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PROPOSAL OF AN ANALYTICAL MODEL FOR A CLT WALL AND STEEL FRAME HYBRID STRUCTURE

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Abstract: *In Japan, the possibility of multi-storey timber buildings is not yet practiced at a large scale due to the complex structural design process. The Japanese structural design code for mid-to-high-rise buildings requires horizontal strength evaluation in which push-over analysis is widely applied. Because the CLT technology is rather new, the conventional model applied in the structural design practice is still not optimized and an easier-to-apply model is desired to further develop the timber construction industry. The objective of this paper is to propose an easy-to-apply analytical model of a novel CLT-Steel hybrid structure system. The model is based on the rotational spring approach to represent the rocking behaviour of the CLT wall under horizontal forces. The proposed model for the rotational spring was assumed to be a trilinear model with specific values for steel connection yield point and ultimate compression stresses for the CLT. The model was validated with a CLT shear wall experiment. The proposed model was validated by both the conventional model and the experiment.*

1. Introduction

In recent years, timber has been receiving increasing attention as an eco-friendly construction material for mid-to-high-rise buildings. In Japan, "the Law for the Promotion of Timber Use in Public Buildings" was enacted in 2010, and research and development of Cross-Laminated Timber (CLT) buildings began around 2012, Suganumata et al. (2012). Based on this research, the Explanation Manual on CLT-related Notifications (2016) was published which provided standard strength of CLT and design methods for CLT structures. For buildings up to 13m high, a simplified structural design method for timber structures is applicable, as prescribed in, as prescribed in the Japanese CLT Building Design Manual (2016). For buildings higher than 13 m, pushover frame analysis is required. This complex analysis results in higher design costs and other difficulties in structural design, posing a significant barrier to promoting mid-to-high-rise timber buildings. In the analytical model in the current design practice (hereinafter referred to as "conventional model"), the CLT wall panel is modelled as an elastic wall element and the connection systems are modelled as uniaxial tension and compression springs or shear springs. In this conventional model, a large number of springs are used, as can be seen in Figure 1. Azumi et al. (2019) proposed a model using a multi-spring model and they found that this model can reproduce the load-deformation curve of a building with good accuracy. However, as this multi-spring mode has multiple springs, it is difficult to handle with general structural analysis software, and there is room for further improvements in terms of simplicity for structural design.

An easy-to-apply analysis model is required to reduce the time and cost of the design process for mid-to-high-rise CLT buildings, and therefore the objectives of this paper are: 1) to propose a simple and practical analytical model based on a uniaxial rotational spring model that can reproduce the rocking behaviour of CLT walls, 2) to validate the proposed analytical model using the results of a full-scale CLT wall experiment, and 3) to verify the applicability and practicality of the proposed model to multi-storey buildings using a case study of a four-storey CLT hybrid building.

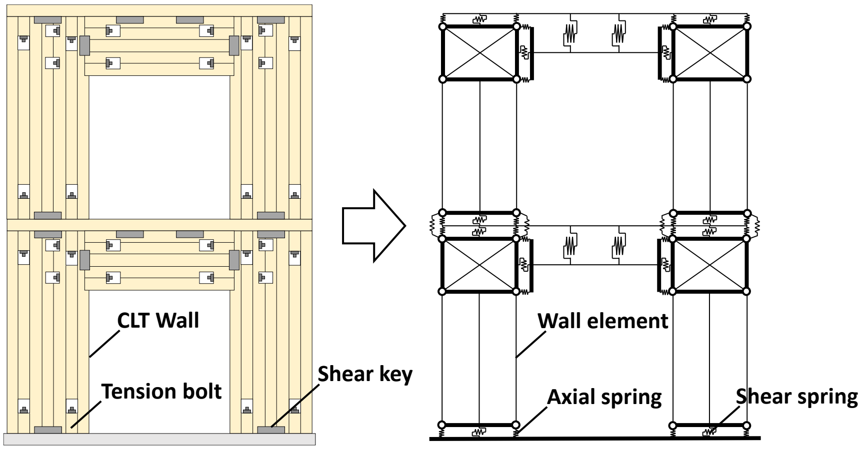


Figure 1. Analysis model of CLT panels frame as per the Japanese CLT Design Manual.

2. Proposal of CLT wall-steel frame hybrid structural system

Because of its high stiffness and strength, CLT panels have been expanding as a new building material that can be used as shear walls in mid-to-high-rise buildings. However, the strength and stiffness of the connections in CLT panels are significantly lower than the potential of CLT itself. Therefore, the structural advantages of CLT panels, in terms of stiffness and strength, are underutilised. For instance, the standard in-plane shear strength of CLT wall panels as provided in the Japanese CLT Building Design Manual (2016) is equal to 2.31 N/mm^2. However, in the current CLT buildings design practice, the average shear strength of CLT walls using conventional joints (such as tensile bolts as shown in Figure 2) is about 0.15 N/mm^2 in the case of most conservative design, as prescribed in the Explanation Manual on CLT-related Notifications (2016). In addition, the commonly used connections in Japan, such as tensile bolts and screws shear connectors, are complex, require high precision on construction sites, and a large number of connections are needed. Therefore, in order to promote CLT mid-rise timber buildings, there is potential for developing a rational CLT-steel hybrid structural system. By adopting this system for mid-rise buildings and using connections with large-diameter steel bolts, the higher strength and stiffness of CLT can be exhibited, and easier construction can be expected. In this study, a new structural system of CLT walls and steel frames was developed. This structural system is designed to have superior structural performance and is expected to offer better workability when compared to conventional joint systems. The final goal is to propose a structural analysis model of this CLT-steel hybrid structural system and to achieve the design of cost-competitive mid-rise timber buildings.

