Low and Mid-rise hybrid CLT Infilled Steel Moment Resisting Frames Displacement Based Design and application advantages

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Research Article

Keywords:

Posted Date: August 2nd, 2022

DOI: https://doi.org/10.21203/rs.3.rs-1908558/v1

Low and Mid-rise hybrid CLT Infilled Steel Moment Resisting Frames Displacement Based Design and application advantages

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Abstract

Recently, a novel hybrid structure has been developed as an alternative lateral-load resisting system at The University of British Columbia. The hybrid structure includes Cross Laminated Timber (CLT) shear panels as an infill in steel moment-resisting frames (SMRFs). To increase the suitability of the suggested system, first in this paper types of hybridization for timber-steel structure are discussed. In the second step, an iterative design method is expanded and formulated for a hybrid timber-steel structure. The iterative design procedure comprise the following primary modeling variables: the gap between CLT panel and steel frame, bracket spacing, panel thickness and strength, and post-yield stiffness ratio. Later, the design displacement profile is developed by allocating primary relative strength between the CLT wall and frame component. This profile is then applied to acquire the distinctives of an equivalent single degree of freedom (SDOF) system. A mathematical term for the system ductility is formulated based on the proportions of the overturning moment resistance of the CLT wall and SMRF. A determined equivalent viscous damping-ductility relationship is applied to get the energy dissipation of the equivalent SDOF system. In the third study, A new reiterative direct displacement based design method SMRFs with CLT-infill walls has been expanded and analytically confirmed by designing 3-, 6-, and 9-story hybrid buildings primary shear proportions between the wall and frame are allocated at the start of the design process. The system ductility and equivalent viscous damping are clearly accounted.

Introduction

The world's population is increasing annually and, as a result, urbanization and the carbon footprint of construction (United Nations 2015). Compared with conventional construction materials, such as concrete and steel, timber as a renewable material has the lowest environmental collision (Connolly et al. 2018). With the progress of mass timber engineering products and related connector technology (Brandner et al. 2016), timber construction is becoming a more practical and economical alternative, also driven by prefabrication and onsite construction. Of particular interest is cross-laminated timber (CLT). CLT panels are made of several layers of orthogonally glued lumber boards to form a solid panel and show high in-plane strength and stiffness (Brandner et al. 2017), they can withstand lateral wind and seismic loads as floor and roof diaphragms, as well as shear walls, also in earthquake-vulnerable regions (Tannert et al. 2018). Over the past years, to extend the employment of renewable materials within the housing industry, a unique steel-timber hybrid building System was developed and studied at the University of British Columbia and FP Innovations. The hybrid structural System was a steel moment-resisting frame (MRFs) with Cross Laminated Timber (CLT) infill walls. Moderately ductile steel moment frames (SMF) are a familiar lateral load resisting system (LLRS). They allow for a plastic architectural design for low- and mid-rise buildings but become uneconomical for high-rises due to the large member sections to convince the drift requirement. In contrast, lightweight cross-laminated timber (CLT) shear walls supply sufficient stiffness for resisting lateral loads and controlling drift in high-rise construction. A hybrid building uses two or more materials in combination to get the most out of each material. CLT structures are constructed in two ways: platform-type construction and balloon-type

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framing. In the platform building, each floor is a stage for the story above, with the walls connected to the foundation and the floors with steel brackets and hold-downs (HDs). In balloon framing, the walls are constant, and no story limitation is caused by compression perpendicular to the grain strength. Mass timber construction has been comprised of the 2020 version of the National Building Code of Canada (NBCC) (NBCC 2015).

