# RACKING BEHAVIOUR OF LIGHT PREFABRICATED CROSS-LAMINATED MASSIVE TIMBER WALL DIAPHRAGMS SUBJECTED TO HORIZONTAL ACTIONS

# WANDSCHEIBENTRAGFÄHIGKEIT LEICHTER VORGEFERTIGTER KREUZWEISE VERKLEBTER MASSIVHOLZELEMENTE BEI HORIZONTALER BEANSPRUCHUNG

# PORTANCE DE PANS DE MUR PREFABRIQUES MASSIFS ET LEGERS EN BOIS LAMELLE-CROISE SOUS CHARGEMENT HORIZONTAL

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## SUMMARY

Light prefabricated timber elements and structures become increasingly popular in the European building market. Especially multi-storey timber structures up to five storeys are in trend, now. Hereby profound attention has to be paid to buildings located in earthquake prone areas of Middle and South Europe. Appropriate guidelines for the design have to be set to assure sufficient seismic resistance. It is obvious in this context that there is a need for development of an European standard covering cyclic testing of construction parts and wall diaphragm segments. The new standard should also include the criteria for determination of the limitations of inter-story drifts according to the concept of performance based earthquake engineering design.

In the paper presented the importance of experimental investigations related to the earthquake response of anchored horizontally loaded wall diaphragms for earthquake safe design of light prefabricated timber buildings is addressed. Cyclic tests of single panels and testing of panel assemblies on a shaking table are presented. The benefits of the experimentally obtained data for the design process are explained. The importance of a proper consideration of the anchorage / boundary conditions, of the influence of vertical and type of horizontal loading is evident from comparison of the results for the different panels. The dynamic tests have proven the ductile behaviour of the connections in case of both diaphragm built-ups presented and further showed a good correlation with the results from quasi-static tests.

## ZUSAMMENFASSUNG

Leichte vorgefertigte Holzelemente und Bauwerke werden auf den europäischen Baumärkten zunehmend nachgefragt. So liegen jetzt insbesondere mehrgeschossige Holzbauten bis zu fünf Geschossen im Trend. Vertiefte Beachtung ist hierbei Bauwerken zu widmen, die in erdbebenanfälligen Gegenden, insbesondere in Mittel- und Südeuropa liegen. Es müssen geeignete Vorgaben implementiert werden, um eine ausreichende seismische Widerstandsfähigkeit garantieren zu können. In diesem Zusammenhang ist es offensichtlich, dass eine Notwendigkeit für die Entwicklung eines umfassenden europäischen Normenwerks bezüglich zyklischer Versuche an Bauwerksteilen und Wandscheibenabschnitten besteht. Die neue Norm sollte auch die Kriterien zur Bestimmung der Grenzen von Geschossverschiebungen nach dem Konzept einer verhaltensbasierten Erdbebenbemessung umfassen.

Der vorliegende Aufsatz beleuchtet die Bedeutung experimenteller Untersuchungen zum Erdbebenverhalten verankerter Wandtafeln im Hinblick auf eine erdbebensichere Bemessung leichter, vorgefertigter Holzgebäude. Es werden zyklische Versuche an einzelnen Elementen sowie Schütteltisch-Versuche an Element-Zusammenbauten vorgestellt. Die Vorteile der experimentell erhaltenen Ergebnisse für die Bemessung werden erläutert. Die Bedeutung der korrekten Berücksichtigung der Lagerungs- / Randbedingungen, des Einflusses der vertikalen Belastung und der Art der horizontalen Beanspruchung wird aus dem Vergleich der unterschiedlichen Elemente evident. Die dynamischen Versuche belegen einerseits das duktile Verhalten der Verbindungen bei beiden Wandscheibenaufbauten und zeigen zum anderen eine gute Korrelation mit den Ergebnissen der quasi-statischen Versuche.



## RESUME

Les éléments et ouvrages préfabriqués légers sont de plus en plus demandés sur le marché européen de la construction. En particulier, les bâtiments en bois à plusieurs étages (jusqu'à cinq) sont à la mode. Une attention particulière doit être prêtée au ouvrages construits dans les régions à haut risque de tremblements de terre, spécialement en Europe centrale et du sud. Des directives appropriées doivent être développées afin de garantir une résistance suffisante aux séismes. Dans ce contexte, il est évident qu'il faut développer une norme européenne sur les essais cycliques sur les éléments et segments de murs. Cette nouvelle norme devrait également contenir les critères permettant de déterminer les limites de déplacement d'un étage selon le concept du dimensionnement parasismique basé sur le comportement.

L'article présent traite l'importance des études expérimentales sur le comportement sismique de panneaux de mur ancrés quant au dimensionnement parasismique de bâtiments préfabriqués légers en bois. Des essais cycliques sur des panneaux individuels sont présentés, ainsi que des essais à la table de vibration sur des assemblages de panneaux. Les avantages des résultats expérimentaux pour le dimensionnement sont expliqués. La comparaison des différents éléments met en évidence l'importance de la prise en compte judicieuse de l'appui, des conditions aux limites, de l'influence du chargement vertical et du type de chargement horizontal. Les essais dynamiques prouvent d'une part le comportement ductile des éléments de liaison des deux pans de murs individuels présentés et démontrent une bonne corrélation avec les résultats des essais quasi-statiques.

