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PROPOSAL FOR A STANDARDIZED DESIGN AND MODELING PROCEDURE OF TALL CLT BUILDINGS

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Abstract: *A crucial issue in the design of a mid-rise Cross Laminated Timber (CLT) building under horizontal seismic action, is the definition of the principal elastic vibration period of an entire superstructure. 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. 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. A design procedure suitable for a reliable definition of the connection stiffness is proposed referring to code provisions and experimental tests. Discussion addresses primary issues associated with the usage of proposed procedure for numerical modeling of case study tall CLT buildings is reported.*

Keywords: *CLT structures, core structures, seismic design, shear walls, tall buildings*

1. Introduction

In recent years the Cross Laminated Timber (CLT) panels have become widely employed in Europe and elsewhere to build multi-story residential and commercial buildings. These buildings are often characterized by the presence of many internal and perimeter shear walls. Such typology has been widely studied through experimental tests and numerical simulations methods.

The most comprehensive experimental investigation to date on seismic behaviour of CLT buildings was carried out by CNR–IVALSA, Italy, within the SOFIE Project (Ceccotti, 2008; Ceccotti *et al.*, 2013). Other important investigations have been conducted at the University of Trento, Italy regarding CLT structures (Tomasi and Smith, 2015) and hybrid steel-CLT technologies (Loss *et al.*, 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 characterization failure mechanisms in large shear wall systems (Pozza and Scotta, 2014).

Mostly traditional timber structural systems have employed post and beam arrangement to resist effects of gravity loads, while effects of lateral loads are resisted by cross-

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bracing or non-timber frame infill material (Smith *et al.*, 2009; Tlustochowicz *et al.*, 2010; Bath, 2013). In the modern context an equivalent construction technology is to use beams and columns to resist effects of gravity loads, while CLT building core substructures and other CLT shear walls to resist effects of lateral loads (i.e. earthquake or wind). Such structures are structurally effective, and fail in predictable stable ways if overloaded (Smith *et al.*, 2009). Advantages of such systems include the creation of large open interior spaces, high structural efficiency, and material saving.

Although examples of the new typology have been built already, there has not been full study of the structural behaviour. This means that there is not yet clear understanding of optimal structural configurations, dimensional limitations of spans, minimum number of columns or maximum number of storeys that is feasible.

A preliminary investigation about the response of such building typology is firstly reported in (Polastri *et al.*, 2014) where the behaviour of multi-storey buildings braced with CLT cores and additional shear walls was examined based on numerical analyses of various 3-dimensional configurations adopting two different calibration of the numerical model according to codes and experimental test data respectively Researcher investigate the seismic behaviour of tall CLT building developing new technologies and hybrid steel-timber structures (Ashtari *et al.*, 2014; Bath *et al.*, 2014; Liul *et al.*, 2014). However the most crucial aspects relate to the construction method for CLT building cores and the pertinent issues relate to vertical continuity between storeys, connections between building core elements and elevated floors, and core to foundation connections is not completely investigated yet.

Some additional studies are reported in Polastri *et al.* (2015). In this work a proposal for a standardized procedure to realize a reliable design of CLT superstructures was presented and validated by means of modal response spectrum analyses on various building configurations. In addition the issue of diaphragm in plane behaviour is treated and the interaction with CLT wall detailed.

Despite several studies, it has to be noted that available seismic codes do not provide guidance on the most crucial aspects of how to design structural systems that combine post and beam type frameworks with CLT building cores and shear walls (Smith *et al.*, 2009). The primary issue is that, the estimation of the principal vibration periods, of buildings with Seismic Force Resisting Systems (SFRS) containing CLT wall panels, can be grossly inaccurate if proper attention is not paid to accurate representation of connection stiffnesses.

Estimates of principal elastic period (T_1) obtained using the simple formula in Eurocode 8 (CEN, 2013) 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 (CEN, 2014) 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-story drift.

The aim of this work is provide design guidance related to determination reliable principal elastic vibration period of CLT superstructures starting form proper definition of initial stiffnesses as well as capacities of connections. Therefore a suitable calculation process for design of CLT SFRS is developed, starting form procedure detailed in Polastri *et al.* (2015).

2. Design and modelling procedure for tall CLT buildings

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, Lucisano *et al.*, 2016). 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.*, 2015). 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 the superstructure. During the design process engineers are required to find following a iterative procedure the principal natural frequency ($f_1 = 1/T_1$). Figure 1 represents the proposed iterative procedure: (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 uniaxial 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. In addition: (5) the interaction between the tensile and shear force (or displacement) acting on the connection must be verified in order to avoid working condition inconsistent with the resistant domain of the connection. This interaction can be checked adopting the resistant domain prescribed by connection manufactory (e.g. EOTA) or referring to specific experimental tests that investigate the bi-directional behaviour of the connections. An experimental campaign aimed to define such interaction is on-going at CIRI Buildings & Construction Laboratory of the University of Bologna – Italy.

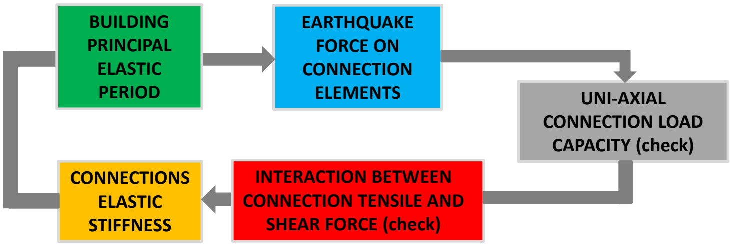


Figure 1. Calculation process for design of connections

According to the scheme in Figure 2, 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, Pavlovic and Fragassa, 2016).

