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Experimental Test on Novel Seismic Hold-Downs for Timber Structures

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Abstract

The structural use of wood in North America so far was mostly related to low-rise and mid-rise residential light-frame construction. Lately, mass-timber engineered wood products such as laminated-veneer-lumber (LVL) and cross-laminated timber (CLT), enable the use of wood in tall and large wood and wood-based hybrid buildings. Compared to other materials, one of the main advantages of using wood as structural material in tall buildings is the reduction in weight and the resulting reduction in foundation costs. Under high seismic loads, a lower mass, however, will also lead to less inherent resistance to overturning forces. The overturning resistance of tall mass-timber structures needs to be addressed, either by reducing the uplift forces or by improving the existing hold-downs. This paper summarizes experimental studies on a practical high-capacity hold-down solution for tall timber structures. The connection assembly is comprised of the Holz-Stahl-Komposit (HSK)™ system: perforated steel plates that are adhesively bonded to the wood. In the conventional HSK application, the perforated steel plates are welded to a section-reduced steel side plates that provide the required ductility. In the modified design investigated herein, duct tape was used to cover some rows of holes in the perforated steel plate. This arrangement allows the perforated steel plates to yield inside the wood, while at the same time preventing bucking of the hold-down. In the first step of the research presented herein, small-scale static and cyclic tests on perforated steel plates embedded in CLT panels with different numbers of rows covered with duct tape, were conducted. In the second step, full-scale tests on in CLT panels were conducted. Based on the results, it can be concluded that the modified HSK hold-down assembly provided high initial stiffness and strength and the required ductility for seismic applications, and can therefore be considered a viable alternative for high-rise wood-based structures in moderate and high seismic zones.

Keywords: Tall wood buildings, wood-hybrid buildings, high-capacity hold-downs, failure mechanism

1. Introduction

1.1 Tall wood buildings

Tall wood building systems are increasingly in the focus of developers around the world. The building industry in North America has the ability to take advantage of the inherent benefits of wood from both an environmental and economic standpoint [1]. Today, wood structures in North America are usually limited to six-storey buildings [2] with the majority not being taller than four storeys due to limitations in the past building codes. Building taller wood buildings may help the growing urbanization of cities while still preserving the environment [3]. Utilization of mass-timber panels can significantly increase the use of timber products in the construction market. The three main mass timber products in North America are Cross-Laminated-Timber (CLT), Laminated Strand Lumber (LSL) and Laminated Veneer Lumber (LVL). These products can be used in wall and floor components of tall wood buildings. Some of the mass-timber products, such as CLT, can rely on the solid nature of the wood panels to provide inherent fire-resistance to the structure [4].

Significant technical efforts have targeted wood-based high-rise buildings, e.g. FPInnovations has developed a technical guide for design and construction of tall wood buildings in Canada [3] and as part of the Canadian NewBuildS network, a 20-storey demonstration wood building, located in North Vancouver, an area of high earthquake, wind and rain loads, was designed, analyzed and shown to be structurally viable [5]. Existing tall wood-hybrid buildings include the LifeCycle Tower One [6], which erected in 2012 was the world's first wood-based hybrid modular building that achieved a passive house standard, the world's leading standard in low energy building [7]. Different conceptual designs are being proposed for tall-wood buildings up to 30 storeys including the so-called FFTT system [8] and the wood concrete hybrids developed by SOM [9]. A number of buildings are already planned or under construction such as the 18-storey tall wood student residence at the University of British Columbia which is set to open in September 2017 [10].

The load effects on high-rise timber structures are different from those on low-rise buildings. While low-rise timber buildings have natural periods of vibration below 1sec, which is within the range of the peak spectral accelerations of most earthquakes [11], high-rise timber buildings have much larger periods, and wind effects may become more critical. Because of higher lateral wind loads to be considered, high-rise wood buildings face increase hold-down demands. Although the smaller self-weight of the wood structure reduces the foundation costs, it also leads to less inherent resistance to overturning forces [12]. The overturning resistance of tall timber structures needs to be addressed, either by reducing the uplift forces or by improving the existing hold-down solutions.

