



ADVANCED MASTERS IN STRUCTURAL ANALYSIS
OF MONUMENTS AND HISTORICAL CONSTRUCTIONS

Master's Thesis

Francesco Isalberti

**Modelling of timber floors in
strengthened conditions for
seismic improvement**



UNIVERSITY OF PADOVA, ITALY



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Erasmus Mundus

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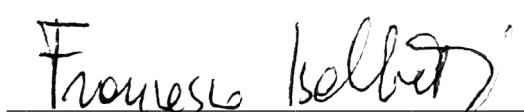
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Alla mia famiglia

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ABSTRACT

Thesis: Modelling of timber floors in strengthened conditions for seismic improvement

This research deals with the numerical modelling of timber floors strengthened against seismic actions. The topic of timber floor is very important considering the masonry building heritage that surrounds us for example here in Italy. Several losses occurred during the last seismic events and different problems related to the masonry building behaviour were highlighted by them. Floors and in particular timber diaphragms that are common in masonry buildings are covering a key role in possible ways to counteract the horizontal forces of an earthquake. Considering the consequences of strong retrofitting techniques suggested by national codes, the current work is aiming to provide a study of compatible retrofitting techniques without strongly compromising the timber floor.

The work starts from an already done thesis performed focusing on several laboratory tests carried out in the laboratories of the Department of Civil, Architectural and Environmental Engineering (DICEA) at the University of Padova. The numerical modelling herein presented has been developed by using the software DIANA improving the already done models with another software.

The first part of the work focused on the calibration of the numerical curves of specimens made of a beam and boards nailed and screwed on it. The curves have been evaluated checking the correspondence of the numerical results with the laboratory ones. These first evaluations were done just to calibrate the modelling of the timber floors and evaluating where the difficulties could be connected with.

As for the second part of the thesis, starting from the tests done at the DICEA, just the floor with nailed boards was modelled. The simple configuration with variable board-to-joist connections has been considered. In addition the global behaviour of the diaphragms was checked with the presence of friction law between the boards.

Finally the work moved to the analysis of a floor with more realistic dimensions than the ones properly defined for the laboratory tests. Herein two different load distributions were evaluated defining the overall behaviour of the timber floor and investigating the results.

RIASSUNTO

Tesi: Modellazione di solai lignei rinforzati per il miglioramento sismico

La tesi tratta della modellazione numerica di solai lignei rinforzati contro l'azione sismica. Il tema dei solai lignei è un argomento rilevante nel panorama degli edifici in muratura specialmente considerando come caso esemplificativo l'Italia. Gli ultimi eventi sismici hanno determinato numerose perdite ed evidenziato una serie di danni ed effetti negativi su edifici del patrimonio storico in muratura. Considerando inoltre le conseguenze di sistemi di intervento invasivi suggeriti dalle normative nazionali, il qui presente lavoro si prefigge lo studio di tecniche di rinforzo non invasive per il sistema solaio.

Il lavoro si sviluppa, avendo come punto di partenza una tesi già elaborata presso l'Università degli Studi di Padova. La tesi si basava sull'analisi numerica di test sperimentali eseguiti presso i Laboratori del Dipartimento di Ingegneria Civile, Edile ed Ambientale (DICEA) dell'Università di Padova. I modelli presentati nella corrente tesi sono stati sviluppati utilizzando il software DIANA.

La prima parte della tesi si concentra sulla calibrazione e controllo della risposta dei modelli numerici dei provini di laboratorio costituiti da un tronco di trave e da uno o più strati di tavolato chiodati e/o avvitati ad esso. Le curve numeriche di risposta dei connettori sono state verificate controllando la loro corrispondenza coi dati sperimentali. Queste iniziali analisi sono state effettuate per valutare eventuali difficoltà e come condizioni esemplificative dei rinforzi applicabili ai solai lignei.

La seconda parte della tesi si è concentrata sul tema dei solai. Partendo dai test di laboratorio precedentemente menzionati, è stato considerato solo il solaio base costituito da semplice tavolato chiodato alle travi. Sono stati valutati solai semplici con variazioni di configurazione della connessione tavolato-trave lignea. Il comportamento del solaio è stato inoltre controllato considerando l'applicazione di una legge di attrito tra le tavole simulando il giunto maschio-femmina presente lungo tutta la loro estensione.

Come conclusione della tesi sono stati modellati anche due solai aventi dimensioni reali e ne sono stati valutati i rispettivi comportamenti considerando due differenti configurazioni di carico.

RESUMEN

Tesis: Modelos de pisos de madera reforzados para la mejora frente a las acciones sísmicas

La tesis trata sobre el modelado numérico de pisos de madera reforzados contra las acciones sísmicas. El tema de los pisos de madera es muy importante cuando se trabaja con edificios construidos en mampostería, especialmente considerando como ejemplo Italia. Los últimos eventos sísmicos han presentado muchas perdidas y evidenciaron varios daños negativos con respecto a los edificios del patrimonio histórico en mampostería. Además considerando las respuestas del sistema de intervenciones invasivos sugeridos por los códigos nacionales, el estudio aquí presentado busca desarrollar las técnicas de refuerzo poco invasivas de los pisos.

El trabajo se desarrolla, contando con una tesis previamente desarrollada por un estudiante de la Universidad de Padova. La tesis se refiere al análisis de elementos finitos de tests hechos en los laboratorios del Departamento de Ingeniería de Caminos, Construcción y Ambiental (DICEA) de la Universidad de Padova. Los modelos aquí presentados están desarrollados con el software DIANA.

La primera parte de la tesis se focaliza en el control y la calibración de la respuesta de los modelos numéricos de los especímenes hechos con una viga central de madera y una o dos tablas de madera conectadas por clavos o tornillos. Las curvas de respuesta de los conectores fueron verificadas controlando sus correspondencias con las experimentales. Dicho análisis fue necesario para evaluar los errores y dificultades para establecer las condiciones iniciales para el estudio de los pisos de madera.

La segunda parte de la tesis está focalizada sobre el tema de los pisos. Empezando con los tests de laboratorio, fue considerado solo el piso con las tablas de madera conectadas por clavos de hierro a las vigas y sin ningún tipo de refuerzo. Se han evaluado muchos pisos con diversas configuraciones de clavos en la conexión entre tablas y vigas. El comportamiento de los pisos fue además evaluado considerando la aplicación de una fricción entre las tablas, simulando la conexión entre ellas.

Como conclusión de la tesis fueron modelados dos pisos con dimensiones comparables a las que se encuentran en edificios reales. Con respecto a estos, sus comportamientos fueron evaluados considerando dos configuraciones de cargas solicitantes.

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1. INTRODUCTION AND OBJECTIVES

Timber floors are the most frequent structural diaphragm that can be found in a masonry building. These systems were used for several reasons, from the availability of materials to the easy way to compose them in order to build a floor system. These applications found place in almost all countries and the ways in which the techniques developed during years are impressive.

Recently several seismic events affected many countries through the World and the most relevant damages involved both ancient and new masonry structures. Horizontal forces acting on masonry walls often generate local mechanisms that affect the global stability of the building. Several studies underlined that the presence of timber diaphragms not properly connected to the lateral walls contributes to the generation of these mechanisms.

The necessity of a global box behaviour of the all structure defined the need to find refurbishing solutions for the floors. The role of the in-plane stiffness of a timber diaphragm is of fundamental importance and for this reason. The general aim is to improve it. In addition to the in-plane stiffness, the floor-to-wall connection had a relevant role too in the general development of the techniques. A proper connection allows the transfer of shear forces from the floor to the lateral shear walls.

Several types of interventions have been introduced in past years but during the most recent earthquakes the already refurbished structures anyhow presented important damages. Among the possible interventions the ones involving a reinforced concrete tie beam all along the perimeter of the structure was deeply used. The effects of heavy masses (reinforced concrete elements) and lack of connection between the concrete and the wall caused various damages like out-of-plane rotations of the walls.

For these reasons low invasive and reversible techniques have to be defined and introduced.

The aim of this thesis is to model timber diaphragms having some changes in the number of board-to-joist connectors and introducing friction laws between boards. The aim is to reproduce the experimental behaviour of already tested specimens in order to calibrate FE models using a more complete software than the one previously used.

The thesis is developed already having experimental data from past experiences and models done by a Master student of the University of Padova. He drew the numerical models

with a software, reproducing the non-linearity of the specimens. The thesis herein presented has the aim to improve the models by using another software that allow the application of interfaces in order to simulate the non-linearity of the materials. In addition, a possible application of the strengthening technique would be evaluated on a numerical model reproducing a floor with realistic dimensions.

1.1. Organization of the thesis

The thesis is divided in eleven chapters including the introduction (Chapter 1).

The second chapter presents general considerations related to the technologies and the typical construction techniques used to build masonry structures and their timber floors. The third chapter is related to the typical problem of these kind of building under a seismic event. The role of the floor against the horizontal action is shown with some examples. As for the fourth chapter, it is focused on a general overview about the possible retrofitting techniques that can be found in literature with some suggestions for future developments. The analytical calculation of the stiffness of a timber floor is presented with some possible applications. It is shown the theoretical calculation from the simplest configuration just with nails to the more complex with fibres. The chapter six is based on the presentation of the samples evaluated in laboratory tests for the previous Master's thesis. Description of the procedures and the typologies of samples are here offered. Regarding the seventh chapter an overview about the possible models is defined. Examples with simplifications are showed starting from the correct literature. Eighth chapter is referred to the description of the models and the results of pushover analysis. Numerical models of sample A and B are evaluated and their response curves presented. Ninth chapter shows the several numerical models of the tested timber floors and the results. The following chapter is dedicated to the evaluation of a numerical model of a realistic floor considering two different load configurations.

Last chapter is dedicated to the explanation of the general results obtained and some observations related to the problems found during the modelling process. Concluding some further investigations and proposals are suggested for future developments.

2. STATE OF THE ART: GENERAL CONSIDERATIONS ON TIMBER FLOORS

Masonry constructions are parts of a common tradition in the field of building techniques in several countries like Italy. Considering Italy as an example of this tradition it is easy to see that various masonry typologies have characterized in different ways the evolution of the building tradition into the regions. The typology herein described and analyzed identify one of the most important component of the historical heritage in the country.

Different typologies of masonry structures can be identified:

- Stone masonry structures
- Clay (tiles or adobe)
- Adobe structures (rammed earth structures)
- Brick-mortar structures
- Concrete bricks

The use of one of the previous materials depends on the availability of it in the surroundings of the building site. The concrete brick typology has been mentioned just because it enters inside the construction characters but not inside historical cases. The Italian tradition is affected by the presence of the two first typologies in which the stone masonry one is defined by stones not previously worked. In this way a great importance is not assumed by the unit (stone) but by the layer of mortar. The quality of masonry has to be determined by considering not only the stress and strain of it but also a qualitative aspect. Several factors have to be taken into consideration during the studies of a masonry structures. Geometrical characters and physical properties are fundamental in the general definition of a masonry construction. Some geometrical rules and properties are widely studied and largely affect the global behaviour of a masonry building. For instance some of the most common factors that are involved in the correct behaviour of a structure are the following:

- Linearity of the rows
- No vertical continuity of the head joints
- Dimensions and shape of the unit
- Presence of transversal connections linking together the leaves of the wall (transversal tying=diatoni)

- Quality of the mortar
- Adequate strength of bricks and stones

Masonry structures work under the effect of the floors or roofs that distribute laterally the load providing the general stability of the structure as a whole. For the general stability of the structures the study of the horizontal elements is of fundamental importance. The overall distribution and disposition of the horizontal elements is one of the most relevant part in the study of the masonry structure. Traditionally masonry building were designed in order to resist vertical loads. The common historical construction is characterized by the presence of timber floors and roofs. Timber floors were generally very simple structures consisting on joists and cross boards linked to the main elements by using iron nails. Timber floors are used either one-way for limited spans or two-way floors for larger ones (Fig.1). Fig. 1 One-way and two-way timber floors The use of timber floors is justified by their limited weight and the compatibility with the masonry structure.

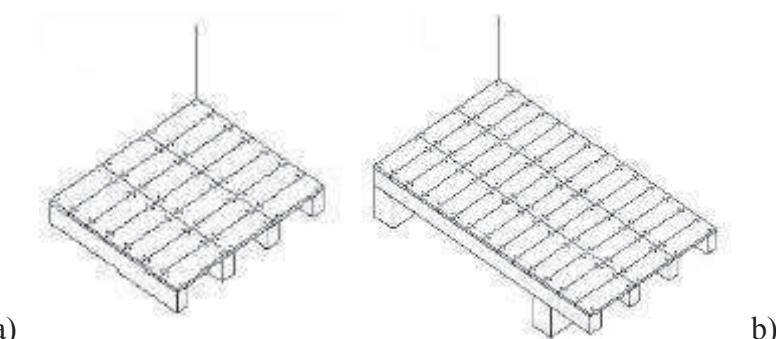


Fig. 1 One-way and two-way timber floors (Brignola, Pampanin and Podestà 2009)

The connection of the beams to the walls are a fundamental part of the total strength and stability of the floor. Joists generally run across the shortest distance between the walls. This is the case of one-way timber flooring. In case of greater widths to be covered, one or more sleeper beams are installed in order to reduce the total span (sleeper beams present bigger dimensions than the secondary joists). The secondary timber joists are placed on the top of the sleeper beams defining the area where to place a deck of wooden tables. In addition to the previous mentioned timber floors there are other possible typologies of horizontal structures that can be mentioned. For instance the so called panelled ceiling in which the secondary frame of beams has the same thickness of the main joists. By using mechanical connections (steel strips nailed on the joists) a timber grid is obtained. In particular this specific type of floor was applied on structure showing a rectangular shape allowing the better distribution of the loads along the lateral walls.

Moreover another typology of timber floor historically adopted is the “Serlio’s” timber floor based on the reciprocating design of timber elements. It was based on the possibility of bridging large span with timber beams that are shorter than the span and hence have a less depth in the section (Fig.2).

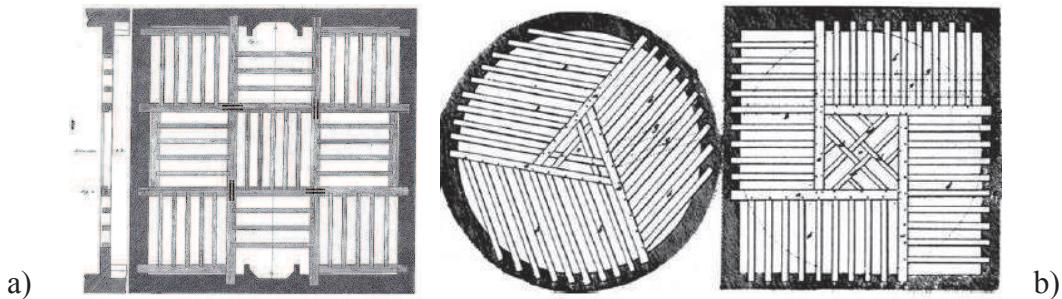


Fig. 2 Serlio's timber floor:a) rectangular configuration (Gelfi and Giustina n.d.); b) various configurations (Munafò 1990)

Historically it was identified in some cases the use of composite timber beams especially in the case of two-way flooring. The composite main beam was properly shaped in order to better resist the vertical loads coming from the upper elements. The global beams were forced to correctly work by adding steel rings wrapping the two sides of the beam (the upper and the lower parts). The beams were adopted in case of larger spans than the traditional ones.

Existing timber floors are usually connected to the lateral walls by simple interlocking between timber beams and masonry or by means of steel ties to improve the local link between the walls and the beams.

A relevant part of the existing buildings suffered significant damages in ancient and recent seismic events, defining economic and cultural losses. Typically these structures are part of the cultural heritage and most of them are still used as housing. The most common structures are based on a simple geometry based on a rectangular shape with common dimensions around $4.00 \times 6.00 \text{ m}^2$, $6.00 \times 8.00 \text{ m}^2$ or $6.00 \times 12.00 \text{ m}^2$. The walls can be characterized by single or more leaves reaching important thickness. As aforementioned timber floors were the widest used in order to define the horizontal storeys. A number of three storeys was very common with some exceptions. The importance of the study of timber floors and their behaviours increased during the last years due to the several effects seen during and after earthquake events affecting masonry constructions. Among the parameters that influence the global response of a structure (masonry structure) the role of timber flooring is fundamental.

3. SEISMIC BEHAVIOUR OVERVIEW AND TIMBER FLOORS

The last seismic events occurred in Italy in 1997-98 with the Umbria-Marche earthquakes and the most recent L'Aquila and Emilia earthquakes respectively in 2009 and 2012 showed several damages that affected the cultural heritage. From the experimental data, material characteristics, construction technologies and phases play an important role in the global structural response under a seismic event. From what was clear from the surveys after the earthquakes, masonry structures are distinguished by the formation of local and particular collapse mechanisms. In this way local modalities of loss of equilibrium of portions of the structure are identified and classified. Among the wide variety of collapse modalities the most common could be the following:

- Rotation out-of-plane of bearing masonry
- Rotation out-of-plane of the top part of the walls
- Masonry disconnections and masonry expulsion
- In-plane mechanisms for shear stresses with diagonal cracks

Generally the most common mechanisms are the ones with the out-of-plane rotation of the wall due to the lack of orthogonal connection with the other walls and with the timber floors. The lack of connection between the orthogonal walls allows them to work separately instead of as a unique element. Generally masonry constructions should work under horizontal actions involving a global box-behaviour, by considering a good connection between the walls. One of the first documents in which masonry structures are secured by using wooden floors come from the ancient Leonardo Da Vinci who wrote (Barbisan and Laner 1995):

«Each beam shall be passed through the walls and be secured there with sufficient chains, since beams have often been dislodged, ruining walls and floors in an earthquake; where they were secured with chains, they kept the walls firmly together and the walls in turn have secured the floors».

Even Leon Battista Alberti suggested the use of timber flooring instead of vaults or arches in order to resist a seismic event. During the centuries many researchers claimed the role of timber floors as anti seismic role in masonry constructions. The capacity of timber in the field of anti seismic construction (not just referring to the floors) can be indentified in the applications introduced after the Calabria earthquake occurred in 1783. The reconstruction of

the cities followed new roles in terms of construction techniques that in general involved wood as the main important element. Moreover the elasticity of wooden floors suggested the idea of dynamic energy absorption of the earthquake. The theory was introduced from a theoretical and practical point of view by Nicola Cavalieri di San Bertolo in 1831. Even the Padova scholar Favaro in 1883 underlined various applications on seismic prevention. Among the suggestions offered by Favaro the main important are the ones referred to the connection of the joists to the walls provided by adding a flat iron bar at the top and in the middle of the beam (Barbisan and Laner 1995) (Fig.3).

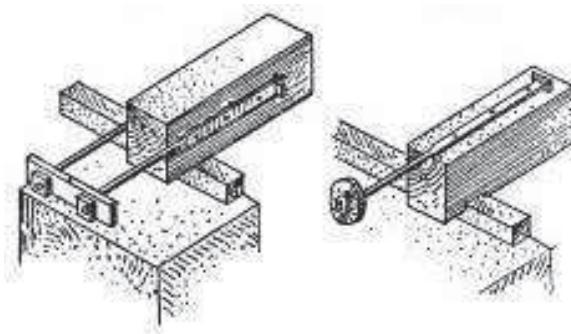


Fig. 3 Beam to wall connection suggested by Favaro (Barbisan and Laner 1995)

The last earthquakes heavily damaged the historical heritage and several applications were introduced also taking example from past experiences as a retrofit intervention on wooden flooring. As aforementioned the main problem is related to the disconnection between walls and the loss of box-behaviour of the global structure.

From several studies it was clear that just a simple connection between the walls provided by placing for example ties or acting as a refurbish solution on the timber floors enhance the global behaviour of the structure. Focusing on the timber floors two different typologies of floors can be evaluated, the deformable diaphragms and the rigid ones. The key role of diaphragm behaviour in the global response of the building was justified focusing on the collapse mechanisms and in general the global response. The high flexibility of timber floors and the lack of tie-in connection of floors and walls contribute to excessive displacements in correspondence of the storeys and possible overturning of the walls. Obviously the quality of the walls can influence the overall behaviour (Gattesco and Macorini 2014). Poor quality masonry elements leads to heavy damages linked to shear, sliding-shear and rocking mechanisms (Fig.4).

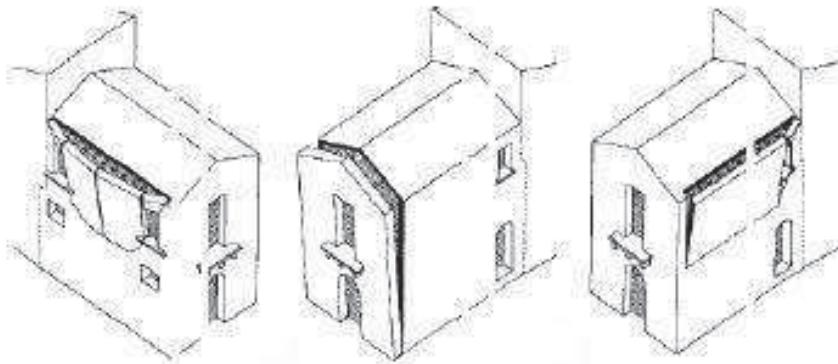


Fig. 4 First-mode collapse mechanisms: out-of-plane wall overturning (Giuriani and Marini 2008)

The inadequate link between the timber floor and the walls defines the poor capacity of the wooden system to redistribute shear forces among the lateral walls.

On the other hand several applications of rigid timber floors took place in masonry structures. The aim of enhance the overall behaviour of the structure reaching a three-dimensional system has been reached increasing the in-plane stiffness of the ancient timber floors. The main aspects linked with the behaviour of the floor are its in-plane stiffness and wall-to-diaphragm connections. Generally old timber systems were substituted by much stiffer diaphragms made of brick and reinforced concrete structures. These kinds of applications have been connected to the walls throughout a concrete beam inserted in the thickness of the wall. Considering this kind of application, issues that can be evaluated are its invasiveness in the historical structure and the consequences from a structural point of view. Excessive stiff diaphragm can lead to undesirable effects under an earthquake with the formation of mechanisms. Mechanisms are due to the compatibility problems of the used system but also for the lack of connection of the new configuration with the lateral walls. The excessive rigidity of the floor associated to the weakness of the ancient masonry walls lead to local collapses. As presented in the following images a possible effect of a reinforced concrete tie-beam in a masonry wall is that it is only supported by the internal leaf of the multi-leafs masonry causing load eccentricity and reduction of resisting area (Fig.5a).



