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Experimental and analytical evaluation of the in-plane behaviour of as-built and strengthened traditional wooden floors

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ABSTRACT

Traditional timber floors cannot normally withstand horizontal seismic loads without large deformations. This may lead to a corresponding out-of-plane collapse of masonry walls in existing buildings. This situation is even more critical in the Netherlands, around the city of Groningen, where human-induced earthquakes started to take place. Since no seismic events have been experienced until recently, none of the existing buildings was designed with seismic events in mind, with no exception for the timber floors: therefore, it was necessary to characterize their in-plane response. To obtain representative results, firstly floor and roof samples were extracted from existing buildings. The relevant material properties were determined, together with the plank-joist connections behaviour. Replicas were then built with new material and tested to confirm the similarity in response compared to extracted samples. Based on these results, full-scale replicated diaphragms were constructed, and tested quasi-static reversed-cyclic in their plane, either parallel or perpendicular to the joists. Besides characterizing as-built diaphragms, a simple strengthening technique with plywood panels was applied as well, improving their in-plane response in terms of strength, stiffness and energy dissipation, as test results confirm. This study is concluded with an analytical characterization of the diaphragms' in-plane response, for as-built and strengthened configurations.

Abbreviations: a, Heart-to-heart distance between the main joists (mm); a', Heart-to-heart distance between the secondary purlins (mm); Av., Average value of a quantity measured in the experimental tests; b, Width of the cross-section of the main joists (mm); b', Width of the cross-section of the secondary purlins (mm); B, Width of the diaphragm, intended as the dimension parallel to the horizontal load (mm); b_p, Width of the plywood panel (mm); CLT, Cross-Laminated Timber; CoV, Coefficient of variation; E, Young's Elastic Modulus of wood (MPa); E_{dyn}, Dynamic Modulus of Elasticity of wood (MPa); $E_{dyn,0}$, Dynamic Modulus of Elasticity of plywood panels in the direction parallel to the fibres of the outer layers (MPa); $E_{dyn,90}$, Dynamic Modulus of Elasticity of plywood panels in the direction perpendicular to the fibres of the outer layer (MPa); E_{joists}, Dynamic Modulus of Elasticity measured for the joists (MPa); E_{planks}, Dynamic Modulus of Elasticity measured for the planks (MPa); F, Horizontal load (kN); FRP, Fibre Reinforced Polymers; G_{floor}, Equivalent shear modulus of the timber diaphragm, calculated according to test results and static scheme (MPa); G_{eq}, Equivalent shear stiffness of the timber diaphragm (N/mm); h, Height of the cross-section of the main joists (mm); h', Height of the cross-section of the secondary purlins (mm); I, Moment of inertia according to the loading direction (mm^4); K, Stiffness of the diaphragm at a certain level of displacement (kN/mm); K_0, Initial stiffness of the connection according to Foschi's exponential model (kN/mm); K_1, Post-yielding stiffness of the connection according to Foschi's exponential model (kN/mm); K_2, Initial secant stiffness of the diaphragm, calculated at 2 mm displacement (kN/mm); K_{20}, Equivalent secant stiffness of the diaphragm, calculated at 20 mm displacement (kN/mm); L, Length of the diaphragm, intended as the dimension orthogonal to the horizontal load (mm); l_p, Length of the plywood panel (mm); l_u, Ultimate displacement reached in a monotonic test of timber shear walls (mm); LVDT, Linear Variable Differential Transformer; LVL, Laminated Veneer Lumber; M_0, Initial moment given by the connection according to Foschi's exponential model (kNmm); M_i, Moment generated by a nail couple, opposing to the deflection of the diaphragm (kNmm); M_p, Plastic bending moment of a nail (kNmm); *m. c.*, Moisture content of wood (%); n, Number of tested specimens; n_{joists}, Total number of joists of the diaphragm; n_{planks}, Total number of planks of the diaphragm; OSB, Oriented Strand Board; PGA, Peak Ground Acceleration (m/s^2); t, Thickness of the floor planking (mm); T, Tensile strength of a nail (MPa); t_p, Thickness of the plywood panels (mm); w, Width of the planks (mm); X_i, Coordinate of the i-th nail couple along the joist (mm); δ, (Maximum) in-plane deflection measured on top of the diaphragm (mm); φ, Rotation of the nail couple at the plank-joist intersection (rad); ρ, Density of wood (kg/m^3)

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1. Introduction

1.1. Background

Timber diaphragms can be found in many old or existing buildings and houses, and are therefore part of the architectural heritage of several countries. Due to the fact that each area had its own (historical) construction techniques, it is still difficult to get comprehensive data on the structural behaviour under seismic loading.

Several tests have been conducted on as-built and retrofitted diaphragms in the recent years, especially in order to develop and assess various strengthening techniques to improve the in-plane behaviour: a number of signifying research studies will be summarized, covering different contexts in Europe, the United States and New Zealand, and investigating the diaphragms' structural response.

In [1] and [2] an experimental campaign on as-built and differently refurbished timber floors has been presented: the tested strengthening methods included some innovative options, like use of FRP laminae. In [3] the use of FRP is studied as well, and tests on various types of as-built flexible floors and traditionally adopted strengthening techniques are reported.

The in-plane behaviour of flexible diaphragms is also analysed in [4,5], together with a large number of strengthening techniques, both traditional and innovative.

In [6] the stiffening of existing floors with screwed CLT panels is proposed, but also tests on an original and a lightly strengthened diaphragm are reported. Use of CLT as retrofitting material is discussed in [7] as well, where it is compared to the response of timber diaphragms strengthened with OSB panels.

In [8] the behaviour of different flexible and strengthened timber floors is analysed and checked against the provisions of American standards FEMA 273 [9] and FEMA 356 [10].

A flexible floor is studied in [11] with respect to the context of New Zealand, and a strengthening technique with plywood panels is proposed.

In [12] the cyclic in-plane behaviour of flexible and strengthened diaphragms is examined, focusing on their orthotropic behaviour.

An in-situ experimental campaign is reported in [13], evaluating existing vintage timber diaphragms before and after retrofitting them with new fasteners and a plywood panels overlay.

A cyclic test on a whole as-built timber roof is presented in [14]. This test took place after the extraction of it from a replicated Dutch terraced house with masonry walls: before the cyclic test on the roof, the whole building was subjected to increasingly more intense earthquakes by means of a shaking table; no strengthening techniques were applied.

In general, the aforementioned research studies highlighted the poor response of as-built floors, and showed that by applying different strengthening techniques a considerable improvement in the in-plane stiffness can be gained, quantifiable as follows with respect to the as-built conditions:

- 3 to 10 times if CLT or OSB is fastened on the existing diaphragms [6,7];
- 5 to 15 times when applying light steel gauges [1,2,4];
- 5 to 20 times when superposing a layer of planks arranged at 45 degrees with respect to the existing sheathing [1,2,4];
- 5 to 20 times with a screwed or nailed plywood panels overlay [8,11–13];
- 25 to 100 times when using FRP [1,2,3];
- 75 to 200 times with the construction of a reinforced concrete slab [1,2].

The variation in the reported values depends on the characteristics of the as-built diaphragms, the entity of the intervention itself, and on how the stiffness is evaluated by different authors.

1.2. Timber diaphragms in the Netherlands

For the Dutch context a lack of knowledge can be observed; this is because on the one hand the area around Groningen started to be subjected to more intense earthquakes generated by gas extraction only in recent years. Therefore, the seismic assessment of existing or historical buildings was no issue until recently.

On the other hand, the timber diaphragms have specific characteristics different from other floors analysed in literature. With reference to the features of the aforementioned tested diaphragms, some typical properties of traditional Dutch timber floors are of relevance:

- The structural elements are normally smaller than the ones belonging to other contexts, such as those studied in [1–8,11–13]; for instance, a main joist of a floor can have a cross section of 60 × 130 mm (or even 50 × 105 mm for a roof), with spacing ranging from 600 to 900 mm;
- The floor sheathing is realized with continuous planks: since these elements are not interrupted, the global behaviour depends above all on the flexural stiffness of the planks (or of the joists, according to the loading direction) and on the rotational stiffness of the connections between the planks and the main joists, as is stated when formulating the analytical model presented in [15];
- Roofs of detached houses are normally composed of main and secondary beams, the former supported by a wall plate, i.e. a timber element positioned on top of the external walls of the building. Due to the very simple connection between main rafters and wall plate, this kind of roof structure appears to be extremely flexible when subjected to horizontal loads, because the rafters can be considered as practically hinged on the wall plate.

1.3. In-plane stiffness of timber diaphragms

One of the essential parameters which characterizes the behaviour of timber floors subjected to horizontal loads is their in-plane stiffness. American [9,10,16] and New Zealand standards [17] treat the diaphragms as horizontal shear walls, proposing also a value of stiffness based on the configurations and conditions of the existing floor or on the adopted strengthening method. The draft of the new seismic guidelines for the Netherlands [18] includes shear stiffness values for entire diaphragms taken from the above-mentioned standards, therefore their application for the Dutch case has to be validated. The proposed value of stiffness, defined as an equivalent shear stiffness G_{eq}, is specified as being independent of the dimensions of the diaphragms and is commonly used to describe their in-plane behaviour:

$$G_{eq} = G_{floor}\, t \tag{1}$$

where G_{floor} is a test result that depends on the static scheme of the diaphragm, and is an equivalent shear modulus describing the global behaviour of the floor; t is the diaphragm's thickness.