KEYWORDS: Timber structures, light prefabricated structures, shear wall diaphragms, racking strength, shear stiffness, seismic parameters, ductility class, seismic behaviour factor

# 1 INTRODUCTION

Wooden panel structures have a general reputation to perform well when subjected to strong earthquakes for the following reasons: large strength-to-weight ratio of wood, enhanced strength under short term loading, high degree of structural redundancy and energy dissipation. Nowadays the non-professional general opinion is that prefabricated structures are all wood based structures and that they behave well during seismic actions due to their lightness. But some assembled materials which form the main load-bearing structure show brittle failure when ultimate load is reached. Therefore they are not able to dissipate seismic energy and to behave in a ductile manner. Those systems have to be regarded very carefully during the design process. In particular in combination with other ductile construction materials and elements such elements have to be prevented against failure. Hence, design rules have to limit the load bearing capacity of ductile elements against brittle ones. With such a combination of strengths the brittle elements are prevented against failure and the joint ductility of the compound structure can be taken as the basis of the nonlinear behaviour of ductile elements.

Comprehensive research was performed in recent years at University of Ljubljana to enable a better understanding of the response of different prefabricated timber wall systems exposed to earthquake action (Dujic, 2001; Dujic and Zarnic, 2002; Dujic et al. 2004; 20005). Recently, Division of Timber Construction of MPA University of Stuttgart joined the research program with testing and seismic evaluation of timber frame wall systems where gypsum fiber boards are used as sheathing material for transferring the horizontal loads in the wall diaphragms (Reinhardt et al., 2005; 2006)

Timber construction as such is not a guarantee for adequate seismic performance. The reputation of the good behaviour stems from many positive post earthquake observations, but there are also cases where the behaviour of timber buildings was not as expected (Fig. 1). Experiences from the recent earthquakes show that damages and even collapses of wooden buildings do not endanger human lives to the same extent as it is the case with buildings constructed from heavy massive materials such as concrete or masonry (Table 1).

*Figure 1: Northridge, ZDA, 1994; global failure because of the effect of soft storey mechanism*

*Table 1: Casualties in some recent earthquakes*

| Earthquake | Magnitude | Buildings Shaken | Persons killed (Approximate) | |
|---|---|---|---|---|
| | | | Total | Wood-frame |
| Alaska, 1964 | 8.4 | | 130 | <10 |
| San Fernando, 1971 | 6.7 | 100,000 | 63 | 4 |
| Edgecumbe, 1987 | 6.3 | 7,000 | 0 | 0 |
| Saguenay, 1988 | 5.7 | 10,000 | 0 | 0 |
| Loma Prieta, 1989 | 7.1 | 50,000 | 66 | 0 |
| Northridge, 1994 | 6.7 | 200,000 | 60 | 16 + 4 |
| Hyogo-ken Nambu, 1995 | 6.8 | 8,000 | 6,300 | 0 |

In this paper the importance of an experimental investigation of the earthquake response of anchored massive wooden panels for earthquake safe design of wooden buildings is addressed. A description of the cyclic testing of single panels and testing of panel assemblies on a shaking table are presented to explain the benefits of experimentally obtained data for the design practice. The development of experimentally supported knowledge about the response of solid wooden houses is illustrated by description of test procedures and by a brief presentation of the test results. The importance of a proper consideration of the boundary conditions and of the influence of vertical and type of horizontal loading is revealed.

A significant part of the European territory, especially in the South-East, is endangered by earthquakes (Fig. 2); on the other hand wooden houses are becoming increasingly popular for residential and business use in theses regions. The post earthquake observations of damaged wooden houses and the analysis of experimentally tested structural elements developed the world wide knowledge about response of wooden buildings on earthquake and strong wind. One of the major problems of understanding is related to boundary conditions and the influence of vertical loading on building elements. Learning from the on-site observations and from experimental investigations, researchers have developed different test protocols and test set-ups trying to simulate the natural behaviour of buildings as realistically as possible.