Timber-Steel Hybridization

Even though the material properties of steel and wood are different, by merging them into a hybrid structure one can take advantage of the internal benefits of each material and control their limitations. Steel shines in tension while wood reacts much better to compression. The compression strength of timber ranges from about 20 MPa through to 40 MPa. These quantities are in the range of concrete compressive strengths. Timber is brittle in tension, much like concrete. Many of the principles of reinforced concrete design could apply to reinforced timber design. In general terms, this equates to using the timber components to resist compressive forces and using steel, or other materials, to resist tensile forces. Of course, timber has a remarkable tensile strength, which can be used, as long as the principles of capacity design are appealed, to ensure that a brittle failure mechanism will not happen. Therefore, when making a hybrid wood/steel truss, the wood should be located up on top of the truss in compression, all the hybridization techniques aim to optimally use each material. Hybrid materials can be combined at component levels and/or at the building system levels. Hybrid systems design is often contemplated for aesthetic purposes, sustainability, maximum use of different material properties, etc. Connection detail is the major provocation associated with hybrid structures. Possible new ways of connecting the two materials are discussed (Leckie J 2008). Hybridization can be divided into three types: Component Hybridization, System Hybridization, and Building Hybridization. Component-level hybridization mixed more than one material type within a member. One common example of component level hybridization for the timber-steel hybrid is a flitch beam, made of one or more steel plate(s) sandwiched between pieces of timber. There are several superiorities to this type of system. The steel beam has remarkable higher strength than the timber members, but a steel plate will generally have remarkable issues with lateral-torsional buckling. The timber members supply lateral restraint for the steel preventing lateral-torsional buckling or local buckling. The steel and wood are joined using bolts spread over the length of the beam to convey shear. This connection is a major part of the design to guarantee that the suitable distribution of forces happens without splitting the wood. System-level hybridization requires the use of multiple material types within a structural system. The connections between these members are usually the most complex issue in system-level hybridization. An ordinary example of a hybrid system is steel and wood trusses. Timber trusses can have definite advantages for use in buildings where the stored materials may give an increase to environments that could be corrosive to steel. In such a situation, the amount of steelwork can be kept to a minimum so that expensive anti-corrosion costs are decreased. Typically these are made with a timber top chord for the truss and steel bottom chord. This works to each material's benefit as timber has high compression Strength and steel is best used in tension. Building level hybridization is the composition of building systems of several materials. One example of this is a vertically mixed system. Vertically mixed systems have been finished around the world, with one timber vertically mixed system in Australia recently becoming the tallest timber residential building in the world (Harris M 2012). The lower floor(s), which sustain more load and often have a higher story height, is made entirely from concrete or steel framing; the stories above are then timber-framed. The result is a remarkably reduced total weight of the structure, decreasing the size of the foundations.

Steel and Hybrid Timber-Steel Lateral Systems

Although, in structural design practice, concrete- and steel-based LLRS are still more usual than mass timber (Zhang et al. 2018).The Canadian Steel Code CSA-S16 (CSA 2014) contains many prequalified steel LLRS, counting different arrangements of braced frames, steel plate shear walls, and moderately ductile steel moment-resisting frames (SMF). SMFs were widely used because they permit wide-open spaces and a flexible architectural design (Hou and Tagawa 2009). SMFs comprise a series of beams and columns and rigid connections that convey both lateral and gravity forces. The NBCC specifies a seismic ductility force reduction factor of R_d =3.5 for moderately ductile SMF. The system is regarded a "sway" frame, and the design of SMFs must account for increased P-delta effects. In high buildings, the frame members (beams and columns) are larger and denser than those in the other steel systems, leading to high construction prices and the associated large environmental footprint (Tremblay 2002). Because controlling later drift in taller SMF buildings is burdensome, SMFs are more practical for low- and mid-rise structures. Structural hybridization produces the chance of increasing the stiffness of tall mass-timber buildings while preserving sufficient load resistance. The important advantage in

the hybridization of timber with other materials, from a seismic design point of view, is the low weight of timber that results in smaller seismic forces (Li et al. 2019). Parametric studies assessing the panel adjustment in different bays indicated that the addition of CLT panels to SMFs slightly reduced the system's ductility, significantly increasing system strength and stiffness. The increases are linearly equivalent to the number of additional CLT panels.

Objectives of present research

The aim of present research is the investigation of iterative design procedure for hybrid timber-steel moment frames and also evaluating the effects of CLT infill with different thicknesses, crushing strength, and gaps on shear capacity curves.

