were recorded at the bars in infill walls and the improvement in terms of lateral resistance was of 62 and 30 % for the walls submitted to the lower and higher pre-compression load levels respectively. Nonetheless, the behaviour of the wall changes from a flexural one to a mainly shear one, so the retrofitting was able to guarantee the expected results.

For timber frame walls, from the comparison between the unreinforced and retrofitted walls (Fig. 5.6d) it is clearly visible a considerable improvement of the lateral response, with the increase on the lateral resistance of about 197 and of 64 % for the retrofitted walls submitted to the lowest and highest levels of pre-compression respectively. Notice that in this case too, the vertical load level has minimal influence on the results, reaching similar values of load capacity and stiffness. Since no severe damage was observed in the walls, the timber frame wall was submitted to a monotonic test after the cyclic test in order to characterize the failure mechanism. The lateral resistance obtained of 179 kN (Fig. 5.6d) was 121 % higher than the one recorded in the unreinforced timber frame wall, which confirms that the increase on the lateral resistance recorded in the cyclic test did not mobilize all the contribution of the steel flat bars and does not correspond to the failure configuration of the wall.

All walls strengthened with steel flat bars experienced pinching, but it was more severe for timber frame walls, as observed for walls retrofitted with steel plates. It appears that pinching manifests itself more when there is less confinement, being it given by the infill or by the strengthening.

Typical Damages

As was the case for unreinforced walls, in retrofitted specimens too the damages are concentrated at the connections. The retrofitted walls experienced severe damages after failure, contrary to what observed in unreinforced walls. Nonetheless, in most cases, the strengthening was still able to work and guarantee an adequate resistance of the wall.

Walls strengthened with bolts exhibited severe damages. The walls developed damages in the central connections, which failed with tearing off of the central beam (Fig. 5.7a) and crushing of the central post; additionally, the nailed connections between the diagonals and the main frame detached.

In the case of walls retrofitted with steel plates the damages observed were similar for all walls and they consisted in: (1) failure of the half-lap connection linking two diagonal member in a cell for walls in which the steel plates linked the diagonals to the main frame (Fig. 5.7b); (2) failure of the central middle connection for walls where the diagonals were not linked to the main frame through the steel plates (Fig. 5.7c). The creation of strong retrofitted points (steel plates linking main frame with diagonals) resulted in the failure of the weakest zones of the wall, which were the half-lap connections of the diagonals. Notice that no damages were observed in the main wood members of the connection. An ovalization of the holes for the bolts in the diagonals was observed for timber frame walls too. When the diagonals were free to move, the failure occurred in the main member of the frame (Fig. 5.7c), due to the shear action imposed by the diagonals elements.

For infill walls with NSM retrofitting plastic deformations were observed in the steel bars, but no failure occurred. For the timber frame wall tested monotonically at the end of the cyclic test, failure occurred at the central connection, see Fig. 5.7d, associated to the failure of the bar and further propagation of cracking in the wood. With the deformations reached, the approximate strength estimated

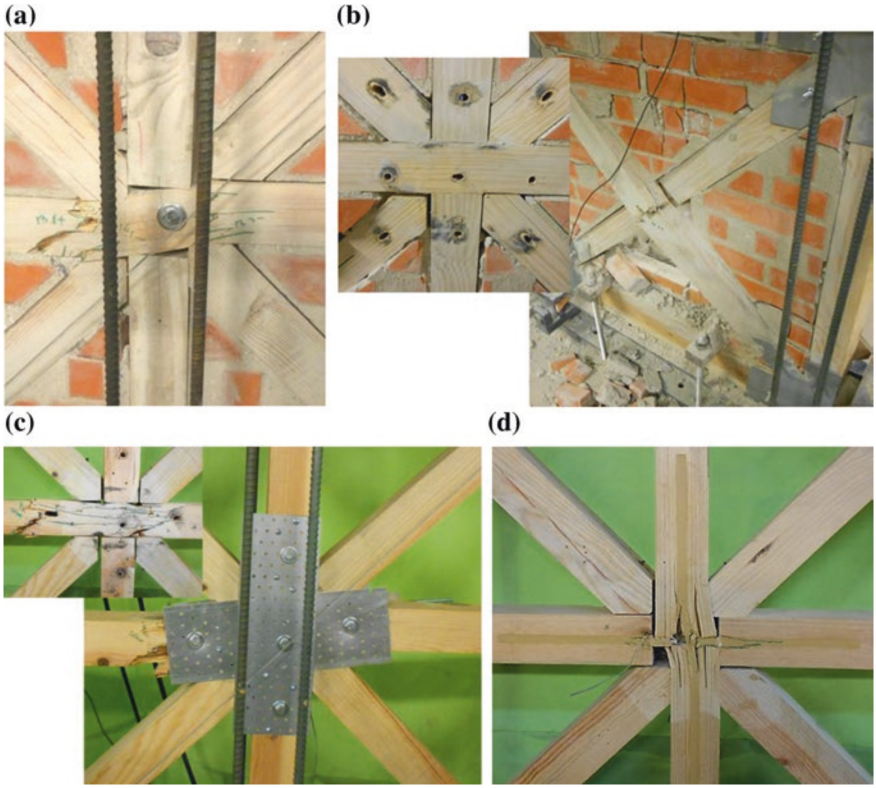

Fig. 5.7 Typical damages in walls: **a** tearing of timber element in infill wall retrofitted with bolts; **b** failure of half-lap joint of diagonals in infill wall retrofitted with steel plates; **c** failure of central connection in timber frame wall retrofitted with steel plates; **d** failure of steel bar at central connection in timber frame wall

in the bars for a deformation of 6 % was of 627 MPa, a value 50 % higher than the yield strength. For both wall typologies, the bars deformed in the plastic regime.

Seismic Parameters and Comparison

In the seismic design of new timber structures or in the rehabilitation of existing structures, including historic timber frame walls, the study of the seismic performance is of paramount importance. Since the seismic response of timber structures is very complex and time dependent, a better understating of the hysteretic factors that govern the problem is important for a safe and economical seismic design or for the adoption of the most adequate retrofitting measures.

A major parameter used for the assessment of the seismic performance is the ability of a structural element to dissipate energy during cyclic testing. The energy dissipated by the walls at each cycle is computed by calculating the area enclosed by the loop in the load-displacement diagram and it represents the amount of energy dissipated during cyclic loading. Energy can be dissipated through friction in the connections, yielding of nails, yielding and deformation of bolts, steel plates and bars and permanent deformation accumulated in the walls as observed during the tests.

Other parameters that can be analysed are initial stiffness and stiffness degradation, ductility and equivalent viscous damping ratio.

Dissipated energy for unreinforced walls is presented in Fig. 5.8a. Timber walls presented the lowest amount of dissipated energy for both vertical load levels. This can be attributed to the strong pinching present in these walls, which clearly diminishes the dissipative capacity of the walls. For both load levels, lath and plaster walls had the highest dissipative capacity, showing how this alternative infill guarantees a good seismic behaviour.

Taking into account the dissipative capacity of retrofitted walls (Fig. 5.8b), all retrofitting techniques adopted were able to guarantee greater energy dissipation during the tests. The highest dissipative solution is provided by retrofitting with steel plates linking the diagonals. For the walls tested without linking the diagonals, the dissipative capacity was lower.

Retrofitting with bolts showed results comparable to the ones obtained in unreinforced walls, improving only for high values of drift in case of the higher precompression load, given that the solution changed the failure mode of the wall.

NSM retrofitting guaranteed higher values of dissipated energy when compared to unreinforced ones for both infill and timber walls, being the difference more significant for timber frame walls.

The vertical load level tended to increase moderately the dissipative capacity of the walls.

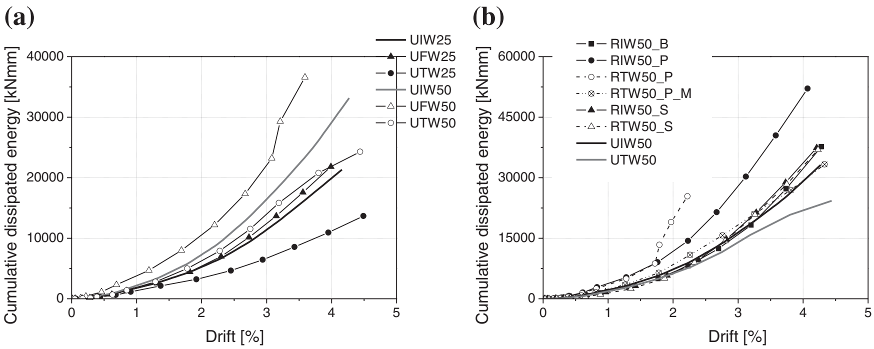

Fig. 5.8 Cumulative dissipated energy: **a** average values of unreinforced walls; **b** values of retrofitted walls, higher vertical load

Table 5.1 Comparison among values of seismic parameters for unreinforced and retrofitted walls

Vert load (kN/post)	Parameter	Infill walls				Timber frame walls			
		UIW	Bolt	Steel plate	NSM	UTW	Steel plate with diag	Steel plate no diag	NSM
25	Max load (kN)	63.85	76.86	157.36	102.99	48.92	177.30	139.42	145.06
	ULT DISPL (mm)	84.35	97.60	79.02	81.68	87.05	76.06	82.80	76.98
	Initial stiff-ness (kN/mm)	3.03	1.63	3.98	4.19	2.14	3.80	2.78	4.19
	Tot cum energy (kN mm)	21,332	20,931	41,840	26,633	13,679	22,333	31,734	35,668
	Viscous damping	0.12	0.13	0.12	0.12	0.12	0.12	0.13	0.13
50	Max load (kN)	105.19	86.53	175.09	136.62	98.64	193.84	133.19	162.03
	ULT DISPL (mm)	81.89	84.30	77.76	80.71	84.73	55.35	81.76	79.62
	Initial stiff-ness (kN/mm)	3.75	2.96	4.28	3.57	3.16	4.76	4.06	4.06
	Tot cum energy (kN mm)	33,154	37,675	52,097	37,424	24,279	25,388	33,290	36,950
	Viscous damping	0.12	0.13	0.13	0.12	0.13	0.12	0.14	0.12

Table 5.1 shows the values of the parameters for all walls. In terms of maximum values, apart from retrofitting performed with bolts, generally steel plates and NSM steel flat bars retrofitting techniques tended to play a major role in the lateral resistance of the walls, reaching an increase in terms of maximum load capacity up to almost 200 %, for the lower load level and 70 % for the higher. For all kind of strengthening, the loss in terms of ultimate displacement was usually in the order of 3 % or less. Considering the values of viscous damping, the values obtained for retrofitted walls are similar for all retrofitting solutions, pointing out that an innovative retrofitting technique could be a comparable alternative to a traditional one when approaching a strengthening problem.

Future Developments

Future developments for this work include the definition of an analytical hysteretic model for traditional timber frame walls based on experimental results and execution of numerical models calibrated on the experimental results obtain in order to perform parametric analyses.

Conclusions

Aiming at gathering a better insight on the seismic response of traditional timber frame walls, characteristic of ancient construction all over the world, and on the improvement of their seismic performance, an experimental campaign was designed based on static cyclic tests, taking into consideration different types of infill and different retrofitting solutions.

As concluding remarks, it is important to point out some results of this work: (1) the presence of infill changes considerably the response of the walls in terms of predominant resisting mechanism, due to the confining effect on the timber frame; (2) the retrofitting technique with steel plates greatly increased the stiffness of the walls, particularly when the diagonal elements were linked to the main frame, while NMS steel flat bars proved to be more appropriate for timber frame walls, since the deformation of the timber elements was not hindered by the infill and exploitation of the flat bars was greater; (3) for all walls, damages were concentrated in the connections, pointing out their key role in the response of the walls.

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Chapter 6
Experimental Study on Timber-Framed Masonry Structures

Andreea Duţu, Hiroyasu Sakata and Yoshihiro Yamazaki

Abstract Timber-framed masonry buildings are found all over the world, from common residential houses to important cultural heritage buildings. However, there is no design standard or published method to systematically evaluate the seismic capacity of this type of building. Therefore, an experimental study was developed to assess the mechanical behaviour of these structures. This study uses the construction details of Pombaline style Portuguese timber-framed masonry heritage buildings but without timber diagonals (St. Andrew's cross). This Pombaline style timber-framed masonry structure without diagonals can be found in other countries such as Romania, Italy, Nepal, and China. Given the cultural heritage value of these types of buildings a research project was initiated at the Center for Urban Earthquake Engineering of Tokyo Institute of Technology with the objective of quantifying the effect of different seismic retrofitting methods using aramid fiber sheets in different layouts. However, to evaluate the effect of different retrofitting methods the evaluation of a non-retrofitted structure is needed first. This paper presents the results of an experimental program conducted on non-retrofitted masonry panels, timber frame, and a timber-framed masonry wall.

Keywords Timber · Masonry · Assessment

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Diagonal Compression Tests on Masonry Panels

Diagonal compression tests on unreinforced masonry panels were performed. Such "indirect" tests are usually preferred over "direct" shear-compression tests because of their short duration and setup preparation time, reduced cost, and ability to provide estimates of mechanical properties used in analytical models [1]. The results of the tests are considered according to ASTM E 519-07 [2]. This test method was used with smaller than normal test panels to evaluate the effects of variables such as type of masonry unit and mortar and workmanship. The dimensions of the masonry panels were chosen to match the dimensions of the masonry panels within the timber frame.

Masonry panels with dimensions of 100 × 870 × 852 mm were tested. The masonry panels were made of burned clay bricks with dimensions of 100 × 60 × 210 mm and mortar joints with 12 mm thickness (Fig. 6.1). Diagonal compression tests were carried out on three panels, each built with the same characteristics. In ASTM E 519-07, a shear stress state of zero confining pressure is assumed at the centre of the specimen, resulting in a shear stress equal to both tensile and compressive principal stresses. Diagonal cracking is then assumed to occur when the masonry tensile strength f_t is reached, where f_t is equal to the shear strength at zero confining stress τ_0. Based on displacement readings on both sides of the panel (Fig. 6.2), axial strains along a single direction were derived as the average displacement divided by the gage length. Figure 6.1 shows the test setup.

Fig. 6.1 Dimensions of masonry panels

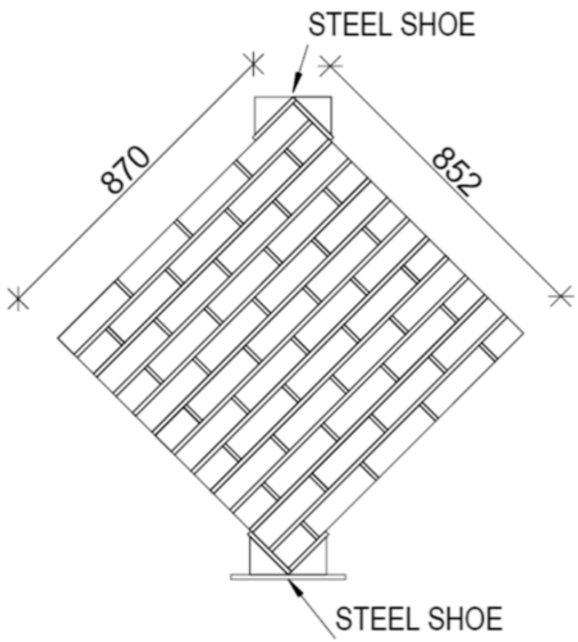

Fig. 6.2 Position of displacement transducers

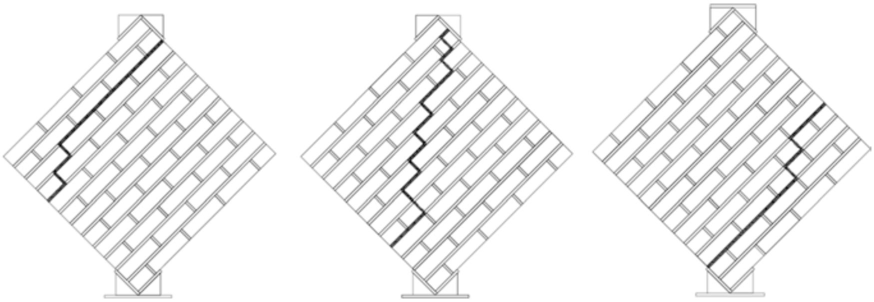

Fig. 6.3 Failure modes of masonry panels. Masonry panel1 (M1). Masonry panel2 (M2). Masonry panel3 (M3)

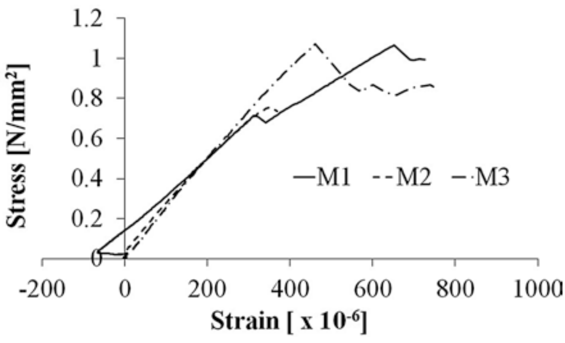

Fig. 6.4 Stress–strain *curves* for compression

Two of the masonry panels suffered shear sliding involving both bed and head joints in a stepwise way (Fig. 6.3, M1 and M3). Bed joint sliding typically produces higher energy dissipation. Specimen 2 suffered diagonal cracking involving both mortar joints and masonry units (Fig. 6.3, M2).

Figure 6.4 presents the stress–strain curve for compression. The stress was calculated as the force divided by the cross sectional area.

Figure 6.4 shows that masonry panels that failed through joint sliding (M1 and M3) showed higher shear strength than the masonry panel that failed through diagonal cracking (M2). For timber-framed masonry structures the weakest component is typically the mortar. Therefore, the failure should theoretically occur due to mortar cracking in the joints that causes bricks to slide, which is the assumed behaviour of the panel within the studied structure type. Figure 6.3 shows that for specimens 1 and 3, the failure mode was similar to the assumed one.

Static Cyclic Tests on Walls

Two types of wall specimens were tested, one timber-frame (S1) and one timber-frame with masonry infill (S2) (Fig. 6.5).

Figure 6.6 shows the dimensions of the wall specimens. The timber frame of both specimens had the same dimensions. The connections between the timber elements were cross-halving type, reinforced with screw nails (Fig. 6.7).

Test Setup

The tests were carried out on a reaction frame with a 200 kN capacity actuator and jack stroke of 500 mm. For S2 a vertical force of 60 kN was initially introduced through steel tie rods (Fig. 6.8). This value was calculated as the equivalent force

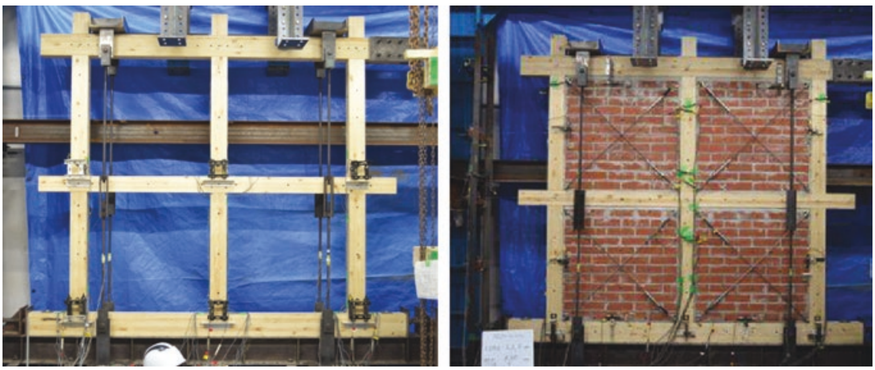

Fig. 6.5 Wall specimens subjected to cyclic static tests under vertical loading. Pure timber-frame (S1). Timber-frame with masonry infill (S2)

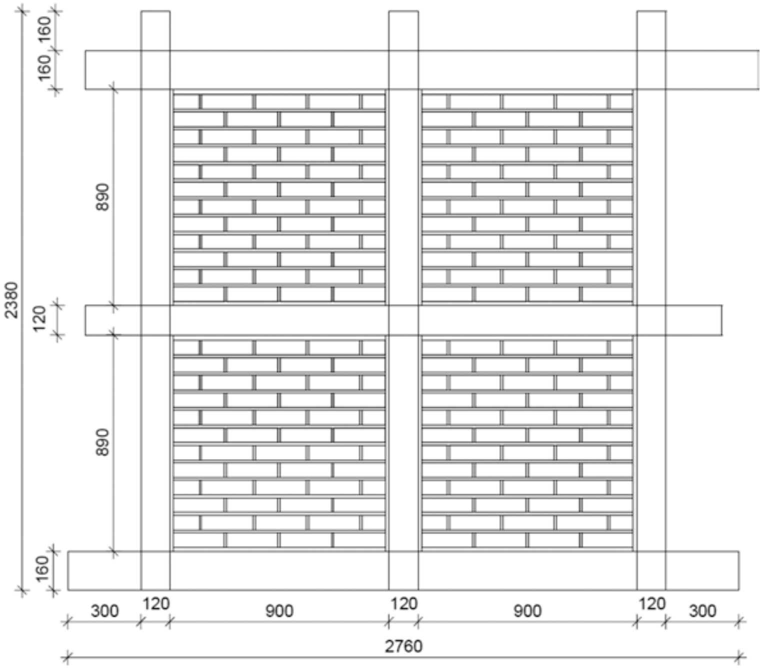

Fig. 6.6 Timber framed masonry wall

acting on a first floor wall in a four storey tall building. The vertical force was uniformly distributed on the top of the wall specimen using steel plates connected to the upper beam with screw nails. For S1, only 30 kN were applied.

The CUREE—Caltech standard loading protocol for wood frames (Fig. 6.9) was used for two reasons; first, because the failure modes appear to be the most consistent with seismic behavior [3], and second, because the same protocol was used in a previous experimental program developed at the Technical University of Lisbon.

The CUREE protocol was specifically developed for wood frame shearwall testing and is based on the hysteretic response of wood frame structures [3]. The ASTM E 2126-02a standard includes the CUREE protocol as one of the loading options [4]. The loading protocol consists of initiation cycles, primary cycles and trailing cycles. Initiation cycles are executed at the beginning of the loading history. They serve to check loading equipment, measurement devices, and the force-deformation response at small amplitudes. A primary cycle is a cycle that is larger than all of the preceding cycles and is followed by smaller cycles, which are called trailing cycles. All trailing cycles have amplitudes that are equal to 75 % of the amplitude of the preceding primary cycle [4].

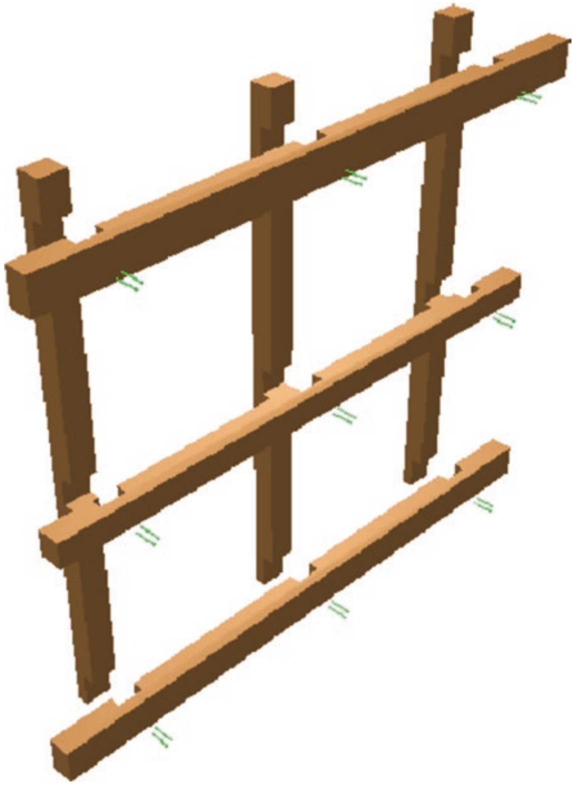

Fig. 6.7 Cross-halving connections reinforced with screw-nails

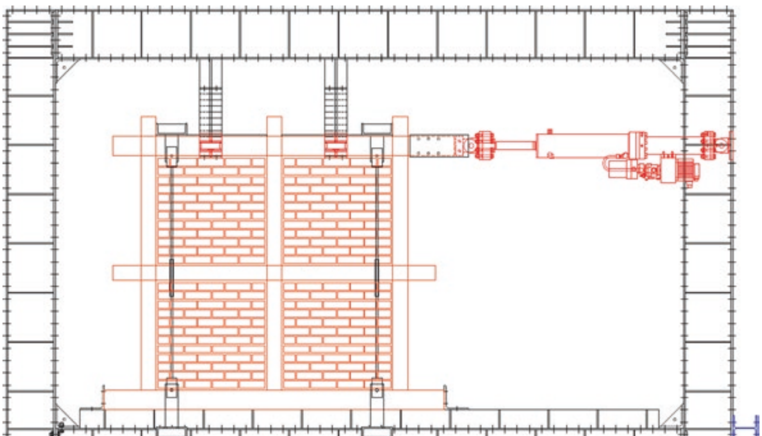

Fig. 6.8 Test setup for the timber framed masonry panel

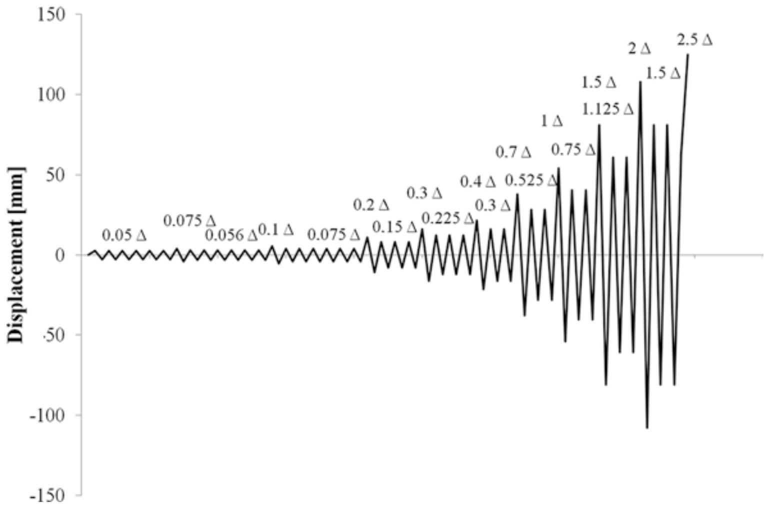

Fig. 6.9 CUREE loading protocol

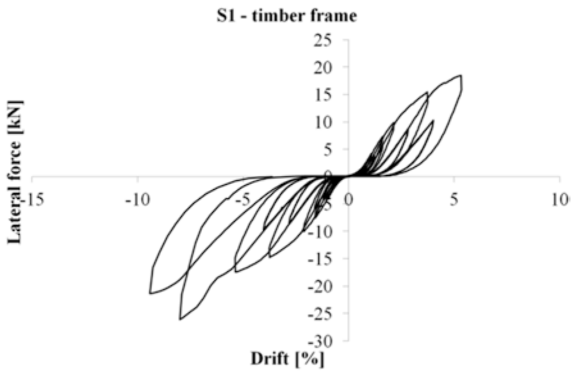

Fig. 6.10 Lateral force–drift diagram of S1

S1 Test Results

The purpose of this test was to establish the response of the timber-frame with no masonry infill, so that the effect of the masonry infill in test S2 could be isolated and examined directly. The hysteretic behaviour (Fig. 6.10) shows a reduced stiffness, a high ductility and important slippage in the timber connections. The rotation angle in the connections was similar to the shear angle, which means that the connections were flexible and did not allow the timber elements to bend.

After reaching the maximum stroke of the jack used to introduce the lateral force at 5.33 % drift, monotonic loading was performed until 9.4 % drift was reached. The wall specimen did not exhibit a distinct failure mode, but the upper timber beam cracked starting from the middle connection (Fig. 6.11). Moment diagrams shown below (Fig. 6.12) indicate that the connections were relatively strong in-plane, with the bottom connections carrying a moment around 2 kNm, while the middle connections carried a moment around 1 kNm.

