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ANALYTICAL EVALUATION OF PLATFORM-TYPE MULTI-STOEY CLT SHEAR WALLS

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ABSTRACT: Many preceding studies have focused on investigating the seismic behaviour of platform-type single-storey cross-laminated timber (CLT) shear walls, encompassing both experimental investigations and analytical developments. However, no experimental validation has been conducted for analytical expressions related to lateral resistance, deflection of multi-storey shear walls, and the associated capacity-based design procedure. In this study, three examples of two-storey experimental tests are compared with analytical solutions. Subsequently two six-storey shear wall case studies were analytically examined to study the contribution of various components, i.e. the different connections, to lateral behaviour. A recently proposed capacity-based design procedure was used to verify the yielding sequence and ensure protected elements remain elastic. The significant influence of aspect ratio of shear wall panels was also observed. With an appropriate connection design, rocking deformation was dominant, while bending deformation accounted for less than 30% of the total deflection, meeting the requirements of the Canadian Wood Engineering Design Standard.

KEYWORDS: Cross laminated timber; multi-panel, shear wall, multi-storey, platform-type

1 – INTRODUCTION

Platform-type buildings encompassing CLT walls resisting gravity and lateral loads and floor between each level, have gained attention worldwide and design approaches have been incorporated into structural design standards, e.g. CSA O86-24 [1]. Extensive research has been undertaken on the lateral behaviour of multi-storey CLT shear walls, highlighting the rigid-like behaviour of CLT panels and significant contribution of connections in providing lateral resistance and ductility [2-5]. Key connections, as illustrated in Figure 1 (b) include Vertical Joints (VJ), which attach adjacent CLT panels and resist shear force between them; Hold-downs (HD) at the wall ends, which resist uplift forces; HD or Tension Straps (TS) attaching the bottom of each level to the top of the bottom level to resist uplift; and Angle Brackets (AB) and Floor connections (FC) along the wall length at the bottom and top of the shear wall at each level, which resist both uplift and sliding shear forces.

Several analytical approaches have been developed for multi-storey CLT shear walls. Casagrande et al. [6] and Masroor et al. [7] developed analytical approaches for single storey walls where the contribution of AB in resisting uplift was neglected and incorporated, respectively. Masroor et al. [8] extended the analytical methods introduced in [7] by establishing a Capacity-based Design (CD), in which system ductility is ensured through a yield hierarchy among ductile connections while capacity protecting brittle elements. Casagrande et al. [9] introduced an elastic analytical approach determining the lateral deflection of multi-storey buildings consisting of multi-panel CLT shear walls. A matrix format to calculate the cumulative lateral deflection of a CLT lateral load resistant system (LLRS).

2 – RESEACH GAP AND OBJECTIVES

The aforementioned analytical procedures have led to recent updates in CSA O86-24 [1]. While these approaches have been investigated and validated against single storey experimental tests [10], their predictive accuracy for multi-storey CLT shear walls remained unexplored. This study aims to validate the analytical approach [9] for calculating elastic lateral displacement by comparing its predictions against two-storey, two-panel wall experimental tests [11]. To extend the analysis to multi-storey, multi-panel CLT shear walls, two cases of six-storey walls with varying connection properties and panel aspect ratios were considered. Additionally, the study evaluates the contribution of TS to uniform inter-storey drift (ISD) and assesses compliance with the requirement in [1], which limits wall panel bending deformation to 30% of the total lateral deflection.

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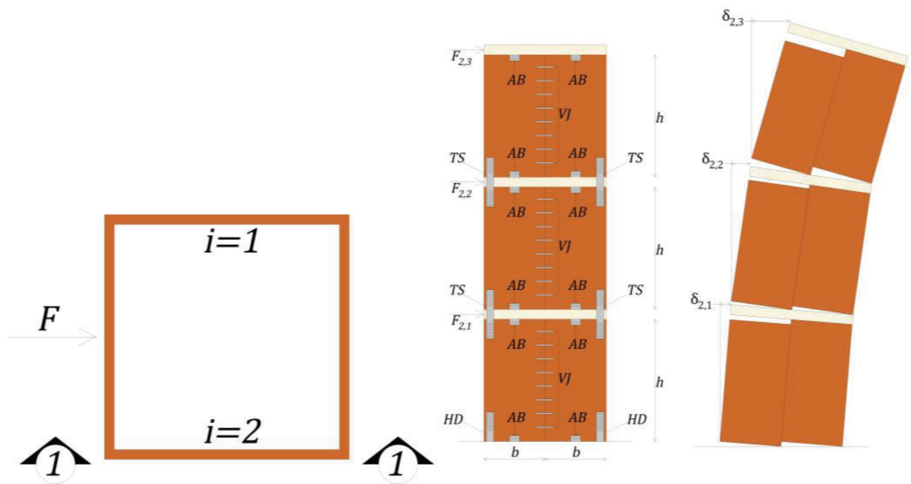