Fundamentals of Direct Displacement Based Design for a Hybrid Timber-Steel Structure

A thorough discussion on the basics of the DDBD method for several types of structures is given in Priestley et al. (2007). In this section, the basic steps and formula of DDBD method are discussed. For frame structures, the first step in the DDBD is change of system to an equivalent single degree of freedom system (SDOF) system (Fig. 1). The equivalent SDOF system is defined by secant stiffness at the maximum response. For this changing process, a design displacement profile is required. For the frame type building, the design displacement belong on the drift limits of lower stories Priestley et al. (2007). The characteristics of equivalent SDOF system, i.e., design displacement (Δ_d), effective mass (m_{eff}), and effective height (h_{eff}) are given in Equations 5, 6, and 7, respectively.

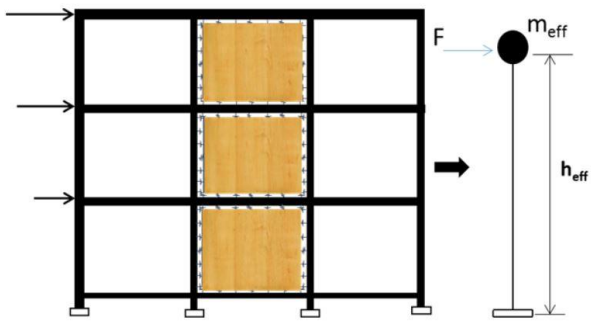

Fig. 1 SDOF representation Priestley et al. (2007).

Once the design displacement (Δ_d) and equivalent viscous damping (ξ_{eq}) are established, the effective period can be obtained from the highly damped displacement spectra (Fig. 4). The corresponding effective stiffness (k_{eff}) (Fig. 2), design base shear, and distributed force vector can be calculated using Equations 14, 15, and 16 respectively.

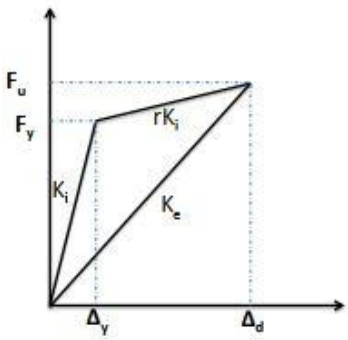

Fig. 2 Effective stiffness Priestley et al. (2007)

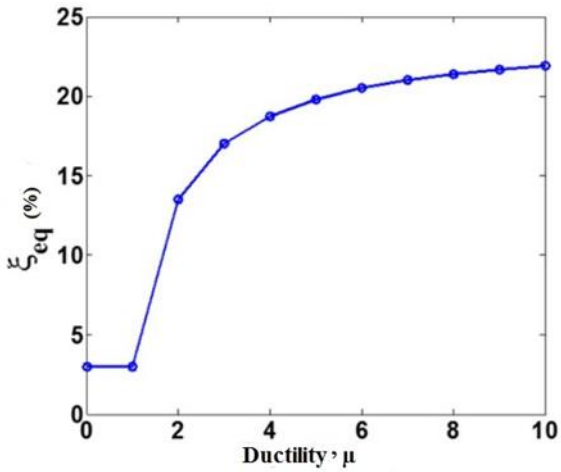

Fig. 3 EVD VS. Ductility Priestley et al. (2007)

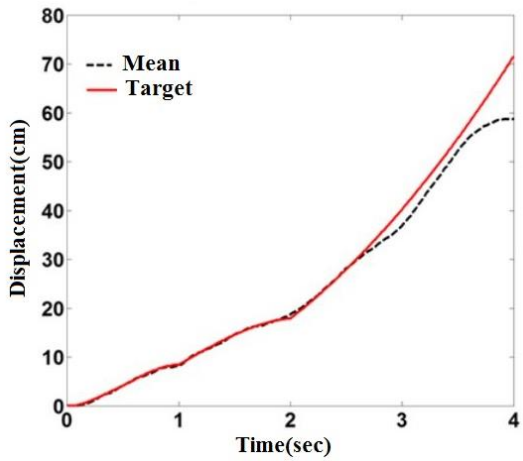

Fig. 4 Design displacement spectra Priestley et al. (2007)

