Technical Note

Seismic Design of Cross-Laminated Timber Platform Buildings Using a Coupled Shearwall Concept

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Abstract: Cross-laminated timber (CLT) is an engineered wood material that was introduced in the last decade as a promising candidate for building wood structures higher than 10 stories. Thus far, a handful of tall residential CLT buildings have been built in low seismic regions around the world. Previous full-scale seismic shaking table tests of multistory CLT buildings revealed that this system is susceptible to overturning damage as a result of lateral seismic loads. To effectively resist overturning, a new floor connection detail was proposed to engage CLT floor panels as coupling elements for CLT shearwall stacks in the building floor plan. This approach is fundamentally different from traditional isolated shearwall stack design methods used in multistory light-framed wood buildings. The proposed method was illustrated through the seismic design of a 12-story CLT building located in Los Angeles, California, which was then subjected to the design equivalent lateral force to evaluate the conservativeness in the proposed simplified calculation. **DOI: 10.1061/(ASCE)AE.1943-5568.0000257.**

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Introduction

Because of the higher urban land price relative to suburban areas, tall buildings in the range of 8 to 20 stories may potentially be cost-effective for residential and light commercial applications in modern cities. Traditionally, these buildings were all built with steel and concrete material because traditional light-framed wood construction can become economically uncompetitive at these heights due to fire regulations. In the last decade, a new engineered wood construction material called cross-laminated timber (CLT) has emerged as an alternative construction material for tall buildings around the world. CLT is a heavy timber construction material that uses multiple layers of 1x lumber or 2x lumber that is glue-laminated together in layers that are perpendicular to each other. This new material facilitates fast panelized construction using wood material and has been used to build tall wood buildings in many cities around the world (BSLC 2014).

Most of the completed tall CLT buildings are constructed in low seismic regions. Seismic performance of CLT building systems have been investigated by a number of researchers around the world. A detailed review and summary of significant research development on the CLT lateral system before 2014 can be found in Pei et al. (2014). In addition, some recent studies (Reynolds et al. 2016) highlighted the difference between the dynamic response characteristics of CLT and traditional light-framed wood buildings. Generally, the research revealed that CLT panels are very rigid under earthquake excitation, and the building's performance was dictated mostly by connections. If CLT walls with a large length-to-height ratio (aspect ratio) were used (as the case for most existing buildings), the CLT system is less ductile than traditional light-framed wood systems because the CLT panel components are quite rigid (ductility mainly comes from connections). To achieve better ductility, the aspect ratio of the CLT panels used as shearwalls needs to be limited. Currently there is no universally accepted seismic design process for CLT building in the United States. Based on isolated CLT wall test results, Pei et al. (2013) proposed that a force-based design using the equivalent lateral force procedure (ELFP) [ASCE7-10 (ASCE 2010)] with a seismic force modification factor (R factor) close to 3.0 can produce good performance for a 6-story CLT building, given that the CLT shearwall panel aspect ratio can be limited to provide ductility. In the Italian SOFIE project (Ceccotti et al. 2013) a 7-story specimen was designed based on the Eurocode (CEN 2004) using a q factor of 1.5; it performed relatively well in strong earthquakes with mostly tie-down damage from excessive overturning. Currently, there is a comprehensive effort in the United States to develop an R factor for CLT walls though the FEMA P695 approach (Amini et al. 2014). It is likely that the results from this research will eventually led to a usable R factor for a CLT shearwall in ASCE7-10 (ASCE 2010). Once such a force-based seismic design approach is in place, it is foreseeable that engineers will start conducting CLT shearwall designs similar to that for light-frame wood shearwalls, which is to treat multistory shearwall stacks as isolated without coupling action. However, the CLT floor diaphragm has significantly more out-of-plane stiffness and strength than traditional light-framed diaphragms. This leads to an opportunity to use the CLT floor plates as coupling elements to engage the CLT shearwall stacks as a coupled system, which has been demonstrated previously for cast-in-place (Paulay and Taylor 1981) and precast concrete (Seible et al. 1991) systems.

In this study, an alternative overturning load path for CLT platform buildings is proposed to enable a force-based seismic design that is more cost-effective than the isolated wall stacks design. The discussion is limited to panelized CLT platform construction, in

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which CLT shearwalls serve as both the lateral force resistant systems (LFRS) and gravity load bearing components, and the walls are separated by floor diaphragms between stories. A good example for this construction style is the Stadthaus Building in London (Thompson 2009). The proposed approach introduced a new floor connection detail for CLT floor panels to serve as coupling elements for CLT shearwall stacks within the building, eliminating the need for an anchor tie-down system (ATS) for individual wall stacks. Instead, the ATS will be placed around the perimeter of the building floor plan to resist a global overturning from the entire building.

Assumptions for Lateral Load Resisting Mechanisms

CLT shearwalls are the main LFRS for the panelized CLT building. Because they are separated by floor diaphragms (platform framing), the lateral force within each story needs to be resisted by the shearwalls in that story and transferred down to the story below and eventually into the foundation. As mentioned previously, the CLT walls with various aspect ratios often behave differently under lateral loads. When the length-to-height ratio of the panel is large (i.e., very long walls), the lateral force will engage the wall panels primarily in shear, and the CLT wall resistance is determined by the shear strength of the connection between the wall and the floor/ceiling. Friction also plays an important role in resisting shear, but it is not very reliable during earthquakes because vertical seismic ground motion may negatively impact friction. In this study, the discussion is limited to the long CLT walls whose shear strength can be calculated conservatively as the shear strength of the connector between the wall and the floor/ceiling. Friction is simply neglected in the design. With this assumption, the wall connectors can be designed (selected) based on story shear demand calculated based on ASCE7-10 (ASCE 2010) given an R factor (which will be in place in the not so distant future).