The hybrid structural system proposed in this study is shown in Figure 3. The system comprises CLT shear walls with steel frames as structural members. CLT wall panels are inserted in the steel frame to increase its stiffness and strength, and the steel beams are designed to yield first to secure sufficient ductility for the structural system. The steel frames are constructed first, and then CLT wall panels are inserted inside the frames. In this system, the steel frame is designed to support the entire gravity load, while the horizontal load is resisted by both the CLT shear walls and the steel frame. In general, the concept of this structural system is similar to steel frame structures with steel braces. Therefore, it is relatively easier for structural engineers and practitioners to design the proposed hybrid system with the conventional design process.

As shown in Figure 4, the CLT shear wall is a single-unit wall consisting of two CLT wall panels joined together with four connections at the four corners of the wall. These connections comprise high tension bolts (HTBs), that work under shear, and steel plates. The coupled CLT panels are then connected to the steel beam with the four connections using HTBs (Aljuhmani et al., 2023). The tension force acting on the connection's steel plate is resisted by the CLT panel with embedment stresses between the HTBs and the CLT. Although the ultimate failure of the connections will be a brittle shear failure, the connections are designed to provide sufficient deformation capacity before failure. Also, CLT with its cross-laminated structure can avoid total loss of load-bearing capacity after the sudden brittle shear failure of the layers parallel to the load direction. The initial stiffness and strength of this bolted joint can be calculated by the model proposed by Minegishi et al. (2022).

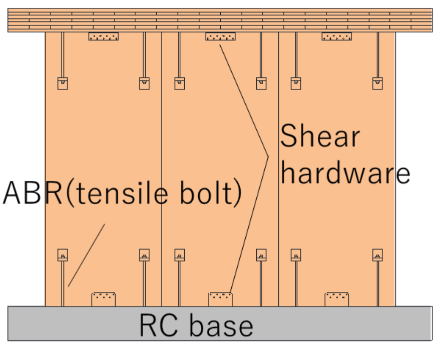


Figure 2. CLT panels conventional structural system.

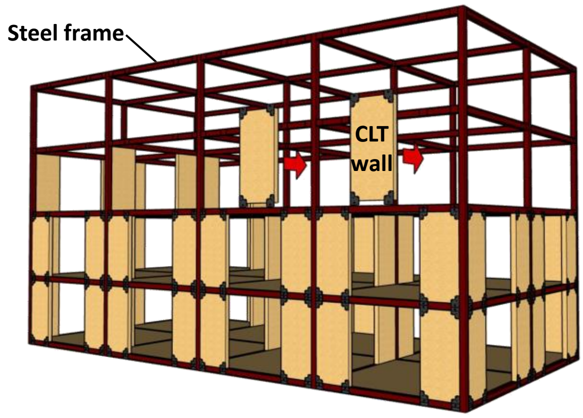


Figure 3. Basic concept of the proposed CLT-steel hybrid structure.

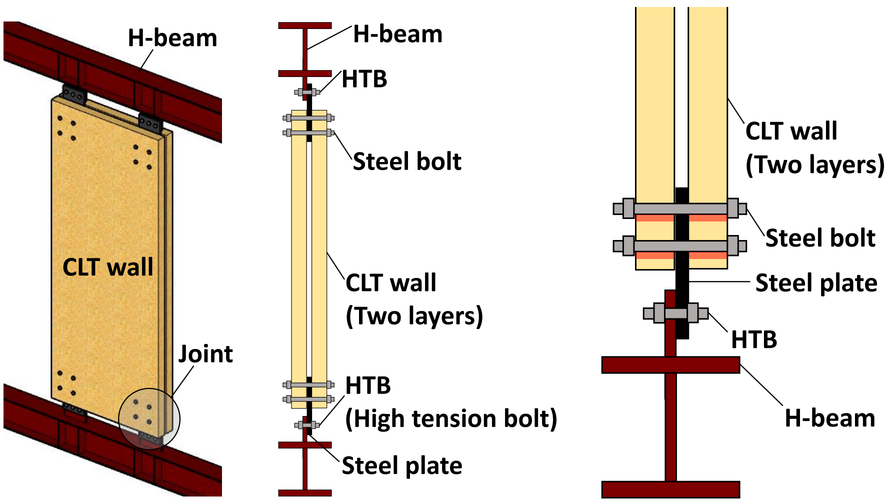


Figure 4. Details of the proposed Joint system.

3. Proposal of simplified structural analysis model for CLT wall element

In this chapter, a simple analytical model that can predict the structural performance of a single CLT wall is proposed and verified by a CLT wall test.

3.1 Proposal of analytical model

In the design practice of CLT buildings in Japan, the CLT wall panels are designed to exhibit rocking behaviour under seismic loads. Under this rocking deformation, the first failure of the CLT wall can either be the yielding of the tensile connections or compression failure of the bottom of the CLT wall panel. To model this rocking behaviour, the conventional model prescribed in the Japanese CLT Building Design Manual is used. In this study, as shown in Figure 5, the connections of each CLT wall are modelled as a single uniaxial elastic-plastic rotational spring (hereinafter referred to as "proposed model"). The rotational (rocking) behaviour of the wall is modelled with this equivalent rotational spring. The CLT wall itself is modelled as an elastic axial element.