1.2 Hold-downs for timber structure

Different types of hold-downs (or anchorage) systems are currently available for houses and low-rise wood-frame structures such as traditional straps, shown in Fig.1a, that are attached to the foundation and nailed to shear walls. Other conventional solutions for low-rise structures are off-the-shelf hold-downs like the Simpson Strong-Tie® [13] which usually reach their load carrying capacity at around 50kN. As a result, there is an increased need for restraint systems that can meet multi-story structural demands without sacrificing installation efficiency or cost considerations. For multi-story wood-frame structures, rod systems such as the Simpson Strong-Tie® Strong-Rod™ Systems (ATS), shown in Fig1b, are available. This system consists of a combination of rods, coupler nuts, bearing plates and shrinkage-compensation devices instead of using metal connector brackets that work together to create a continuous load path to the foundation and can provide a capacity of 100kN [13]. A novel round tube connection developed by Schneider [14], as shown in Fig.1c, can also be used as hold-down in mass timber structures. This connector consists of hollow steel tubes placed inside CLT panels and can avoid damage to wood at capacities more 60kN based on the size of the tube, while providing both high stiffness and ductility.

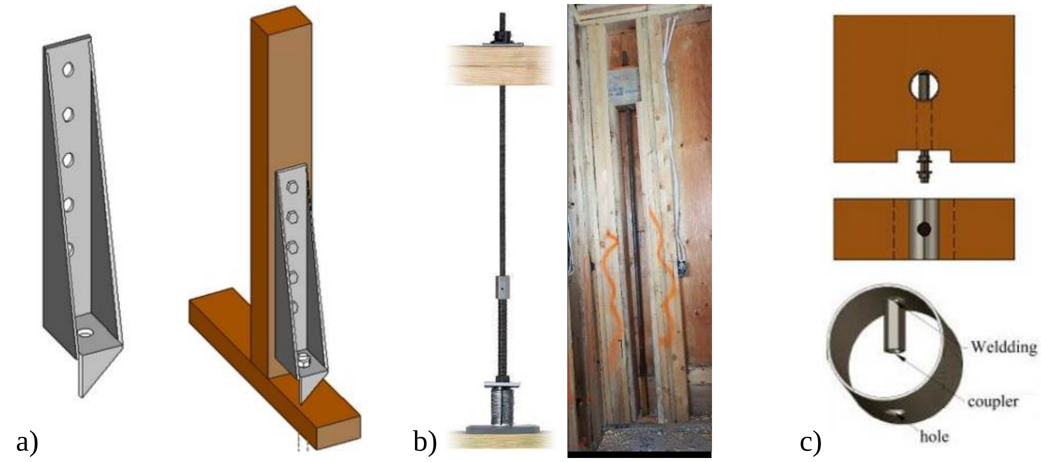


Fig. 1 – Hold-downs for timber structures: a) Traditional option; b) Strong-Rod™; c) Tube Connector

1.3 HSK System

Traditional hold-downs or continuous rod tie-down systems are suitable for low-rise construction as their maximum capacities are around 100kN. These hold-downs would not be able handle the forces in tall mass-timber buildings. Consequently, high-capacity hold-down solutions that maintain the structural integrity need to be developed. One option is the Holz-Stahl-Komposit-System (HSK-System)™ which consists of perforated steel plates, shown in Fig.2a, which are adhesively bonded to the wood, as shown in Fig. 2b. The capacity of an HSK connection is governed by the minimum of the steel plate, the adhesive bond, and the wood capacity.

The geometric parameters of a typical perforated steel plate are shown in Fig.2a. The holes in the plate are filled by the adhesive after insetting it into the wood, forming so-called "adhesive dowels" (AD). The "adhesive bond capacity" is based on the sum of the individual AD capacities. The links between the plate holes, called steel links (SL), determine the ductile steel plate capacity. Fig.2c shows the conventional HSK hold-down design in high-rise timber structure. The perforated steel plates are welded to section-reduced steel side plates which provide the ductile fuse for capacity design. Preliminary investigations [15] demonstrated the stiff performance, a ductile behavior and at the same time a predicable fatigue performance. Therefore, the HSK system can be a viable option for hold-downs of high-rise timber structures. It can be designed to produce ductile steel failure in the range of ultimate load under static as well as dynamic loading [15] and has been successfully applied in timber-tower applications [16]. The connector has been applied in several tall building projects such as the Wood Innovation Design Centre in Prince George, which is currently North America's tallest contemporary wood building [10]; however, it has not been used in timber structures located in high seismic zones.