Such definition follows the engineering design practice disregarding the effect of the interaction between tensile and shear action on the connection resistance. A more accurate approach may consider the effective capacity of connection in both tensile and shear. However no experimental results are available to define the interaction between tensile and shear connection capacity therefore such approach is applicable only for research purposes (Pozza *et al.*, 2015). The preliminary calculation does not involve the definition of T_1 accounting for effects of connection stiffness but refers to a priori definition (e.g. according to CEN, 2013) of the periods referring just to the number of storey and to the building typology.

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 and therefore realistic preliminary estimation of T_1 using a more precise Natural Frequency analyses in a specifically developed building numerical model. Then T_1 can be used 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. Finally it is necessary to verify the compatibility of the bi-directional action and displacement of the connection with the relative interaction domain. In detail the displacement achieved by connection should be verified bot for tensile and shear actions in order to avoid brittle failures or relevant strength reductions.

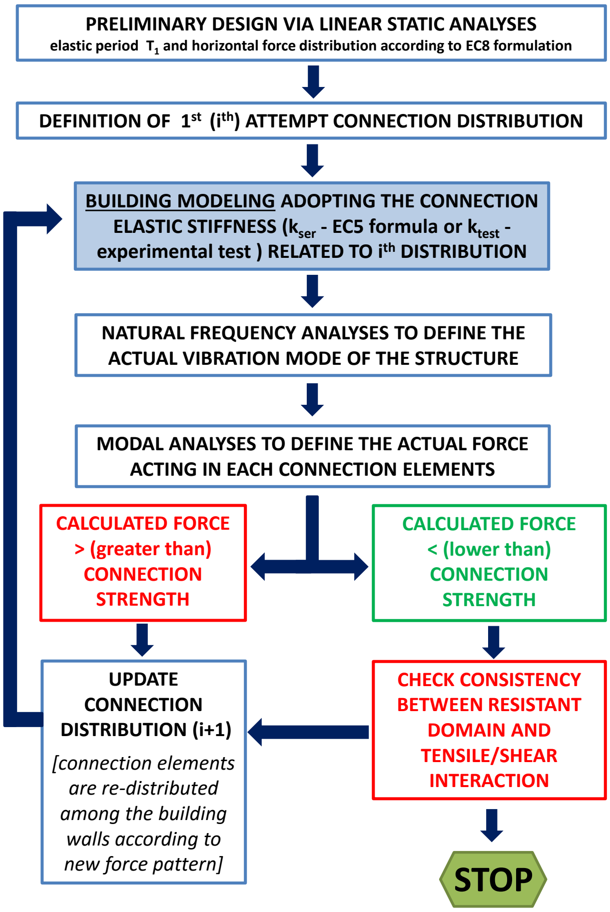


Figure 2. Calculation process for design of connections

Afterward it is possible to perform a more iterative precise frequency analyses until solutions, including connection designs. Results obtained in different interaction (until convergence) should be significantly different as reported in section 4.5.

Proposed procedure is based on simplified Linear Dynamic Analyses (i.e. Modal Response Analyses) adopted to calculate connection's actions and displacements. Finally it is necessary to highlight that when more refined numerical models are used to reproduce connection behaviour (e.g. those used in Pozza and Scotta, 2014) and Nonlinear Analyses are performed, some steps of the proposed procedure are unnecessary, in particular those regarding the check of consistency of calculated action and one- or bi-directional capacity of connections.

3. Connectors mechanical characterization

The most important step in the procedure for the design of CLT buildings, consist in the reliable definition of the elastic stiffness of the connections.

The connection's elastic stiffness and capacity can be evaluated referring to results from monotonic or cyclic tests or to analytical calculations according to appropriate design code.

Below stiffness and capacity values implemented into the numerical models described in Section 4 were calculated directly from experimental data and from analytical formula provided by code.

3.1. Experimental characterization of CLT metal connector elements

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 studies conducted in the last years. The first considered study was carried out at CNR-IVALSA (Gavric *et al.*, 2011), the second study at the University of Trento (Tomasi and Smith, 2015). Both 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.

Figure 3 reports the geometrical characterization the fastening systems adopted in the four examined connections: angle brackets BMF 100 x 100 and Rothoblaas TITAN TTF200 (EOTA, 2012) used as shear elements and hold downs Rothoblaas WHT 540 and WHT 620 (EOTA, 2011).

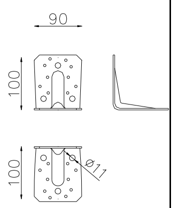
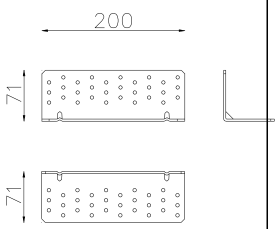
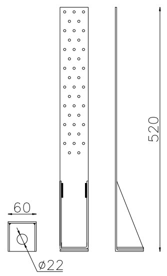
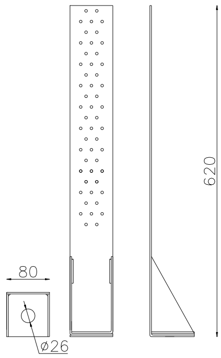
Connector	angle brackets BMF 100 x 100	angle TITAN TTF200	holdown WHT 540	holdown WHT 620
Geometrical detail				
n° and type of nails	n°12 Anker 4x60	n°30 Anker 4 x 60	n°12 Anker 4 x 60	n°32 Anker 4 x 60

Figure 3. Geometrical detail and fastener systems of investigated connections

Figure 4 reports the experimental load displacement curve for holdown WHT 540 and angle brackets BMF 100 obtained by Gavric *et al.* (2011). Figure 5 shows the experimental load displacement curve for holdown WHT 620 and angle brackets TITAN TTF200 obtained by Tomasi *et al.*, (2015) and reported in Polastri *et al*. (2015).

a) b)