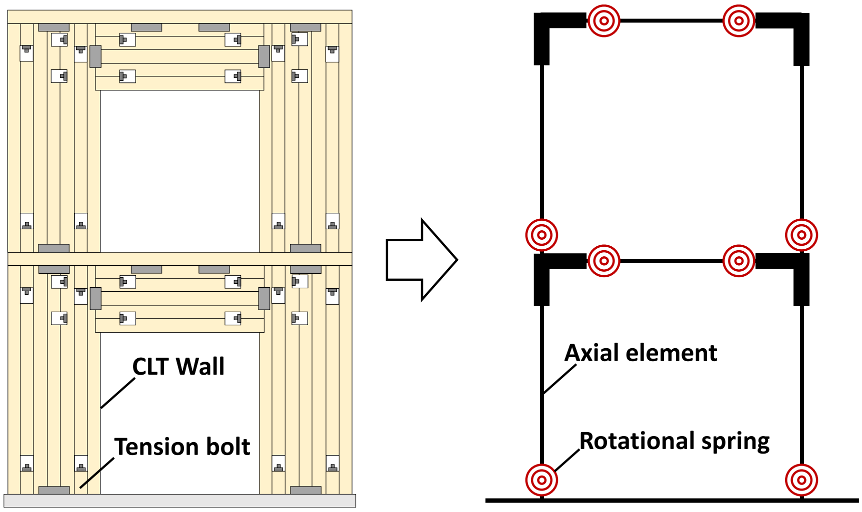


Figure 5. Proposed analysis model.

For the proposed model, shown in Figure 6, the rotational spring is modelled as a trilinear moment-rotation curve for wall-to-floor connections (top and bottom of the wall), and as a bilinear moment-rotation curve for the wall-to-foundation connections (i.e., reinforced concrete foundations). The trilinear model was adapted to account for the embedment deformation of the CLT floor under the CLT wall panel (i.e., sinking of the CLT wall panel into the CLT floor). In the proposed model, the yield bending moment (M_y) and the yield rotation angle (θ_y) at the bottom of the CLT wall are calculated by taking the stress and strain distribution in the cross-section of the wall in the allowable stress state, as shown in Figure 7 and Figure 8(a). The yield bending moment of the wall (M_y, in kN·m) is calculated, as shown in Equation (1), by taking the minimum value of the yielding moment based on tensile connection yielding (M_{ty}) and the yielding moment based on the compression side of the CLT wall reaching the allowable compression stress (M_{cy}). The allowable compression stress (σ_{cy}, in N/mm^2) is assumed to be equal to two-thirds of the maximum compression stress of CLT (σ_{cu}). M_{ty} and M_{cy} are calculated, as shown in Equations (2) and (3), by taking the moment equilibrium of the tensile force of the connection (T, in kN) and the compressive force of the wall (C, in kN), where N (kN) is the axial load applied to the wall.

$$M_y = \min\left(M_{ty}, M_{cy}\right) \tag{1}$$

$$M_{ty} \ or \ M_{cy} = T \times \left(\frac{D}{2} - d_c\right) + C \times \left(\frac{D}{2} - \frac{x_n}{3}\right) \tag{2}$$

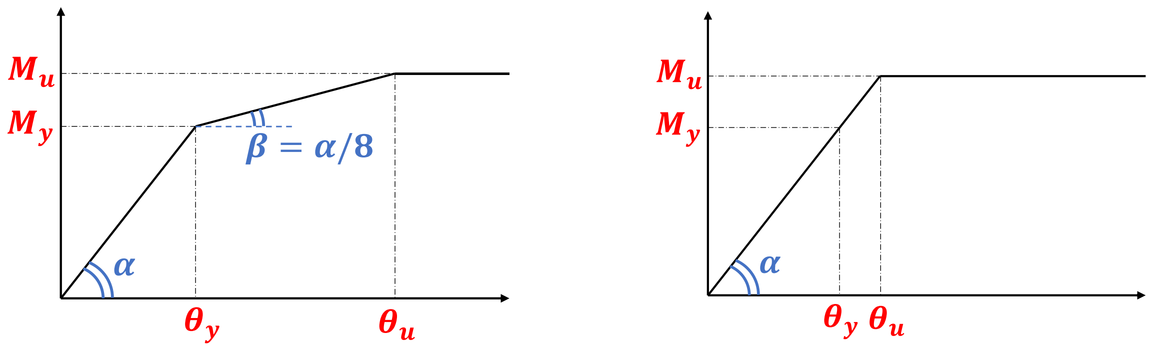
$$C = T + N \tag{3}$$

If M_{ty} is smaller than M_{cy}, the yielding tensile force of the connection (T_y) can be calculated by Equation (4) according to Minegishi et al. (2022). Compression force (C) can then be calculated based on the strain distribution shown in Figure 8(a), as shown in Equation (5). The neutral axis distance (x_n, in mm) can be then calculated by finding the solution for Equation (6), by substituting Equation (4) and (5) into Equation (3).

$$T_y = \sigma_{ty}\phi t \tag{4}$$

$$C = \frac{E_c \sigma_{ty} t (x_n)^2}{2E_t (d - x_n)} \tag{5}$$

$$\frac{E_c \sigma_{ty} t}{2E_t}(x_n)^2 + \left(\sigma_{ty}\phi t + N\right)x_n - \left(\sigma_{ty}\phi t + N\right)d = 0 \tag{6}$$



(a) Wall-to-floor connections *(b) wall-to-foundation connections*

Figure 6. Rotational spring model.