Fig. 5 Damages on masonry buildings after a seismic event. On the left 1997 Umbria-Marche earthquake (Italy), on the right 2002 Molise earthquake (Italy) (Brignola, Pampanin and Podestà 2009)

In addition to the previously mentioned mechanisms one of the most frequent is the expulsion of the building corners (Fig.5b). This kind of collapse is linked with the poor connection between the floor system and the walls. The angular deformation due to the different shear resistance of the walls leads to shear distribution in the timber diaphragm. As a result the diagonals of the horizontal element result either under compression and tension. Consequently concentration of compression forces in correspondence to the compressed corners activates the out-of-plane mechanism of expulsion of the building corners (Brignola, Pampanin and Podestà 2009). Besides as the diaphragm in-plane flexibility increases, torsional effects occur. As a result of this last consideration, when torsional effects are a concern, a no-intervention or a reduction of the stiffness could be desired. For this reason strengthening solutions like the one involving reinforced concrete slabs is not proposed anymore and just more traditional techniques, aiming only a reduction of excessive deformability, are proposed.

Moreover some considerations should be done referring to possible interventions on the masonry walls. Sometimes just a refurbishment of timber diaphragm is not enough to guarantee the box-behaviour of the structure. As aforementioned the quality of the walls strictly affects the general performances of the building. Due to this in addition to applications on the floors some other strategies should be applied on the walls. Stiff diaphragms lead to a different distribution of seismic actions among the walls. Among the possible techniques criteria for the seismic improvement of the walls can be considered the grout injection and repointing applications. It is of fundamental importance provide a good connection between corner walls by adding for instance anchoring ties, reinforcing rings or as said before floor-to-wall connection.

4. RETROFITTING TECHNIQUES ON TIMBER DIAPHRAGMS

Strengthening interventions on timber diaphragms can be done either at the extrados or intrados side. Generally strengthening solutions can be divided into two categories, wet and dry applications. Frequently the first one involves the use of lightweight reinforced concrete slabs layered on the top of a wooden board plank. On the other hand dry techniques involve the use of timber elements nailed to the previous floor system, applications with steel plates and diagonals or more recent FRP strips.

Herein is presented a short overview about the retrofitting techniques (Brignola, Pampanin and Podestà 2009). The possible interventions are the followings:

- Concrete topping for composite action: it is a common and widely used technique, consisting of a lightweight concrete slab (40-50 mm thick) with or without steel connectors. The slab is superimposed on a deck of wooden table nailed to the timber beams. The diffused reinforcement is a steel wire-mesh (5-6 mm diameter, 100×100 mm wire mesh). The connection between the slab and the below timber joists is obtained through several types of elements (e.g. L-shaped steel bars, nails, axial connectors). As a result of this application a T-shaped composite resistant section is attained (Fig.6).

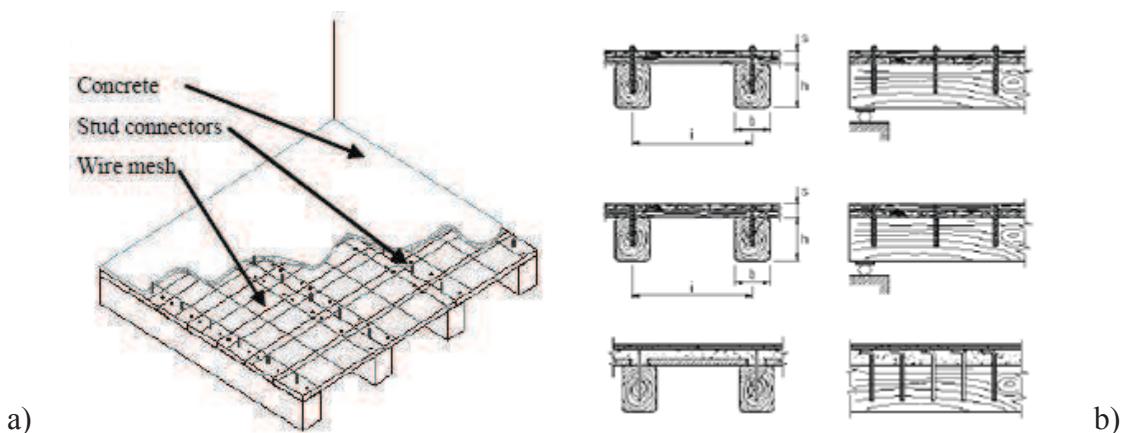


Fig. 6 Concrete topping: a) concrete topping layer with a steel wire-mesh and connectors (Brignola, Pampanin and Podestà 2009); b) different types of connectors (Piazza, Tomasi and Modena 2005)

Pros of the application is the higher stiffness reached by the timber diaphragm that allows a proper redistribution of shear forces among the lateral walls. From the cons point of view should be considered the increase of the weight at floor level and the possible not so good interaction with the masonry walls if an adequate connection is not provided.

- Cross laminated plywood sheets: the technique involves the superimposition of one or more layers of wood planks or plywood panels over the existing sheathing. The connection between the new deck and the past one is provided by using nails and screws. Normally the new planks are applied perpendicularly to the previous ones but it is used to arrange them also diagonally. Application of nail plates (gang nails) to join adjacent timber boards is a quite common solution that provide a better adherence between the wooden elements. This technique prevents the slip of the board due to shear forces (Fig.7).

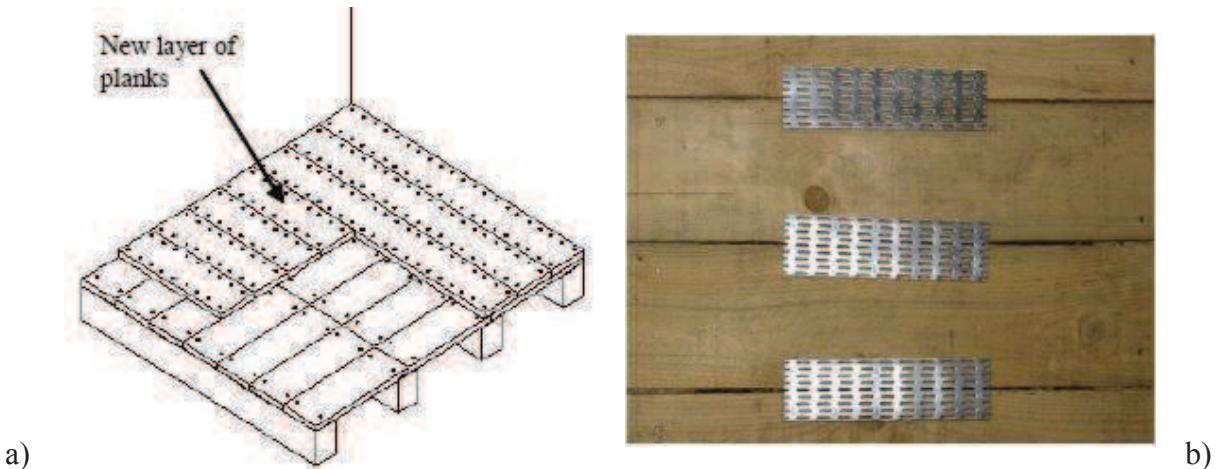


Fig. 7 Strengthening applications: a) new layer of wooden boards (Brignola, Pampanin and Podestà 2009); b) on the right floor stiffened by using gang nail plates (Gattesco and Macorini 2014)

- Fibre reinforced Polymers (FRP, CFRP) or steel plates: the techniques involves the use of diagonal bracing to the existing wood planks. The FRP strips are bonded to the wooden boards gluing them by using epoxy-based resins. On the other case the steel plates are nailed or screwed (Fig.8). The application of diagonal elements on the extrados of the timber floor can be also done by using wooden boards placed as a San Andrea's cross.

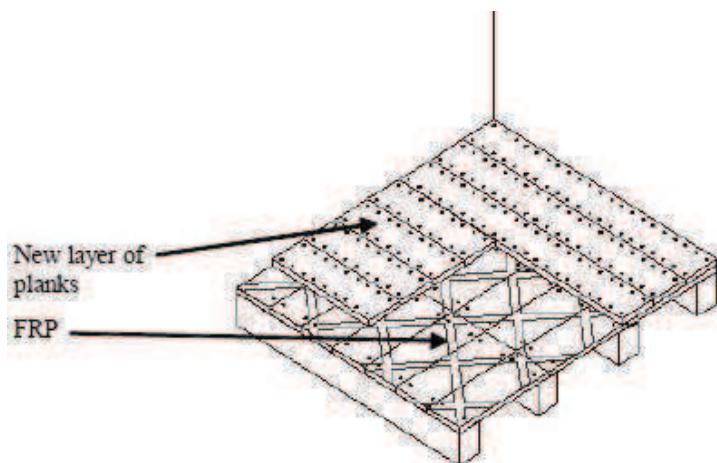


Fig. 8 Timber floor stiffened by applying FRP strips (Brignola, Pampanin and Podestà 2009)

Considering the first possible application with lightweight reinforced concrete slab, even if it is lighter and more compatible with the structure than the fully reinforced one is generally avoided. More reversible and adequate interventions are normally preferred. In addition to these kind of applications the lateral floor-to-wall connection can be provided in different ways. U.S. and New Zealand buildings' solutions involve steel rods embedded inside the masonry at the level of the timber joists. A metallic anchor on the external side of the building provide the appropriate link. Controversy this solution in Europe did not find real applications because of the irregularities of the walls connected with the roughness of the beam shape did not allow its correct effect. On the other hand European solutions involved the use of Y-shaped anchor plates applied on the extrados of the timber floor. Moreover recently an L-shaped steel plate was applied on the extrados of the diaphragm all along the boundary. The steel plate is linked to the walls with threaded steel bars and mechanically or chemically connected inside the masonry walls.

5. RETROFITTING STRATEGIES: ANALYTICAL APPROACH

The overall behaviour of a masonry structure is strictly linked with the diaphragm in-plane response. The general resisting capacity of a floor against horizontal forces and redistribute them among the walls defines the possibility to reach a more effective hierarchy of strength. Generally the evaluation of a strengthening approach has to be justified considering the in-plane and out-of-plane behaviour of URM buildings. Conventionally structural analysis are based on the assumption that roofs and floors diaphragms are enough stiff. Diaphragms are assumed to distribute the horizontal force among the shear walls oriented in the same direction. Real cases have shown the invalidation of these assumptions because timber floors are so deformable and the walls so rigid that the function of the floors is not reached. The analysis should be done considering the adequate stiffness of the diaphragm and the out-of-plane response. For these reasons parameters linked with the in-plane behaviour of the floor and the lateral response of the walls have to be properly evaluated. Nowadays there are no real analytical approaches to the evaluation of the in-plane stiffness of existing timber floors. Some guidelines offer general contributions in this direction. The general behaviour of horizontal wood diaphragms is influenced by the decking system in terms of size and the amount of fasteners. Other factors involved into these calculations are the wheelbase between the joists and the presence of perimeter chord. Acting in this sense the presence of lateral chord allows a limited reduction of in-plane stiffness of the floor. As reported in the NZSEE Assessment Guidelines (NZSEE 2006) in case of strengthening techniques are available some possible calculations involving three contributions to the diaphragm deflection of the floor. The global deflection is used to define the stiffness of the global system. The first factor is coming from the flexural deformation considering the chord as a moment resisting couple. Secondly the contribution due to the shear deformation of timber beams and finally the nail slip.

Four different applications can be evaluated depending on the type of the adopted strengthening system. Considering the Italian guidelines local mechanisms have to be evaluated in terms of the activation of the mechanism (Damage Limit State) and the maximum displacements limits (Ultimate Limit State). If the required parameters are not satisfied a strengthening strategy should be applied. The strategy modifies the equivalent stiffness of the floor in order to enhance the global behaviour. The steps involved in the

definition of a strengthening technique are the following (Brignola, Pampanin and Podestà 2009):

Evaluation of the in-plane stiffness of the sole diaphragm ($k_{eq,d}$) and the stiffness of the floor-to-wall connectors (k_c). Once defined, the equivalent stiffness of the entire floor system have to be considered.

Several in-plane and out-of-plane collapse mechanisms have to be studied and their collapse multipliers shall be defined. By considering the adequate Limit States (Damage and Ultimate) and the proper design earthquake, the evaluation of the hierarchy of strength has to be done. The lowest values of the multipliers are the ones that define the correct methodology for the strengthening technique.

The target variation of equivalent floor stiffness ($\Delta k_{eq,c+d}$) required to achieve the desired performance and the hierarchy of strength are evaluated.

The retrofit design is chosen and the evaluation of the new stiffness is done by considering the $k_{eq,d}^*$ (stiffness of the sole diaphragm) and k_c^* (stiffness of the connectors). The general stiffness is estimated with the following approach

$$k_{eq,c+d}^* = k_{eq,c+d} + \Delta k_{eq,c+d}$$

The diaphragm stiffness is composed of two factors that are the stiffness of the sole diaphragm and the stiffness of the connectors. They are working in series so that the final displacement of the floor is given by the sum of the two factors.

$$\delta_{FIN} = \delta_c + \delta_d$$

The two contributions can be considered in different cases. The first ideal case takes into consideration rigid connectors ($k_c \rightarrow \infty$) and as a result the global deformation is just due to the internal diaphragm stiffness. Likewise if the assumption is a rigid diaphragm ($k_{eq,d} \rightarrow \infty$) the overall deformation of the floor is due to the connectors stiffness factor. The equivalent stiffness of the floor that should be used for the assessment, design and retrofit interventions is provided by a combination of both contributes with the following expression:

$$\frac{1}{k_{eq,c+d}} = \frac{1}{k_{eq,d}} + \frac{1}{k_c}$$

Equivalent stiffness should be correctly evaluated before and after the intervention in order to quantify the real enhance to the floor system.

The analytical evaluation of the sole diaphragm stiffness can be done considering simple loading conditions. Examining a simple non-strengthened floor system with a wooden deck

made of boards of 20 mm thick and 100-200 mm wide nailed to the cross beams, the global stiffness of the system is composed by three separate factors: δ' as flexural deformation of the single board, δ'' as shear deformation of the single board and δ''' the rigid rotation of the board caused by the nail slip.

$$\delta = \delta' + \delta'' + \delta''' = \left(\frac{F'}{k_{ser}} \cdot \frac{2}{s_n} + \frac{\chi}{GA} \cdot F + \frac{l^2}{12EI} \cdot F \right) \cdot l$$

The previous equation has F'/k_{ser} = nail slip coming from the shear force F , k_{ser} = nail deformability determined through experimental studies, χ = shear factor, G = shear modulus of timber boards, E = flexural modulus of timber boards, A = area of plank section, I = moment of inertia of boards sections, l = wheelbase between beams and s_n = nail spacing.

If the wooden deck is composed of boards interrupted at each beam it is easy to obtain an equivalent shear modulus of the floor:

$$G_{eq} = \frac{\chi \cdot F_T}{Bt} \cdot \frac{L}{A} = \frac{\chi}{A} \cdot \left(\frac{l}{k_{ser} \cdot s_n^2} + \frac{\chi}{GA} + \frac{l^2}{12EI} \right)^{-1}$$

Where B = total width of the floor, t = thickness of the boards, F_T = total shear force acting on the diaphragm. In case of strengthening timber floors by using gang nails or CFRP fibres two other analytical procedures are herein proposed. The main aim is to define the in-plane stiffness of a timber floor strengthened using the previous techniques.

The non-strengthened diaphragm is characterized by a specific stiffness. Firstly it is of fundamental importance the definition of the rotational stiffness (k_n) for a single nail couple (the nail couple is an ideal rotational spring). The rotational stiffness (k_ϕ) can be calculated using the rotational one as shown in the following equations.

$$k_n = \rho_m^{1,5} \cdot \frac{\phi^{0,8}}{30} \quad k_\phi = k_n \cdot \frac{a^2}{2}$$

Finally the total response of a wooden floor can be evaluated considering the response of a single board or joist subjected to a horizontal force respectively parallel to the joists or to the boards.

$$\frac{F}{n_b} \cdot \delta = n_j \cdot k_\phi \cdot \alpha^2 \quad \frac{F}{n_j} \cdot \delta = n_b \cdot k_\phi \cdot \alpha^2$$

where δ is the displacement, α is the rotation of the element defined as $\alpha = \delta/L$.

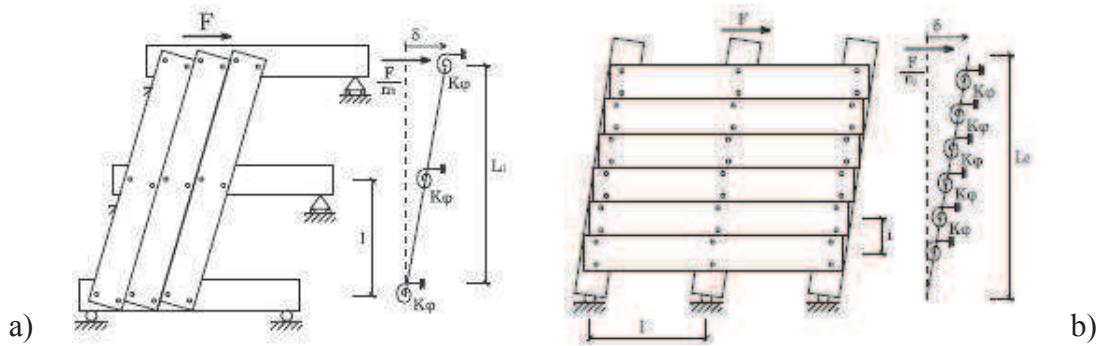


Fig. 9 Mechanical model for a non-strengthened timber floor: a) shear force parallel to joists; b)shear force parallel to boards (Gattesco and Macorini 2014)

The in-plane stiffness of a non-strengthened timber floor is herein presented:

$$K_{unstr} = \frac{n_j}{L_i^2} \cdot n_b \cdot k_\varphi$$

The same procedure is carried out taking into account the presence of gang nails that are used in order to guarantee a much better friction between the wooden boards. Even in this case the approach to the problem is similar to the previous one considering respectively one board or a joist subjected to horizontal force.

$$\frac{F}{(n_b-1)} \cdot \delta = (n_j - 1) \cdot k_{np} \cdot \frac{i^2}{L_1^2} \cdot \delta^2 \quad \frac{F}{(n_j-1)} \cdot \delta = (n_b - 1) \cdot k_{np} \cdot \frac{i^2}{L_2^2} \cdot \delta^2$$

where the k_{np} value is the stiffness of the linear spring that ideally is the gang nail element.

Once defined the contribution of the gang nails the total stiffness of the sum of the contributions with the stiffness corresponding to the non-strengthened condition.

$$K_{str,gn} = (n_j - 1) \cdot (n_b - 1) \cdot k_{np} \cdot \frac{i^2}{L_i^2} + \frac{n_j}{L_i^2} \cdot n_b \cdot k_\varphi$$

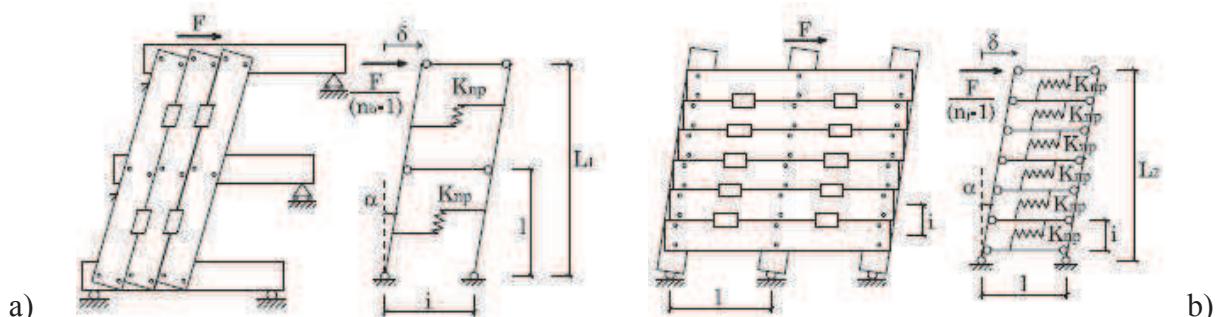


Fig. 10 Mechanical model for a strengthened timber floor by using gang nails: a) shear force parallel to joists; b)shear force parallel to boards (Gattesco and Macorini 2014)

A similar approach is used to define the final stiffness of a timber floor strengthened using diagonal CFRP strips. Even in this case the proposal is based on the possibility of considering respectively board or joist. The CFRP diagonal strips are idealized as linear diagonal springs.

The total stiffness of the strengthened diaphragm is obtained by the addition of the two components as presented in the following expression:

$$K_{rc} = n_h \cdot n_v \cdot \frac{(E_f A_f)}{d} \cdot \frac{c_i^2}{L_i^2} \cos^2 \beta + \frac{n_j}{L_i^2} \cdot n_b \cdot k_\varphi$$

The design procedure of a strengthened floor should also consider in-plane shear capacity and bending capacity. The first one depends on the shear resistance of gang nails or the axial resistance of the fibre strips.

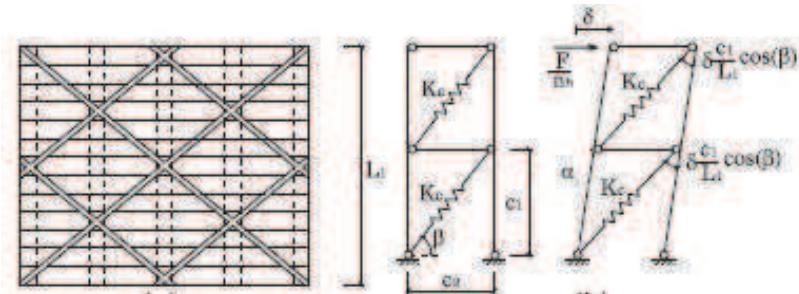


Fig. 11 Mechanical model for a non-strengthened timber floor with the addition of diagonal strips of CFRP material (Gattesco and Macorini 2014)

Even if the cases with the applications of gang nails or CFRP diagonal strips are not treated in this thesis, they could be studied in further analysis and works. The current work is just focusing on the simple configuration of a non-strengthened timber floor. Only some improvements in the connections between boards and beams and between the boards are taken into account.

6. EXPERIMENTAL TESTS CONFIGURATIONS

This thesis is based on a previous work done by a student of the University of Padova for his Master dissertation. His work focused on the experimental evaluations of laboratory tests performed in the University laboratories.

The tests were performed inside the laboratories of the Department of Civil, Architectural and Environmental Engineering (DICEA) of the University of Padova following the code UNI EN 26891. Starting from the experimental data the work focused on the drawing of the response curves of different configurations of timber samples and the calibration of the FE models. The curves were done by considering a force displacement diagram. The configurations of the specimens were the simplest one with wooden boards nailed on the central timber beam. The second and third samples were made applying two and three layers of wooden boards connected to the timber beam with nails and screws. In these two cases the boards presented the same thickness (20-25 mm thick). The last configuration was characterized by the superimposition of two layers of boards with different thickness nailed and screwed to the beam.



Fig. 12 Machine configuration used during the tests (Borsatto 2013-2014)

From the tests, response curves were defined, evaluating the different contribution of the nails and screws to the general strength of the system. The overall load test was based on the application on the top of the beam of an incremental displacement up to 30 mm and the reaction was calculated at each step. The procedure was carried out for the first six steps with the following increase (0,001 0,01 0,02 0,04 0,08 0,12 0,20). From this last step the range of increase of 0,1 stays constant up to the displacement of 0,5 mm. Later on the increment is maintained steady with a value of 0,5 up to the final displacement of 30 mm.