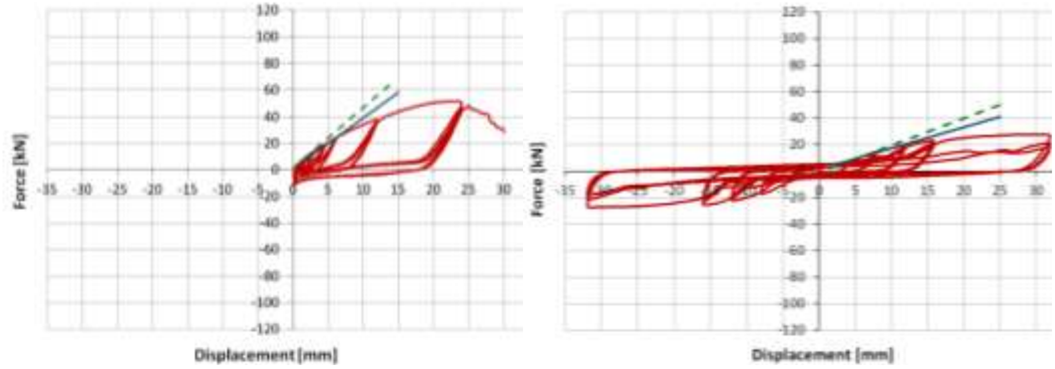


Figure 4. Typical tests results: holdown WHT 540 (a) and angle bracket BMF 100 (b)

a) b)

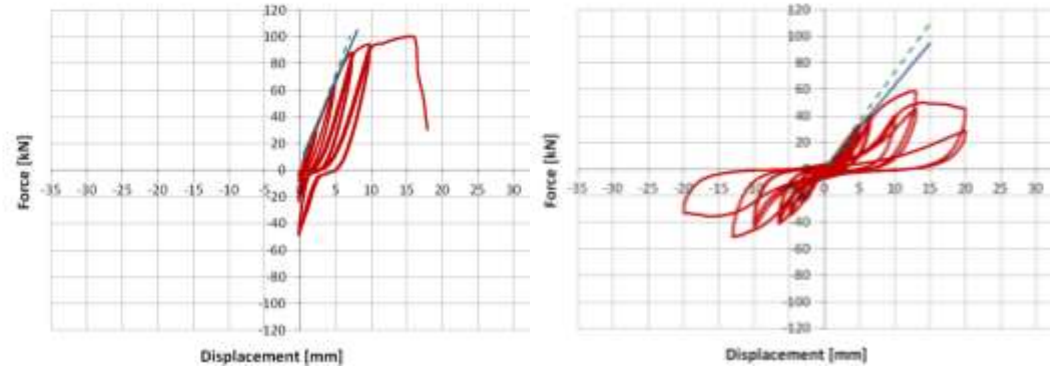


Figure 5. Typical tests results: holdown WHT 620 (a) and angle bracket TTF200 (b)

Since this paper deals with Linear Dynamics Analysis of superstructure systems only the parameters that characterize the maximum load at failure (F_{max}) and the elastic properties of connections (k_{test}) are reported here, Table 1.

The maximum load at failure is evaluated as the peak force achieved during the tests (CEN, 2006) and listed in Table 2 for the examined connection elements.

The initial stiffness was firstly calculated according to ‘method b’ specified by EN 12512 (CEN, 2006) that permits description of the mechanical behaviour of trends representing elastic phase and post-elastic phase responses (Piazza *et al.*, 2011). Such approach define the elastic branch of bilinear approximation of experimental curve referring to the gradient between the point on the load-slip curve corresponding to 0.1 F_{max} and the point on the load-slip curve corresponding the 0.4 F_{max}.

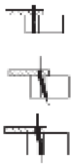
Since the elastic branch provided by “method b” is conventional in some particular cases it does not fit properly the experimental elastic stiffness of the connection. Consequently an additional estimation of the experimental elastic stiffens is provided referring to the actual initial stiffens according to the results provided by “method a” (CEN, 2006). The experimental stiffness estimation of examined connections are reported in Table 2 and in Figure 4 and Figure 5: dashed line represents the stiffness calculated according “method a” whereas continuous line represents the stiffness according “method b”.

3.2. Analytical definition of strength and stiffness according to Eurocode 5

The lateral load bearing capacity of connectors can be calculated according to Johansen's yield theory (Johansen, 1949). Standards (CEN, 2014) base formulas for estimating the lateral capacity of slender fasteners like nails on the aforementioned theory. Here, the Eurocode 5 (CEN, 2014) definition is used to calculate the characteristic shear capacity per nail, $F_{v,Rk}$ (CEN, 2014). For the pertinent case, thick steel plate, representing the vertical leg of hold-down anchor or of angle bracket, the load capacity per nail is (Equation 1).

$$F_{v,Rk} = \min \begin{cases} f_{h,k} t_1 d \\ f_{h,k} t_1 d \left[\sqrt{2 + \dfrac{4 M_{y,Rk}}{f_{h,k} d t_1^2}} - 1 \right] + \dfrac{F_{ax,Rk}}{4} \\ 2{,}3 \sqrt{M_{y,Rk} f_{h,k}}\, d + \dfrac{F_{ax,Rk}}{4} \end{cases} \quad (1)$$

c) d) e)



Where: $f_{h,k}$ = characteristic embedment strength in the timber member; t_1 = depth of penetration of the fastener into the timber member; d = diameter of fastener; $M_{y,Rk}$ = characteristic fastener yield moment; $F_{ax,Rk}$ = characteristic withdrawal capacity of the fastener (associated with pulling the fastener out of the timber member). Characteristic embedment strength $f_{h,k}$ was computed according to Eurocode 5 for nails without predrilled holes (CEN, 2014) (equation 2):

$$f_{h,k} = 0.082\, d^{-0.3}\, \rho_k \quad (2)$$

Where ρ_k is the characteristic value of panel density. Fastener yield moment $M_{y,Rk}$ and withdrawal capacity of fastener $F_{ax,Rk}$ are parameters computed according to the manufacturers' technical certifications and are therefore product-specific. Application of equations (1) and (2) results in calculation of a basic design resistance, which does not include the necessary adjustments to obtain the proper design resistances, taking into account also the partial coefficients of materials.. The design capacity per fastener/nail is therefore computed according to equation (3):

$$F_{d,\,nail} = F_{v,Rk}\, k_{mod} \,/\, \gamma_m \quad (3)$$

Where kmod is the modification factor with regards to the combined influences of duration of loading and moisture; γm is the partial coefficient of the materials.