The moment–rotation relationship was obtained by measuring the strain (ε_1 and ε_2) and the displacements (D_1 and D_2) as indicated in Fig. 6.13, and using Eqs. (6.1), (6.2) and (6.3). In this case, the distance between the strain gauges was the same as the one between the displacement transducers (L).

$$\theta = \frac{D_1 - D_2}{L} \tag{6.1}$$

$$M = EI\varphi \tag{6.2}$$

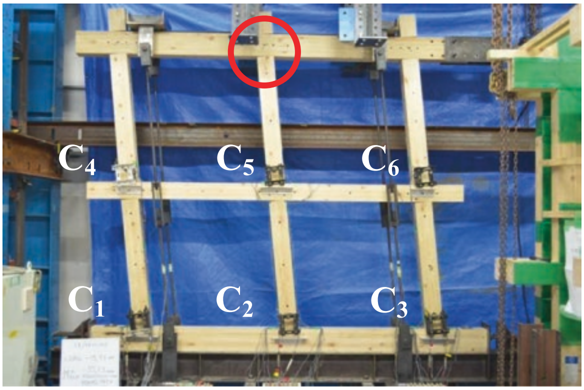

Fig. 6.11 Deformation of S1 at 9.4 % drift and the slight damaged area highlighted

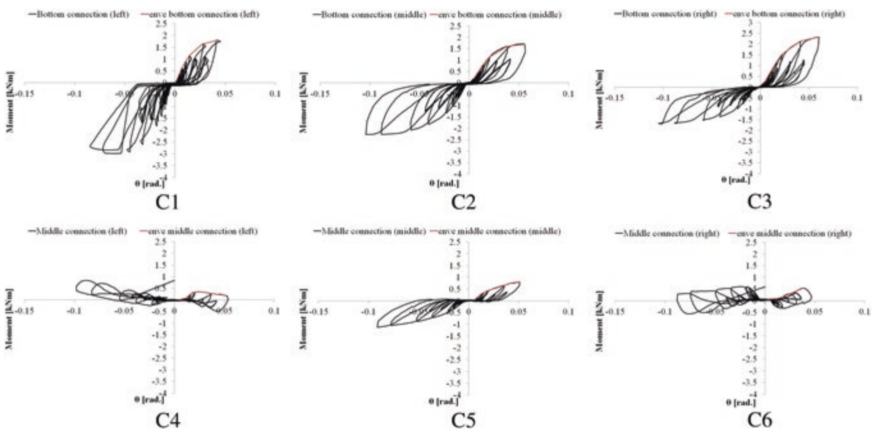

Fig. 6.12 Moment–rotation diagram for the connections of S1 (for location see Fig. 6.11)

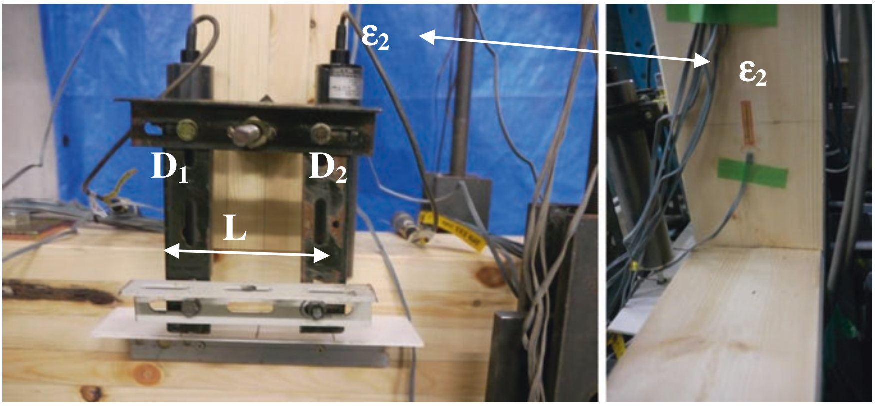

Fig. 6.13 Instrumentation to determine the moment–rotation relationship

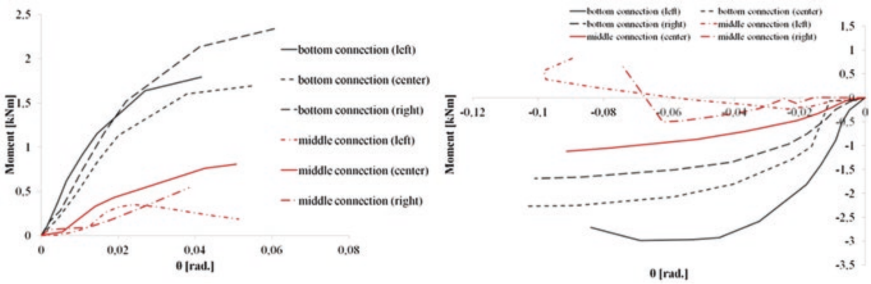

Fig. 6.14 Comparison of moment–rotation diagram for all of the studied connections in positive loading (*left*) and negative loading (*right*)

$$\varphi = \frac{\varepsilon_1 - \varepsilon_2}{L} \tag{6.3}$$

Figure 6.14 shows the envelope diagrams for both positive and negative loading.

Figures 6.12 and 6.14 show that each connection behaves differently depending on its neighbouring elements. Therefore, the prediction of the moment is mainly considered for the middle centre connection.

S2 Test Results

Figure 6.15 shows the hysteretic behaviour of the timber-framed masonry panel (S2) represented in terms of lateral force at the top of the wall specimen with respect to the height (drift). It shows important stiffness and ductility characteristics of the system. Wall specimen S2 failed at 5.3 % drift due to shear forces on the upper horizontal beam, which in turn caused the bending failure of the right column (Figs. 6.16, 6.17 and 6.18).

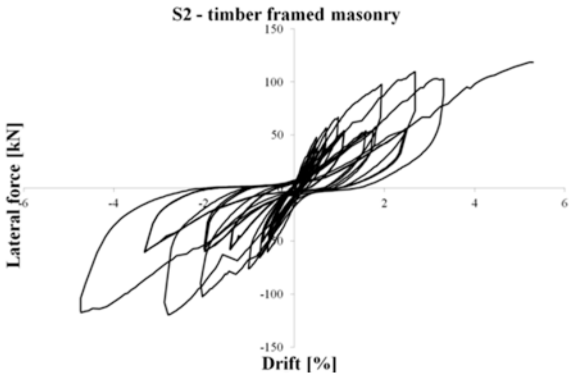

Fig. 6.15 Lateral force–drift diagram of S2

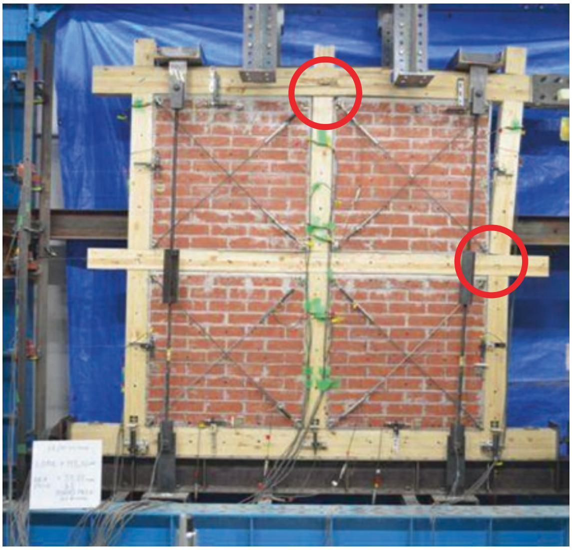

Fig. 6.16 Failure mode of S2

Fig. 6.17 Damage of the *upper* timber beam, which fractured in tension

Fig. 6.18 Damage of the *right* column, which failed in shear (*back side*)

Fig. 6.19 Damages in the *middle* joint

Stiffness degradation was directly proportional to cyclic loading and was strongly influenced by reverse cycles. Masonry panels separated from the timber frames from the first cycle. The masonry started to crack in the bottom right panel from the eighth cycle, and from the bottom left panel in the fourteenth cycle. The masonry was considered to have failed when one crack visibly extended from one end of the panel to the other. Although the first masonry panel failed around 1.93 % drift at 97 kN lateral force, the masonry infill continued to ensure the system's stiffness and to dissipate energy until the timber frame failed. The cracks in the masonry panels started from the bottom and continued with every new cycle, layer after layer, towards the top of the panel.

The middle timber joint showed significant damages at 0.7 % drift and was subjected to strong shear forces, resulting in compression by the masonry perpendicular to the grain of timber elements (Fig. 6.19). The upper masonry panels did not show any cracks, but near failure at 4.7 % drift the left masonry panel exhibited out of plane displacement (Fig. 20).

Fig. 6.20 Out of plane displacement of the *upper* right masonry panel

Comparison of S1 with S2

Comparison of S1 with S2 (Fig. 6.21) shows that the masonry infill significantly influenced the strength of the system. For S2, the masonry carried lateral forces even after it cracked. The timber frame had a significant contribution to the ductility of the wall, behaving differently than reinforced concrete frames with masonry infill where after separation damage usually appears in both the infill and frame. In timber framed masonry the infill and frame separate early, but after separation they still work together, with the infill ensuring lateral resistance and stiffness, and the flexibility of the timber frame confining the masonry. Therefore, only at large deformation do damages appear. Considering the ratio between the ultimate displacement and the yielding displacement, a ductility factor of 2.03 was determined. Since in S1 no significant bending of the timber elements was observed, for S2 the bending was very clear for all of the timber elements except the bottom beam (Fig. 6.20).

Figure 6.22 shows that S1 has negligible stiffness compared to S2, this means the presence of masonry panels is crucial to the system, acting as reinforcement for the timber frame and being able to carry the lateral loads.

In terms of energy dissipation S2 showed a significantly higher total value (37.69 kNm) than in the case of S1 (2.94 kNm). The energy dissipation, ΔW, was calculated as the area of the hysteretic curve for each specimen. Figure 6.23 shows the energy dissipation of S1 and S2 in each cycle.

Equivalent damping ratio h_e of the system was also calculated using the formula:

$$h_e = \frac{1}{4\pi} \cdot \frac{\Delta W}{W} \tag{6.4}$$

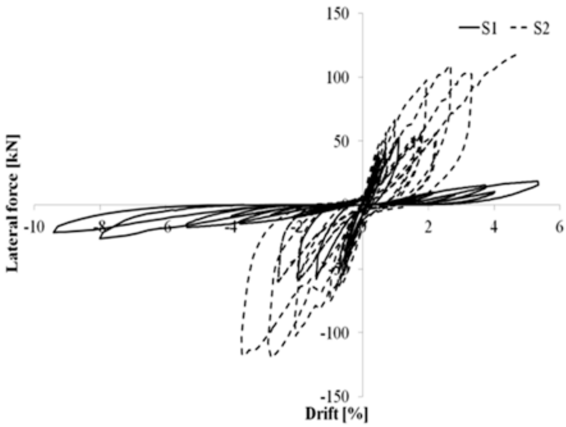

Fig. 6.21 Comparison of hysteretic diagrams between S1 and S2

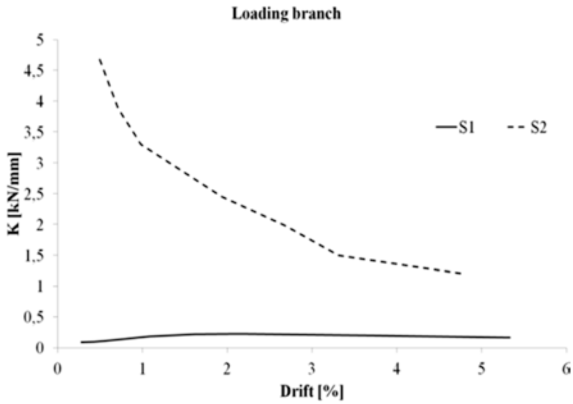

Fig. 6.22 Stiffness degradation comparison between S1 and S2

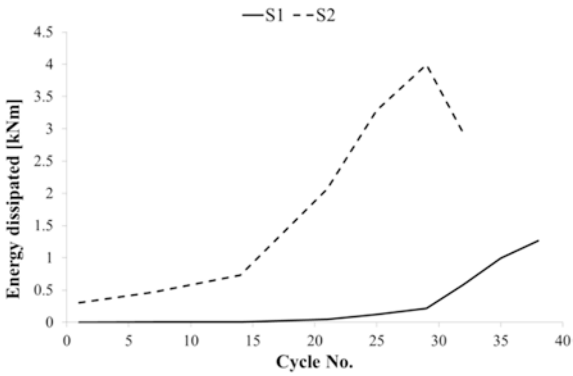

Fig. 6.23 Energy dissipation comparison between S1 and S2

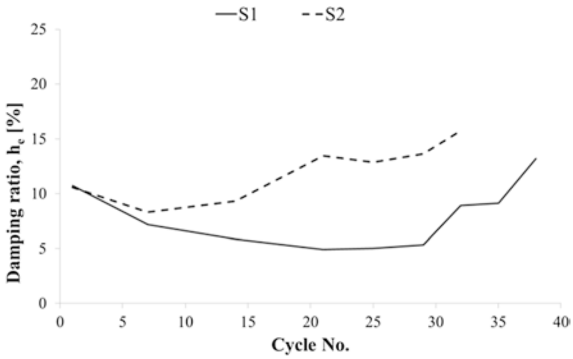

Fig. 6.24 Damping ratio comparison

where ΔW is the total dissipated energy of the specimen under cyclic loading and W is the potential energy in a cycle, calculated as the average of the negative and positive peaks. Figure 6.24 presents the values. Figure 6.24 shows that the masonry infill produces a significant increase in damping when it starts to crack after the 21st cycle.

Conclusions

The present experimental study confirms the good in plane behaviour of the timber-framed masonry system. The masonry infill determines the strength and stiffness of the system, while the timber frame is responsible for the high ductility.

The principle of timber-framed masonry structures is that the failure of the masonry infill occurs due to mortar cracking in the joints because the mortar is the weakest element in the system, which then causes bricks to slide. The results of both diagonal compression and static cyclic tests confirmed that the failure mode of the masonry was a shear sliding mechanism.

The failure mechanism of the timber framed masonry system can be described by three phases:

- the masonry works in an uncracked state and separates from the timber frame;
- after cracking the masonry starts to slide, dissipating energy, while the timber frame deforms with no damages due to the greater flexibility of timber;
- the timber frame starts to show damages and its elements are subjected to shear forces caused by compression of the masonry perpendicular to the grain of the timber.

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Chapter 7
Shaking Table Test of Full Scale Model of Timber Framed Brick Masonry Walls for Structural Restoration of Tomioka Silk Mill, Registered as a Tentative World Cultural Heritage in Japan

Toshikazu Hanazato, Yoshiaki Tominaga, Tadashi Mikoshiba and Yasushi Niitsu

Abstract Tomioka Silk Mil is the first model silk-reeling factory that the Japanese Government established in 1872. The main building is structurally characterized by timber framed brick masonry walls. The buildings were constructed using European and Japanese traditions. Walls were built of locally produced bricks, manufactured using a technique introduced from Europe, and the buildings were roofed with traditional Kawara tiles. The scope of the paper is to present the outcomes of the shaking table tests using full-scale model of the timber framed brick masonry walls. Those tests were successfully conducted to study the seismic behaviors of such composite structures under extremely strong motions. As well as, they were performed to examine the effectiveness of the proposed strengthening technique using aramid fiber wires. In the present study,

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3-dimensional dynamic displacement was directly measured by the image processing technique utilizing high-speed optical cameras. This new technology revealed both the dynamic deformation of the walls and the safety limit of the displacement in dynamic phase.

Keywords Timber frame brick masonry · Earthquake · Retrofitting · Shaking table test · Masonry wall · Out-of-plane · Heritage structure

Introduction

Tomioka Silk Mil [1] in Gunma Prefecture, located 150 km to the north of Tokyo, is the first model silk-reeling factory that the Japanese Government established in 1872. The Japanese Government introduced the modern machine silk reeling from France to spread the technology in Japan. Since the site and the buildings of the factory were designated as the national historic site and the national important cultural properties, respectively, the main buildings of the mill complex such as the east and west cocoon warehouses have been preserved in good condition (See Figs. 7.1 and 7.2). The buildings were constructed in combination of European and Japanese traditional manner. In particular, most of those buildings were structurally characterized by timber-framed brick construction. However, there were only a few studies of the seismic performance of such combined construction in Japan. Therefore, it should be needed to ensure the seismic safety of such timber-framed brick masonry structure in order to open them to public. In the present study, the shaking table test was conducted to know actual performance of the timber-framed brick walls during earthquakes, as well as, to verify the strengthening method of the walls against earthquakes. In general, seismic safety problem of such construction walls must exist in out-of-plane behaviors. By taking into account this condition, the present study was focused on seismic performance of out-of-plane behaviors. In particular, deformability of out-of plane behaviors of

Fig. 7.1 View of the West Cocoon

Fig. 7.2 Inside of the West Cocoon

walls in dynamic phase was discussed, as there found no past studies on the out-of plane deformability (limit state) of those brick masonry walls. The strengthening technique introduced in the present study was proposed by Tominaga [2, 3], one of the authors. The method was to install aramid fibers into joints. By applying this newly developed technique, the exterior of brick face wall can be conserved as cultural heritages. Furthermore, the full-scale model was examined in the shaking table tests in order to know the actual dynamic behaviors subjected to earthquake motions. Simulation law affects such shaking model tests for failure, therefore, full-scale models should be utilized.

Model Structure for Test

The model structure was designed to examine seismic safety in out-of-plane direction. Illustrated in Figs. 7.3 and 7.4, the test model structure fabricated in the present study was 1-span full-scale models taken from the west cocoon building shown in Fig. 7.1. The model structure composed of two brick walls and timber frames composed one model structure with height and width of 4,800 and 3,600 mm, respectively. The brick walls were fabricated by French bond with thickness of 230 mm. The timber frames were made of glued laminated timber with section of 303 mm^2. Not only dimensions of the structures but also their mechanical material properties were reproduced to fabricate the model structures. In particular, as the joint mortar of the original structure was

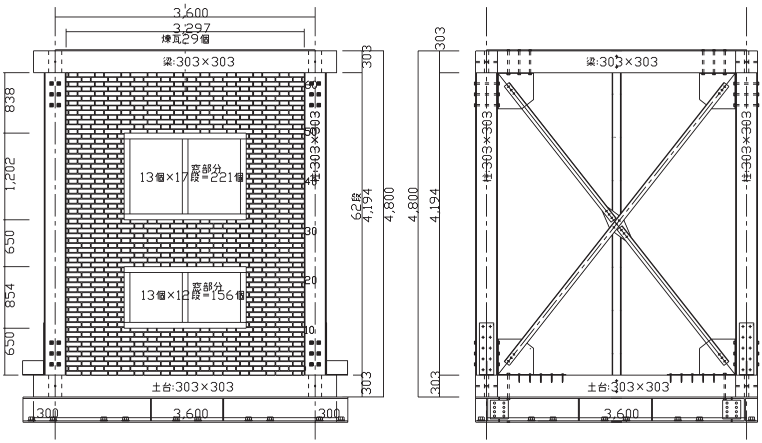

Fig. 7.3 Elevation of model structure

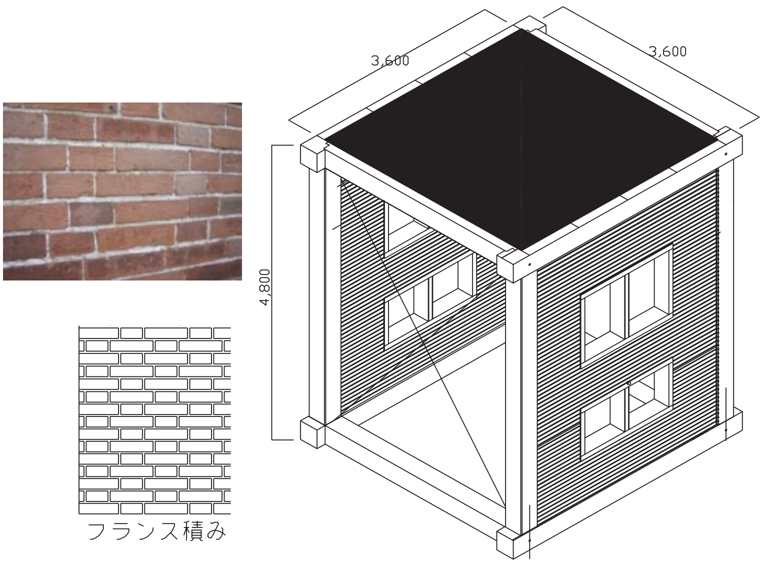

Fig. 7.4 View of model structure

lime one, lime mortar was also used for the construction. Both the bricks and the joint mortar were made so that the material strength should be as weak as the original structure constructed in 1870’. Table 7.1 shows the mechanical material properties of the model structure. Two model structures were shaken at the same time on the shaking table, shown in Fig. 7.3. Each model structure had two brick walls, meaning that a total of four timber-framed brick walls were tested

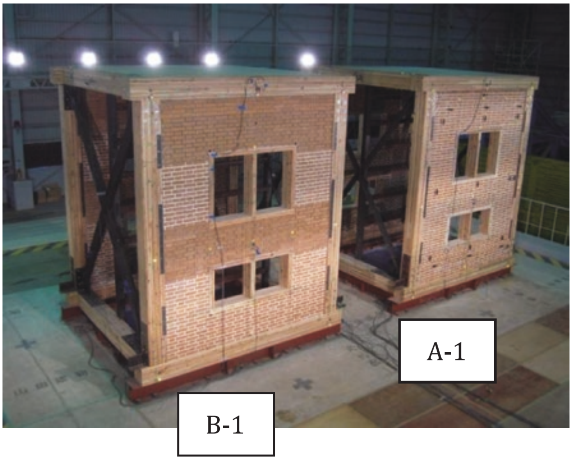

Fig. 7.5 Model structure on shaking table

Table 7.1 Mechanical material properties

Material	Properties	Strength (N/mm^2)	Stiffness E_{50} (N/mm^2)	Initial strength E_0 (N/mm^2)
Brick	Compressive	10.2	1800	3470
Prism		5.7	278	192
	Tensile	0.0026	–	–

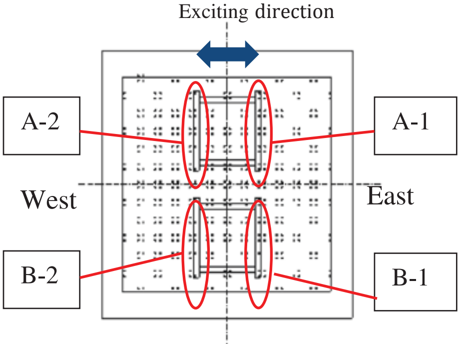

Fig. 7.6 Arrangement of model structure on shaking table

at the same time. Figure 7.6 shows arrangement of the model structures on the shaking table test. Listed in Table 7.2, two of them were the wall model structures strengthened by the joint replacement method utilizing aramid-rods with diameter of 3 mm. This strengthening technique developed for structural restoration of historical brick masonry buildings was proposed by Tominaga [2, 3]. The procedure was summarized as (1) Joint mortar was removed to depth of 40 mm,

Table 7.2 Wall models for tests

	Model name	Existence of gap[a]	Reinforcement by joint replacement method
Unreinforced model	A-1	Existence	No
	A-2	No	No
Reinforced model	B-1	No	One-side
	B-2	No	Both-sides

[a] Gap between timber frame and brick wall, produced by shrinkage of brick wall

(2) Aramid-rods were installed in both horizontal and vertical joints. (3) Joints were filled with no-shrinkage mortar. As listed in Table 7.2, two brick wall model structures were strengthened by the above-mentioned technique; while one (Wall model B-1) was strengthened in front or back face, another (Wall model B-2) was strengthened in both front and back faces. On the other hand, the gap between the bricks and the timber frame at the top of the brick wall was examined for the unreinforced model structures, because there found such gaps in the original structure, caused by shrinkage of joint mortar during the long period. Wall model A-1 with gap and Wall model A-2 with no-gap were examined at the same time in the present shaking table test (Fig. 7.5).

Shaking Table Excitation

The shaking table tests were performed at the large shaking table in NEID, Tsukuba, Japan. Excitation cases for the tests were as the follows; (1) Step excitation of amplitude 0.5 mm to investigate natural frequency and damping of the wall models, (2) Ground motions simulated by fault model as extremely strong motions at the site (D1F), (3) Floor response motions at the second floor when the simulated ground motions were input to the structure(D2F), (4) Earthquake ground motions recorded at JMA Kobe during Kobe Earthquake of 1995. The excitation direction was perpendicular to the wall's plane, i.e. the out-of-plane response of the timber-framed brick walls was examined.

Damage to Structure

Table 7.3 summarizes the damage to the structure, in which the excitation number and input motion level are described to show the development process of the damage up to failure. The minor displacement was first found in Model A-1 at Excitation No. 4 (0.10G). Next, cracks between the timber beam and the brick wall appeared in Model A-2 at Excitation No. 8 (0.44G). When half level of JMA Kobe was input at Excitation No. 9 (0.41G), only one brick was fallen

Table 7.3 Excitation and damage description

Excitation no.	Input wave form	Input motion level		Damage description
		Acceleration (G)	Amplitude (mm)	
1	DF2 (0.8 %)	0.0025	1.3	No damage
2	DF2 (7.7 %)	0.025	10.2	No damage
3	DF2 (11.7 %)	0.051	20.4	No damage
4	DF2 (23.4 %)	0.103	39.0	A-1: slight brick displacement
5	DF2 (35.1 %)	0.150	58.9	
6	DF2 (46.8 %)	0.206	76.6	
7	DF2 (100 %)	0.326	122.6	
8	D1F (50 %)	0.440	31.3	A-2: crack at joint between frame and brick wall
9	JMA Kobe (50 %)	0.409	87.5	A-1: one brick falling
10	D1F (100 %)	0.879	76.9	A-1: some brick layer falling
11	JMA Kobe (100 %)	0.818	184.0	A-1: collapse A-2: brick wall displacement
12	STEP		±15	
13	JMA Kobe (110 %)	0.900	212.2	A-2: partial wall collapse B-1: no significant damage B-2: no significant damage
14	JMA Kobe (110 %)	0.900	212.2	A-2: collapse B-1: partial collapse B-2: no significant damage
15	JMA Kobe (110 %)	0.957	225.6	B-1: partial collapse B-2: crack development

Max. response ratio

from the wall of Model A-1. At the following excitation No. 10 (0.88G), some brick layers were fallen from the wall of Model A-1. This vulnerable Model A-1 collapsed when JMA Kobe at the full level (0.88G) was input to the structure at Excitation No. 11 However, the other models survived even when the devastating strong motion record of JMA Kobe was input, although it caused the horizontal residual displacement of Model A-2. This experimental result demonstrated that, even for such non-reinforced brick wall, had inherent potentialities against large earthquake ground motions, when there was no gap between the frame and the brick wall, i.e. connection between the wall and the timber frame was perfect. Here, it should be emphasized that those test models had no structural defect such as cracks or deformation at the initial condition as the model structures were fabricated by the professional Japanese carpenters who worked well at the construction site of the traditional buildings. In the present study, in order to cause cracks to the model structures, step excitation with amplitude of 15 mm was input at Excitation No. 12. After this step excitation, when the model structures were excited by JMA Kobe of 110 % amplitude (0.90G, Excitation No. 13), some of bricks were fallen from the wall of Model A-2, while there found no significant damage to the reinforced models of Model B-1and B-2. This indicated that the strengthening technique applied in the present study was greatly effective in improvement of the brick walls. At the next excitation of No. 13 of the same JMA Kobe of 110 % amplitude, Model A-2 that was severely damaged at the previous excitation finally collapsed. However, the reinforced walls survived even against such extremely strong motions. At the final excitation No. 15, both reinforced model structures survived although a part of the brick wall was fallen for Model B-1, while the cracks were developed in Model B-2.