Figure 1. Multi-panel CLT shear wall (a) plan view (b) elevation view 1 (b) lateral deformed shape

3 – ANALYTICAL EXPRESSIONS

In the LLRS system shown in Figure 1, the total lateral deflection of the i^{th} shear wall at the j^{th} level can be determined by summing the ISD from the first level up to the j^{th} level. The ISD for the at the j^{th} level for the i^{th} shear wall, $\delta_{i,j}$, is presented in Eq. (1), consisting of the lateral deflection due to the rocking of CLT wall panels ($\delta_{r,i,j}$), sliding at the based of each wall in the connections between the shear wall and the floor below ($\delta_{s,i,j}$), bending and shear deformations of CLT wall panels ($\delta_{b,i,j}$ and $\delta_{d,i,j}$, respectively), cumulative of rotation of the walls at lower levels ($\delta_{c,i,j}$) and the sliding at the top of the shear wall in the connections between the floor and the shear wall below ($\delta_{f,i,j}$).

$$\delta_{i,j} = \delta_{r,i,j} + \delta_{s,i,j} + \delta_{b,i,j} + \delta_{d,i,j} + \delta_{c,i,j} + \delta_{f,i,j} \quad (1)$$

The analytical expressions for each lateral displacement contribution are presented in Eqs. (2)-(7). In these equations, b and h_j refer to the width and height of each wall panel, respectively, while $h_{int,\mathrm{j}}$ represents the j^{th} level inter-storey height. m stands for the number of CLT shear wall panels. $M_{f,i,j}$, $M_{f,top,i,j}$ represent the bending moments at the base and top of the shear wall at the j^{th} level, respectively, while $V_{f,i,j}$ and $q_{i,j}$ correspond to the base shear force and the cumulative gravity load at the j^{th} level, respectively. $k_{AB,x,i,j}$ and $n_{AB,i,j}$ are the elastic stiffness of AB in the horizontal direction and the number of AB in each wall panel. $k_{FC,x,i,j}$ and $n_{FC,i,j}$ are also the elastic stiffness of connections resisting horizontal sliding at the top of the shear wall, where it is attached to the floor and their corresponding number in each wall panel. $t_{CLT,i,j}$ is the total thickness of the CLT wall panel.

It is important to note that the rocking deformation expression presented here applies to the case of coupled-panel behaviour, where each panel has a separate center of rotation while maintaining contact with the ground.

$$\delta_{r,i,j} = \max\left[\left(\frac{M_{f,i,j}}{b^2} - \frac{q_{i,j} \cdot m}{2}\right) \cdot \frac{h_{int,\mathrm{j}}}{k'_{r,i,j}} ; 0\right] \quad (2)$$

$$\delta_{s,i,j} = \frac{V_{f,i,j}}{k_{AB,x,i,j} \cdot m \cdot n_{AB,i,j}} \quad (3)$$

$$\delta_{b,i,j} = \frac{V_{f,i,j} \cdot {h_j}^3}{3 \cdot EI_{eff,i,j}} + \frac{M_{f,top,i,j} \cdot {h_j}^2}{2 \cdot EI_{eff,i,j}} \quad (4)$$

$$\delta_{d,i,j} = \frac{V_{f,i,j} \cdot h_j}{G_{eff,i,j} \cdot t_{CLT,i,j} \cdot m \cdot b} \quad (5)$$

$$\delta_{c,i,j} = \theta_{i,j-1} \cdot h_{int,\mathrm{j}} \quad (6)$$

$$\delta_{f,i,j} = \frac{V_{f,i,j}}{k_{FC,x,i,j} \cdot m \cdot n_{FC,i,j}} \quad (7)$$