However, depending on the configuration of each floor, this description of its in-plane response might not always be representative: for flexible floors, if the planks (or the joists) are continuous, their flexural stiffness is governing the in-plane behaviour. Thus, the assumption of a floor acting as a shear wall might be questionable, at least for non-strengthened diaphragms, because the response in terms of their deflection would not be properly captured. This emerged also from the numerical study presented in [19], where G_{eq} was found to be not size-independent for as-built floors. Instead, when the diaphragm is retrofitted, it might be able to bear a lateral load as a single slab and therefore its behaviour could be more shear-dependent.

Another unclear aspect is related to the definition of G_{eq} itself: since the behaviour of the floor is strongly nonlinear in terms of stiffness, the value of G_{eq} depends on the in-plane displacement of the diaphragm. However, normally only a single value of G_{eq} is assumed, neglecting the total load–displacement response of the diaphragm and leading to not comparable data due to the different adopted methods to calculate the stiffness.

Finally, in [12] the tested floors exhibited an orthotropic behaviour: this property can be of relevance depending on the diaphragms' size and dimensions [19], and has therefore to be investigated for traditional Dutch floors as well, as the earthquake response may depend on it.

1.4. Objectives of the research study and outline

The main aim of the present work was to assess the in-plane response of as-built and strengthened timber diaphragms with Dutch features. This characterization took place both experimentally and analytically.

From the experimental point of view, a testing campaign was conducted on full-scale diaphragms that were replicated based on the properties of original samples extracted from existing buildings. The target of the experimental tests was firstly to characterize the as-built floors' in-plane response, and secondly to develop and investigate simple and effective strengthening methods: these retrofitting interventions have to be quite adaptable, because the PGA levels in Groningen are characterized by a large variation within a small region.

From the analytical point of view, the in-plane behaviour of the tested diaphragms was characterized by examining their flexural or shear-related response, nonlinearity and orthotropy. Moreover, the amount of collected information aims to provide professional engineers with relevant properties, such as strength, stiffness and energy dissipation, properly describing the response of these diaphragms under earthquakes.

Therefore, in this article, after a description of the replication phase and the test setup, the experimental results are presented and discussed, with particular reference to the aforementioned aspects. Furthermore, the analytical characterization of the in-plane response for as-built and strengthened diaphragms is illustrated.

2. Materials and methods

2.1. Introduction

Because it was not possible to extract complete existing diaphragms and directly test them in laboratory, small-scale samples of floors and roofs from existing buildings were taken, and replicas were accurately manufactured after the characterization of the extracted specimens: this allowed to test the diaphragms as representatively as possible. The process of replication was based on both the study of the configurations of extracted diaphragms and the determination of their material properties.

In section 2.2 the main properties of the collected samples and of the afterwards replicated diaphragms are reported. The procedure followed for the extraction, characterization and replication of original samples is described in Section 2.3, which presents also the experimental setup for full-scale diaphragms testing.

2.2. Materials

2.2.1. Extracted samples

After a survey in the most common typologies of traditional timber diaphragms in the Groningen area, four representative samples (Fig. 1) were extracted from existing detached houses (built 1890–1930) to be demolished. Three of them were wooden floors, while the fourth was part of a pitched roof; the extracted portions measured approximately 2 × 2 m and were labelled with letter *G* (Groningen), followed by the progressive number.

In most samples, joists and boards were made of spruce (*Picea abies*), but in sample G4 pine (*Pinus sylvestris*) planks were found. The joists and the planks had dimensions, geometry and fasteners (common round or square nails) as given in Table 1. These properties were taken as the basis for the construction of the replicas and the strengthened samples described in Section 2.2.2.

2.2.2. Replicated specimens

Tables 2 and 3 present the characteristics of the non-strengthened and retrofitted replicated diaphragms, and in Figs. 2–4 their configurations are shown. The five diaphragms were labelled with the initials *DF* (Detached house Floor) or *DR* (Detached house Roof), followed by *par* or *per* (loading parallel or perpendicular to the joists, respectively), and by the progressive number. For the strengthened samples, letter *s* was added at the end.

The structural elements were made of spruce (*Picea abies*) timber with strength class C24 [20], and the planks presented a tongue and groove configuration, as was found in original samples: in order to represent the situation observed in reality, the tongues were not fully pushed to the side of the next plank, leaving a gap of approximately 2 mm.

Due to the low in-plane stiffness which characterized the floors in their original configuration, a simple retrofitting technique was adopted, in order to improve their in-plane behaviour and their dissipative properties: the target in this case was to develop a simple and light strengthening technique, which was properly found in the superposition of a layer of plywood panels to the original floor.

It was chosen not to cut the plywood panels in too large dimensions, to guarantee an easier installation of them in practice. In the very first strengthening (specimen DFpar-1s), approximately 600 × 1200 mm panels were used, which were positioned and screwed without taking into account the underlying layer of existing planks.

From the second diaphragm on, the panels were cut, placed and screwed in such a way that all the fasteners were effectively connecting the existing and the additional layer. In other words, the connectors were always crossing the total thickness of the plywood panels and the planks, while for the first sample in some cases the screws were crossing the existing sheathing in the tongue between two planks. The position of underlying joists was not considered for cutting and placing the plywood panels and the screws, except for the top joist (Fig. 2) and the timber blocks (Fig. 3).

The structure of the roof pitch was slightly more complex than the floors' one, according to the situations found in practice: between the wall plate and the top main beam of the roof, rafters were arranged, to which the purlins (supporting in turn the planks) were connected (Fig. 4).

The diaphragms were constructed to be tested in two directions, in order to analyse their orthotropic behaviour, for both the as-built and the strengthened configurations.

When loading the specimens perpendicular to the joists, their connection to a (masonry) wall was also simulated by means of tropical hardwood elements, shown in the detail of Fig. 3. Two configurations were considered:

- If the masonry pocket is totally void of mortar, then a hinged configuration can be assumed (specimens DFper-3 and 3s, Fig. 3);
- If the joist is inserted in the masonry pocket and mortar is present, a slight clamping effect might be introduced (specimens Dfper-4 and 4s, Fig. 3).

The plywood panels were fastened to the sheathing along their perimeter with different screws depending on the specimen (see Table 3). In addition to that, in four specimens some further strengthening elements were applied:

- For specimens DFpar-1s and DFpar2-s, the upper row of screws corresponded to the fastening of the strengthening panels directly to the top joist of the floor: hence, to improve the shear transfer in this area, 5 × 70 mm screws were used;

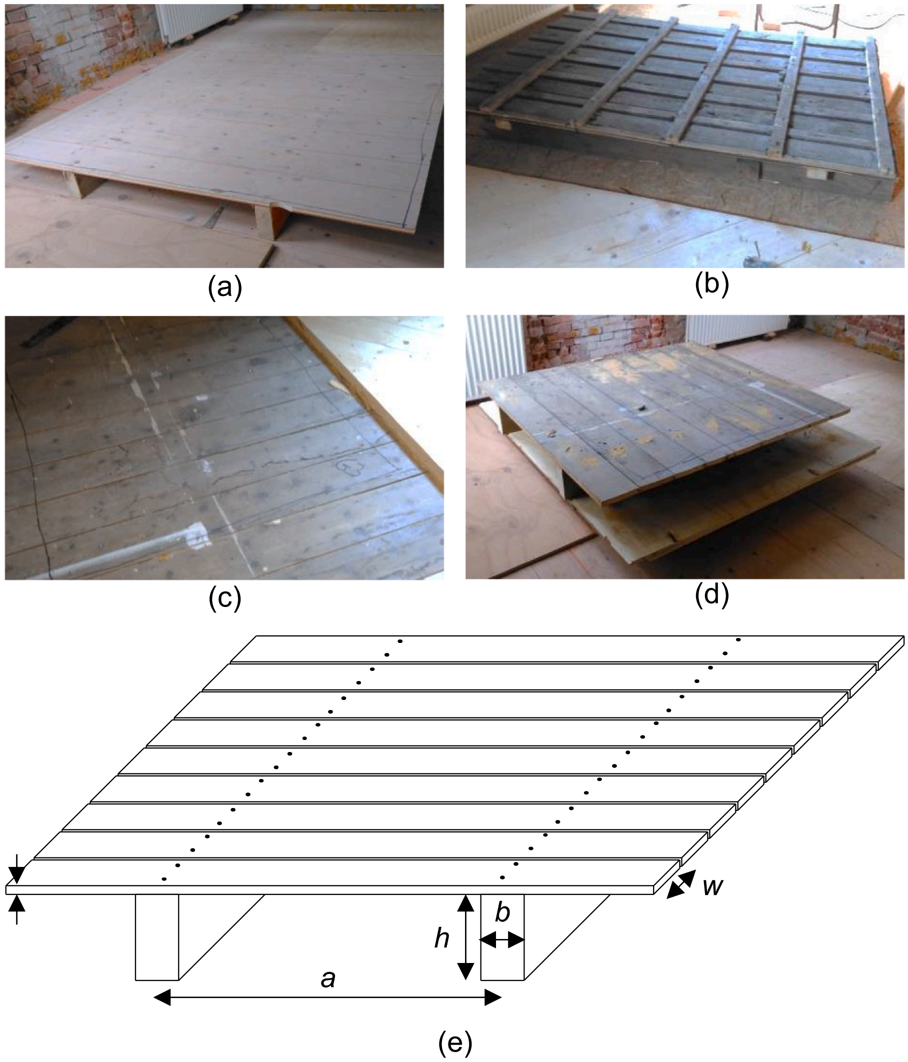

Fig. 1. Extracted samples: G1 (a), G2 (b), G3 before cutting (c), G4 (d); geometrical parameters of interest for replication (e).

Table 1
Characteristics of the extracted samples.

