STRUCTURAL CHARACTERIZATION OF MULTI-STOREY CLT BUILDINGS BRACED WITH CORES AND ADDITIONAL SHEAR WALLS

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CLT structures, core structures, seismic design, shear walls

1 INTRODUCTION

In last twenty years the CLT panels have become widely employed to build multi-storey residential and mercantile buildings. These buildings are often characterised by the presence of many internal and perimeter shear walls. Such structures have been widely studied through experimental and numerical simulation methods. The most comprehensive experimental investigation to date on seismic behaviour of CLT buildings was carried out by CNR-IVALSA, Italy, under the SOFIE Project (Ceccotti 2008, Ceccotti et al. 2013). Other important investigations have been conducted at the University of Trento, Italy (Tomasi and Smith 2015). European seismic performance related tests have also been conducted at the University of Ljubljana, Slovenia where the behaviour of 2-D CLT shear walls with various load and boundary conditions were assessed (Dujic et al. 2005). FPInnovations in Canada has undertaken tests to determine the structural properties and seismic resistance of CLT shear walls and small-scale 3-D structures (Popovski et al. 2014). Those and other studies have enabled characterisation failure mechanisms in large shear wall systems (Pozza et al. 2013). Multi-storey building superstructures in which beam-and-column frameworks resist effects of gravity loads and cross-braced or core substructures and exterior CLT shear walls resist effects of lateral forces from earthquakes or wind have been found structurally effective, and fail in predictable stable ways if overloaded (Smith et al. 2009). Advantages of such systems can include creation of large open interior spaces, high structural efficiency, and material economies.

Recently innovative connection solutions that create discrete panel-to-panel, or panel-to-other material joints have been developed in Italy (Polastri and Angeli 2014). The method results in point-to-point mechanical connections that only connect corners of individual CLT panels in ways that fulfil hold-down and lateral shear resistance functions (Gavric et al. 2013). This has the advantage of making the load paths within superstructures unambiguous. Different connectors have also been tested to find the best ways to make point-to-point connections between CLT panels and steel structures (Loss et al. 2014).

During recent years connector designs had evolved considerably making them suitable for much large systems that place high capacity demands on connections, with emphasis on requirements for high seismicity areas (Polastri et al. 2014). During such development attention was paid to avoiding the possibility of brittle behaviour of joints to CLT panels having many nails.

Structural performance issues not fully studied are those related to using CLT building cores as replacements for one constructed from reinforced concrete or masonry. Pertinent issues relate to vertical continuity between storeys, connections between building core elements and elevated floors, and building core-to-foundation connections.

2 MECHANICAL CHARACTERIZATION OF THE CONNECTORS

2.1 Experimental studies

The mechanical behaviour of connection systems for CLT structures that employ thin metal elements fastened to panels with nails or other slender metal fasteners is well known, as demonstrated by numerous scientific papers (Ceccotti et al. 2008, Pozza et al. 2013). The behaviour of such connectors is determined largely by the elastoplastic response of the fasteners, and to a lesser extent by the response of steel elements. Stiffness and capacity values implemented into the numerical models described in Section 4 were calculated directly from experimental data.

The first study was carried out at CNR-IVALSA (Gavric et al 2011), the second study at the University of Trento (Tomasi and Smith 2015). In both cases tests were conducted according to the European standard EN 12512 (CEN 2006). The CEN 2006 protocol provides a load history characterized by load cycles of increasing intensity and is intended to apply to structures in seismic regions. As suggested by the standard, a preliminary monotonic test was undertaken to define the magnitudes of cyclic load excursions, Figure 1.

Initial stiffness was calculated according to ‘method b’ specified by EN 12512 that permits