Fig. 13 Tested samples: a)sample A; b)sample B; c)sample C; d)sample D (Borsatto 2013-2014)

These samples were used as examples of possible retrofitting strategies for timber floors (addition of nails, wooden cross on the extrados side of the floor with screws etc.).

Once evaluated the response curves of these examples, the work concentrated on the estimation of the global behaviour of refurbished timber floors. The evaluated configurations were considered by adding for instance on the extrados of the floor diagonal timber elements, San Andrea's cross with wooden boards or carbon fibres and finally steel plates.

The idea at the base of this work was the numerical evaluation of the response of a laboratory sample of a timber floor under the effect of an horizontal force. The aim was the definition of alternative strengthening techniques defining a calculation methodology for the in-plane stiffness of timber floors. The tested floors (2190×2190 mm 2) presented in all

configurations a perfect joint as a base constrain. The timber joist at the base of the floor is connected to the machine and the static representation of the global behaviour of the floor is a cantilever. The maximum flexural stress is defined on the left and on the right where the floor is linked to the machine, while a zero value is present in the central part. On the other hand the shear stress is maximum in the central area and null on the extreme parts.



Fig. 14 Tested samples of timber floors: a) non-strengthened configuration; b) strengthened configuration with steel strip; c) strengthened configuration with one diagonal wooden board; d) strengthened configuration with two diagonal elements; e) strengthened configuration with diagonal CFRP fibres; f) strengthened configuration with two diagonal steel plates (Borsatto 2013-2014)

7. MODELLING TYPOLOGY OF TIMBER DIAPHRAGMS

The scientific literature based on the evaluation of the in-plane behaviour of a timber floor system is characterized by several typologies of modelling. Depending on the characters of the tests, many cases can be taken as examples of how to properly model a wooden diaphragm.

As previous mentioned there are many refurbish applications that can be applied to timber floors and each of them needs a specific definition in terms of modelling.

Numerical modelling are developed in order to verify already done tests or to forecast the possible and more probable behaviour of the materials and the structure itself.

The material behaviour of timber can be assumed to be orthotropic with the axial direction parallel to the grain, radial and tangential directions are principal directions. Even if the real behaviour of wood is modelled as orthotropic, generally it can be studied and adopted as isotropic. The examples provided in this chapter were treated considering wood as an orthotropic material.

Starting with the modelling of a timber diaphragm composed of joists and a wooden plank a simple model that can be evaluated is the one proposed by Brignola (Brignola, Pampanin and Podestà 2009). The behaviour of a timber flooring system can be evaluated creating a 3D model in which joists are modelled as elastic beam elements in two dimensions. The wooden plank with a section 3×20 cm is modelled defining plate elements. Where the nails pass from the wooden deck into the beams, links are introduced parallel to the plank in order to reproduce the slip behaviour of the nails and a rigid rotation of the deck.

Simplified modelling for a non-strengthened floor is the one used by Gattesco (Gattesco, Macorini and Benussi 2007). He modelled every member by using beam elements. The connection element (nail) was replaced by a non linear spring. The higher deck of wooden tables was drawn considering other beam elements orthogonally placed on the previous ones (Fig.15a).

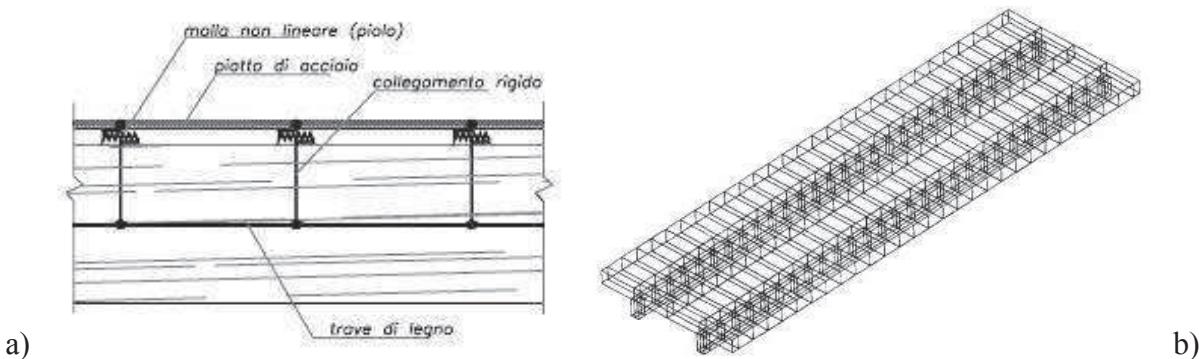


Fig. 15 Types of numerical modelling of timber floor: a) model with linear beam and plastic springs (Gattesco, Macorini e Benussi 2007); b) solid elements and plastic springs for a composite timber-concrete floor (Ahmadi and Saka 1993)

A possible retrofitting technique involving the light weight concrete slab on the top of the timber system can present the following approach. The concrete slab and the timber joists are represented by solid elements (block elements) and the wooden deck by quadrilateral bending-plate elements. Where the connectors are placed, the solid elements of the concrete slab, wood beams and bending-plate elements have common points (nodes). The elements are divided into solid parts where the connectors are located. There are also the possibilities to substitute the previous punctual connectors with spring elements. The connectors can be replaced by spring elements parallel oriented to the global axis (Fig.15b). Acting in this way a more realistic behaviour of the connectors is reached. The non-linear behaviour of the nails is perfectly modelled by this spring system (Ahmadi and Saka 1993).

Moreover a numerical model for a timber diaphragm system is the one that involves elastic beams elements representing the timber joists. These elements are connected to the perimeter walls by pinned joints. The finite elements that can be applied on these structures are 4-noded shell elements. These kinds of elements can be used both for the modelling of a concrete slab and also to represent the characteristics for a floors strengthened with lower invasive approaches like the use of gang nails or FRP strips. A bit more complicated is also the modelling of the improved floor-to-wall connection. Generally the connection is provided by the addition of an L-shaped plate along the perimeter of the floor. The L-shaped steel element can be defined by using another beam element (Gattesco and Macorini 2014)

Besides another type of modelling can be found in the applications involving fibres or steel plates or shaped steel plates. In this case we can consider the modelling carried out by Corradi (Corradi, et al. 2006). Some tests were performed on timber floors strengthened with traditional techniques just adding a layer of planks or with more innovative configurations considering fibres. The numerical models developed were characterized by the use of 2 nodes

beam elements for the joists, the higher rafters and the composite materials fibres. A 4 node shell element was chosen for the wooden plank floor. The wood was treated as an orthotropic material (axial direction: parallel to the grain, perpendicular to the grain, tangential and radial: principal directions). In the modelling of a floor strengthened with a superimposition of wooden planks the plank-to-plank connection is provided by a gap element (spring elements).

8. MODELLING PROCESS: FEM ANALYSIS PUSHOUT SPECIMENS

8.1. Introduction

In this chapter is presented a simplified explanation about the procedure to create a numerical model.

Firstly two simplified models of specimens are modelled evaluating the response of the finite element analysis and comparing the data with the experimental ones. The first sample is composed of a central timber beam on which are nailed wooden boards on opposite sides. The second specimen presents the same configuration as before but other two thicker wooden boards are screwed on the previous ones. As a result the sample is constituted by two board with different thickness on opposite sides. For each of the samples three measurements were taken in order to have a probabilistic distribution of the data. The aim of this first part of the thesis is to compare the numerical distribution of the data (Force-Displacement law) of the nails and screws with the experimental ones. The two specimens considered in this first part of the thesis have been used to calibrate the future work. Considering the difficulties and the results obtained from these two models the work on the floors was set.

The second part of the thesis is based on the modelling of timber floors in a non-strengthened and strengthened conditions. The first model used as starting floor for the other strengthened is the one composed of five timber joists on which wooden boards are fixed. The most common timber floors present simple boards just put close one to the other one. In this case the boards are considered having a rabbet joints in order to obtain a better and uniform in-plane behaviour. The finite element representation of this type of joint is a linear interface with a frictional behaviour.



Fig. 16 Types of wooden boards: a) without rabbet joint; b) with rabbet joint (Vincentz 2008)

8.2. Applied constitutive models

The work is based on the elaboration of previous data collected during laboratory tests. The main aim is to reproduce numerically the behaviour of the considered samples. Structural non-linear analysis are performed with a finite element model. The software DIANA 9.6 is used for this purpose (Manie 2014).

8.2.1. Diana 9.6

DIANA (DIplacement ANAlyzer) version 9.6, is a dedicated software for a wide range of applications spacing from structural, geotechnical and earthquake engineering. Several applications are also found in the branches of engineering in historical constructions and masonry buildings.

The software, having an integrated solver with pre- and post- processing tools, it becomes a powerful code. Several types of analysis can be performed such as structural linear static, non-linear analysis and dynamic ones. Among the possibilities of non-linear applications some of the most relevant are the ones involving plasticity, creep, seismic analysis, buckling and post-buckling. The capabilities of the software allow to evaluate a wide range of possibilities covering 2D and 3D applications.

DIANA presents a wide choice in terms of structural finite elements like flat/curved shell elements, beam elements, interface elements, solid elements etc. The availability of different constitutive models integrated with these structural elements allows the solution of complex structural problems in the fields of concrete, steel, masonry and wood sources.

8.3. Sample A

The first sample has to be modelled according to the geometry used during the experimental tests. This sample is tested in order to verify the response of the connections between the central timber beam and the lateral wooden boards. The sample presents a $110 \times 140 \text{ mm}^2$ section wooden beam and 600 mm high. The lateral boards are $140 \times 20 \text{ mm}^2$ section and 600 mm high. On each side eight steel nails are used to fix the boards on the beam (total nail number=18).

A pushover analysis was applied on the sample, imposing an increasing displacement to the top of the beam. The maximum applied displacement was 30 mm.

The finite element analysis are performed assuming wood as an isotropic material well knowing that this is an approximation of its real behaviour.

Both beam and boards belong to the *Spruce Pine* family and they are classified as C24 by UNI 11035-2:2010.

	Beam	Board
Base [mm]	110	110
Width [mm]	140	20
High [mm]	600	600

Table 1 Geometric dimensions of the sample A

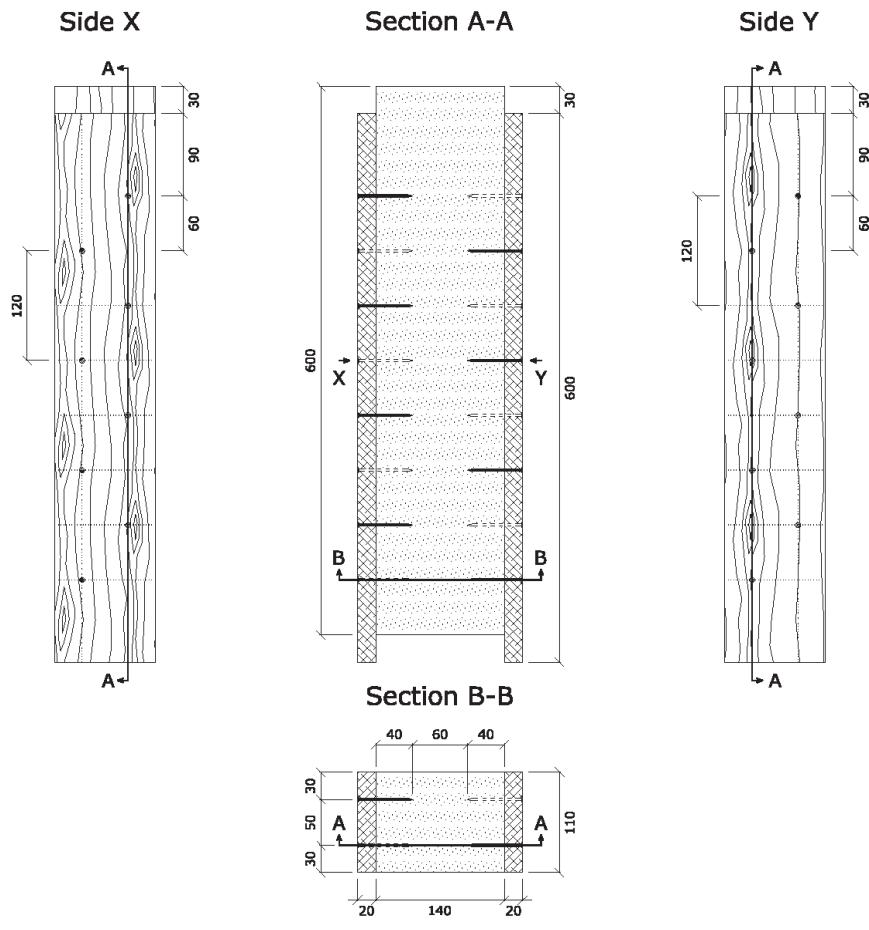


Fig. 17 Sample A

	Beam	Board	Nail
Young's Modulus [MPa]	12000	12000	210000
Poisson's Ratio	0,35	0,35	0,3

Table 2 Mechanical properties of the sample A

This sample represents the most common solution in existing building with timber diaphragms. It was made of a simple connection between board and beam by using nails. This

application allow us to have a reference in the case of the floor without strengthening condition.

8.3.1. General observations: geometric simplification and modelling

From the geometrical modelling point of view some problems were found during the evolution of the project. Initially the samples have been drawn as solid elements defined by extrusion of faces. The central timber beam and plates were constituted by several solid prisms. On the other hand nails were defined as simple lines and the punctual interface was applied. The principle of symmetry was used in order to simplify the single specimen and for this reason constrains that take into account this simplification were attached to the model.

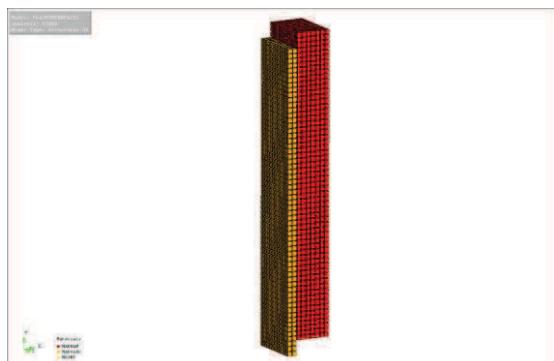


Fig. 18 Initial configuration of the specimen with solid elements defining joist and board and punctual interface for the nails

Some problems related to the orientation of the local axis of every single prism contributed to the impossibility of carrying out the calculations. As aforementioned prisms were drawn by solid extrusion. For this reason a properly order should be used to model all the sample. Inside the dat-file coming from the model, the local x,y,z axis should have the proper orientation by considering as a reference the global X,Y,Z reference axis. The impossibility of changing the orientation of the local reference system forced to change the model shape and its simplifications.

As a result the final samples were modelled with predefined beam elements, the boards by considering plates shapes and the connections with rigid connections.

8.3.2. Modelling of Sample A

The model is simplified by considering a line as the body of the timber beam and plates as the wooden boards. Nails are drawn as simple lines connected at the correct level to the beam. In order to simulate the real behaviour of the nails, rigid connections constraints are attached to the lines. Between the nails and the board a non-linear interface that allows to connect two

nodes is applied (the interface simulates the non-linear behaviour). Considering the wide variety of mesh typologies that iDiana can offer, a user defined mesh size and type is adopted. Geometric and physical properties are attached to the model's parts. The following table presents the fine element typologies considered for the model.

	Beam	Plates	Contact node-node
Element type	LI13BE	CQ48S	N6IF
N° nodes	2	8	2

Table 3 Typologies and number of nodes of the selected finite elements for the sample A

The eight-node quadrilateral curved shell element is here presented. It allows a faster integration than a Q24SF used in previous applications.

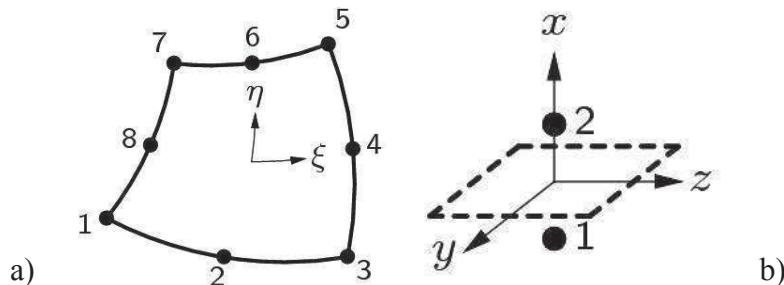


Fig. 19 Fine elements used: a) quadrilateral element CQ48S with eight nodes; b) punctual interface N6IF (Manie 2014)

Three samples with same shape and characters were tested. The force-displacement law was modelled starting from experimental results and later compared with the numerical response coming from the fem analysis.

A multi-linear bond-slip behaviour is applied to the interfaces in order to reproduce the non-linearity of the nails during the test. Bond-slip describes a relation between the shear traction t_t and the shear slip Δu_t^0 , considering a linear relation between normal traction and normal relative displacement. iDiana offers four types of bond-slip law and a multi-linear behaviour is selected.

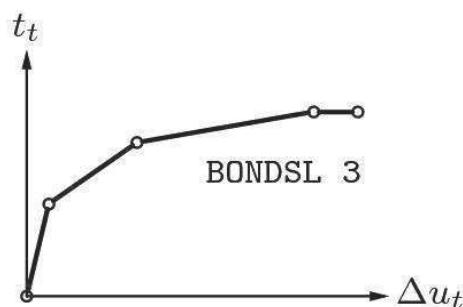


Fig. 20 Multi-linear bond-slip behaviour (Manie 2014)

8.3.3. Characters of the analysis: Parameters

Regarding the characters of the analysis a structural non-linear calculation was set. The analysis were carried out under displacement control and due to this no arch length control systems have been adopted. Considering the properties of load, an user's specified step size of 0,08 has been applied for 500 times. The use of a so small size step was due to the bond slip behaviour attached to the punctual interfaces. The high slope of the first part of the curve imposed a very small size step. The maximum number of iteration defined was 200 with a Secant (Quasi-Newton) method of iteration. The chosen type of iteration was the BFGS one. In order to stabilize the convergence of the system a Line Search algorithm with default values parameters was considered. With regard to the convergence criteria, displacement and force controls have been chosen rather than the energy based one.

Set	Characteristics
Method	Secant (Quasi-Newton) - BFGS
Load steps	0,08 (500)
Max. number of iterations	200
Line search	Yes
Displacement Criteria	Tolerance 0,01
Force Criteria	Tolerance 0,01
Solution of system of equations	Sparse Cholesy with a tolerance of 1e-08

Table 4 Characteristics of the analysis

8.3.4. Results of the numerical model

Once performed the analysis into Diana Mesh Editor, applying in several steps the displacement to the beam up to 30 mm as in the real tests, the experimental curve is obtained. Therefore the numerical curve is overlapped the experimental ones of the three samples. Herein the graph is presented and it is easy to confirm that the numerical results are following the experimental ones.

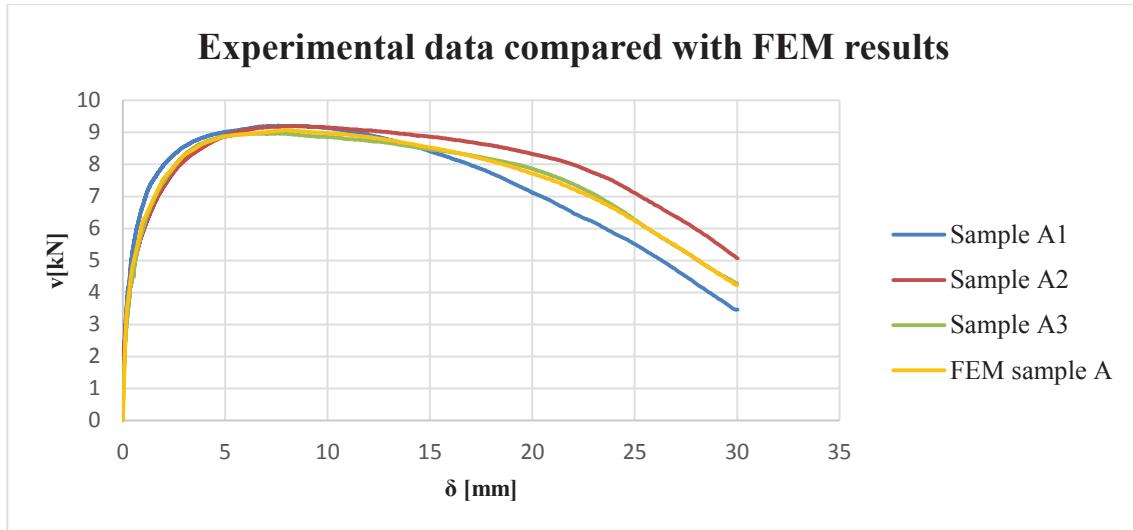


Fig. 21 Comparison between experimental data and numerical ones

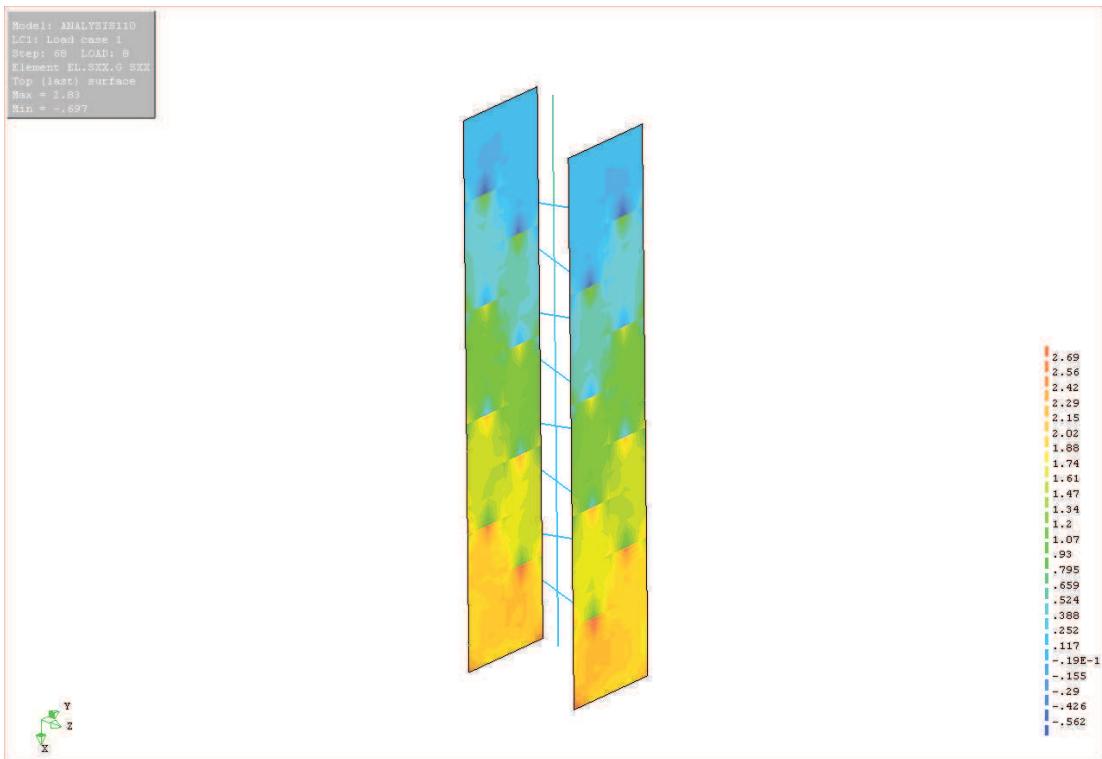


Fig. 22 Distribution of σ_{xx} stresses in the model

8.4. Sample B

The second sample follows the same approach in terms of elements and simplifications, already presented for the previous case. The specimen is composed of the central timber beam and two layers of wooden boards laterally connected to it. The first layer of boards is nailed and screwed to the beam and the outer and thicker one is just screwed to the joist. On each side eight steel nails and eight screws are used.