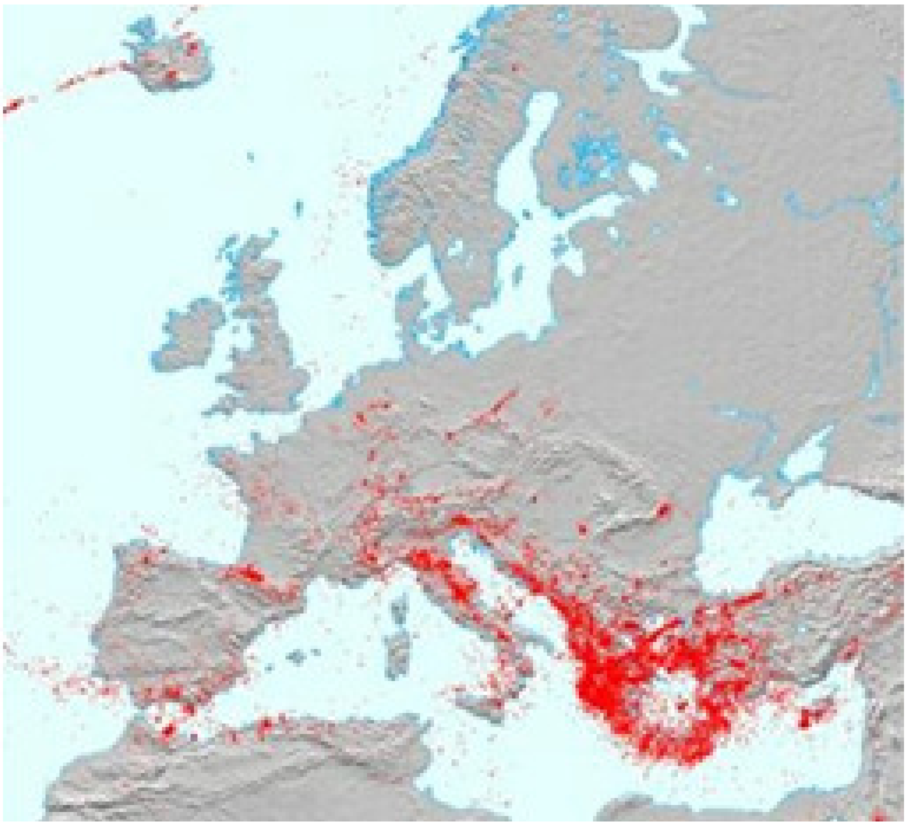


*Figure 2: Seismic activity in Europe 1973 -2002; Magnitude > 3*

## 2 SEISMIC RESISTANCE OF WOODEN BUILDINGS

The seismic resistance of buildings is governed by three main parameters: stiffness, strength and ductility on the level of the constituent structural and non-structural elements as well as on the level of the whole building. Assessing the behaviour of wooden structures one has to bear in mind that wood is an anisotropic material where strength and stiffness properties vary with the orientation of the wood fibers. High stiffness of structural elements provides a limitation of the lateral deflections, but vice versa attracts the increase of forces being proportional to stiffness. Element strength makes the building resistant to seismic forces and prevents the development of damages up to a certain limit. Ductility enables a reduction of seismic forces, but the interaction of structural and non-structural elements of different ductility has to be studied carefully. In addition, the ability of structural elements and of whole buildings to dissipate energy imposed by earthquake diminishes the destructive action of an earthquake. All these parameters are inter-related (Rainer and Karacabeyli, 1999).

Seismically well designed buildings encompass the benefits of all cited parameters and their interrelations.

In the case of wooden buildings especially the appropriate design of joints and anchorages contributes to a structure with high potential for dissipation of energy and thus increased resistance to earthquake action. Typical wooden structures have a fundamental period of 0,2 s to 0,8 s. During an earthquake, the stiffness of a structure is decreasing due to softening of joints and anchors leading to an increase of the fundamental periods. Resonance is likely to occur on rather soft soils. Typical damping values of heavy timber structures are in the range between 3% and 5%, while in the case of platform frame structures the damping values are in the range between 3% to 20%. The most important parts regarding the earthquake response of timber structures are the connections and the anchors of the wooden panels. Slender bolts, dowels, timber rivets or nails should be used for connection of wooden panels to enable ductile behaviour of the entire building. According to the basic principles of capacity design, connections should be weaker than the members they connect. In the case of wooden structures connections leading to stresses in the wood perpendicular to grain should be avoided in order to mitigate damages due to splitting of wooden elements. The hysteretic behaviour of connections and anchors is governed by the material properties of the wood and the fasteners. The hysteretic lines are usually smooth without well defined yield points. The loops are mildly to severely pinched (Fig. 3). The pinching effect is caused by the loss of stiffness at small deformations. A cavity around the fastener is formed by wood crushing.

The behaviour of a building during an earthquake can be predicted by mathematical models that usually depend on various parameters, which can only be defined by experimental investigations of the constituent elements of the (wooden) building. The experiments comprise cyclic testing of connections, followed by cyclic testing of anchored panels and finally by shaking tests of structural assemblies and/or prototypes of entire buildings (Hristovski and Stojmanovska, 2005).

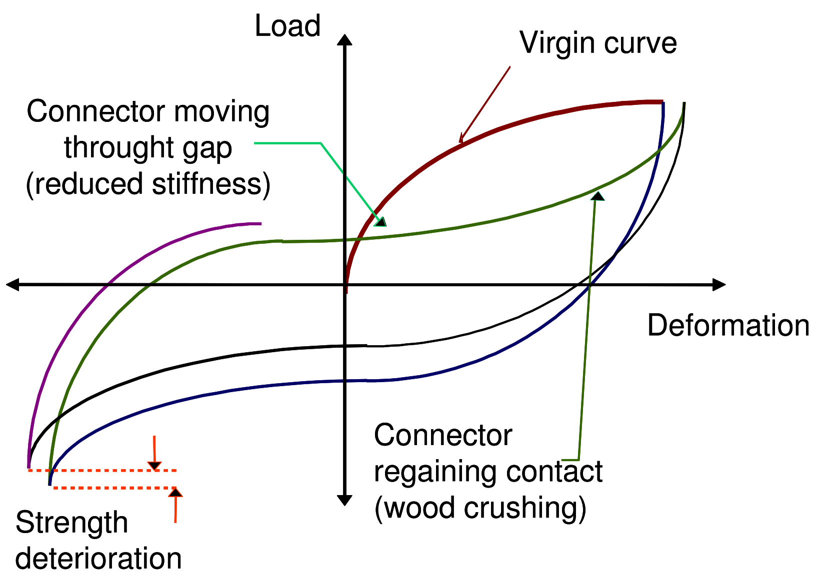



*Figure 3: Hysteretic behaviour of a ductile timber connection capable to dissipate energy.*

Testing protocols may be standardized or developed individually. The main task of the testing consists in the simulation of a realistic response mechanism. This necessitates the simulation of realistic boundary conditions, taking into account the influence of the magnitude of the vertical load and of the influence of the horizontal loading protocol. In this paper the brief insight in cyclic and shake table testing of anchored cross-laminated (x-lam) solid wooden panels is presented.