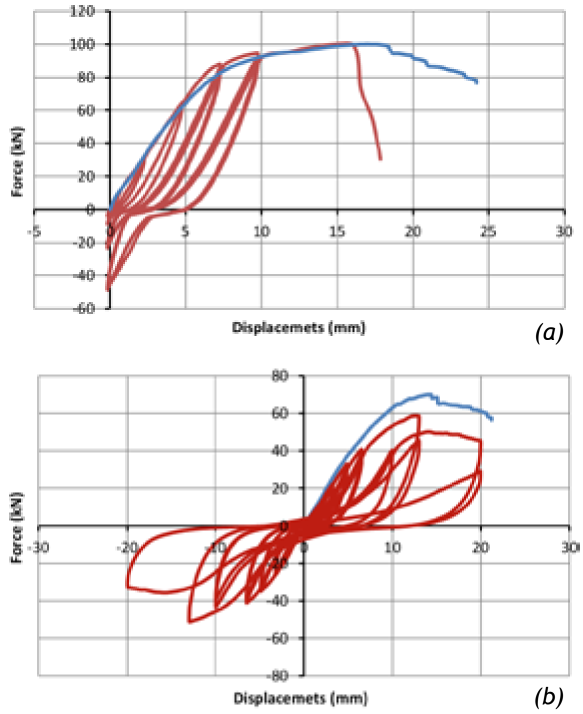


Figure 1: *Typical tests results: hold-down (a) and angle bracket (b).*

description of the mechanical behaviour of trends representing elastic phase and post-elastic phase responses (Piazza et al. 2011). However, as this paper deals with Linear Dynamics Analysis of superstructure systems only the parameters that characterize the elastic properties of connections (k_{test}) and the maximum load at failure (F_{max}) are reported here, Table 1.

2.2 Analytical definition of stiffness according to Eurocode 5

The Finite Element (FE) model presented in Section 4 implements hold-down Rothoblaas WHT 620 (EOTA 2011) and angle brackets TITAN TTF200 (EOTA 2012) connectors joined to CLT panels manufactured from class C24 wood boards using 32 4x60 or 30 4x60 Anker nails. The initial stiffness of connectors was calculated taking into account the stiffness of the steel-to-timber nailed joints in shear and hold-down connections. Deformation of steel parts within the connections are very small, compared deformation of nailed joints, and was therefore neglected. Characteristic load-carrying capacities, $F_{v,Rk}$, and slip moduli, k_{ser}, were calculated based on Eurocode 5 (CEN 2014), Table 1.

Table 1: Experimental and Eurocode 5 derived connection properties

Connection type	Elastic stiffness (kN/mm)		Capacity (kN)	
	Test (k_{test})	EC5 (k_{ser})	Test (F_{max})	EC5 ($F_{y,Rk}$)
TITAN TTF 200	8.2	23.1	70.1	35.5
WHT 620	12.1	24.8	100.1	85.2

3 ESTIMATION OF T_1 AND DESIGN OF CONNECTIONS

A crucial issue in the design of a CLT building under horizontal seismic action, is the definition of the principal elastic vibration period (T_1) of an entire superstructure (CEN 2013). Such vibration period depends on the mass distribution and on the global stiffness of the buildings. In a CLT structure the global stiffness of the buildings is highly sensitive to deformability of the connection elements (Pozza et al. 2013). Consequently for a precise control of the vibration period of the building it is crucial to define the stiffness of each connections used to assemble a superstructure. During design engineers are required to solve iteratively to find the principal natural frequency ($f_1 = 1/T_1$) using a scheme such as that in Figure 2. Under the shown scheme: (1) the stiffness of the connections influences the global stiffness of the building and therefore its principal elastic period; (2) the external force induced by earthquake in each connection is a function of the principal vibration period; (3) the load bearing capacity of the connection must be compatible with the external force; (4) the strength and the stiffness of the connection are linked through the effective number of fasteners.