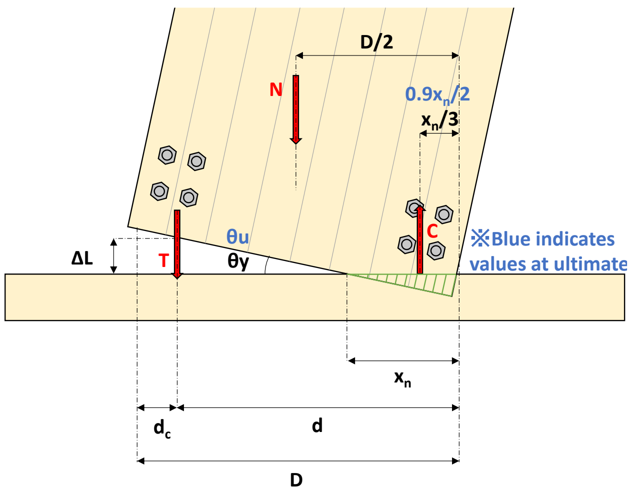
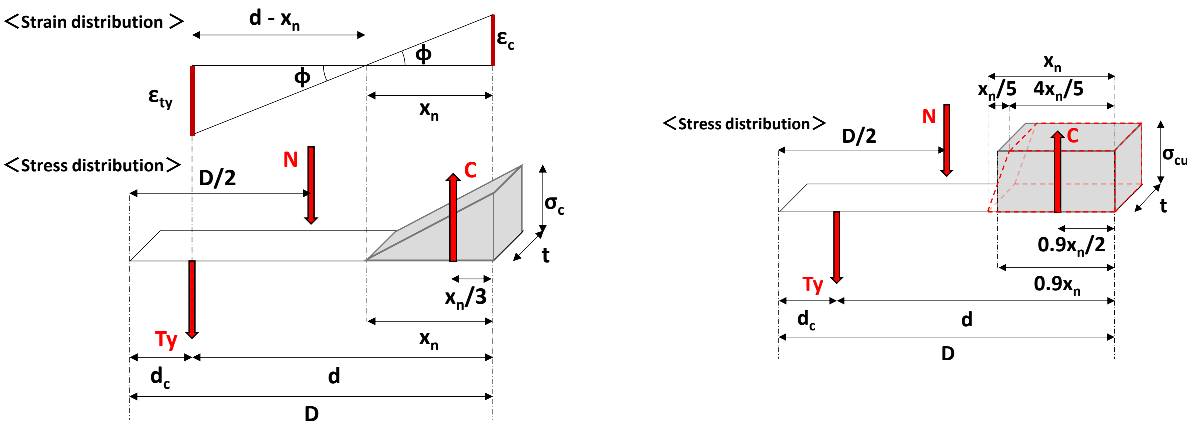


Figure 7. Stress and deformation state of the CLT wall.



(a) allowable stress state *(b) ultimate state*

Figure 8. Assumption of stress and strain distribution (at yield of tensile connection).

Here, D (mm) and d_c (mm) are the wall width and the distance from the edge of the wall on the tensile side to the tensile connection, respectively. d (mm) is the distance from the edge of the wall on the compression side to the tensile connection. σ_{ty} (N/mm^2) is the embedment strength of CLT. ϕ (mm) is the diameter of the steel bolt, and t (mm) is the thickness of the CLT panel. E_c (N/mm^2), E_t (N/mm^2) are the compression Young's modulus and the embedment Young's modulus of CLT.

If M_{cy} is smaller than M_{ty}, the same approach is used to calculate the compression force (C_y), tensile force at compression failure (T), and x_n using Equations (7)-(9).

$$C_y = \frac{\sigma_{cy} t x_n}{2} \tag{7}$$

$$T = \frac{E_t \sigma_{cy} \phi t (d - x_n)}{E_c x_n} \tag{8}$$

$$\frac{E_c \sigma_{cy} t}{2E_t}(x_n)^2 + \left(\sigma_{cy}\phi t - N\frac{E_c}{E_t}\right)x_n - \sigma_{cy}\phi t d = 0 \tag{9}$$

The yield rotation angle (θy, in rad.) at yielding moment, for both cases (M_{ty} or M_{cy}) is calculated by Equation (10) using the calculated x_n, with reference to Figure 7, where ΔL (mm) is the uplift displacement of the connection on the tensile side.

$$\theta_y = \frac{\Delta L}{d - x_n} \tag{10}$$

At the ultimate state, the plastic bending moment of the wall (Mu, in kN·m) can be calculated by Equations (11) and (12), with the same approach for the yielding moment (M_y). In these calculations, based on the calculation method provided in the Japanese CLT Design manual, it is assumed that the compression side of the wall could be replaced by an equivalent rectangular stress block, as shown in Figure 8(b). In this research, it is assumed that the width of this equivalent stress block equals $0.9x_n$, as it is presumed that compressive stress distribution of the CLT wall at its ultimate state has a trapezoidal distribution with ultimate stress developed until $0.9x_n$ from the edge of the wall, as indicated by the red dashed line in Figure 8.b.

$$M_u = \min(M_{tu}, M_{cu}) \tag{11}$$

$$M_{tu} \; or \; M_{cu} = T \times \left(\frac{D}{2} - d_c\right) + C \times \left(\frac{D}{2} - \frac{0.9x_n}{2}\right) \tag{12}$$

Here, M_{tu} represents the ultimate bending of the wall when its compression side of the wall reaches the ultimate stress after the yielding of the tensile connection. M_{cu} represents the ultimate bending of the wall when its compression side of the wall reaches the ultimate stress, and the tensile connection does not yield.