$k'_{r,i,j}$ can be determined using Eq. (8), where $k_{HD,ij}$ and $k_{VJ,i,j}$ are the uplift and shear stiffnesses of HD and VJ, respectively, and $n_{VJ,i,j}$ the number of VJ between panels.

$$k'_{r,i,j} = k_{HD,ij} + (m-1) \cdot n_{VJ,i,j} \cdot k_{VJ,i,j} \quad (8)$$

$EI_{eff,i,j}$ and $G_{eff,i,j}$ can be determined using Eqs. (9) and (10), respectively. $t_{CLT,0,i,j}$ and $t_{CLT,90,i,j}$ are the panel's vertical and horizontal plies thickness, while $E_{0,i,j}$ and $E_{90,i,j}$ are the moduli of elasticity for these plies. $G_{0,i,j}$ is the vertical shear modulus of the wall panels. $t_{mean,i,j}$ and $w_{i,j}$ denote the average thickness of wall panel and the board width of a ply. $\alpha_{i,j}$ for a 5-ply CLT wall panels can be obtained using Eq. (11).

$$EI_{eff,i,j} = \frac{m \cdot b^3}{12} \cdot \left(t_{CLT,90,i,j} \cdot E_{0,i,j} + t_{CLT,0,i,j} \cdot E_{90,i,j}\right) \quad (9)$$

$$G_{eff,i,j} = \frac{G_{0,i,j}}{1 + 6 \cdot \alpha_{i,j} \cdot \left(\frac{t_{mean,i,j}}{w_{i,j}}\right)^2} \quad (10)$$

$$\alpha_{i,j} = 0.425 \cdot \left(\frac{t_{mean,i,j}}{w_{i,j}}\right)^{-0.79} \quad (11)$$

$\delta_{c,i,j}$ is equal to zero at the first level (*j=0*) since no rotation is assumed at its base (foundation). $\theta_{i,j-1}$ can be calculated using Eq. (12).

$$\theta_{i,j-1} = \theta_{i,j-2} + \frac{\delta_{B,i,j-1} + \delta_{R,i.j-1}}{h_{j-1}} \quad (12)$$

For more details on deflection equations and expressions related to kinematic mode requirements and CD, readers can refer to [12].

4 – MODEL VALIDATION

The analytical approach was validated by calculating lateral deflection of three cases of two-storey two-panel walls, as shown in Figure 2, with different connections.

Figure 2. Two-storey experimental test

The mechanical properties of connections used at each level (*j=1,2*) for each case, #H2a, #H3a, and #H3b are summarized in Table 1. The number of AB and FC at both levels ($n_{AB,i,j}$ and $n_{FC,i,j}$) were equal to one, while $n_{VJ,1,1}$ and $n_{VJ,1,2}$ were sixteen (16) and eleven (11), respectively.

Table 1: Mechanical properties of connections in validation cases

Parameter	Stiffness (kN/mm)		
	#H2a	#H3a	#H3b
$k_{HD,1,1}$	24	24	24
$k_{HD,1,2}$	13.7	10.3	10.3
$k_{VJ,1,1} \cdot n_{VJ,1,1}$	16	16	16
$k_{VJ,1,2} \cdot n_{VJ,1,2}$	11	11	11
$k_{AB,1,1}$	40	40	40
$k_{AB,1,2}$	30	40	20
$k_{FC,1,1}$	40	40	30
$k_{FC,1,1}$	30	20	20

The mechanical properties of CLT wall panels, essential for evaluating the panels' bending and shear deformations, were determined for V2 grade [13]. The modulus of elasticity values, $E_{0,i,j}$ and $E_{90,i,j}$ were set at 9500 MPa, 300 MPa, respectively, while the shear modulus, $G_{0,i,j}$, was 594 MPa. The thickness parameters considered included $t_{CLT,i,j} = 139$ mm, $t_{CLT,90,i,j} = 34$ mm, $t_{CLT,0,i,j} = 105$ mm, and $t_{mean,i,j} = 27.8$ mm, with a panel width ($w_{i,j}$) of 150 mm. The total distributed gravity loads applied at each level, $q_{i,j}$, amounted 6.65 and 13.3 kN/m, for the first and second level, accounting for the cumulative dead load weight and the self-weight of CLT panels, including both floors and shear walls. Please refer to [11] and [12] for a full description of the tests, as well as additional details on connection configurations and the types of fasteners used in each connection.