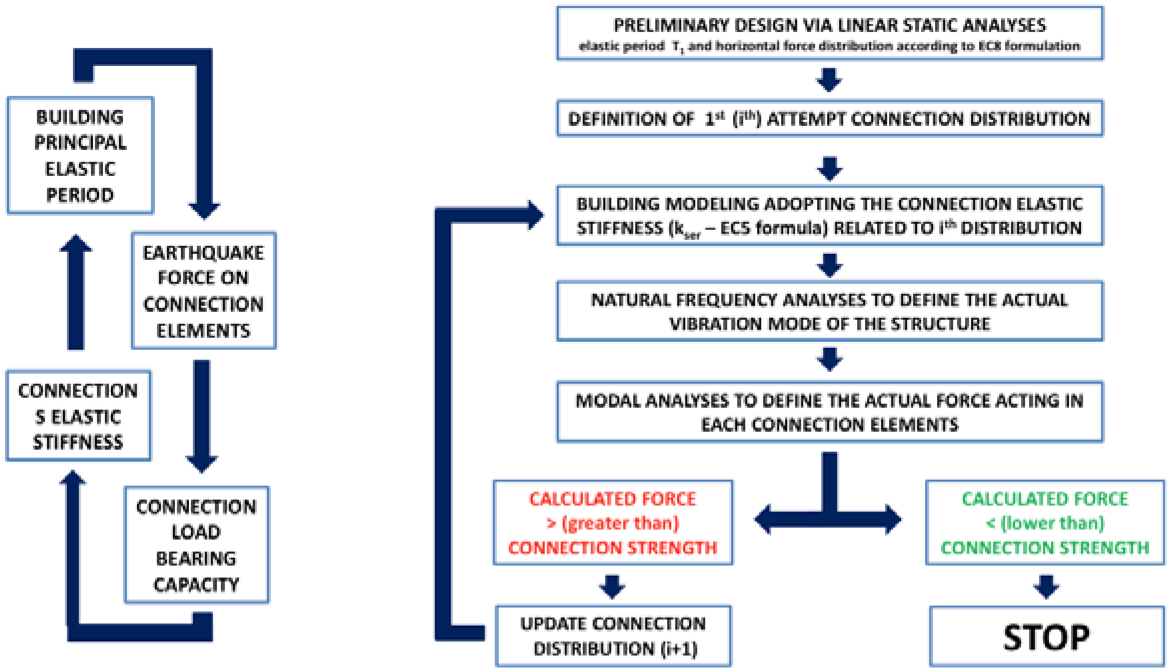


Figure 2: *Calculation process for design of connections*

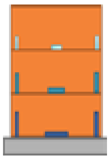
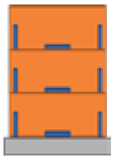
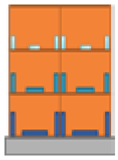
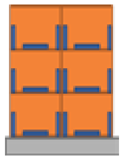
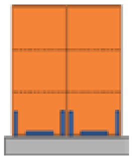
An efficient approach to design a CLT structure is to start from a preliminary definition of the external force induced by earthquake in each wall panel according to the common equivalent static force linear static analysis approach (CEN 2013). This does not involve the definition of T_1 accounting for effects of connection stiffness. Once static forces on each CLT wall panel are defined connection capacities can be designed to be compatible with external static forces. This allows estimation of the connection elastic stiffness (k_{test} or k_{ser}), and therefore realistic preliminary estimation of T_1. Then T_1 can be in modal analyses to calculate the effective forces induced in connections by earthquakes. Obtained connection forces may or may not be compatible with the connection strength, and if not it is necessary to redesign connections. Afterward it is possible to perform a more iterative precise frequency analyses until solutions, including connection designs, are convergent.

4 NUMERICAL ANALYSIS OF CORE TALL BUILDINGS

4.1 Case study buildings

The aim is to characterize behaviour of multi-storey CLT buildings braced with cores and additional shear walls from the seismic design perspective based on effects of varying design parameters. Varied design parameters are: number of storey (3-5-8), lateral shear wall panels width, construction methodology, and regularity of connectors as a function of the height within a superstructure, Table 2.

Table 2: Examined building configuration

Case Study ID	3(5-8) A R	3(5-8) A I	3(5-8) B R	3(5-8) B I	3(5-8) C R
Graphical schematization of building cores (ex. 3-storey case)					
Panel assembly	Joint free wall panels		Jointed wall panels		Joint free wall panels
Elevation regularity	Regular	Irregular	Regular	Irregular	Regular
Construction methodolgy	Platform system				-

4.1.1 Geometric configurations

Examined case-study building superstructures have footprint dimensions of 17.1m (direction X) by 15.5m (direction Y). Seismic Force Resistant Systems (SFRS) include a building core that is 5.5m by 5.5m on plan, and partial perimeter shear walls constructed from CLT panels with a total base length of 6m, Figure 3. Storey height is 3m in all cases, resulting in total superstructure heights of 9m, 15m and 24m respectively. All CLT panels in the core walls have a thickness of 200mm. CLT panels in perimeter shear walls are 154mm thick, except for those in the lowest four storeys of the 8-storey SFRS which are 170mm thick. Floor diaphragms are composed of 154mm CLT panels in all cases.