The rotation angle at the ultimate state (θ_u, in rad.) is then calculated using Equation (13), based on the assumption that the secondary stiffness is equal to 1/8 of the initial stiffness for wall-to-floor connections (Equation (13.a)), based on AIJ Manual for Timber Structural Joints (2017). As explained before, this secondary stiffness is caused by the embedment deformation of the CLT floor under the CLT wall. In the case of wall-to-foundation connections, the compression stiffness of the CLT wall was taken as prescribed in the Japanese CLT Design Manual. The Japanese CLT Design Manual assumes that the compressive secondary stiffness of the wall panel is negligible and is equal to zero (Equation (13.b)).

$$\theta_u = \theta_y + \frac{(M_u - M_y)}{M_y / 8\theta_y} \tag{13.a}$$

$$\theta_u = \frac{M_u}{M_y}\theta_y \tag{13.b}$$

3.2 Validation of proposed analysis model

3.2.1 Methodology

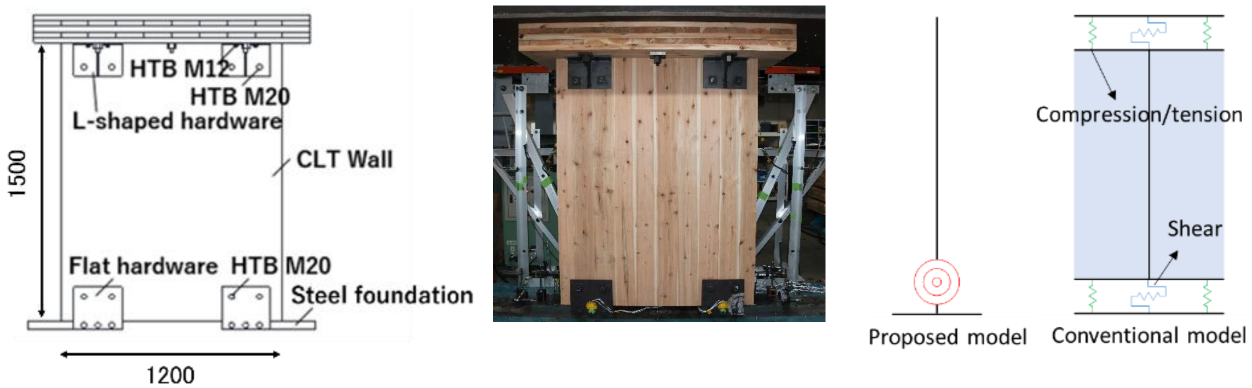
A half-scale CLT shear wall test was used to validate the proposed model. The shear wall test was modelled using the proposed analysis model as well as the conventional analysis model. Results from both analysis models were then compared with the experimental results.

The illustration of the CLT shear wall test specimen is shown in Figure 9(a) and (b). The tested specimen was a 150-mm thick single CLT panel (Japanese cedar, *Cryptomeria japonica*) made of Mx-60-5-5 (5-ply 5-layer) grade and configuration, based on classification provided in the Japanese CLT Design Manual. A CLT wall panel with 1500 mm in height and 1200 mm in width was used. A steel foundation was used under the tested CLT wall. On top of the wall, a CLT floor was attached, and the lateral load was applied to this floor. For the steel connections used at the four corners of the wall, steel plate-bolt connections, as detailed in section 2, were used. In this wall test experiment, a double-plate configuration of the connections was used (i.e., two steel plates were attached to both sides of the single CLT wall panel and then joined together using HTBs.) Both proposed and conventional analytical models of the shear wall test experiment are shown in Figure 9(c).

The CLT in the analysis model is modelled using the material property values from the material tests, as shown in Table 1. In the conventional analytical model, shear springs and the tension properties of the axial springs were modelled as proposed by Minegishi et al. (2022). Compression properties of the axial springs were modelled based on the model provided by the Japanese CLT Design Manual. In the proposed model, the CLT shear wall was modelled as a single rotational spring. Characteristics of the rotational spring were calculated as explained in section 3.1. A pushover analysis was performed for both models.

3.2.2 Application of analytical model to experimental results

Results of this analysis were compared to experimental results as shown in Figure 10. In terms of initial stiffness, both conventional and proposed models overestimated the stiffness values, compared to the experimental results. This can be attributed to the effect of the slip of the HTBs used in the steel connections which is not included in the analytical models. Regarding the maximum strength, the value derived from the proposed model was about 10% lower than that from the conventional model. This variation is thought to be due to the distance between centers of tensile and compressive stresses in the bottom of the wall, as explained in section 3.1. In general, it was found that the proposed model can evaluate the strength of the wall without exceeding the experimental maximum strength. As a result, it was found that the proposed model overestimates the stiffness; however, further improvements in accuracy can be expected by incorporating the effect of the slip of connections in the model. The proposed model can evaluate the strength with generally good accuracy and conservative values, for the scope of a single CLT wall.



(a) Illustration (b) Photograph (c) proposed and conventional analysis model

Figure 9. Test specimen in experiment and analytical model (dimensions in mm).

Table 1. Results of the CLT material tests used in the analysis model.

	Compression		Shear		Embedment	
Strength (MPa)	Major	Minor	Major	Minor	Major	Minor
	19.6	15.1	5.0	4.7	23.8	19.2
	Young (compression) [E]		Shear [G]		Embedment stiffness	
Modulus (MPa)	Major	Minor	Major	Minor	Major	Minor
	4817	3459	698	649	1858	1515