A pushover analysis is applied on the top of the beam with an increasing displacement reaching the maximum of 30 mm.

A finite element analysis is performed assuming a simplified isotropic behaviour of the wood instead of an orthotropic one.

Even in this case the wood comes from the *Spruce Pine* family and it belongs to the C24 resistant class according the UNI 11035-2:2010.

	Beam	First Board	Second Board
Base [mm]	110	110	110
Width [mm]	140	20	40
High [mm]	600	600	600

Table 5 Geometric dimensions of the sample B

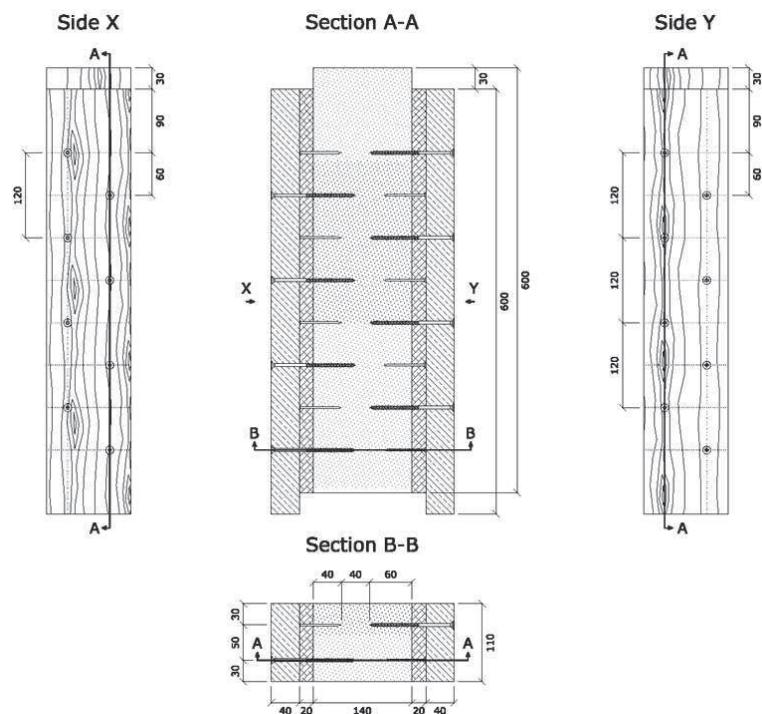


Fig. 23 Sample B

	Beam	First Board	Second Board	Nail/Screw
Young's Modulus [MPa]	12000	12000	12000	210000
Poisson's Ratio	0,35	0,35	0,35	0,30

Table 6 Mechanical properties of the sample B

This sample represents a possible strengthening application in which a thicker layer of boards can be placed on the existing one. As for this configuration, the analyzed strengthening condition presents a timber cross on the extrados simply screwed to the lower boards and joists.

8.4.1. Modelling of sample B

The model is defined using the same procedure of the basic one. The central timber beam is drawn as a line to which geometrical and physical properties are attached. In this case two plates per side are representing the two layer of wooden boards. Nails and screws are exemplified as rigid connection starting from the beam up to the plates. As aforementioned non-linearity of the nails are simulated by considering an interface element where the nails are passing from the beam to the boards. The same typology of element is also attached in correspondence to the screws passing from the beam to the first boards and from the first to the second boards. The bond-slip is contemporary applied to the interfaces for nails and to the ones for the screws.

As for the previous example, same types of elements were used and applied to the geometries of the sample. Three specimens were tested and the data have been collected and graphed. The results are compared with the numerical ones coming from the finite element calculations.

8.4.2. Characters of the analysis: Parameters

As in the first case, structural non-linear analysis has been defined. Due to the fact that a displacement control analysis was considered, no arch length control was set. The user's specified step size was 0,25 for 240 times. The maximum number of iterations was 300 and the selected method was, as in the previous one, the Secant (Quasi-Newton) type BFGS. The stabilization of the calculation was obtained by adding the Line Search algorithm. The convergence criteria adopted were the displacement and the force one.

Set	Characteristics
Method	Secant (Quasi-Newton) - BFGS
Load steps	0,25 (240)
Max. number of iterations	300
Line search	Yes
Displacement Criteria	Tolerance 0,01
Force Criteria	Tolerance 0,01
Solution of system of equations	Sparse Cholesky with a tolerance of 1e-08

Table 7 Characteristics of the analysis

8.4.3. Results of the numerical model

Herein the graphs of the laboratory tests and the FEM analysis are presented. It is easy to confirm that the numerical result is very closed to the experimental ones.

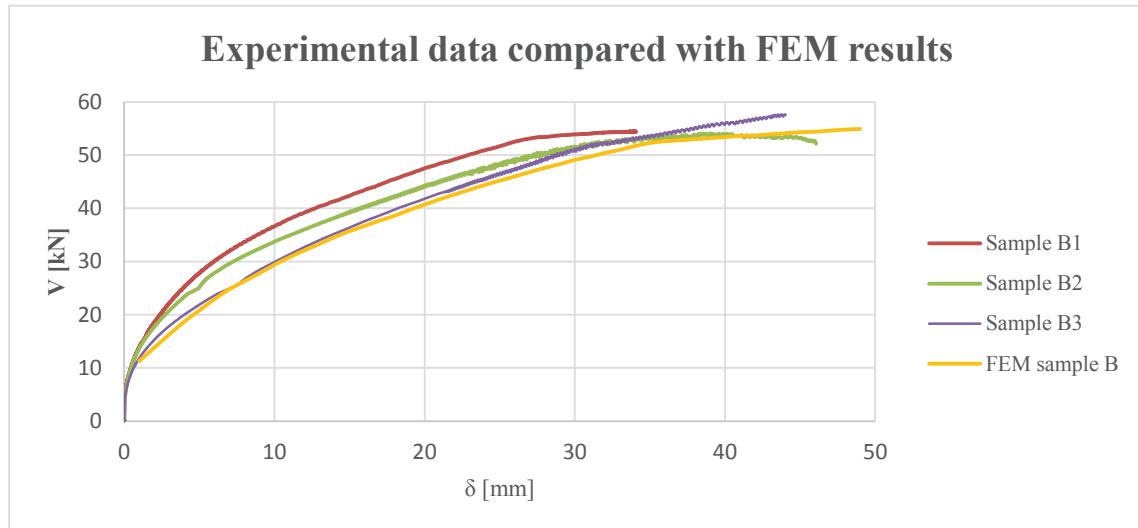


Fig. 24 Comparison between experimental data and numerical ones

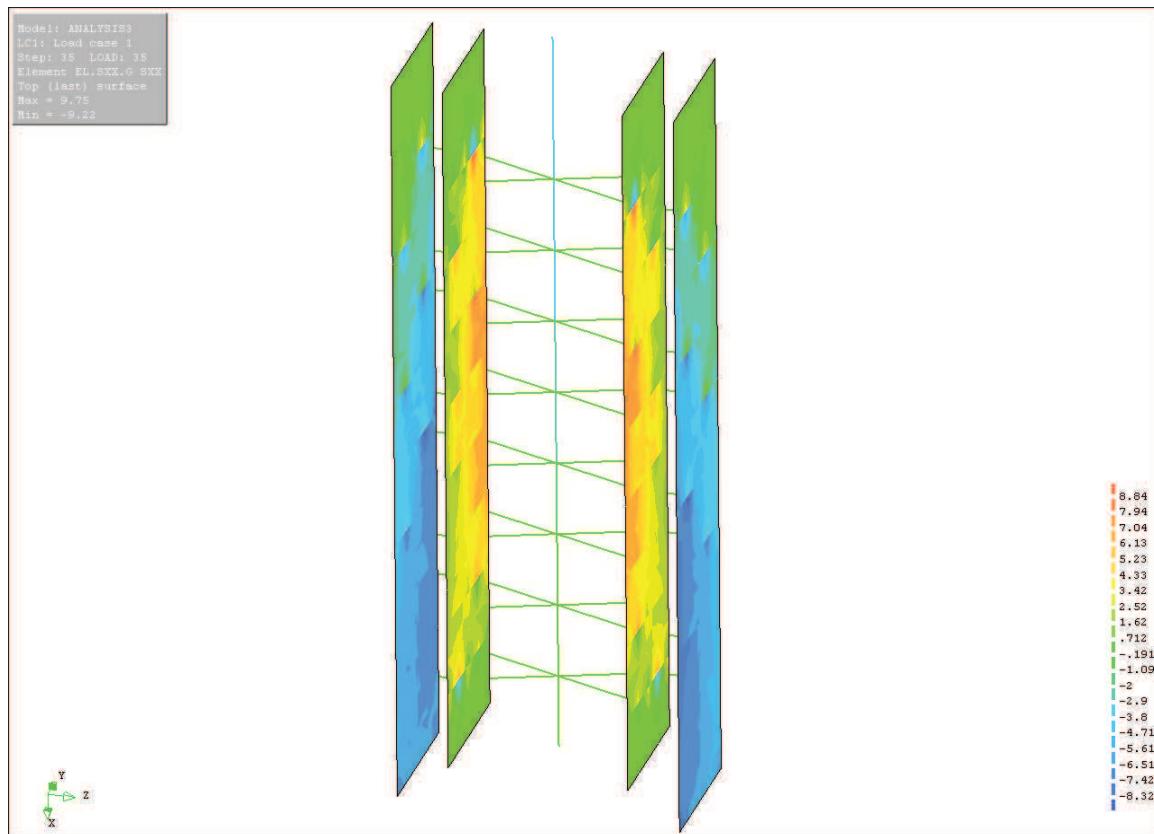


Fig. 25 Distribution of σ_{xx} stresses in the model

9. MODELLING PROCESS: FEM ANALYSIS OF TIMBER FLOORS

9.1. Introduction and previous studies

This part of the thesis is based on the evaluation of timber floors comparing the already taken results with the newest ones.

The previous studies were developed and deep studied taking into consideration the experimental results coming from laboratory tests and the finite element analysis of the samples.

Finite element models were defined by considering as in this case a simplified approach in terms of geometries. The beams were modelled as linear axial beams and the wooden boards as plates. In order to simulate a non-linear behaviour of the interfaces between the boards *Connectors* elements were used. Those elements were distributed every 10 cm and 50 cm in order to reach the proper stiffness defined during the laboratory tests. By applying those elements a friction behaviour was attached but it did not give the correct behaviour of the system. The problems were related to the punctual distribution of the *Connectors* compared to the real situation.

Considering this thesis, some improvements to the models have been developed and applied. From the geometrical point of view the simplifications were the assumption of linear elastic beam with a rectangular predefined shape section. Curved shell elements have been adopted simulating the wooden boards. Rigid connectors as nails or screws were modelled. Punctual non-linear interfaces were attached to the nails, simulating their behaviour. Along the boards, linear interfaces have been introduced. Considering that the wooden deck is constituted by wooden boards with rabbet joints along their length, those interfaces presented a frictional Coulomb behaviour. This mathematical approach allows a more accurate interaction and response between the plates than the one provided by *Connectors*.

Acting in this way the analyzed diaphragms are more able to represent the proper behaviour of the components of the floors and of the floor as whole.

With regard to the finite elements considered, same types of elements of the push-out analysis have been used. A two-node, three dimensional beam was used for the timber joists and the steel nails. As for the boards, eight-node curved shell elements were used. Non-linear punctual interfaces have been applied between the nails and the timber boards. In addition the

boards were characterized by a linear interface to which friction laws were attached. The following table resumes the used elements.

	Beam	Plates	Node to node contact	Board to board contact
Element type	LI13BE	CQ48S	N6IF	CL24I
N° nodes	2	8	2	3

Table 8 Typologies and number of nodes of the selected finite elements for the timber diaphragms

The considered floors present as a strengthening condition just the variation of the distance between nails and the addition of more nails connecting the boards to the joists. The possible solutions are the following:

- The nails are placed along the diagonal of a rectangle 60 mm large and 10 mm width.
- The nails are placed along the diagonal of a rectangle 60 mm large and 30 mm width.
- The nails are placed along the diagonal of a rectangle 60 mm large and 50 mm width.

Each of those possible solutions is evaluated firstly without the application of a frictional Coulomb behaviour between the boards and later by applying it.

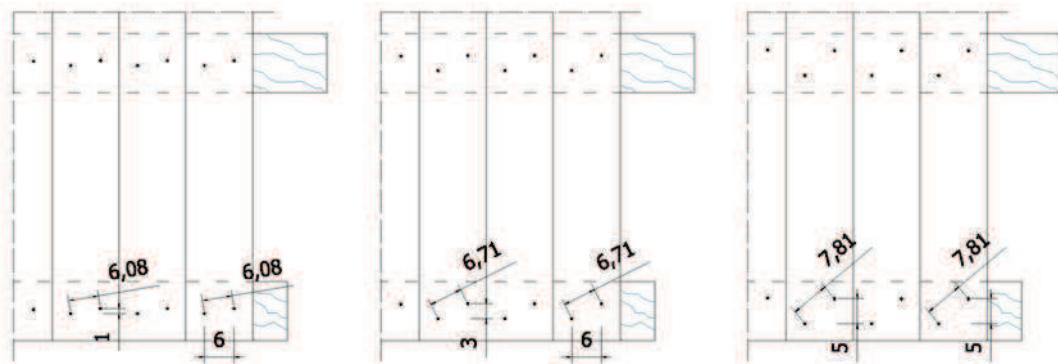


Fig. 26 Disposition of the nails on the boards: first on the left 60×10 mm disposition, on the right 60×30 mm disposition, the last one 60×50 mm disposition

In addition to these possibilities another evaluation can be performed by considering the double of the previous nails. Acting in this way on all the corners of the rectangles previously mentioned, nails are applied. Even in this case the evaluation can be done by considering both solutions, without and with the frictional Coulomb behaviour.

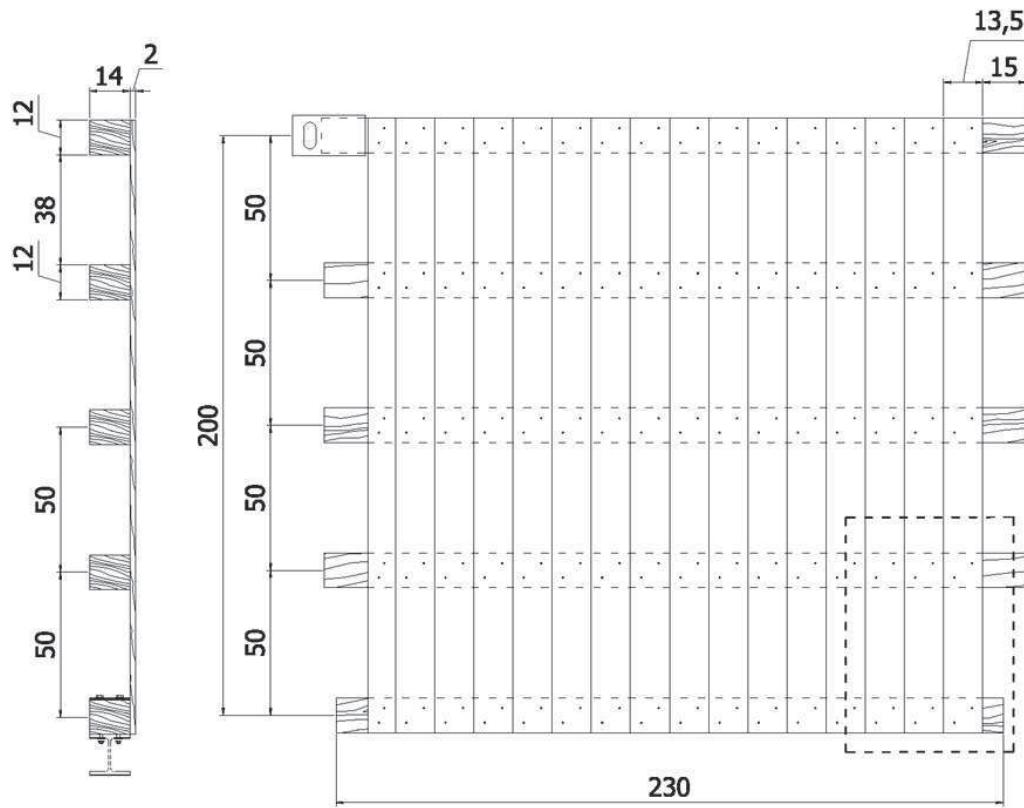


Fig. 27 Sample of a timber floor with a 60×50 mm nail distribution and rabbet joints

9.2. Characters of the analysis: Parameters

All the analysis herein explained have been performed by considering a structural non-linear type of analysis under displacement control. The calculations were done by considering a user's specified load step size of 0,25 for 320 times. As for the equilibrium iteration process a maximum of 400 iterations was set. The chosen method has been a Secant (Quasi-Newton) with a default type of iteration BFGS. The line search algorithm was set in order to stabilize the convergence. Line search is able to increase the convergence speed. As for the convergence criteria, displacement and force criteria were imposed instead of the energy based one.

Set	Characteristics
Method	Secant (Quasi-Newton) - BFGS
Load steps	0,25 (320)
Max. number of iterations	400
Line search	Yes
Displacement Criteria	Tolerance 0,1

Force Criteria	Tolerance 0,01
Solution of system of equations	Sparse Cholesy with a tolerance of 1e-08

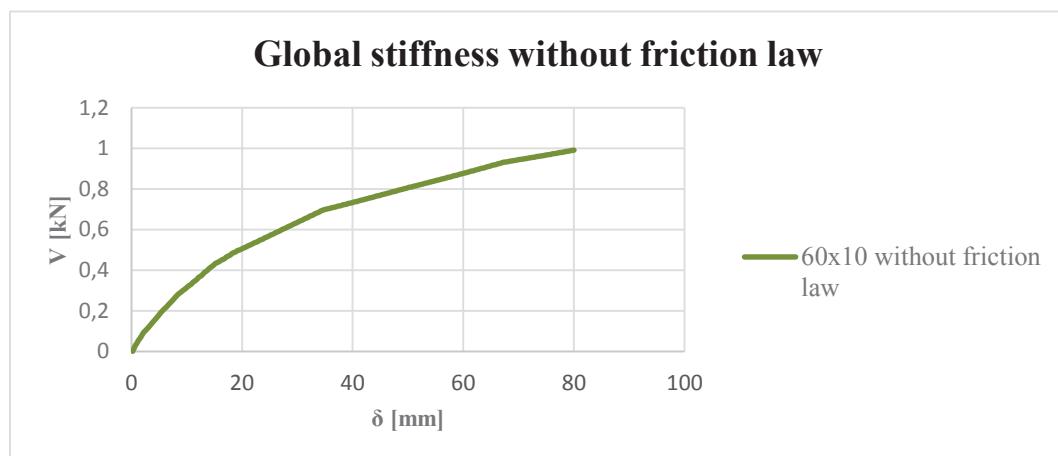
Table 9 Characteristics of the analysis

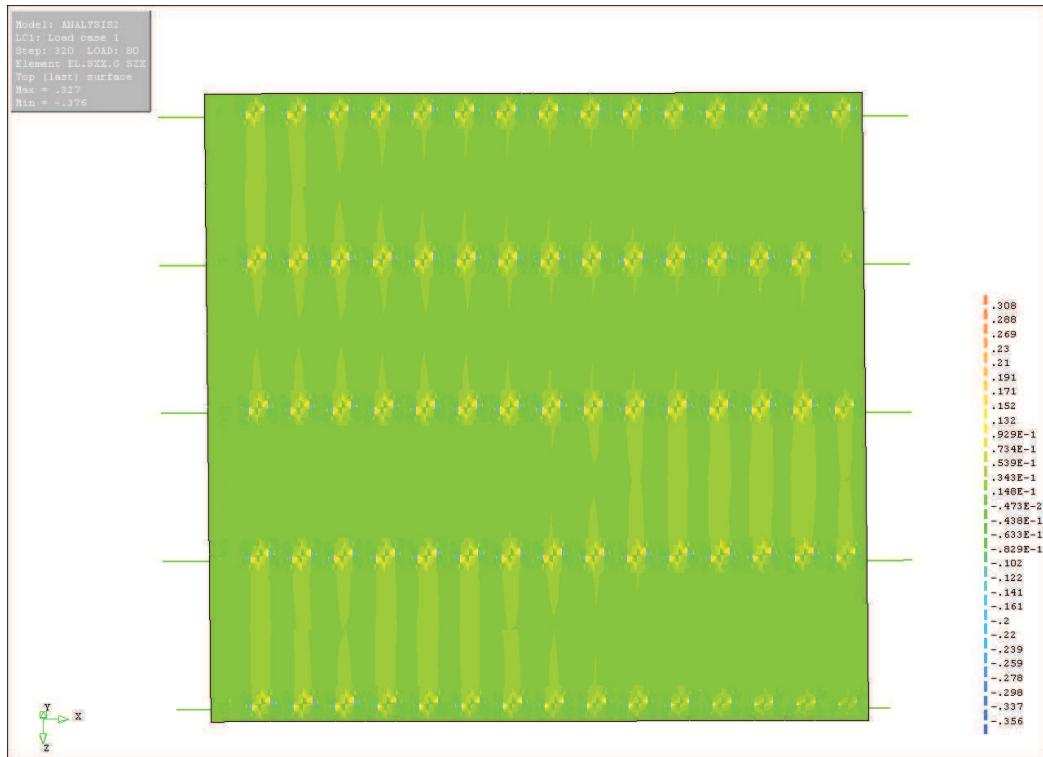
As for the criteria adopted, the defined tolerance of the systems were low due to several problems related with the integration procedure during the calculations.

9.3. Timber floor–FM 60×10 mm

9.3.1. FM 60×10 mm without frictional behaviour

The model presents the simplest connectors (nails) placed on the corners of the diagonal of the rectangle 60 mm large and 10 mm width. The first model was calculated without considering the frictional Coulomb behaviour. In this first application just the bond-slip behaviour is attached to the punctual interfaces of the nails.