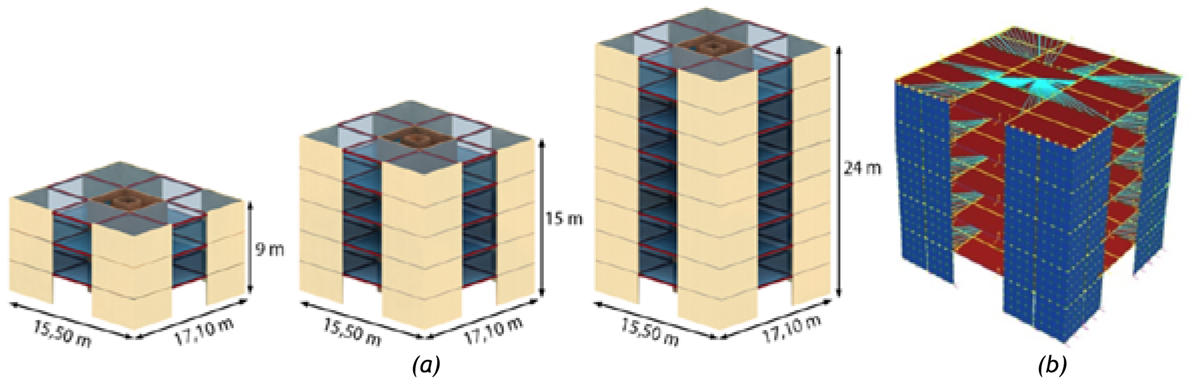


Figure 3: *SFRS wall configurations of case study buildings (a) and typical FE model (b).*

4.1.2 Design method

The earthquake action for these case study buildings was calculated according to Eurocode 8 (CEN 2013) and the associated Italian regulations (MIT 2008) using design response spectra for building foundations resting on ground type C*, assuming the PGA equal to 0.35g (the highest value for Italy) with a building importance factor of λ = 0.85. [*Deep deposits of dense or medium dense sand, gravel or stiff clay with thickness from several tens to many hundreds of meters]. The seismic action was calculated starting from the elastic spectra and applying an initial q-reduction factor of 2

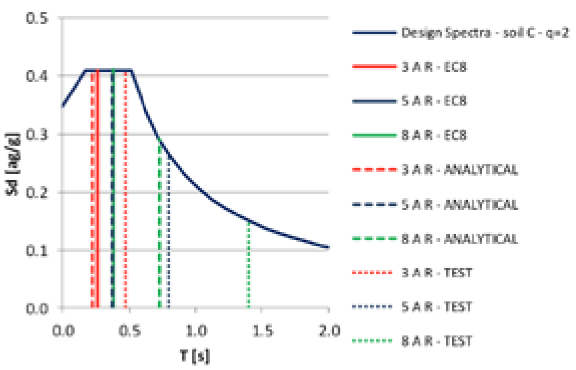
(CEN 2014). The coefficient k_r was taken equal to 1.0 for regular configurations and 0.8 for non-regular configurations. Figure 4 shows the adopted design spectra and other relevant design parameters. The figure shows T_1 values determined by simplified formula and numerical frequency analyses methods for configurations A R 3-5- 8.

Connections were first designed using the force pattern obtained applying linear elastic static analysis (CEN 2013) and the seismic action defined by taking $T_1 = T_{1\_EC8}$. Connection designs were then refined using the rotation and translation force equilibrium approach described by Gavric et al. (2011) and Pozza and Scotta (2014) and the iterative design process in Figure 2.

seismic area	zone 1
soil type	C
Peak Ground Acceleration	0.35 g
q_0 (behaviour factor)	2
k_R (regular configuration)	1
λ (importance factor)	0.85

	3 A R	5 A R	8 A R
H [m]	9	15	24
$T_{1\_EC8} = 0.05\ H^{3/4}$ [s]	0.26	0.38	0.54
M [t]	276	482	800
$S_{d\text{-}el\,(T1)}$ [ag/g] *	0.82	0.82	0.78
$q = k_R\ q_0$	2	2	2
$S_{d\,(T1)}$ [ag/g] *	0.41	0.41	0.39
*using elastic period evaluated via code (CEN 2013)			

(a)



(b)

Figure 4: *Input data for seismic analysy (a); design spectra and calculated periods (b).*

4.1.3 Finite element (FE) models

Numerical models of the investigated building were realized using the finite-element code Strand 7 (2005). The illustrative FE model in Figure 3 (b) uses linear elastic shell elements to represent CLT panels and link elements to simulate the elastic stiffnesses of connectors. Beam elements with pinned end conditions were used to represent beam members interconnecting perimeter shear walls and shear walls in the building core at the top of each storey. Horizontal slabs elements in floor and roof diaphragms were assumed to be rigid in-plane.