## 3 EXPERIMENTAL INVESTIGATION ON CROSS-LAMINATED SOLID WOOD A DIAPHRAGMS

The investigations reported in this paper have been carried out within a project supported by the Austrian Company KLH Massivholz GmbH. The main target of the project was to obtain reliable data on the principle behaviour of massive cross-laminated (x-lam) wooden wall panels/diaphragms that will contribute to the design and construction of seismically resistant and safe timber buildings.

In the scope of the project many shear wall racking tests were carried out at University of Ljubljana, Faculty of Civil and Geodetic Engineering, on walls with a length of 2.44 m and story height at combined constant vertical load and monotonous or cyclic horizontal load applied according to different loading protocols.

The wall panels were tested at various boundary conditions which enabled wall deformation features from cantilever behaviour up to a pure shear deformation. The influences of the boundary conditions, of the magnitudes of vertical load and of the type of anchoring systems were evaluated with respect to deformation mechanisms and racking strengths of the wall segments. The differences in the mechanical behaviour between monotonic and cyclic responses were studied.

Additionally, two full-scale one story element assemblies/built-ups have been constructed and tested on the shaking table at IZIIS Laboratory in Skopje, Macedonia, in order to investigate the response of massive x-lam wooden diaphragms under earthquake excitation. The basic idea was to correlate the results of the quasi-static and of the shaking table tests. The seismic tests were performed with panel assemblies, also termed models, built-ups, constructions, consisting of two parallel walls of length 2.44 m and story height linked together with wooden ceiling. The assemblies were tested dynamically with harmonic and seismic excitation. The main task was to obtain the mechanical properties of the dynamic responses of the tested models. Dynamically developed failure mechanisms on the wall diaphragms should confirm the validity of the boundary conditions set at quasi-static racking tests. With an establishment of such relationships, dynamic properties also for other types of light load-bearing systems could be defined on the base of known quasi-static cyclic or monotonic mechanical properties.

### 3.1 Cyclic testing of diaphragms

Following the experiences obtained from testing of masonry diaphragms, a versatile shear wall test set-up was developed and installed at UL FGG in 1999 (Fig. 4). The main idea of the test apparatus is to use a gravity load induced by ballast as a constant vertical load and a displacement controlled hydraulic actuator as a driver of the cyclic horizontal load. The main challenge is to simulate realistic boundary conditions that may occur during the action of an earthquake. In reality, the boundary conditions may change during an earthquake excitation because of changes of the building characteristics due to development of damages. Therefore, the testing device should allow the altering of boundary conditions from one to another test run. Following this idea, a set-up was perceived which can be easily adapted to various boundary conditions to be applied to diaphragms.

*Figure 4: Setup for cycling testing of panels*

Basically, three major cases of boundary conditions are most likely to appear in reality (Fig. 5):

a) shear cantilever mechanism, where one edge of the panel is supported by the firm base while the others can freely translate and rotate ("Case A")

b) restricted rocking mechanism, where one edge of the panel is supported by the firm base while the others can translate and rotate as much as allowed by the ballast that can translate only vertically without rotation ("Case B")

c) shear wall mechanism, where one edge of the panel is supported by the firm base while the other can translate only in parallel with the lower edge and rotation is fully constrained ("Case C")

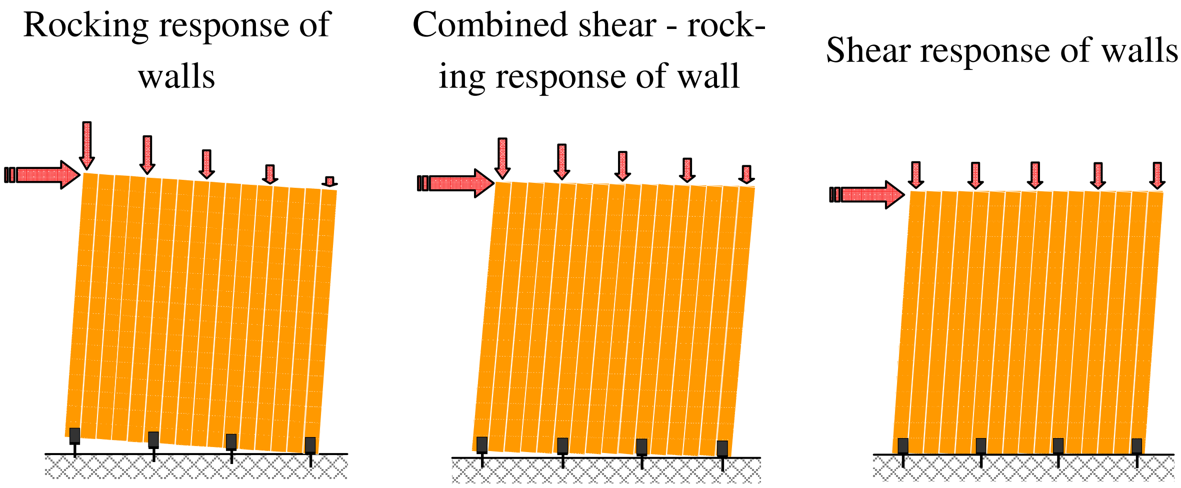
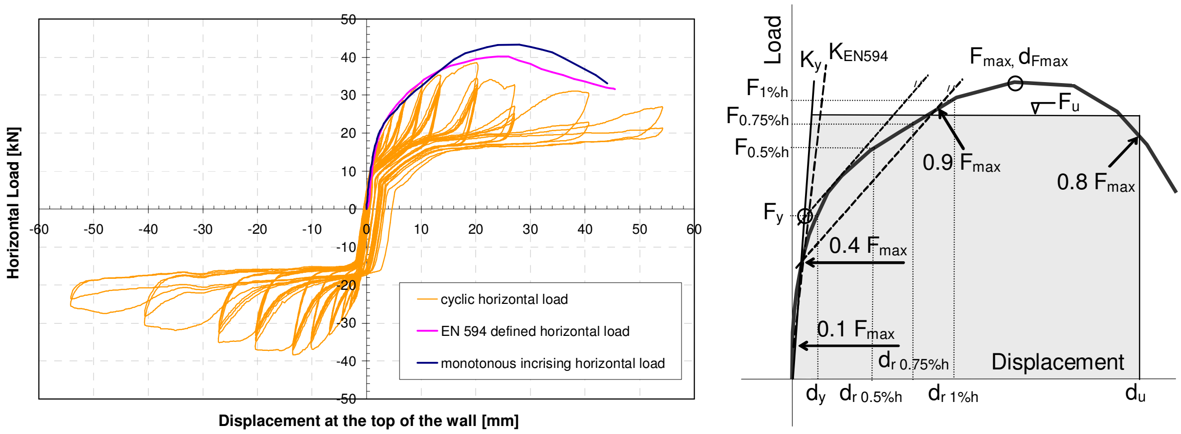