Figure 7.7 shows the maximum acceleration amplitude ratio of the wall of Model A-2 to the shaking table in out-of-plane response, where variation of the ratio through the excitation series can be noticed. Model A-2 was the unreinforced model of which the gap between the timber frame and the brick wall was filled with mortar. At Excitation No. 8 (0.44G), horizontal cracks were caused along the joint mortar layer between the timber beam frame and the brick wall. After this excitation in which cracks were caused in Model A-2, the maximum acceleration amplitude ratio became larger, shown in Fig. 7.7. In this figure, the measuring point at No. 13 was on the timber frame, while the points at No. 8 and 14 were on the brick wall. This figure suggests a possibility for non-destructive test to detect existence of gaps or cracks between timber frames and brick walls by application of such measurement of vibration.

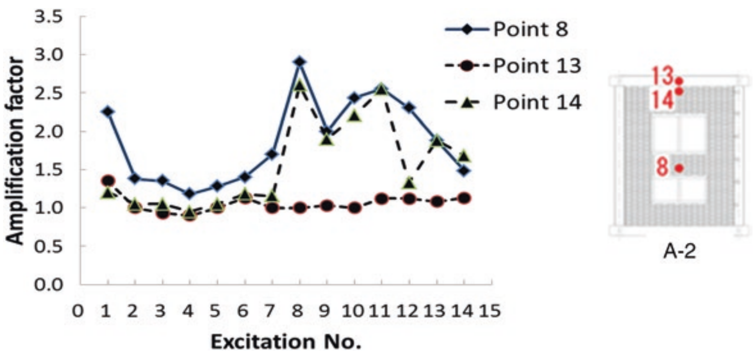

Fig. 7.7 Maximum acceleration amplification in out-of-plane direction

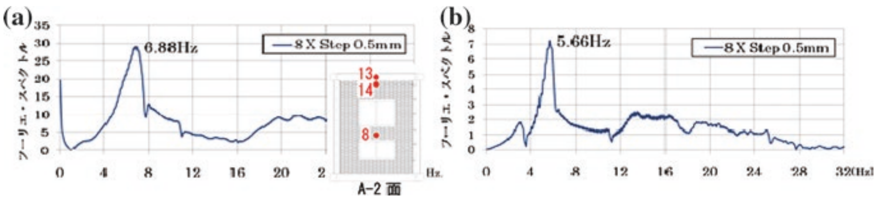

Fig. 7.8 Fourier spectra of response at point 8 of Model A-2. **a** Initial condition (after Excitation No. 1); **b** Damaged condition (after Excitation No. 12)

Variation of Natural Frequency in Out-of-Plane Behavior

It was found in the present tests that the natural frequency varied in accordance with development of cracks. Figure 7.8 compares Fourier spectra of the acceleration recorded in Model A-2 at the step excitation with amplitude of 0.5 mm, which was recorded at the center measuring point. Figure 7.8a, b describe the initial behavior and the one after severely damaged at Excitation No. 12. It should be noticed that the natural frequency varied from 6.9 Hz at the initial condition with no damage to 5.7 Hz at the condition with severe damage. Such variation indicated that the rigidity of the brick wall was reduced by 30 %.

Dynamic Response Displacement

For assessment of seismic safety, it is essential to evaluated deformability of timber-framed brick masonry wall. In particular, deformability of out-of-plane behavior was of concern in the present study. Therefore, dynamic 3-dimensional displacement was measured by the imaging processing technique using high-speed cameras and LED markers. Figure 7.9a describes the peak displacement recorded in the unreinforced walls, Model A-1 and Model A-2 at Excitation No. 10 when both model structures survived. It can be noticed that the peak displacement reached 83 mm at the upper boundary of the brick wall of Model A-1. On the other hand, it was 27 mm at the center measuring point of the wall for Model A-2. These figures indicate that filling gap between the timber frame and the brick wall was so effective in reduction of the response displacement. Figure 7.9b also shows the peak displacement of the reinforced brick masonry walls utilizing joint replacement method at Excitation No. 13. It should be noticed that, the peak displacement was 57 mm in Model B-1 (one-side reinforcement), while that was 19 mm in Model B-2 (both-sides reinforcement). The implementation of reinforcement at both sides reduced the response displacement of the wall to 1/3 of that of one-side reinforcement, indicating the effectiveness of the both-sides reinforcement in improvement of the out-of-plane seismic performance of the walls.

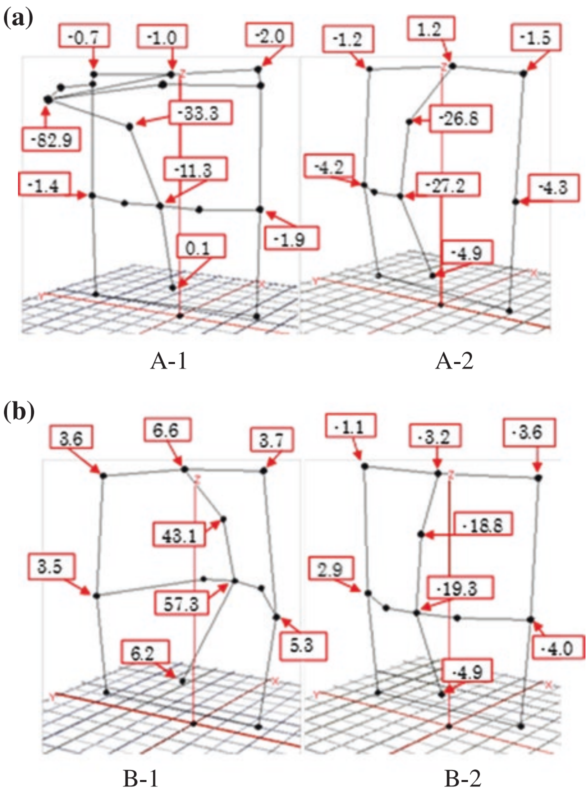

Fig. 7.9 Deformation of wall at maximum response: **a** Excitation No. 10 (D1F 100 %); **b** Excitation No. 13 (JMA Kobe 100 %)

Dynamic Peak and Residual Displacement

Table 7.4 summarizes the peak and the residual displacements measured in out-of-plane direction, indicating that the test structure dynamically survived such deformation in the out-of-plane response. Deformability in the out-of-plane behaviors in dynamic phase can be recognized in this table. For the unreinforced model structures of A-1 and A-2, they showed large deformability that reached 80–110 mm in dynamic phase. It should be noticed that the residual displacement of those models,

Table 7.4 Max displacement in dynamic phase and residual displacement

Model	Measuring point	Ex. no.	Max. disp (mm)	Residual disp (mm)
A-1	Top of wall	10	82.9	–
	Center	10	11.3	0.5
A-2	Center	13	113.4	0.4
B-1	Center	15	111.9	–
B-2	Center	15	36.3	0.6

however, was less than 1 mm. On the other hand, the reinforced model structures of Model B-1 and B-2, the peak displacements were 36 mm and 110 m, respectively. Such large deformability observed in the shaking table tests was corresponding to the deformation angle as large as 1/16. Hence, the structural issue was raison from this test results, i.e., why such large deformability of the brick walls was observed in out-of-plane during the shaking table tests should be discussed from a mechanical point of view. In particular, the minimum intervention should be applied to buildings with cultural and historical values when they structurally restored. The criteria of deformation in out-of-plane in dynamic phase, i.e. allowable displacement, should be evaluated to respect such large deformability in the future.

Concluding Remarks

The shaking table tests of the full-scale model structures in out-of-plane excitation demonstrated the deformability of the brick walls in dynamic phase, as well as, the effectiveness of the strengthening technique proposed by Tominaga [2, 3]. In the present tests, the peak deformation in out-of-plane direction reached 110 mm corresponding to the angle of 1/16, for both unreinforced and reinforced brick walls with a span of 3,600 mm. Furthermore, the tests showed that filling the gap between the upper timber frame and the brick wall improved the seismic safety of the structure. The strengthening method that is called "the joint replacement method" using aramid-rods has significant effect on improvement of the seismic performance.

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Chapter 8
Seismic Performance Evaluation of Timber—Framed Masonry Walls Experimental Tests and Numerical Modelling

Stefano Galassi, Nicola Ruggieri and Giacomo Tempesta

Abstract The Borbone constructive system used in Calabria at the end of the 1700s consisted of a particular composite structure realized by means of a timber frame suitably embedded inside masonry walls. This system used with similar purposes, although in different ways, in other places in the world (especially in seismic regions), can represent, with good reason, the synthesis of scientific knowledge in eighteenth century seismic engineering. The aim of the paper is to investigate and evaluate the seismic performance of the structure described above through a comparison between experimental tests, carried out by means of cyclic tests on 1:1 scale models, and the results obtained by the numerical modeling of the mechanical system that is capable of interpreting the actual contribution of the wooden structure, as well as that of the masonry, to the overall stiffness of the wall. In the numerical procedure, the masonry infill is modeled by rigid blocks connected by unilateral elastic contact constraints. A convenient way to define the contact device which links the blocks, through which a mortar joint or dry joint could be simulated, is to consider a set of elastic links, orthogonal to the contact surface between two adjacent blocks, and an additional link, parallel to the interface through which the shear forces can be transmitted. Reasonable hypotheses can be assumed for the link parallel to the contact surface in order to calibrate both the shear behaviour and the influence of the friction between the blocks. Furthermore the timber frame is modeled by using finite elements with elastic and bilateral

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behaviour. Unilateral contact constraints are again used in the contact interfaces between elements in wood and masonry blocks which take into account the actual contribution of friction. The mechanical parameters used in the numerical model were deduced from the experimental laboratory tests.

Keywords Seismic behaviour · Masonry reinforced · Timber frames · No tension behaviour

Introduction

The Borbone constructive system, constituted by masonry reinforced with timber frames, represents the application of the most ancient of European anti-seismic codes.

The Mileto Bishop's building in Calabria, constructed immediately after the catastrophic earthquake of 1783, is characterized by a load bearing system executed exactly according to the Borbone rules. Therefore, after a detailed structural and geometric survey, including the material features, on the Mileto construction, the latter, in particular a wall modulus, had been reproduced in full scale and subjected to a cycling test in the CNR Ivalsa laboratory in Trento. The tested specimen was constituted by timber framing devoid of Saint Andrew crosses and stiffened, to the in plane seismic action, by means of the masonry infill. The wooden skeleton was characterized by half lap joints in which the stiffness was improved by the presence of pyramidal nails.

Data to be used in the numerical model proposed were obtained by comparing the experimental campaign results to the seismic behaviour deduced from historic photos and documents that depict seismic failures after the 1905 and 1908 telluric events [1]. In fact, the aim of this theoretical investigation is to provide researchers with data, obtained on the basis of these experimental results, to be used to propose new methods for assessing the seismic behaviour and the vulnerability level of this constructive system.

Several authors have investigated timber framing with different arrangements of wooden elements and stiffness devices by computing non-linear analysis carried out through various F.E. software. Kouris and Kappos [2] applied a numerical analysis in ANSYS on masonry walls reinforced with timber elements found in Greek traditional edifices. This numerical approach provided the modelling of horizontal and vertical elements through a linear-elastic beam while the diagonals of the timber frame were modelled with a link pinned at its ends and characterized by the presence of a plastic axial spring.

The use of the DIANA F.E. software distinguished the work of Ramos and Lourenço [3]. These Portuguese researchers applied a numerical modelling on traditional buildings, with and without the interior "frontals" walls, to assess the internal panels contribution to the overall building under seismic actions.

Fig. 8.1 The specimen under cyclic loading in the CNR-Ivalsa laboratory

The analyses were validated by means of three specimens removed from existing Pombaline edifices and tested under cycling horizontal loading.

The DRAIN2DX software, developed by the University of California in Berkeley, was implemented with the *Florence Pinching* (Ceccotti, Lauriola, Follesa) to analyze, in a simplified way, structures characterized by timber frames. The researchers of the University of Florence introduced rotational semi-rigid elements to simulate pinching hysteretic behaviour of the joints based on cyclic tests results [4].

A similar quantitative investigative approach was carried out at the *Earthquake Engineering Center* in Peshawar, Pakistan. In fact, an equivalent model with elastic beam-column element, with assigned moment-rotation plastic hinges derived from an experimental campaign, was employed to obtain a non-linear static pushover tool by means of SAP2000 software and relative to Dhajji-Dewari structure, a timber braced frame masonry wall [5] (Fig. 8.1).

The Cycling Test Results

Two specimens, timber framing with infill masonry frame and empty timber framing, which reproduce the Mileto panel in real scale, were tested at the CNR Ivalsa in Trento according to the UNI EN 12512:2003 "*Timber structures—Test methods—Cycling testing of joints made with mechanical fasteners*" protocol.

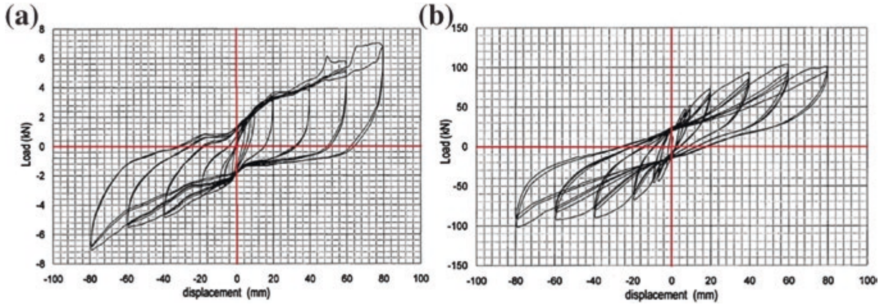

Fig. 8.2 **a** Hysteresis loops from the experimental survey; **b** empty timber frame and masonry wall reinforced with timber frame

The samples were tested with positive and negative horizontal displacements, applied at the top of the wooden framing, using an hydraulic actuator with a 500 kN capacity.

A uniformly distributed load (18.7 kN/m) was applied to the models with the aim of replacing the self weight of the timber post king truss bearing on the wall of the Bishop's building.

The tests were interrupted at a maximum displacement of approximately 80 mm, as a consequence of excessive deformation.

The specimen characterized by timber framing with masonry infill showed a low rocking mechanism with a maximum value of uplift displacement of 30 mm at peak load. The lateral resistance, relative to the first cycle, reached 103.64 kN in positive direction corresponding to a displacement of 59.18 mm (2.0 % drift) and −101.62 kN ultimate load in negative direction which is related to a displacement of −79.02 mm (2.6 % drift). Hence, the model showed an impairment of the strength, calculated between the first and the third cycle for each ductility level, variable between a peak of approximately 13 % in "compression" charge to a maximum value of approximately 15 % relative to a displacement of −40 mm.

The energy dissipation value was approximately 1,500 kN mm in correspondence to the 1st half cycle with maximum displacement and approximately 300 kN mm for the half cycle concerning a displacement of 20 mm. The hysteresis equivalent damping ratio (Veq) presented constant values between 6 and 7 % for each displacement analyzed; even if a peak of 8.9 % was recorded relative to an "in-tension" displacement of 20 mm.

The maximum ductility value ($\mu = V_u/V_y$) reached by the tested model was 7.6. Namely the specimen has emphasized a ductility response (Fig. 8.2).

The experimental survey pointed out a correct response of the Borbone constructive system under horizontal force. This kind of structure dissipated energy by means of interface frictions generated by the slips of the stones both between the infill masonry and the frame and also thanks to some fissures generated in the mortar, as well as the expulsion of a few stones. The overall timber skeleton, both elements and joints, acted, during the cycles, in elastic field (with the exclusion of the beam at the frame bottom that presented some shear cracks, however without losing the structural integrity). The timber reinforcement provides the masonry with a major deformability and simultaneously the infill frame provides a confinement for the wooden structures.

The model devoid of the infill masonry frame emphasized a weak behaviour characterized by a high deformability under cyclic actions.

The main purpose of the experimental program described above is to provide data to assess seismic capacity of the Borbone system by means of multi-scale numeric modelling.

Preliminary Numerical Modelling

The original software *BrickWORK* [6], specifically developed by some of the authors for the analysis of general masonry structures, is used in the herein numerical calculation to simulate the behaviour of the *Baraccato* constructive system, masonry wall reinforced with timber framing, under earthquake action.

The numerical model is characterized by the masonry modelled by a collection of rigid blocks (bricks or stones) connected by mortar joints, where the elastic-brittle behaviour of the material is concentrated. Consequently, relying on these mechanical features, the main type of damage mechanism considered in the mortar joints is a tensile failure and until such a failure occurs, the joints are supposed to retain an elastic behaviour.

Therefore, such an approach involves that the masonry, as a whole, has a good capability to carry compression loads and, taking into account that the masonry to which we want to refer is that of historical architecture heritage, the tensile strength of the material is limited to the poor cohesion between mortar and bricks.

Based on the above assumptions, the mechanical characterization of masonry refers to a system of rigid blocks connected by unilateral contact and frictional links.

In the numerical model the contact devices located in the joints are described by a set of fictitious links, arranged orthogonal to the interface surfaces, capable of transmitting only compressive forces or, at most, weak tensile forces which do not exceed the assigned limit values, and, by an additional link, tangent to the interface surface, to transmit the shear force.

In the case of brittle-rigid joint only two normal links are strictly necessary. Instead, in the case of elastic-cracking joint it is better to consider at least four normal links in order to highlight the actual cracking pattern with the possibility of measuring the width and depth of the cracks inside the mortar joints.

An example of this numerical model to a real case can be found in [7, 8] (Fig. 8.3).

Moreover, it is reasonable to think that a model which considers the rigid blocks linked together by means of deformable surfaces, with no tension behaviour, is the most correct model to interpret the influences which the dimensions of the blocks and the orientation of the joints have on the behaviour

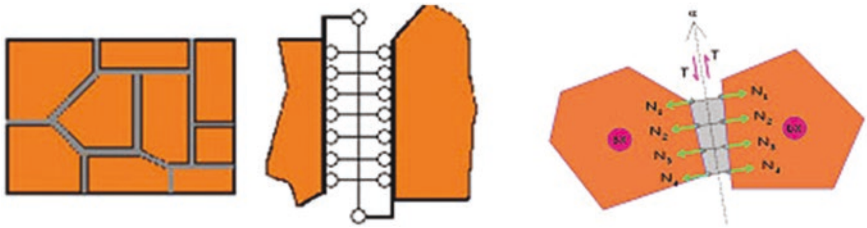

Fig. 8.3 Discrete model of the joint device

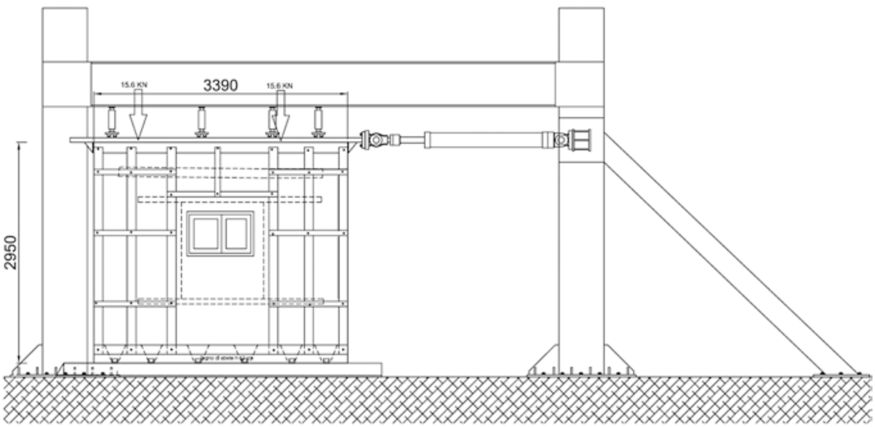

Fig. 8.4 Full-scale specimen of a masonry wall reinforced with a timber frame built according to the Borbone constructive system. Laboratories Ivalsa—CNR

of historical masonry buildings. In this way the model is capable of very clearly describing the progression of the damage to masonry under load conditions.

The original numerical model, developed for the analysis of a structure consisting of only masonry blocks [9], has been modified to consider the peculiar mechanical characteristics of the *Baraccato* system, a masonry wall reinforced with a timber frames. Specifically, it was necessary to properly define the contact joint between wood and stone, which was assumed to have a no-tension, and the joint between wood and wood, which was considered to be perfectly elastic (Figs. 8.4 and 8.5).

The results obtained from the experimental tests performed at the CNR–Ivalsa laboratory, on a full-scale specimen of a masonry wall made on the basis of the Borbone constructive system (summarized in Table 8.1) were used for calibrating the mechanical parameters to be assigned to the contact joints between the finite elements constituting the general mesh of the model.

The first step was to define the mechanical and geometrical characteristics to be assigned to the discrete model with concentrated elasticity in correspondence to the joints so as to reproduce the same field of deformation and displacements obtained by the experimental tests carried out on the structure consisting of only a timber frame.

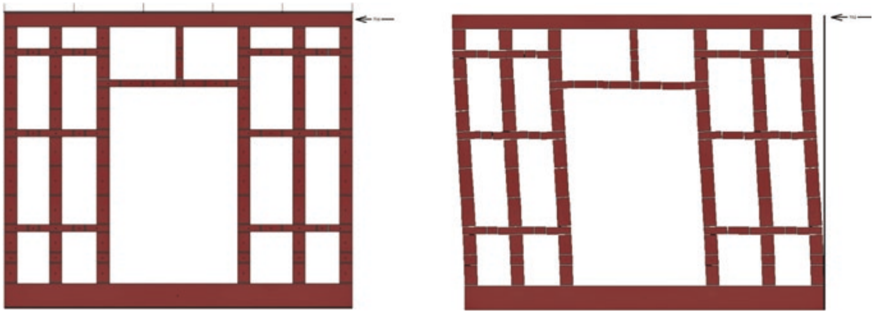

Fig. 8.5 Discrete model of the timber frame specimen

Table 8.1 Main results from the experimental survey relative to the compound specimen masonry with timber frame

Direction	F_{max} (kN)	V_{max} (mm)	F_u (kN)	V_u (mm)	Envelope curve 1st cycle
P	103.6	59.2	100.6	79.12	
N	−101.6	−79.02	−101.6	−79.02	

The experimental survey showed that the timber frame, even at the maximum value of the applied load, never cracked in any section. For this reason no ultimate tensile and compressive strengths in correspondence to the wood joints have been defined because they can be conventionally assumed to be infinite.

The next step was to define the discrete model of the masonry infill, taking into account the shape and the arrangement of the stone elements as well as the thickness of the joints so as to reproduce, as closely as possible, the actual experimental model.

In order to define the mechanical characteristics of the contact joints between stone and wood, a zero tensile strength limit was assumed, while for the contact joints between the stones, an ultimate tensile strength equal to 0.5 MPa was considered.

Relative to the boundary conditions of the mechanical model subjected to the numerical analysis, fixed supports were assumed at the base and a slider-type connection at the top, with the aim to reproduce the choices made for the experimental tests (Fig. 8.6).

The results obtained with the numerical modelling have provided an interpretation of the behaviour of the wall very close, both quantitatively and qualitatively, to that of the specimen subjected to the cyclic tests in the laboratory (Fig. 8.7).

The final results for a load, applied at the top, equal to 103.64 kN was achieved after 311 iterative steps of the calculation algorithm with a final horizontal displacement, measured at the top of the specimen, equal to 59.90 mm. Such a displacement is very close to the actual one.

It is interesting to notice how, in terms of fracture and detachment, the crack pattern obtained by the numerical analysis has shown a significant similarity to the real one.

Fig. 8.6 Mechanical modelling of the masonry wall built according to the Borbone constructive system

Fig. 8.7 Ultimate deformed shape and cracking pattern. Comparison between experimental test and numerical model

Conclusion

This paper provides a preliminary report on the experimental survey of the *Baraccato* system, a masonry wall reinforced with timber frames, as well as a preliminary numerical approach to analyzing this system based on a mechanical model composed of rigid blocks and elastic joints.

The results of the analysis conducted by means of the original software *BrickWORK*, suitably modified to consider the presence of wooden elements, are perfectly coherent with the ones obtained by the cyclic tests of the Borbone system performed in the CNR-ivalsa laboratory.

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Chapter 9
A Proposal for a Procedure to Evaluate the Seismic Vulnerability of Historic Timber Frame Buildings

Ario Ceccotti and Carmen Sandhaas

Abstract The seismic vulnerability of historic timber frame building heritage is difficult to quantify in retrospect. However, in order to maintain, restore or even retrofit these buildings, it is indispensable not only to understand their performance under an earthquake loading, but also to gain quantitative information on stiffness, load bearing capacities and failure modes of the used timber frame technique. A possibility to assess earthquake behaviour is full-scale testing, but this is very expensive and it is nearly impossible to cover all the different timber frame systems, the variations between the different systems and different earthquake loadings. Therefore, small-scale tests on components or substructures are more suitable whose outcomes can be used for nonlinear dynamic modelling of buildings. This contribution proposes a simple testing-modelling approach to quantify the seismic vulnerability of timber frame buildings. The testing includes monotonic and quasi-static reversed cyclic tests on shear walls which are re-built specimens mirroring as exactly as possible the historic archetypes for both wall dimensions and used materials. The test results are then used to develop nonlinear dynamic lumped mass models which are subjected to various earthquake accelerograms. By increasing the single earthquake's peak ground acceleration (PGA) values, the seismic performance of the investigated historic structure can be evaluated and suggestions can be given with respect to restoring or retrofitting measures. On the basis of a valid and reliable mathematical model, also parameter studies varying, for instance, the number of fasteners in joints are more readily carried out than with experimental methods.

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Keywords Seismic behaviour · Historic timber frame buildings · Cyclic tests · Shear walls · Non-linear dynamic model · Time domain

Introduction

The building asset in many earthquake-prone areas comprises timber frame buildings with an infill of, among others, bricks, stones or wattle and daub (e.g. [1]). The seismic vulnerability of this historic building heritage is difficult to quantify in retrospect. The complex interaction between timber frame and infill together with unknown joint properties can hardly be determined and is influenced for instance also by manufacturing skills of the builders.

However, in order to maintain, restore or even retrofit these buildings, it is indispensable not only to understand their performance under an earthquake loading, but also to gain quantitative information on stiffness, load bearing capacities and failure modes of the used timber frame technique. Deterioration and damage that is commonly assessed by qualitative visual inspection must be quantified as well in order to develop bespoke restoring interventions.

A possibility to assess earthquake behaviour is full-scale testing, but this is very expensive and it is nearly impossible to cover all the different timber frame systems, the variations between the different systems and different earthquake loadings. Therefore, small-scale tests on components or substructures are more suitable whose outcomes can be used for non-linear dynamic modelling of buildings.

This contribution proposes a simple testing-modelling approach to quantify the seismic vulnerability of timber frame buildings. The testing includes monotonic and quasi-static reversed cyclic tests on shear walls which are re-built specimens mirroring as exactly as possible the historic archetypes for both wall dimensions and used materials. The test results are then used to develop non-linear dynamic lumped mass models which are subjected to various earthquake accelerograms. By increasing the single earthquake's peak ground acceleration (PGA) values, the seismic performance of the investigated historic structure can be evaluated and suggestions can be given with respect to restoring or retrofitting measures. On the basis of a valid and reliable mathematical model, also parameter studies varying for instance the number of fasteners in joints are more readily carried out than with experimental methods.

In this contribution, the proposed methodology is explained and illustrated by means of an example taken from Ceccotti et al. [2]. The proposed assessment method is similar to the already proposed approach by Ceccotti and Sandhaas [3] to establish the seismic behaviour factor according to Eurocode 8 [4].