Existing shaking table tests of CLT platform buildings revealed that the system is prone to overturning damage. In design, the global overturning demand can be estimated using an ELFP. In the design of a multistory light-framed wood building, the story shear was typically distributed to shearwall stacks within the floor plan through diaphragm action. Then tie-down elements and compression stud packs were sized to resist this overturning on stacked walls assuming no coupling effect (i.e., isolated wall stacks). This approach is conservative, but reasonable for light-framed wood construction because the out-of-plane strength/stiffness of the traditional wood floor diaphragm is typically low (especially in the direction perpendicular to the floor joists). However, for a panelized CLT building, because the floor diaphragm is made of solid CLT panels (potentially also with concrete overlay), it is possible to design the diaphragm as the coupling element for neighboring CLT wall stacks. This will enable a new load path for overturning resistance, which can be used in design to save material and labor. The following sections illustrated the feasibility of using the CLT floor as the coupling elements to achieve a more efficient global overturning resisting mechanism.

Potential Benefit of the Coupling CLT Floor

A simple two-dimensional (2D) parametric study can help illustrate the efficiency in a fully coupled design to overturning resistance against a design using isolated wall stacks. Consider a shearwall line in an N-story building with M shearwall stacks (Fig. 1). The equivalent lateral force on each story has been calculated as F_i ($i = 1$ to n). The self-weight of each story is donated as W_i.

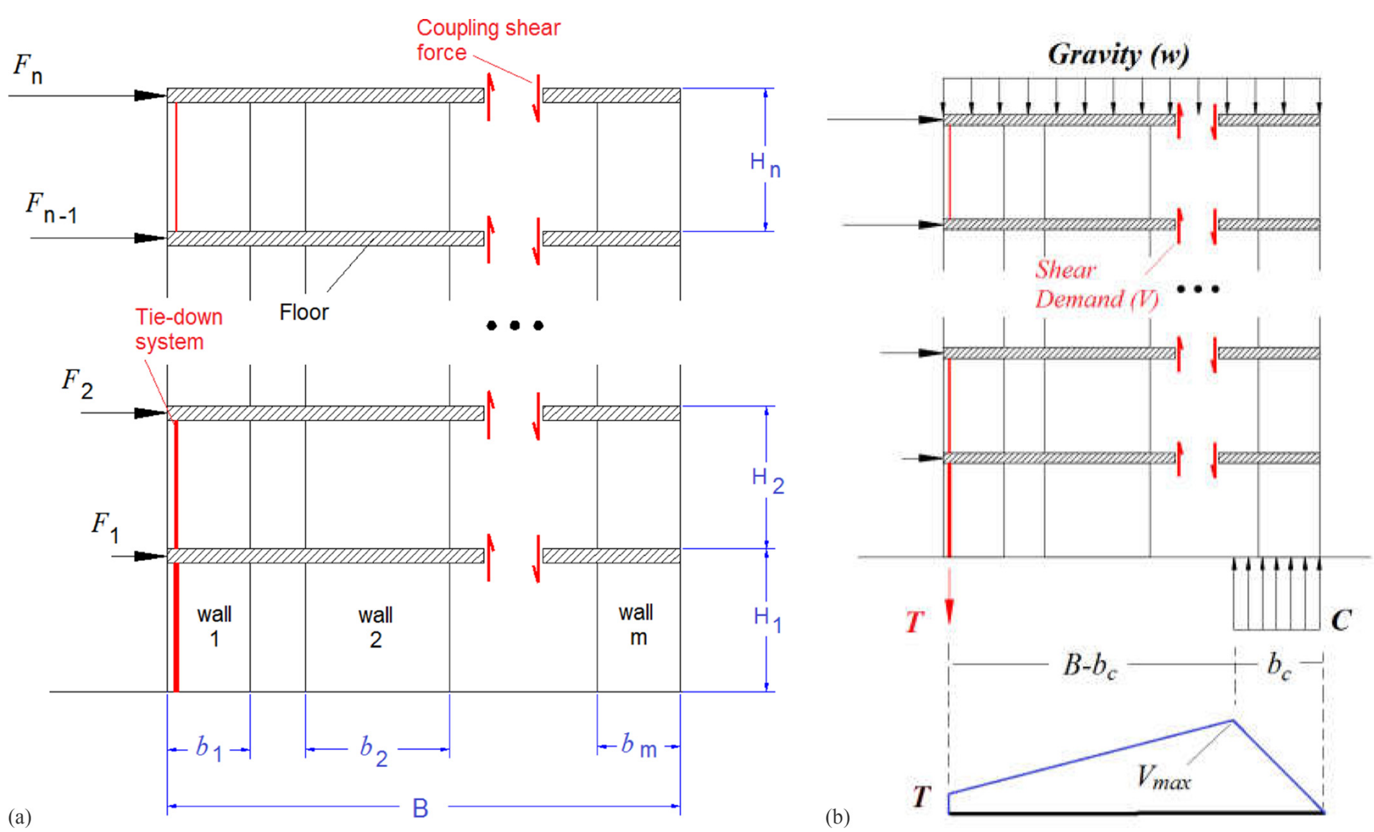

Fig. 1. Shearwall calculation diagram for (a) overturning force demands and (b) coupling shear demands

If the floor diaphragm can transfer shear in the vertical direction to allow full coupling, the most efficient way to design for overturning is to place the hold-down elements only at the ends of the building, as shown in Fig. 1(a). The coupling shear forces will be carried through by the CLT diaphragm as shown. The force demand for the tie-down element at story x can be calculated as

$$T_x = \frac{\sum_{i=x}^{n} F_i \times \left(\sum_{j=x}^{i} H_j\right) - \sum_{i=x}^{n} W_i \times {}^{B}/_{2}}{B - {}^{b_c}/_{2}} \tag{1}$$

where b_c = width of the compression zone on the other side of the building (assuming uniform compression). For a given building design, the value $P_{ax} = \sum_{i=x}^{n} F_i \times \left(\sum_{j=x}^{i} H_j\right)$ and $P_{bx} = 1/2 \sum_{i=x}^{n} W_i$ is constant when story height, seismic weight, and lateral force are determined. Thus, the overturning demand for story x can be written as

$$T_x = \frac{P_{ax}}{B - {}^{b_c}/_{2}} - P_{bx} \tag{2}$$

Alternatively, if there is no coupling effect and each shearwall stack acts independently, then each shearwall stack needs to be individually anchored down. Assuming the total shear will be distributed among the wall stacks proportional to their lengths, the force demand for the tie-down element for wall k at story x can be calculated as

$$T_{xk} = \frac{\sum_{i=x}^{n} F_{ik} \times \left(\sum_{j=x}^{i} H_j\right) - \sum_{i=x}^{n} W_{ik} \times {}^{b_k}/_{2}}{b_k} \tag{3}$$