Specimen	G1 (floor)	G2 (roof)	G3 (floor)	G4 (floor)
w (mm)	162	164	163	166
t (mm)	18	15	23	23
h (mm)	112	105	118	165
b (mm)	51	52	62	61
a (mm)	788	912	736	650
h' (mm)	N. A.	35	N. A.	N. A.
b' (mm)	N. A.	62	N. A.	N. A.
a' (mm)	N. A.	820	N. A.	N. A.
Fasteners	Two 3 × 65 mm nails at every intersection plank/joist	- Two 3 × 55 mm nails at every intersection plank/purlin; - One 5 × 110 mm nail at every intersection purlin/rafter	Two 3 × 65 mm nails at every intersection plank/joist	Two 3 × 65 mm nails at every intersection plank/joist

- In specimen DFper-4s, additional 60 × 130 mm timber blocks were placed on top of the floor between each couple of joists; this configuration allows not only to improve the in-plane stiffness of the floor through a better fastening of the panels, but can also be a possible solution to realize a diffused connection between diaphragm and wall in practice, by fastening these blocks to the latter;
- In the extracted roof samples, the rafters supporting the pitch were fastened to the wall plate with only one 5 × 110 mm nail: this situation was replicated also in specimen DRpar-5. Because this was the only connection between the whole structure of the roof and the walls, it was chosen not only to improve the stiffness of the diaphragm, but to make the transfer of horizontal forces more effective as well. Therefore, between the rafters 75 × 150 × 8 mm steel angles were fastened both to the roof structure and to the wall plate with 6 × 70 mm screws at 150 mm centres. Besides, the plywood panels were cut in such a way that they could be inserted between the purlins, allowing to perform an intervention from the inner part of the roof.

2.3. Methods

2.3.1. Introduction

The in-plane behaviour of timber diaphragms is governed not only by the material properties of timber, but also by strength, stiffness and energy dissipation of the fasteners connecting all the structural elements [21]. Fig. 5 shows the followed process for the determination of these characteristics: after extracting original samples from detached houses (Section 2.3.2), all elements were subdivided into smaller specimens to be tested. After determining the properties of timber and

Table 2
Characteristics of the five tested diaphragms, reported for the as-built configurations.

Properties of non-strengthened diaphragms					
Specimen	DFpar-1 (floor)	DFpar-2 (floor)	DFper-3 (floor)	DFper-4 (floor)	DRpar-5 (roof)
Loading direction	Parallel to the joists		Perpendicular to the joists		Parallel to purlins
L (mm)	2400	2400	2300	2300	2730
B (mm)	3800	3960	3800	3800	3800
w (mm)	165	165	165	165	165
t (mm)	18	24	18	18	18
h (mm)	130	130	110	110	105
b (mm)	60	60	50	50	50
a (mm)	650	650	750	750	925
h' (mm)	N. A.	N. A.	N. A.	N. A.	35
b' (mm)	N. A.	N. A.	N. A.	N. A.	60
a' (mm)	N. A.	N. A.	N. A.	N. A.	820
Fasteners	Two 3 × 65 mm nails at every intersection plank/joist				- Two 3 × 55 mm nails at every intersection plank/purlin; - One 5 × 110 mm nail at every intersection purlin/rafter

Table 3
Characteristics of the strengthened versions of the five tested diaphragms.

Additional properties of strengthened diaphragms					
Specimen	DFpar-1s	DFpar-2s	DFper-3s	DFper-4s	DRpar-5s
l_p (mm)	1200	1200	1200	1200	820
b_p (mm)	600	670	770	770	760
t_p (mm)	18	18	18	18	18
Screws	4.5 × 40 mm	5 × 60 mm	5 × 60 mm	5 × 70 mm	4.5 × 40 mm
Spacing	100 mm along the perimeter of each panel for all specimens				
Other remarks	5 × 70 mm screws used for top row at 150 mm spacing	5 × 70 mm screws used for top row at 100 mm spacing	N. A.	Between the joists: top 60 × 130 mm blocks	75 × 150 × 8 mm steel angle fastened at roof bottom with 6 × 70 mm screws at 150 mm spacing

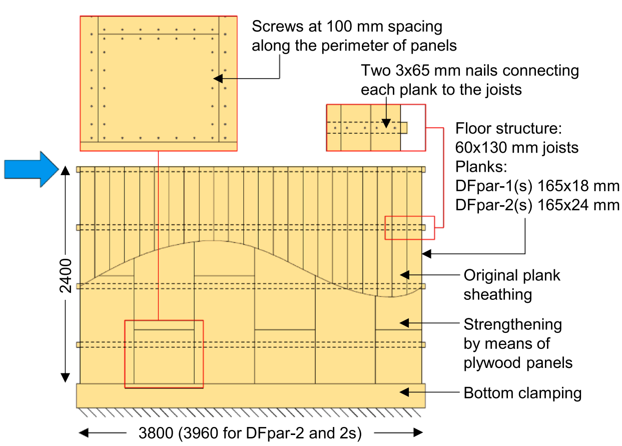

Fig. 2. Schematic representation of the diaphragms tested parallel to the joists (specimens DFpar-1 and 1s, DFpar-2 and 2s); view from decking side. Dimensions in mm.

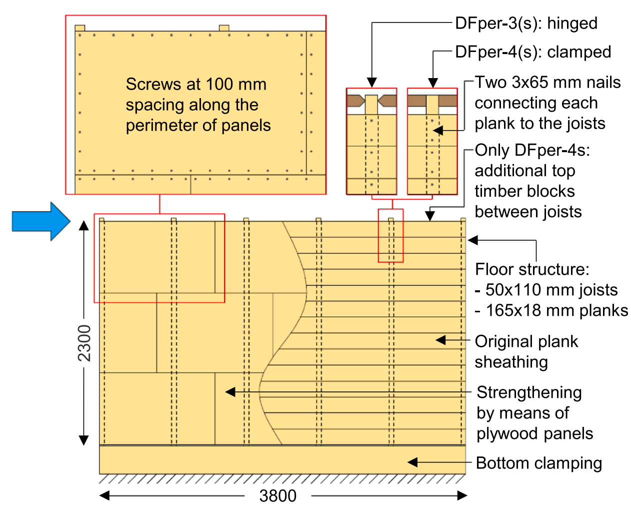

Fig. 3. Schematic representation of the diaphragms tested perpendicular to the joists (specimens DFper-3 and 3s, DFper-4 and 4s); view from decking side. Dimensions in mm.

fasteners from original samples, materials with similar properties were ordered and tested for realizing the replicated floor specimens with good accuracy. Additionally, cyclic tests on both extracted and replicated plank-joist connections were performed (Section 2.3.4), to confirm the representativeness of the replication. These tests were conducted perpendicular to the joists (configuration A), parallel to the joists (configuration B) and in rotation (configuration C), according to Fig. 5.

2.3.2. Extraction of original samples of diaphragms from detached houses

The extraction of samples was conducted according to the following process:

- A visual survey was performed prior to the selection of timber samples;
- Existing finishes, such as carpeting, were removed, in such a way that the sample itself was void of anything but the structural floor: joists, planks and nails; for roof samples the tiles were removed;
- The sample location in the building was marked and identified, and a sketch or photograph from each proposed sampling location was prepared;
- Any imperfections on the samples were marked;
- The extraction of the samples was performed carefully and accurately;

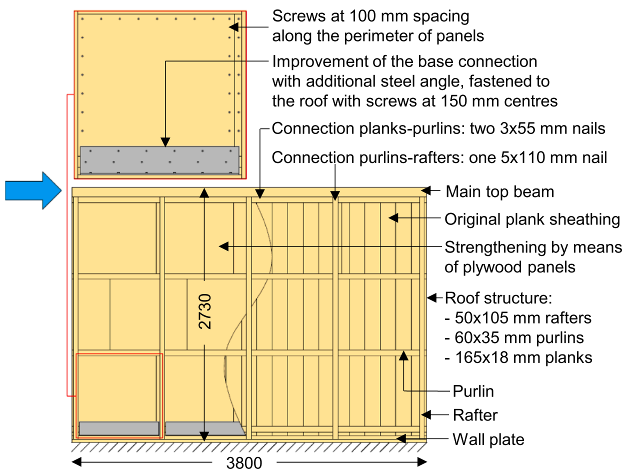

Fig. 4. Schematic representation of the tested roof pitch (specimens DFpar-5 and 5s); view from the inner side. Dimensions in mm.

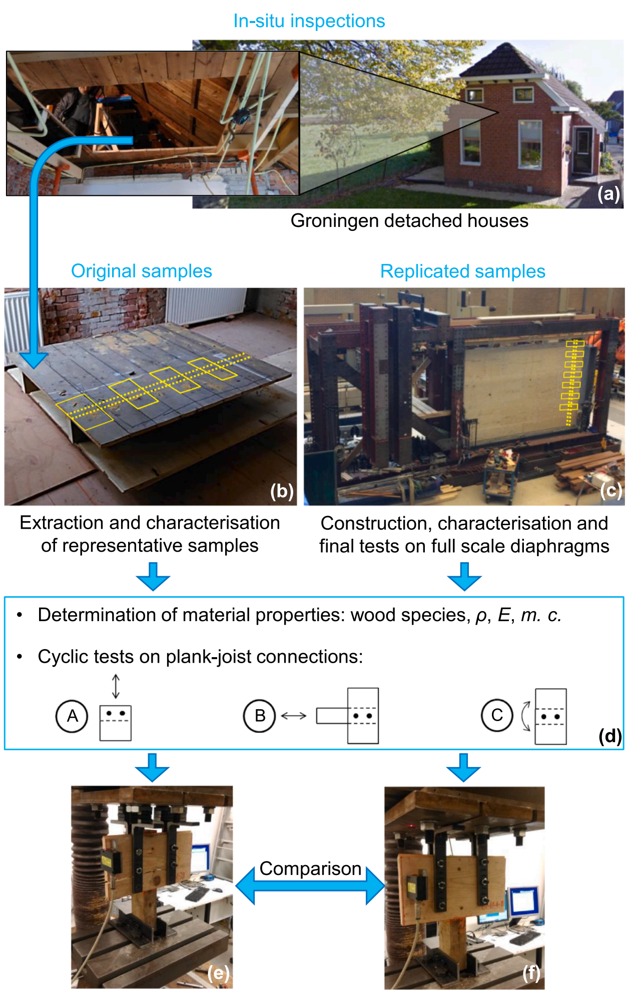

Fig. 5. Process followed for the determination of material properties of timber and plank-joist connections for original and replicated samples: (a) in-situ investigation and selection of samples; (b) extraction of floor and roof samples; (c) construction of replicated specimens on the basis of the characteristics of original ones; (d) characterization of material properties and cyclic behaviour of plank-joist connections, with comparison between extracted (e) and replicated samples (f).