The analytical fastener stiffness is calculated according to the formula given by Eurocode 5 (CEN, 2014) for steel to timber connections reported in Equation 4.

$$K_{ser} = 2 \cdot (\rho_m 1.5\, d^{0.8} / 30) \quad (4)$$

where ρm is the mean density of the panel and d is the nail diameter.

The following values have been assumed according to information on the material property of fastener and CLT: My,Rk = 6.55 Nm; Fax,Rk = 1.32 kN; t1 = 55.6 mm; ρk = 350 $kg/m^{3,}$ ρm = 420 kg/m^3. Moreover the following design coefficients has been used: kmod = 1.10, γm = 1.00, matching values suggested by Eurocode 5 (CEN, 2014) in a seismic design perspective. This results in the predicted design values for lateral load resistance $F_{d,\ nail}$, listed in Table 1 for the three failure mode provided by Johansen's yield theory.

Table 1. Eurocode 5 derived nail capacity and stiffness

SINGLE NAILS (ANKER 4 x 60)		mode c	mode d	mode e
Characteristic capacity	$F_{v,k}$ [kN]	4.06	2.16	1.93
Design capacity	$F_{v,d}$ [kN]	4.47	2.38	2.12
Slip moduli	k_{ser} [kN/mm]	1.32		

Starting from the load capacity and the elastic stiffness of the nail it is possible to calculate the analytical global properties of the examined connections. In this case, due to the large spacing of the nails, no reduction effects, both for capacity and stiffness, are considered.

In addition the initial stiffness of connectors was calculated taking into account only the stiffness of the steel-to-timber nailed joints. The deformation of steel parts within the connections has been neglected because it is very small compared deformation of nailed joints. Characteristic load-carrying capacities, $F_{v,Rk}$, and slip moduli, k_{ser} are listed in Table 2.

Table 2. 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 ($F_{y,Rd}$)
	“method a”	“method b”			
BMF 100	3.2	2.5	7.3	23.5	14.3
TTF 200	9.8	8.2	23.1	70.1	35.5
WHT 540	6.1	4.5	15.8	48.3	25.4
WHT 620	16.3	12.1	42.2	100.1	67.8

The axial resistance and slip moduli calculated above refer to the hypothesis that the load carrying capacity of the nails was weaker than the steel plate one according to the capacity design principles in timber structures (Fragiacomo *et al.*, 2011). Moreover the calculations follows the engineering design practice disregarding the effect of the lateral action on the holdown resistance and stiffens. The analytical procedures provided by codes don’t account for the elastic and plastic deformability of the steel plates and base bolts consequently the calculated stiffness values result higher than those obtained from experimental tests.

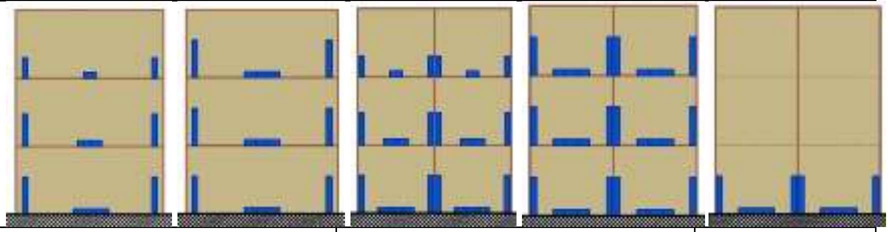
4. Numerical analysis of core tall buildings

The behaviour of multi-storey buildings braced with cores and CLT shear walls is examined using numerical modal response spectrum analyses, with connection properties calibrated based Eurocode 5 (CEN, 2014) and experimental test discussed in Section 3. Analyses followed the scheme in Figure 2 and are presented in terms of principal elastic periods, base shear and up-lift forces, and inter-storey drift. In addition the variability of these results will be presented in the various interactions of the methods, still the convergence.

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 (i.e. jointed or un-jointed wall panels), construction methodology (i.e. storey by storey shear-walls or multistorey shear-walls), and regularity of connectors as a function of the height within a superstructure, Table 3.

Table 3. 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	Unjointed wall panels		Jointed wall panels		Unjointed wall panels
Elevation regularity	Regular	Irregular	Regular	Irregular	Regular
Construction methodology	Storey by storey shear wall system				Multy-storey shear wall system

4.2. 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. Storey height is 3m in all cases, resulting in total superstructure heights of 9m, 15m and 24m respectively (3-5-8 configurations). 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. Finally the distribution of connection elements adopted among the height of the examined building configurations is reported in Table 4.