Assessment Methods

The most common assessment methods to evaluate the seismic behaviour of historic buildings are visual inspections after major earthquakes as for example carried out by Langenbach [5]. Langenbach [5] stated that damage patterns observed in traditional Turkish houses in the Marmara region helped to understand their seismic behaviour which the author attributes to the low stiffness of the lateral load bearing elements, the so-called shear walls. Although the lateral load bearing stiffness and strength is low, the energy dissipating capacity of the shear walls is high which helps to avoid catastrophic collapse whilst allowing for plastic deformation and damage [5]. Another, even more fundamental conclusion is the identification of the leading role of shear walls for seismic resistance.

Visual inspection methods are very powerful for understanding the general seismic behaviour of structures and for identifying critical structural components. However, they cannot provide quantitative information in terms of strength, stiffness and amount of dissipated energy. These values can be determined with experimental assessment methods such as cyclic tests as for instance applied in Vieux-Champagne et al. [6] where traditional Haitian timber frame walls with stone and mortar infill were tested. When using experimental methods, the first decision is on the testing scale. Cyclic tests can be undertaken on joint level, component or substructure level such as wall or floor elements or (theoretically) even on full-scale level. In Vieux-Champagne et al. [6] for instance, timber frame walls were tested on joint, sub-wall and shear wall level and the authors concluded that due to their high ductility and energy dissipation capacity, these historic structures perform well under seismic actions. The chosen testing level depends not only on the feasibility of the experiments and the gain in information provided by the tests, but also on the scope of testing in terms of subsequent modelling purposes.

Cyclic tests however cannot provide information on the sustainable level of seismic action. After cyclic tests, it is not known up to which PGA the tested historic structure could resist. Furthermore, the behaviour of a shear wall under a cyclic loading may be different from the behaviour of a real three-dimensional building under a seismic loading. As full-scale shaking table tests are expensive and therefore difficult to realise, other means must be found to investigate the seismic behaviour of buildings. Therefore, instead of undertaking full-scale testing, the buildings are modelled. The modelling can take place at many different modelling levels; each level requiring test results as input parameter show ever. The structural scale at which these tests must be undertaken depends on the scale of the model itself. For instance, if the modelling starts at material level, then material tests must be undertaken to establish the mechanical properties necessary for mathematical modelling. An example would be explicit modelling of nails fixing the sheathing to a timber frame. The next hierarchical level are models starting at the scale of structural components such as shear walls using cyclic tests on shear walls as input parameters and neglecting the behaviour of the single nails.

An adequate assessment method seems to be a hybrid approach with higher-level substructure testing such as cyclic testing of shear walls combined with numerical modelling. Testing is necessary to establish system properties under reversed cyclic loading. The complex loading conditions appearing during an earthquake are thus simplified using cyclic loading protocols. With this approach, the models are calibrated on larger-scale testing than pure material testing. These models (and the test results) hence include phenomena such as friction between timber frame and infill that are contributing to the load bearing capacity of a shear wall, but which cannot be captured by smaller joint tests. For an efficient determination of the seismic vulnerability, non-linear dynamic computer models of simplified buildings are better suited than models simulating every single building component. The here proposed hybrid testing-modelling method will be presented in the following section.

Proposed Assessment Method

Testing and Modelling of Shear Walls

As already stated, there are two main issues connected with testing. Firstly, as much relevant information as possible on the seismic behaviour of the investigated timber frame typology should be gained. The second issue is the strong simplification of entire buildings for modelling purposes in order to obtain efficient numerical models for the assessment of seismic vulnerability. Therefore, timber frame typologies are simulated at the "biggest logical scale" at which testing is still possible in terms of costs, time and feasibility. This mostly means tests on complete shear walls which include many contributory effects, e.g. friction, that are difficult to capture at smaller testing levels. Simplified models calibrated on these more realistic test results will be efficient and reliable.

As information on the hysteretic behaviour of the tested shear walls must be collected, quasi-static reversed cyclic tests are usually carried out, e.g. according to EN 12512 [7]. Apart from tests on shear walls, other information may be needed in order to properly model a complete structure. For instance, information on the behaviour of interstorey connections or vertical joints between wall elements may be necessary together with information on the rigidity of the floor diaphragms.

In the following, the proposed procedure is explained by means of traditional timber frame houses of the Cibiana di Cadore region in the Italian Dolomites which were presented in Ceccotti et al. [2]; further information on used materials, building configuration and modelling can be taken from Ceccotti et al. [2]. From the investigated buildings in Cibiana di Cadore, three traditional timber frame wall panels were identified. Figure 9.1a shows a typical timber frame wall panel with an infill of hazelnut branches and mortar, the panel of Fig. 9.1b has a brick infill and Fig. 9.1c shows a panel with stone infill. The timber members of the panel with branches infill were connected by dovetail joints and the members of the panels with brick and stone infill were connected by lapped joints as shown by the sketches on the right side of Fig. 9.1.

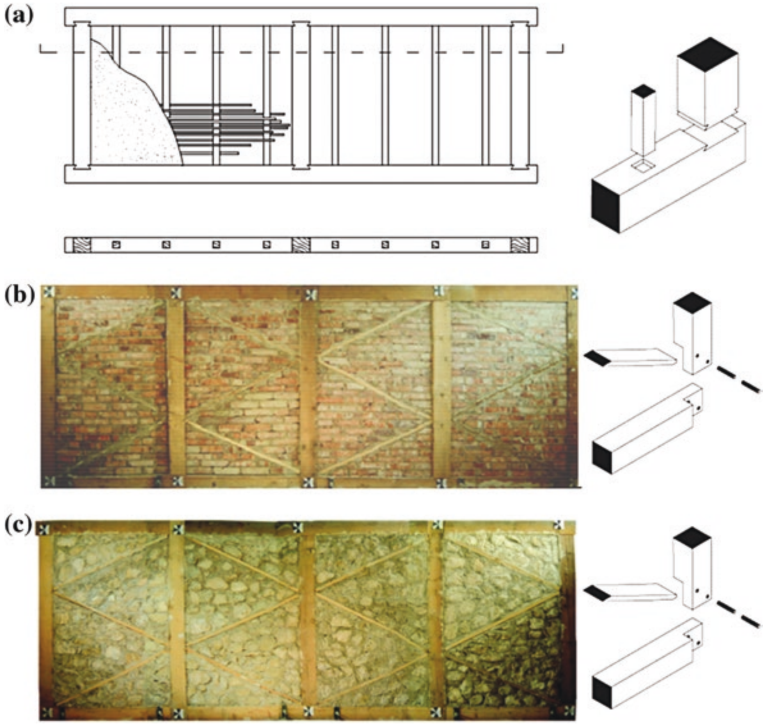


Fig. 9.1 Traditional timber frame panels and joinery, **a** hazelnut branches infill and mortar, **b** brick infill, **c** stone infill

On these three shear wall types, cyclic tests were carried out [2]. Typical results can be seen in Fig. 9.2 where the increase in load bearing capacity and sustainable maximum horizontal displacement for the wall types with stone and brick infill can be seen. All three panel types show plastic hysteretic behaviour with a considerable amount of dissipated energy and can be modelled by spring models with lumped masses where rigid members are representing the timber frame and rotational springs are representing the global behaviour of the panel. Such a simplified model of the timber frame walls shown in Fig. 9.1 is presented in Fig. 9.3. The rotational springs are calibrated on the cyclic test results (Fig. 9.2); only these springs represent the behaviour under horizontal cyclic loading. The timber frame is modelled as being rigid in order to be sure that only the springs are working. The masses are added as lumped masses on the upper nodes of the panels. Due to the model approach, the shear walls are assumed to deform in shear only. Any other deformation, for instance uplift and translation due to rigid body motion, is included in the rotational springs. This assumption is valid for historic timber frame walls as can be shown with test results where these wall typologies mainly fail in shear deformation (e.g. [2, 6]).

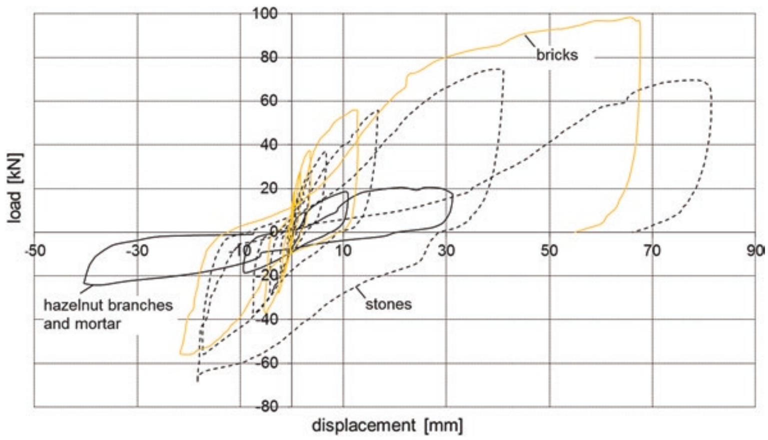
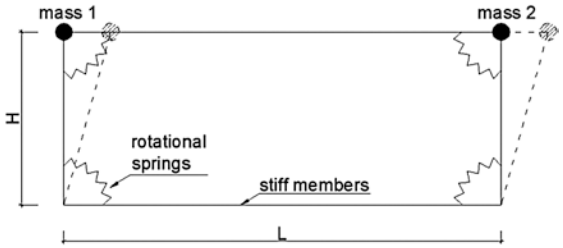


Fig. 9.2 Typical cyclic test results for different timber frames

Fig. 9.3 Model of timber frame wall



Any contributory effects such as friction are implicitly included in the spring models as they are part of the test results. Therefore, precise values for friction are not necessary when modelling at a larger structural scale as proposed here. This is advantageous as friction is difficult to determine, but is a powerful contribution to energy dissipation. By including it implicitly, no rough guesses must be made on friction coefficients used for modelling.

The proposed shear wall model is easy to control as its behaviour is completely governed by the four spring elements. With this approach, it is possible to model traditional timber frame elements whose seismic behaviour is defined by horizontal displacements of the wall elements with no significant uplift.

Calibration of Springs

Plastic deformation capability, hysteresis and energy dissipation are important concepts that determine earthquake behaviour. Models must hence be able to reflect the ductility and energy dissipation capacity of the modelled structure. As

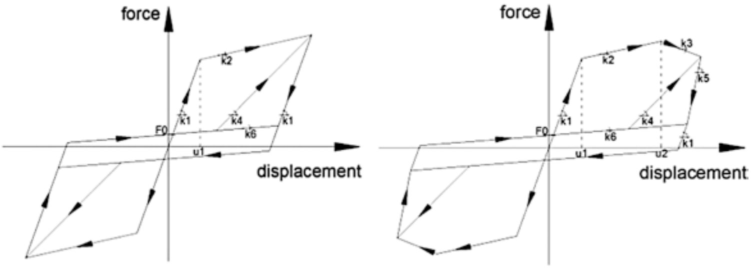


Fig. 9.4 Hysteresis cycle of spring model, *left* 4 inclinations, *right* 6 inclinations

the mechanical behaviour of the structures is governed by the springs due to the chosen modelling approach, a suitable spring model must be used. Ceccotti and Vignoli [8] developed such a spring model to represent the behaviour of semi-rigid joints under reversed cyclic loading. In their piecewise linear model, springs can reproduce the non-linear pinching hysteresis loops of typical semi-rigid joints in timber constructions as can be seen in Fig. 9.4, but their model cannot represent strength degradation at higher cycles.

There are no standardised methods to calibrate the springs. The calibration procedure is iterative and the calibration parameters are maximum horizontal load, maximum displacement and amount of dissipated energy. The correct representation of the envelope curve in terms of maximum load and maximum displacement is prerequisite to proper modelling; a calibration only in terms of dissipated energy could lead to totally different results. With a first estimation of the stiffness values, the displacements and the residual force F_0 as needed by the chosen spring models shown in Fig. 9.4, the cyclic test is practically repeated by applying the time-displacement history of the tests on the wall model of Fig. 9.3. The values for stiffness, displacements and residual force are modified until a good overlap between model (Fig. 9.3) and cyclic test result is reached. In Ceccotti and Sandhaas [3], the iterative spring calibration method is thoroughly explained. As an example, the result of the calibration for the timber frame wall with branch infill (Figs. 9.1a and 9.2) is shown in Fig. 9.5.

Modelling of Buildings

Shear wall models according to Fig. 9.3 are then assembled to represent whole buildings. If the models are in 2D, no decision must be taken on the rigidity of the floor diaphragms. If instead the models are in 3D, then of course the diaphragms must be modelled as well. Either they are considered being rigid or further spring elements are modelled to simulate their elastic or plastic flexibility. However, as discussed by Langenbach [5], after a seismic event, historic timber frame buildings usually show failure in their lateral load bearing elements, i.e. shear walls,

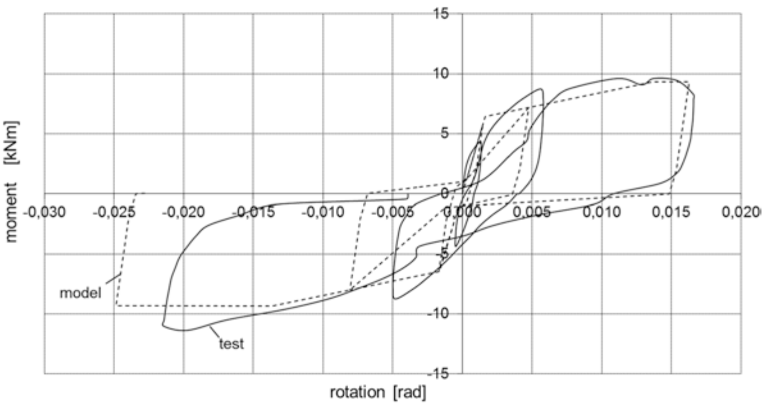


Fig. 9.5 Overlap of model result (*dashed line*) and test result (*solid line*) for timber frame with branch infill, graph in terms of moment-rotation as rotational springs are used

which usually have a low lateral stiffness and strength and thus govern the seismic behaviour. Moreover, historic timber frame buildings generally show symmetry in plan and elevation, consist of one to three stories and do not have a prominent eccentricity between mass and stiffness barycentres which reduces the influence of torsional movements. As a consequence, mostly simple 2D models are sufficient to represent these buildings' seismic behaviour and the influence of floor diaphragms or torsion can be neglected.

Once the springs are calibrated by "repeating" the cyclic tests, the next step is the assembling of a whole building with the calibrated wall elements in order to undertake a non-linear dynamic analysis with earthquake accelerograms as loading. Important parameters such as vertical loads or geometry may differ between cyclic test specimens and a wall in a building. A strategy must be developed in order to transfer the hysteretic springs calibrated on cyclic test specimens into springs determining the behaviour of a building under earthquake loading.

For historic timber frame buildings, this is rather straightforward. In most of the cases, the lateral stiffness can be assumed to be linearly proportional to the wall length; the stiffness of a wall with a length of 2 m is assumed to be half the stiffness of the wall with a length of 4 m. Therefore, to transfer the springs into the buildings, the stiffness values (k_1–k_6 in Fig. 9.4) must be linearly adjusted as well as the residual force at zero displacement (F_0 in Fig. 9.4).

The influence of vertical loads on the stiffness and on the maximum load-carrying capacity of the specimens is considerable. The procedure to solve this is first of all the execution of cyclic tests with well-defined vertical loading derived from a real building situation. Tests with different vertical loading to determine the rate of change of stiffness values can also be undertaken. Basically, few tests are sufficient to estimate the rate of change of stiffness values of structures under different vertical loading. Within the scope of the development of a straightforward and simple method to evaluate the seismic vulnerability whilst needing few tests, a conservative approach is the execution of cyclic tests with a lower vertical load than in real buildings. The springs calibrated on those results are then transferred unchanged into building models which are subsequently underestimating the load bearing capacity and lateral stiffness of the modelled buildings in comparison to the real structure. Then the modelling results are on the safe side.

An important parameter in dynamic modelling is the assumed viscous damping value of the modelled structure which has a high influence on the quality of the results. Already minor changes in viscous damping may change the modelled structural response significantly [9]. At low levels of displacement, the hysteretic spring models do not provide any energy dissipation which means that all energy dissipation at that load scale must be provided by the estimated viscous damping. In Folz and Filiatrault [9], viscous damping values of 2–5 % were used to model modern platform frame buildings, and the importance of the assumed damping values on the model results was discussed; choosing inappropriate damping values may lead to significantly wrong model results. Values of viscous damping between 2 and 5 % are rather high and it is recommended to use a viscous damping of about 1 % in order to obtain modelling results that are lying on the safe side.

Now, all necessary parameters for a non-linear dynamic analysis are defined except for the masses. The determination of masses and their distribution to lumped nodes will however be the least problem as the construction type and the used materials will be known.

Non-linear Dynamic Analysis

Once the building models are developed, they are subjected to a series of earthquakes increasing the PGA values until a previously defined near-collapse state is reached. The necessary near-collapse criterion is usually an ultimate lateral displacement that can just be sustained by the timber frame shear wall before collapsing. If no collapse is reached, the near-collapse displacement can also be taken at 80 % of the maximum load bearing capacity. The near-collapse criterion can be established by means of the test results as horizontal displacements are direct derivatives from the tests. The chosen near-collapse criterion is directly related to the construction typology; it is natural to choose a maximum horizontal displacement (the so-called interstorey drift) as a criterion for historic timber frame buildings and not a maximum uplift.

A remark is given on the choice of earthquakes. In order to represent one specific seismic region, geologically possible earthquakes for this seismic region should be chosen. This may include the use of artificially generated earthquakes. The frequency content of the earthquakes has to cover a broad range in order derive reliable information on the seismic vulnerability of the investigated traditional timber frame construction technique.

Summary

The proposed method is a simple and straightforward approach to assess the seismic vulnerability of historic timber frame buildings and can be summarised as follows:

- Monotonic and cyclic tests on exact copies of historic timber frame walls are carried out and analysed as to their maximum horizontal load and displacement, energy dissipation and near-collapse failure criterion (mostly ultimate horizontal displacement).
- Global cyclic test data on shear walls are fitted to shear wall models assigning the mechanical behaviour to rotational hysteretic springs which are able to reproduce pinching behaviour. The fitting parameters are envelope curve and energy dissipation.
- The shear wall models are used to generate building models with properly determined and distributed lumped masses and an estimation of the damping rate. The building's behaviour is completely governed by the hysteretic springs which are transferred from the shear wall models into building models by adjusting their stiffness values in order to consider e.g. different wall lengths or numbers of fasteners.
- The building models are subjected to accelerograms of various earthquakes covering a wide range of frequencies. The earthquakes' PGA values are increased until the near-collapse state is reached and therefore, the seismic vulnerability of the buildings can be assessed.

Example

The symmetric two-storey historic timber frame building presented in Ceccotti et al. [2] is investigated. The timber frame building typical of the Cibiana di Cadore region is 5 m wide and 10 m long with an inner wall and consists of wall panels with an infill of hazelnut branches and mortar (Fig. 9.1a). The inner wall with a length of 5 m is subjected to an earthquake excitation. The masses are calculated considering a influence width of 5 m, assuming a traditional timber frame construction with a timber joist floor and adding 30 % of additional load to account for furniture and occupancy. A viscous damping of 5 % was assumed. The calibration of the shear walls was shown previously in Fig. 9.5 and the building model is shown in Fig. 9.6.

From the cyclic test results, a maximum load-carrying capacity and a near-collapse criterion can be derived. Considering Fig. 9.2, the maximum load bearing capacity of a timber frame wall with an infill of hazelnut branches and mortar is 20 kN and the near-collapse displacement is chosen to be 30 mm.

A non-linear dynamic analysis in the time domain is carried out on the fully defined building using different earthquakes. The PGA values of the earthquakes are increased until the near-collapse displacement of 30 mm is reached. The same procedure was carried out for a whole series of earthquakes. The results are shown

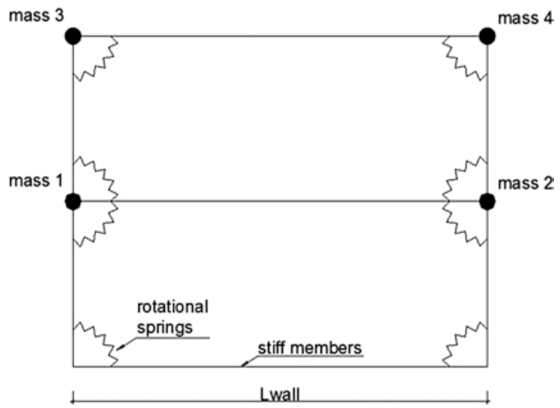


Fig. 9.6 2D model of traditional timber frame

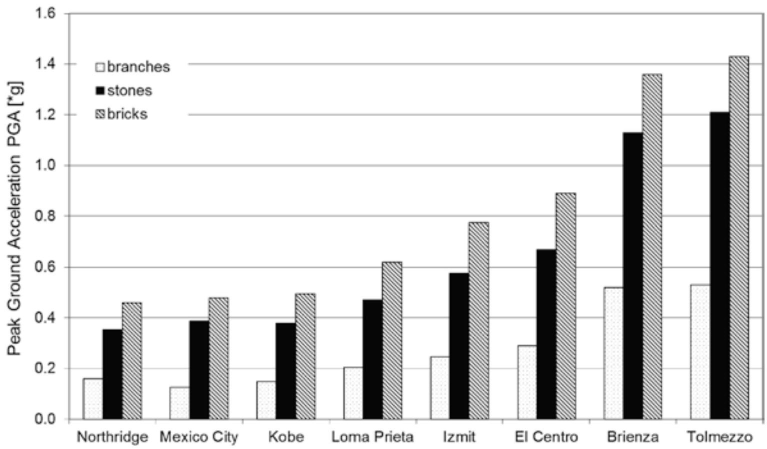


Fig. 9.7 Peak ground acceleration at near-collapse state of a two-storey traditional timber frame house

in Fig. 9.7 in terms of $PGA_{near\text{-}collapse}$ for eight different earthquakes and a two-storey building with the three different panel types shown in Fig. 9.1. It can be seen that the historic building with timber frame walls with a brick infill can sustain the highest PGA values whereas the typology with stone infill showed similar cyclic behaviour, but reaches the near-collapse state already at lower PGA values. The building typology with an infill of branches and mortar can sustain only smaller earthquakes because it has the lowest load bearing capacity and ultimate displacement of all types as can be seen in Fig. 9.2.

But, if the ratio between "static" and "seismic" load bearing capacity is considered, the panels with branches infill are gaining on the panels with stone infill. Whereas the panels with stone infill have about three times the static load bearing capacity of panels with branches infill (see Fig. 9.2), their seismic load bearing capacity is only about two times the one of the panels with branches infill (see Fig. 9.7). Furthermore, in Fig. 9.7, the dependency of the structural answer on the earthquake input is clearly visible. The brick-infill-type for instance reaches its near-collapse state during the Northridge earthquake at a PGA of 0.46 g, but during the Tolmezzo earthquake, it would have survived a PGA of 1.43 g.

Retrofitting

The potential application fields of the proposed method can be extended to the assessment of retrofitting interventions. In order to investigate this, the dovetail joints of the timber frame panel with an infill of hazelnut branches and mortar shown in Fig. 9.1a have been reinforced in order to increase the lateral strength. Two retrofitting actions have been undertaken and are shown in Fig. 9.8. One modification comprised the insertion of inclined screws and the second modification generated dovetail joints with inclined screws and EPDM damping pads.

Figure 9.9 shows the cyclic test results of the three panel types with branches infill. In comparison to the original panel with no reinforcements, the panel reinforced with inclined screws could reach twice the maximum load at the same maximum displacements whereas the panel reinforced with inclined screws and EPDM pads reached again twice the maximum load, but also higher displacements. At the same time, the retrofitting measures did not decrease the energy dissipation capacity; the hysteresis loops are still broad.

The findings from cyclic test results already indicate that the retrofitted panels are expected to show better earthquake behaviour than the original timber frame panel. If the above-presented two-storey timber frame house model is updated with the new shear walls and then subjected to earthquake accelerograms, these speculations can be verified with a non-linear dynamic analysis. The analysis results are shown in Fig. 9.10. The near-collapse state of the retrofitted timber frame buildings is indeed reached at higher PGA values where, except for the two Italian earthquakes, no significant improvement could be found between the panels reinforced with screws only or with screws and damping pads. The earthquake of Mexico City represents another exception to this finding. During the Mexico City quake, reinforcement with screws does not significantly change the sustainable PGA whereas the sustainable PGA is increased when reinforcement with screws

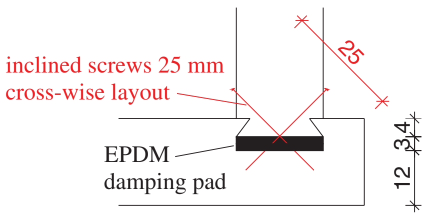


Fig. 9.8 Timber frame panel with infill of branches and mortar, detail of reinforced dovetail joint

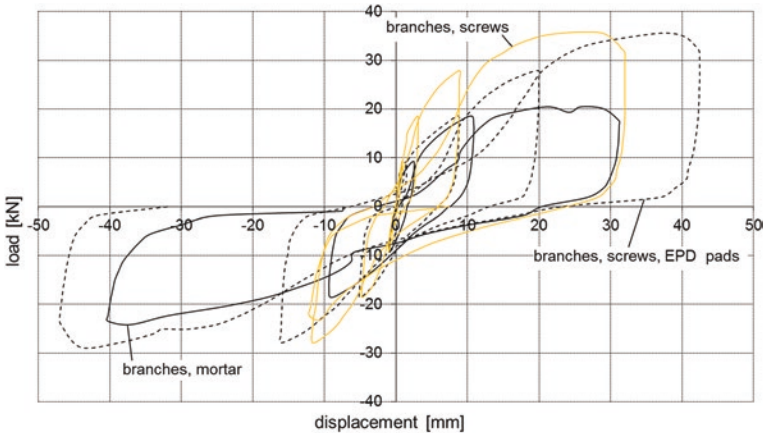


Fig. 9.9 Typical cyclic test results for different timber frames

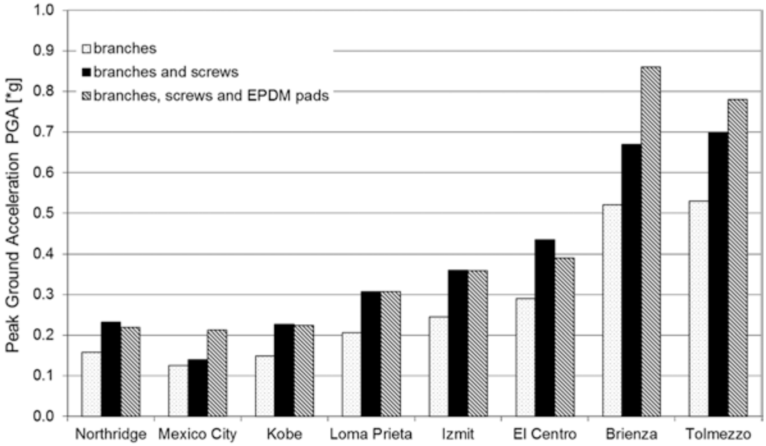


Fig. 9.10 Peak ground acceleration at near-collapse state of a retrofitted two-storey traditional house

and damping pads is chosen. The structural response during a seismic event is thus clearly depending on the seismic event itself.

Conclusions

The proposed method is a simple and straightforward procedure to assess the seismic vulnerability of historic timber frame structures and to evaluate the effectiveness of restoring and retrofitting interventions. Only a minimum of tests is needed to develop a computationally efficient nonlinear dynamic model where energy dissipation and pinching behaviour of typical semi-rigid timber joints are considered.

It could be shown that cyclic tests alone are not sufficient to fully assess the seismic vulnerability of structures which underlines the necessity of the proposed method. Timber frame typologies that show higher lateral stiffness and resistance, e.g. with brick or stone infill, do not necessarily behave better during a seismic event than weaker typologies, e.g. with branches infill, as the ductility and energy dissipation capability of the used typologies play an important role. Structures with a high lateral stiffness have to sustain higher seismic loads than structures with a low lateral stiffness which in the case of an earthquake event may penalise the stiffer structures. Also, typologies that behaved similar during cyclic tests may still sustain different peak ground accelerations at near-collapse state.