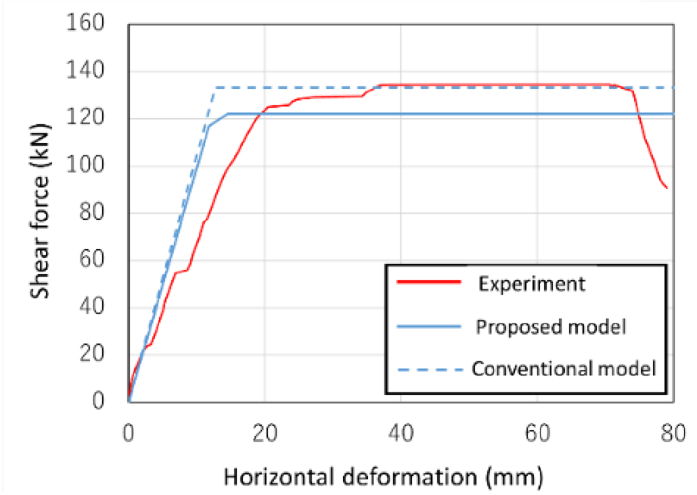


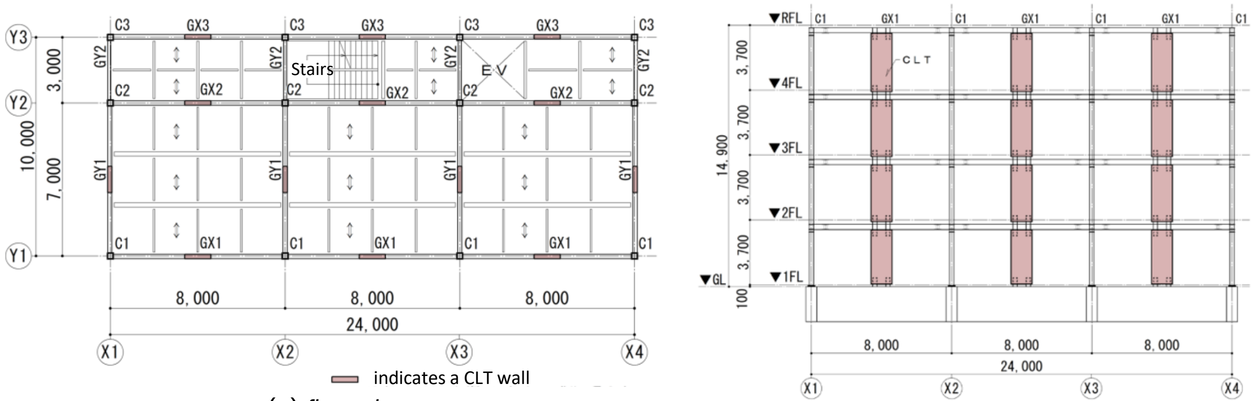
Figure 10. Comparison of experimental and analytical results.

4. Case study of proposed hybrid structural system

A case study was conducted to confirm seismic performance of the proposed hybrid structural system and its advantage of reduction of amount to use of steel volume comparing with conventional steel frame strucutures.

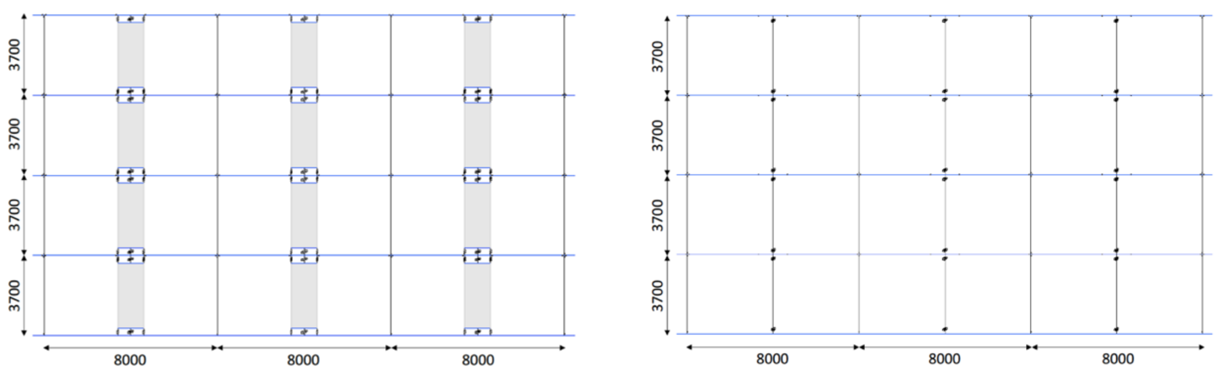
4.1. Seismic performance analysis of a prototype 4-storey hybrid building

A prototype for an application example of CLT wall-steel frame hybrid four-storey building was modelled using the proposed analytical model as well as the conventional model. The layout of the studied building is shown in Figure 11. The analytical study of this building was conducted on a single frame of the building using both analytical models (proposed and conventional) shown in Figure 12.



(a) *floor plan* (b) *elevation*

Figure 11. Layout of the studied building.



(a) *floor plan* (b) *elevation*

Figure 12. Analytical model of the studied building.

The dimensions of each component are shown in Table 2. The CLT panels were made from Japanese cedar with Mx-60-3-3 (laminar grade 60, 3-ply, 3-layer) grade and composition, The material property values from material tests are shown in Table 3. The CLT panels system proposed in Chapter 2 were applied tothe prototype building. For the steel frame, H-section steel beams (H-294x200x8x12 in the Japanese Industrial Standards (JIS G 3192)) and square hole section columns (SHS-250-250-16) were used. Yield strength and plastic section modulus for steel columns and beams are listed in Table 4.

Table 2. Structure types considered in this study.