**Fig. 28 Global stiffness of the timber diaphragm without Coulomb friction law**

Fig. 29 Distribution of σ_{zx} stresses in the model

9.3.2. FM 60×10 mm with frictional behaviour

The herein presented study followed the previous configuration but in addition the frictional behaviour is attached. The Coulomb friction behaviour is applied to the linear interfaces between the wooden boards. DIANA provides a wide range of properties that can be linked to the general Coulomb friction law but in this particular case the simplified approach was chosen. The defined parameters for the frictional behaviour were the cohesion of the material that it was imposed equal to zero. In this way a frictional law without cohesion was evaluated. Regarding the tangent of the friction angle ϕ a value of 0,1 was applied. The dilatancy property expressed by the dilatancy angle was not considered and it was imposed equal to zero. Both normal and shear stiffness of the linear interface were considered equal to 1 N/mm³. The absence of cohesion physically means that the wooden boards even if are joined along their length, the movement normal to the interface is easy to reach. The tangent of the friction angle demonstrates the adherence between the boards and the slip of the boards.

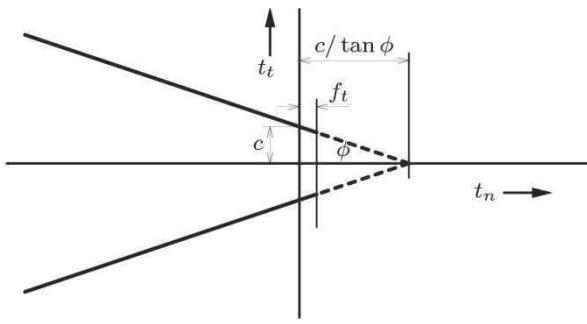


Fig. 30 Coulomb friction behaviour (Manie 2014)

Considering the following graph it is clear that just applying the Coulomb law the floor reacts in a better way. The behaviour is better and the non linearity is reached at an higher level of load.

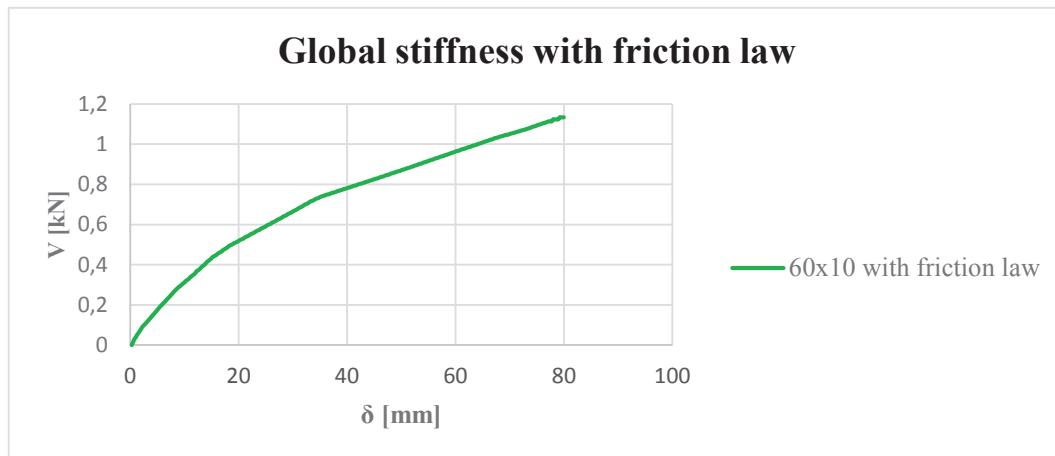


Fig. 31 Global stiffness of the timber diaphragm with Coulomb friction law

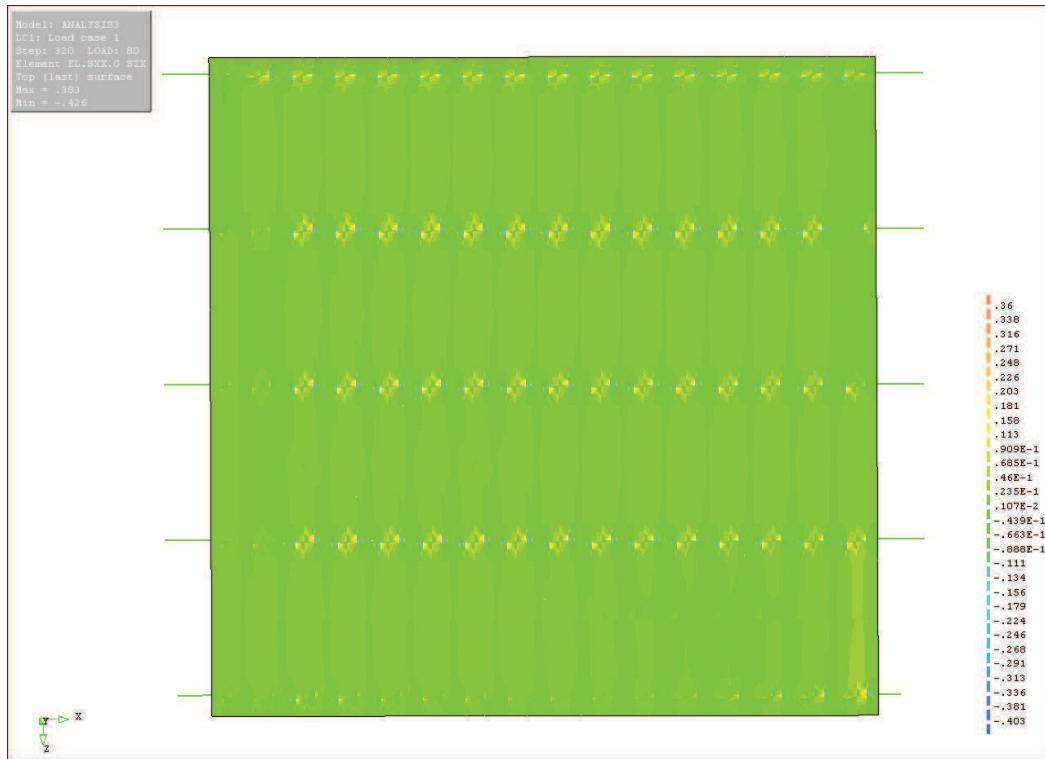


Fig. 32 Distribution of σ_{xx} stresses in the model considering the presence of the Coulomb friction law
The difference between the two graphs is due to the added friction.

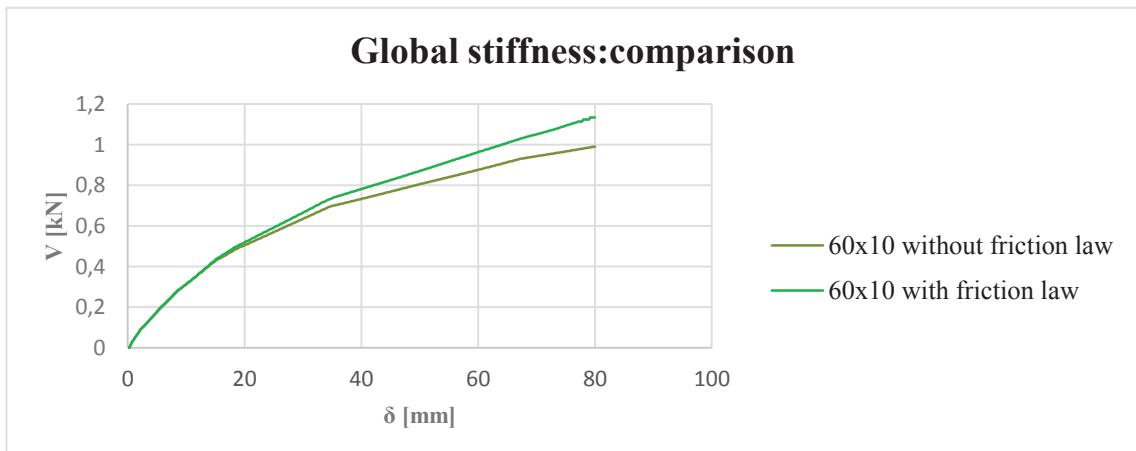


Fig. 33 Comparison between the two diaphragms in terms of stiffness

Considering the last graph it is clear that the presence of a friction law along the boards provides an improvement of the global behaviour of the floor. The influence of the distance between the nails is relevant and the contribution to the general improvement of the in-plane stiffness provided by the friction increase a lot the response.

9.4. Timber floor–FM 60×30 mm

9.4.1. FM 60×30 mm without frictional behaviour

Considering this new model the only difference compared with the previous case was the different position of the nails. The nails are applied following a rectangle shape 60 mm long and 30 mm width as shown in Fig.26. This application allowed to estimate how the distance between nails influences the overall behaviour of the system.

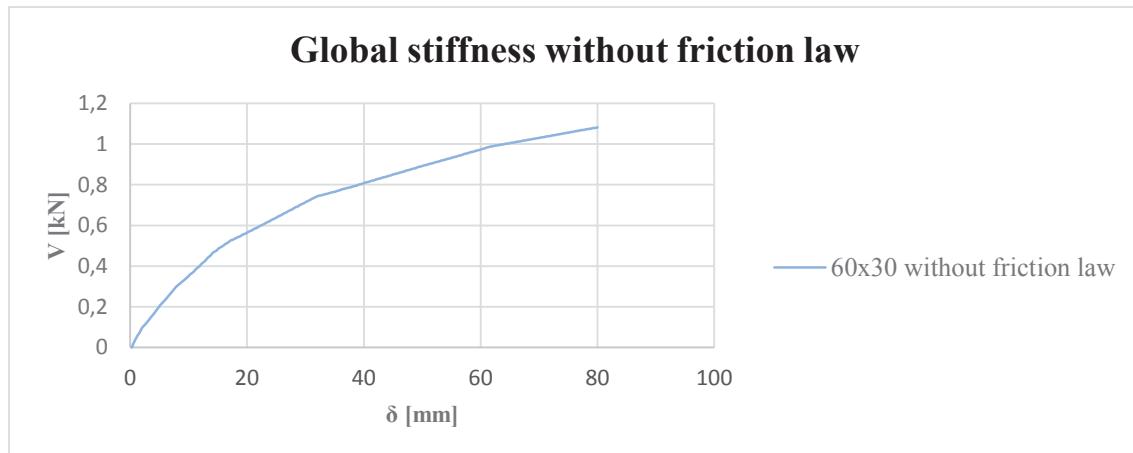


Fig. 34 Global stiffness of the timber diaphragm without Coulomb friction law

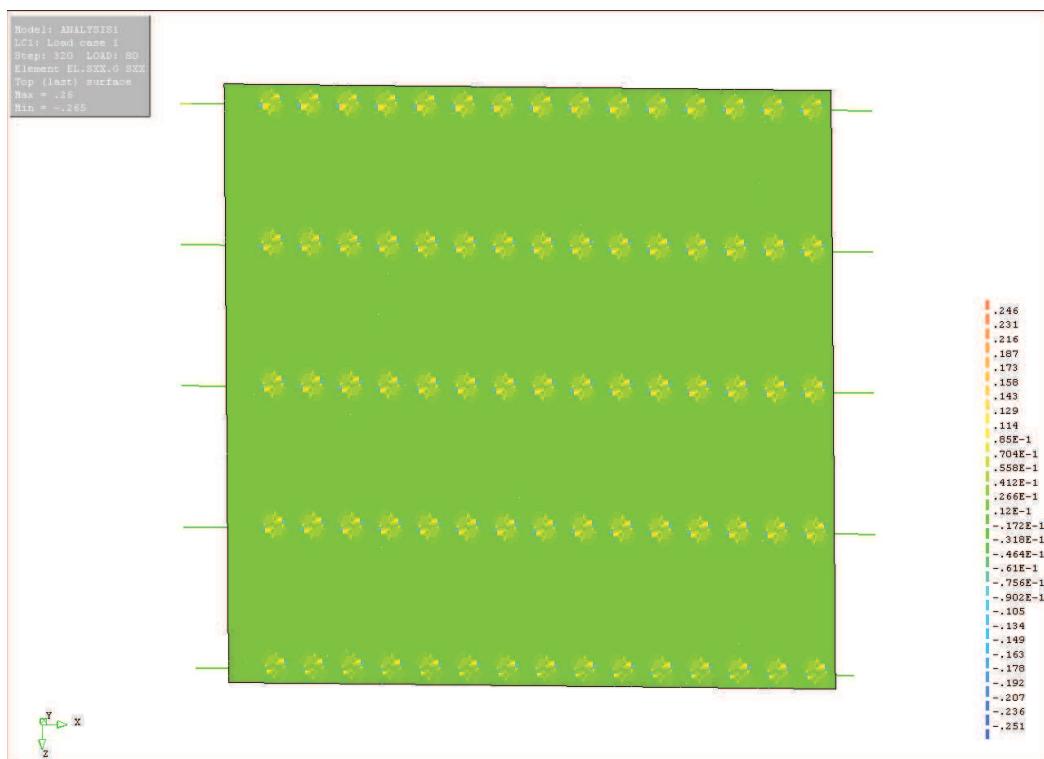


Fig. 35 Distribution of σ_{xx} stresses in the model

9.4.2. FM 60×30 mm with frictional behaviour

Even in this case as in the previous one there was the application of the frictional behaviour along the interfaces of the boards. It is easy to define also in this case an improvement of the stiffness of the floor. The Coulomb's parameter attached to the interfaces are the same considered before.

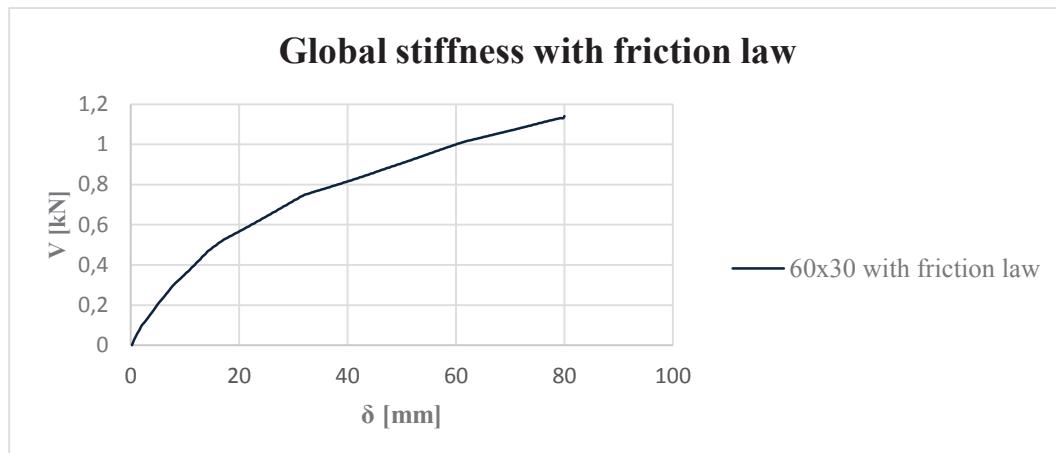


Fig. 36 Global stiffness of the timber diaphragm with Coulomb friction law

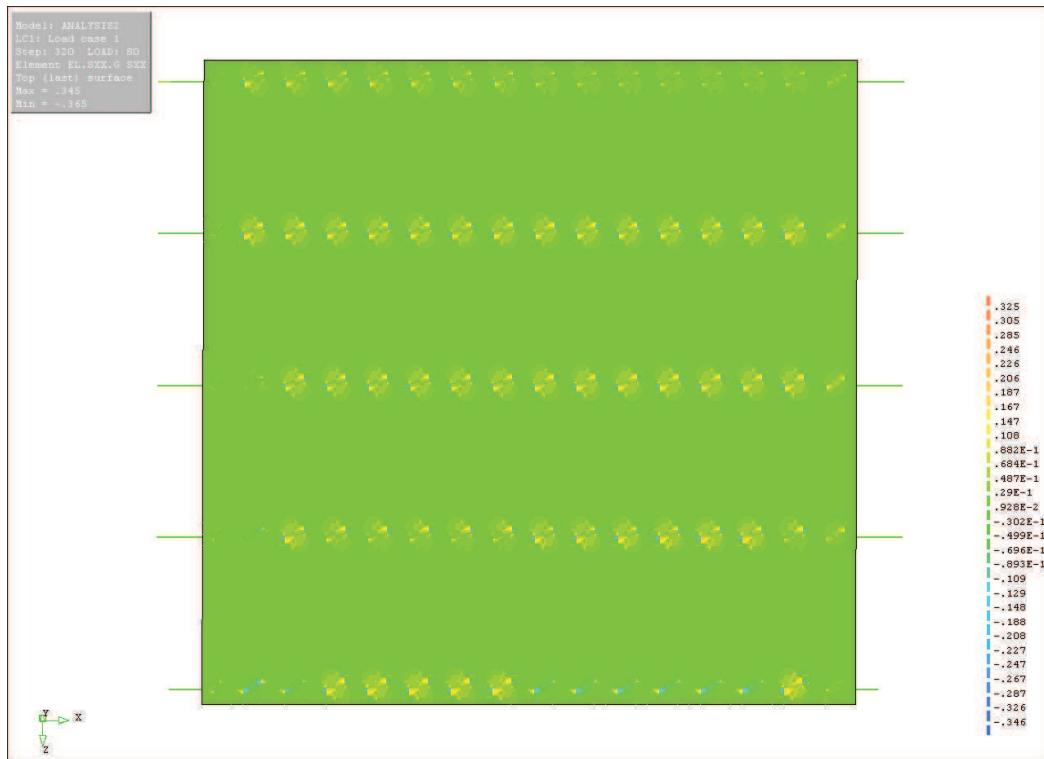


Fig. 37 Distribution of σ_{xx} stresses in the model considering the presence of Coulomb friction law

Considering both the previous graphs their difference is due to the friction between the boards. Herein the graph is presented.

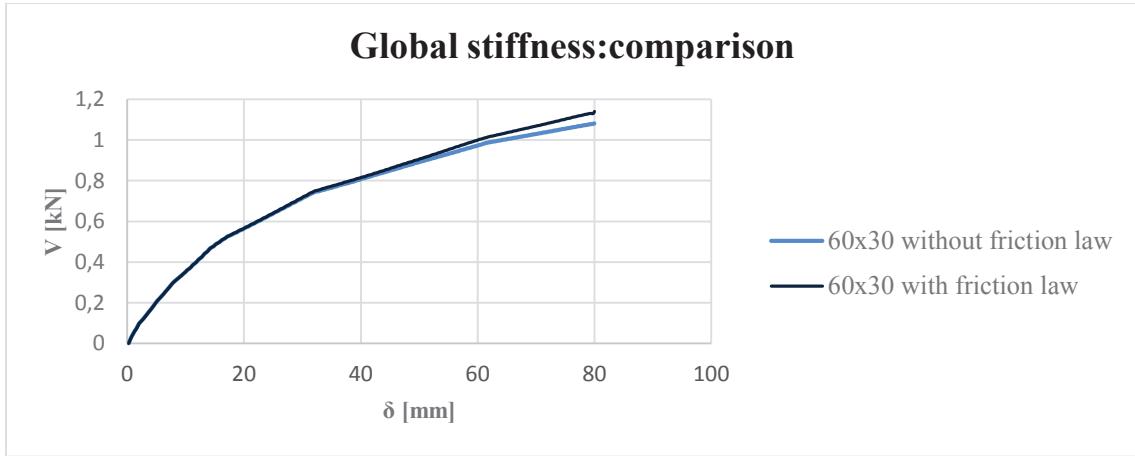


Fig. 38 Comparison between the two diaphragms in terms of stiffness

Even in this case as in the previous one, the addition of a friction law to the boards enhance the system. Considering that the distance between the nails in the connection is higher than the previous one, the general increase of the stiffness is not so consistent.

9.5. Timber floor–FM 60×50 mm

Here the floor is characterized by the presence of nails distributed on a rectangular shape 60 mm long and 50 mm width. As in the previous cases the comparison between the floor without and with frictional law is observed.

9.5.1. FM 60×50 mm without frictional behaviour

The following graph shows the global stiffness reached by the floor considering no frictional law and the nails placed as aforementioned. The comparison presents the difference between the previous numerical solutions and the newest ones. Even if the new curve is following the previous one, there is a significant difference. It seems that the non-linearity is reached later in the newer model than the past one.

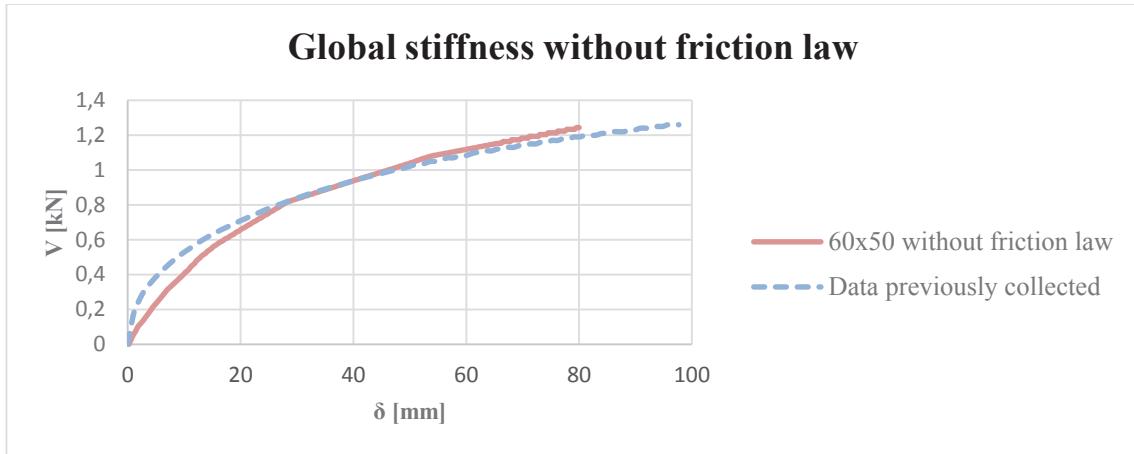


Fig. 39 Global stiffness of the timber diaphragm without Coulomb friction law compared with previous data

Considering the analytical approach proposed by Brignola and Gattesco for determining the stiffness of a non-strengthened floor the results are herein presented. The case studied is the one having the shear force parallel to the joists as shown in Chapter 5.

ROTATIONAL STIFFNESS OF A SINGLE NAIL		
Diameter	φ [mm]	2,57
Density of timber	ρ_m [kg/m ³]	350
Rotational stiffness of single nail	k_n [N/mm]	500
ROTATIONAL STIFFNESS OF A COUPLE OF NAILS		
Distance between nails	b [mm]	60
Width between nails	w [mm]	50
Distance between nails	a [mm]	78,10
Rotational stiffness of a couple of nails	k_ϕ [N/mm]	1525000
GEOMETRY OF THE SYSTEM		
Number of boards	n_b	16
Number of joists	n_j	5
Floor dimension along direction of boards	L_1 [mm]	2000
Floor dimension along direction of joists	L_2 [mm]	2000
Boards' wheelbase	i [mm]	135
Joists' wheelbase	l [mm]	500
IN-PLANE STIFFNESS OF NON-STRENGTHENED FLOOR		
In-plane stiffness	K_{un} [N/mm]	30,5

According to the previous graph the stiffness of the floor is around 30 N/mm even if there is a slight error between the numerical curve (red one) and the experimental one (blue dotted curve).