- The samples were removed from the construction site and set on a dry and stable horizontal surface;
- All samples were moved to the site of preparation for transport and their condition on all exposed sides was documented; the samples were kept out of the sun to prevent deformations;
- The samples were protected on all sides to prevent damage during transportation.

After the extraction and transportation, all the samples were delivered at TU Delft Stevin II Laboratory.

2.3.3. Determination of material properties of original and replicated timber elements and fasteners

With regard to timber elements, the determined material properties were dimensions, density (ρ), dynamic modulus of elasticity (E_{dyn}), moisture content (*m. c.*).

The density was measured by weighing the test sample and dividing the weight by its volume. The modulus of elasticity was determined by longitudinal vibration measurements with a *Brookhuis MTG 960* [22]. With this apparatus the first natural frequency of the timber pieces was determined, and by knowing the density the dynamic modulus of elasticity E_{dyn} was calculated, which is on average 8% higher than the static one [23], determined in accordance with EN 408 [24]. The moisture content of the timber was determined with the oven-dry method as specified in EN 13183-1 [25].

With regard to fasteners, and namely the nails connecting the main structural elements, two material properties were determined: tensile strength (T) and plastic bending moment (M_p), in agreement with EN 409 [26].

2.3.4. Determination of cyclic behaviour of original and replicated plank-joist connections

A well-performed replication of the cyclic behaviour of plank-joist connections is of importance, because the in-plane response of timber diaphragms largely depends on them. With newly-built samples, the connections might display much better strength and stiffness compared to existing ones [21]; therefore it was chosen to test and replicate their cyclic behaviour, to investigate whether adjustment factors for replicated samples' properties were needed.

The specimens representing plank-joist connections were loaded quasi-static reversed-cyclic in accordance with ISO 16670 [27], in order to determine their hysteretic behaviour. These tests were conducted perpendicular and parallel to the joists, and also in rotation: the loading directions corresponded to cases A, B, and C of Fig. 5, respectively.

After characterizing existing specimens, the same tests were conducted also on the replicated ones: the timber elements ordered according to the measured material properties were used to build each diaphragm, from which plank-joist connection samples were extracted. The fasteners used for the construction of the replicated diaphragms had identical or, where not possible, similar characteristics in terms of diameter and yield strength to the ones found in the original samples from the Groningen area.

In this way, as shown in Section 3.2, it was possible to compare the properties and the behaviour of the extracted samples with replicated ones: similar values in terms of strength and maximum displacement (with respect to the usual scatter which affects the tests performed on timber joints) were found, thus no adjustment factors were needed.

2.3.5. Full-scale experimental setup for the cyclic tests on replicated diaphragms

The test setup was designed taking into account the characteristics of Dutch timber floors described in Section 1.3. It was chosen to test in a vertical configuration half of the diaphragm, according to the principle shown in Fig. 6: considering the behaviour of the planks (or of the joists), it is possible to test only one half of the diaphragm by clamping its bottom part (centre of symmetry of the floor). In this way, the applied

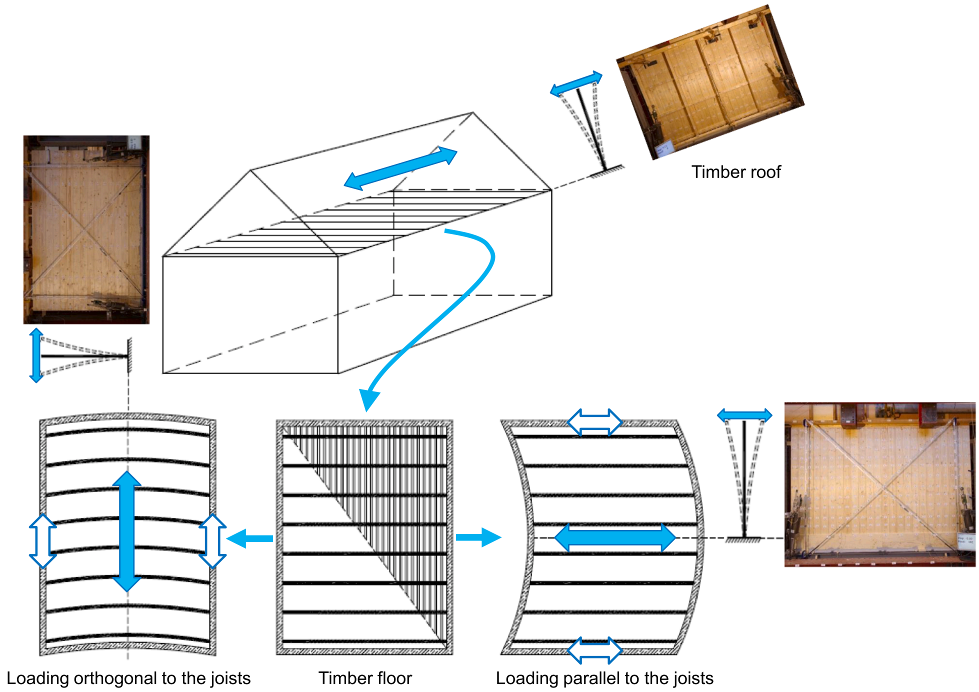

Fig. 6. Principle for the adopted test configuration of floors and roofs.

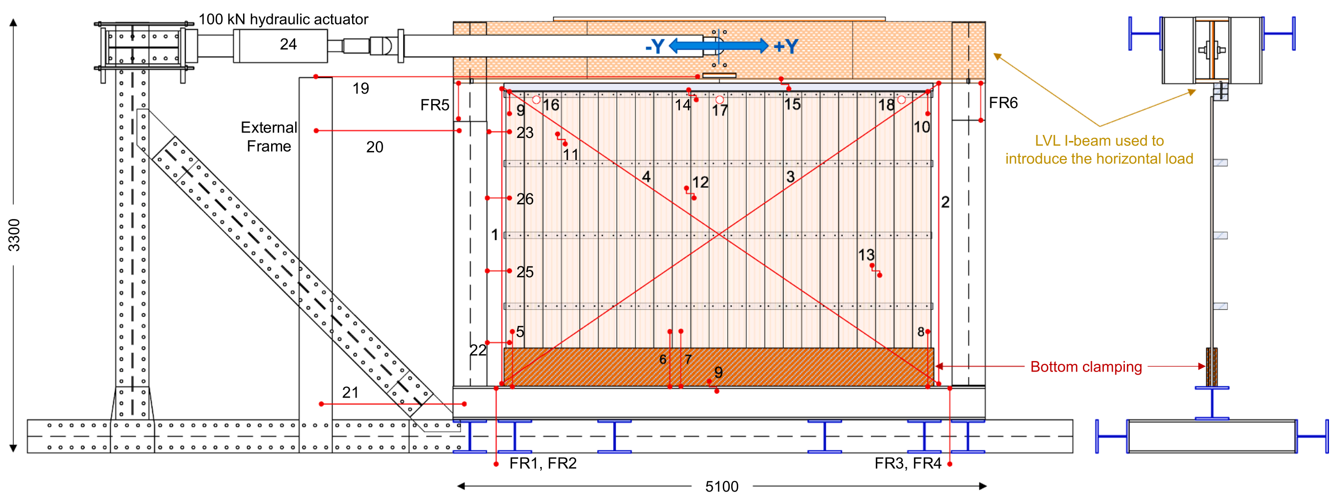

Fig. 7. Front view and cross-section of the test setup; the position of the sensors is also shown.

force corresponds to the reaction that the floor is able to bear.

The tested diaphragm was glued to a bottom HEB 300 steel beam, which was bolted to the part of the test setup connected to the laboratory floor (Fig. 7). The horizontal load was introduced by means of an LVL I-beam, fastened to the top joist (or to the wooden blocks shown in Fig. 3, when loading orthogonally to the joists) with screws having a diameter of 10 mm and spaced 150 mm. Lateral out-of-plane displacement of the LVL I-beam during the test was prevented by applying vertical steel elements, covered with Teflon to allow low-friction sliding.

In order to ensure the cantilever scheme assumed for the tests, the floors were clamped on the bottom part using two layers of plywood elements glued together and screwed on both sides. As regards the roof, it was sufficient to test only one of the two pitches; the clamping was not necessary, as this was already provided by the wall plate.

The measurement plan is given again in Fig. 7: depending on the configuration of the specimens, the position of the sensors was slightly adapted for the different tests. For the strengthened versions of each diaphragm, extra sensors were positioned to record also the horizontal and vertical sliding of the plywood panels. All sensors consisted of LVDTs, except for the ones used to record the out-of-plane displacements of each diaphragm, which were lasers.