Furthermore, the effectiveness of retrofitting interventions can be studied. As above, also for retrofitting interventions, not always better behaviour during cyclic tests results in better earthquake resilience. With the proposed method, the extent of retrofitting that gives a desirable increase in seismic resilience can be defined. The proposed method also allows for the use of different earthquake accelerograms as load input which has been shown to be necessary to assess seismic vulnerability. Different earthquakes may lead to different peak ground accelerations at near-collapse state which again cannot be captured by cyclic testing only.

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Chapter 10
An Overview on the Seismic Behaviour of Timber Frame Structures

Graça Vasconcelos, Paulo B. Lourenço and Elisa Poletti

Abstract Masonry and timber are materials used since ancient times in construction. Masonry buildings constitute an important percentage of the existing building stock and their preservation should be considered, since a large part of historic buildings are actually in masonry.

Keywords Timber frame buildings · Traditional construction · Seismic performance

Introduction

Masonry and timber are materials used since ancient times in construction. Masonry buildings constitute an important percentage of the existing building stock and their preservation should be considered, since a large part of historic buildings are actually in masonry. A drawback on the use of unreinforced masonry is its vulnerability to seismic actions, which is in part associated to the low resistance to tensile stresses. A historic construction solution to improve the mechanical behaviour of ancient masonry adopted in different locations at different times, namely in seismic regions, has been the reinforcement of masonry with timber.

Traditional timber frame walls are an important structural element of many buildings and are usually composed of vertical posts and horizontal beams with diagonal bracing elements. In Portugal, timber frame walls, known as *frontal*

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walls, are usually part of Pombalino buildings, which were introduced by the Marquis of Pombal, who was responsible for the reconstruction of Downtown Lisbon after the great earthquake of 1755. The timber-framed walls are connected to the external masonry walls by means of the timber floor beams, which are connected both to the timber-framed and to the external masonry walls [1]. This system can be beneficial to reduce the out-of-plane vulnerability of masonry walls. Timber frame walls are also identified in several countries, particularly in local vernacular architecture, due to the low cost of such structures composed of timber and varying infill materials, from brick and stone masonry to mud and cane.

Given the increasing interest of the research community in this structural system, it is important to promote the discussion of the main findings that can contribute to the advance on the knowledge of the mechanical behaviour of timber frame buildings under seismic action.

Therefore, this paper intends to: (1) give an overview of the different solutions of timber frame structures in different countries with special focus on the frontal walls characteristic of Pombalino buildings; (2) provide some examples of the reasonable behaviour of timber frame buildings in past earthquakes; (3) summarize the experimental research carried out in recent years, analysing the behaviour under in-plane cyclic loading.

A Brief Overview on the History of Timber Frame Buildings

The origin of timber frame structures probably goes back to the Roman Empire, as in archaeological sites half-timbered houses were found and were referred to as Opus Craticium by Vitruvius [2]. But timber was used in masonry walls even in previous cultures. According to [3, 4] in the Minoan palaces in Knossos and Crete, timber elements were used to reinforce masonry. Later, half-timbered constructions spread not only throughout Europe, such as Portugal (edifícios Pombalinos), Italy (casa baraccata), Germany (fachwerk), Greece, France (colombages or pan de bois), Scandinavia, United Kingdom (half-timber), Spain (entramados) etc., but also in India (dhaji-dewari) and Turkey (himis) [2, 5]. In each country, different typologies were used, but the common idea is that the timber frame can resist to tension, contrary to masonry, which resists to compression, thus providing a better resistance to horizontal loads. Besides, timber elements are viewed as a sort of confinement to the masonry structure, improving the mechanical properties to shear loads. In general, the cross section of timber elements in the distinct case studies is very similar (approximately 10 × 12 cm).

Timber frame buildings were common all over Greece in different periods, as reported by many authors [6–8]. Examples of this system are the monastic buildings in Meteora and Mount Athos, the post byzantine (Ottoman period) buildings in Central and Northern Greece and the traditional buildings in the island of Lefkas. These buildings consisted of a stone masonry ground floor plus one or two

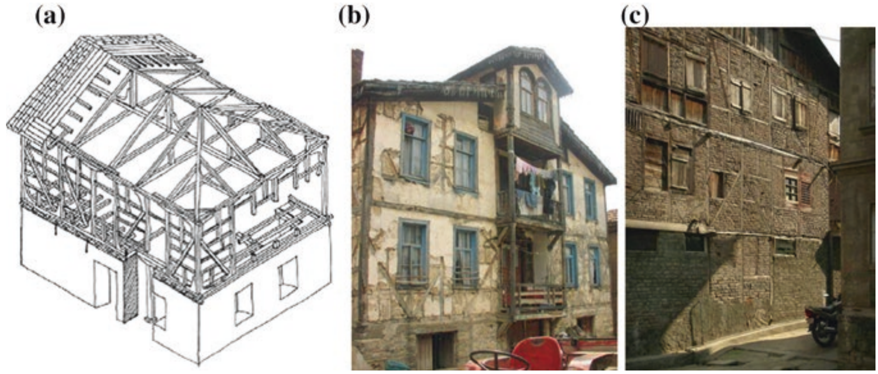

Fig. 10.1 Some examples of timber frame buildings, **a** typical house of Lefkas island in Greece, built with the local aseismic technique [7], **b** in Turkey—hatil at ground floor and himis in upper storeys [8], **c** India—dhajji-dewari building in Kashmir [2]

timber-framed masonry storeys (Fig. 10.1a), which represent a common disposition in timber frame buildings. In the case of timber frame construction in Lefkas, an innovation that was introduced (which proved to be particularly effective for seismic loads) was the introduction, at the ground floor, of timber columns stiffened by angles that constituted a secondary load bearing system in case of failure of the masonry walls, since they were connected to the timber-framed structure of the upper storeys [5].

In Germany, *fachwerk* construction was very popular and several examples of timber frame constructions are present all over the country. Different timber frame styles can be found, characterized by a varying number of storeys and geometry of the timber frame. In Germany, this construction system was introduced in the 7th century and it flourished particularly in the 16th and 17th century. Three main styles can be recognized (Alemannic, Lower Saxonian and Franconian), differentiating mainly with regardto spacing between the elements, dimensions and disposition of the framing. An example of the German constructions is presented in the lexicon by Otto Lueger [9].

Another example of timber frame construction is the *casa baraccata* in Italy. After the 1783 earthquake in Calabria, authorities adopted construction methods similar to those imposed some decades before in Lisbon. The same construction technique, with slight changes, was also adopted after the Messina earthquake in 1908. In particular, Vivenzio proposed a 3-storey building with a timber skeleton aiming at reinforcing the external masonry walls, avoiding their premature out-of-plane collapse. The timber-framed walls constituted the internal shear walls, presenting a bracing system of S. Andrew's crosses, similar to what can be found in Lisbon [10].

A difference to the Portuguese solution is the continuity of the vertical timber posts from the foundation to the roof, being anchored in the foundation (especially in buildings built after 1908) [11]. Similar houses were also found in India and

Turkey. Turkey is a prone seismic zone and is frequently subjected to strong earthquakes, meaning that the buildings need to be able to resist to seismic actions. Besides, Turkey has an abundance of wood, as well as stone and clay, which promoted the growth of timber frame structures. The typical timber frame construction used in the upper floors is called *himis* and it is typically constituted of a timber frame filled with rubble or brick masonry [12] (Fig. 10.1b). An alternative to masonry infill can be found in *bagdadi* constructions, where short rough pieces of timber are used as infill material. These led to lightweight, seismic resistant, economical structures, but they were more disposed to decay [12]. Among India's traditional buildings, a half-timbered construction typology can be distinguished in the *dhajji-dewari* (patchwork quilt wall) system, which is a braced timber frame with masonry infill, frequently used for the upper storeys of buildings (Fig. 10.1c). Buildings date as back as the XII century [2].

Timber frame construction has also been used in South America. In Peru, for example, the *quincha* presents a one-storey timber frame made of round or square wood (bamboo is often used) and filled with canes covered with earth and gypsum [13]. This type of construction was for example proposed by Peruvian experts for the reconstruction of Haiti after the severe earthquake of 2010 [14]. In Haiti, timber frame construction was introduced by immigrants, and it performed well during the earthquake. The posts are grounded in a concrete foundation, the infill consists of canes covered with clay and mud and, once dried, everything is covered with a cement plaster.

The Portugese Example

In Portugal, typical half-timbered structures are known as Pombalino buildings, which are old masonry buildings constructed after the 1755 Lisbon earthquake, which destroyed Downtown Lisbon. The new buildings took their name from the prime minister of the time, the Marquis of Pombal, who encouraged the reconstruction of the city. A Pombalino building is characterized by external masonry walls up to 5 storeys. The ground floor consists of stone masonry columns supporting stone arches and clay brickwork vaults and above the first floor it develops an internal timber structure, named *gaiola* (cage), see Fig. 10.2. The *gaiola* consists of horizontal, vertical and diagonal bracing members, forming a three–dimensional braced timber structure. These timber-framed walls are filled with rubble brick or rubble stone masonry and act as shear walls. The length of a typical building is 8 to 16 m and the width is about 10 m. The internal walls of the *gaiola* (paredesem frontal) may have different geometries in terms of cell dimensions and number of elements, as it depends greatly on the available space and the manufacturer's customs [1]. The main horizontal and vertical elements are reasonably long, whereas the diagonal ones are very short. The timber elements are notched together or connected by nails or metal ties.

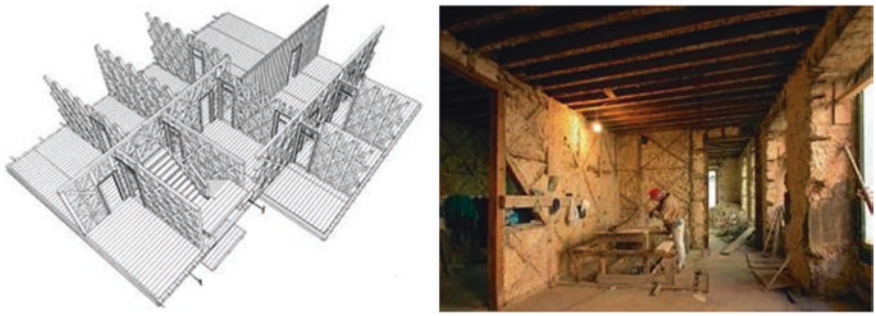

Fig. 10.2 The Gaiola system in Pombalino buildings [1]

Traditional connections used for timber elements varied and could be mortise and tenon, half-lap, dovetail connections, and other types of notched connections. A wide range of sectional dimensions can be found in the elements: diagonal members are usually smaller (10 × 10 cm or 10 × 8 cm), whereas vertical studs and horizontal members are bigger (usually 12 × 10 cm, 12 × 15 cm and 14 × 10 cm or 15 × 13 cm, 10 × 13 cm and 10 × 10 cm).

The sectional dimensions of the elements are usually bigger for the lower storeys, decreasing progressively with the height of the building. The frontal walls have a width of 15–20 cm, with a grout thickness covering the masonry infill of about 2.5 cm but it could vary up to 5 cm [1, 15]. The frontal walls act as shear walls in the building but can be considered also as partition walls. The peculiarity of this type of building is that under a seismic event, it is admissible that the heavy masonry of the façades falls down, as well as the tiles of the roof and the plaster of the inner walls, but the timber skeleton should remain intact, keeping the building standing. It should be stressed that if the connections between the external masonry piers and the internal timber-framed walls are adequate, the out-of-plane collapse mechanisms of the external façades is also minimized. Some timber elements can be found in the external walls to promote the connection between the *gaiola* and the external masonry walls [16, 17].

Seismic Performance of Timber Frame Structures—Evidences from Past Earthquakes

Based on the analysis carried out on the damage state of traditional timber frame buildings located in high prone seismic regions after important seismic events, it has been seen that a very reasonable behaviour is exhibited by this structural system in distinct countries with high seismicity [18]. Timber frame structures combine the best features of masonry and timber, offering a better overall behaviour of the

Fig. 10.3 Examples of damages in timber frame buildings, **a** out-of-plane collapse of masonry infill (Lefkada, Greece) [6], **b** comparison of damages to traditional and modern building after the 1999 Duzce earthquake, **c** failure of connection in timber frame (1999) Kocaeli earthquake [14]

buildings under seismic actions. With this respect, it is important to consider that the state of conservation of the traditional buildings can influence their seismic behaviour.

After the strong earthquake in 2003 in Lefkas, a high prone seismic island, it was observed that in spite of damages developed in traditional timber frame buildings, they were not as severe as the ones observed in reinforced concrete buildings and no collapse of traditional buildings was recorded. The damages observed included vertical and diagonal cracks and, in some cases, collapse of the stone masonry walls at the ground floor, shear cracks at the interface between timber frame and masonry infill, which in a certain extent promoted the out-of-plane collapse of the infill (Fig. 10.3a), crushing of the infill masonry. Almost no damage was found in the wood elements of the timber frame [7]. Another example, where the efficiency of timber frame structures was tested, consists of the traditional timber frame buildings in Turkey, already described. Turkey is frequently exposed to severe earthquakes, being one of the few countries with the shortest return period in earthquakes causing often loss of lives [14]. Different authors have pointed out the reasonable earthquake resistance of timber frame buildings, especially compared to other structural systems such as masonry or reinforced concrete structures (Fig. 10.3c), namely during the 1894 Istanbul earthquake, 1970 Gediz earthquake and more recently the 1999 Marara (Kocaeli) earthquake [14].

According to Gülhan and Güney (2000) [14], in Kocaeli-Gölcük, in the Sehitler district, 51 % of the buildings are RC buildings (up to 7 storeys), while the rest are traditional (either half-timbered or timber-laced masonry or plain masonry up to three storeys). Among these, only 0.5 % of the traditional structures presented heavy damages or collapsed against 7.4 % of the RC structures, 0.6 % of the traditional structures presented moderate damage versus 8.6 % of the RC and 10 % and 16.5 % respectively presented light damages. In all the mentioned earthquakes, a low number of total collapses of traditional buildings was recorded, even if light to severe damage could develop depending on the conservation of the structure, on the materials, and on the structural features of the system. Typical damages in timber frame buildings under seismic actions include: (1) cracking and failure of plaster as the result of deformation of the braced elements and posts. When reduced space of the posts exists no propagation of the cracking occurs for the masonry infill; (2) loosening and failure at the connections (Fig. 10.3c). In fact, the connections take a central role on the seismic behaviour of traditional timber frame buildings as they are the elements keeping the structure together during the earthquakes, being understandable that important deformations and damages can develop; (3) large lateral displacements, which can result from soft-storey mechanism, resulting from the changes carried out on the original buildings at the first floor (e.g. removal of timber braced elements and studs aiming at having free spaces for commercial purposes, resulting in the alteration of the structure stiffness in height, leading to global collapse of the buildings).

The earthquakes of India 2001 and El Salvador 1986 are other two examples where the timber-laced masonry buildings and the *Bahareque* timber frame buildings behaved considerably better than reinforced concrete or unreinforced masonry [18]. The heavy damage and inadequacy of timber frame buildings under earthquakes, as occurred in Nicaragua 1936, can often be attributed to the poor condition of the connections due to inadequate conservation. More recently, during the earthquake of Haiti in January 2010, it was seen that a great number of concrete block and reinforced concrete buildings were heavily damaged, resulting in the loss of a dramatic number of human lives and in a huge economic impact on society [19]. Contrarily, the behaviour of traditional timber frame buildings did not exhibit such severe damage. Both the braced timber frame and the colombage, with more flexible and energy dissipating systems, tended to perform better than the other structural systems (masonry and reinforced concrete) [19].

Experimental Research on Timber Frame Walls

In spite of timber-frame walls being very common all over the world and behaving reasonably well during past earthquake events, very little information is available on their experimental seismic behaviour that enables to understand the resisting mechanisms under lateral loading. In fact, this type of construction system has not been taken into great consideration from the scientific research community but a great number of historic buildings consists of timber frame structures, which means that the evaluation of its mechanical performance, particularly to seismic actions, can be valuable. Moreover, the great variability found in these buildings in terms of geometry, materials and modifications introduced in the structures makes their seismic assessment a relevant research issue.

With this respect, only in the last decade experimental studies have been carried out in different countries for the evaluation of the in-plane lateral performance of distinct types of timber frame walls. Therefore, this section aims at giving an overview on the experimental analysis of timber frame walls under in-plane cyclic/monotonic loading by presenting the main outcomes.

Experimental Research on Frontal Timber Frame Walls

In relation to Pombalino timber frame walls, few experimental information is available so far. From the *frontal* walls point of view, the first experimental work was carried out by Santos (1997) [20], in the scope of a rehabilitation program of ancient masonry buildings. Three specimens of real walls were taken from an existing building which was going to be demolished and tested under static cyclic loads. It should be noticed that no vertical load was applied and a concrete beam was cast to secure the base beam. The hysteresis loops of the tested wall, shown in Fig. 10.4a, are indicative of the good deformation capacity and energy dissipation capacity of the structure. Cyclic tests were also carried out by Meireles et al. [21] on walls similar to the ones tested by Santos (1997) [20]. The wood specie selected was Pinus pinaster, a typical Portuguese softwood. The nails were used they were assembled according to what is seen in existing walls (number and positioning).

For the beams and posts a cross section of 12×8 cm^2 was used and for the diagonals a section of 10×7 cm^2 was adopted. Half-lap connections were used between beams and vertical posts and between diagonal bars, and two nails were additionally inserted. The diagonal bars were connected to the beams and posts through nails. A nail was also used to connect each two diagonal braces. For the infill material it was decided to use brick masonry made with low strength hydraulic lime. The walls were tested under cantilever boundary conditions, as the top of the wall was free to rotate. The bottom beam was fixed to the reaction structure so that uplift was avoided. The vertical load applied was about 80 kN aiming at simulating the dead and live load of a building, with typical three stories plus the ground floor, by means of four hydraulic jacks. The tests were carried out under displacement control by using the CUREE-Caltech loading protocol. The hysteresis loops obtained for the two *frontal* walls allow to observe that in-plane lateral response is characterized by a considerable non-linear behaviour, with the hysteresis loops predicting reasonable energy dissipation (Fig. 10.5a). The response is also characterized by pinching, which is associated to cumulative damage at the connections and progressive increase on the plastic deformations, similarly to what recorded in the tests of Santos (1997) [20]. The collapse of the

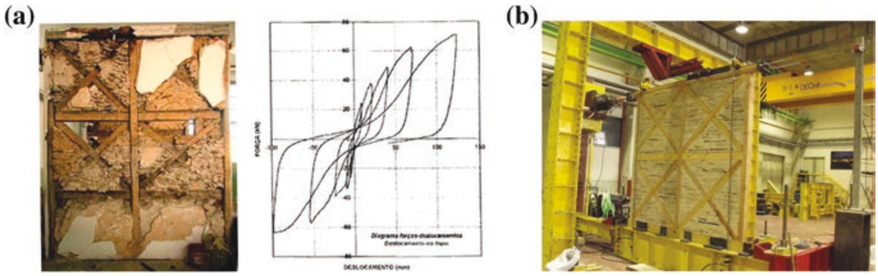

Fig. 10.4 Experimental testing of frontal walls, **a** Santos (1997) [20], **b** Meireles et al. (2012) [21]

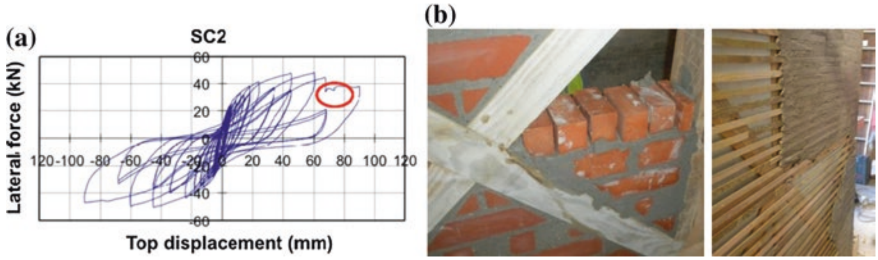

Fig. 10.5 **a** Typical force–displacement diagrams [21], **b** infill material for frontal walls [22]

walls occurred for a lateral drift of 3.5 %. In-plane cyclic tests were carried out by Poletti and Vasconcelos (2014) [22] on the same type of walls. In this case, the dimension of the braced diagonal cell is lower (total height and length 8 % lower). Only one nail was used in all half-lap connections and regular brick masonry was considered as infill material. Alternatively, lath and plaster was adopted as infill material, see Fig. 10.5b. Besides the timber frame infill walls, also empty timber frame walls were tested.

The vertical load was applied directly to the posts. Two levels of vertical loads were considered, namely 25 and 50 kN per post. The typical load-displacement diagrams are presented in Fig. 10.6 for brick infill and empty timber frame walls for the two levels of vertical load. From the analysis of these diagrams, it is possible to observe that: (1) the timber frames filled with masonry and lath and plaster present similar behaviour, being the predominant resisting mechanism characterized by flexure, corresponding to the uplift of the lateral posts and rotation of the wall. This resisting mechanism leads to plastic deformation of the nails placed at the bottom half-lap connections, which should be responsible for the plateau characterising the unloading branches; (2) the timber frame walls exhibit typical shear behaviour being the force-displacement diagrams characterized by pinching resulting from the cumulative deformations observed in the walls, particularly at the connections. The failure mode is characterized by the shear collapse of the central connections; (3) the infill materials (masonry brick and lath and plaster) influence the resisting mechanism of the timber frame walls. The resisting shear mechanism of plane timber frame wall is replaced by flexural rocking mechanism in case of infill material being added. Infill acts as a confining element, influencing the deformation of the connections; (4) the vertical load applied on the posts influences the lateral resistance and the overall behaviour of the walls. The increase on the vertical load results in the increase of the lateral resistance. On the other hand, higher vertical loads lead to the decrease of the vertical uplift of the posts, mainly in case of filled walls, meaning that the flexural rocking mechanism that prevails in the response of the lowest vertical load is reduced. It is possible that the higher stiffness of the brick masonry used in case of Poletti and Vasconcelos (2014) [22] results in the higher stiffening effect of the connections leading to a predominant flexural behaviour, contrarily to the shear behaviour achieved by Meireles et al. (2012) [21].

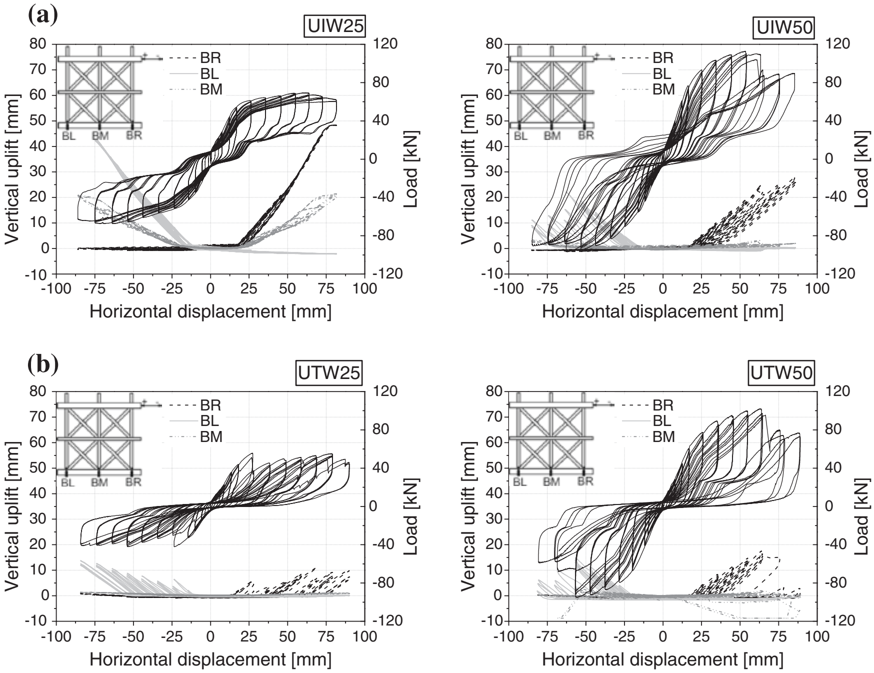

Fig. 10.6 Force-displacement diagrams obtained for *frontal* walls tested by Poletti and Vasconcelos (2014) [22], **a** walls filled with brick masonry submitted to a vertical load of 25 kN/post and 50 kN/post, **b** empty walls submitted to a vertical load of 25 and 50 kN/post

This appears also to be valid for the lateral resistance, as the lateral strength obtained by the authors is higher than the one pointed out by Meireles et al. (2012) [21], taking into account that the same vertical load was applied. The predominant flexural behaviour found for the lowest vertical pre-compression levels was also obtained by Gonçalves et al. (2012) [23], who carried out in-plane cyclic tests in the same type of walls tested by Poletti and Vasconcelos [22]. It should be noticed that in these two works only the brick masonry infill was not the same. In all mentioned experimental works the timber frame detached from the masonry for increasing lateral displacements. In the tensile part of the frame masonry does not work at all, being only active in the neighbourhood of the compression strut of the opposite side. The detachment pointed out by Meireles et al. [21] is more associated to the shear deformation of the timber frame. In terms of lateral drift, the values obtained by Poletti and Vasconcelos (2014) [22] for all walls was close to 4 %, meaning that it was a little higher than the value pointed out by Meireles et al. (2012) [21] of 3.5 %. It should be mentioned that the values obtained by the authors could be even higher for some walls, particularly the ones submitted to the lowest levels of pre-compression, as the maximum displacement did not correspond to the collapse of the walls. In relation to the values of equivalent viscous damping, it should be mentioned that the authors found higher values for low lateral drifts when compared to the values found for higher lateral drifts, being in average 0.1 for infill timber frame walls and 0.12 for timber frame walls for high lateral drifts. These values are of the order of the ones found by Gonçalves et al. (2012) [23], on similar traditional Portuguese *frontal* walls, which obtained values of viscous damping for low values of drift of 0.17–0.20 for infill walls and 0.19–0.20 for empty timber frame walls. The values then decreased to 0.11–0.13 and 0.10–0.11 for infill walls and empty timber frame walls respectively, confirming the trend of having higher values of equivalent viscous damping for low drifts. The values of the equivalent viscous damping obtained by Vasconcelos et al. (2013) [24] for 1:2 reduced scale "frontal" walls tested under in-plane cyclic loads was about 0.15. This higher value can possibly be attributed to the distinct "frontal" walls typology and connections: additional vertical and horizontal bars in the braced cells and mortise and tenon connections in the intersection of beams and posts. From this work, it was possible to observe that the equivalent viscous damping depends on the resisting mechanism, being higher when shear response predominates. In these walls, lateral drifts of about 3.5 % were obtained, being comparable with the values obtained in the other studies.

Experimental Research on Other Timber Frame Systems

In this section a very brief overview is made in relation to experimental research carried out on timber frame walls that are characteristic of different countries, namely Peru and Turkey (Fig. 10.7). Some notes are also given about the work carried out on the construction system used in the reconstruction of Haiti after the earthquake of January 2010 [25].