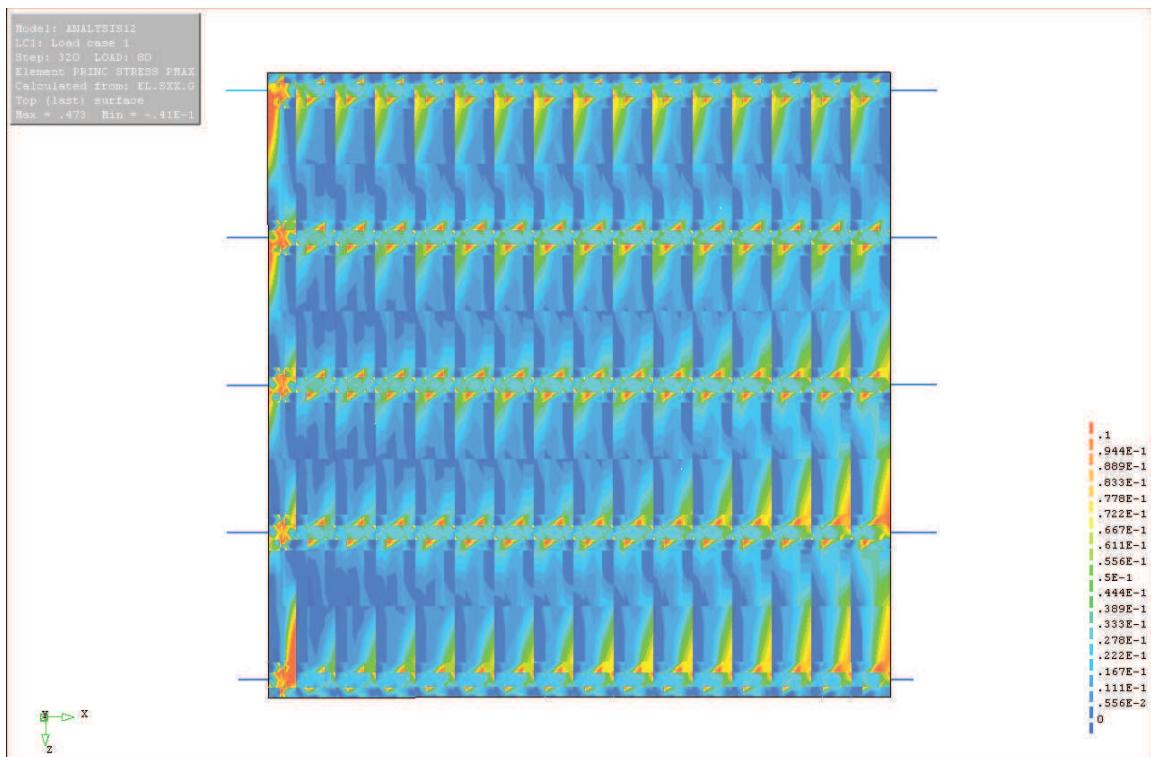


Fig. 40 Distribution of σ_x principal stresses in the model

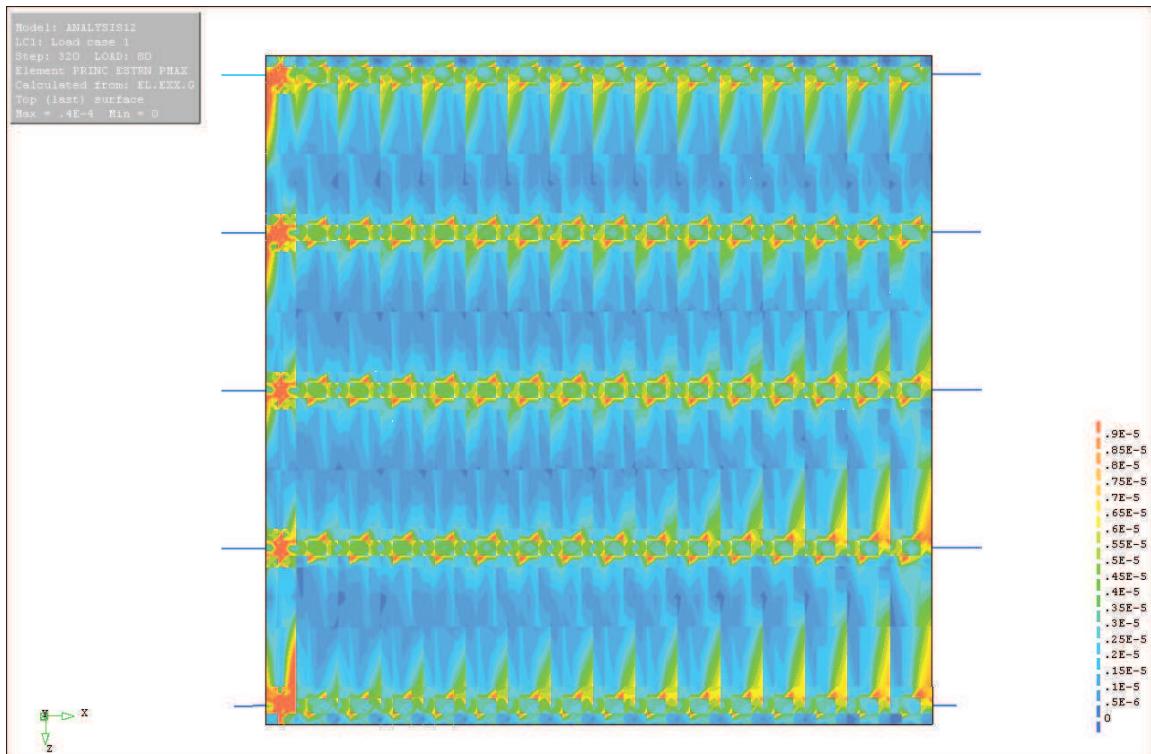


Fig. 41 Displacement distribution along x of the model

It is clear that concentration of forces around the nails are very high. The use of low tolerance influenced a lot the results. During the integrations negative and positive values

around these zones defined a not so smooth surface of results. Due to this there are several peaks concentrated in the nails.

9.5.2. FM 60×50 mm with frictional behaviour

The following graph presents the global stiffness of the floor considering the application of frictional behaviour. It can be seen that the general behaviour is improving just considering the addition of a limited friction.

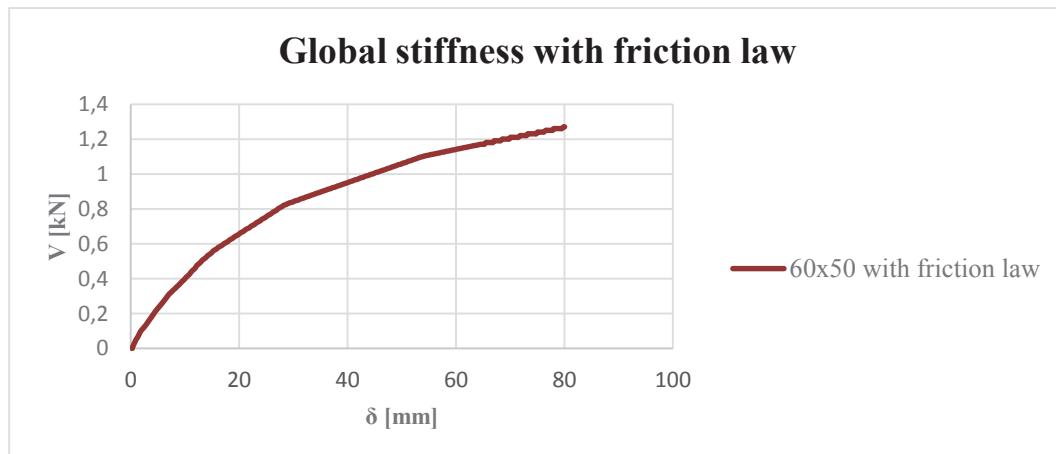


Fig. 42 Global stiffness of the timber diaphragm with Coulomb friction law

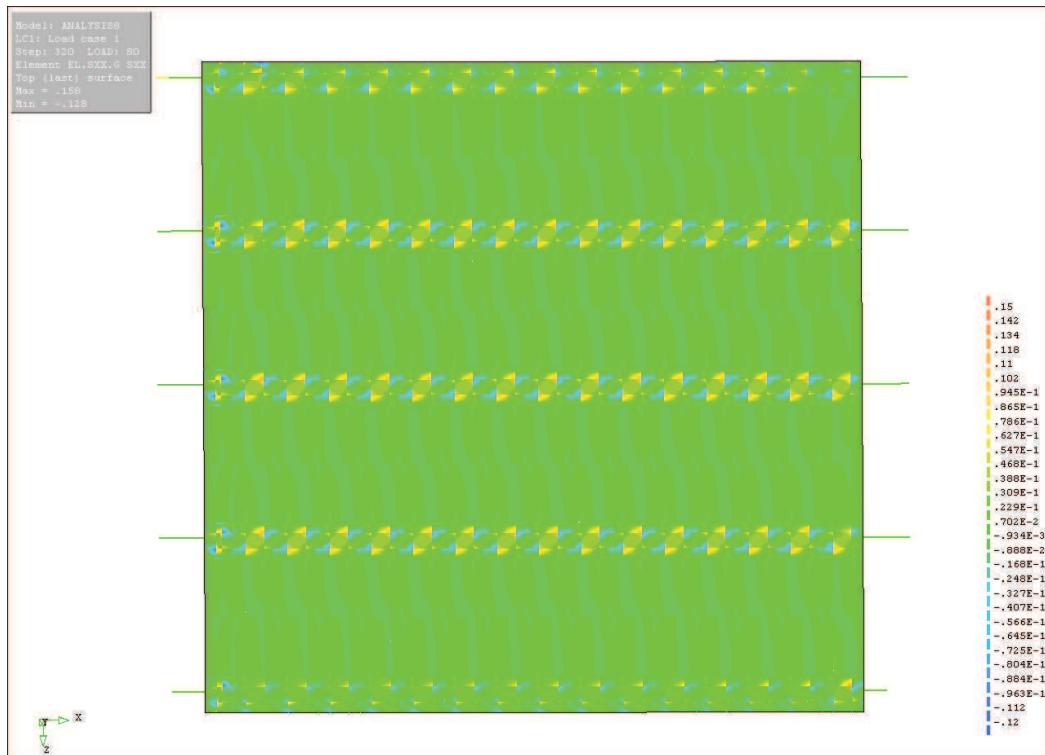


Fig. 43 Distribution of σ_{xx} stresses in the model considering the presence of Coulomb friction law

Considering a comparison between the two graphs, it is possible to check the difference between the two systems.

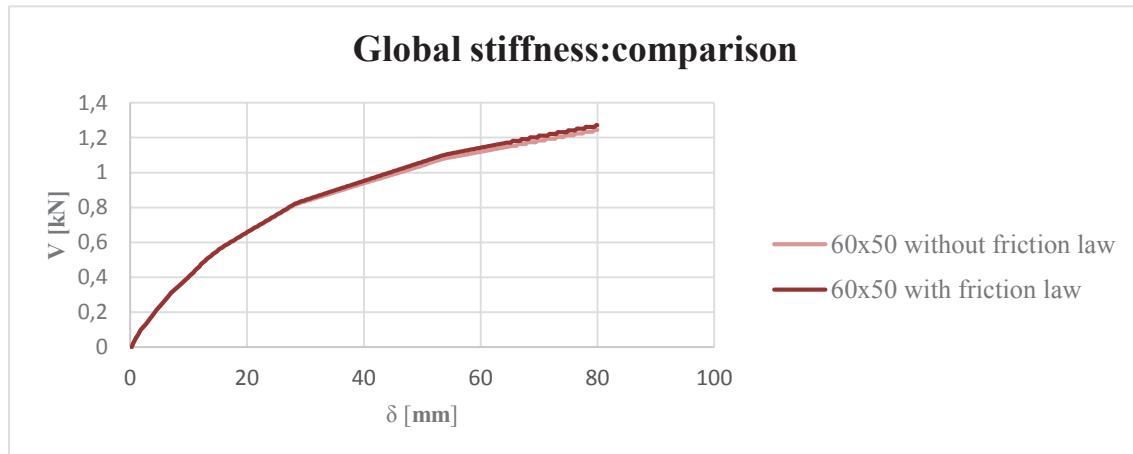


Fig. 44 Comparison between the two diaphragms in terms of stiffness

From the last graph it is clear that the presence of a friction law between the wooden boards does not improve the response of the system. Globally the in-plane stiffness of the diaphragm is much more influenced by the distance between the nails than considering the friction law.

9.6. Observations

The models showed an overall improvement of their in-plane stiffness passing from the most simple configuration up to the more complicated. Observing the general enhance of the stiffness passing from the 60×10 mm configuration up to the 60×50 mm, it can be said that a relevant effect on the in-plane stiffness is due to the distance between the nails especially considering the configurations 60×10 mm and 60×30 mm.

In addition it has been confirmed that the presence of inter board friction advances the global stiffness. Herein the comparison between graphs is shown.

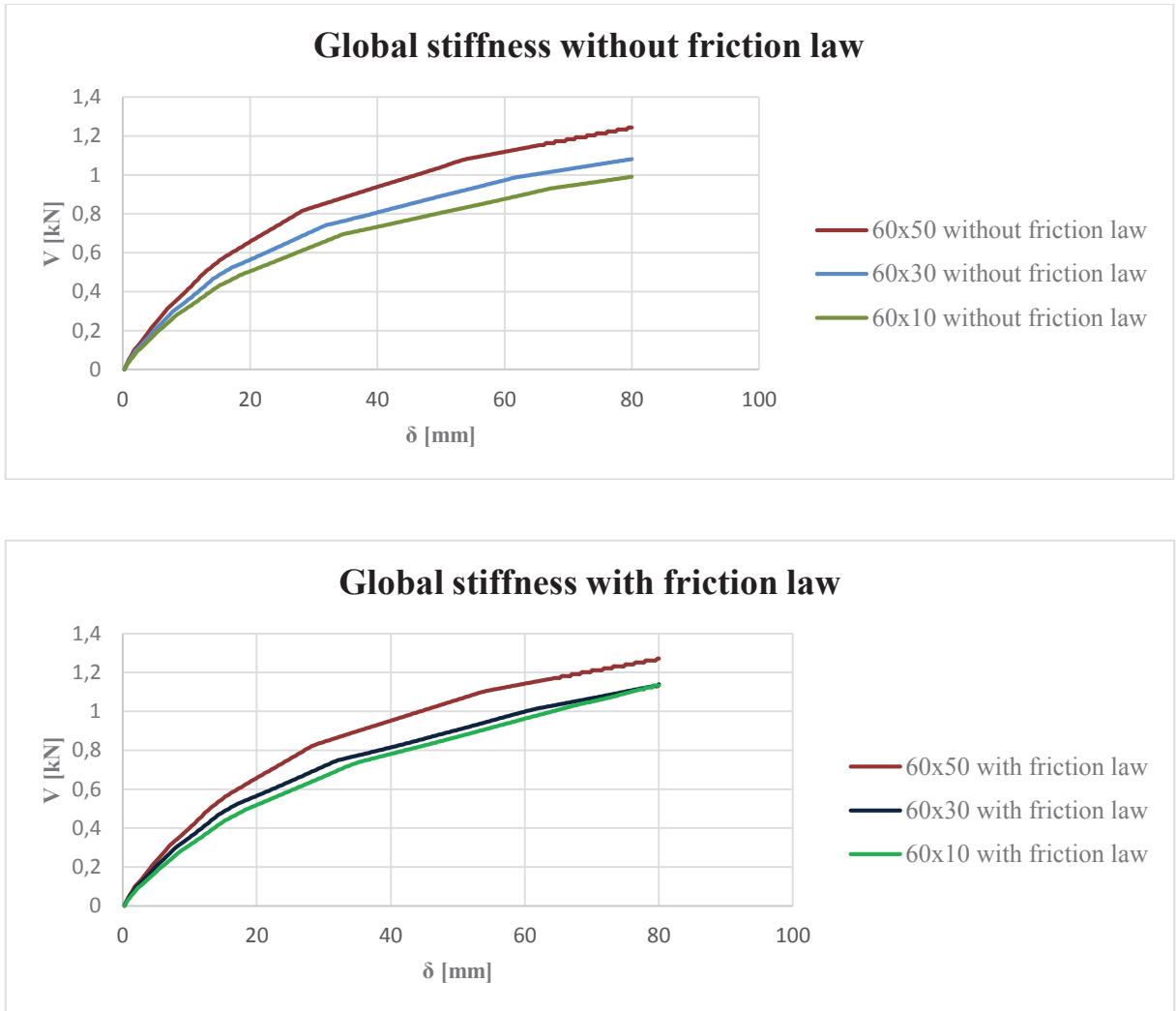


Fig. 45 Upper image is the comparison among the results without friction, the lower one presents the comparison of the results taking into account the friction

9.7. Timber floor–FM 60×10 mm with four nails

The presented work shows the response of the previous samples considering the application of four nails at each board to joist connection instead of two. The geometry of the problem is equal to the previous one. An incremental displacement is applied to the upper joist and the finite element results are collected and elaborated.

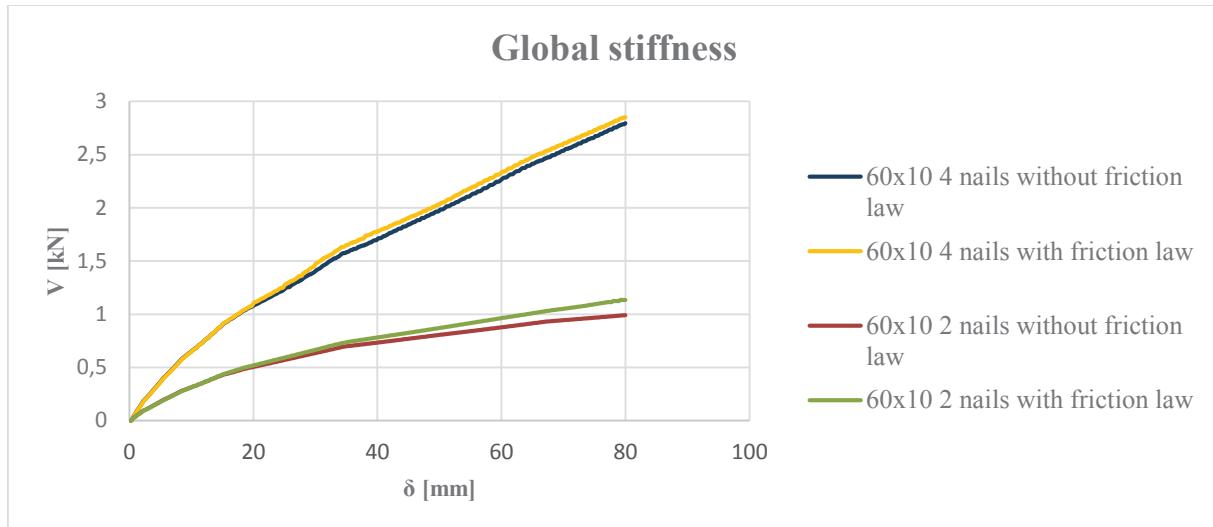


Fig. 46 Comparison between the two diaphragms in terms of stiffness

9.8. Timber floor–FM 60×30 mm with four nails

The following graph is showing the global behaviour of the floor considering the nails placed on the corners of a rectangle 60 mm large and 30 mm width. Herein there is the comparison between the two applications with and without friction law.

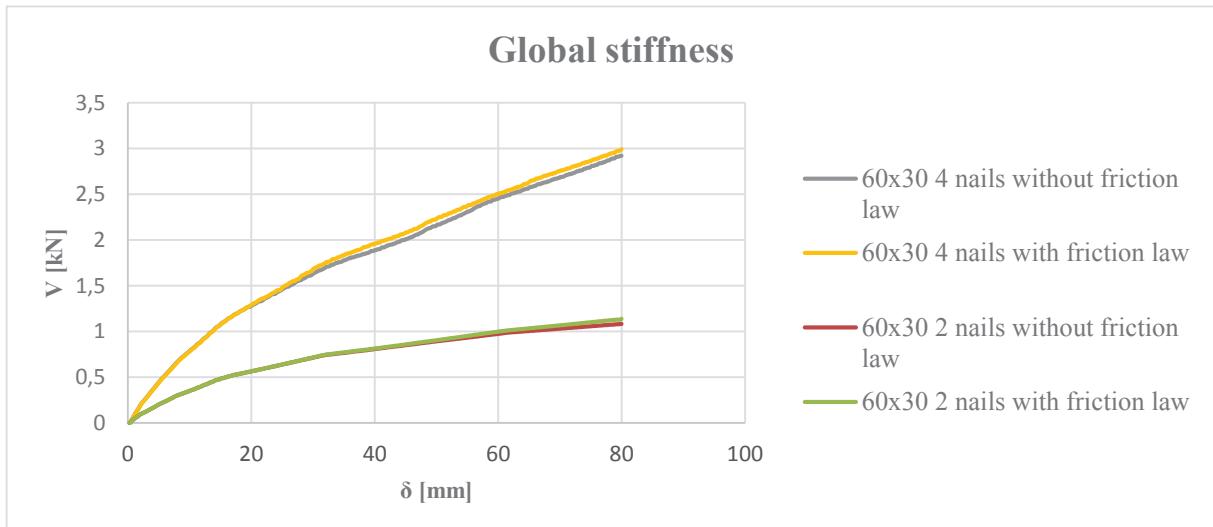


Fig. 47 Comparison between the two diaphragms in terms of stiffness

9.9. Timber floor–FM 60×50 mm with four nails

The cases with the four nails as connection elements ends with their application on the corners of a rectangle 60 mm large and 50 mm width. This is the last case considered as a possible strengthening condition for a timber diaphragm. Even in this case as in the previous the floor is considered both without and with Coulomb friction law.