$$F_{ik} = F_i \times \frac{b_k}{\sum_{i=1}^{m} b_i} = R \times F_i \quad \text{and} \quad W_{ik} = W_i \times \frac{b_k}{\sum_{i=1}^{m} b_i} = R \times W_i \tag{4}$$

$$T_{xk} = \frac{P_{ax}}{\sum_{i=1}^{m} b_i} - R \times P_{bx} \tag{5}$$

Note for overturning demands in high seismic regions, it is quite likely for $\sum_{i=1}^{m} b_i \leq B - b_c/2 \sum_{i=1}^{m} b_i \leq B - b_c/2$ to hold true; furthermore, one will always have $R < 1$. Comparing Eqs. (2) and (5), one can come to the conclusion that overturning force demand for a single isolated wall stack will likely be greater than that for the entire wall line with full coupling. If one ignores the contribution of gravity (the term P_{bx}), the ratio of the demands can be simplified as $B - b_c/2/\sum_{i=1}^{m} b_i$, which is likely to have a value greater than 1 in realistic building floor plans under high overturning demands (i.e., small compression zone width). Note that this is the uplift force demand ratio between a single wall stack to the entire wall line. If one assumes the cost of material and installation for the tie-down system increases proportionally with the force demand and the number of tie-downs needed, then the ratio of the total anchor system cost between isolated wall stack design to the fully coupled wall design is

$$m \times \frac{B - b_c/2}{\sum_{i=1}^{m} b_i} \tag{6}$$

For example, a shearwall line with four isolated shearwall stacks will be around four times more expensive in tie-down cost than the same wall line that can enable full coupling between the shearwall stacks.

To enable full coupling, the coupling shear force needs to be transferred by the floor diaphragm in out-of-plane shear between shearwall stacks. Consider the force equilibrium condition shown in Fig. 1(b); the total coupling shear across any *vertical cut* through the height of the building can be calculated in a way similar to the development of a beam's shear diagram based on free-body equilibrium [also shown in Fig. 1(b)]. To simplify derivation, one can assume that the building self-weight is uniformly distributed along the width of the building B, and the compression zone width is b_c, then the maximum shear demand can be calculated as

$$V_{\max} = T + \frac{B - b_c}{B} G \tag{7}$$

where G = total building weight. Thus, the range of the coupling shear demand on the floor diaphragms will range from T to $T + G$, depending on the size of the compression zone. When the overturning moment is large enough to cause *global decompression* on one side of the building as shown (with a small b_c relative to B), this coupling force demand is quite significant and requires the floor diaphragms to be strengthened, especially at splices.

Although there are potential savings when using hold-down installation, the coupled wall design is not necessary or feasible for a traditional light-framed wood design for two reasons: (1) the height limit for multistory light-framed wood construction is very limited due to fire safety concerns (less than five stories in most U.S. jurisdictions), thus, the building system will not generate a high level of overturning, resulting in a minimal cost tie-down system even if they need to be installed at every single shearwall stack, and (2) it is very difficult to satisfy the coupling demands with a light-framed wood floor diaphragm that does not have a high out-of-plane shear strength. The cost to strengthen the floor diaphragm to achieve coupled action will likely exceed the cost of simply installing an individual tie-down system for each shearwall stack. These two conditions do not apply to tall CLT platform construction, in which the height of the building can reach 10 stories (e.g., Forte Building in Melbourne, Australia), and the CLT floor system is strong in out-of-plane shear and bending, which can serve as the coupling element for stacked shearwalls if correct connection details at CLT floor splicing are properly designed.

CLT Floor Design to Enable Coupling

Depending on the location of the CLT floor splices relative to shearwall stacks, there are two scenarios for which coupling action is needed. If the floor panel is continuous between two wall stacks, the continuous floor panel can be sized to transfer shear between wall stacks based on coupling force demand calculated based on Eq. (7). Typically, the size of the CLT floor is controlled by vibration requirements and can adequately transfer the coupling demands (see the next section). However, when the floor diaphragm is not continuous (i.e., has splice) between shearwall stacks, additional connection details will be needed to transfer the out-of-plane shear. In this study a connection detail shown in Fig. 2 is proposed that includes a metal casing at the CLT panel splicing and steel dowels to transfer shear. This design is similar to the traditional shear plate connection used in heavy timber construction simply applied to shear transfer between CLT panel ends.

The size and number of the shear connections can be designed based on the shear demand calculated in Eq. (7). Keep in mind that the illustrated detail in this figure is just one of many possible connection details that can be used to transfer shear. Specifically, the connection proposed in Fig. 2 is embedded in the floor panel and can be prefabricated at the CLT floor manufacturing facility. In the next section, the seismic design process of a 12-story CLT building based on the coupled wall action was illustrated using an assumed R factor and the ELFP. This technical note focuses on presenting the design and detailing to enable floor coupling action; interested readers can obtain the detailed design calculation of the entire design process in Lenon (2015).

Design Example

Because there is currently no widely adopted procedure for the force-based seismic design of CLT buildings, a rational design process was followed in this study, as shown in Fig. 3.

The example building is a 12-story CLT platform building with a floor plan (Fig. 4) similar to the Stadthaus Building in London (Thompson 2009). The story height was 2,743 mm for all stories.

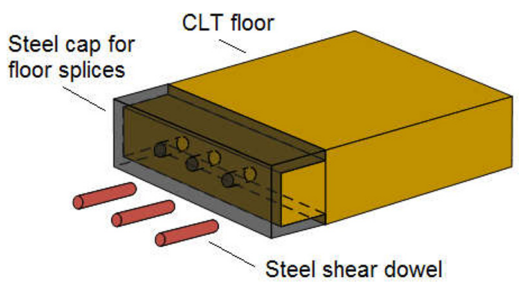

Fig. 2. CLT floor splice shear plate connection details for coupling force transfer

As shown in Fig. 4(a), the coupled shearwall design allows the ATS rods to be placed only at the perimeter of the building to resist global overturning. The building is located in Los Angeles, California. The seismic design parameters were obtained based on the ASCE7-10 hazard map, with Sa0.2 = 2.377g and Sa1 = 0.832g. The CLT material used was assumed to be Grade E1 based on APA Standard PRG 320 (material density assumed to be 419.7 kg/m^3). The dead load consists of the weight of the CLT panels as well as a 1.44-kPa supplementary dead load to account for the weight of the slab and carpet. Additionally, a dead load of 0.24 kPa is added to each wall to account for a 12.7-mm-thick layer of gypsum on each side for fire resistance. The live load is 2.39 kPa, conservatively assuming office use.