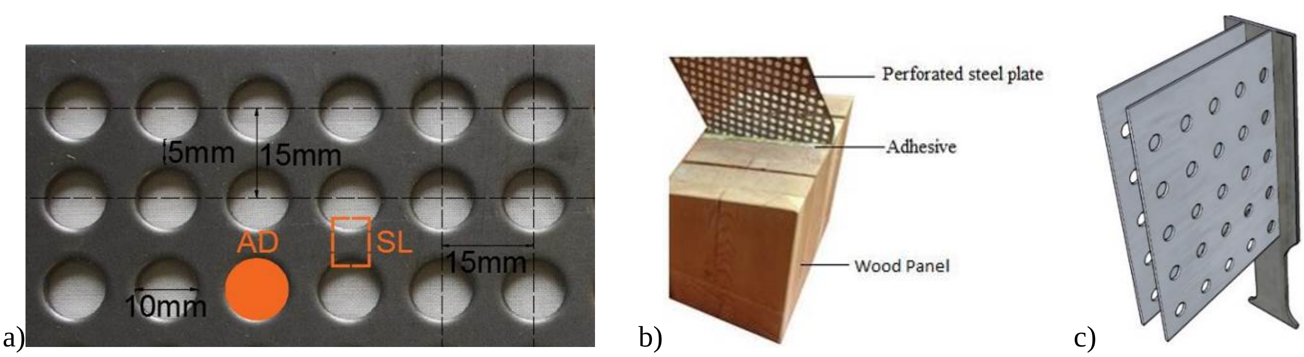


Fig. 2 – HSK connection: a) Perforated steel plate; b) HSK assembly; c) conventional HSK hold-down

2. Experimental investigation

2.1 Objective

There is little research available on the HSK system under cyclic loading. Furthermore, the conventional solution of using section-reduced steel side plates could buckle under reversed cyclic loading. In this research, the HSK hold-down system is modified by attaching the perforated steel plates to a HSS hollow beam and using duct tape to cover holes in the perforated steel plate to facilitate a secondary yield mechanism inside the wood assembly which is restrained from buckling, see Fig.3. The objective of this research was to investigate the performance of modified HSK connectors in a hold-down application.

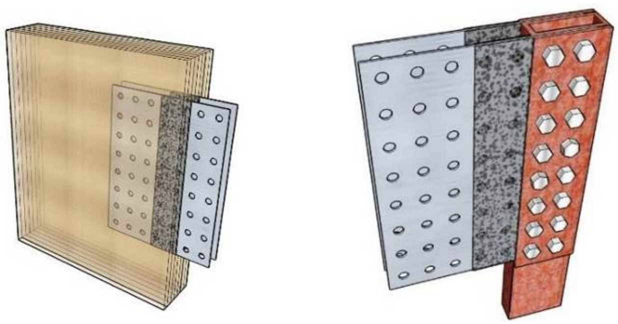


Fig. 3 – Modified HSK hold-down design

2.2 Materials

As wood material, 7-ply CLT panels, grade V2M1, supplied by Structurlam Products Ltd. and fabricated according to ANSI/APA PRG 320-2012 [17] were used. The wood species was SPF No.1/No. 2, with a density of 420~450kg/m^3, Purbond polyurethane as adhesive, and the moisture content was 12% (+/-2%). The perforated steel plates were Grade S275 with a uniform hole size of diameter d=10mm. The length of the SL was 5mm and the thickness of the plate was 2.55mm. The applied adhesive was CR-421 by Purbond® (two-component polyurethane-PUR), commonly used for on-site applications with a viscosity of 4,000 cps, an open work time of 10-20 min and an expected shear strength when bonded to wood of around 3.6~7.8MPa [18].

Two different configurations of perforated steel plates were selected with two rows of holes covered by the duct tape. The short size contained 22 by 8 holes with 23SL in one row and the long size contained 66 by 7 holes with 67SL in one row, as shown in Fig.4a. Steel Tube (3.5×2.5×0.25 inch), steel side plate (3.5×0.75 inch) and 3/8 inch Grade 8 bolts and nuts were also needed clamp the HSK mesh and fixed to the foundation, as shown in Fig. 4b.