*Figure 5: Typical responses of wooden wall diaphragms exposed to combined vertical and horizontal load*

In "Case A" and "Case B" the diaphragms are exposed to a constant vertical load at every stage of the cycling excitation or horizontal deformation induced along the upper edge where the ballast is acting. In "Case C" the vertical load increases when the panel tends to uplift due to displacements along the upper horizontal edge. The advantage of the proposed test procedures "Case A" or "Case B" is avoiding the boundary conditions of the "Case C". In practice, "Case A" represents mostly the behaviour of narrow panels and of diaphragms located in the attic and vertically loaded only by flexible roof constructions. The "Case B" is typical for panels carrying the floor construction above it and the "Case C" is typical for an infill embedded in a stiff surrounding frame.

The complete information about the lateral resistance characteristics of wooden diaphragms and their anchoring can be obtained from responses both to monotonous and cyclic lateral loading with a proper combination of vertical forces (Fig. 6). The loading protocol of EN 594 is sufficiently covering the monotonous loading of wall panels. Unfortunately, the loading protocol of EN 12512 deals only with the cyclic testing of particular joints made with mechanical fasteners, what is an insufficient tool for evaluation of the behaviour factor "q" needed for design of earthquake resistant buildings. Similarily, the ISO standard 16670 addresses exclusively joints; the proposed loading protocol, however, can be also used for testing of wooden wall diaphragms. The reason therefore is that in ISO 16670 ultimate joint displacement is used instead of yield slip (EN 12512) which is difficult to define. Since the ISO loading protocol is based on ultimate displacement it can forward a behaviour factor "q" as addressed in Eurocode 8. It is obvious that there is a need for development of an European standard that would cover both monotonous and cyclic testing of wall diaphragms. The new standard should also include the criteria for determination of limitations of inter-story drifts according to the concept of performance based earthquake engineering design.




a) b)

*Figure 6: Comparison of monotonous and cyclic response of solid wooden x-lam wall diaphragms with a length of 2.44 m and vertically loaded with 15 kN/m(a) and procedure for evaluation of the racking properties of a tested wall panel from monotonic response or from cyclic envelope (b)*

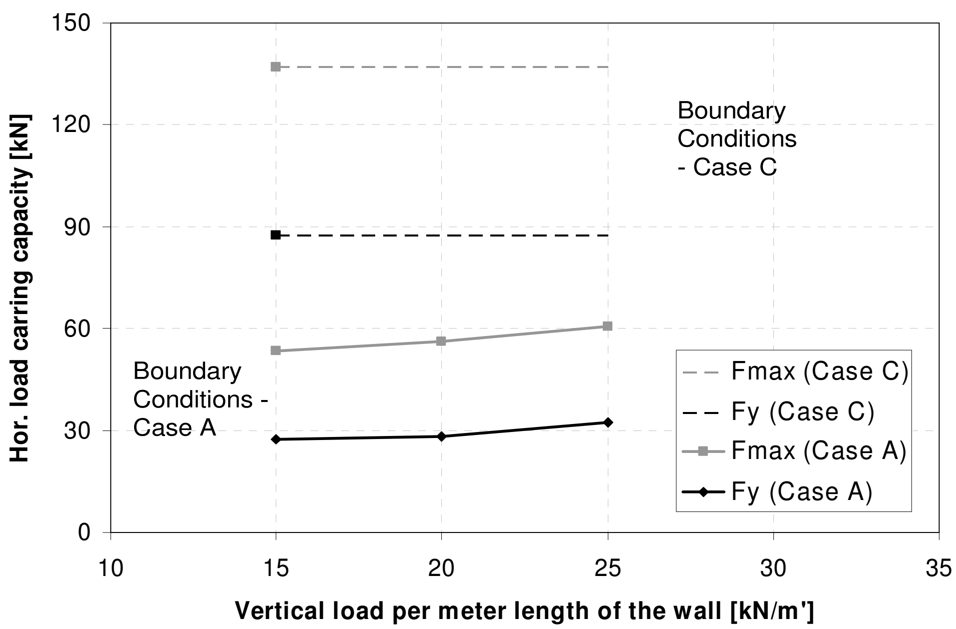



*Figure 7: Influence of vertical load intensity on load carrying capacity of solid wooden x-lam wall diaphragms with a length of 2.44 m*

A comparison of the responses of solid wooden x-lam wall diaphragms subjected to monotonous and cyclic loading (Fig. 6) illustrates the importance of cycling testing. In the case presented the load carrying capacity of the diaphragms, when exposed to cyclic loading, was 10 to 20% lower than the resistance of the panel exposed to monotonous loading. The cyclic response shows higher initial stiffness due to hardening of the fasteners (low-cycle fatigue) and lower ductility down to 50% of the ductility reached by monotonous loading. The example reveals that earthquake design of wooden buildings can not be properly performed without experimental data obtained from cyclic testing of wall diaphragms whereby different intensities of vertical loads have to be examined.