All the 15 building configurations have been modelled respecting the geometrical features and connection stiffnesses in Tables 1 and 2.

4.2 Analysis results

Results presented here were obtained by modal response spectrum analyses of case study buildings, Tables 3 to 6 and Figure 5. Those tables and figure show calculated building principal elastic periods (T_1), base shear forces (v) on angle brackets at the Ultimate Limit State (ULS), uplift forces on base hold-down anchors at ULS (N), and the maximum inter-storey drift values (δ) at Damage Limitation State (DLS). The alternative values given represent effects of taking connection stiffnesses (k_{conn}) equal to values derived from Eurocode 5 (k_{ser}) versus values derived from experiments (k_{test}). Plus in the case of T_1, the simple formula value T_{1_EC8} is included (CEN 2013). Inter-storey drift was calculated for each case study building using the Modal Response Spectrum Analyses and the DLS design spectrum.

Observing Figure 5 it is apparent that can be seen that in most cases use of experimental connection stiffnesses (k_{conn} = k_{test}) leads to much larger T_1 values than those predicted based Eurocode 5 based estimates of connection stiffnesses (k_{conn} = k_{ser}). Similarly using the simple formula given by Eurocode 8 leads to low estimates of T_1 values. Interestingly use of Eurocode 5 based estimates of k_{conn} results is estimates of T_1 relatively close to simple formula values. However results suggest that neither of those approaches are reliable ways of estimating

Table 3: Predicted principal elastic periods (T_1)

[s]	3AR	3AI	3BR	3BI	3CR	5AR	5AI	5BR	5BI	5CR	8AR	8AI	8BR	8BI	8CR
T_{1_EC8}	0.26	0.26	0.26	0.26	0.26	0.38	0.38	0.38	0.38	0.38	0.54	0.54	0.54	0.54	0.54
$T_{1\_kconn.=kser}$	0.22	0.19	0.22	0.20	0.18	0.37	0.34	0.38	0.35	0.30	0.73	0.69	0.73	0.68	0.50
$T_{1\_kconn.=ktest}$	0.47	0.39	0.44	0.37	0.33	0.80	0.61	0.73	0.57	0.45	1.40	4.14	1.30	1.08	0.66

Table 4: Predicted base shear per unit of length (v)

[kN/m]	3AR	3AI	3BR	3BI	3CR	5AR	5AI	5BR	5BI	5CR	8AR	8AI	8BR	8BI	8CR
$V_{\_kconn.\,=\,kser}$	28.2	33.2	32.7	40.3	35.8	42.3	51.7	51.8	64.2	50.4	50.2	64.6	61.6	82.6	79.7
$V_{\_kconn.\,=\,ktest}$	28.1	34.9	33.4	36.4	29.2	29.7	49.3	38.4	58.4	50.5	34.0	47.4	42.7	76.3	61.6

Table 5: Predicted free edge base uplift forces (N)

[kN]	3AR	3AI	3BR	3BI	3CR	5AR	5AI	5BR	5BI	5CR	8AR	8AI	8BR	8BI	8CR
$N_{\_kconn.\,=\,kser}$	128.1	170.9	152.0	178.1	161.3	316.0	420.6	355.4	447.3	373.2	511.6	884.1	559.5	759.7	897.0
$N_{\_kconn.\,=\,ktest}$	138.5	185.7	149.4	232.1	138.5	246.2	403.8	260.1	455.7	366.9	334.4	527.3	339.5	516.1	678.8

Table 6: Predicted maximum inter-storey drift (δ)

[mm]	3AR	3AI	3BR	3BI	3CR	5AR	5AI	5BR	5BI	5CR	8AR	8AI	8BR	8BI	8CR
$\delta_{\_kconn.\,=\,kser}$	2.9	1.8	2.5	1.9	1.4	5.6	4.4	5.1	4.3	2.6	9.9	8.8	8.8	7.8	4.7
$\delta_{\_kconn.\,=\,ktest}$	12.3	7.8	10.9	6.8	3.8	15.6	9.7	14.0	8.8	5.1	21.3	13.9	18.5	12.4	5.6