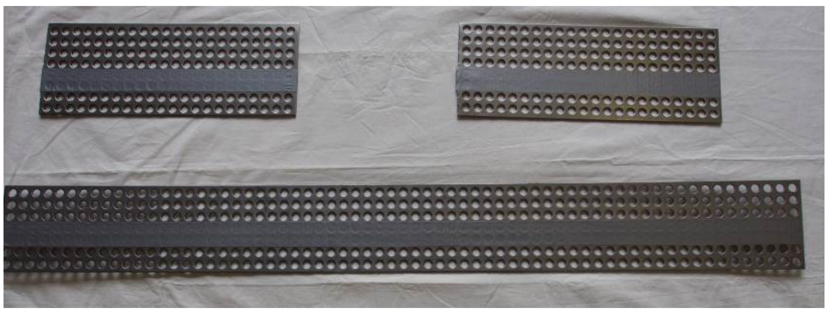
a)  b) 

Fig. 4 – Materials: a) Perforated steel plates; b) HSS tube, side pate and bolts

2.3 Component level investigations

In a first phase, the tensile capacity, shear capacity and ductility of the modified HSK connection at the material and component level were determined [19]. At the material level, quasi-static monotonic tension tests validated the basic connector properties. Fig.5a illustrates the tests to determine the tensile strength of the perforated steel plates; Fig.5b shows the setup which was conducted to determine the strength of the adhesive bond. In component level tests, the influence of the number of the SL in perforated steel plate and the number of covered rows of duct tape on the assembly performance were studied under quasi-static monotonic loading (illustrated in Fig.5c) and reversed cyclic loading (shown in Fig.5d).