The testing protocol for cyclic loading according to ISO 21581 [28] was adopted, with a variable rate to achieve the ultimate displacement l_u between 1 and 30 min.

3. Experimental results

3.1. Material properties of original and replicated samples

Tables 4 and 5 show the comparison between the characteristics of original and replicated timber elements and fasteners, respectively: taking into consideration the usual scatter affecting timber structural members, the replication was regarded as sufficiently accurate, leading to similar properties for replicated samples with respect to the extracted ones. A slightly larger difference was instead detected for the fasteners,

Table 4
Comparison of the material properties of timber for extracted and replicated samples.

Property	Extracted specimens			Replicated specimens		
	n	Av.	CoV (%)	*n*	Av.	CoV (%)
ρ (kg/m^3)	27	481	9.6	35	474	10.2
E_{dyn} (N/mm^2)	27	12990	18.3	35	11830	21.2
m. c. (%)	21	9.2	2.0	39	11.3	16.2

Table 5
Comparison of the characteristics of fasteners for extracted and replicated samples.

Property	Extracted specimens			Replicated specimens		
	n	Av.	CoV (%)	*n*	Av.	CoV (%)
T (MPa)	28	655	14.8	8	792	2.0
M_p (kNmm)	23	3.2	10.1	8	3.9	3.2

essentially because of two reasons:

- Newly produced nails are characterized by higher quality compared to older ones, which had poorer and not standardized properties;
- The majority of the extracted nails were slightly rusty.

It is however worth remarking that, although this difference in terms of material properties was detected for the single fasteners, the behaviour of the whole plank-joist connections proved to be similar, as noticeable in Section 3.2.

This phenomenon can be explained considering that in [29] timber joints with rusty nails showed an increase of 20–25% in capacity compared to normal ones, because of the higher friction between wood and steel that could develop. This increase is within the same range as the recorded difference in properties between extracted and replicated samples: the higher values obtained for the new nails are therefore compensated in the whole plank-joist connection by the improvement in capacity given by rusty nails.

Additionally, material properties of the plywood panels used for the strengthening were determined (Table 6).

3.2. Cyclic behaviour of original and replicated plank-joist connections

The hysteretic cycles of plank-joist connections were obtained as described in section 2.3.4. In total, for the original samples, 10 tests were conducted for each configuration A, B and C. For the replicated samples, 7 tests were performed for configurations A and C, 4 for configuration B.

Fig. 8 shows a typical example for comparison: the two cycles are practically coincident until an already large displacement (10 mm) with similar values of strength and stiffness, considering the normal scatter which characterizes tests on timber joints. Furthermore, by visual observation, the hysteretic behaviour itself appears to be similar as well. It can be concluded that the replicated samples were able to show a comparable response under cyclic loading with respect to the extracted ones and were therefore regarded as representative for them.

Table 6
Material properties of the plywood panels used for strengthening.

Property	*n*	Av.	CoV (%)
ρ (kg/m^3)	39	473	3.5
$E_{dyn,0}$ (N/mm^2)	5	7130	6.8
$E_{dyn,90}$ (N/mm^2)	5	6310	11.2
m. c. (%)	29	9.2	19.3

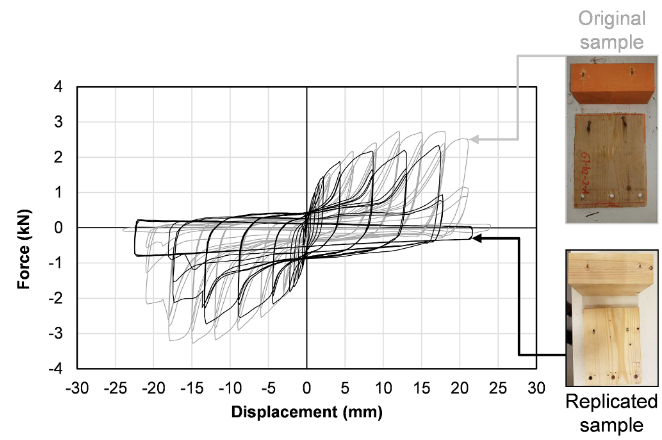

Fig. 8. Comparison between the hysteretic cycles of original and replicated plank-joist connections.

3.3. Test results on full-scale diaphragms

3.3.1. Introduction

The hysteretic cycles of the diaphragms are reported according to the loading direction for floors, while for the roof pitch specimen a separate graph is shown.

In all charts also the stiffness of the various floors is indicated; for its calculation, two values of displacement were considered:

- 2 mm of displacement, corresponding to the value of a very initial secant stiffness K_2;
- 20 mm of displacement, where nonlinearities have already taken place, corresponding to an equivalent elastic secant stiffness K_{20}.

This displacement corresponds to the horizontal deflection of the diaphragms measured at their top (sensor 19 in Fig. 7).

3.3.2. Specimens loaded parallel to the joists

Fig. 9 depicts the hysteretic cycles obtained for specimens DFpar-1 and 1s, and DFpar-2 and 2s. Both non-strengthened diaphragms displayed a flexible in-plane behaviour with very limited energy dissipation. The small difference in stiffness was given by the diverse thicknesses of the planks (18 mm for DFpar-1, 24 mm for DFpar-2). No signs

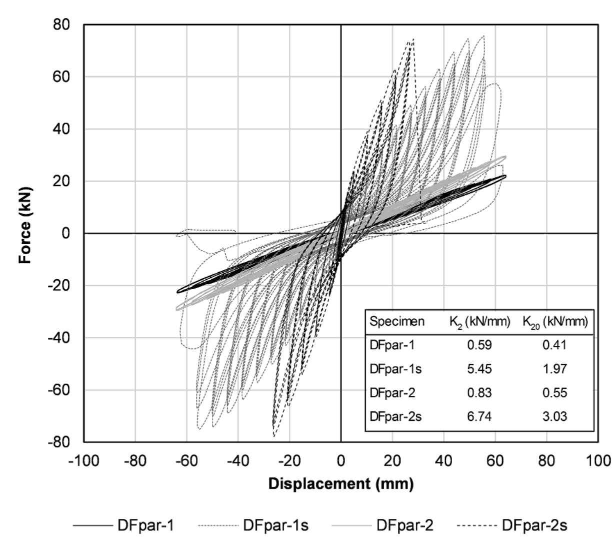

Fig. 9. Experimental hysteretic cycles obtained for specimens loaded parallel to the joists; the graph reports also the values of the initial and equivalent elastic stiffness for each tested floor.

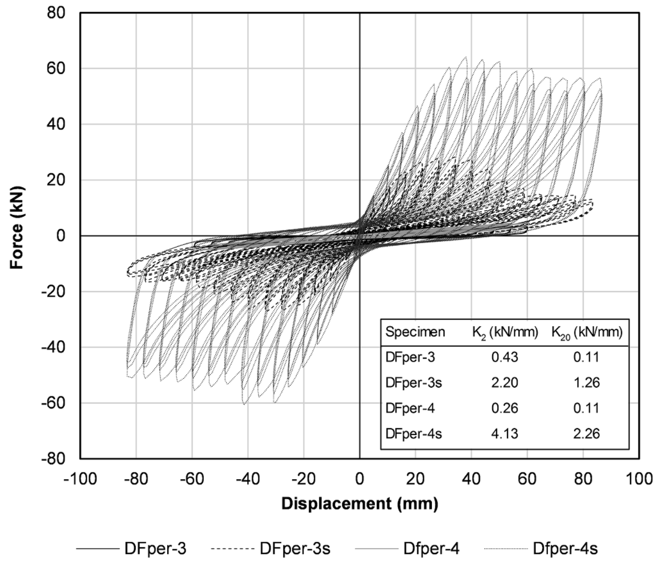

Fig. 10. Experimental hysteretic cycles obtained for specimens loaded perpendicular to the joists; the graph reports also the values of the initial and equivalent elastic stiffness for each tested floor.

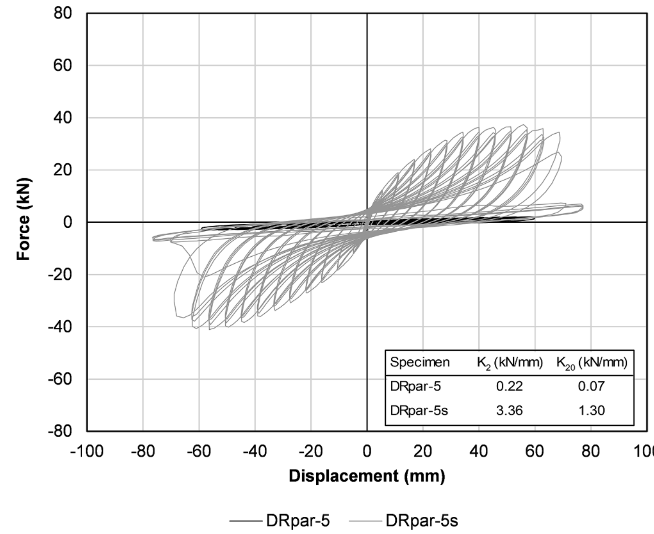

Fig. 11. Experimental hysteretic cycles obtained for the roof pitch specimen; the graph reports also the values of the initial and equivalent elastic stiffness for the tested diaphragm.

of global failure of the diaphragms were present after the end of the tests.

The first retrofitted floor (DFpar-1s) strongly improved the stiffness and strength with respect to the original configuration. At large displacement values, failure of nails and screws was observed on top of the floor, together with an overall plasticization of fasteners across the diaphragm. This can also be noticed from the large amount of dissipated energy visible.