Table 4. In height connections distribution for the examined building configuration

BUILDING ID	CONNECTION TYPE	CONNECTION LINE AMONG THE BUILDING HEIGHT							
		Base	Floor 1	Floor 2	Floor 3	Floor 4	Floor 5	Floor 6	Floor 7
3 A R	HOLDOWN ANGLE BRACKET	WHT 620 TTF 200	WHT 540 BMF 100	WHT 540 BMF 100	-	-	-	-	-
3 A I	HOLDOWN ANGLE BRACKET	WHT 620 TTF 200	WHT 540 BMF 100	WHT 540 BMF 100	-	-	-	-	-
3 B R	HOLDOWN ANGLE BRACKET	WHT 620 TTF 200	WHT 540 BMF 100	WHT 540 BMF 100	-	-	-	-	-
3 B I	HOLDOWN ANGLE BRACKET	WHT 620 TTF	WHT 540 BMF	WHT 540 BMF	-	-	-	-	-

		200	100	100					
3 C R	HOLDOWN ANGLE BRACKET	WHT 620 TTF 200	-	-	-	-	-	-	-
5 A R	HOLDOWN ANGLE BRACKET	WHT 620 TTF 200	WHT 620 TTF 200	WHT 540 BMF 100	WHT 540 BMF 100	WHT 540 BMF 100	-	-	-
5 A I	HOLDOWN ANGLE BRACKET	WHT 620 TTF 200	WHT 620 TTF 200	WHT 540 BMF 100	WHT 540 BMF 100	WHT 540 BMF 100	-	-	-
5 B R	HOLDOWN ANGLE BRACKET	WHT 620 TTF 200	WHT 620 TTF 200	WHT 540 BMF 100	WHT 540 BMF 100	WHT 540 BMF 100	-	-	-
5 B I	HOLDOWN ANGLE BRACKET	WHT 620 TTF 200	WHT 620 TTF 200	WHT 540 BMF 100	WHT 540 BMF 100	WHT 540 BMF 100	-	-	-
5 C R	HOLDOWN ANGLE BRACKET	WHT 620 TTF 200	-	-	-	-	-	-	-
8 A R	HOLDOWN ANGLE BRACKET	WHT 620 TTF 200	WHT 620 TTF 200	WHT 620 TTF 200	WHT 620 TTF 200	WHT 620 TTF 200	WHT 540 BMF 100	WHT 540 BMF 100	WHT 540 BMF 100
8 A I	HOLDOWN ANGLE BRACKET	WHT 620 TTF 200	WHT 620 TTF 200	WHT 620 TTF 200	WHT 620 TTF 200	WHT 620 TTF 200	WHT 540 BMF 100	WHT 540 BMF 100	WHT 540 BMF 100
8 B R	HOLDOWN ANGLE BRACKET	WHT 620 TTF 200	WHT 620 TTF 200	WHT 620 TTF 200	WHT 620 TTF 200	WHT 620 TTF 200	WHT 540 BMF 100	WHT 540 BMF 100	WHT 540 BMF 100
8 B I	HOLDOWN ANGLE BRACKET	WHT 620 TTF 200	WHT 620 TTF 200	WHT 620 TTF 200	WHT 620 TTF 200	WHT 620 TTF 200	WHT 540 BMF 100	WHT 540 BMF 100	WHT 540 BMF 100
8 C R	HOLDOWN ANGLE BRACKET	WHT 620 TTF 200	-	-	-	WHT 620 TTF 200	-	-	-

The number and the distribution of the connection in the SFRS is calculated for each examined configuration following the procedures reported in Figure 2.

4.3. 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 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, 2013). The coefficient kr was taken equal to 1.0 for regular configurations and 0.8 for non-regular configurations.

Figure 6 shows adopted design spectra, and 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.

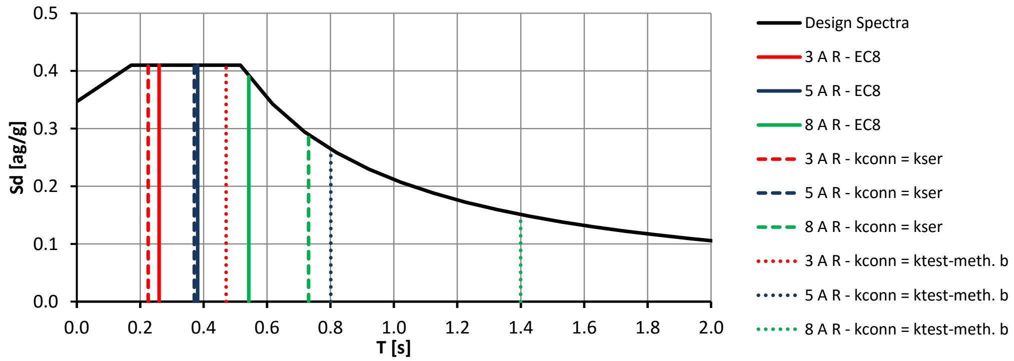


Figure 6. Design spectra and calculated periods according to CEN 2013

4.4. 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 7 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 stiffness's in Table 2.

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.

4.5. Finite element (FE) models

Following the iterative procedure reported in Figure 2, the variation of calculated building principal elastic periods (T_1) among the iterations was investigated by means of modal response spectrum analyses of the case study buildings. The procedure converges in tree iterations for all examined building configurations. Figure 8 reports obtained values for each iteration of the procedure. 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 ($k_{test\text{-}method}$ *"a"* / "b"). The reference T_1 values obtained using the Eurocode 8 (CEN 2013) approach are also reported.

Figure 9 shows that the difference between the predicted values of principal elastic period (T_1) at first iteration and at convergence (delta) can be relevant and spanning form 10% to 45% for the investigated case study buildings. In addition it is found that such error depends substantially from two variables: (1) in height building regularity and (2) number of storey.

Regarding to the in height regularity (Figure 9) it results that the error “delta” for the non-regular configuration (ie. A-B I 3-5-8) is about 10% greater than he correspondent regular configuration (ie. A-B-C R 3-5-8). It means that the 1st temptative connection distribution provided by approximated Linear Static Analysis is acceptable only for regular configuration and cannot be used for non-regular configuration without relevant error.