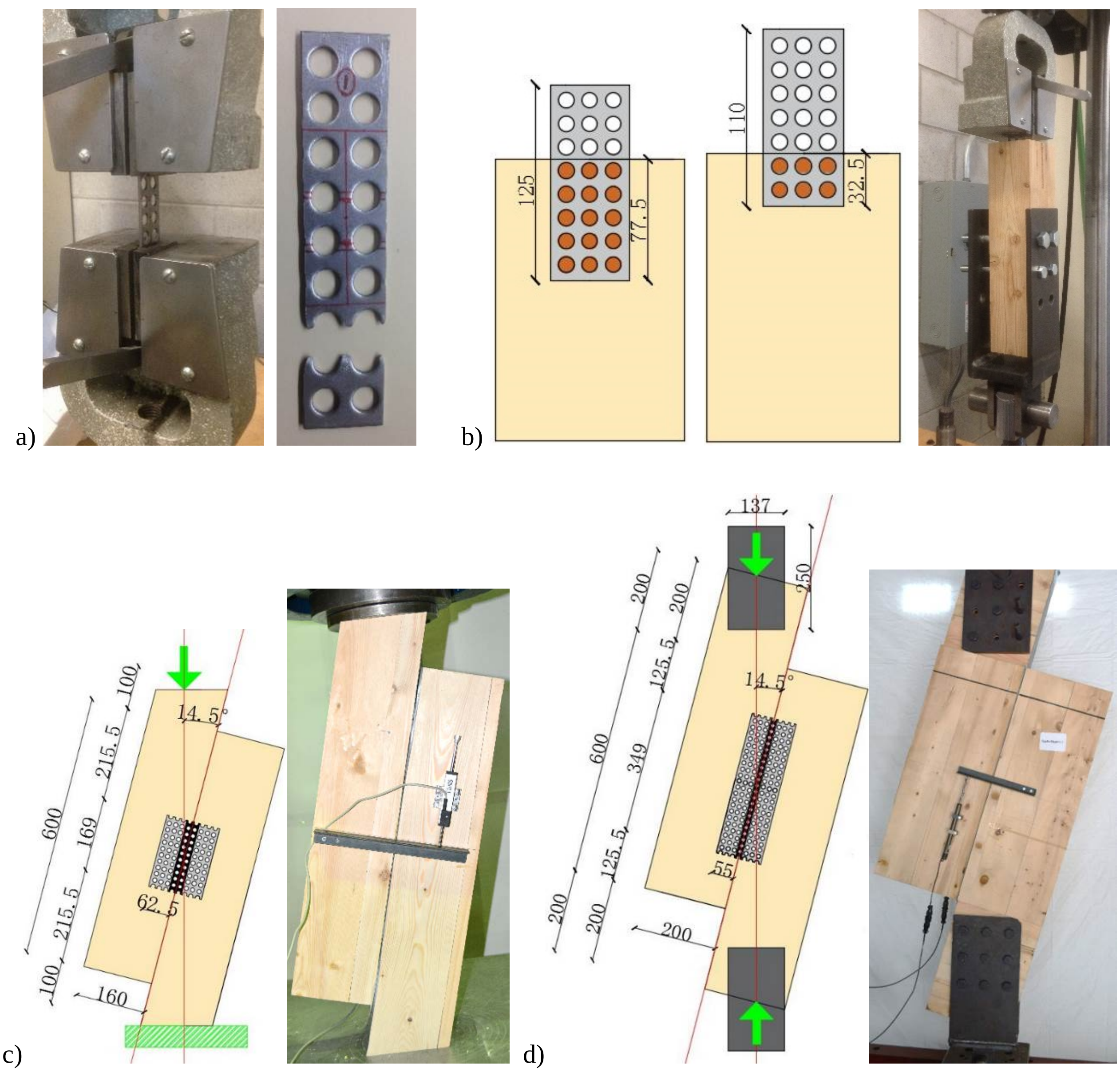


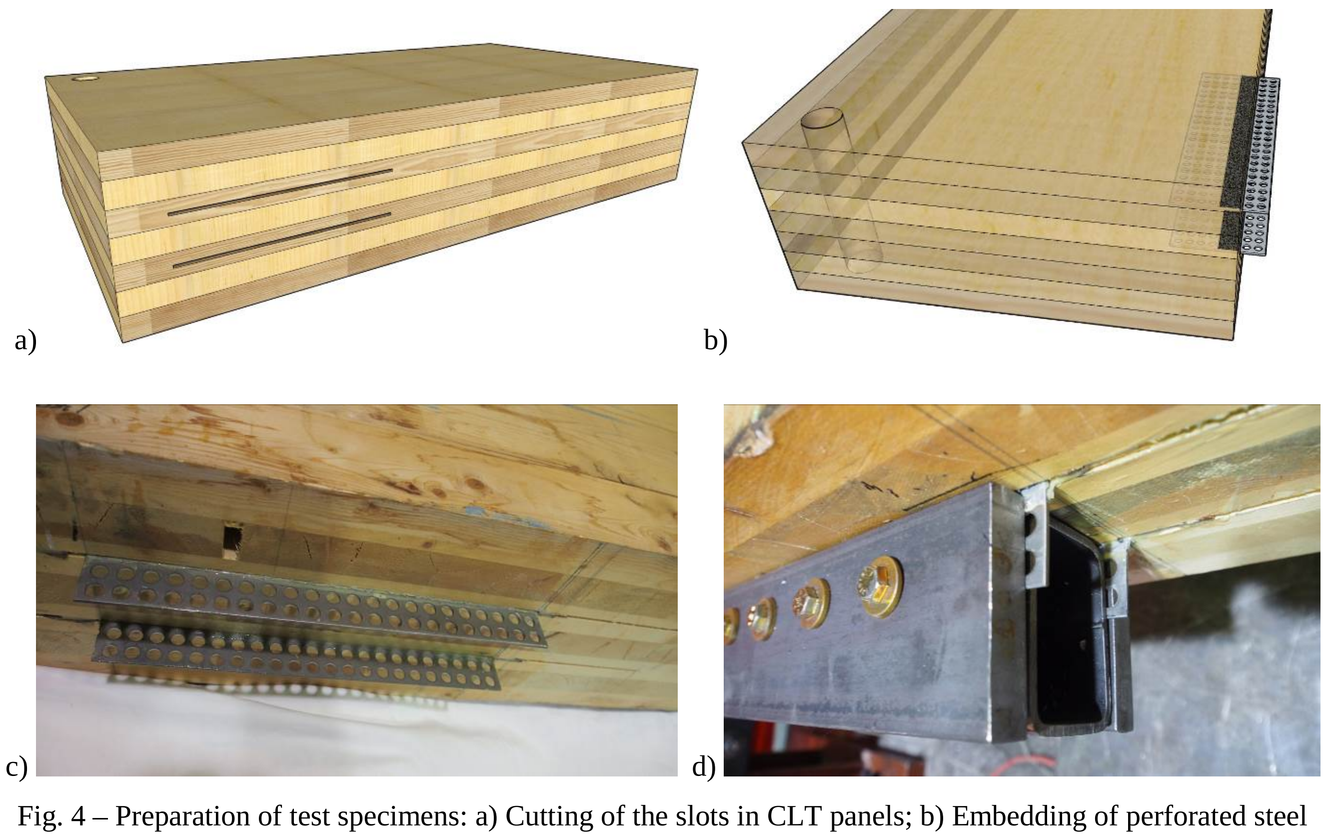
Fig. 5 – Preliminary tests: a) Tension tests on steel plate; b) Tension test on HSK assembly; c) Monotonic shear tests d) Reversed cyclic shear tests [20]

The results demonstrated that the modified HSK connector assembly has high initial stiffness while providing large ductility. The design of this hold-down is based on the capacity of the AD and the capacity of the perforated steel plate. The total AD capacity equals the number of AD multiplies one AD capacity and the total steel plate capacity equals the number of SL in the loading direction times one SL capacity. Once the total AD capacity is larger than the total maximum force of SL, the maximum bearing capacity of this connection is the capacity of the SL and ductile steel failure instead of the brittle adhesive failure can be achieved.