The traditional timber frame walls used in the reconstruction of houses in Haiti, whose shape is similar to *frontal* walls of Pombalino buildings (Fig. 10.8a), exhibit a clear nonlinear behaviour under in-plane lateral cyclic loading with important pinching effect. The shape of the hysteresis is very similar to the one obtained in the experimental results pointed out by Meireles et al. (2012) [21]. In this work, it is possible also to assess the influence of the infill rubble stone masonry. Similarly

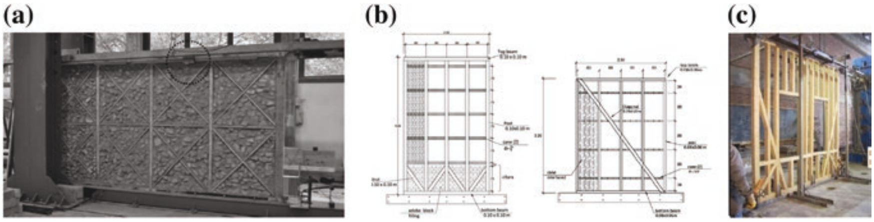

Fig. 10.7 Timber frame walls: **a** system used in the reconstruction of Haiti [25], **b** *quincha* walls [26], **c** timber frames characteristic of himis construction, Turkey [27]

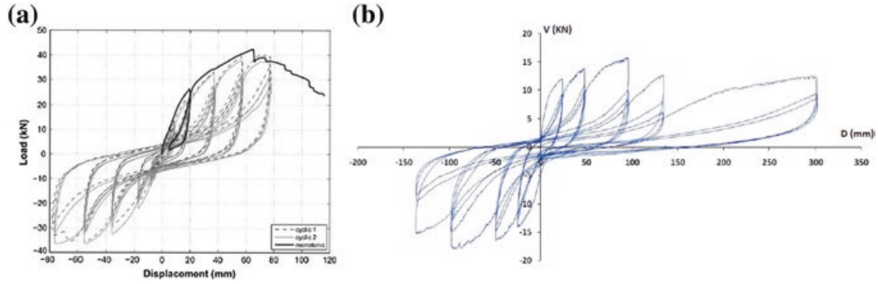

Fig. 10.8 Typical force-displacement diagrams, **a** reconstruction of Haiti [25], **b** *quincha* walls [26]

to what was pointed out by Poletti and Vasconcelos [22] and Gonçalves et al. [23], the addition of an infill material leads to an increase on the lateral strength and stiffness.

From the monotonic envelope it appears that the timber frame wall without infill has more ductility. The force-displacement diagrams obtained in the *quincha* walls (Fig. 10.8b) are also characterized by remarkable nonlinear behaviour and pinching effect.

These walls presented an important capacity to deform nonlinearly, having great lateral drifts of 7.5 % for the walls with Citara (struts at the base of the wall, Fig. 10.7b—*left*) and of 9.375 % for the wall with diagonal (Fig. 10.7b—*right*). The experimental work carried out by Aktas et al. (2012) [27] on in-plane cyclic testing of timber frames characteristic of himis construction, with and without brick masonry and wood laths and plaster (bagdadi), revealed that the behaviour of timber frames is controlled by the behaviour of their connections, being the damage concentrated at the connections. Besides, the high capacity to deform in the nonlinear range is confirmed by the great lateral drifts of about 6 % in case of empty frames, of about 5.5 % in case of brick masonry infill and of about 4.9 % in case of lath and plaster finishing (bagdadi cladding). Additionally, it was seen that according to what was pointed out by other authors [22, 25], the addition of an infill material and cladding is responsible for the increase on the lateral stiffness and lateral resistance.

Concluding Remarks

This paper aimed at providing an overview on the seismic behaviour of traditional timber frame construction based both on the evidences from the past earthquakes and from the recent experimental works on in-plane cyclic testing. From the analysis made, it is observed that there are several indications of the very reasonable behaviour of timber frame construction subjected to important seismic events. This is, in certain extent, also observed from the experimental works, as timber frame walls exhibit a large capacity to deform in the nonlinear regime with remarkable lateral drifts and controlled damages under in-plane cyclic loading. In general, their in-plane behaviour is considerably better than that of unreinforced masonry walls, used in vernacular architecture in several countries with important seismicity, meaning that traditional timber construction deserves to be conserved and can be viewed as a true alterative for reconstruction and strengthening purposes of traditional vernacular construction.

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Chapter 11
Practical Simulation Tools for the Seismic Analysis of Timber-Framed Masonry Structures

Leonidas Alexandros S. Kouris

Abstract A non-linear (NL) empirical macro-model is presented for NL static analysis of Timber-Framed (T-F) masonry structures. This model is based on the diagonal strut approach with NL axial hinges in the struts. The constitutive law for the hinges in terms of axial force vs. axial deformation is a function of the main factors affecting the response of T-F masonry panels subjected to horizontal loading which are the geometric features of the panel and the strength of wood. A parametric analysis is performed using a detailed model based on Hill-type plasticity to derive the constitutive law of the hinges. It is shown that in the studied X-braced T-F wall the masonry infills do not contribute significantly towards its lateral load resistance. Empirical expressions are proposed for the yield and maximum displacements and shears of a horizontally loaded T-F panel consisting of two X-braced diagonals which constitute the basic module of a structure. Thus, the model is readily applicable to NL static analysis for the assessment of the seismic capacity of T-F masonry buildings by meshing a structure to its T-F modules and applying the empirical expressions to each one of them. The model is verified against available experimental data, and is found to capture well the envelopes of the experimental loops.

Keywords Timber-framed masonry · Timber diagonals · Empirical model · NL static analysis · Lateral resistence

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Introduction

An initial experimental research on T-F masonry structural system carried out in Portugal in 1997 with an experimental campaign involving three specimens extracted from an existing building in the historic centre of Lisbon [1]. Walls were subjected to horizontal reversed cyclic loading at the top beam, and developed considerable ductility and energy dissipation capacity. Recently, another series of T-F walls were tested, also in Portugal [2]. This experimental research involved three large-scale specimens constructed in the laboratory Joints were constructed as close as possible to those found in old buildings. Meireles et al. [2] have observed early detachment and low influence of masonry infill in the overall response of the T-F walls. Another experimental investigation, also conducted in Portugal, involved seven T-F panels (1 m^2) with diagonal braces [3]. Materials and construction techniques were as in the previous tests. That study reconfirmed the key role of the diagonals and the early detachment of the masonry infill from the surrounding frame.

Three full-scale walls each including 16 X-braced T-F panels [4] were tested with a cyclic horizontal force and a constant vertical load. Joints were constructed using the mortise (groove) and tenon scheme, supplemented with mild steel nails, commonly used in India, which is highly dependent on the axial load of the columns. The conclusions drawn are: (a) highly NL response of the walls with separation of the connections under tensile stress, (b) minor contribution of the masonry infill to lateral stiffness and strength but rather important contribution to energy dissipation, and (c) rocking response due to the mortise and tenon joints.

Simplified models for T-F structures with progressive removal of the failed elements from the model were proposed by Cardoso et al. [5] in an approximate procedure. Masonry infills were ignored in the simulation and diagonal struts were assumed pinned at the connections and carrying compression only. A similar approach is suggested by Vintzileou et al. [6] focusing on possible variations of the damaged structure and the collapse mechanism. A distinction is made regarding the connections of timber elements; rigid connections are assumed between timber posts and beams, while the diagonals are taken as pinned to the surrounding timber frame. Similarly, Ferreira et al. [3] assumed carpentry joints to be rigid and diagonals to be pinned at the connections in a model comprised of beam, strut and plane elements. They found exclude the masonry infills from the model. A high modification factor (over 35) was proposed for reducing the axial stiffness of diagonals. A NL macro-model was proposed by Ahmad et al. [4] for the previously described type of T-F that is found in parts of India. They assigned NL hinges only to timber posts, while beams and diagonals were assumed to behave elastically.

Analysis Using the Micro-Model Approach

Seismic Behaviour of T-F Masonry Panels

Referring to the partially plastered T-F masonry infilled wall of Fig. 11.1a, loaded with a horizontal force V at its top beam, its response is characterised by an initial brief elastic phase, during which some cracks appear especially in the region of the diagonal braces; these are visible if stucco or plaster do not conceal them. The origin of this cracking is the initiation of relative sliding between masonry infill and diagonal braces. It is clear that apart from the first elastic phase of the response, which is brief compared to the full range of response and negligible in terms of energy dissipation (hence of seismic resistance), the governing element of the X-braced panel in all other phases is the compression diagonal, whereas masonry infill play (prior to their failure) a positive, yet secondary, role. It has to be noted here that in other configurations of T-F masonry, especially those that do not include proper diagonal elements, the role of masonry infill is more significant.

Description of the Micro-Model

The parametric analysis is performed using a detailed plasticity-based finite element model previously developed by the authors [7]. In this model NL behaviour of timber elements is described by a Hill plasticity model [7, 8]. Isotropic hardening occurs for natural species of wood and is considered here to occur at a stress corresponding to 40 % of its strength. The response of timber in uniaxial stress is assumed trilinear; the second branch has modulus of elasticity equal to 10 % the initial one, while the third branch is horizontal (fully plastic behaviour).

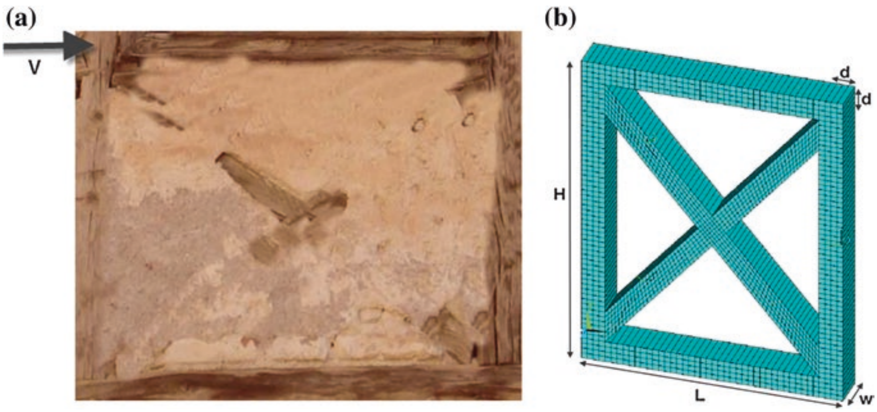

Fig. 11.1 **a** T-F panel with two diagonal braces subjected to a horizontal force and **b** geometry

The response of a T-F masonry panel (especially of the type found in traditional buildings) is highly affected by the inadequate detailing of the connection between the diagonal braces and the surrounding timber frame; this results to un-nailing of the iron nails, quite visible in the tests. Therefore, proper modelling of the joints should capture their opening and sliding, as well as the transmission of compressive and (interface) shear stresses. A simple contact is considered based on a friction-only constitutive law for the shear stress at the contact areas $\tau = \mu_f \cdot \sigma,\ \tau \leq \tau_{\max}$ where μ_f is the friction coefficient for isotropic friction, and σ is the normal stress at the friction area.

The connection of the timber beams and timber posts and the connection of the diagonals are materialised through T-shaped lap carpentry joint and two iron nails.

Parametric NL Analysis of T-F Masonry Panels

Several finite element analyses were carried out to study the main parameters affecting the response of T-F masonry walls subjected to monotonic horizontal load. Then, best fit equations were established for the main input parameters required for the macro-model. The *independent (input) parameters*, identified from a series of preliminary analyses as those having the largest influence on the seismic response of a T-F wall, in terms of the height *H,* length *L,* axial load *P* on the posts, and the distributed (here considered as uniform) loading *p* on the span, are:

1. The area *A* of the T-F panel including masonry infill, i.e. the product of height *H* times length *L* of the panels.
2. The ratio *R* of the dimensions of the panel, i.e. the ratio *H/L*.
3. The section of timber elements, i.e. the width *w* (which coincides with the width of the panel) and the depth *d* of the section (which is perpendicular to length and in-plane of the wall).
4. The strength of timber, i.e. compressive timber strength (which is assumed equal to the tension strength) $f_{c,t}$.
5. The vertical loading, i.e. the axial load *P* on the posts and the linear distributed (here considered as uniform) load *p* on the span of the T-F walls.

It should be noted that the thickness *w* of timber elements is assumed to coincide with the thickness of the panel, while the compressive strength of timber $f_{c,t}$ is assumed equal to the tensile strength. The definition of the dimensions *H, L, w* and *d* is shown in Fig. 11.1b. The objective of this study is to investigate the characteristics of the response of the T-F walls and express them in terms of the following *input parameters*: the yield δ_y and maximum (failure) δ_u displacements, the yield V_y and maximum (failure) V_u base shear and elastic K_{el} and post-elastic K_{inel} shear stiffness with respect to the following *independent parameters*

Therefore, the problem that needs to be solved has a rank of correlation 6 × 6 between the *response parameters* Ω_i and the *input parameters* Λ_i. In other words, *response parameters* $\{\Omega_i\} = \{V_y, V_u, K_{el}, K_{inel}\ \delta_y, \delta_u\}$ and *input parameters* $\Lambda_i = \{A, H/L, w, d, f_{c,t}\}$ is a system $Y_{ij}\ \{\Omega_i\} = f(\Lambda_i)$ where Y_{ij} is a 6 × 6 matrix. The key assumption is that each response parameter Ω_i is independent of the others which means Y_{ij} is a diagonal matrix ($Y_{ii} = \Omega_i$ and $Y_{ij} = 0$) and consequently, $\Omega_i = f(A, H/L, w, d, f_{c,t})$. So, evaluation of each of the six input parameters is carried out independently from the others, which substantially simplifies the analysis. The investigation domain over each input parameter Λ_i is determined as a range of values typically found in common buildings. Then, the influence of this parameter on the six response quantities is quantified. Practically, only four of the previous quantities are generally needed to express the seismic response of a T-F panel. However, as it will be shown, some of them are also affected by other parameters not included in the above mentioned input parameters and so, they are not selected as representative quantities.

Reference values of the input parameters for the T-F walls under examination were as follows:

1. Area A equal to 1.44 m^2 (1.2 x 1.2 panel)
2. Ratio R of the external dimensions H/L equal to 1.
3. Section of timber elements equal to 10×10 cm^2.
4. Strength of timber $f_{c,t}$ equal to 18.9 MPa.
5. Vertical loading, i.e. axial load P on the posts equal to 4.26 kN and uniform loading p on the T-F walls equal to 5 kN/m.

In order to estimate yield displacement δ_y, yield shear V_y and elastic K_{el} and post-elastic K_{inel} lateral stiffness, an appropriate definition of the yield point is necessary. Indeed, the calculated pushover curves for the T-F panels have been approximated by bilinear curves. This process is based on an energy balance, i.e. equating the areas above and below the bilinear curve with respect to the original pushover curve [9] retaining the first and the last point of the actual curve.

Parametric Analysis for the Panel Area

Parametric analysis for the effect of the area A was carried out selecting eleven T-F walls with A varying from 0.36 up to 6.76 m^2 (reference value 1.44); dimensions of the panel vary from $L = H = 0.6$ to 2.6 m corresponding approximately

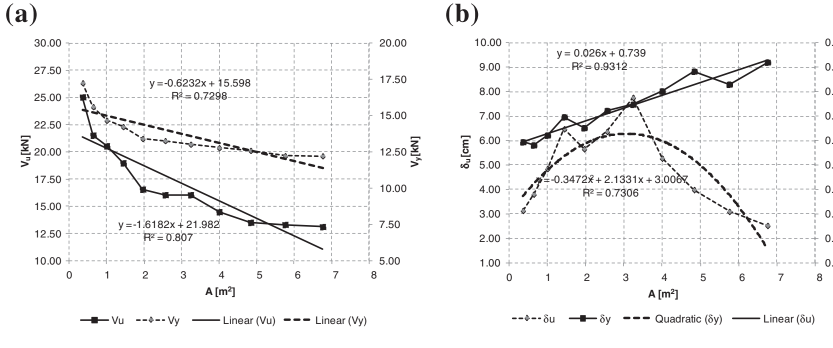

Fig. 11.2 Variation with A of: **a** lateral strength and **b** displacements at yielding and failure

to one quarter of a storey height up to a full storey height. Figure 11.2 displays the variation of yield and maximum shear, elastic and post-elastic lateral stiffness, and yield and maximum displacement, with the area *A*. The horizontal shear at yield and failure increases with *A* in an almost linear fashion. The same also holds for the elastic lateral stiffness and the yield displacement which decreases and increases, respectively, almost linearly with high correlation coefficient (higher than 80 %). On the contrary, post-elastic stiffness changes with the area in a bilinear mode; the boundary between the two regions is the intermediate area wall. The first region is characterised by a steep decrease of K_{inel} with increasing *A*, whereas the second region has a smooth decrease which can also be assumed constant. Rather than reflecting an intrinsic characteristic of the panel, this behaviour is primarily controlled by the way the bilinear approximation is made, in particular that no negative slopes are allowed. Regarding the variation of the ultimate displacement (Fig. 11.2b), this is a parabolic function with its maximum close to the intermediate area wall. It is concluded that a harmonious combination of wood and masonry contributes to maximizing its displacement ductility (i.e. the ratio of failure to yield displacement), while a heavily or poorly reinforced (with timber elements) wall lead to lower ductility. An analogy can be recognised here with the influence of the reinforcement ratio on the ductility of reinforced concrete sections.

Parametric Analysis for the Aspect Ratio

A similar parametric analysis is carried out for T-F walls with aspect ratio *H/L* varying from 0.5 to 2.0 (reference value 1.0). Eleven walls are analysed which have the typical properties 1, 3, 4 and 5 of Sect. 4.1 and the resulting variation of the response variables is identified by means of a least squares regression analysis of the parametric analysis results. This regression gives fitting functions with high correlation factor, over 90 % [8]. The variation of yield V_y and maximum V_u shears, as well as the variation of lateral stiffness K_{el} and K_{inel} with *H/L*, is exponential. On the contrary, the variation of δ_y and δ_u displacements with *H/L* is linear.

Parametric Analysis for the Panel Thickness

The parametric analysis for values of the thickness *w* of the panel within a range from 10 to 20 cm (reference value 10 cm) reveals a linear trend in the variation of shears V_y and V_u and lateral stiffness K_{el} and K_{inel}, while displacements δ_y and δ_u remain practically constant, with mean value of 1.03–8.91 cm, and standard deviations 0.03–0.60 cm, respectively [8]. This can be justified on the basis of the nature of the influence of *w* on the strength and stiffness of the panel, i.e. the fact that in-plane displacements are not substantially affected by out-of-plane geometric properties.

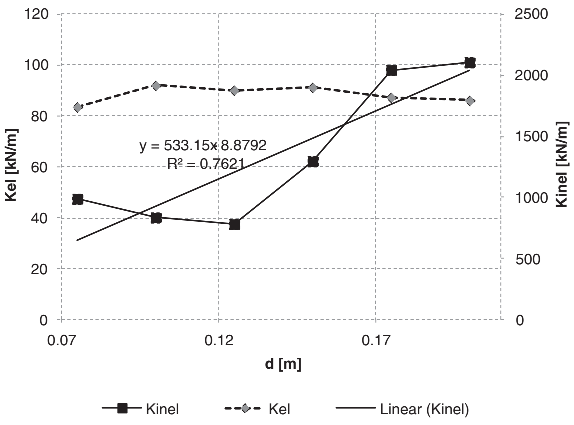

Fig. 11.3 Variation of elastic and post-elastic lateral stiffness with depth *d*

Parametric Analysis for the Timber Section Depth

The reference panel in this investigation has dimensions 1.8×2.0 m^2 because of the increased thickness $w = 15$ cm and depth d. The range of d varies from 7.5 to 20 cm which are typical timber sections found in existing T-F masonry structures. Regarding the effect of depth d on the response parameters a similar trend is observed as with w, i.e. linear increase with increasing values of d, however, the elastic lateral stiffness K_{el} remains almost constant with d (Fig. 11.3). This unexpected result stems from the way the bilinear approximation is made, i.e. in applying the equal area concept, the 'yield' point of the bilinear curve does not in principle represent the real onset of the NL response but it is always located beyond this point. The smoother the pushover curve the larger is the distance between the yield point and the real onset of yielding. Although elastic stiffness increases with increasing d values the curvature of the pushover curve near the yielding area also increases and this results to practically constant values of K_{el}.

Parametric Analysis for the Timber Strength

Ten T-F panels with varying timber strength $f_{c,t}$ from 16 to 26 MPa were examined in the parametric analysis for the effect of strength. The remaining input parameters were kept constant, the dimension of the panels being 1.3×1.1 m^2. The shears V_y and V_u are linearly correlated to $f_{c,t}$ with correlation coefficient exceeding 90 %. Also linear is the variation of lateral stiffness K_{el} and K_{inel}. Timber strength does not affect displacements δ_y and δ_u, which remain almost constant.

Parametric Analysis for the Vertical Loading

Vertical loads N acting on a T-F panel originate from their self-weight of the structure and from permanent and live loading in the part of the structure above them. Self-weight is directly modelled but loads from the upper structure should be added to posts (axial forces of the upper storeys, P) and to the beams (vertical distributed loading from the floors, p). These two patterns constitute the external gravity load (N_w). Further contribution (N_s) to the vertical loading of the T-F panels is made by the overturning moment due to horizontal (seismic) loading of the structure. However, due to the reversed cyclic nature of the seismic action effects, on average the mean value of the total external axial load (N) is approximately equal to the one corresponding to gravity loads (N_w). Therefore, in the investigation of the vertical loading N only equal loads are applied to both posts of a T-F panel. The total vertical load varies from $N = 10$ to 90 kN, retaining a 4:1 proportion between the concentrated and the distributed vertical loading.

The variation of shears at yield V_y and failure V_u with vertical load N are presented in Fig. 11.4. Both V_y and V_u slightly decrease with increasing vertical load. This negative slope of the V–N curves can be attributed to second order effects; however, the reduction of shears with the vertical load is rather negligible. Mean

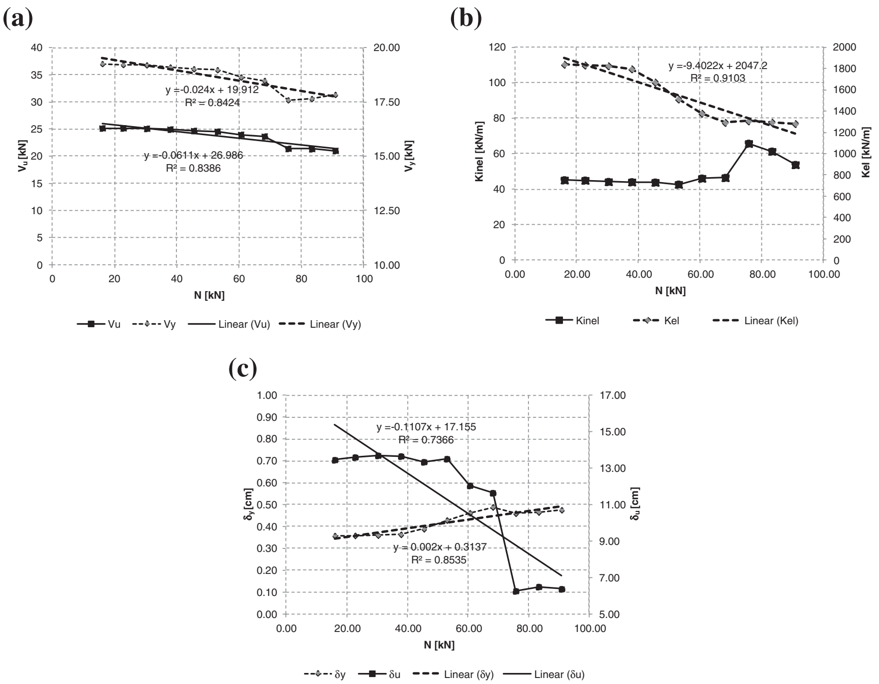

Fig. 11.4 Variation with N of: **a** shear strength, **b** lateral stiffness, and **c** displacements at yielding and failure

values for the entire range of considered vertical load are $V_u = 12.13$ kN and $V_y = 9.53$ kN whereas the corresponding COVs are 7.0–3.5 %, respectively, i.e. very small. Consequently, in a practice-oriented model the influence of the vertical load on these parameters can be neglected.

Elastic lateral stiffness seems to be substantially influenced by the vertical load N (Fig. 11.4b); the linear correlation function has a negative slope $\tan(-83.93^{\circ}) = -9.40$. However, closer observation of the actual pushover curves shows that their initial stiffness does not change at the same rate but it is either constant or very slightly influenced by N. Again the bilinearization procedure affects the 'elastic' stiffness, while second order phenomena do not play such a substantial role in the elastic lateral stiffness. Post-yield stiffness remains practically constant with N (Fig. 11.4b).

Regarding the influence of the vertical load N on yield displacement δ_y it remains almost constant around an average $\delta_y = 1.23$ cm with a COV of 13 %. On the contrary, maximum displacement δ_u (Fig. 11.4c) is strongly influenced by the vertical load N. The mean value for maximum displacement δ_u is 7.89 cm with a COV of 28.5 % cm which is comparatively high. It appears that there are two regions with an almost constant maximum displacement δ_u; a region with low N (< 70 kN) and a region with high N ($\geq$ 70 kN) where a significant decrease in the maximum displace-ment δ_u has taken place; after the threshold of 70 kN the maximum displacement δ_u decreases substantially and then remains constant (Fig. 11.4c).

Proposed Empirical Model

Empirical Expression for Shear at Yield

The main parameter affecting the shear V_y, as it is evident (and not surprising) from the above analysis, is the compressive strength $f_{c,t}$ of wood that defines the strength of the diagonal strut. The basic formulation of the expression for shear V_y is as follows:

$$V_y = \lambda_{\mathrm{A},V_y}\lambda_{R,V_y}\lambda_{w,V_y}\lambda_{d,V_y}\lambda_{N,V_y}\cdot\varphi_{V_y}(f_{ct}) \tag{11.1}$$

In Eq. (11.1) λ_{A,V_y} is a correction coefficient that takes into account the area of the panel, λ_{R,V_y} is a correction coefficient that takes into account the aspect ratio H/L of the of the panel, λ_{w,V_y} is a correction coefficient that takes into account the thickness of the panel, λ_{d,V_y} is a correction coefficient that takes into account the depth of the cross section of the timber elements, λ_{N,V_y} is a correction coefficient that takes into account the vertical loading and $\varphi_{V_y}(f_{ct})$ is a function depending on the compressive strength $f_{c,t}$ of wood. It is pointed out that all correction coefficients are dimensionless. The function $\varphi_{V_y}(f_{ct})$ was found from linear regression to be:

$$\varphi_{V_y}(f_{ct}) = 1.02\cdot f_{ct} - 0.59 \tag{11.2}$$

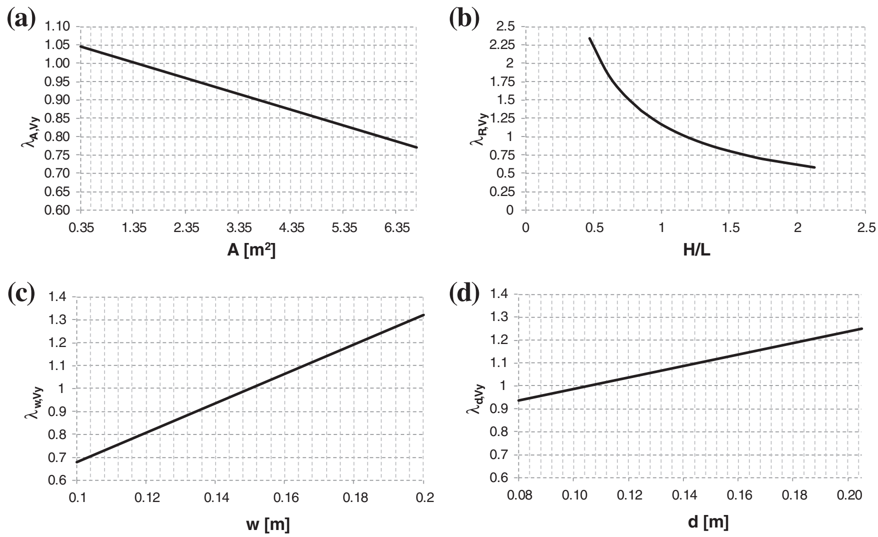

Fig. 11.5 Correction coefficients for shear at yielding V_y

where $f_{c,t}$ is the compressive strength of wood in MPa. The correction coefficient λ_{N,V_y} may be taken equal to 1 and the remaining four should be estimated according to Fig. 11.5.