The graphs in Fig. 7 clearly illustrate the influence of vertical load intensity, both on the load carrying capacity and on the type of response mechanisms as discussed above. In the case of the stiff solid wood diaphragm, the shear mechanism does not develop in spite of varying boundary conditions from “Case A” to “Case B”. The shear mechanism was reached only when the boundary conditions were set to the “Case C”. Finally, however, cyclic testing does not provide all information on the behaviour of structural elements during an earthquake. The necessary information about the real earthquake response of anchored diaphragms can only be obtained by shake table tests.

### 3.2 Shake table tests of anchored x-lam wall diaphragms

Two full-scale models have been constructed and tested on the shake table at IZIIS Laboratory, Skopje, Macedonia in order to investigate the response of massive wooden x-lam wall panel systems under earthquake excitation. The basic idea was to deduce correlations between the results from the cyclic tests performed at the UL FGG in Ljubljana and the results from the shake table tests.

The shake table on which the structural elements were installed in order to be subjected to a biaxial earthquake motion is a prestressed reinforced quadratic concrete plate with an edge length of 5 m. Four vertical hydraulic actuators (total force capacity: 888 kN) located at four corners and at a distance of 3.5 m in both orthogonal directions support the table. The total weight of the shake table is 330 kN.

The maximum applicable accelerations are 0.5 g and 0.7 g in vertical and horizotal directions, respectively. The maximum possible displacements in vertical and horizontal directions are ± 0.050 m and ± 0.125 m, respectively. The frequency range is 0 to 80 Hz. The total bearing capacity for static loads is 720 kN. The shaking system controls five degrees of freedom of the table, two translations and three rotations. The analog control system controls displacements, velocity, differential pressure and acceleration of the six actuators.

Both tested specimens No.1 and 2 were made of cross-laminated solid wooden panels, produced by the Austrian company KLH Massivholz GmbH. Specimen No. 1 was assembled by two one-unit wall elements. Specimen No. 2 resembled specimen No. 1 with respect to outer dimensions. Contrary, however, both wall elements consisted of two parts screwed together at mid-length. Both constructions had a roof element with dimensions (thickness x length parallel to wall elements x length perpendicular to wall elements) of 16.2 cm x 244 cm x 210 cm. Both models were stiffened by two additional panels (dimensions: 190.5 cm x 272 cm x 9.4 cm) placed in lateral direction (see Fig. 8). The tested models were instrumented by LVDTs and accelerometers. LVDTs were used for measuring the horizontal displacements at the top of the elements, of the slip between the walls and the roof element, of diagonal deformations, of uplift of wall and of slip between the diaphragms and the reinforced concrete foundation.

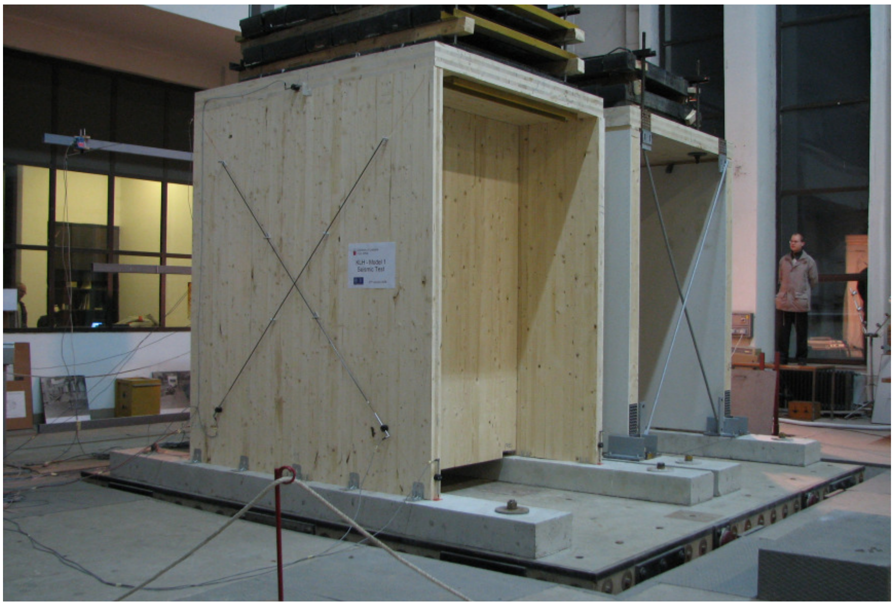


*Figure 8: Instrumented specimen models fixed on the shake table and loaded with dead load*