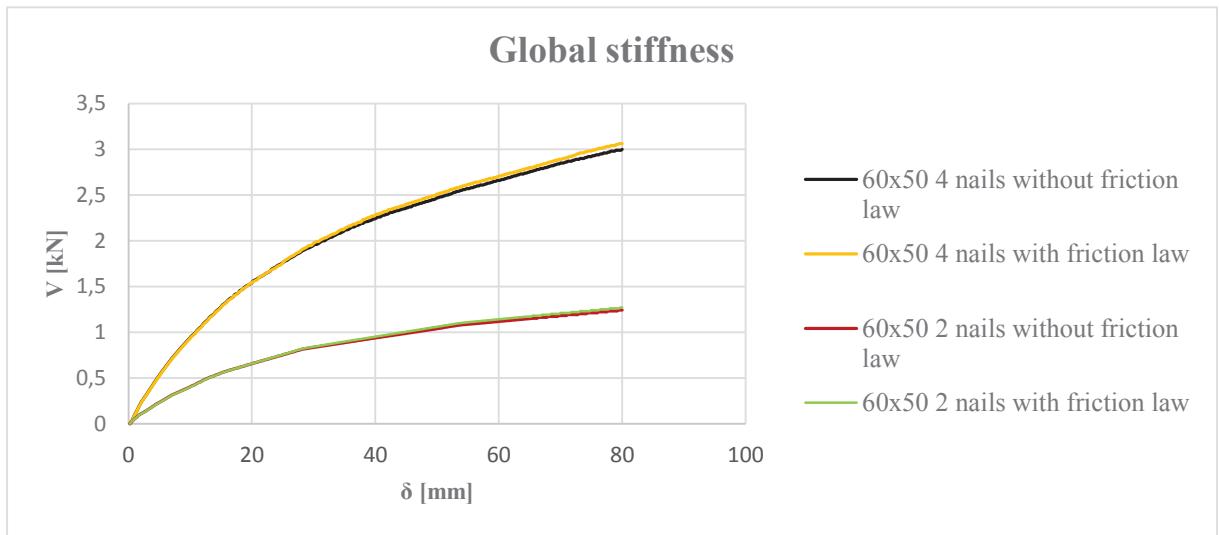


Fig. 48 Comparison between the two diaphragms in terms of stiffness

9.10. Observations

Considering the last evaluations shown by the graphs it is clear that the addition of two more nails per board-to-joist connection significantly increases the overall behaviour of the floor. Even for these cases the application of a friction law to the boards was studied but the results were not similar to the ones obtained for the just two nailed boards. The presence of four nails per each board-to-joist connection determine contribution to the global in-plane stiffness of the floor.

9.11. Application of Coulomb friction law

The behaviour of the timber floor and more precisely of the wooden deck is strongly influenced by the friction. A Mohr-Coulomb friction law has been applied to the boards as previously illustrated. This paragraph wants to focus on one typology of diaphragm among the ones before mentioned, applying several types of friction laws. The selected type of timber floor was 60×50 mm board-to-joist connection with just two nails on the corners of the rectangle (the simplest configuration).

DIANA offers the possibility of setting the cohesion between materials, the tangent of the friction angle ϕ and the dilatancy. As aforementioned the dilatancy was not taken into consideration. The work in this sense was carried out defining increasing values of cohesion. The tangent of the friction angle was maintained at a constant value of 0,1. In addition the Coulomb friction law was cut at certain point, defining the tensile strength f_t . Acting in this

way it was decided not to consider a normal Coulomb law but a limited one (Fig.30). The value of f_t was defined as the cohesion divided for the tangent of the friction angle. Considering this kind of friction law the software found serious difficulties during the integration process giving bad results. Two friction laws with cohesion have been considered in order to check which the program response was. The following table shows the different values of cohesion evaluated and the corresponding tensile strength.

Type of floor	$\tan \phi$	Cohesion	Tensile strength f_t ($0 < f_f < c/\tan \phi$)
FM 60×50	0,1	0,02	0,1
FM 60×50	0,1	0,05	0,3

Table 10 Parameters of friction laws

The results obtained are resumed in the following graph.

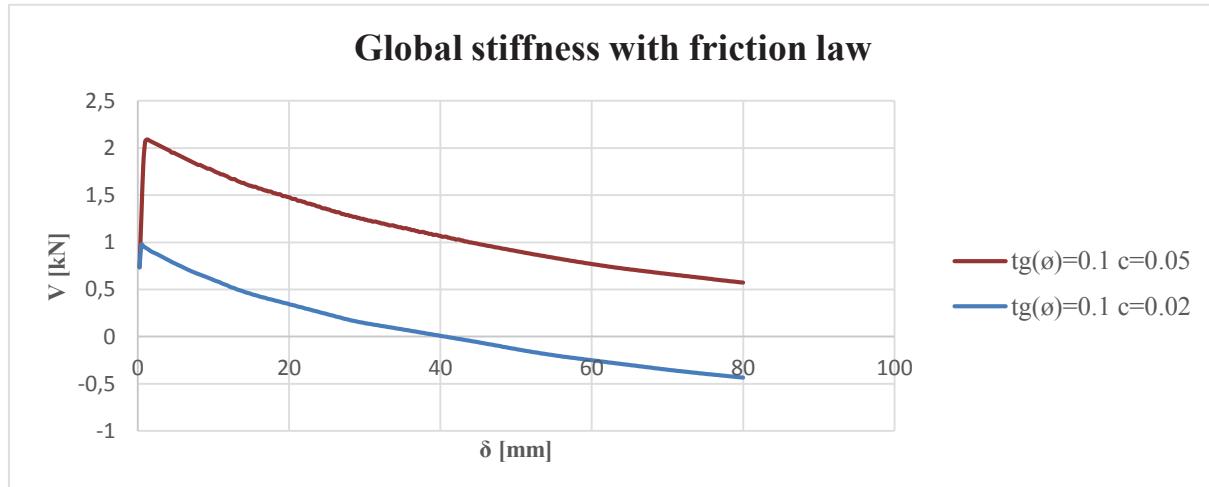


Fig. 49 Behaviour of the floor considering a more complicated friction law with cohesion and a tensile strength

As it is clear from the graph the application of a cohesion to the common friction law involved problems during the calculation.

Starting from this first approach, a simple friction law without cohesion has been evaluated as shown in the following table and graph.

Type of floor	$\tan \phi$
FM 60×50	-
FM 60×50	0,1
FM 60×50	0,2
FM 60×50	0,4

Table 11 Considered friction laws applied to the floorboards

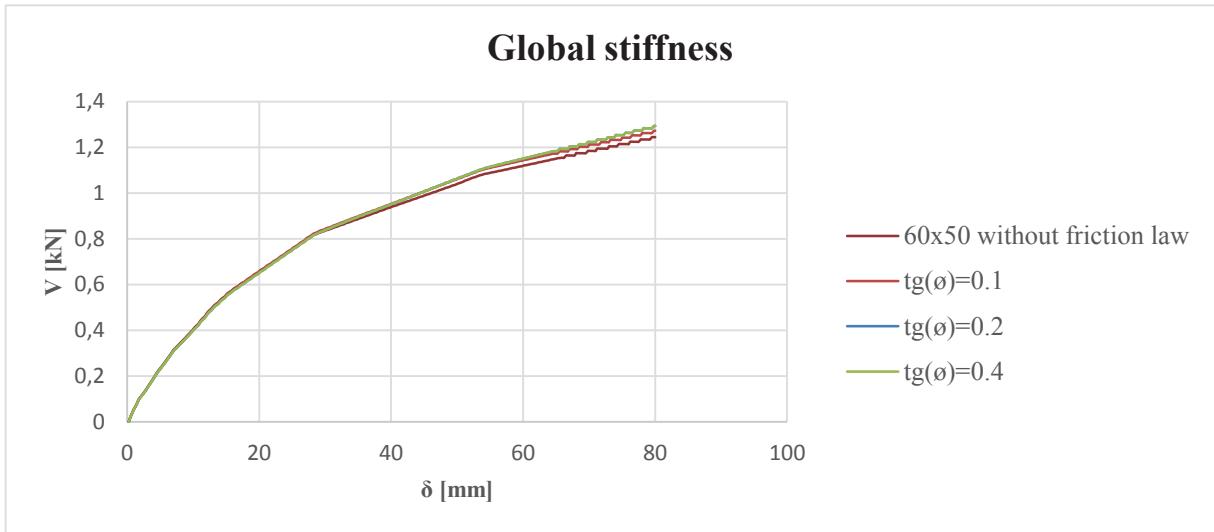


Fig. 50 Global stiffness considering different friction laws

9.11.1. Observations

Evaluating the data provided by numerical models some conclusions can be done regarding the application of friction law. As for the application of a simplified friction law with just the presence of the tangent of the friction angle (ϕ), it is clear that there is a global enhancement of the in-plane floor behaviour. As for the last graph, the curves referred to the tangent of ϕ equal to 0,2 and 0,4 are overlapped. The global behaviour of the system should increase but most probably the high tolerance used for the numerical analysis affected the final result (used tolerance 0,1). As a result of it the global behaviour it is the same.

With regard to the more complicate friction law, with cohesion and tensile strength f_t the problems are most probably relates to the contemporary presence of strong non-linearity of the nails added to the friction law along the boards. Due to these effects the results provided by the calculations cannot be considered reliable.

10.TIMBER FLOOR WITH REAL DIMENSIONS

10.1. First Load Configuration

Once performed the analysis on the full scale samples a possible real case is evaluated. The timber floors considered until now were characterized by limited dimensions due to the laboratory needs.

Considering what was developed at the University of Trento by Professor Piazza working on full-scale timber floors, here the aim is to evaluate numerically the experiment (Piazza, et al. 2008). Full-scale tests were performed on timber floors in strengthened conditions. The aim of his work was to evaluate the response of the floor and the parameters for the calculation of the in-plane stiffness of the diaphragm. The peculiarities of those tests were the boundary conditions. The floor was allowed to have free in-plane deformation. The common conditions of the interaction between floor and masonry walls was considered unpractical and affected by several problems. The applied load was considered uniformly distributed due to the fact that the lateral forces during a seismic event are proportional to the vertical load on the floor. Two external hinges were defined at the neutral axis level in order to guarantee the in-plane deformation (Fig. 51).

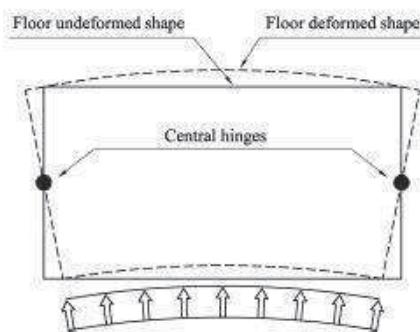


Fig. 51 Configuration used in the laboratory tests at University of Trento (Piazza, et al. 2008)

Considering as a reference these tests, herein is presented a numerical evaluation of a similar configuration. The set up of our case study is slightly different than the one previously presented. Starting from the dimensions the adopted sizes for the specimen were 5 m span and 4 m width. Our case study presents a specimen 3 m span and 6 m width. Both configurations represent ordinary dimensions of timber floors in historical buildings in Italy. The evaluated model is made of timber joists ($0,12 \times 0,14 \times 3$ m) and wooden planks ($0,135 \times 0,02 \times l$ m with l

varying from 2,05 m to 4,05 m). The joists are spaced 50 cm because it is a common configuration for the floor structure in Italian historical buildings. The constraints are applied following a different configuration. Rotation of joists around their main longitudinal axes are not allowed, and the two lateral joists are fixed. The load was applied at 1/4 of the width.

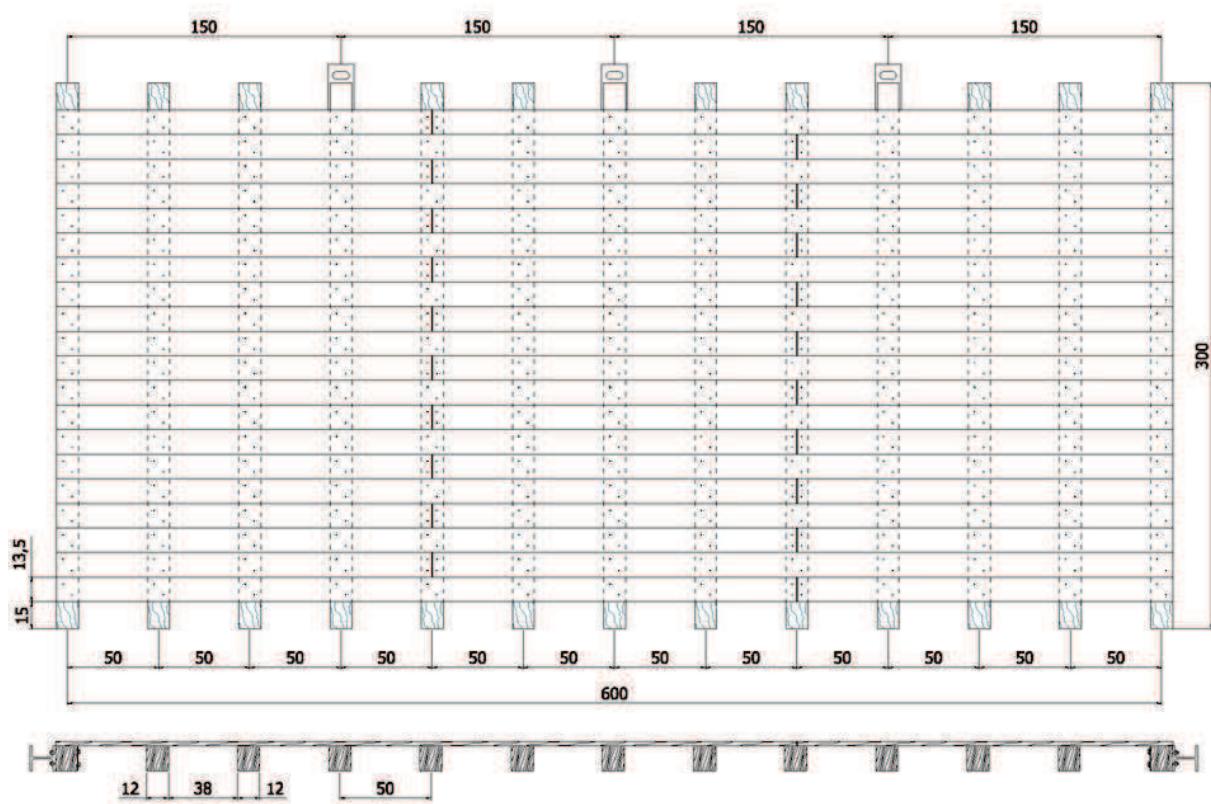


Fig. 52 Configuration of the load applied to the real timber diaphragm

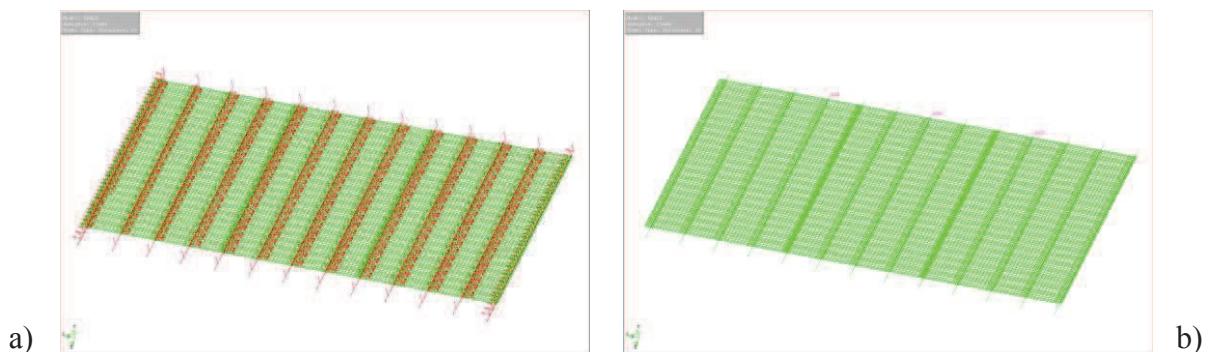


Fig. 53 Model of the real timber floor: a) the constrains of the full-scale timber floor; b) the applied loads

Herein the results of the application of three equal loads to the floor are shown.

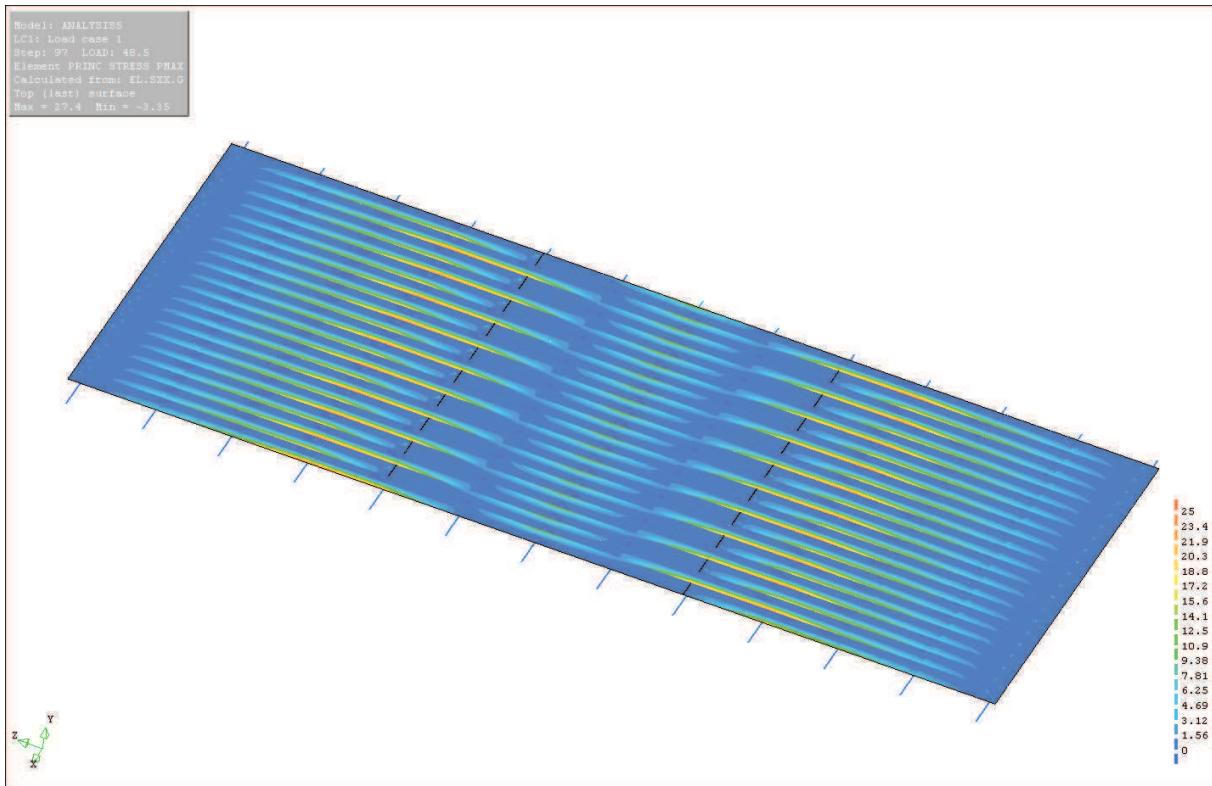


Fig. 54 Displacement distribution along x direction of the model

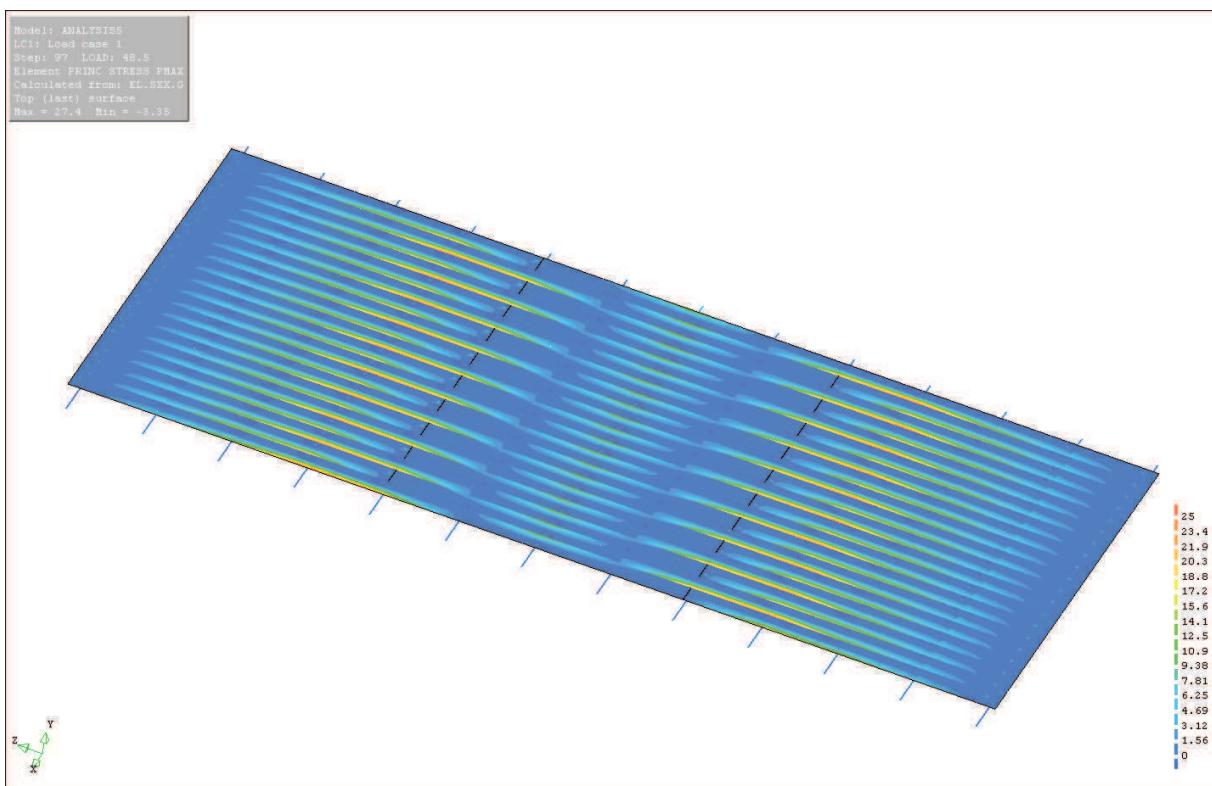


Fig. 55 Principal stress distribution σ_x

From the herein presented images, the distribution of the stresses is highlighting that the floor is subjected much more to a shear force than an in-plane flexural one.

10.2. Second Load Configuration

Considering the results obtained from the previous calculations, another load configuration has been evaluated. According to what American Society of Civil Engineers (ASCE) ((ASCE) 2006) recommends for the Seismic Rehabilitation of Existing Buildings, the load acting onto a timber floor presents a parabolic shape distribution. The in-plane deflection of the timber floor shall be calculated for an in-plane distribution of lateral force consistent with the distribution of mass and the forces have to be applied at the diaphragm level. The horizontal distribution provided by the ASCE is the following and is the one used for determining the acting load.