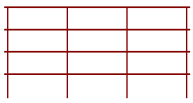
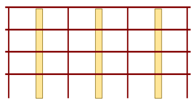
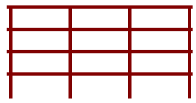
	1. Steel frame (small sections)	**2. CLT wall-steel frame hybrid structure**	**3. Steel frame (large sections)**
Structural system			
Element details	Columns：SHS-250x250x16 (STKR400) Beams：H-294x200x8x12 (SN400)	Columns: SHS-250x250x16 (STKR400) Beams: H-294x200x8x12 (SN400) CLT: Mx60-3-3(Cedar) L:1200xH:3000xt:90 CLT wall connections：HTBs Φ30x4 (Strength classification=10.9)	Columns: SHS-400x400x19 (STKR400) Beams: H-450x200x9x14 (SN400)
Component selection methodology	･The cross-sectional area of the columns and beams were chosen to satisfy the long-term forces design. ･The structure was not design for horizontal forces.	･The cross-section area of the steel columns and beams is similar to 1. steel frame (small sections). ･The required CLT connections to resist the horizontal forces were selected with the virtual work method (i.e., plastic analysis of the frame). ･Pushover analysis was conducted to confirm the short-term allowable deformation limit.	･Cross-sectional area of the steel columns and beams were chosen to satisfy the design horizontal strength using the virtual work method. ･Pushover analysis was conducted to confirm the short-term allowable deformation limit.

Table 3. CLT material property values from material tests.

	Compression		Shear		Embedment	
Strength (MPa)	Major	Minor	Major	Minor	Major	Minor
	23.7	10.9	3.8	2.6	21.6	15.5
	Young (compression) [E]		Shear [G]		Embedment stiffness	
Modulus (Mpa)	Major	Minor	Major	Minor	Major	Minor
	5927	2322	299.5	199	1767	1189

Table 4. Material properties of steel columns and beams.

	Element sections	**Yield strength σ_y (MPa)**	**Plastic section modulus Z_p (cm^3)**
Steel column	SHS-250x250x16	235	1240
Steel beam	H-294x200x8x12	235	842

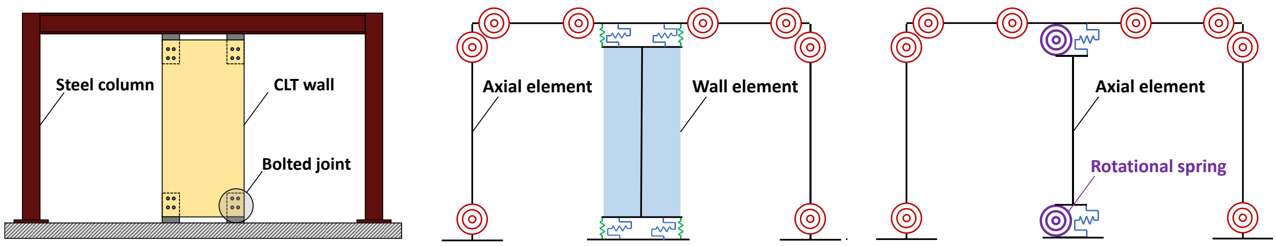
An illustration of the analysis model for CLT wall-infilled steel frame is shown in Figure 13. The basic concept of the analysis model for the CLT wall is as described in section 3.1; however, the method of calculating the connection rotational spring has been modified due to the change in the support conditions at the bottom of the CLT wall (i.e., the bottom of the CLT wall does not resist compression stresses). Rotational springs of the connections were modelled as a bilinear moment-rotation curve, as shown in Figure 14(a). The moment-rotation curve is calculated using Equations (14) and (15) by assuming the stress-deformation state of the CLT wall as shown in Figure 15. As shown in Figure 15, the steel connection resists a combination of tensile force (T) or compressive force (C), and shear force (Q). The tensile and compressive force (T, C) is represented by the rotational spring, while the shear force (Q) is represented by a conventional shear spring, as shown in Figure 13(c). The steel frame was modelled as an elastic linear element with perfectly elasto-plastic sprong at both ends, as shown in Figure 14(b). The yielding moment capacity of the steel beam and clumn was calculated using Equation (16).

$$M_{clt} = T \times j \tag{14}$$

$$\theta_{clt} = \delta/(D/2 - d_c) \tag{15}$$

$$M_y = \sigma_y \times Z_p \tag{16}$$

Here, T (kN) is the tensile force of the connection, and j (mm) is the distance between tension (T) and compression (C) at the bottom of the CLT wall. δ (mm) represents rocking deformation of the CLT wall, with reference to Figure 15(b). D (mm) and d_c (mm) represent the wall width and the distance from the edge of the wall to the tensile connection, respectively. σ_y (MPa) is yielding stress of the steel, and Z_p is the plastic section modulus.

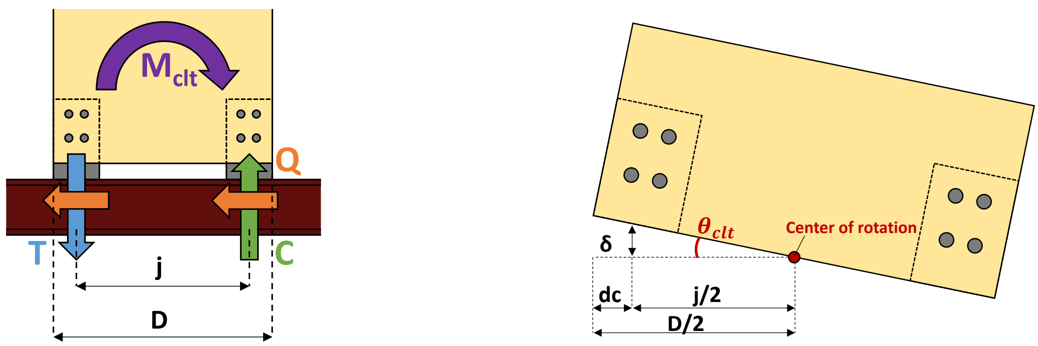


(a) actual structure (b) conventional model (c) proposed model

Figure 13. Illustration of structural analysis models.