Gravity Design

Initial gravity design was conducted to determine the thickness of the floor diaphragm and the wall panels. The floor was designed as a one-way slab for bending strength based on Chapter 3 of the CLT handbook (Karacabeyli and Douglas 2013). Then a vibration check was conducted in accordance with Chapter 7, Section 3 of the CLT handbook and was found to control the design of the floor. The longest span in this example was 4.72 m based on the floor plan, which led to a 5-ply CLT panel design [thickness = 174.6 mm (6.875 in.) total] for the floor system. The shearwall compression capacity can be calculated based on Chapter 3 of the CLT handbook. However, the shearwall thickness was controlled by the compression perpendicular to grain bearing strength of the CLT floor. The initial calculation assumes uniform stress distribution among all bearing walls. The required shearwall thickness based on this compression force demand is 104.8 mm, indicating a 3-ply CLT panel for walls. From the design experience of the Stadthaus Building, 3-layer panels are able to reach a fire class of F-30, retaining structural integrity for at least 30 min in a fire. This is likely not adequate for a 12-story structure. In the Stadthaus Building, 5-layer panels were used to obtain a

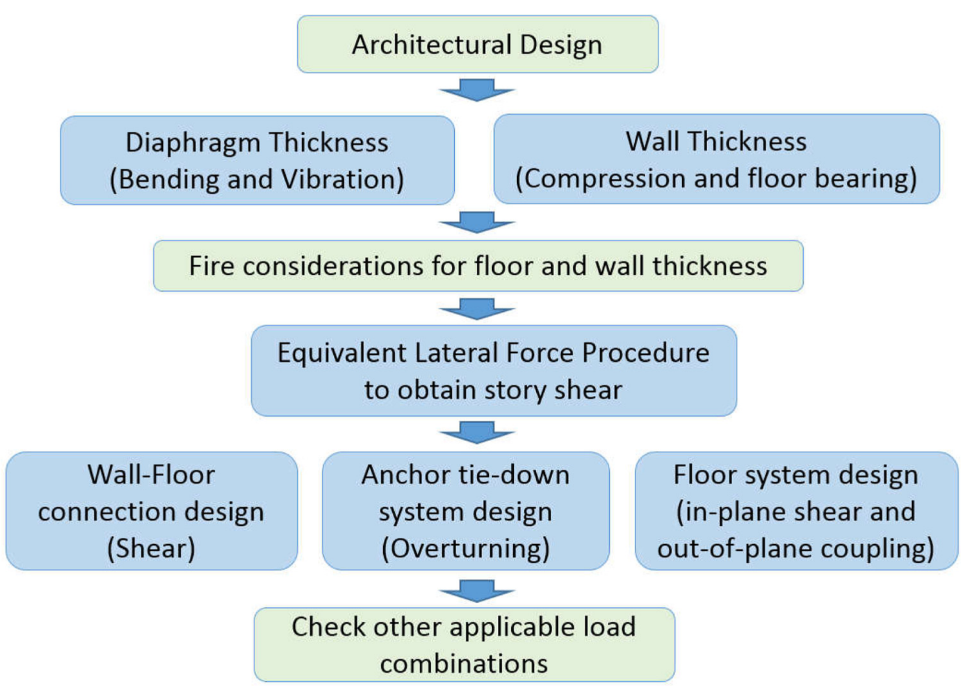

Fig. 3. Rational seismic design procedure for CLT platform building

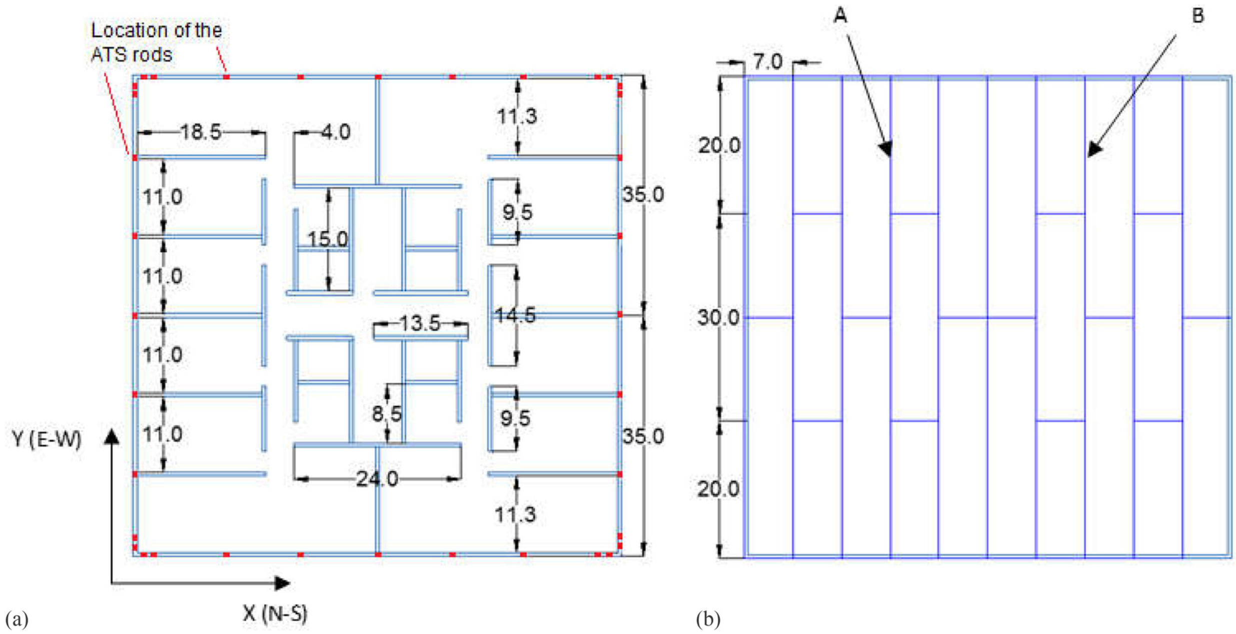

Fig. 4. Example building floor plan with (a) shearwall configuration and (b) floor diaphragm panel layout

fire protection class of F-90 for the main structural elements and F-60 for the other elements. Although fire resistance design is out of the scope of this study, to make the design example more realistic, 5-ply panels were used as the minimal thickness for shearwalls throughout this example.