Figure 3 illustrates the base shear-ISD relationship at each level for all three cases, along with an inclined red line representing the elastic deflection obtained from the presented analytical approach. The results demonstrate that the analytical method predicts the elastic deflection of CLT shear walls with reasonable accuracy across all three cases at both levels.

Table 2 summarizes the deviation in elastic stiffness between analytical and experimental results. At the first level, the difference ranges from 2% to 15%, while at the second level, the deviation is higher, ranging from 12% to 25%. The close match at the first level can be attributed to the highly accurate mechanical properties used for HD. In contrast, the greater deviation at the second level may result from assumptions made in defining the HD in the second level, represented as the TS, and from neglecting the contribution of TS in resisting uplift between the floor and the shear wall below.

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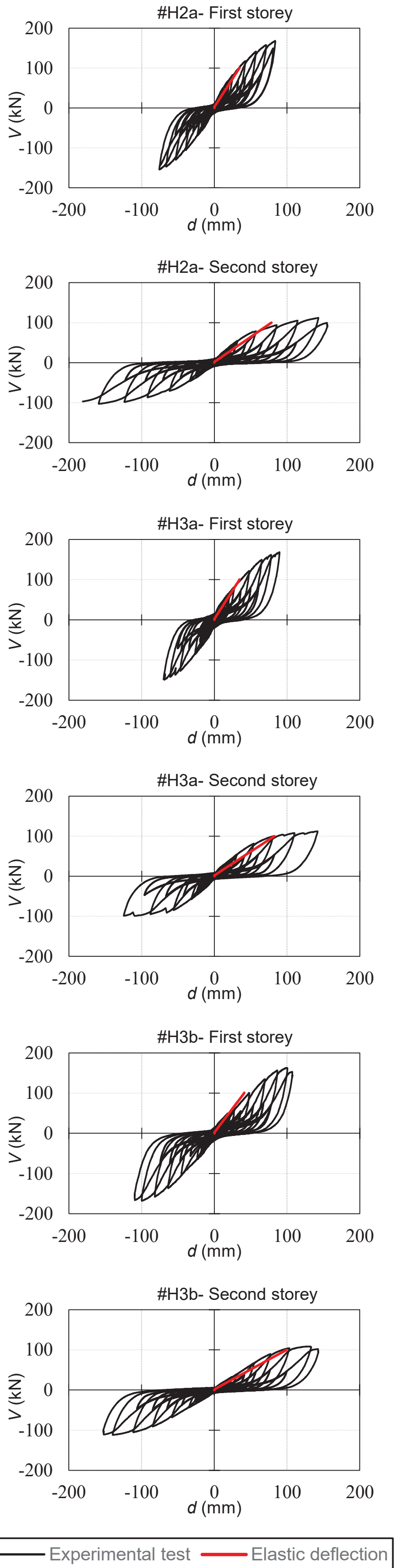

Figure 3. base shear-ISD curves along with the analytical elastic deflection

Table 2: Difference between the elastic stiffness in analytical and experimental results

	Level	Stiffness (kN/mm)		
		#H2a	#H3a	#H3b
Deviation [%]	1st	2%	4%	15%
	2nd	25%	25%	12%

5 – SIX-STOREY CASE STUDY

This section presents two case studies on six-storey platform-type CLT shear walls, examining the effects of varying the elastic stiffnesses of connections on the ISD of mid-rise platform-type CLT shear walls. Figure 4 shows a six-storey building with two shear walls, each resisting the total base shear. Both walls have the same total length, 2400 mm, but one consists of three panels ($b=800$ mm), and the other of two panels ($b=1200$ mm), allowing for an investigation of different wall panel aspect ratio, 3:1 and 2:1, respectively.