The second strengthened floor (DFpar-2s) was characterized by an even stronger and stiffer behaviour, probably due to the more accurate positioning of the plywood panels on the existing sheathing; again, high energy dissipation took place in the fasteners. However, it was not possible to test this floor until large displacements, due to the sudden failure of the bottom glue layer. Despite this inconvenience, the reached level of displacement was still sufficient to characterize the in-plane behaviour of this retrofitted diaphragm.

3.3.3. Specimens loaded perpendicular to the joists

Fig. 10 shows the hysteretic cycles obtained for specimens DFper-3 and 3s, and DFper-4 and 4s. Very flexible in-plane behaviour was observed with low energy dissipation, and the difference in the response between the hinged (DFper-3) and the clamped (DFper-4) configuration displayed in Fig. 3 was not noticeable. Besides, the stiffness measured at 2 mm for sample DFper-4 (clamped) is lower than the one recorded for sample DFper-3 (hinged), due to the observed higher friction among the planks for the latter specimen. No signs of global failure of the diaphragms were present after both tests.

The first retrofitted floor (DFper-3s) displayed a great improvement in its strength, stiffness and energy dissipation due to the applied strengthening method. However, after 30 mm displacement a softening phase took place, caused by the progressive plasticization and failure of the top nails. This fact caused the joists to move independently of the sheathing and therefore not the whole resistance of the floor could be activated.

For sample DFper-4s, a solution allowing better transmission of shear forces was adopted, leading to an overall much stiffer behaviour: the presence of timber blocks between the joists helped the diaphragm to deflect as a whole shear wall, unlike the previous strengthened case. The consequence was an improvement in strength and stiffness, but also in the capacity to withstand high lateral forces at large levels of displacement. As can be noticed, compared to sample DFper-3s, specimen DFper-4s is characterized by a much smoother softening phase (less than 20% of the peak value), although widespread plasticization in the fasteners and cracks in timber at large displacements occurred. This is also the reason of the high energy dissipation noticeable from the graph. Apart from plastic behaviour, few screws were also subjected to tensile failure.

3.3.4. Roof pitch specimen

In Fig. 11 the hysteretic cycles of the tested roof sample are shown. In this case, due to the very flexible connection between each rafter and the wall plate, the roof was almost not able to withstand horizontal forces. No signs of global failure of the diaphragm were observed after the end of the test.

After the strengthening of the roof with plywood panels and bottom steel angles, the behaviour of sample DRpar-5s was strongly improved in terms of strength, stiffness and energy dissipation. The test was stopped after the pull-out failure of top nails and of few bottom screws of the steel angle.

3.3.5. General observations on the tests

From the aforementioned results some general considerations can be drawn. First of all, the as-built timber diaphragms appeared to be very flexible, especially in the direction orthogonal to the joists. With the applied strengthening technique, on the contrary, strength and stiffness of the diaphragms were largely improved.

Besides, another interesting benefit of such an intervention is related to energy dissipation. In fact, refurbishment techniques such as use of FRP or concrete slabs lead to high stiffening of floors, but normally the amount of dissipated energy is limited [2]. Instead, with this adopted strengthening method, the floor is at the same time stronger, stiffer and also able to helpfully dissipate part of the energy provided by the earthquake.

These characteristics were found in all tested diaphragms, but are of importance especially for the roof sample: strengthening a roof is a delicate intervention, because on the one hand an excessive flexibility may cause the whole building to be locally or globally damaged, on the other hand if the stiffening method is associated with a not negligible increment of mass and lack of dissipative properties, the overall performance of the building can even be worsened. This is because the highest load provided by the earthquake occurs on top of the building, and the presence there of a too stiff and heavy structural element increases the horizontal load transferred to the walls. The proposed strengthening technique allows instead to maintain at the same time light and sufficiently stiff diaphragms, which are also able to beneficially dissipate energy.

The tested diaphragms clearly showed a difference in their response depending on the loading direction. After their strengthening, this orthotropic behaviour is still present, but much more mitigated if compared to the scatter observed for the as-built diaphragms: especially when besides the plywood panels timber blocks were placed, the behaviour in the two loading directions was very similar.

With regard to the observed deformation of the floors, as will be further discussed in Section 4.1, a behaviour dominated by bending stiffness for the as-built floors was noticed, while for their strengthened version the response was more shear-related. The only specimen showing a shear behaviour also for its as-built configuration was the roof pitch (DFpar-5), due to the weak and flexible connection between wall plate and rafters.

4. Analysis

4.1. In-plane deflection of the diaphragms

As can be observed from all the reported hysteretic cycles, for the strengthened diaphragms not only a great improvement in strength and stiffness is reached, but also high energy dissipation is achieved. This is due to the large amount of screws used to connect the plywood panels to the planks and the fact that diffused yielding and plasticization took place, together with friction among the panels.

Besides, from the reported deformed shapes (Fig. 12), with the exception of the roof sample (Fig. 13), the behaviour from flexural became more shear-related after strengthening the diaphragms. This could be explained by imagining that instead of a number of small panels, a single large plywood element is connected to the original floor: in this way, a real shear action can be obtained, with the fasteners around the perimeter that are responsible for the initial stiffness and the final deformation. Such a purely theoretical diaphragm, however, would not allow high energy dissipation due to the limited number of fasteners and the high stiffness which would be required to them.

For the tested specimens the plywood overlay is composed of a number of panels: even if the stiffness of the diaphragm is less than the one that could be reached with a single panel, the energy dissipation increases due to friction among the panel edges, and above all to the large number of fasteners. In this case, by renouncing to a slightly higher stiffness, the panels are allowed to small sliding and rotation, which can bring all the fasteners across the diaphragm into play.

Some more observations regarding the different behaviour of the as-built and strengthened versions are of interest. As already mentioned before, it is quite common to consider timber diaphragms as shear walls, characterizing them with the equivalent shear stiffness G_{eq}. However, for floors with continuous planks like the Dutch ones, this representation might be questionable.

The deformed shape of the diaphragms was recorded at different levels of displacement. For the strengthened floors the shear behaviour is recognizable and G_{eq} can properly represent their stiffness at a certain level of displacement, while for the as-built configurations the deformation is mainly flexural. This is shown in Fig. 12, which reports the deformed shape of the diaphragms at approximately 10 and 30 mm displacement on top of the floor. It is interesting to notice that, for larger displacement, even if the behaviour is still shear-related, it is possible to perceive the mutual sliding of the panels. This fact can be recognized also for the roof sample (Fig. 13), in which the panels undergo slightly larger displacements than the purlins.

According to the static schemes shown in Figs. 2–4, G_{eq} can be derived as follows:

$$\delta = FL/(G_{floor}Bt) \rightarrow G_{eq} = G_{floor}t = FL/(\delta B) = KL/B \tag{2}$$

The advantage of such a formulation is that G_{eq} is a size-independent measure of stiffness for the floors, and for the strengthened configuration it can be properly used taking into account an average shear deflection throughout the diaphragm. Therefore, also for strengthening interventions with different dimensions of the floors, it is possible to define a reference value of the improvement in stiffness that can be gained with the adopted technique.

On the contrary, for as-built flexible floors with these characteristics, the definition of G_{eq} appears not to be suitable, because the behaviour is mostly flexural. It is only possible to derive this parameter for the tested samples at different displacements, but it would not be representative for the observed behaviour and also not size-independent. This can immediately be noticed considering that in a cantilever static scheme, such as the test setup one, the flexural deflection is linked to the cube of the span, whereas the shear deformation is linearly related to it. Thus, a scale effect is present when bending deformation is governing. The use of G_{eq} could therefore lead to wrong estimations of the actual stiffness and deflection of the diaphragms for other sizes than tested. Furthermore, it has to be considered that longer spans and/or smaller structural elements imply a more relevant contribution of resisting moments given by nail couples and friction effects.

However, with the application of relations from mechanics it is still possible to correctly represent the behaviour of these as-built floors. In the calculations, for loading parallel to the joists, friction in the planks as well as the small resisting bending moment given by each nail couple according to the applied rotation, were neglected given their limited contribution; this assumption is valid especially when the specimens undergo sufficiently high forces. For the other direction, however, the influence of friction is noticeable from the hysteretic cycles of the samples tested orthogonally to the joists. This is because in the loading and unloading phases an almost infinitely rigid behaviour occurs, before the stiffness related only to the geometry of structural elements composing the floor comes into play. This is because the diaphragms are so flexible, that they can undergo only very limited horizontal loads (up to 4 kN), and therefore even this friction's contribution can play a non-negligible role.

The same considerations can be drawn with regard to the resisting bending moment given by nail couples. The calculation in this case is more complex, because these moments are nonlinearly depending on the rotation of the joists, which in turn is due to the moments themselves, leading to an iterative procedure similar to the one presented in [15].

Therefore, the calculation of the diaphragms' deflections remains still possible. Together with the geometrical dimensions of each as-built sample, the following material properties were adopted from test results:

- $E_{planks} = 11210$ MPa;
- $E_{joists} = 13120$ MPa;
- $I_{planks} = (t_{planks}\, w^3_{planks}/12)n_{planks}$ when loading parallel to the joists;
- $I_{joists} = (h_{joists}\, b^3_{joists}/12)n_{joists}$ when loading perpendicular to the joists;

By substituting the amount of horizontal load reached during the test at again 10 and 30 mm displacement, the deflections on top of the floors can be obtained with the following relations:

- Loading parallel to the joists:

$$\delta = FL^3/(3EI_{planks}) \tag{3}$$

- Loading perpendicular to the joists (see Fig. 14 for better clarification):

$$\delta = FL^3/(3EI_{joists}) - \sum_i [(M_iX_i^2)/(2EI_{joists}) + (M_iX_i)(L - X_i)/(EI_{joists})]n_{joists} \tag{4}$$

$i = 1 \cdots n_{couples}$

The results of these calculations are shown in Table 7.