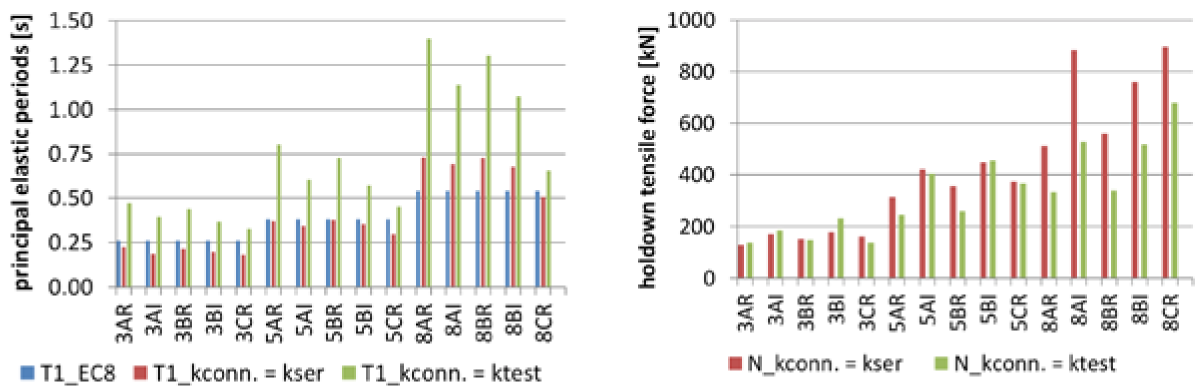


Figure 5: *Comparison of estimates of principal elastic periods and base free edge uplift forces*

principal natural periods of buildings having SFRS consisting of CLT cores and perimeter shear walls. Consequences of discrepancies in k_{conn} values from those found by testing varied in their effects on v, N and δ values, but in general results show that how connection stiffnesses are estimated can alter design force and lateral drift estimates by substantial amounts. For example, estimates of δ were especially sensitive for eight-storey buildings.

It is important to underline that the adopted FE model is a limiting condition representing the maximum deformability of the system since the interaction between the orthogonal walls and the out of plane stiffness, provided by the interposed floor slabs, are neglected. On the other hand, FE models did not take into account nonlinear deformability or large displacements effects.

It is possible to achieve large vertical reaction forces using a group of hold-down anchors working “in parallel”. To obtain the required uplift force resistances, that can be greater than 600kN (configuration 8CR), it is necessary to use more than eight hold-down anchors; however it is not demonstrated that the hold downs, disposed in the aforementioned group configuration, are able to spread the total force between the different reaction elements.

5 CONNECTION SOLUTIONS FOR INNOVATIVE DIAPHRAGMS

Diaphragms are an integral part of the building any SFRS and if they have high stiffness and capacity any non-linear behaviour of the entire structure is primarily defined by the response of the vertical bracing elements and complexity of the seismic analysis is reduced. CLT multi-storey buildings are erected using panels with limited dimensions because of production and transportation limitations (FPInnovations 2011). In floors and roofs, the different CLT panels are commonly joined together at the edges using dowel-type mechanical fasteners like self-tapping screws, Figure 6 (a).

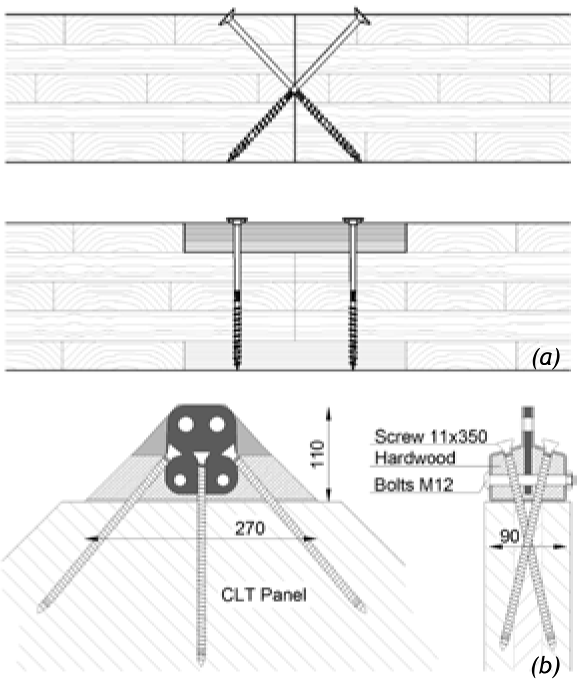
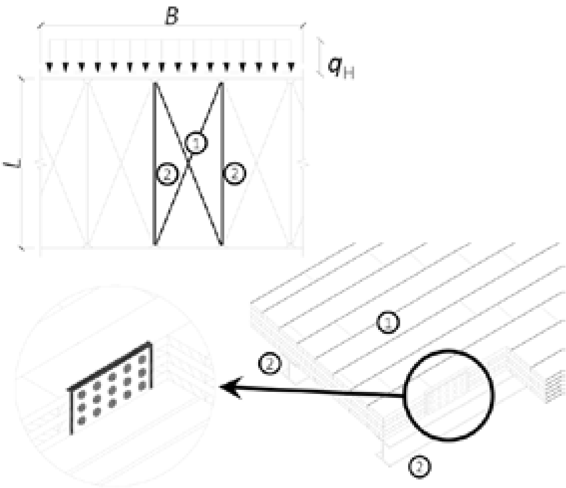