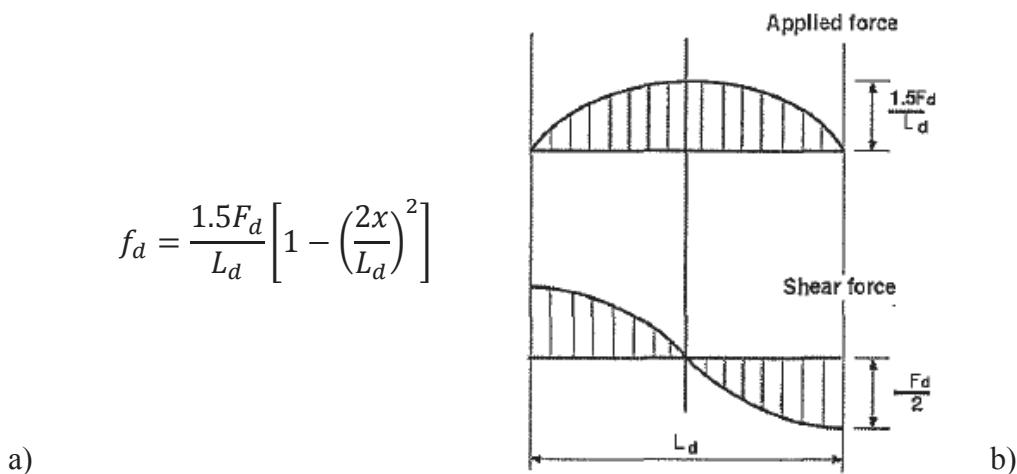


Fig. 56 Suggestions from ASCE: a) load distribution formula; b) load distribution ((ASCE) 2006)

Where:

f_d is inertial load in kN/m

F_d total inertial load on a flexible diaphragm

x is the distance from the centre line of flexible diaphragm

L_d is the distance between lateral support points for diaphragm

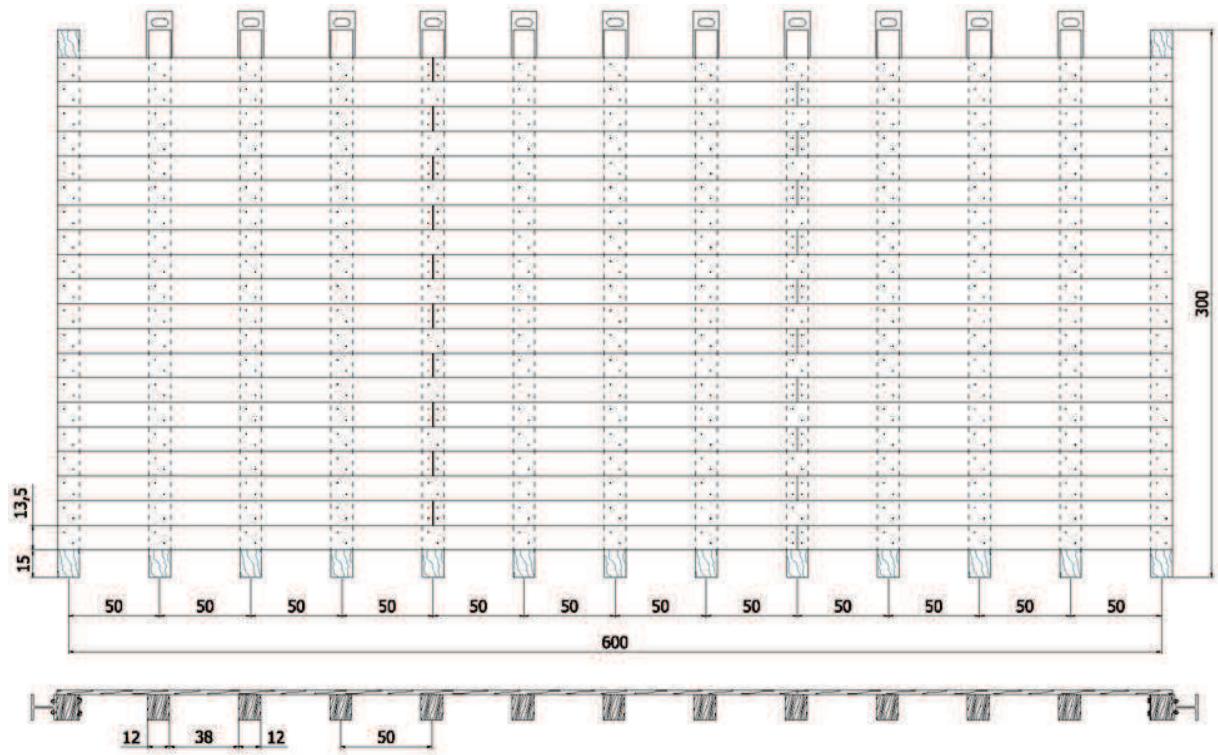


Fig. 57 Configuration of the load applied t the real timber floor system

Herein the loads applied to the heads of the joists are presented considering that all of them were calculated by using the upper formula.

Load	A _i	A _i /A _{tot}	F [N]
V1	0,003	0,005115	0
V2	0,021	0,038293	4,92
V3	0,039	0,069863	16,31
V4	0,052	0,094417	29,76
V5	0,062	0,111956	41,82
V6	0,068	0,122479	50,04
V7	0,070	0,125987	52,95
V8	0,068	0,122479	50,04
V9	0,062	0,111956	41,82
V10	0,052	0,094417	29,76
V11	0,039	0,069863	16,31
V12	0,021	0,038293	4,92
V13	0,003	0,005115	0

Table 12 Load values of the parabolic distribution applied on the head of the timber joists

From the following image it can be seen how the load was considered and applied. In addition the deformed shape of the timber floor is presented.

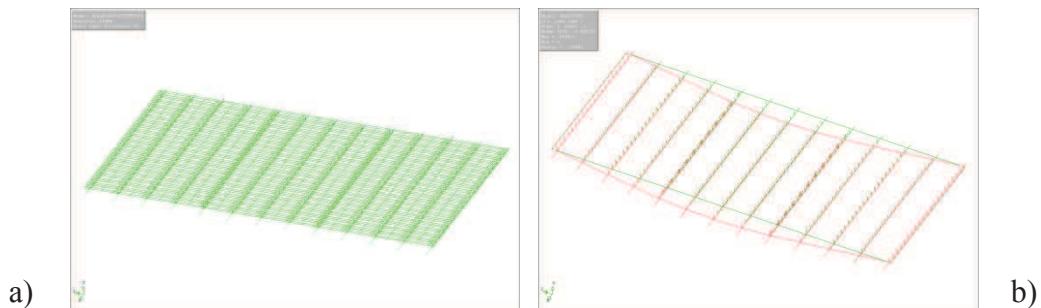


Fig. 58Model of the real timber floor: a) load distribution; b) deformed shape configuration under parabolic load

Herein the results coming from the analysis are shown.

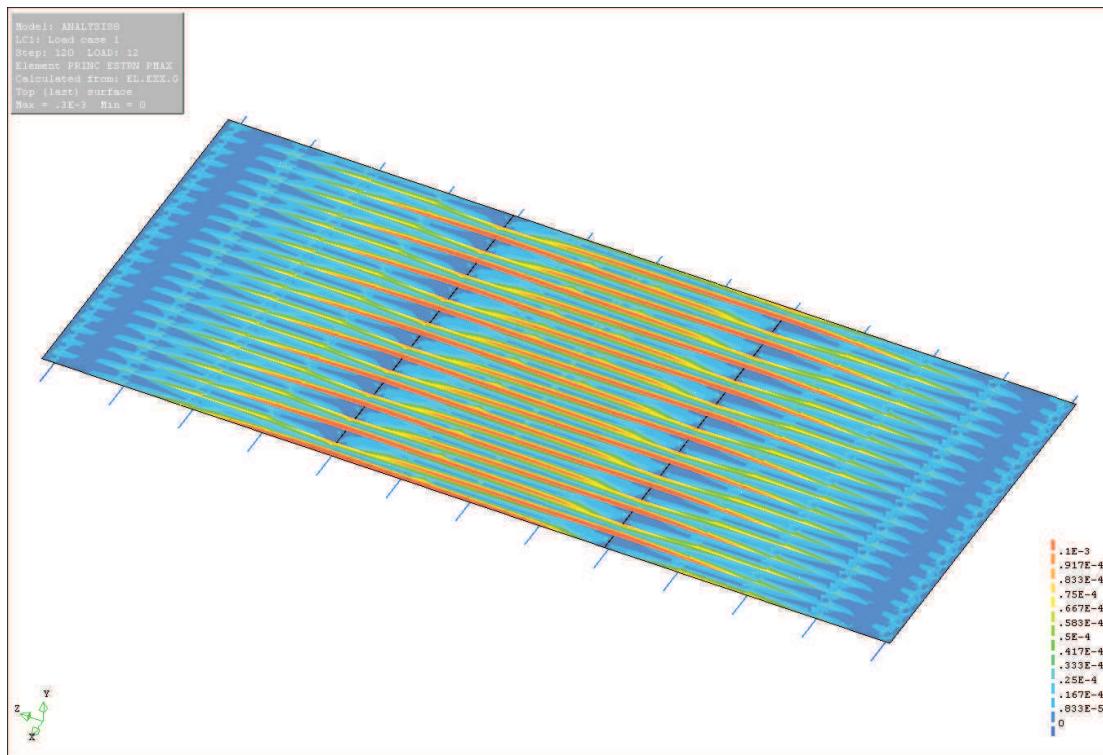
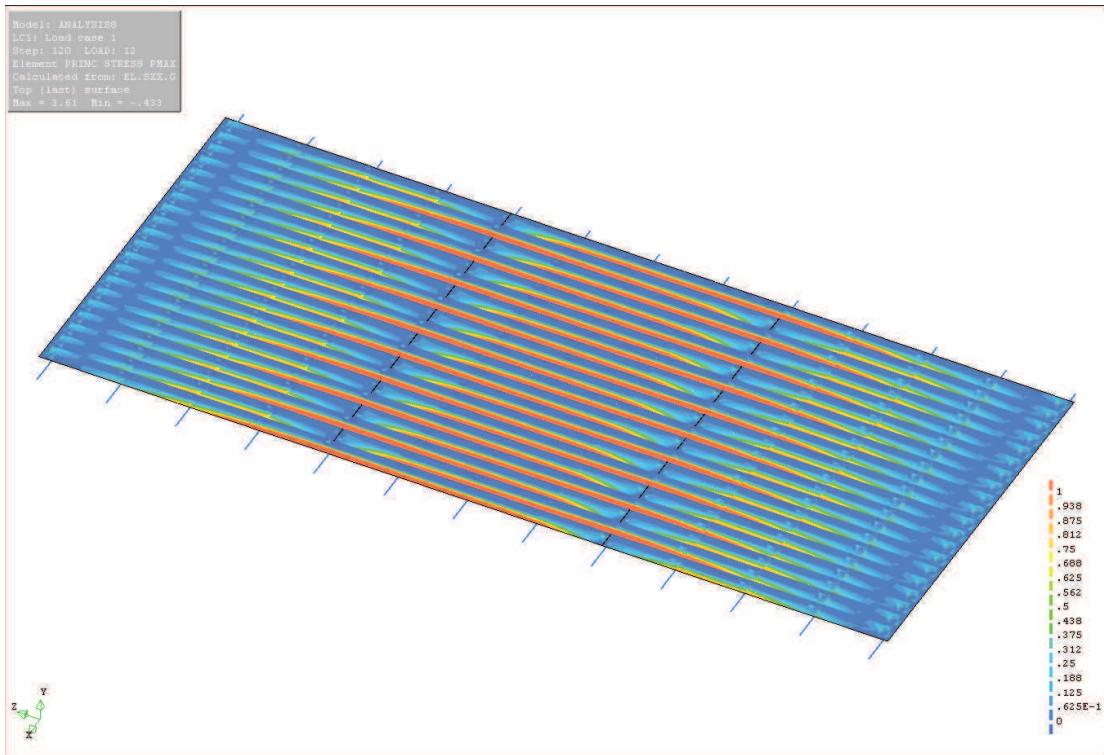


Fig. 59 Displacement distribution along x direction of the model

Fig. 60 Principal stress distribution σ_x

10.3. Observations

Focusing on the results obtained from the first and the second load configurations some considerations can be done. Firstly the two models presented a different deformation shape induced by the load conditions.

Diaphragm stiffness is generally converted to shear stiffness G_d in order to reach results independent from the geometric configuration. Herein some examples from scientific literature are provided in order to compare the results with the one coming from the current model. Dividing the diaphragm stiffness for the board thickness t the shear modulus of the timber system is obtained.

	Sample	G_d [kN/mm]	t [mm]	G [N/mm ²]
1	Simple boards sheathing	0,123	20	6,15
2	Simple boards sheathing	0,290	20	14,5
3	Simple boards sheathing	0,263	25	10,52
4	Simple boards sheathing	0,079	20	3,95
5	Simple boards sheathing	0,198	20	9,9
6	Simple boards sheathing	0,06	20	3

Fig. 61 Results of several calculations and approaches

The first and second results are coming from cyclic and monotonic experiments carried out at the University of Padua. It is clear that the influence of the cyclic load application influences a lot the overall shear modulus of the floor (Valluzzi, Di Bella and Garbin 2013). According to what is provided by American Wood Council the third value is given (Council 2005). Considering the experiments done by Gattesco on simple floors with a chord ringing the system, the fourth result is obtained (Gattesco and Macorini 2014). As for the fifth one, it is coming from the laboratory experiences made by Brignola (Brignola, Pampanin and Podestà 2012). For all the considered configurations the simple supported constrains as boundary conditions were used. Just the configuration of Gattesco was a little bit different and closer to the one used for the current numerical model. The constrains of the FE models were laterally fixed elements and they were closed to the system of Gattesco. Even if the configuration is a fixed-fixed beam the calculation is carried out considering the following formula for a supported beam. The stiffness of the system was calculated using the Timoshenko's approach without taking into account the flexural component with the following formula.

$$G = \frac{F/2}{B} \cdot \frac{a}{\Delta}$$

Where:

F was the applied force

B was the width of the diaphragm

a was the distance between the force and the lateral support

Δ was the maximum displacement

The last result is the one coming from the configuration used in the numerical system and compared with the one coming from the experimental results of Gattesco it can be said that the parameter is influenced by the constrains and the nails' behaviour.

11.CONCLUSIONS AND FURTHER PROPOSED INVESTIGATIONS

The thesis had the aim of performing numerical analysis on specimens and timber floors already analyzed with another software. The results of this work reached a good level but there are some aspects that should be deeper investigated and solved.

Considering the first part of the thesis focused on the numerical evaluation of the simple specimens, the obtained results, perfectly fitted the expected solutions. The two curves obtained from the sample A and from sample B exactly followed the experimental curves. Considering that these evaluations have been considered to calibrate the future model, the response of the software was perfect. As a conclusion of this first part it can be said that the punctual interfaces of the nails properly worked connected to the beam by using rigid connections.

Taking into consideration what most influenced the final results of the calculations for samples A and B, were the method of integration and the number of steps. Comparing the Newton-Raphson method with the Secant (Quasi-Newton) one, the first one was very instable, providing several times unreasonable solutions. In addition the small load step [0,08(500) steps] used for the sample A was compulsory due to the great amount of data attached to the interfaces defining the non-linear behaviour. As for the sample B, the data attached to the interfaces were two, the one for the nails and the one for the screws. In this case a larger load step was set due to the smaller non-linearity in terms of data connected to the interfaces [0,25(240) steps]. Moreover the tolerance was left as the default one. The good numerical results obtained for these two samples most probably were also influenced by the limited amount of both geometric and fine elements involved in the calculations. A better tolerance could be set according to a reduction of the load step.

On the other hand, considering the timber diaphragm analysis, the results were no so confident as it was expected. Unfortunately the software, during the calculations, found several difficulties in terms of integration procedures and also interaction between the rigid connectors and the punctual interfaces of the nails. Moreover considering the addition of the inter-boards linear interfaces to which friction laws were attached, the software was not able to show accurate results. For all the analysis on timber diaphragms just the combination of large load steps and high tolerance allowed to reach the final curves [0,25(320) steps]. Comparing the parameters used for the floors with the ones applied to the pushout analysis,

the integration method was the same but the tolerance influenced a lot the results (tolerance of 0,1). Probably, due to the complexity of the system the program was not able to carry out the analysis. An higher tolerance was needed to obtain the solutions. Due to these considerations, even if the results almost fitted the previous analysis (see the case of two nails placed in the rectangle 60×50 mm) and the analytical approach furnished quite reliable result (30,5 N/mm), some other models and analysis should be carried out. An improvement of the tolerance of the integration system should be reached and maybe a substitution of the elements considered should be taken into account. Acting in this way further refinement and better results are going to be obtained.

Regarding the floors having real dimensions the results showed two different behaviour and deformed shapes under different load configurations. Considering the suggestions given by ASCE, the floor under a parabolic distributed load behaved as it was expected. In addition the analytical calculation of its stiffness provided a quite reasonable result for the parabolic load configuration.

Taking these considerations into account, some further development can be suggested:

- Considering the simple model of the sample B involving two layers of boards and the contemporary behaviour of nails and screws, strengthened floors can be modeled. Due to the good results obtained for the sample B, strengthened floors with the contemporary presence of nails and screws can be considered.
- Taking into consideration that the work just focused on the modeling of 2000×2000 mm² timber floors, some calculations can be carried out on bigger and more realistic timber diaphragms. During the elaboration of the thesis two configuration of realistic floors were modeled. The first one considering just two nails as a board-to-joist connection (60×50 mm), and the other one with four nails placed in the same rectangle. Starting from these two models better results can be obtained.
- Regarding the possible improvements to the floor systems, the application of multipoint connections to the nails could be a reasonable solution. According to this methodology the integration area around the nails is bigger and it could provide better results and a smoother solution.

For this reason a numerical evaluation has been considered just for the simple pushout specimen A in order to evaluate if some reasonable results can be obtained. The model used was the same proposed in Chapter 8 but some improvements were considered. A better mesh was defined and rigid connections were attached all

around the nails. Acting in this way the aim was to enhance the integration along the lines connecting the mesh around the nail on the nail-to-plate connection.

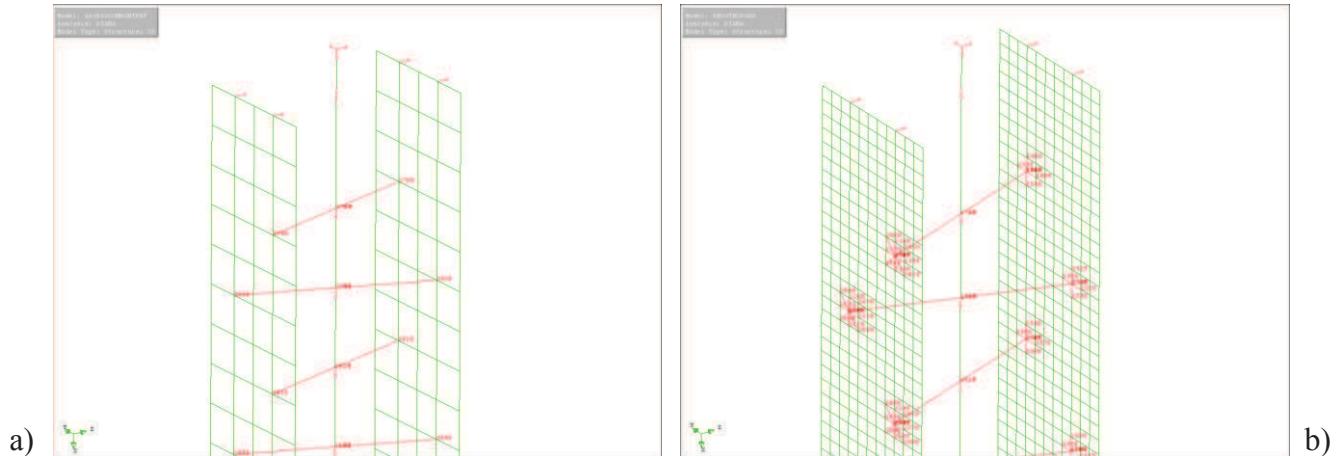


Fig. 62 Models of the pushout test: a) previous modelling; b) new model with rigid connections

As it can be seen the rigid connections applied on one plate are defining two crosses overlapped and rotated of 45°, and on the other plate just a vertical cross connecting the up and down points and the left and right ones is applied.

The parameters for the numerical calculations were exactly the same of the ones used in Chapter 8.

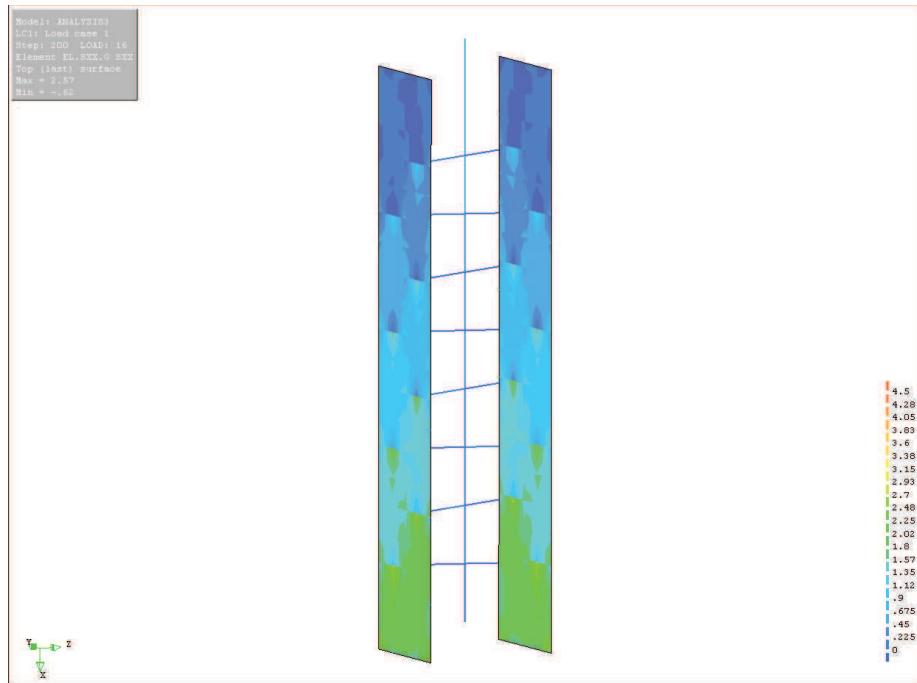


Fig. 63 Distribution of σ_{xx} stresses in the first model



Fig. 64 Distribution of σ_{xx} stresses in the improved model

It is clear that applying rigid connection all around the nail-to-plate connection, a smoother stress distribution is obtained. The rigid connections allowed a better integration along the lines of the fine elements and defined an area where the average value of integration is more stable than the ones provided by the first application.

As a conclusion of this proposal it can be said that a more coarse mesh provide a less precise non-linear curve behaviour of the nails but faster integrations. Peak values are concentrated in correspondence of the nails. On the other hand a finer mesh with rigid connections define a more precise non-linear curve of the nails and a slow integration. Peak values are reduced and not concentrated in the nails due to presence of a plate configuration given by a multi point constrains.

- Some improvements can be done in terms of the elements involved in the floors considering instead of linear beam and curved shells, a solid beam where in correspondence of the nails a plate interface is applied. Probably the application of linear interfaces to that points instead of punctual interfaces linked to rigid connection, presents a faster integration and the possibility to use a better tolerance.

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