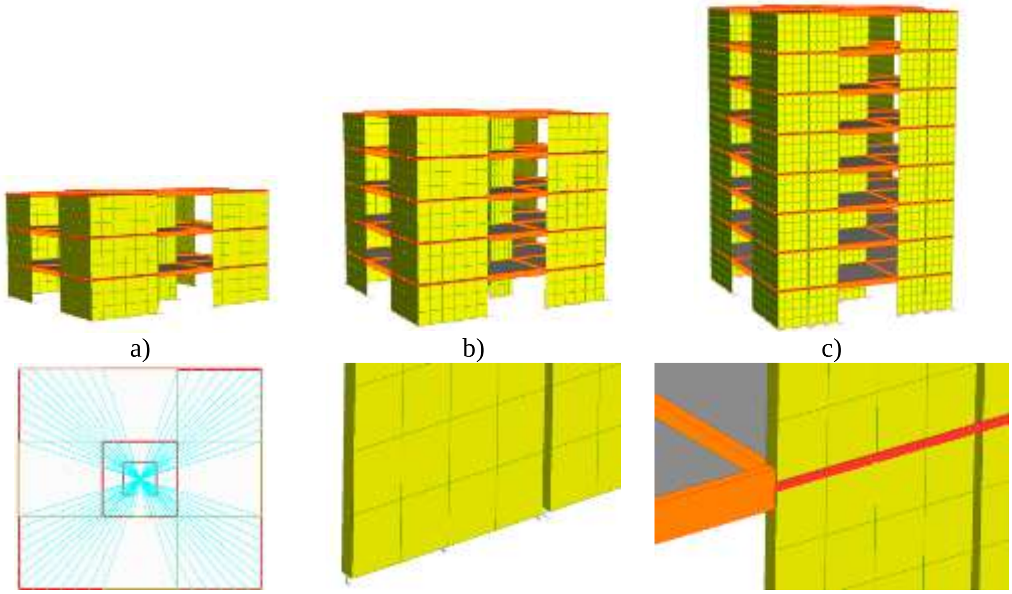

a) b) c) d) e) f)

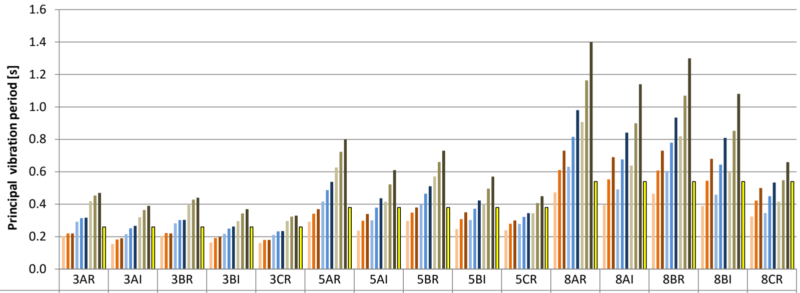
Figure 7. View of the adopted FE model: a) 3AR configuration, b) 5AI configuration, c) 8BR configuration, d) rigid story diaphragm detail, e) base connection detail and f) inter story connection detail

Similarly the storey number affect the reliability of the results obtained with the preliminary Linear Static Analysis: increasing the height of the building the error “delta”, on the principal elastic period values (T_1), increases of about 10% switching from 3 storey to 5 storey and of 25% switching from 3 storey to 8 storey.

Results in Figure 9 shows also that the different values of stiffness assigned to connections does not affect significantly the variability of results among the iterations still convergence.

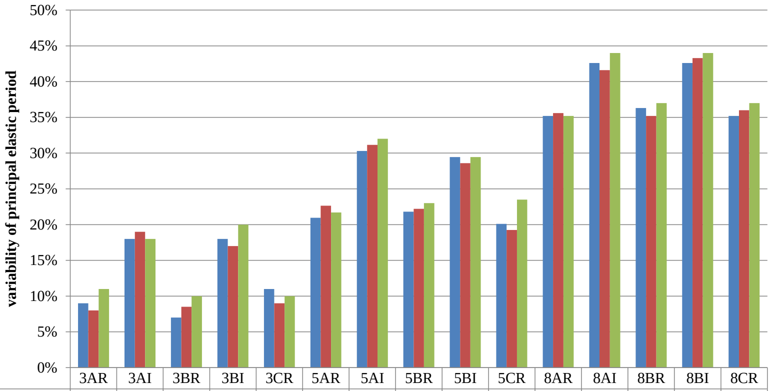
4.6. Analysis results

Results presented here refer to the convergence condition of the iterative procedure and were obtained by modal response spectrum analyses of case study buildings, Figure 10 to 13. Those figures 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).



	3AR	3AI	3BR	3BI	3CR	5AR	5AI	5BR	5BI	5CR	8AR	8AI	8BR	8BI	8CR
1st - kconn. = kser	0.20	0.16	0.20	0.16	0.16	0.29	0.24	0.30	0.25	0.24	0.47	0.40	0.47	0.39	0.32
2nd - kconn. = kser	0.22	0.18	0.22	0.19	0.18	0.34	0.30	0.35	0.31	0.28	0.61	0.55	0.61	0.55	0.42
conv.- 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
1st - kconn. = ktest_"meth. a"	0.29	0.22	0.28	0.22	0.21	0.42	0.30	0.40	0.30	0.28	0.63	0.49	0.61	0.46	0.35
2nd - kconn. = ktest_"meth. a"	0.31	0.25	0.30	0.25	0.23	0.49	0.38	0.46	0.37	0.32	0.82	0.68	0.78	0.64	0.45
conv. - kconn. = ktest_"meth. a"	0.32	0.27	0.30	0.26	0.23	0.54	0.44	0.51	0.42	0.35	0.98	0.84	0.93	0.81	0.53
1st - kconn. = ktest_"meth. b"	0.42	0.32	0.40	0.30	0.30	0.63	0.41	0.57	0.40	0.34	0.91	0.64	0.82	0.60	0.42
2nd - kconn. = ktest_"meth. b"	0.45	0.36	0.43	0.34	0.32	0.72	0.52	0.66	0.50	0.41	1.16	0.90	1.07	0.85	0.55
conv. - kconn. = ktest_"meth. b"	0.47	0.39	0.44	0.37	0.33	0.80	0.61	0.73	0.57	0.45	1.40	1.14	1.30	1.08	0.66
Eurocode 8	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