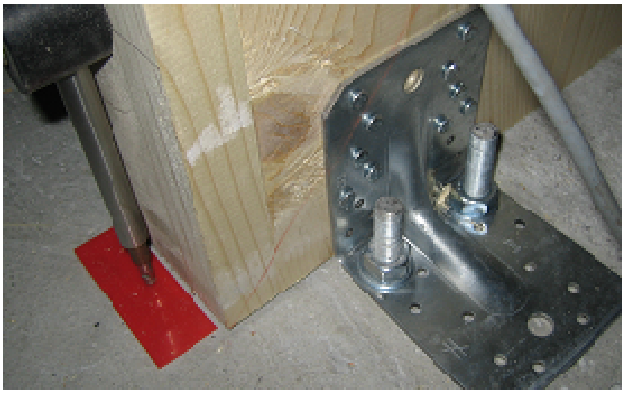


*Figure 9: Detail of wooden diaphragm foundation anchor.*

The wooden diaphragms were anchored to the RC foundation beams by ribbed steel angles of 105 mm mounted at every 60 cm of wall length and fixed to the KLH panels by 10 annually nails 4.0/60 mm and to the RC foundation beams by two bolts M12 (Fig. 9). The ballast applied atop each model corresponded to the usual weight of 3-storey structures. Preliminary estimations defined a mass of 5 tons acting on a single wall panel with length of 2.44 m representing a ground floor wall of the considered building type. Following this estimation and taking into consideration that each test configuration / model consisted of two wall panels and one roof panel an additional mass of 9.6 tons had to be applied on the roof panel. This was realised by 24 steel ingots (3 layers by 8 ingots, 400 kg each) which were placed and connected rigidly to the roof panel (Fig. 8).

After their installation on the shake table and their loading with ballast, the test procedure was equal for both structures. The test started with a measurement of the fundamental period of the models by ambient and low level random vibration tests. The results of both models matched well. For built-up No. 1 the natural frequencies 3.4 Hz and 7.4 Hz were obtained in lateral and in longitudinal direction, respectively, and for built-up No. 2, 3.6 Hz and 7.4 Hz were received. In the following several seismic test runs were carried out on the shake table. The peak ground acceleration was increased gradually from 0.06 g to 0.38 g. Hereby, the following earthquake excitations were applied: El Centro 1940, N-S, California, USA; Petrovac 1979, Montenegro; Kobe 1995 E-W, Japan; and Friuli 1976 E-W, recorded in Tolmezzo, Italy. All earthquakes excitations were applied with the real intensities. An exception was the Kobe earthquake where the intensity was scaled with a factor of 0.5 due to technical reasons.

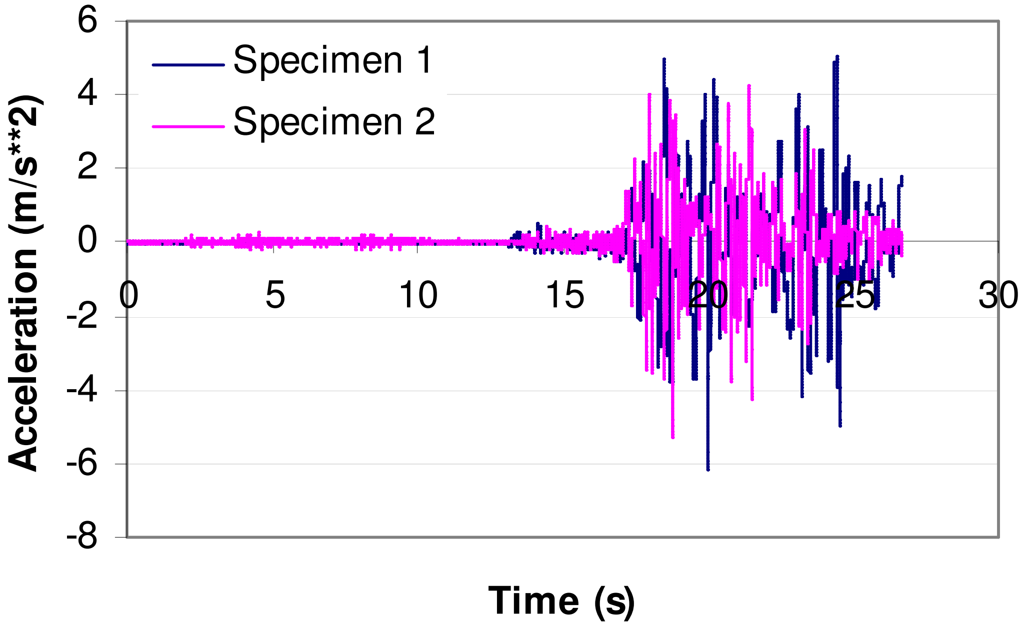



*Figure 10: Comparison of the measured acceleration response of both structure excited by the Petrovac 1979, Montenegro earthquake*

After excitations by the different earthquakes, the structures (built-ups) were also subjected to harmonic excitation with frequencies of 7.5 Hz and 5.0 Hz. Each earthquake and each harmonic excitation was followed by a low-level random vibration test in order to monitor the change of the natural frequencies due to eventual damages of the specimens. The maximum acceleration responses have been obtained from the Petrovac excitation. The displacements of the shake table were up to 68.75 mm, and the maximally obtained accelerations of the structure were $a_{max}$ = 0.6 g for specimen No. 1 and $a_{max}$ = 0.5 g for specimen No. 2. Due to the increased energy dissipation between the screwed panels of specimen No. 2 demonstrated a far more ductile behaviour than specimen No. 1. In Figure 11 relative displacements measured at the top edge of the walls of specimen No. 1 and specimen No. 2 in case of the El Centro earthquake record are given. Relative displacement of specimen No. 2 was 42% higher than relative displacement of specimen No. 1 while acceleration at the top was 57% higher. The relative displacement reached 0.4% of the story height or H/250, where H is wall height.

The shaking tests proved the non-linear behaviour of the massive wooden cross-layered wall panel diaphragms. The solid wooden wall panel itself behaves linear-elastically for most of the cases and, of course, as an orthotropic material. So, the main source of the non-linearity is certainly the connection, i.e. the steel anchorage system (Fig. 9).