supposing modeling parameters of CLT infilled steel moment resisting frames

Results from multi-objective optimization of drift demands of the hybrid system are used to set the primary modeling parameters CLT infilled SMRFs. A bracket spacing, panel thickness and strength of CLT are used as starting parameters. A bracket spacing 0.8 m, panel thickness and strength of 99 mm and 17.5 MPa, respectively, are used as starting parameters for the current example. Post yield stiffness ratio of 1% is used as an initial starting point

Allocation of lateral load proportions between CLT infill walls and steel moment frames

A concept of beginning strength assignment to compute the characteristics of an equivalent SDOF is acquired from Sullivan et al. (2006). Since the CLT shear panels act in a pure shear manner, it is rational to designate the shear strength proportion at the beginning of the process, as their bending strength is not remarkable. As the first starting point, 70% of the total shear is directly allocated to the frame. The shear resistance for the CLT wall is then obtained by subtracting the SMRF shear ($V_{i,frame}$) from the total shear Sullivan et al. (2006):

$$\frac{V_{i,CLT}}{V_b} = \frac{V_{i,total}}{V_b} - \frac{V_{i,frame}}{V_b} \tag{1}$$

Where V_b is design base shear, $V_{i,CLT}$ is the shear resisted by the CLT infill panels at story i, and $V_{i,total}$ is the total shear the total shear of the system is established from frame plus CLT shear.

$$V_{total} = V_{frame} + V_{CLT} \tag{2}$$

Expanding design displacement profile

The properties of an equivalent SDOF system belong on the drift limit of the lower stories of moment frames and an presumed displacement profile. This displacement profile communicates to the inelastic first mode response of the structure under seismic excitation Priestley et al. (2007).To guarantee the satisfactory performance of the structures under seismic actions, building codes determine limits on lateral story drift values. The NBCC 2010 (2010) puts a 2.5% inter-story drift limit to define extensive damage to the buildings. Previous researches on the CLT infilled SMRFs propose that the CLT panel crushing can be an essential way of energy dissipation and easy for conservation purposes. In their research, the authors indicated that panel crushing happens before the 2.5% inter-story drift limit of the system. Therefore, the drift limit θ_d of 2.5% corresponding to the lower story drift demand of the hybrid system is selected as a target drift limit. The displacement profile is accepted using Equation 3 Sullivan et al. (2010).

$$\Delta_i = \omega_\theta \theta_c h_i \left(\frac{4H_n - h_i}{4H_n - h_1}\right) i \tag{3}$$

where Δ_i is the displacement at level i, h_i is the height of ith floor from the ground, H_n is the total building height, ω_θ is the factor to account for the effects of higher modes and is given as:

$$\omega_\theta = 1.15 - 0.0034 H_n \leq 1 \tag{4}$$

Determining elements of equivalent SDOF system

Equations 5, 6, and 7 are used to compute the design displacement (Δ_d), effective mass (m_{eff}), and effective height (h_{eff}), receptively, for the replacement equivalent SDOF system from the masses lumped in each story (m_i) and height of each story from the base (h_i) Sullivan et al. (2012).

$$\Delta_d = \frac{\sum_{i=1}^{n} m_i \Delta_i^2}{\sum_{i=1}^{n} m_i \Delta_i} \tag{5}$$

$$m_{eff} = \frac{\sum_{i=1}^{n} m_i \Delta_i}{\Delta_d} \tag{6}$$

$$h_{eff} = \frac{\sum_{i=1}^{n} m_i \Delta_i h_i}{\sum_{i=1}^{n} m_i \Delta_i} \tag{7}$$

Determining Hybrid System ductility (μ_{sys})