Figure 8. Calculated principal elastic periods (T_1) for the three iterations of the procedure

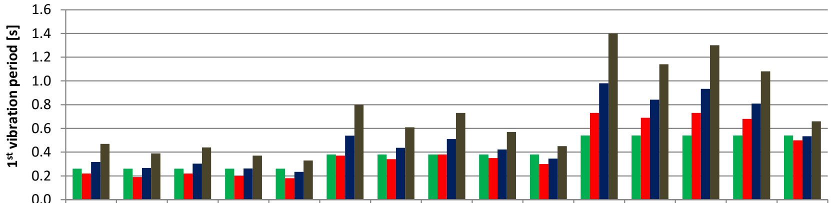


	3AR	3AI	3BR	3BI	3CR	5AR	5AI	5BR	5BI	5CR	8AR	8AI	8BR	8BI	8CR
delta - T1_kconn. = kser	9.0%	18.0%	7.0%	18.0%	11.0%	21.0%	30.3%	21.8%	29.5%	20.1%	35.2%	42.6%	36.3%	42.6%	35.2%
delta - T1_kconn. = ktest_"meth. a"	8.0%	19.0%	8.5%	17.0%	9.0%	22.7%	31.2%	22.2%	28.6%	19.3%	35.6%	41.6%	35.2%	43.3%	36.0%
delta - T1_kconn. = ktest_"meth. b"	11.0%	18.0%	10.0%	20.0%	10.0%	21.7%	32.0%	23.0%	29.5%	23.5%	35.2%	44.0%	37.0%	44.0%	37.0%

Figure 9. Calculated free edge base uplift forces (N) for the three iterations of the procedure

The given alternative values represent effects of taking connection stiffnesses (k_{conn}) equal to values derived from Eurocode 5 (k_{ser}) versus values derived from experiments (k_{test}). Figure 10 include also T_1 calculated with the simplified formula given by (CEN, 2013). Inter-storey drift was calculated for each case study building using the Modal Response Spectrum Analyses and the Damage Limit State design spectrum.

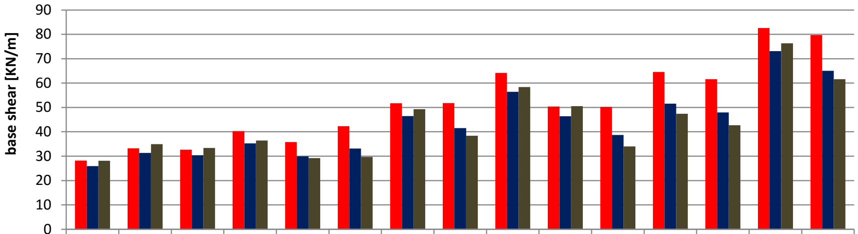
Observing Figure 10 it is apparent that: (1) in most cases use of experimental connection stiffnesses (k_{conn} = k_{test}) leads to much larger T_1 values than those predicted based Eurocode 5 based estimates of connection stiffnesses (k_{conn} = k_{ser}); (2) using the simple formula given by Eurocode 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; (3) "method a" of EN 12512 (CEN, 2006) provide stiffness values closer to those form analytical prediction based on Eurocode 5 (CEN, 2014) formula. However results suggest that neither of those approaches are reliable ways of estimating principal natural periods of buildings having SFRS consisting of CLT cores and perimeter shear walls.



	3AR	3AI	3BR	3BI	3CR	5AR	5AI	5BR	5BI	5CR	8AR	8AI	8BR	8BI	8CR
T1_Eurocode 8	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
T1_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
T1_kconn. = ktest_"method a"	0.32	0.27	0.30	0.26	0.23	0.54	0.44	0.51	0.42	0.35	0.98	0.84	0.93	0.81	0.53
T1_kconn. = ktest_"method b"	0.47	0.39	0.44	0.37	0.33	0.80	0.61	0.73	0.57	0.45	1.40	1.14	1.30	1.08	0.66

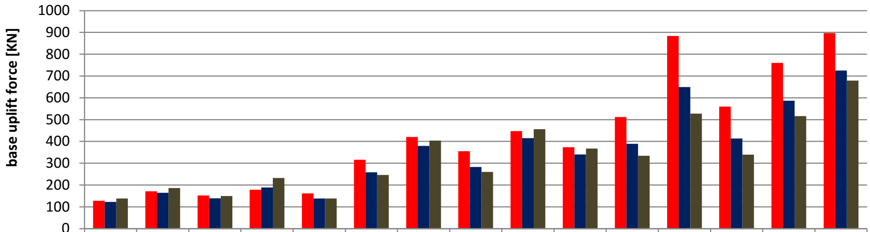
Figure 10. Predicted principal elastic periods (T1)

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 the method used to estimate the connection stiffnesses can alter design force and lateral drift estimates by substantial amounts, Figure 11 to 13.