Further, it can be assumed that the entire non-linear behaviour results from the behaviour of the nailed connection of the steel anchor (Fig. 9) to the wooden panel. The vertical bolts anchored into the concrete foundation along with the steel plate form a very stiff part of the connection, so that these components contribute hardly to any non-linear vertical and/or horizontal relative displacements between the RC foundation and the wooden diaphragm.

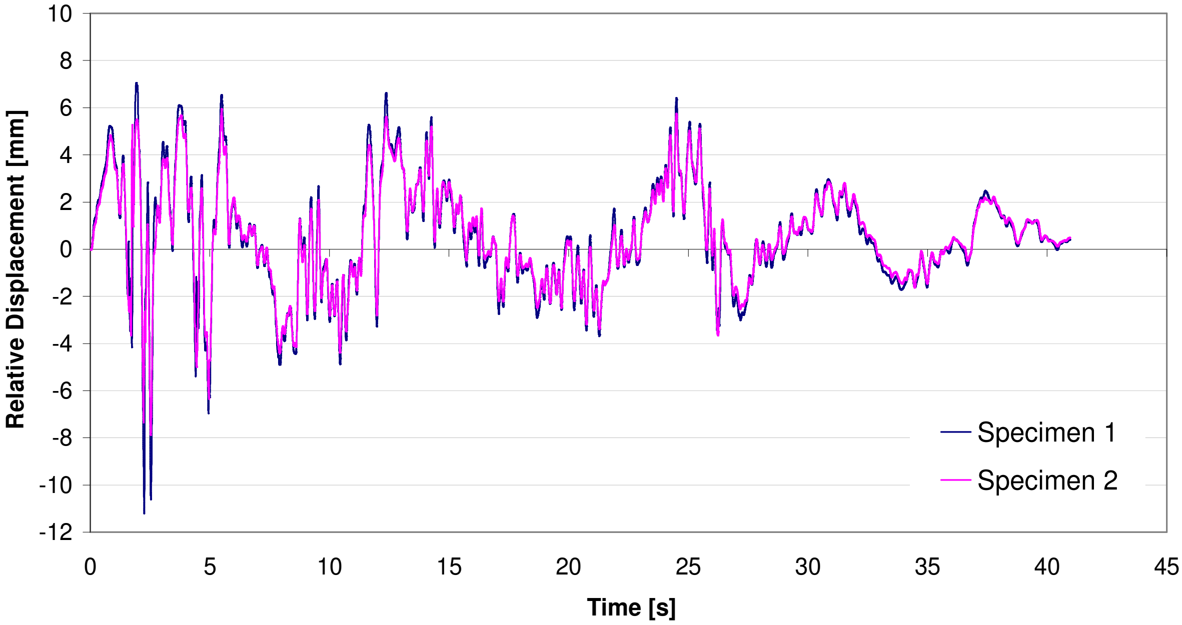



*Figure 11: Comparison of the relative horizontal displacement of both tested diaphragm specimens measured at the top edge of the walls during the excitation with the El Centro 1940 earthquake, the shaking table displacement was up to 106.25 mm*

## 4 CONCLUSIONS

The paper presents a first brief information on shake-table tests with diaphragms made of cross-laminated solid wood panel assemblies where anchorage and inter-panel connection systems were of prime interest of the investigations. Two full scale wooden wall assemblies termed specimen No. 1 and No. 2 were investigated. Both assemblies were made of cross-laminated massive wooden panels, produced by the Austrian Company KLH Massivholz GmbH.

Specimen No. 1 was assembled by two one-unit wall elements 244 x 272 x 9.4 cm and specimen No. 2 consisted of two wall elements of half width screwed together. Both test configurations had a roof element of size 244 x 210 x 16.2 cm. The built-ups were stiffened by two additional panels sized 190.5 x 272 x 9.4 cm and placed in lateral direction.

A series of dynamic tests has been performed in order to investigate the diaphragm behaviour of the massive wooden wall systems under seismic excitation. Several earthquake records, significant for Central and Southern Europe have been applied to the constructions. The maximum acceleration response has been the recorded for the Petrovac earthquake excitation, $a_{max} = 0.6g$ for specimen No. 1 and $a_{max} = 0.5g$ for specimen No. 2. Both of the test specimens behaved according to expectations and no visible damage was registered. In both diaphragm built-ups the seismic energy has been primarily dissipated by the mechanically fastened anchorage system vs. the foundation. In addition, specimen No. 2 also dissipated energy through the screwed connection in vertical wall direction between the two in-plane jointed wall units. The dynamic tests have proven the ductile behaviour of the connections in case of both diaphragm built-ups and a good correlation with the results from quasi-static tests was obtained.

The development of experimentally supported knowledge about the response of solid wooden houses is illustrated by description of test procedures and a brief information about results of some tests. A consistent experimental verification of any diaphragm behaviour starts with static tests of elements and joint details, is followed by cyclic testing of anchored wooden walls and concludes with real shake table tests of wall assemblies. The importance of a proper consideration of the boundary conditions and of the influence of vertical and type of horizontal loading is evident from comparison of the behaviour of differently tested and composed diaphragms. There is a need for further development of standard protocols for wooden wall diaphragms used for structures located in earthquake prone areas. New standards should implement the concept of performance based earthquake engineering design to obtain the experimental data needed for evaluation of the behaviour factor “q” and to set the values of story drifts defining the limit states of the story base shear diagram.

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