As crushing of the wood at the join with the nails happens at relatively low drift values, the related ductility is large. However, for the SMRFs the nonlinear response happens at a relatively larger drift value. Therefore, for this design reason, this effect is collected by taking the weighted average ductility correlated with panel crushing and steel yielding as:

$$\mu_{sys} = \frac{M_{CLT}\mu_{CLT} + M_{frame}\mu_{CLT}}{M_{frame} + M_{CLT}} \tag{8}$$

The displacement ductility value of CLT panels and frames is calculated as follows:

$$\mu_{CLT} = \frac{\Delta_d}{\Delta_{crush,CLT}} \tag{9}$$

The crushing displacement for the CLT wall ($\Delta_{crush,CLT}$) is calculated using the deflected shape of the CLT panel (Fig. 5) as follows:

$$\Delta_{crush,CLT} = \frac{d.f_{CLT}}{E_0.\cos\theta} \tag{10}$$

Where E_0, f_{CLT} are the modulus of elasticity and crushing strength of CLT panel, respectively.

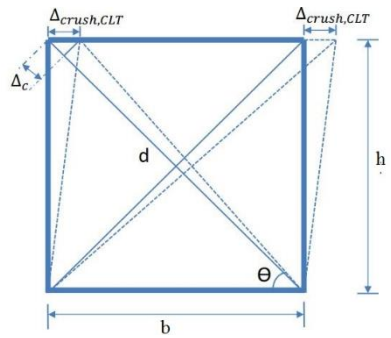

Fig. 5 CLT panel representation with a compression strut

The displacement ductility of the steel moment resisting frame is calculated using Equation 11 Garcia R et al. (2010).

$$\mu_{frame,i} = \frac{\Delta_i - \Delta_{i-1}}{h_i - h_{i-1}}\left(\frac{1}{\theta_{y,steelframe}}\right) \tag{11}$$

Where $\mu_{frame,i}$ the ductility is demand of ith story and $\theta_{y,steelframe}$ is the yield drift of the steel frame as given by Equation 12.

$$\theta_{y,steelframe} = 0.65\varepsilon_y \frac{L_b}{h_b} \tag{12}$$

Where L_b and h_b are the beam span length and depth, respectively.

Developing System equivalent viscous damping

An expression and plots of EVD for SDOF hybrid systems are expanded in Bezabeh (2014) and Bezabeh et al. (2015). Fig. 3 shows the damping ductility law of the SDOF hybrid system with reciprocal modeling variables specified in the primary modeling parameters

Acquiring effective period of the system

The effective period T_{eff} of the equivalent SDOF system is acquired from the highly damped displacement spectrum. A scaling factor is deliberated using Equation 13.

$$\eta = \sqrt{\frac{10}{(5 + \xi)}} \tag{13}$$

Obtaining effective stiffness and design base shear

the effective period and design base shear are calculated in Equations 14 and 15 as follows:

$$K_{eff} = 4\pi 2\frac{m_{eff}}{T^2_{eff}} \tag{14}$$

$$V_b = K_{eff}\Delta_d \tag{15}$$

Performing structural analysis

The above-calculated design shear force is dispensed to execute the structural analysis of the system. The design shear forces at each level of the building (F_i) are computed by Equation 16 Sullivan et al. (2012):

$$F_i = V_b \frac{m_i \Delta_i}{\sum_{i=1}^{n} m_i \Delta_i} \tag{16}$$

A portal method of analysis has been selected to perform the structural analysis because of its simplicity and accuracy.

Designing steel frame members

The plastic moment strengths for the beams and columns are computed in performing structural analysis. Afterwards, the plastic section modulus is computed to select a proper section that persuade the demand. It should be distinguished that the choose of steel cross-sections is accomplished with only the governing load (seismic actions). This supposition is correct for building designs in high seismic terrain. Moreover, Pinto (1997) established a minor difference in the seismic response of structures with or without gravity loads. Priestley et al. (2007) strongly proclaimed the fallacy related with combining seismic actions with gravity actions for the DDBD process. In line with Priestley et al. (2007), the members sections are selected for the higher gravity and seismic moments. As needed by CSA S16-09 (CSA, 2010) CSA (2010), both beams and columns are supposed to be constructed by considering bracing against lateral-torsional buckling. With the above suppositions, the member selection is carried out for beams and columns in the lower stories. Unvaried cross-sections of beams, exterior columns, and interior columns are used all over the height of the building. The section modulus of beams and columns is used to choose the sections from the CISC Handbook (CISC (2010).