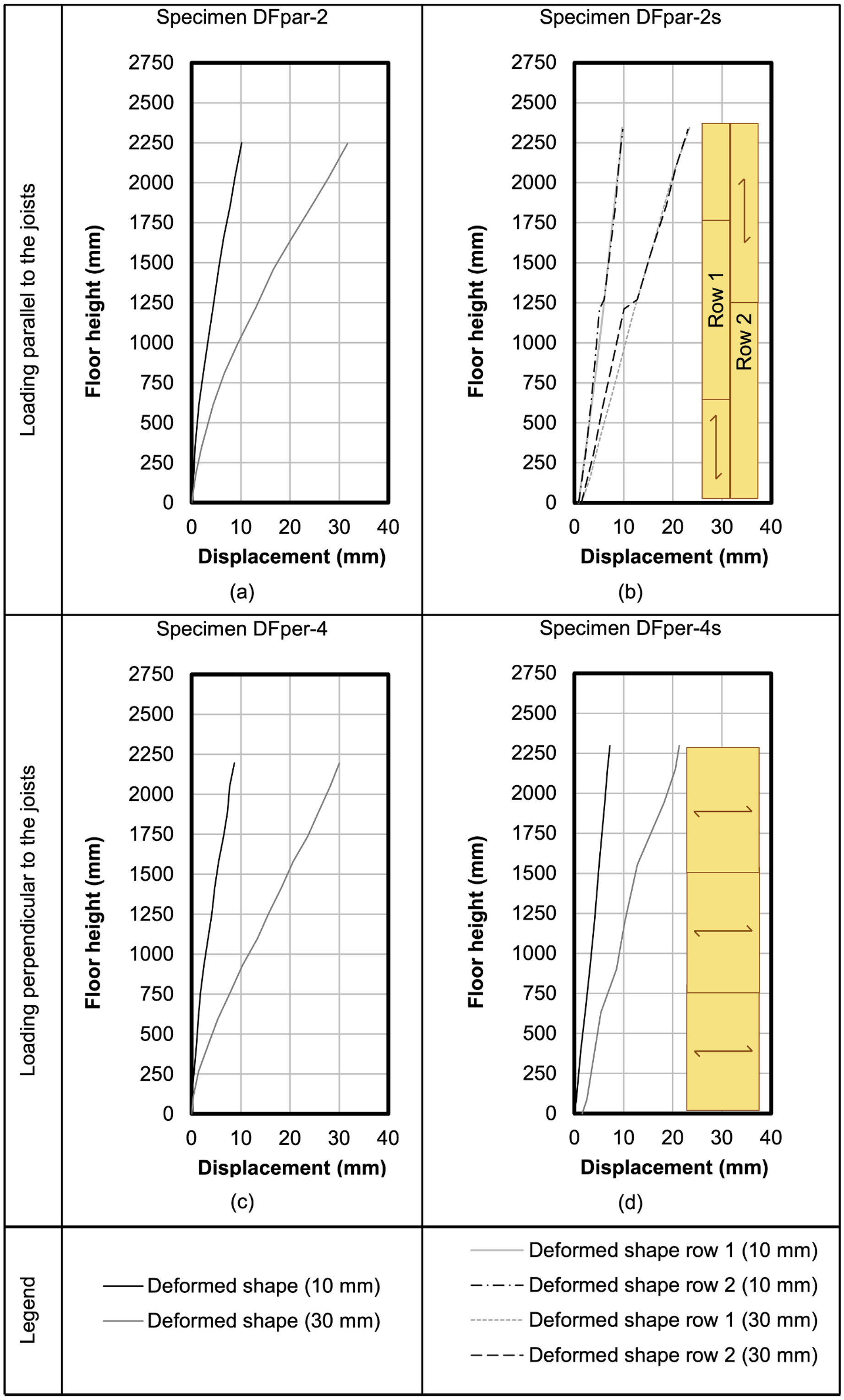

Fig. 12. Examples of in-plane deformed shape: as-built (a) and strengthened (b) floors loaded parallel to the joists (reported for two adjacent rows of panels); as-built (c) and strengthened (d) floors loaded perpendicular to the joists.

The flexural behaviour observed during the test is then also confirmed by these analytical calculations, especially for the floors loaded parallel to the joists. For the specimens tested orthogonally to the joists, the values show a slight scatter, probably because for the calculation of the bending moments given by nail couples always the same average backbone curve was used (Fig. 14), derived from the rotational tests on plank-joist connections. The obtained values are nevertheless still comparable to the ones experimentally recorded.

The average backbone curve was calculated according to Foschi's exponential model [30] adapted for torsional behaviour, by means of the following equation calibrated on the whole set of results (10 reference backbones, $R^2 = 0.84$):

$$M = (M_0 + K_1\varphi)[1 - exp(-K_0\varphi/M_0)] \tag{5}$$

with:

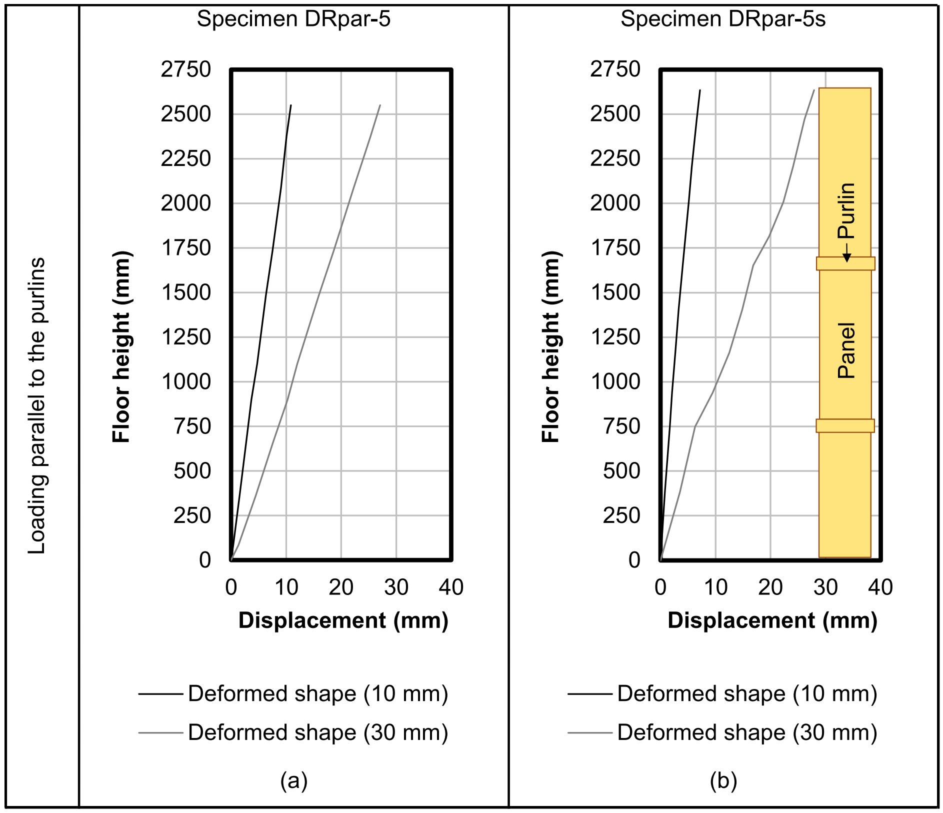

Fig. 13. In-plane deformed shape of as-built (a) and strengthened roof (b).

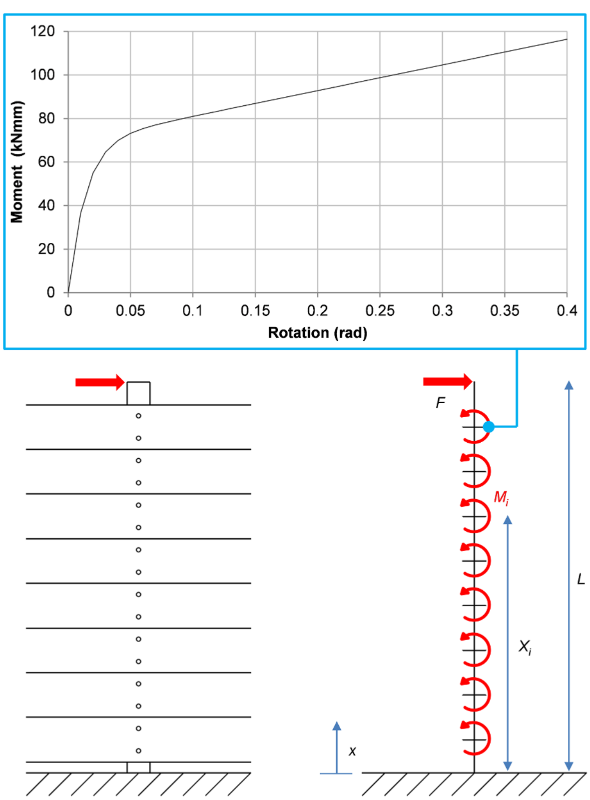

Fig. 14. Scheme for the analytical calculation of the floor deflection in the direction orthogonal to the joists: for the moments given by nail couples, the shown average backbone curve calibrated from the conducted experimental tests on plank-joist connections was used.