The length of the perforated steel plate which relates to the number of SL determines the capacity and stiffness as these metrics are proportional to the number of SL in the inserted perforated steel plate within the loading direction. The yield force for one SL, $F_{y,SL,s}$, was determined to 2.2kN; the capacity for one SL $F_{max,SL,s\,(e)}$ equaled 3.85kN [19]. Based on these results from material level and component level test, the capacity of the modified HSK hold-down can be predicted according to the number of the SL in the perforated steel plate.

2.4 Methods for Hold-down tests

The preparation of the test specimens consisted of four steps: 1) two 4mm width slots were cut in two layers of the 7-ply CLT panel with the dimension of 1200mm×600mm×239mm, the length of the slot is based on the length of the perforated steel plate which is 335mm long and depth of the slot is 92.5mm, Fig.6a; 2) two perforated steel plates with two rows of covered duct tape were embedded into the two slots of panel independently which filled of adhesive, each perforated steel plates has 23 SL and 88 AD, as shown in Fig.6b and Fig.6c; 3) After two days when the adhesive was cured, the part of the perforated steel plate which is outside of the CLT panel was clamped to the HSS steel tube with two steel side plates as shown in Fig. 6d. Four replicates were fabricated, two each for the quasi-static monotonic and reversed cyclic tests.



a) b) c) d)

Fig. 4 – Preparation of test specimens: a) Cutting of the slots in CLT panels; b) Embedding of perforated steel plates; c) Curing of adhesive; d) Clamping the perforated steel plate to steel profile

The tests were carried out under displacement-controlled loading utilizing a MTS actuator with (445kN capacity) in the Structures Lab of UBC. Linear Variable Differential Transformers (LVDT's) were attached to both sides of the CLT panel to capture the uplifting displacement of the hold-down and a string-pot was used to measure the lateral movement of the CLT panel. Fig. 7 illustrates the test set up from the side view and top view.