Empirical Expression for Maximum Shear

The shear strength V_u is expressed by a similar empirical equation as for V_y (which is set as a lower limit to V_u, to avoid negative slopes that create numerical problems):

$$V_u = \max\begin{cases} \lambda_{\mathrm{A},V_u}\lambda_{R,V_u}\lambda_{w,V_u}\lambda_{d,V_u}\lambda_{N,V_u}\cdot\varphi_{V_u}(f_{ct}) \\ V_y \end{cases} \tag{11.3}$$

Again λ_{i,V_u} are correction coefficients and $\varphi_{V_u}(f_{ct})$ is a function depending on the compressive strength $f_{c,t}$ of wood. Note that shears V_y and V_u may be very close to each other for some extreme cases of the geometry when displacement ductility is rather low.

The function $\varphi_{V_u}(f_{ct})$ is given by the following expression:

$$\varphi_{V_u}(f_{ct}) = 1.31\cdot f_{ct} + 0.03 \tag{11.4}$$

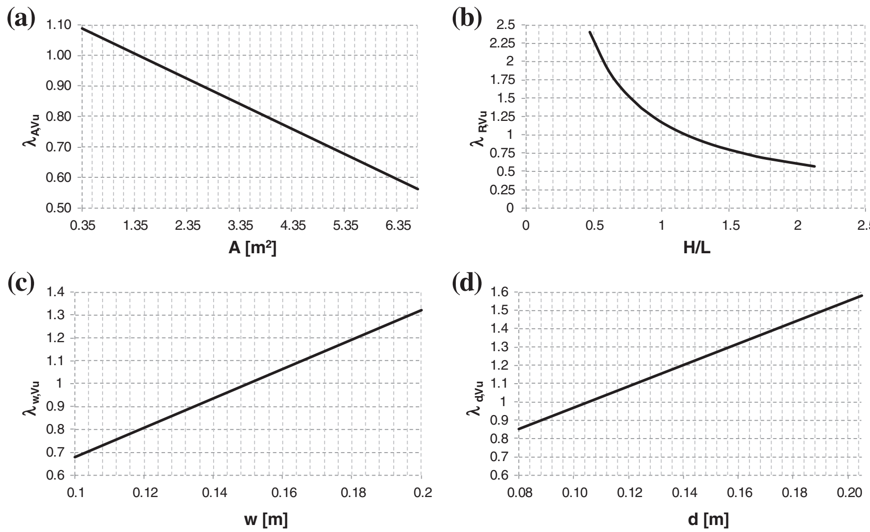

Fig. 11.6 Correction coefficients for maximum shear V_u

where f_{ct} is the compressive strength of wood in MPa. The dimensionless correction coefficients should be estimated from Fig. 11.6, while λ_{N,V_u} can be taken equal to 1.

Empirical Expression for Yield Displacement

Displacements are basically affected by the geometry of the panel; yield displacement δ_y is mainly affected by the area of the panel. The empirical formula to determine the yield displacement is given by:

$$\delta_y = \lambda_{R,\delta_y} \lambda_{d,\delta_y} \cdot \varphi_{\delta_y}(A) \tag{11.5}$$

In Eq. (11.5) λ_{R,δ_y} is a correction coefficient that takes into account the aspect ratio of the panel, λ_{d,δ_y} is a correction coefficient for the depth of the cross section of the timber elements, and $\varphi_{\delta y}(A)$ is a function depending on the area of the T-F panel. This function $\varphi_{\delta y}(A)$ is given by the following expression:

$$\varphi_{\delta_y}(A) = 0.03 \cdot A + 0.74 \tag{11.6}$$

where A is the area of the panel in m^2. The dimensionless correction coefficient λ_{R,δ_y} should be estimated from Fig. 11.7 and λ_{d,δ_y} may be taken equal to 1 for usual values of d.

Empirical Expression for Maximum Displacement

In Sect. "Proposed Empirical Model" it was found that maximum displacement is not only affected by the geometry of the panel but also from the vertical loading. The main parameter should be the aspect ratio of the panel. The empirical relation to determine the maximum displacement is given by:

$$\delta_u = \max\begin{cases} \lambda_{A,\delta_u}\lambda_{d,\delta_u}\lambda_{N,\delta_u} \cdot \varphi_{\delta_u}(H/L) \\ 1.5 \cdot \delta_y \end{cases} \tag{11.7}$$

In Eq. (11.7) $\lambda_{A,\delta u}$ is a correction coefficient that takes into account the area of the panel and should be estimated from Fig. 11.7, λ_{d,δ_y} is a correction coefficient for the depth of the cross section of the timber elements and may be taken equal to 1, λ_{N,δ_u} is a correction coefficient for the vertical loading, and $\varphi_{\delta_u}(A)$ is a function depending on the area of the T-F panel. This function $\varphi_{\delta_u}(A)$ is given by the following expression:

$$\varphi_{\delta_u}(H/L) = 10.56 \cdot {}^{H}\!/_{L} - 4.32 \tag{11.8}$$

the λ_{N,δ_u} correction coefficient may take the following two values:

$$\lambda_{N,\delta_u} = \begin{cases} 1, & \text{for } p < 65\,\text{kN/m} \\ 0.5, & \text{for } p \geq 65\,\text{kN/m} \end{cases} \tag{11.9}$$

It is recalled that the total vertical loading (N) consists of two parts, the axial loading on timber posts (P) and the uniformly distributed loading (p) on timber beams. In Eq. (11.9) an equivalent uniformly distributed load (p_{eq}) on the span of length L is used, as follows:

$$p_{eq} = p + \frac{2P}{L} \tag{11.10}$$

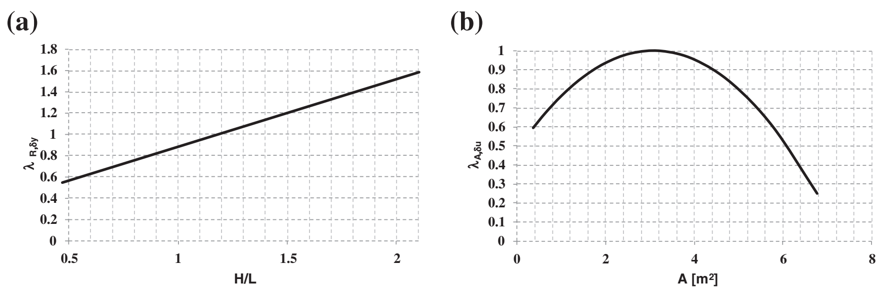

Fig. 11.7 Correction coefficient for yield displacement δ_y and maximum displacement δ_u

Implementation of the Procedure

The proposed macro-model is implemented according to a procedure which follows five steps. The method applies to a structural system which includes only timber elements pinned at their edges and masonry infills are taken into account indirectly. The five steps are as follows:

1. A discretization of the building into individual T-F panels should be performed. Each panel is defined by its surrounding T-F, masonry infills and the diagonals (Fig. 11.1).
2. The equivalent vertical load (Eq. 11.10) should be calculated for each T-F panel. Hence, a static analysis for the definition of vertical loads is necessary.
3. The empirical formulas (Eqs. 11.1–11.9) should be applied to define the constitutive load of each panel in terms of horizontal shear versus displacement. These 3 steps are preliminary and no change of the elastic model is so far implemented.
4. The elastic stiffness of the diagonals (see Fig. 11.1b for notation) should be corrected using the following expression [7]:

$$k_s = \frac{\left(H^2 + L^2\right)^{3/2} + H^3}{EA} \cdot \frac{1}{L^2} \cdot \frac{V_y}{\delta_y} \tag{11.11}$$

Diagonal elastic stiffnesses should be altered using a stiffness coefficient if possible, or by changing the dimensions or better by adding an extra spring.

5. The NL law of the plastic hinges in the diagonal struts should be defined in terms of axial load vs. deformation using the yield and ultimate shear and displacement found in step 3. The following expressions should be applied [7]:

$$\begin{gathered} N_{diag} = V\frac{\sqrt{H^2 + L^2}}{L}, \ \mu_d = \frac{\delta_u}{\delta_y} = \frac{u_{diag,u}}{u_{diag,y}} \\ u_u = 1.2 \cdot u_{diag,\max} \ \mathit{and} \ N_{res} = 0.2 \cdot N_{diag,\max} \end{gathered} \tag{11.12}$$

In Eq. (11.12) u_u is the maximum axial deformation, $u_{diag,\max}$ the axial deformation at maximum axial force, N_{res} is the residual axial force after the drop in strength and $N_{diag,\max}$ is the maximum axial load, i.e. strength degradation takes place prior to the development of the ultimate displacement; μ_d is the displacement ductility, as well as the ductility in terms of axial deformation.

Nonlinear static analysis of the structure consisting of the braced T-F panels defined in the previous steps.

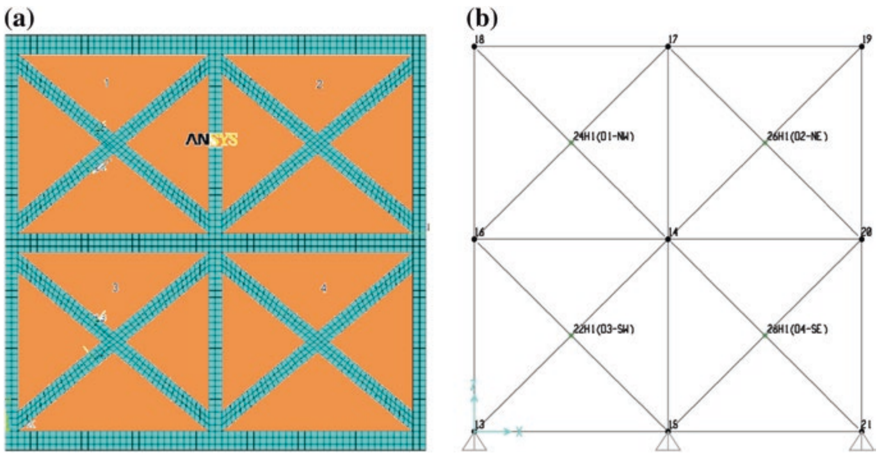

Fig. 11.8 **a** Finite element discretization and **b** macro-modelling of SC1 and SC2 specimens

Table 11.1 Characteristics of the panels in IST specimens [1] SC2 and SC3

H (m)	L (m)	H/L	A (m^2)	w^* (m)	d (m)	f_{ct} (Mpa)
1.3	1.32	0.98	1.72	0.10	0.1	25

Verification of the Model Against Experimental Results

This procedure is applied to the experimental specimens tested at IST Lisbon [2]. The discretization of the T-F specimens into four panels is illustrated in Fig. 11.8a. It is noted that the geometry is not exactly the same as that in the parametric analysis (Fig. 11.1b), since not all elements have the same thickness. For cases where there is small variation in *w* the following equation could be applied to estimate an equivalent thickness

$$w_{eq} = \left(A_{diag} w_{diag} + A_{col} w_{col} + A_b w_b + A_{inf} w_{inf}\right) / \left(A_{diag} + A_{col} + A_b + A_{inf}\right) \tag{11.13}$$

where the weighting coefficients A_i are the area of the ith member (diagonals, columns, beams and infills respectively). Basic data of the specimens necessary to implement the model are presented in Table 11.1. The macro-model of the T-F specimens is shown in Fig. 11.8b. Application of the empirical formulas (Eqs. 11.1–11.9) according to step 3 results in the constitutive load of each panel in terms of horizontal shear vs. displacement. Intermediate and final results of the formulas are presented in Table 11.2. The yield shear resulting from the NL static analysis of the wall is 38.5 kN while the maximum shear is equal to 44.4 kN. The elastic lateral stiffness is 3,256 kN/m and the maximum displacement is 9.5 cm which occurs when the wall carries the maximum shear.

Table 11.2 Values of the coefficients of the independent parameters of the empirical formulas and results for the four response parameters

Parameter	f_{ct}	A	R	W	D	N	Result
V_y	24.90	0.97	1.18	0.67	1	1	19.13 kN
V_u	32.90	0.94	1.19	0.67	1	1	24.60 kN
δ_y	0.78	1	0.88	1	1	1	0.69 cm
δ_u	6.08	0.90	1	1	1	1	5.46 cm

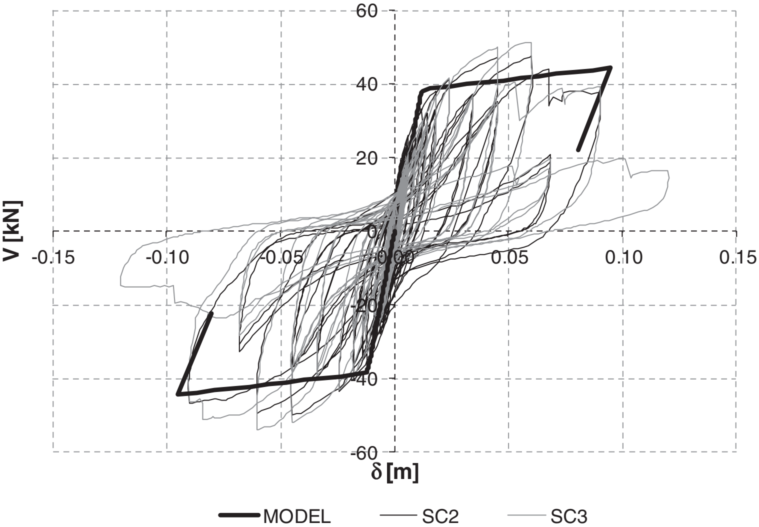

Fig. 11.9 Comparison of the pushover curves from the model against experimental results for SC2 and SC3 specimens

Comparing the model with the envelope of the experimental loops (Fig. 11.9) it is seen that there is a reasonably good match between theoretical and experimental results, deemed sufficient for practical purposes. More specifically, for the first experimental cycle the experimentally measured secant lateral stiffness at the yield point ($\delta_y = 1.2$ mm) of the analytical model is 3,798 kN/m for specimen SC2, while for specimen SC3 it is 3,836 kN/m, which is 16 % higher than the model stiffness, considered as a reasonable difference for a model aiming primarily at the NL response. The maximum shear for specimen SC2 is 48 kN and for SC3 is 51 kN, which are 9 and 16 % higher than the strengths predicted by the model. The shear developed during the last loop before substantial degradation takes place is about 40 kN for either specimen, which is 10 % less than the theoretical one.

Regarding displacements, the maximum value estimated by the analytical model is 9.5 cm, while the maximum experimental one is 9 cm for SC2 and SC3 for the last loop before the substantial degradation in the shear resistance. Specimen SC3 reached an ultimate displacement of 12 cm but that corresponded to a shear near one third of the maximum.

Conclusions

A simple NL model requiring knowledge of a few input parameters is presented in this study developed with the aim of being used in the practical analysis of traditional buildings with T-F panels. These input parameters depend only on the T-F panel geometry and the strength of timber. The selected parameters were related through regression of the results of an extensive parametric study using a refined finite element model to the main quantities required to define the constitutive relationship of the T-F walls in the 'practical' model. The relationship used in the model is the axial force verses deformation of the hinges in the diagonal struts, where damage and consequently inelastic behaviour is expected. The proposed empirical relationships apply only to X-braced T-F walls, a configuration which is most efficient for earthquake resistance. The reliability of the proposed empirical model was validated in respect of experimentally tested T-F wall specimens. A reasonably good match with the envelopes of the experimental loops was found.

The proposed macro-model permits estimation of the lateral load (seismic) capacity of traditional TF buildings not only with relatively limited computational effort, but also with limited knowledge of the properties of the structure, difficult to define in an existing building.

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Chapter 12
Earthquake Response of Historic Buildings at Lefkas Island

Triantafyllos Makarios and Milton Demosthenous

Abstract This paper deals with the earthquake response of the historic traditional buildings of Lefkas Island, Greece. These structures have up to four storeys and are located in the central area of the Lefkas town. The main characteristic of these buildings is their dual bearing system, which consist of a spatial timber multi-storey system and a single-storey masonry wall one, respectively. The first multi-storey wooden system possesses in elevation various diagonal trusses that are infilled by bricks with lime mortar, while at the top of the building there is spatially connected a tiled wooden truss roof. During the strong earthquake at Lefkas Island (August 14, 2003), the seismic behaviour of the above-mentioned buildings was very good, since only damages that can be repaired (without total collapse) have been appeared such partial collapse of the masonry wall sat ground floor and various cracks near to masonry openings. On the contrary, at the rest floors the timber multi-storey frames did not suffer damage, but many bricks cracked and fell out of planes. In this article, an analytical investigation of the earthquake response of such historic buildings is took place, in order to explain their seismic damage due the above-mentioned strong earthquake.

Keywords Lefkas earthquake · Traditional buildings · Structural damages

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Introduction

On August 14, 2003 at 05:15 GMT, a strong earthquake of magnitude M = 6.2 occurred in the Greek Ionian Sea, very close to the Lefkas Island. The strong motion of this earthquake was recorded by many accelerographs of the Institute of Engineering Seismology and Earthquake Engineering (ITSAK) network, where the peaks of the three seismic components were 0.42 and 0.34 g for the two horizontal directions and 0.19 g for the vertical one, while the strong motion duration was 18 s. The traditional building category (Fig. 12.1), which is examined in this article, represents the 34 % of the total building stock of the town and is existed in the historic centre and discussed in the past by others researchers [1–4]. The architectural morphology, the construction materials of these structures, the dual

Fig. 12.1 **a**,**b** Traditional building with dual bearing system. **c** The concept of the dual bearing system of the traditional buildings. **d** The wooden 3D frame of the upper storey

structural bearing system, as well as the observed damages from previous strong earthquakes were investigated by other researchers in the past. In the present work, interesting numerical results by an further extended parametric analysis, which have been applied in the past with reference these traditional buildings with dual structural system, are given.

It is worth noting that the traditional buildings of Lefkas town have up to four storeys and have a dual load carrying system. The first main load carrying system consists of a wooden multistory 3D-frame. The wooden 3D multistory frame system, on the ground floor, is enveloped by the stone masonry walls (Fig. 12.1) and it is connected on the top of the ground floor and on the perimeter of the building, with the second load carrying system that consists of a single-storey stone masonry walls. On the upper floors, the wooden 3D frame possesses diagonal wooden trusses. Single brick with lime mortar fill the wooden trusses. The majority of these buildings are resident houses and the rest of them are public buildings such schools, the old town hall, etc. Despite the age of these buildings (many of them have been existing for more than 200 years), almost all of them are in use nowadays. Poor soil conditions and the high level of underground water characterize the center of the Lefkas town. So, the traditional builders applied a specific type (Fig. 12.2d) of foundation of these buildings that it consists of a

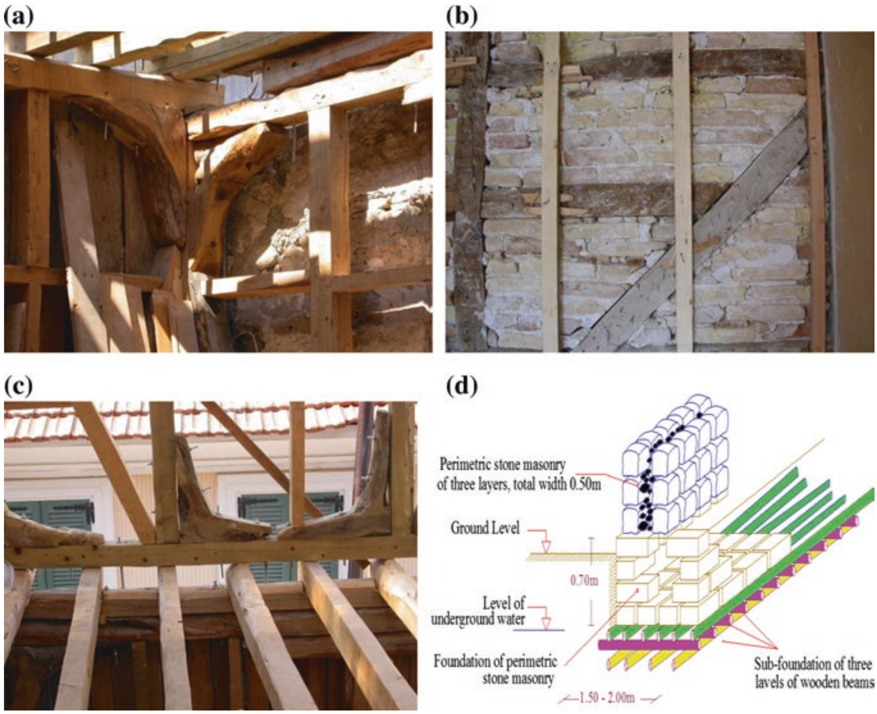

Fig. 12.2 **a**,**b** The wooden frame of the upper storey. **c** Detail of the wooden joint elements. **d** The foundation form of these buildings

sub-foundation of horizontally placed tree trunks in three levels [2, 3], while this sub-foundation resembles approximately as an ancient (primitive) seismic base isolation system (the first case of seismic isolation system in the world), without ditch in the perimeter of the building [2].

Earthquake Damage—Numerical Investigation—Results

The structural damages of the traditional buildings observed after the earthquake can be classified in the following categories: (a) Damages on the tiled roofs due to vertical seismic component; (b) Damages to the stone masonry walls of the ground floor where various cracking at the masonry stones or at the joint of mortars appeared. Moreover, partial collapse of the masonry walls observed (Fig. 12.3a); (c) Damages to the wooden frame with brick masonry infill on the upper storeys (Fig. 12.3b). In the frame of the present paper, a suitable analytical investigation of the seismic behaviour of such traditional buildings is performed in order to explain the observed damages and interpret the reasons of failures. The modeling of the masonry walls of the ground floor has been achieved using suitable shell elements. Different shell elements are used for the infill masonry walls of the upper floors, while for modeling of the wooden frames beam elements are used. Also, suitable failure criteria for masonry walls are adopted.

In analyzes, the recorded time histories of the acceleration along the two horizontal directions at the center of Lefkas town are used as base excitation [5]. Next, the numerical results of representative numerical models of two/three-story traditional buildings are presented and discussed. From the in situ investigation in Lefkas town the special characteristics (dimensions, used materials, type and strength of masonries, existed conditions etc.) of the traditional buildings are recorded [2]. For the needs of analysis, the numerical models have been analyzed

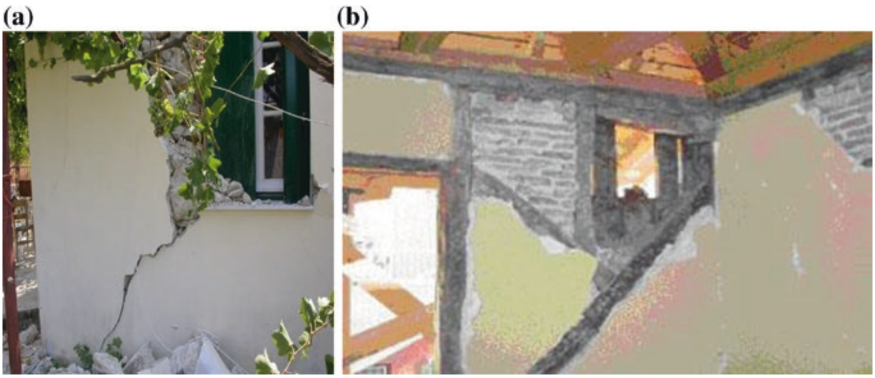

Fig. 12.3 **a** Local partial failure of masonry walls at the ground floor. **b** Out of plane collapse of masonry infill

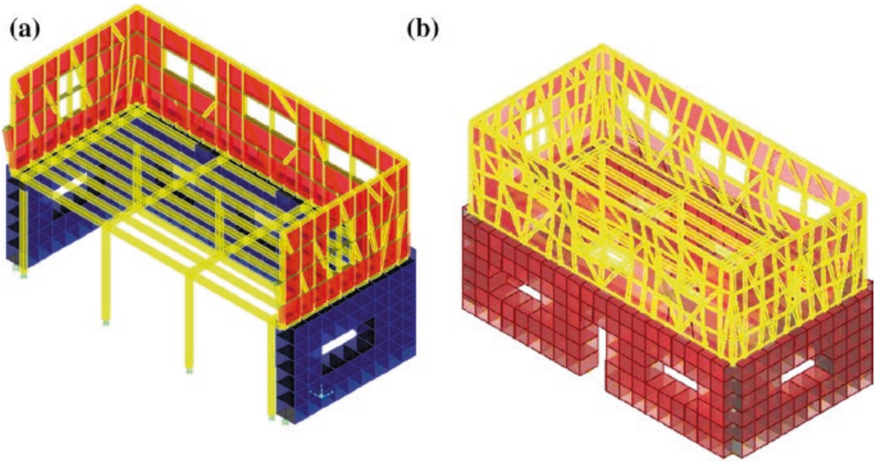

Fig. 12.4 **a** An open view of the 3D finite element numerical model. **b** Horizontal displacements due to Lefkas earthquake

using SAP2000,v15.1, (Fig. 12.4). As can be seen from Fig. 12.4b the maximum displacement response occurred at the mid-height of the upper storey due to the out of plane response.

The distribution of the out of plane response of the examined vertical cross section of the model at different time steps during base excitation is showed in Fig. 12.6. As shown from this figure and according to the conclusions from the modal analysis of the examined model, the out of plain response on the upper floor is governed by the local modes [2]. The time history of the displacement response of the two different positions is showed in Fig. 12.6.