Design for Shear

The seismic force demands were calculated based on ELFP outlined in ASCE7-10. The building fundamental period was estimated using the empirical formula 12.8-7 in ASCE7-10 as 0.67 s. Because the R factor for CLT construction has not been developed for ASCE7-10, an arbitrary $R = 3.0$ was used here to illustrate the design process. Table 1 illustrates the ELFP parameters and the resultant lateral forces and overturning moments for each story. The seismic weight for each story of the building was calculated based on the CLT material self-weight, supplemental dead load, and the added gypsum wall board (GWB) weight.

In this example, the brackets used for shear resistance were selected based on published design strength from the hardware manufacturer Simpson Strong-Tie (2015). Simpson ABR105 brackets selected for this example are 90.5 mm long with an allowable stress design (ASD) shear capacity of 8.36 kN (installed using SD10212 screws). In ASCE7-10 Section 12.4.2.3, a multiplier of 0.7 is used to convert the seismic loading from ASD to LRFD. Therefore, the shear capacity for ABR105 brackets for LRFD can be calculated as 8.36/0.7 = 11.96 kN. Once the total story shear demand was distributed to individual wall lines through the diaphragm, the total number of brackets (and their spacing) needed for each wall line can be determined. Note that the bracket connectors need to be installed to both the wall-to-floor and the wall-to-ceiling interface to ensure continuation of shear load path.

Currently the behavior of the CLT floor diaphragm is not well understood. There is not enough testing data to support its design as either a rigid or flexible diaphragm. In this study, both the rigid and flexible diaphragm assumptions were investigated to identify the worst force distribution for each wall line. A 5% accidental torsion for the rigid floor diaphragm case was also considered. The shear demand on each wall line is different, resulting in different bracket spacing requirements. In Table 2, only the minimal bracket spacing requirement was listed for each story for walls in both directions. Using a single spacing for the entire floor is more practical to avoid confusion during construction. Because of the limited length of this paper, detailed design calculations on diaphragm and wall shear is not fully presented here. Interested readers can review Lenon (2015) for details. For the example floor plan considered in this study, the impact of the floor diaphragm assumption on shearwall force distribution is not very significant. Note that small spacing (close to 180 mm) is required in the lower stories, mainly because the strength of the selected bracket product is limited and the story shear demand is significant. It is possible to develop custom connectors for larger shear demand to reduce the number of brackets needed.

Table 1. ELFP to Obtain Seismic Force Demand Based on an Assumed $R = 3.0$

Story	h_x (m)	w_x (kN)	C_{vx}	F_i (kN)	V_i (kN)	M_i^a (kN·m)
1	2.7	1,629	0.0016	13	7,936	180,802
2	5.5	1,629	0.0065	52	7,923	159,037
3	8.2	1,629	0.0146	116	7,871	137,517
4	11.0	1,629	0.0260	206	7,755	116,513
5	13.7	1,629	0.0406	322	7,549	96,313
6	16.5	1,629	0.0584	464	7,227	77,209
7	19.2	1,629	0.0795	631	6,764	59,502
8	21.9	1,629	0.1038	824	6,133	43,499
9	24.7	1,629	0.1314	1,043	5,309	29,508
10	27.4	1,629	0.1622	1,288	4,266	17,844
11	30.2	1,629	0.1963	1,558	2,979	8,824
12	32.9	1,248	0.1790	1,421	1,421	2,768
Total	—	19,168	—	—	—	—

[a]Note: This is the overturning moment from lateral forces alone. The actual design overturning demand for the building is lower due to the building's self-weight.

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Table 2. Bracket Spacing Required for Each Story Based on Story Shear Demand and Available Shearwall Length

	Shearwall length (mm) according to story number											
Shearwall (diaphragm type)	1	2	3	4	5	6	7	8	9	10	11	12
E-W (*X*) direction walls (flexible)	216	216	216	221	226	236	254	279	330	406	584	1,194
N-S (*Y*) direction walls (flexible)	188	188	188	191	196	206	218	254	279	356	508	1041
E-W (*X*) direction walls (rigid)	254	279	279	279	279	305	305	356	406	508	711	1,499
N-S (*Y*) direction walls (rigid)	191	191	193	196	201	208	224	254	279	356	508	1,067