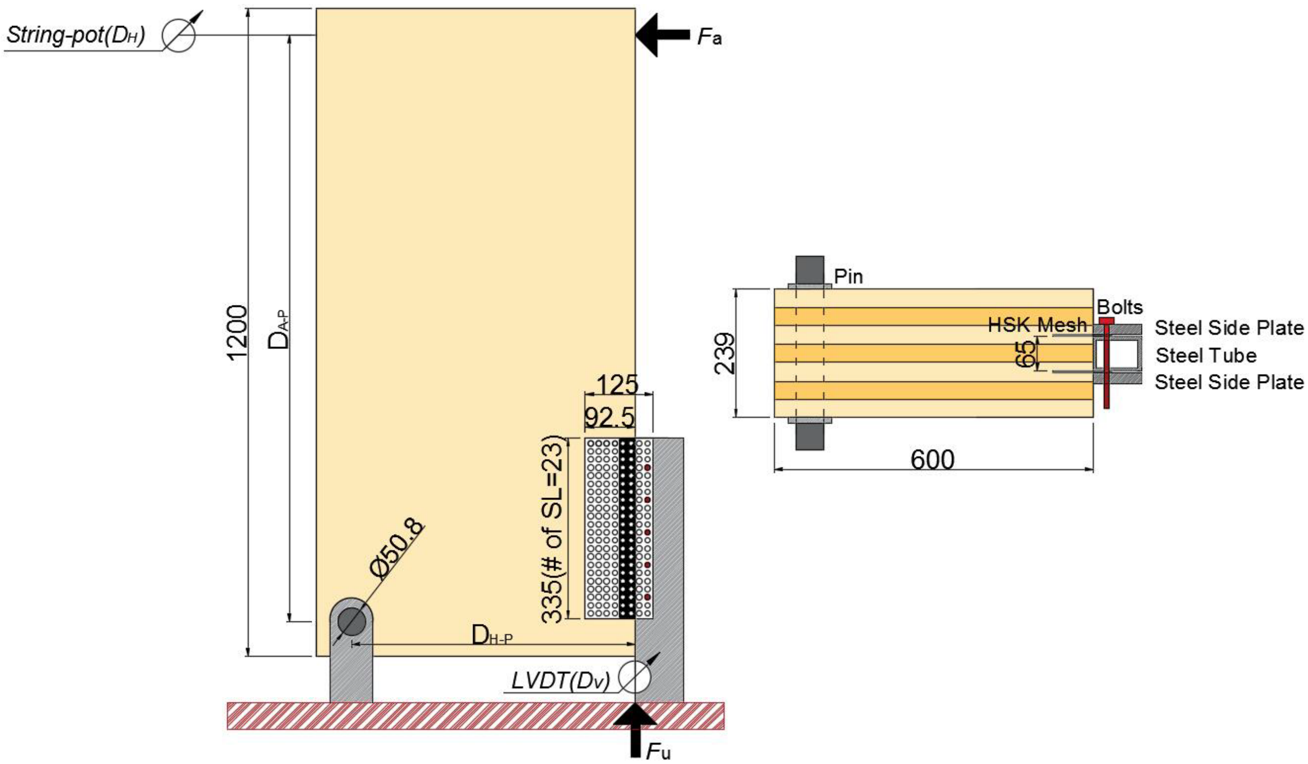


Fig. 5 – Test set up

The monotonic tests were conducted with a constant loading rate of 8mm/min. Then, based on the target displacement obtained from the static tests, the loading protocol for the reversed cyclic tests using the CUREE protocol [20] were developed. Based on the applied actuator force and the geometric properties of the CLT panes, the hold-down force can be calculated:

$$F_u = \frac{F_a \times D_{A-P}}{D_{H-P}} \tag{1}$$

Where F_u is the hold-down force, F_a is the applied load; $D_{A\text{-}P}$ is the distance from the actuator to the center of the pin connection, and $D_{H\text{-}P}$ is the distance from the hold-down to the center of the pin connection. Ductility μ was computed as a ratio of ultimate displacement to yield displacement. The connection stiffness K_e was calculated as ratio of load difference ($0.4F_{max}$ - $0.1F_{max}$) to displacement difference ($0.4D_{max} - 0.1D_{max}$).

2.5 Results and Discussion

The hold-down force (F_u) and the recorded uplifting displacement from LVDT (D_v) were used to plot the force-displacement curves, see Fig.8. The load (F_u) is the hold-down force while the (D_v) is the vertical uplift displacement. The hysteresis loops and envelope curves from the reversed cyclic tests are also shown in Fig. 8. Table 1 summaries the results with F_y representing the yield force; F_{max} representing the maximum force; D_y representing the yielding displacement; D_u representing the ultimate displacement; $F_{y,SL}$ representing the yield force for one SL; $F_{max,S}$ representing the maximum force for one SL, and $K_{e,SL}$ is the stiffness for one SL.

Table 1 – Test Results from static and cyclic test

Test	F_y [kN]	D_y [mm]	F_{max} [kN]	D_u [mm]	K_e [kN/mm]	μ [-]	$F_{y,SL}$ [kN]	$F_{max,SL}$ [kN]	$K_{e,SL}$ [kN/mm]
Monotonic 1	98	1.5	180	19.8	151	13.1	2.1	3.9	3.3
Monotonic 2	102	1.7	186	18.5	109	10.9	2.2	4.0	2.4
Cyclic 1	98	1.3	167	12.3	159	9.7	2.1	3.6	3.5
Cyclic 2	91	1.4	172	11.5	134	8.1	2.0	3.7	2.9