Figure 6: *Typically floor-floor panel connections (a) and new X-RAD connector (b).*

The in-plane behaviour of the horizontal floors constructed from CLT panels and connections is mainly affected by the response of panel-to-panel connections, with the overall length-to-width ratio of the floor and aspect ratio of the CLT panels playing primary role (Ashtari et al. 2014). More generally, the in-plane behaviour of CLT floors depends on the building system, the location of the bracing walls and their stiffnesses (Loss et al. 2015). For multi-storey CLT buildings with cores and additional perimeter shear walls the construction system varies significantly compared to other common CLT structures. In such cases the mechanism of deformation of the floors under in-plane actions can increase the level of shear forces in the connections, due to the distance between the supports and the number and placement of shear walls around the perimeter of the building. Consequently standard connection techniques for CLT elements can be inadequate in terms of capacity and special high performance connections are then required, e.g. Figure 6 (b). Discussion here addresses use of two innovative high-capacity connection technologies suitable for the purpose.

Figure 7 (a) shows a slab made of CLT panels joined together by steel beams, with the beam-to-panel connections designed and engineered from the perspectives of mechanical behaviours of the materials, installation tolerances, feasibility of on-site assembly, and cost. The load-slip curve (*F-δ*) for these connections measured by tests is shown in Figure 7 (b) based on Loss et al. (2014). The operating principle of such floors is similar to a truss system in which each pair of steel beams is braced by the CLT panel and related to characteristics of the beam-to-panel connections. In Figure 7 it is presented a beam-to-panel connection solution obtained by the use of steel plates welded to the beam and glued to the CLT panel.



(a)

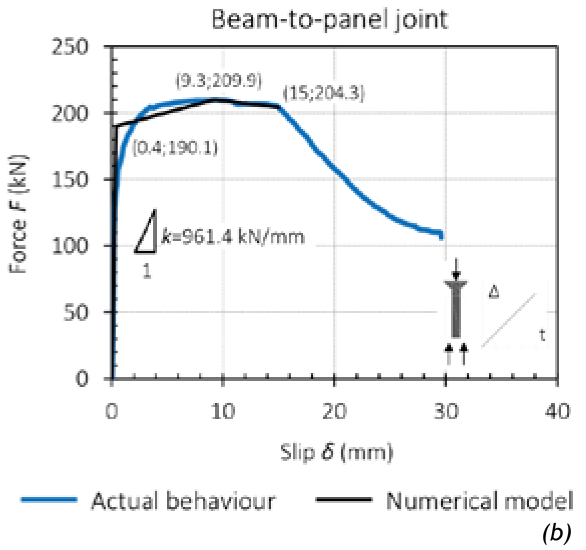


Figure 7: *Innovative hybrid floor system (a) and tests results (b).*

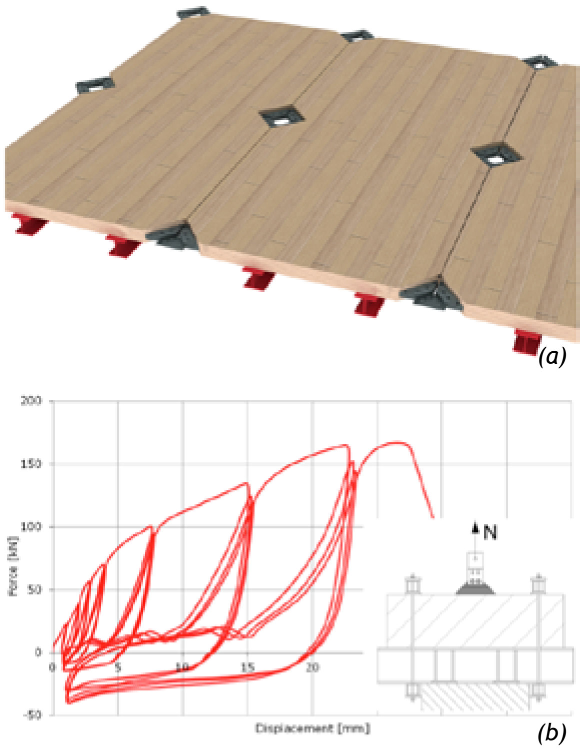


Figure 8: *Innovative hybrid floor system (a); tests setup and test results (b).*