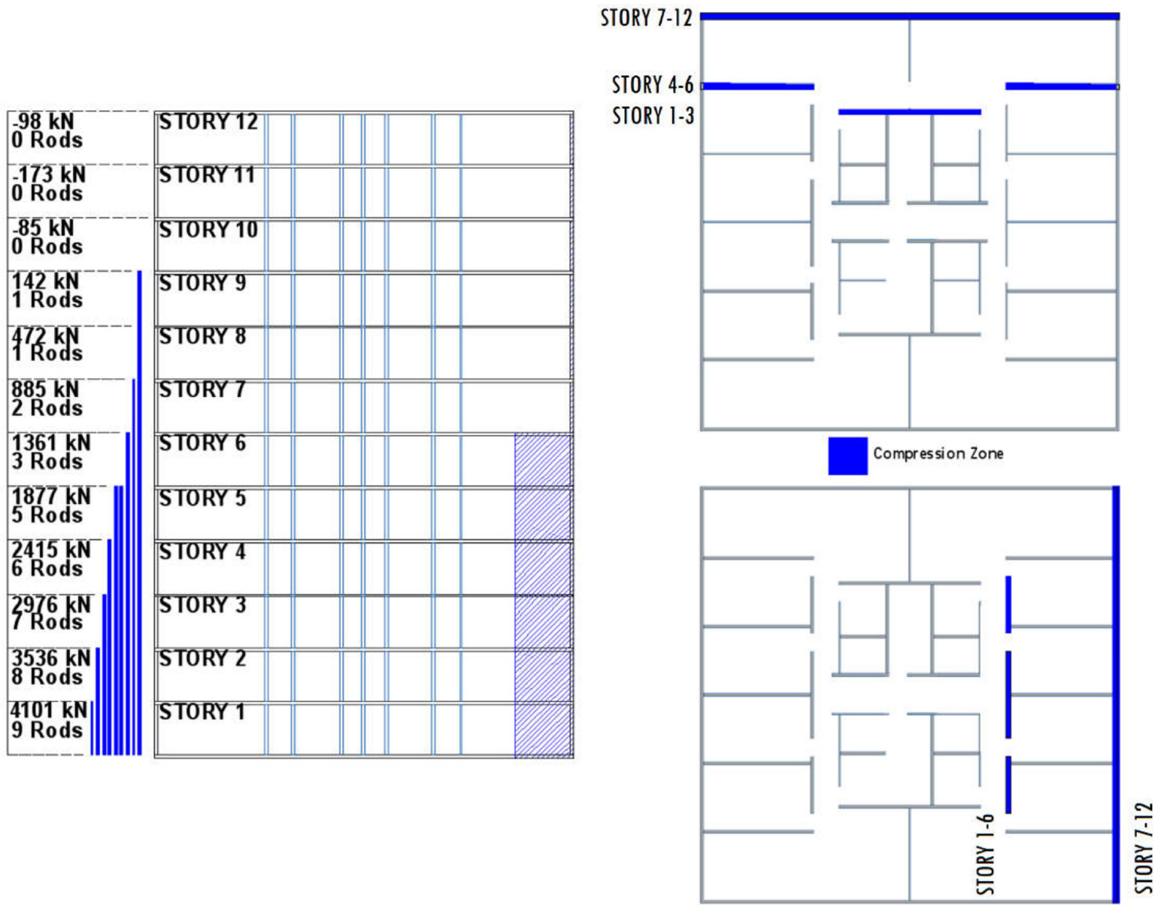

Fig. 5. Design of ATS rods and compression bearing area check for overturning resistance

Design for Overturning

As discussed earlier, the intended global overturning resistance mechanism requires the CLT floor system to fully transfer the out-of-plane shear force throughout the building. Thus, the design for overturning requires three steps: (1) select the ATS system at the perimeter of the floor plan based on the global overturning moment demands calculated using ELFP, (2) conduct a compression demand check for CLT load bearing walls on the opposite side of the floor plan based on demands and the available bearing stress under load-bearing walls (bearing perpendicular to grain at the floor will control), and (3) design floor diaphragm connections at the splices to transfer the coupling shear force.

For a simplified design to determine the amount of ATS rods and CLT wall bearing area in compression needed to resist overturning, a uniform stress distribution for CLT walls in compression was assumed in this study. The strength of the wood bearing was taken as the perpendicular to grain compression strength of the floor, assuming no additional details were used to protect and strengthen the wall-to-floor bearing surfaces. As the dead load will be beneficial to tie-down tension demand and exacerbate compression demand, the tension and compression design for the force couple are actually designed using different load combinations in ASCE7-10 (Case 5 for compression and Case 7 for tension).

Through a trial-and-error process, the needed compressive bearing area was found for different stories. The compression *zones* identified for different stories are shown in Fig. 5. It is clear that for bottom floors more ATS rods and bearing wall area are needed to resist overturning. At higher stories, the exterior wall bearing area is adequate for resisting the overturning couple. It can also be seen from this example that for a residential platform building floor plan with a large number of CLT walls, it is feasible to obtain enough bearing area for coupling action. Note this design process needs to be repeated for both directions of the building, resulting in ATS rod placement around all perimeter's of the floor plan. The final design of the number of rods is also illustrated in Fig. 5.

The design of the floor diaphragm coupling shear detail depends on the relative location of the shearwall stacks and splices. As shown in Fig. 1, when a cross cut is made through the building, the total amount of shear calculated by Eq. (7) must be transferred by either the CLT floor material (when there is no splice) or shear plate splice connector (Fig. 2). When the coupling shear is transferred directly by a continuous CLT floor panel, the out-of-plane shear capacity of the 5-ply CLT floor used in this example (807 kN·m) is adequate for this purpose. The maximum total shear demand from the overturning coupling is 2,682 kN, which requires only 3.32 m of CLT panel. Thus, the coupling requirement for the overturning in the N-S direction can be easily satisfied without special detailing [because of the staggered panel configuration, there is at least a 10 m (half of the floor width) wide solid panel available to transfer shear along any cross cut in that direction]. In the E-W direction, there will be continuous panel splices (marked as Splices A and B in Fig. 4) throughout the entire width of the floor that need to be strengthened to transfer shear. In this study, the buried shear plate detail illustrated earlier in Fig. 2 was used. The number of steel dowels needed will depend on the individual dowel shear strength and the total shear force demand on the floor splice. In this example, a steel dowel with a diameter of 34.9 mm and length of 152.4 mm is used to provide an ultimate shear strength of 142 kN (LRFD) controlled by the cross-sectional area of steel dowels. This connection strength was determined based on a range of potential failure modes of the connection including the shear failure of the steel dowels, the perpendicular to grain compression transferred into the CLT from the end caps, and the bending failure of the top and bottom of the steel end caps. Details on the calculation of this strength value are described in Lenon (2015). The shear strength of the connection can be easily adjusted by altering the diameter and spacing of the steel dowels.

Based on force equilibrium, the coupling shear demand for each story can be calculated. This shear demand will vary depending on the location of the splice and the overturning moment. As shown earlier in Fig. 1(b), the maximum shear demand will occur at the point of decompression. Using this maximum demand to size the connection for all splices will be a conservative approach for design. In this example, the total number of the dowel connections at each floor level was calculated based on maximum shear demands of the entire building divided by the number of floors. This will result in the same dowel spacing requirements at all floors, which is also convenient for prefabrication/construction. Similar to the designation of shear connectors, a constant dowel spacing was selected in this design to minimize the potential for construction errors. It was determined that a spacing of 1.1 m on center for the steel plate dowels along the continuous splices of the floor diaphragm will satisfy the shear transfer demand of the entire building. Note that the CLT floor diaphragm also needs to be designed for in-plane shear for seismic loading. The lateral design approach for the CLT diaphragm is not currently agreed on in the United States and is not the focus of this study, but there is a diaphragm design example white paper document for interested readers. This document was developed by a working group affiliated with the Oregon structural engineers association (Spickler et al. 2015).