Checking for CLT properties

In this section, design investigations have been carried out on at first assumed CLT panel properties. As expressed in the CLT have remarkable impact on the dynamic behavior of the hybrid system. However, as the smallest CLT thickness used is remarkably inflexible, sensitivity of difference in panel strength and panel thickness is negligible. Therefore, smallest CLT panel thickness and strength of 99 mm and 17.5 MPa in the design process are admissible. The steel connection brackets conduct lateral load from the steel frame to the CLT wall Schneider et al. (2012). At the start of the design process, a total of 16 bracket spacing of 0.8 m brackets were used at the top and bottom of the panel. The maximum shear request on the CLT wall is less than the brackets shear force transferring capacity. Therefore, the initially assumed bracket spacing is acceptable.

Low and Mid-rise CLT Infilled Steel Moment Resisting Frames Displacement Based Design Examples

The suggested DDBD procedure is exemplified by designing a case study three buildings, 3,6,and 9 storey -middle bay infilled SMRF. The height of each storey level of the building is 3.2 m. A constant bay width of 6m is used for the whole buildings. The buildings are supposed to be located on a very dense soil and soft rock (site class C) in Vancouver, Canada. The buildings are modelled as a two-dimensional structure and due to its symmetry in plan. Both beam and column component are described based on CSA-S16 code (CSA-S16, 2009) with a yielding strength of Fy = 350 MPa and modulus of elasticity of Es = 200 GPa. A constant floor seismic weight (including the CLT panels) of 253 tons was studied. The suggested design procedure is involved of 11 steps and details of each step are provided for the case study buildings in tables 1-6.