- M_0 = 69.2 kNmm
- K_0 = 5059.7 kNmm/rad
- K_1 = 118.0 kNmm/rad

4.2. Influence of the displacement on G_{eq}

The standards mentioned in section 1.3 define a single value of G_{eq}, which can be used by designers for their calculations and modelling of existing diaphragms to be retrofitted. However, given the presence of nonlinearities, it has to be specified at which displacement this value refers. Therefore, in this work it was chosen to describe the response of the diaphragms by means of two values, taking into account a very initial behaviour (e.g. for seismic serviceability limit states) and the stiffness at a reasonably high level of displacement (closer to global ultimate limit state for a building). These values were already adopted for the calculations of the experimental stiffness of each diaphragm (Section 3.3).

Since only strengthened floors displayed a shear-related behaviour, the results of the calculation of G_{eq} according to Eq. (2) are reported for them in Table 8; a graphical comparison is also given in Fig. 15a. As can be noticed, nonlinearities have indeed a quite remarkable influence on the final results, as expected: the value of the equivalent shear stiffness at 20 mm displacement is on average 50% less than the one calculated at 2 mm. However, the improvement in the stiffness of the diaphragms (Fig. 15b) is increasing at 20 mm for the most flexible ones, i.e. the two samples tested perpendicular to the joists and the roof pitch. This is an important aspect to be considered, because even if at larger displacements the equivalent stiffness is decreasing due to nonlinearities, the strengthened diaphragms are able to withstand lateral loads with an increasing improvement in stiffness (until the peak of force) with respect to the as-built ones. Considering that this happens for the most flexible specimens, the adopted strengthening technique appears to be effective and to guarantee a stable behaviour especially for the diaphragms that need these characteristics at most after their retrofitting.

Table 7
Comparison between the experimental and analytical values of displacement for as-built specimens loaded parallel to the joists.

Specimen name	Horizontal load (kN)	Experimental displacement (mm)	Analytical displacement (mm)
DFpar-1	4.0	10.1	10.6
	11.3	30.0	30.0
DFpar-2	5.0	9.5	9.6
	15.8	30.2	30.2
DFper-3	1.1*	10.3	11.1
	2.2*	30.0	29.7
DFper-4	1.0*	10.1	11.2
	2.5*	30.0	30.1

* after subtracting friction contribution, equal to ≈ 0.9 kN for sample DFper-3 and to ≈ 0.5 kN for specimen DFper-4

Table 8
Values of the equivalent shear stiffness for the strengthened specimens.

Specimen	Value at 2 mm (initial elastic, N/mm)	Value at 20 mm (equivalent elastic, N/mm)
DFpar-1s	3450	1250
DFpar-2s	4100	1850
DFper-3s	1350	750
DFper-4s	2500	1350
DRpar-5s	2400	950

As reported in section 1, the improvement in terms of stiffness for diaphragms strengthened with plywood panels was 5 to 20 times with respect to as-built ones. The obtained values are therefore in line with these reference ones, as can be observed in Fig. 15b: interestingly, this range of values is noticeable for both 2 and 20 mm reference displacements.

In general, the obtained values are similar to those derived from literature, and this can be considered as a further proof of effectiveness of the adopted strengthening technique: the as-built floors had very poor characteristics from the seismic point of view, but it was possible to improve them in such a way that they could perform similarly to diaphragms with larger structural elements and better properties.

4.3. Influence of the loading direction on G_{eq}

As can be noticed again from Table 8, also the loading direction appears to be a parameter that should be considered, independently of the reached level of displacement. The difference in stiffness when loading perpendicular to the joists is in fact quantifiable as 30 to 60% of the value obtained applying the force parallel to the joists. This might be partly caused by the lay-up of rectangular panels, but an important role is surely played by the initial wide difference in the response in the two directions for the as-built diaphragms. Hence, the use of a single value of G_{eq} might be not always appropriate to properly describe the in-plane behaviour of the diaphragms.

Furthermore, for the tested diaphragms this orthotropic behaviour was always present but a distinction before and after strengthening should be made:

- For the as-built specimens, the direction perpendicular to the joists displayed a very flexible behaviour with a low capacity to withstand the horizontal loads, especially compared to the orthogonal direction. This means that for as-built diaphragms the orthotropic behaviour has to be considered, because one direction of load is much weaker than the other;
- For strengthened specimens, the orthotropic behaviour is still present, even if it appears to be mitigated when increasing the deflection (see Table 8). For instance, sample DFper-4 s, strengthened with plywood panels and additional timber blocks between each couple of joists, displayed such a great improvement in its properties, that its equivalent shear stiffness is comparable to the values referred to the other direction, especially at larger displacements. Therefore, a proper strengthening method based on efficient transfer of shear forces strongly reduces the difference in the diaphragm's response

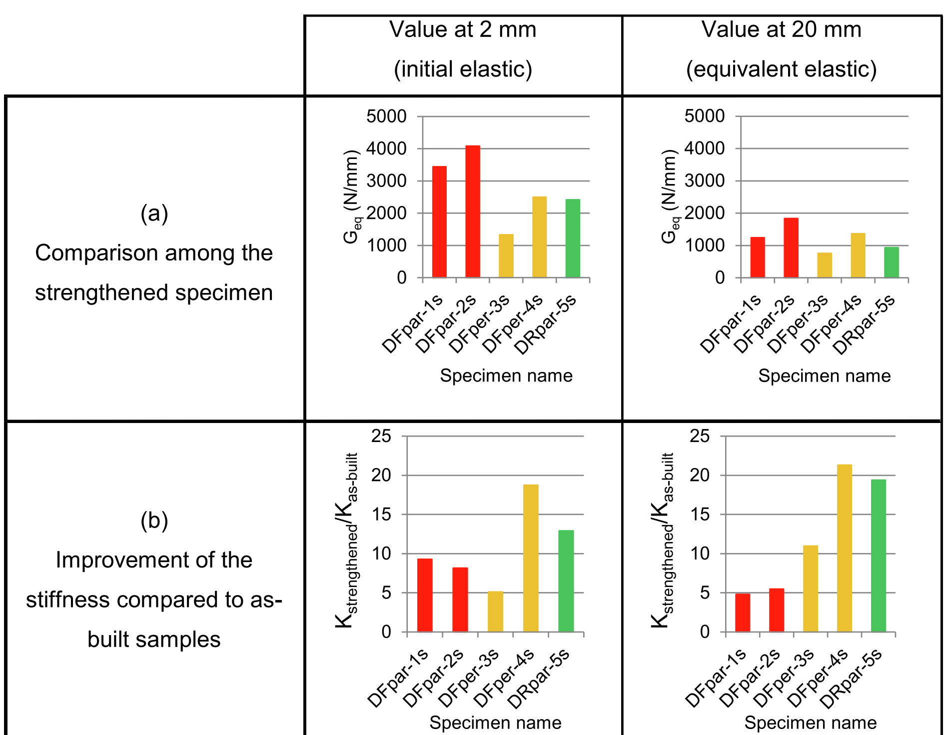

Fig. 15. Graphical comparison among the values of equivalent shear stiffness of the tested floor (a) and improvement in the experimentally recorded stiffness with respect to the as-built versions (b).

depending on the loading direction: this happens when it is managed to make the diaphragm act as a whole shear wall. If this behaviour is not completely enabled or guaranteed, however, the orthotropic response of the floor should be considered also for the strengthened configuration when analysing an historical building.

5. Conclusions

In this work, an experimental campaign and analytical investigation on the in-plane response of replicated as-built and retrofitted timber diaphragms was presented and discussed.

In order to reduce the scatter in mechanical properties between newly built specimens and the existing floors, original samples were extracted and tested. Basing on these results, new materials with close characteristics were ordered for an accurate replication.

Four specimens representing floors and one representing the pitch of a roof were firstly tested in their original non-strengthened configuration: results showed a very flexible behaviour and confirmed the need to develop a simple and effective strengthening technique to improve their in-plane response.

Therefore, the same specimens were strengthened by means of plywood panels screwed to the existing sheathing. Some further measures, like fastening of additional timber blocks or steel angles, were adopted for the most flexible diaphragms. A significant improvement in strength, stiffness and energy dissipation was obtained with this technique, and a change in the floor response was observed, from mainly flexural to shear-related.

It was demonstrated that for flexible floors with continuous planks the use of an equivalent shear stiffness is not appropriate, and that the deflection can be calculated by means of the usual relations from mechanics. On the contrary, for the strengthened diaphragms G_{eq} can be properly defined: an initial and an equivalent elastic value was proposed for the adopted technique, in order to take into account the dependency of this shear stiffness on the reached displacement.

The results displayed also that the loading direction has an influence in the response of the diaphragms and therefore should be taken into account, especially when analysing as-built flexible floors. However, with further strengthening measures, such as the additional insertion of timber blocks between the joists in our case, this orthotropic behaviour can be strongly reduced.

Further research is ongoing to firstly compare the obtained results uniformly with those present in literature, and to define a more refined and general analytical model, allowing to assess the in-plane behaviour of timber diaphragms retrofitted with the presented technique. This calculation model aims to avoid the definition of the equivalent shear stiffness, which appeared to be too general, optimizing then the strengthening interventions and the description of the in-plane response of existing timber diaphragms in historical buildings.

CRediT authorship contribution statement

Michele Mirra: Conceptualization, Methodology, Formal analysis, Investigation, Writing - original draft, Visualization. **Geert Ravenshorst:** Conceptualization, Methodology, Supervision, Writing - review & editing, Project administration, Funding acquisition. **Jan-Willem van de Kuilen:** Writing - review & editing, Project administration, Funding acquisition.

Declaration of Competing Interest

The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

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