In summary, the proposed design method was built on simplified assumptions about the load transfer mechanism of the CLT platform building system. Most important assumptions include the following: (1) story shear was completely resisted by shear connectors between CLT shearwalls and the diaphragm, (2) the overturning was resisted by the moment couple formed by the ATS rod tension and load bearing wall compression, and (3) the amount of load bearing wall needed for compression resistance can be calculated by assuming uniform stress distribution. These assumptions conform to equilibrium conditions at the building system level, but they may not accurately reflect the detailed load paths within the highly indeterminate platform building system. Thus, the conservativeness of the proposed design approach needs to be evaluated through experimental testing and numerical simulation. Because of the limited scope of this study, a finite-element model was developed to evaluate the conservativeness of the design calculation.

Numerical Evaluation

The design method described previously was derived using a simplified calculation that does not take into account the complicated load transferring mechanisms among CLT building components. The actual CLT platform building system is highly indeterminate and has redundant load paths. A nonlinear FEM model was constructed in this study to generate a more realistic connection and anchor demands in a full three-dimensional building configuration. The FEM model for a panelized CLT building has been proven viable by a handful of researchers in Europe (Sustersic et al. 2015). Although the numerical model used in this study is not as comprehensive as these existing studies, it will help to portray the behavior of the building when static equivalent lateral loads were applied. The model was used to simulate the force demands caused by the equivalent lateral loads at the ATS system and floor diaphragm connections. These simulated demands were compared with the assumed values from simplified design calculations. Although the comprehensive validation of the design methodology cannot be conducted without full-scale system-level testing, this nonlinear static analysis can help provide a quantitative estimation of the conservativeness in the proposed design assumptions.

The model was built using general finite-element software *AxisVM*, which is capable of simulating contact between CLT panels with the gap elements. The CLT panels were modeled as an elastic shell element, and all mechanical connections were modeled using linear spring elements. Although wood connections typically will exhibit nonlinear load-resistance characteristics, it is assumed in this study that the connection details will be designed conservatively with enough overstrength to remain close to linear under design level loads. Gap elements were placed between panels to simulate contact and bearing. The ATS rods were modeled using truss elements. The modeling parameters used in this study are described briefly in Table 3. A 3D rendering of the constructed model with the deformed shape (magnified for clarity) under lateral load is shown in Fig. 6. A more comprehensive description of the modeling process can be found in Lenon (2015).

The equivalent lateral forces were applied to the side of the diaphragm in each direction of the building separately, resulting in two loading scenarios. In this study the scenario with lateral forces in the N-S and E-W directions were referred to as NS and EW loading, respectively. The resultant forces in the connection springs (representing shear connectors and coupling steel plate dowels) and the ATS members were recorded from the simulation. These *simulated values* were then compared with the *design values* calculated using the simplified method described in earlier sections. A loading demand ratio of the simulated value to design value for each connector can be calculated. If the design calculation underestimates the simulated demand in a connector, then this loading demand ratio will be greater than 1.0. It is expected that this loading demand ratio will be different for each connector given their different locations. Collecting loading demand ratio of all connectors in the building, one can generate a distribution for each connection type to evaluate the percentage of connectors that falls within the conservative range (i.e., loading demand ratio below 1.0).

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Table 3. Modeling Parameters for Example CLT Building

Physical component	Modeling parameters
CLT panel	Shell elements with the properties of CLT Grade E1; Poisson's ratio is 0.29, material density is 420 kg/m^3, and G is E/16
ATS rods	Truss elements, $E = 200$ GPa, area calculated based on the actual ATS rods selected; pin connections to the floor are used to allow rotation
Shear bracket	Linear spring elements used with stiffness parameters calibrated using proprietary Simpson Strong-Tie connector test data; the contact between CLT panels and wall-to-floor panels are simulated using vertical and horizontal gap elements at the every corner of every wall panel; friction was ignored to help yield a conservative shear demand on the brackets
Floor coupling connection	Linear springs placed between the floor diaphragm panels at the splice locations; the stiffness parameters of the springs were calculated based on the steel dowel cross-section properties

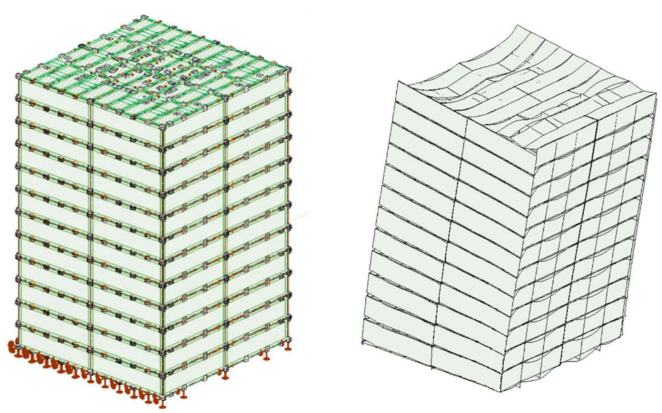

Fig. 6. FEM model of the example building structure and the simulated deformed shape under design lateral loads

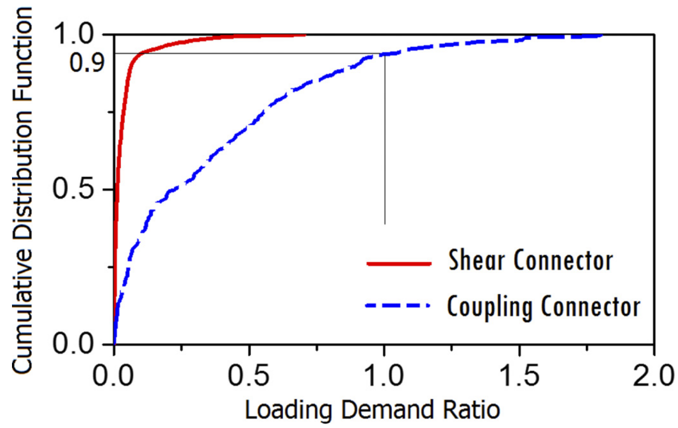

Fig. 7. Loading demand ratio for wall shear connectors and floor coupling connectors

Fig. 7 shows the cumulative distribution function generated from the loading demand ratio of the shear connectors and floor coupling steel dowel connectors in the entire building. All of the shear connector loading demand ratios are smaller than 1, which means the design for shear brackets is conservative. However, about 10% of the diaphragm coupling connectors have a loading demand ratio greater than 1. This is from the concentrated internal force produced numerically due to the use of linear springs for these connections. In realistic connections with a reasonable level of ductility, these overstressed connections can yield and redistribute the loads to other connectors.