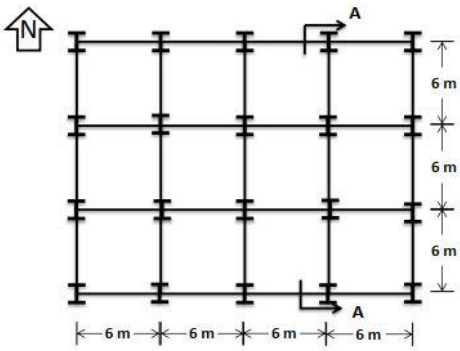

Fig. 6 Buildings floor plan

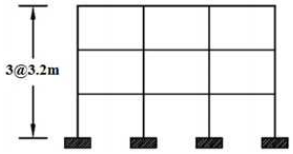

Fig. 7 Elevation view of the 3 storey bare 2D frame

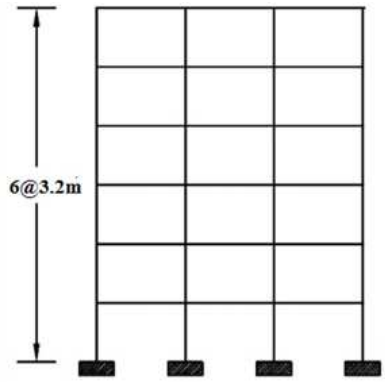

Fig. 8 Elevation view of the 6 storey bare 2D frame

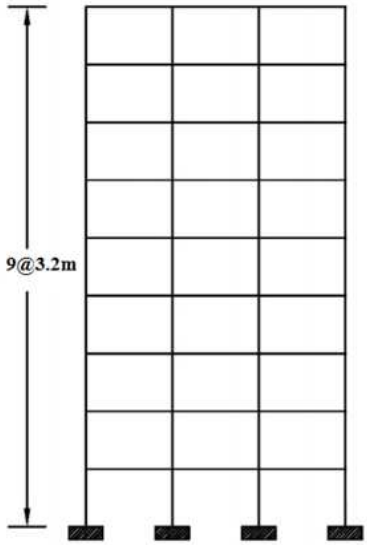

Fig. 9 Elevation view of the 9 storey bare 2D fram

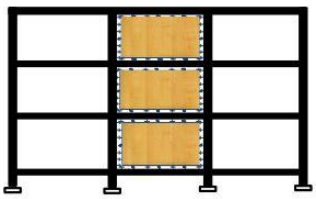

Fig. 10 Elevation view of the 3 storey hybrid 2D frame

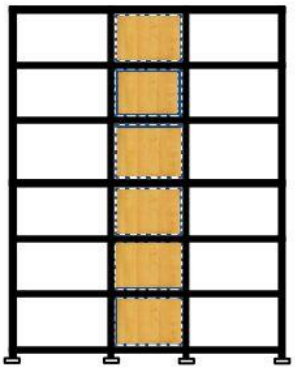

Fig. 11 Elevation view of the 6 storey hybrid 2D frame

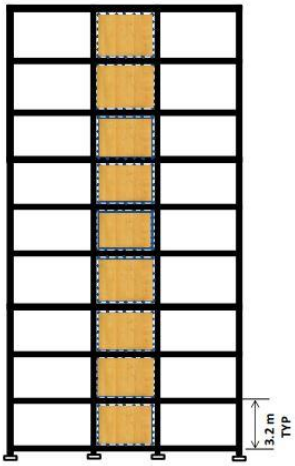

Fig. 12 Elevation view of the 9 storey hybrid 2D frame

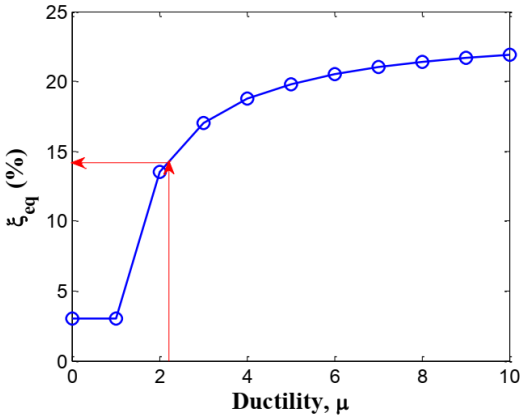

Fig. 13 Equivalent viscous damping (EVD)-ductility relationship for the proposed hybrid system

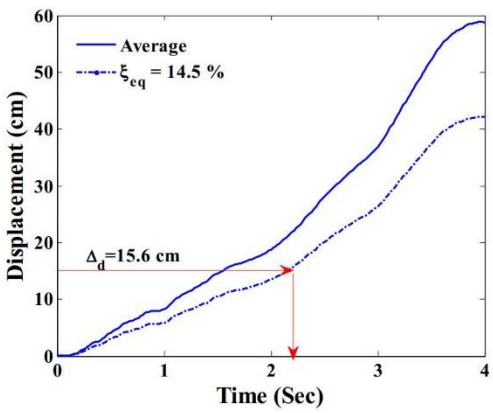

Fig. 14 Effective period from damped displacement spectrum

Table 1 Base shear proportions between frame and walls

Story	$m_i\Delta_i$	F_i (kN)	V_i,total(kN)	V_{frame} (kN)	V_{wall} (kN)
3	49.68	383.6	383.6	268.52	115.08
2	36.8	284.15	667.75	467.42	200.32
1	20.24	156.28	824.03	576.82	247.21

Table 2 Characteristics of equivalent SDOF

Story	H(m)	Δ_i	Θ_i (%)	m_i (ton)	$m_i\Delta_i$
3	9.6	0.196	1.59	253	49.68
2	6.4	0.145	2.04	253	36.8
1	3.2	0.08	2.5	253	20.24

Table 3 Characteristics of equivalent SDOF

Story	$m_i\Delta_i^2$	$m_i\Delta_i h_i$	Δ_d(m)	m_{eff}(ton)	h_{eff}(m)
3	9.75	476.92	0.156	680.9	7.28
2	5.35	235.52			
1	1.61	64.768			