(a) proposed model (b) CLT wall spring

Figure 14. Rotational spring model.



(a) load-transfer mechanism at wall bottom (b) deformation state

Figure 15. Assumed load-transfer mechanism and the deformation state of the wall panel.

A Pushover analysis was performed on the two analytical models, and the resulting shear force-storey drift angle relation is shown in Figure 16. The proposed model slightly overestimates the initial stiffness and strength up to 1.5% storey drift angle but was generally found to be able to evaluate the stiffness and strength with acceptable accuracy.

4.2. Comparison of seismic performance and steel volume with conventional steel frame structures

A comparative case study on two different steel frame buildings was conducted and compared to the proposed hybrid structural system. One of these steel frame buildings, hereinafter referred to as steel frame (small sections), consists of the same steel beams and columns of that for the proposed hybrid system but without the CLT wall panels. In the other steel building, the beams and columns cross sections were designed to satisfy the base shear strength required in the Japanese seismic code by Minegishi et al. (2022), hereinafter referred to as steel frame (large sections). A summary of the three studied buildings is presented in Table 2. A Pushover analysis was conducted on the three buildings using the proposed analytical model. As a result of this analysis (Figure 17), the steel frame (small sections) building did not satisfy the base shear coefficient of C_0=0.2 in the short-term design requirements (at 1/200 rad of story drift), as required by the seismic design method prescribed in the AIJ Allowable Stress Design Standard for Steel Structures (2019). The CLT wall-steel frame hybrid structure satisfied both the deformation limit for short-term design requirements and the required ultimate horizontal load-bearing capacity. Table 5 shows a comparison between the total amount of steel used in the hybrid structure and the steel frame structure (large sections) that satisfied required seismic structural performance. It was found that using CLT wall panels inside the steel frames can reduce the amount of steel by 40%. Thus, the proposed hybrid structural system can be efficiently designed to leverage the structural advantages of CLT panels (i.e., the high stiffness and strength) while reducing the cross-sections of steel frame members.

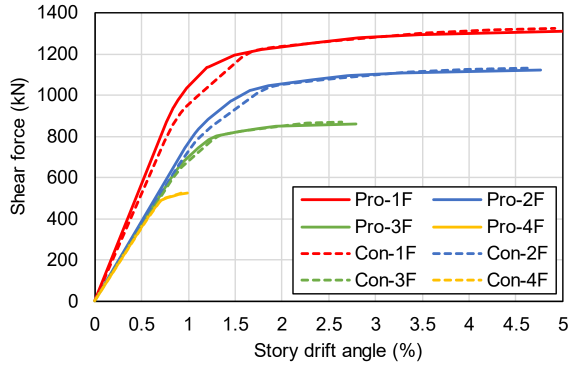


Figure 16. Comparison of analysis results for the conventional model and proposed model. Where "Pro" and "Con" refer to proposed model and conventional model, respectively. 1F-4F refers to the floor number.

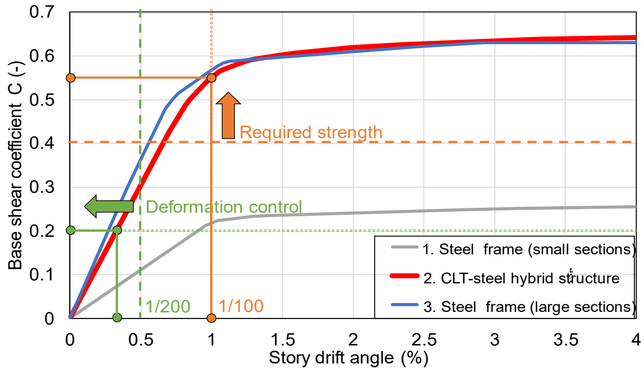
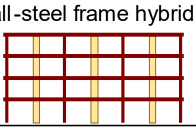
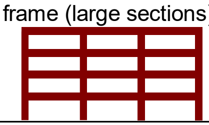


Figure 17. Comparison of analysis results for the different structure types.

Table 5. Comparison of steel quantities for CLT hybrid structure and steel frame structure.

Structural system	Steel element sections	Total weight of elements[t]	
		Steel	CLT
2. CLT wall-steel frame hybrid structure 	SHS-250x250x16 H-294x200x8x12	10.9	2.5
3. Steel frame (large sections) 	SHS-400x400x19 H-450x200x9x14	18.1	0

5. Conclusions

In this study, an easy-to-apply analytical model for a CLT wall-steel frame hybrid structural system has been proposed. The proposed model was compared to the conventional analytical model and validated by a half-scale single CLT wall test. Furthermore, several case studies were conducted on a 4-storey hybrid structure system using the proposed model. The conclusions of this study are summarised as follows:

1. A simple and practical analytical model for a CLT wall-steel frame hybrid structural system was proposed. In this model, the CLT wall panel is modelled as a single uniaxial rotational spring that represents the rocking deformation of the CLT wall under horizontal loads.

2. Based on the experimental results of the CLT shear wall test, the proposed model and the conventional model overestimated initial stiffness since the slip of the HTBs is not incorporated in the analytical model. Both models gave good estimations for ultimate strength.

3. Based on the results of the structural analysis of a 4-storey hybrid building, both proposed and the conventional models produced similar results, indicating the applicability of the proposed simplified model to multi-storey hybrid buildings. Based on the comparative case study, it was found that the hybrid structural system can reduce the amount of steel by 40% compared to the steel frame structure. Using the proposed hybrid system, the high strength and stiffness of the CLT walls can be utilized and required cross-sections of the steel members can be reduced.

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