The second innovative connection method discussed here employs X-RAD connectors, Figure 6 (b) that create discrete panel-to-panel joints. This method results in point-to-point mechanical connections in ways that fulfil hold-down and lateral shear resistance functions (Polastri and Angeli 2014). As for shear walls, making point-to-point interconnections lessens the chances that structural systems will fail in unintended ways if overloaded. Figure 8 (a) shows use of X-RAD connectors in a floor diaphragm with the result being ability to transfer very large forces and achieve very high stiffness (Polastri and Angeli 2014). As shown in Figure 8 (b) the load capacity was 171kN and the elastic stiffness 23.6kN/mm.

Although results are not reported here it is to be mentioned that the authors are currently studying use on the described innovative connectors as ways of creating next generation of CLT floor diaphragms. It is anticipated this will enable new applications of CLT like construction of tall building having large footprints and braced by one or more building cores and perimeter shear walls.

6 DISCUSSION AND IMPLICATIONS FOR DESIGN PRACTICE

As the case studies demonstrate, hold-down and shear connections at the bases of CLT wall panels largely determine the behaviors of SFRS. It is therefore crucial to properly represent the stiffnesses of connections during structural analyses from which T_1, peak dynamic forces flowing through wall and connection elements and inter-storey drift are estimated.

For buildings having three to eight storeys T_1 estimates, shear and uplift forces at bases of wall panels, and inter-storey drift can all be miscalculated by substantial margins, Tables 3 to 6. The remainder of this discussion assumes connection stiffnesses derived from test data ($k_{conn} = k_{test}$) are the most accurate and therefore correct estimates of how connections embedded within SFRS actually behave. Although designers could also estimate connection stiffnesses in many other ways, the authors believe it reasonable to suppose that estimating stiffnesses will often be based on information in Eurocode 5 and similar international codes (i.e. $k_{conn} = k_{EC5}$ in case studies).

Case studies suggest T_1 values being underestimated by up to 50 percent is a realistic scenario unless designers use test data to estimate connection stiffnesses. Large errors occurring during subsequent calculation of shear and hold-down forces and inter-storey drift is also highly feasible. In capacities terms estimation of design forces and sizing shear and hold-down connection the likely consequences of how connection stiffnesses are characterized are lesser, with connections being somewhat overdesigned being normal (i.e. based on assuming $k_{conn} = k_{EC5}$). However, errors in estimation of inter-storey drift are likely to be much greater. As results in Table 6 show, interstorey-drift was estimated to be up to four times larger assuming $k_{conn} = k_{test}$ than assuming $k_{conn} = k_{EC5}$.

Based on findings here it suggested that design standards require testing of all connections intended to be used in SFRS constructed partially or completely from CLT wall panels. Furthermore, it is recommended that testing be required to characterize both initial stiffness and capacities of such connections. Also highly desirable is that engineers be given explicit guidance about what constitute appropriate structural model representations of SFRS and appropriate Modal Response Spectrum Analyses. The calculation process for design of connections in Figure 2 is believed suitable as the basis for such guidance.

7 CONCLUSIONS

The primary finding of work reported here is that estimates of the principal vibration periods of buildings with Seismic Force Resisting Systems containing CLT wall panels can be grossly inaccurate if proper attention is not paid to accurate representation of connection stiffnesses. Estimates of T_1 obtained using the simple formula in Eurocode 8 can deviate greatly from values found using finite element models employing connection stiffnesses test data. Similarly finite element model predictions of T_1 in which connection stiffnesses are estimated from information in Eurocode 5 can differ greatly from values obtained using connection test data. Inaccurate representation of connection stiffnesses can also result in incorrect sizing of elements in SFRS, and gross inaccurate in predictions of inter-storey drift. For these reasons it is important that design standards give specific guidance related to determination of initial stiffnesses as well as capacities of connections. A suitable calculation process for design of connections is required based on proposals here dealing with the structural analyses of CLT shear wall systems.

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