Failure Criteria of Masonry

In Fig. 12.5, the envelope of the tensile stresses in two principal directions (s11 and s22) are presented. Tensile stress concentrations are observed in mid height of the infill walls of the upper floor agreeing with the positions of the observed damages from the in situ observations. At the same time wooden frames remain in the elastic range, presenting no structural failure, a fact also confirmed from in situ investigations. It has been mentioned that the observed structural damages on this type of buildings from the in situ investigation have been developed on the ground floor stone masonry walls and the brick masonry infill of the upper stories. With reference to masonry walls, the failure criterion of Von-Misses have been used, because is the most suitable for this case. The parameters needed for the definition of the surface shown in Fig. 12.7 the $f_c = 4{,}000$ kN/m^2 (compression strength) and the $f_t = 400$ kN/m^2 (tension ultimate strength), namely f_t assumed equal to 0.10 f_c. Cracking of masonry in both principal stress directions (s11 and s22) occurs when

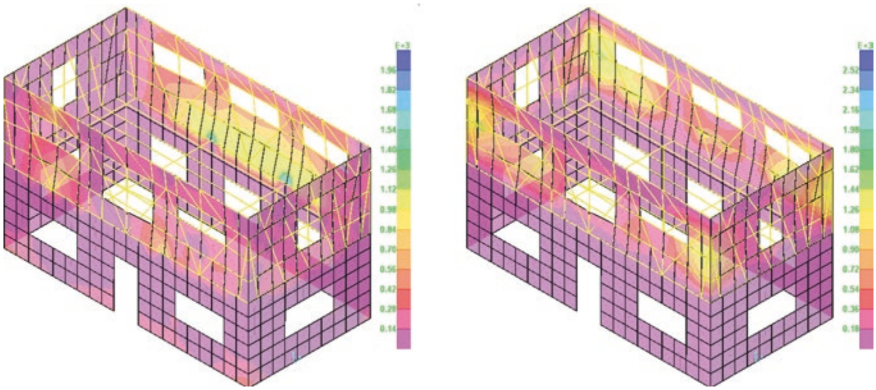

Fig. 12.5 Distribution of stresses s22 (*vertical*) and s11 (*horizontal*) due to Lefkas earthquake

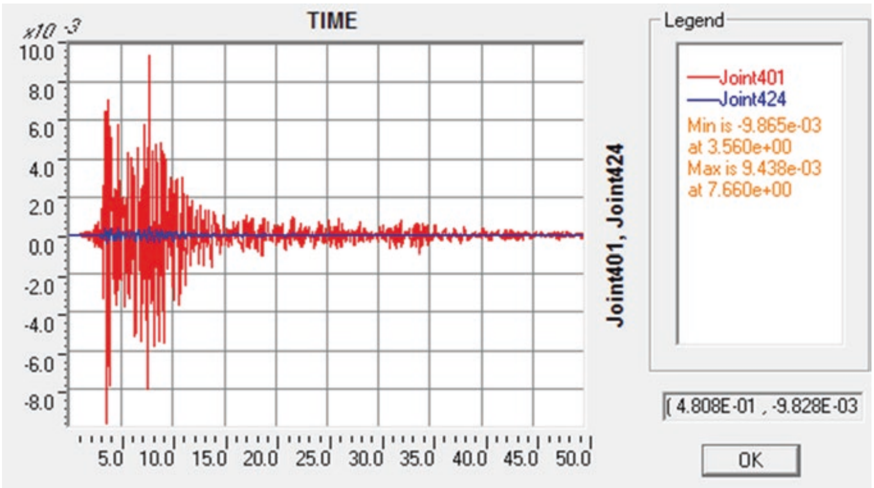

Fig. 12.6 Time history of out of plane displacement response of two different positions (at mid-height and in middle Joint401 and at the end_Joint424) of the upper storey

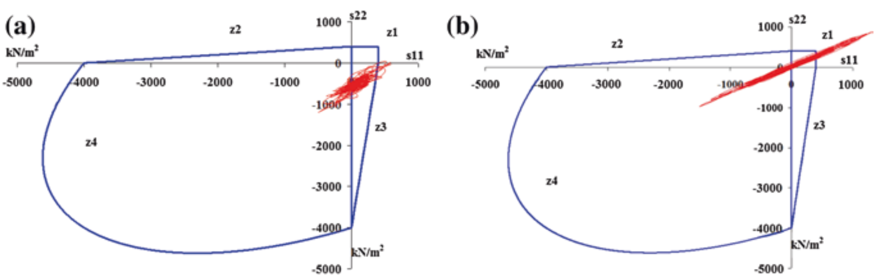

Fig. 12.7 **a** State of stress of the more stressed area at the masonry walls of the first storey. **b** State of stress of masonry infill at the corner of wooden elements at the upper storey

the state of stress is of the biaxial tension-tension type and both of the tensile principal stresses are beyond the tensile—failure envelope, which is designated as zone z1 in Fig. 12.7. In this situation the material loses its tensile strength completely. For a further investigation and in order to explain typical types of failure, the stress state of many positions of the model, in two principal directions, as it is developed during a time history analyzes is comparing with the adopted failure envelope. A relevant fully investigation about the failure can find in our previous work [2]. The final conclusion is that; cracking at the masonry walls of the first floor and crushing at the masonry infill of the upper story can be developed in case of masonry with small values of compressive strength which is in a good agreement with the in situ observations. On the other hand, the shear failure at the interface of masonry infill and wooden elements of the upper storey can be developed even for masonry with higher tensile strength as that adopted in this paper. It is clear that after these failures, the resistance of the masonries to the out of plane response is reduced and the occurrence of the out of plane collapse is more possible to be happened, as observed from the in situ investigation.

Conclusions

a. The traditional buildings with a dual structural system behaved very well, despite the long strong duration of the Lefkas earthquake (August 14th, 2003) and the high peak ground accelerations values (0.42 g).
b. The structural damages of the traditional buildings have been observed after this earthquake can be classified in two main categories, related to the structural system: Cracking and partial collapse of the ground floor stone masonry walls (in few cases) and shear failures, crushing and out of plane collapse of the upper stories brick masonry infill (in very many cases). The wooden frame, both in the ground and upper floor has not been appeared damages. In addition and instead of serious damages have been observed in modern structures and especially to infrastructures by foundation settlings, no one case of such damages have been observed in the traditional buildings, despite the poor soil conditions at the old town district of Lefkas, where the majority of such type of buildings is existed. This due to use of the extended wooden footings of three levels.
c. It is very common the fact, which the observed damages to the brick masonry infill of the upper floors are systematically concentrated around the middle of the height of the floors and usually followed by out of plane collapsed.
d. According to the numerical results of the numerical investigation, the maximum displacement and state of tensile stress have been concentrated around the middle of the height of the upper storey and it is a good verification and explanation of the in situ observations.
e. Comparing the numerical results of the state of stress with an adopted simple failure criterion that is based on stresses in two principal directions, the

following important findings have been underlined: Cracking at the stone masonry walls of the ground floor and crushing at the brick masonry infill of the upper stories can be developed in case of masonry with small values of compressive strength and it is a good verification and explanation of the in situ observations

f. The single story masonry by stones and mortars 3D-system (the first bearing system) has uncoupling fundamental period about 0.015–0.02 s for fixed foundation, while the wooden multistory 3D-system (the second bearing system) has uncoupling fundamental period about 0.20–0.23 s for fixed foundation, namely the difference is 1st order of magnitude. In addition, the two mode shapes of the first bearing system are characterized as global mode shapes, while the first two mode shapes of the second bearing system are characterized as local mode shapes (vibration of infilled walls out of the their plane). The coupling of the two bearing systems is a very smart idea and an important issue that has been applying by the technician and expert builders of Lefkas Island in the last 200 years, because gives to traditional building multiplex lines of capacity during earthquake events and the damages have been appeared and concentrated on the secondly elements, such the infilled walls.

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Chapter 13
A Diagnostic Plan Supporting Conservation Work on Timber-Frame Houses

Caterina Gattuso and Philomène Gattuso

Abstract In order to preserve and/or restore buildings of historic and architectural value, it is crucial to use methodological procedures and specific tools that allow the rational and targeted management of the large amount of information provided by sectorial and specialized research. This work proposes a diagnostic plan that is characterized by innovative elements and by the logic of the digital record and is aimed at describing the constituent materials and the structural elements of timber-framed houses by taking into account their environmental context, case history and pathologies. The methodological procedure is divided into three main steps—pre-diagnosis, diagnosis and post-diagnosis—and is applied to a representative timber-framed building chosen among the various examples still existing in Reggio Calabria. The investigation allows describing the building in detail, thus defining its state of conservation and leading to the creation of user-friendly digital repositories thanks to the implementation of the digital record. Certainly, this will have a positive impact on the definition of a successful project proposal. Therefore, it will be also possible to perform compatibility tests based on the comparison of various knowledge about the constituent and original materials, which were identified by means of studies of origin. This will further qualify the proposal of intervention even in the light of the latest cultural debates.

Keywords Methodological procedures · Timber-frame house · Reggio calabria

Introduction

The complex production and management of the extensive amount of information needed to get a comprehensive knowledge of particularly interesting buildings implies the use of tools and methods which allow identifying, reading, understanding and interpreting this information properly [1–4].

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In particular, information and data collected in a digital case sheet are made available for consultation and processing thanks to a Digital Case Sheet for the Building that is made out following the principles of the 2010 "Seville Charter", which implements the proposals of the 2006 "London Charter".

A digital case sheet is a useful tool supporting the management of methods aimed at controlling the process of knowledge acquisition. In the specific instance, a methodological procedure is defined to tackle the problems related to the overall analysis of the building, from its environmental and territorial context, to the anamnesis, the analysis of its constituent materials and of the construction and structural types, until the identification of the types of deterioration and their processes of formation [5–7]. This action is aimed at achieving strategies of intervention respecting the building and the most advanced cultural instances.

The methodological procedure presented in this paper proposes an organization of the phases that are deemed necessary to achieve the established objectives. In particular, the method includes three main phases, which highlight hierarchies and connections [8, 9].

Such a method has innovatory characteristics and enables to synchronize multidisciplinary contributions from experts involved in various ways and at different levels. This allows creating the technical assistance necessary to work synergically to find the most suitable action of conservation or restoration, which should be based on wide and detailed knowledge resulting from an in-depth analysis of the state of the art.

Particularly, the proposed method is explained through an eloquent demonstration study on one of the few existing timber-frame houses in the historic centre of Reggio Calabria.

The Methodological Procedure

The process of knowledge acquisition requires a rational methodological approach based on both analysis and synthesis as well as on the synergy between various complementary and interacting fields. In particular, according to the complex and specific nature of the diagnostic process, the process of knowledge acquisition aimed at its management can be developed considering the three following levels of interest:

- *specific*, when it is necessary to compartmentalize and study in detail specific situations in order to highlight particular aspects;
- *essential,* in emergency situations, when it is necessary to collect information under considerable time pressure, e.g. in case of natural disasters, such as earthquakes;
- *detailed,* when greater accuracy is demanded owing to the problems to face and to the importance of the examined monument, but also with a view to carrying out comparative studies and finding information that may be decisive for subsequent interventions.

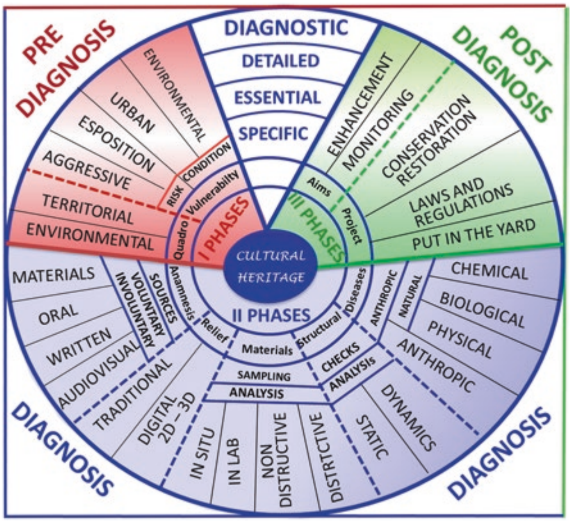


Fig. 13.1 Ideogram of the process of knowledge acquisition

The three types of diagnosis imply the use of logical and operational methods and instruments characterized by different levels of commitment, in terms of financial resources and activities.

Later on in this paper, the complete methodological procedure is proposed with the purpose of highlighting and representing the management of the complex knowledge typical of Cultural Heritage. It is graphically represented by an ideogram (Fig. 13.1) showing the organization of the phases of the process of knowledge acquisition.

The complete process takes a general dimension that includes investigation approaches which, through easy adjustments or simplifications, can become more specialized according to the detailed, essential or specific level to be achieved.

The methodological procedure is divided into three phases:

- *pre-diagnosis*, i.e. a preliminary phase of knowledge acquisition during which the environmental context is examined to identify causes of or tendencies to specific phenomena of deterioration that can be related to the environment;
- *diagnosis,* i.e. a second phase that can be divided into two sub-phases:

  a. anamnesis reconstruction, execution of the survey of the monument and of a specific photo/graphic and stratigraphic dossier with the purpose of collecting and connecting information to grasp the main problems;
  b. analysis of the constituent materials, of symptoms and of the state of conservation in order to understand the processes of deterioration;

- *post-diagnosis*, i.e. a third phase devoted to the search for possible solutions-interventions, which should be in line with the most advanced cultural and legal orientations promoting the respect of the monument and its enhancement.

By integrating and harmonizing the contribution from various experts involved in a restoration project, the proposed method becomes a guideline to reduce discretion though guaranteeing the creativity of the restorer [10, 11].

The process of knowledge acquisition, whose results are collected in the digital case sheet, is the first phase of the restoration and/or conservation project, which, in its turn, is organized in three macro-phases:

- *pre-intervention*, which includes all the actions necessary to define the knowledge base and coincides with the diagnostic plan;
- *intervention*, which is the phase when the choices to make are defined. It also includes the planning of periodical monitoring to check the effects produced by the restoration actions.
- *Post-intervention*, which includes the actions aimed at enhancing and *marketing* the cultural asset.

The Diagnostic Plan: From the Analysis of Materials to the Analysis of Mixed Structures

The analysis of the monument or of the building should consider not only the aspects concerning their constituent materials, but also the types of construction and the aspects related to the structural testing (e.g. their response to seismic actions).

The processing of the data resulting from the diagnostic process carried out on the materials allows consistently defining the behaviours of buildings and performing useful simulations for the structural analysis of structures and for the control of the planned actions [11, 12].

The relationship between materials and structures is more evident in the case of buildings with mixed structures characterized by the use of heterogeneous structural elements assembled to obtain a synergic response which may turn, for instance, into tensile or compressive strength. Structures made up of timber frames and stone, brick or raw earth masonry are a clear example [13–15].

They are characterized by brickwork made of weak, less cohesive mortars that easily break up and allow the surfaces in contact with the hard-wearing bricks to slide over the masonry substructures. This kind of infill assures flexibility and favours the effective dissipation of the energy produced by the seismic action. As a result, it induces damping and reduces the possibility of the structure to resonate with the earthquake [16–18].

The effective interaction between timber structure and infill allows obtaining an elastic behaviour which brings on sliding rather than resisting the earthquake, thus keeping a structural reserve capacity useful to delay the collapse.

Case Study

Though, in Calabria, the traditional timber-frame structures had already proved to resist earthquakes, they started to spread only after the disasters provoked by the 1783 earthquake.

The need to reconstruct cities assuring greater protection from the earthquake action led the Bourbon government to promote this type of buildings, which, today, are commonly known as “timber-frame houses”.

Unfortunately, over the last decades, the sprawling historic centres, the forgetfulness of the earthquake and the development of reinforced concrete, have brought about the loss of numerous timber-frame houses.

A study conducted at the State Archives in Reggio Calabria allowed identifying a timber-frame house located in via Filippini, in the city centre. It was taken as an emblematic case to demonstrate the importance of the diagnostic plan.

The Diagnostic Plan demands the acquisition of information through the direct observation and analysis of the building. Therefore, in order to reconstruct the knowledge base, it is necessary to integrate the study with data from on-site inspections.

Phase of pre-diagnosis

In this phase, the context and the vulnerability to potential risks were analyzed. It was found that the timber-frame house, which is located in the regular urban mesh (Fig. 13.2), on the corner of a block at the intersection of via Filippini and via Osanna and overlooking one of the busiest city roads, is however well-sheltered and not exposed to particular climate action (Fig. 13.3). According to the Italian classification, the house falls into zone 1, which is characterized by high seismic vulnerability.

Fig. 13.2 Location of the timber-frame house

Fig. 13.3 The timber-frame house in its urban context

Phase of diagnosis

In this phase, the timber-frame house was analyzed as a whole. In particular, an anamnestic analysis was carried out, a photographic dossier was produced, the main forms of deterioration were analyzed and a laboratory analysis was performed by using a sample of brickwork with mortar taken from an uncovered area of the façade.

Anamnesis

The timber-frame house belongs to the buildings erected after the 1908 earthquake. At present, it is unoccupied and completely abandoned. A study carried out at the State Archives in Reggio Calabria and at the Urban Cadastre allowed identifying its location on the sheet of the cadastral map n. 126, parcel 292. As a result, it was possible to trace its current and previous owners.

Photographic dossier and description of the house

Since a survey was not available, a photographic dossier was produced to provide useful information to create the knowledge base focusing on the construction elements that enabled to infer the type of structure.

Photos allowed detecting a restoration action carried out by using a hexagonal mesh, which had been covered with cement based mortar spread on the timber frame. The house is composed of two adjacent buildings with pitched roofs and its gabled façade shows two floors (Fig. 13.4): the ground-floor elevations comprise

Fig. 13.4 Main façade

Fig. 13.5 Side view

windows and doors, while, on the second floor, a French window opens on to a small balcony. The plastered exterior surfaces rest on a base which develops along the two sides overlooking the road (Fig. 13.5). Façades end in cornices carved out of the thickness of the plaster.

Materials and structure

The house is structurally made of a timber frame which shows up in a few deteriorated parts (Fig. 13.6). The masonry infills are made of solid bricks and mortar, while the frame is probably made of oak wood (Fig. 13.7). Timber is a strong but light building material; timber beams are more flexible than other materials and, as a result, they allow absorbing and dissipating energy.

The structure shows a rather complex behaviour, related above all to the different stiffness of the constituent materials, to the thickness of the infills and to the

Fig. 13.6 Construction detail

Fig. 13.7 Detail of the timber frame

way timber was assembled. In any case, an important factor, which is also decisive for the sustainability of this kind of wall structures, is the progressive deterioration of timber: the lack of maintenance leads to the gradual decrease in the buffer capacity. The box-like behaviour, caused by the timber frame, allows the various elements involved in the resistance to exchange the horizontal seismic pressures and to trigger a global response by collaborating and distributing pressure on the various parts, also thanks to adequate connections which play a crucial role in restraining the rotation of walls (Fig. 13.8).

As regards materials, it is necessary to carry out specific analyses to characterize them and obtain data useful to trace their origin. Data are essential to know the mechanical properties of the materials and, therefore, to construct models supporting appropriate simulations (Fig. 13.9).

Fig. 13.8 Sampling point

Fig. 13.9 Samples taken

The estimate of the resistance of the existing buildings depends on the knowledge of the quantities and characteristics of materials, considering that certain parameters influence the response of the structure more than others do. To that purpose, it is necessary to prepare a specific sampling plan in order to gather correct information and assure the success of the simulations to perform on the materials, on the structural elements and on the whole structure (Fig. 13.10).

The approach is illustrated in analytical terms by means of an application process. Sampling was carried out on the examined timber-frame house. In particular, samples of brickwork were taken, one of which included a layer of mortar.

The largest sample of brickwork was used to obtain a thin section, which was then analyzed by means of a petrographic microscope to observe its microstructures and texture. Before being sectioned, the sample was macroscopically observed with a Digital Microscope, which allowed finding calcite nodules (Fig. 13.11).

The sample of brickwork included an aggregate made up of a crystalline silico-clastic fraction. It was fine-grained, well sorted and immersed in an

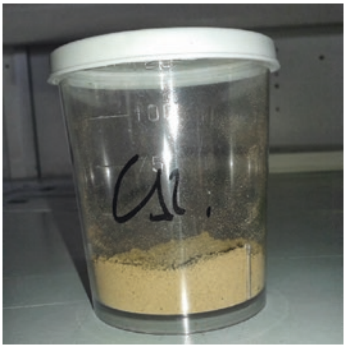


Fig. 13.10 Powdered sample

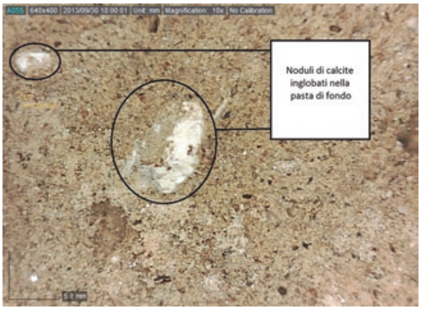


Fig. 13.11 Digital Microscope Photo

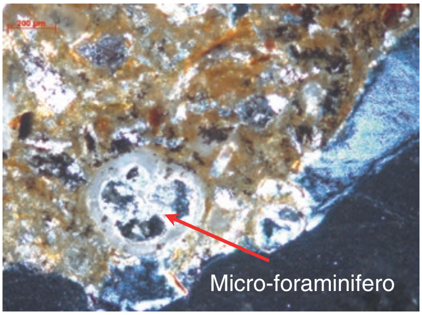


Fig. 13.12 Light Microscope Photo

oxidized matrix which contains plenty of fossil foraminifer species (Fig. 13.12). Microporosity was well distributed and shows incomplete dissolution and various fractures. It also showed a medium-high level of conservation.

The aggregate of mortar (Fig. 13.13) was composed of granite silico-clastic fragments and not well sorted feldspar, quartz and mica. The grain was medium-fine. The binder, which was made of a calcite micro-crystalline matrix, showed widespread micro-fractures.

Other samples were powdered and analysed to obtain the mineralogical composition of the brickwork. The X-ray diffraction analysis allowed detecting the predominant presence of quartz as well as of plagioclase and calcite (Fig. 13.14).

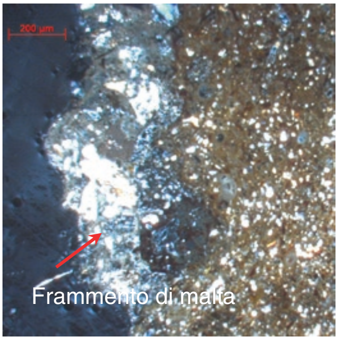


Fig. 13.13 Light Microscope Photo

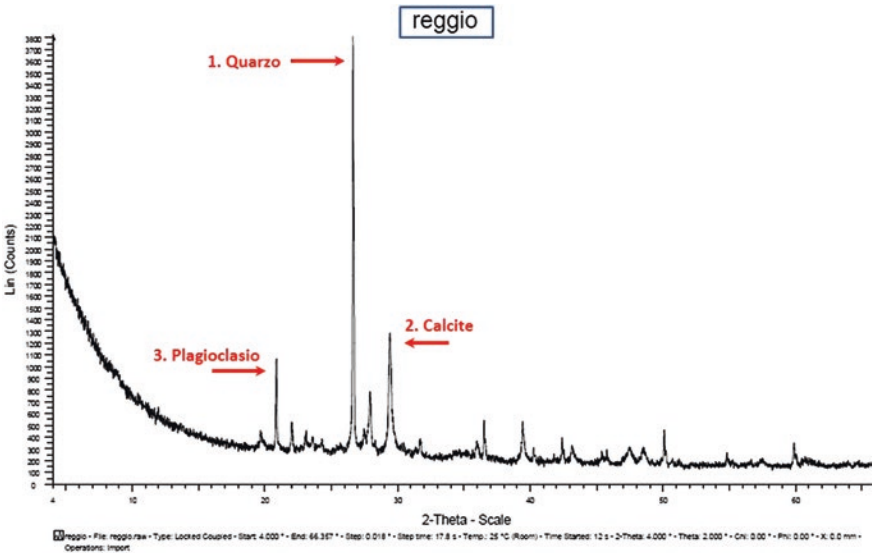
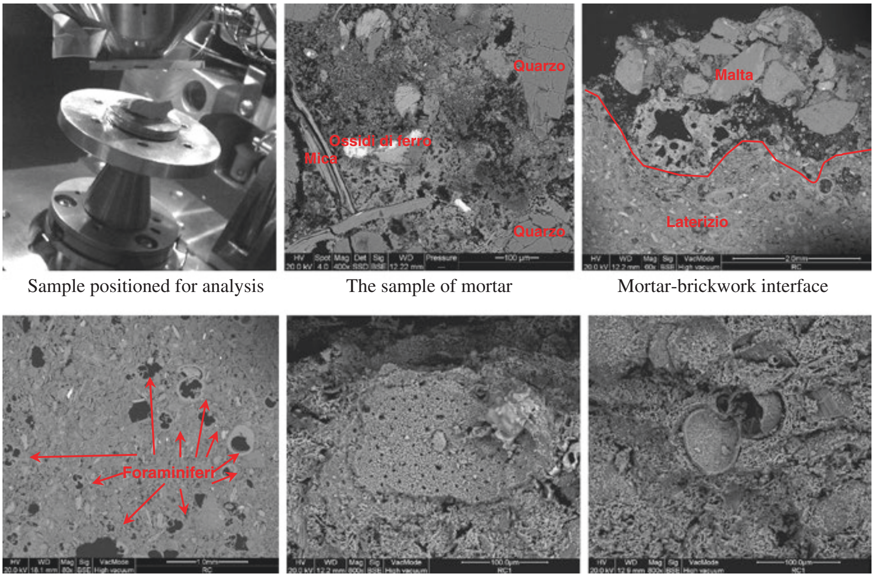


Fig. 13.14 Diffractometer response spectrum

Table 13.1 Anhydrous values of the main elements observed with the SEM. The chemical composition is expressed as weight percent of the main oxides

	Na_2O	MgO	Al_2O_3	SiO_2	Cl_2O	K_2O	CaO	TiO	Fe_2O_3
Campione	Percentuale (%)								
Laterizio int	3,40	3,16	15,89	58	0,54	2,20	12,6	0,8	3,41
Laterizio est	47,7	0	0,81	1,26	49,3	0	0,91	0	0

Finally, in order to obtain detailed information, an accurate chemical analysis on a fragment of the sample and on its thin section (Table 13.1) was carried out by using a SEM (Scanning Electron Microscope), which also allowed obtaining spectra where the main components, identified in two representative points of the



Sample positioned for analysis | The sample of mortar | Mortar-brickwork interface

The sample of brickwork | Brickwork–Mould of foraminifer | Brickwork–Shell of foraminifer

Fig. 13.15 SEM images of a fragment of brickwork with a superficial layer of mortar

sample (the one inside the sample and the other on its external surface, in an area with salt deposits), were highlighted.

This instrument allowed also getting morphological images of the surface of the sample, which showed the presence of shells and moulds of foraminifers (Fig. 13.15).

SEM observations confirmed the presence of fossils in the sample of brickwork, whose matrix has a marly composition (high in calcium) with round-edged pores.

The mortar is quite deteriorated (actually, it easily broke up already during the sampling, thus showing a powdery conformation). The chemical composition demonstrated that the structure was rich in salts, calcium sulfate and sodium chloride. Fragments of quartz, mica, feldspar and oxides were found in the aggregate. The spectra obtained during analyses are shown below (Fig. 13.16).

The state of conservation

The deterioration of the examined house is due above all to its neglect, as shown by graffiti on the walls, by the rubbish in the interstices of the brickwork and by the continuing lack of maintenance, especially of gutters and roofs. The deterioration of the plastered surfaces as well as the loss of considerable parts of plaster in corner posts and in the footing allow seeing the wooden frame, which is also partially deteriorated (Fig. 13.17).

This situation may threaten the structural effectiveness of the construction system. As a matter of fact, maintenance is crucial to assure the sustainability of

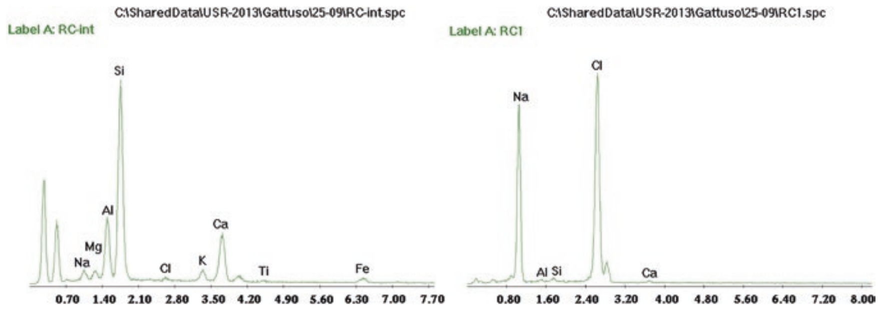


Fig. 13.16 SEM spectra—Accurate qualitative analyses inside the brickwork and on its surface

Fig. 13.17 Deterioration of the brickwork

this type of walls, whose timber frame would otherwise progressively deteriorate involving a gradual loss of dissipative capacity.

As shown by pictures, the situation is made even worse by a series of localized interventions, which were carried out superficially by covering portions of surfaces with cement-based plaster (Fig. 13.18).

The structure

From a structural point of view, the house does not appear to be in poor condition. A more detailed study of the structure may provide further information.

Additional investigation, including mechanical testing and the use of simulation models for static and dynamic analysis, would be needed for a specific supplementary analysis, yet it lies outside the purpose of this paper. However, this confirms the need for multidisciplinary integration in order to obtain sectoral knowledge in relation to the various specific competences.

Fig. 13.18 Localized interventions

Conclusions

This paper illustrates a complex methodological procedure applied to the analysis of one of the rare timber-frame houses still existing in the centre of Reggio Calabria, Italy.

The study confirms the need to manage the knowledge of the situation in order to identify key elements to use for the definition of more adequate proposals of intervention, taking into account their implementation and the following enhancement of assets.

The study could be completed by dealing also with aspects related to the origin of materials, in order to perform simulations, identify the most compatible materials, and construct structural models to test in the laboratory and with dedicated software.

The focus on a rare example of timber-frame house is noticeable also because it offers the opportunity to make a first step towards its enhancement during the phase of post-diagnosis and to conceive a first draft of digital case sheet which could be used for further studies.

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