Table 4 Details of DDBD of three story frame

	3 storey
Proportion of V_b assigned for frames (%)	70
Design storey drift, Θ_d (%)	2.5
Design displacement , Δ_d (m)	0.15
Effective height, h_{eff} (m)	7.28
Effective mass, m_{eff} (ton)	680.8
μ_{frame}, average	1.13
μ_{sys}	2.21
ξ_{SDOF} (%)	14.5
Effective period, T_{eff} (sec)	2.26
K_{eff} (kN/m)	5257.4
V_b (kN)	824.04
Beam section	W310 x 52
Interior column section	W360 x 79
Exterior column section	W310 x 45
Beam strength, M_{bi} (kN.m)	261
Interior column strength, Mint:col,$_i$ (kN.m)	444
Exterior column strength, Mext:col,$_i$ (kN.m)	220

Table 5 Details of DDBD of six story frame

	6 storey
Proportion of V_b assigned for frames (%)	50
Design storey drift, Θ_d (%)	2.5
Design displacement , Δ_d (m)	0.28
Effective height, h_{eff} (m)	13.5
Effective mass, m_{eff} (ton)	1287.3
μ_{frame}, average	1.22
μ_{sys}	7.53
ξ_{SDOF} (%)	20.5
Effective period, T_{eff} (sec)	3.2
K_{eff} (kN/m)	4957.9
V_b (kN)	1399.8
Beam section	W310 x 67
Interior column section	W360 x 91

Exterior column section	W310 x 52
Beam strength, M_{bi} (kN.m)	326
Interior column strength, Mint:col,$_i$ (kN.m)	552
Exterior column strength, Mext:col,$_i$ (kN.m)	261

Table 6 Details of DDBD of nine story frame

	9 storey
Proportion of V_b assigned for frames (%)	50
Design storey drift, Θ_d (%)	2.5
Design displacement , Δ_d (m)	0.41
Effective height, h_{eff} (m)	19.73
Effective mass, m_{eff} (ton)	1887.5
μ_{frame}, average	1.28
μ_{sys}	10.21
ξ_{SDOF} (%)	21
Effective period, T_{eff} (sec)	4.2
K_{eff} (kN/m)	4219.9
V_b (kN)	1726
Beam section	W360 x 122
Interior column section	W360 x 162
Exterior column section	W360 x 101
Beam strength, M_{bi} (kN.m)	705
Interior column strength, Mint:col,$_i$ (kN.m)	975
Exterior column strength, Mext:col,$_i$ (kN.m)	584

Conclusions

A new reiterative direct displacement based design method SMRFs with CLT-infill walls has been expanded and analytically confirmed by designing 3-, 6-, and 9-story hybrid buildings primary shear proportions between the wall and frame are allocated at the start of the design process. The system ductility and equivalent viscous damping are clearly accounted. Better advantages were achieved for low and mid-rise hybrid buildings as follow.

- Increasing the number of infilled bays results in a stiffer and stronger system in all cases.
- The increases in strength and stiffness of structure are linearly equivalent to the number of additional CLT panels.
- The ductility of the steel frame used donates minimally to the behavior; therefore, allowing for the cheaper frame design (limited ductility).
- By reducing the weight of the building by using prefabricated CLT panels instead of heavy structural elements such as floors and walls, the speed of construction of the building is increased.
- In the steel-timber hybrid structures, because CLT vertical panels are placed as shear walls inside the openings of the steel moment frame, the ductile behavior of the steel moment frame with the Suitable and light resistance behavior of CLT panels is combined.
- Moment frame system with CLT shear wall is a hybrid system that in each direction of the structure simultaneously shows resistance to lateral forces. Thus, the lateral stiffness of such systems is the total lateral stiffness of the CLT and moment frame, which will reduce the lateral deformation of the structure. Such systems are very useful for structures that have difficulty controlling drift.
- Lightweight cross-laminated timber (CLT) shear walls supply adequate stiffness for resisting lateral loads and Controlling drift in high-rise construction.
- The important advantage in the hybridization of timber with steel, from a seismic design point of view, is the low weight of timber that results in smaller seismic forces.

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