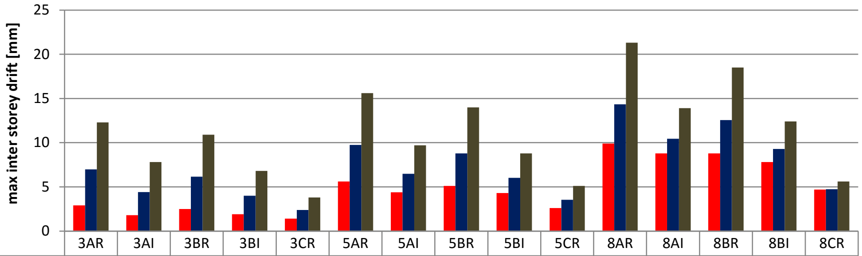
	3AR	3AI	3BR	3BI	3CR	5AR	5AI	5BR	5BI	5CR	8AR	8AI	8BR	8BI	8CR
v_kconn. = kser	28.20	33.20	32.70	40.30	35.80	42.30	51.70	51.80	64.20	50.40	50.20	64.60	61.60	82.60	79.70
v_kconn. = ktest_"method a"	25.90	31.33	30.41	35.28	29.90	33.12	46.46	41.49	56.40	46.41	38.73	51.52	47.98	73.09	65.00
v_kconn. = ktest_"method b"	28.10	34.90	33.40	36.40	29.20	29.70	49.30	38.40	58.40	50.50	34.00	47.40	42.70	76.30	61.60

Figure 11. Predicted base shear per unit of length (v)



	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_"method a"	122.6	164.0	138.6	188.7	137.9	258.6	379.2	283.1	415.4	340.4	389.2	649.2	413.5	586.9	724.9
N_kconn. = ktest_"method b"	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

Figure 12. Predicted free edge base uplift forces (N)



	3AR	3AI	3BR	3BI	3CR	5AR	5AI	5BR	5BI	5CR	8AR	8AI	8BR	8BI	8CR
d_kconn. = kser	2.9	1.80	2.50	1.90	1.40	5.60	4.40	5.10	4.30	2.60	9.90	8.80	8.80	7.80	4.70
d_kconn. = ktest_"method a"	6.99	4.42	6.16	4.00	2.39	9.75	6.49	8.79	6.03	3.54	14.35	10.44	12.56	9.29	4.74
d_kconn. = ktest_"method b"	12.30	7.80	10.90	6.80	3.80	15.60	9.70	14.00	8.80	5.10	21.30	13.90	18.50	12.40	5.60

Figure 13. Predicted maximum inter-storey drift (δ)

Comparing results of Figure 11 (base shear force) with those of Figure 12 (base uplift force) it is possible to observe that the variability of base shear force values induced by different stiffnesses estimation is lower than that of base uplift force values especially for the 5 and 8-storey configuration. This means that for high-rise buildings the lateral deformability is mainly controlled by holdown stiffness that define the rocking behaviour of shear walls. Otherwise the sliding effects is reduced and the seismic horizontal action is uniformly distributed among the angle brackets connections.

The predicted free edge base uplift forces provided by the estimation of connection stiffness according to code (i.e. $K_{conn} = k_{ser}$) are much greater than those given by test stiffness estimation (i.e. $K_{conn} = k_{test}$) particularly for 8-storey configuration. A direct consequence of this is that using analytical estimation of connection stiffness induces an overdesign of holdown connections for high-rise CLT buildings. On the contrary, for low- and mid-rise building configuration the effect of connection stiffness estimation on base shear and uplift forces is small and stable among the different examined configurations.

Another aspect that can be deduced form obtained results shown in Figure 11 and 12 is that the non-regular configuration are characterized by a greater susceptibility to the variation of connection stiffness estimations.

Estimates of predicted values of inter-storey drift (δ) reported in Figure 13 are really sensitive to connection stiffness estimation particularly for eight-storey buildings.
As results inter-storey-drift was estimated to be up to four times larger assuming $k_{conn} = k_{test\text{-}meth.\ “b”}$ than assuming $k_{conn} = k_{EC5}$. Inter-storey drift values provided by connection estimation based on experimental method “a” (i.e. $k_{conn} = k_{test\text{-}meth.\ “a”}$) appear to be the most suitable approach for a reliable estimation of lateral deformation of the SFRS.

5. Conclusions

Results obtained in this work demonstrate that hold-down and shear connections used to assemble and connect 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 principal elastic period, peak dynamic forces flowing through wall and connection elements and inter-storey drift are estimated.

A specific design procedure suitable for an efficient and safe design of mid- and high-rise CLT buildings is proposed basing on reliable definition of the connection stiffness via code provisions and experimental tests. The main feature of this procedure consists in the iterative approach necessary to define the best connection arrangement among the shear walls and therefore the reliable estimation of the building global stiffness.

The iterative process start with a preliminary design of connection systems based on code provisions both for principal elastic period value and connection's parameters. Then the connection distribution can be iteratively refined referring to more precise analyses for the estimation of the principal elastic periods and therefore of the action on the connection elements. The procedure also include a verification of the interaction between the tensile and shear force or displacement according to specific resistant domain prescribed by standards.

The crucial aspect of the proposed procedure consists in the choice of the best approach for a reliable estimation of the connection elastic stiffness. In this work two different approach are used: the first one is analytical and based on Eurocode 5 (CEN, 2014) provisions while the second one refer to the experimental tests and to the stiffness estimation provided by EN 12512 method "a" and "b" (CEN, 2006).

Validation of proposed procedures on 15 different case study building configurations, characterized by different number of storeys and connection arrangement, demonstrate that the difference between the predicted values of principal elastic period at first iteration and at convergence is relevant (form 10% to 45%) and depends substantially from in height building regularity and number of storey. Procedure provided by codes sounds suitable only for building up to 3-storey characterized by in plant and in height regular shear walls and connections distribution.

Regarding to the effect of stiffnesses estimation, case studies suggest that principal elastic periods values 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.

Finally results shows that for buildings having three to eight storeys principal elastic period estimates, shear and uplift forces at bases of wall panels, and inter-storey drift can all be miscalculated by substantial margins if a standardized procedure for calculating and schematizing of connections behaviour is not followed.

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