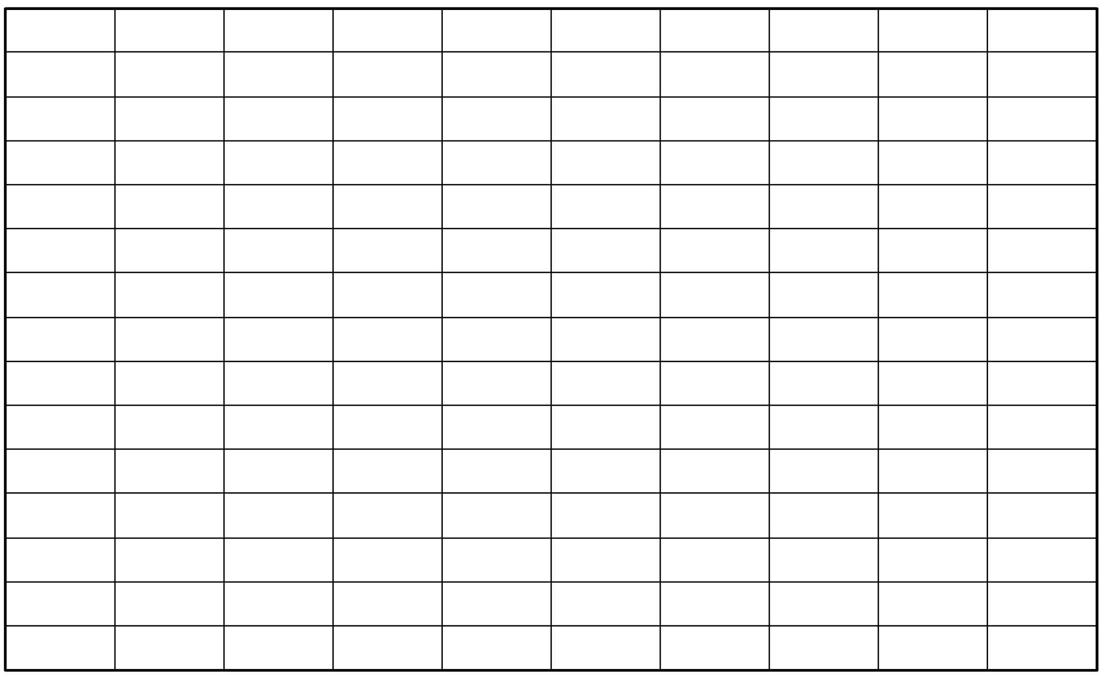


Fig. 6 – Force-displacement curves from static test and cyclic test for two replicates

The obtained results conclusively demonstrate that the modified HSK hold-downs reach high stiffness and reliably predictable capacity.

In the monotonic tests, the yield and maximum force F_y and F_{max} are closed to the expected values according to the material level and component level tests of 103kN and 177kN, respectively. The average differences are within 3% which indicates the good predictability of the hold-down connection. This confirms the capacity of the hold-down can be designed based on the number of the SL as it is proportional to this parameter. While there was higher variability in stiffness between replicates, the values are both large with the average being 130kN/mm ($K_{e,SL}$ equal to 2.9kN/mm). The ductility ratios μ for both replicates were larger than 10 which confirmed the highly ductile behaviour of the modified HSK hold-down connection.

The results from the reversed cyclic tests confirm the findings from the monotonic tests, validating a predictable high stiffness and capacity.The yield force F_y and maximum force F_{max} reahced as an average 93% of the monotonic test results. The difference can be explained by low-cycle fatigue of the steel plate which results the lower capacity in cyclic testing. For the same reason, the ulitmate displacements under cyclic loading are smaller than the monotonic loading, leading to smaller ductility ratios in the cyclic tests. Although cyclic ductility reahced only 75% of monotonic ductility, the ratios μ equals to 8 and 10 still allow the assembly to be classified as highly ductile. The hold-down stiffness, K_e, is similar to the values from the monotonic tests, exhibiting smaller variability between replicates.

Fig.8 also shows one challenge that has no yet been sufficiently addresses: the HSK hold-down system does not exhibit any self-centring capability. The hysteresis loops do not return to zero displacement but after each loading cycle a larger residual displacement remains. This undesired behaviour, however, was partially caused by the chosen displacement controlled loading protocol with the displacement being applied to the top of the panel and some slack in the test-set-up and can be avoided in future testing.

The failure mode of the perforated steel mesh is shown in Fig.9. The photo, taken after testing clearly illustrates the displacement between the two rows of holes which were covered by duct tape and that no plastic deformation occurred in the steel links of the perforated steel plate where the holes where filled with adhesive. This failure mode demonstrated that yielding and plastification can be concentrated inside the CLT panel and the steel mesh can be effectively prevented from buckling.

Fig. 9 – HSL mesh after testing

3. Conclusions

In this research, a high-capacity seismic hold-down was experimentally tested. The results presented in this paper allow following conclusions to be drawn:

(1) The modified HSK hold-down can be used as a strong and stiff yet ductile connection based on the number of steel links as long as the steel-link capacity exceeds the adhesive dowel capacity.

(2) By covering one row of perforated steel plates inside the mass-timber panel using duct tape, the modified HSK hold-down shows a ductile behaviour and buckling of under cyclic loading is prevented.

(3) As a consequence, the modified HSK hold-down can be applied in tall wood buildings. It provides a solution to resist the uplift forces of the building and can dissipate seismic energy.

Acknowledgements

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