The simulated and calculated demands for the ATS rods were also compared. In the simplified design calculation, the tie-down force was assumed to be provided only by the ATS rods on one side of the building, whereas in the realistic building system, the rods on the sides (sized based on the overturning in the other direction) and the CLT wall shear connectors will also help resist uplift forces. Although the simplified calculation assumes uniform distribution of the load amount of all ATS rods, the sharing of the loads in reality will depend on the relative stiffness of the floor system at different locations. Because the trend of ATS rod force comparison is similar for all stories, only the ground floor ATS force comparison is shown in Fig. 8.

From Fig. 8, one can see that the simplified design does not match the state of ATS load distribution in the example building model, although the total overturning resistance force predicted by the model did not exceed the total demands from the simplified calculation. Some of the simulated ATS forces greatly exceeded expected demands from the average demand in the simplified calculation. Although this concentration of the rod forced in the FEM simulation can be related to the artificially strengthened connection in a linear connection model, it also reflects to a certain degree the limitation of the design method assuming uniform rod force distribution. Without more robust model or experimental data, a safety factor of two (2.0) should be implemented for using uniformed rod force design assumption to prevent progressive rod failure.

Conclusion and Discussion

A new seismic design approach for a panelized tall CLT platform building was proposed in this study. Different from traditional stacked shearwall design approach, the proposed method used the CLT floor as the coupling element to ensure shear force transfer across the width of the building, which eliminated the need for an ATS system within the floor plan. This resulted in a concentration of these anchoring elements only at the exterior perimeter of the floor plan. This design approach simplified tie-down installation, but added additional requirements for floor panel connection detailing. Considering the prefabrication level of CLT construction, this new design is expected to help reduce construction time and efforts.

Although there is a lack of experimental validation, a FEM simulation illustrated the rationale of the simplified design approach. It is concluded that the load transferring mechanism proposed here has a good potential to be realized in panelized CLT platform buildings. However, the analysis was only static without considering the impact of dynamic responses to the demands, and the few cases of simulated demand exceeded simplified calculations. These limitations necessitate further analysis and experimental tests before this proposed design procedure can be put into practice.

It is important to keep in mind that the seismic design parameter for CLT walls and systems have not been developed for the United States. Thus, the example building designed in this study is hinged on an assumed factor of $R = 3.0$. A change in R value will alter all

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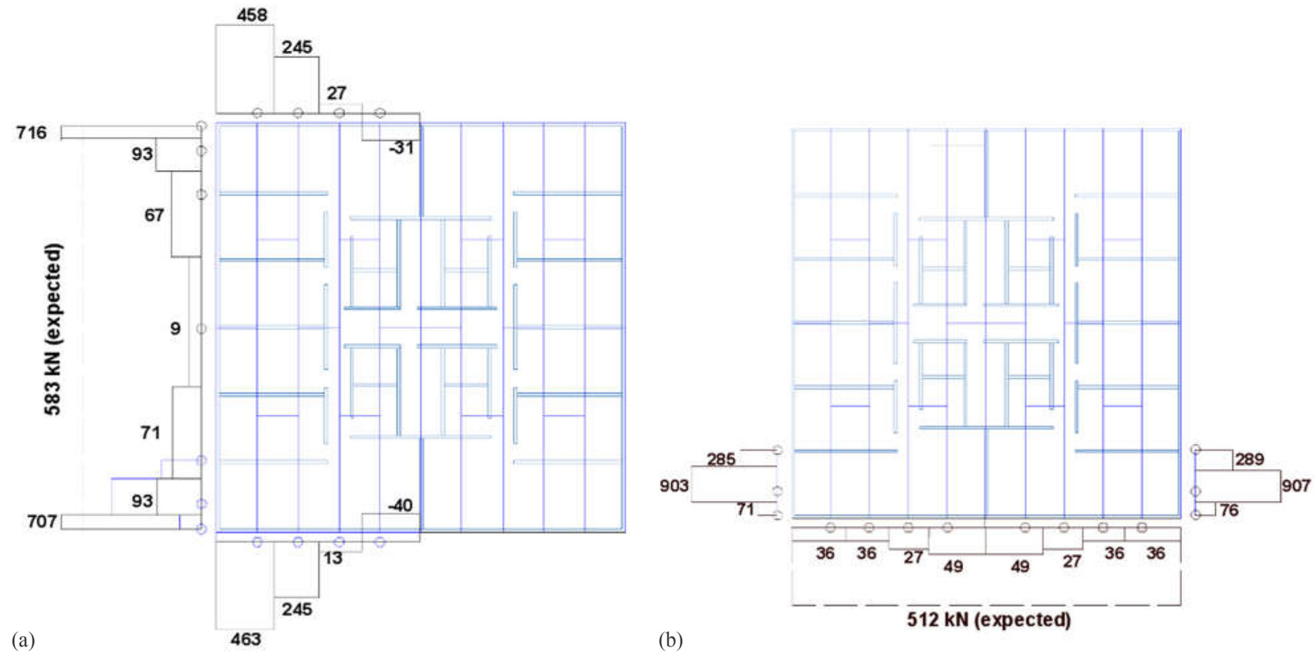

Fig. 8. Comparison of ATS forces under lateral load in (a) E-W and (b) N-S directions

related design and detailing outlined in this study, but will not affect the design process. As a result, all the numerical examples in this study are only to illustrate the design process rather than to conduct a realistic code-compliant design. The application of this design approach is only valid when an equivalent load for CLT building design can be obtained either through an *R* factor using ELFP or through other alternative approaches allowed by the code.

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