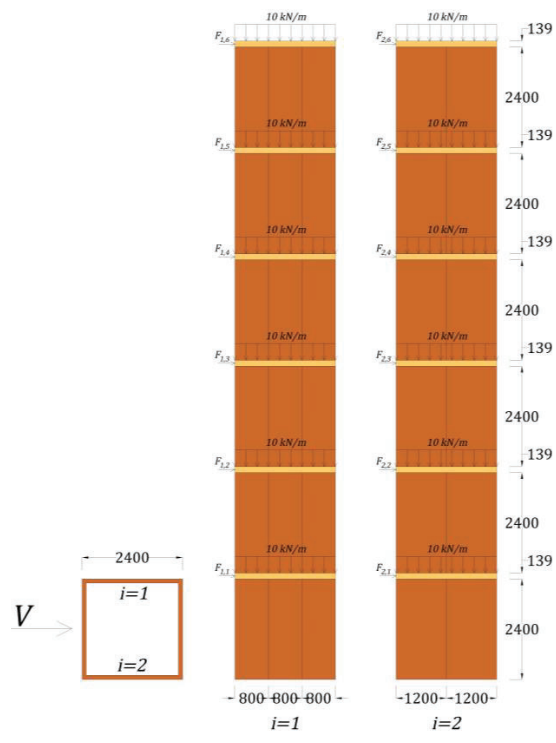

Figure 4. Six-storey CLT building

The lateral loads were estimated in accordance with NBCC [14]. The fundamental period, T_a, was determined using the expression $0.05 \cdot (h_n)^{3/4}$, where h_n is the total height of the building (15.2 m) to be 0.39 sec. The building was assumed to be in New Westminster, BC, Canada, with a site classification C. The higher mode factor, m_v, obtained based on T_a, was 1.0, the importance factor, I_E, for the building was 1.0, the ductility and overstrength-related force reduction factors, R_d and R_0, were taken as 2.0 and 1.5, and the total weight was assumed as 450 kN. The resulting spectral acceleration, $S(T_a)$, was determined as 0.92.

Utilizing these values, the lateral loads acting at each level, F_j, and subsequently the lateral loads acting on each shear wall j, $F_{i,j}$, as listed in Table 3 were computed assuming that each shear wall resists half of the seismic loads $F_j/2$. The table also presents the base shear, $V_{f,i,j}$ and bending moment, $M_{f,i,j}$ at the base of each level, calculated based on the lateral loads. Additionally, the gravity loads, $q_{i,j}$, represent the summation of gravity loads at the j^{th} level and all levels above.

Table 3: lateral loads, base shear, and bending moments at each level

	Unit	Level					
		#1	#2	#3	#4	#5	#6
$F_{i,j}$	[kN]	3.3	6.6	9.9	13.2	16.4	19.7
$V_{f,i,j}$	[kN]	69.1	65.8	59.2	49.3	36.1	19.7
$M_{f,i,j}$	[kNmm]	750	575	409	260	137	47
$q_{i,j}$	[kN/m]	60	50	40	30	20	10

Table 4 presents the elastic stiffness of each connection, along with the number of connections at each level for both building cases. The same AB and VJ were used for both cases, while the HD varied at different levels to capture the impact of TS on ISD. The properties of the CLT wall panels were the same as those used in the validation examples with two-storey walls.

Figure 5 illustrates the ISD at each level when the 6th-level lateral load, $F_{i,6}$, ranges from 2 kN to 10 kN to ensure elastic behavior. **Error! Reference source not found.** also provides the ISD values when $F_{i,6}$ was set to 10 kN, the ISD corresponding to the yielding of VJ, a level-by-level comparison of the ISD between the two walls with three and two panels (i=1,2), $\xi_{ISD,S}$, and the ISD deviation between adjacent levels, $\xi_{ISD,AS}$.

The red points on the graphs represent the ISD corresponding to the yielding of VJ at different levels, as defined in [11]. No yielding was observed at the 5th and 6th levels, except in case #2-2, where VJ yielded at an ISD of 1.53%. It is important to note that other connections, including HD, AB, and FC, did not yield throughout the assessments.

In the context of aspect ratio, the ISD range for the larger aspect ratio (3:1 for *i=1*) was greater, ranging from 0.5% to 1.49%, compared to 0.24% to 0.89% for the aspect ratio of 2:1 for *i* = 2. $\xi_{ISD,S}$ indicated a deviation range of 81% to 108%, highlighting the significant impact of aspect ratio on ISD.

Table 4: Elastic stiffness and number of connections in case study

Properties	Building	Level					
		#1	#2	#3	#4	#5	#6
$k_{HD,i,j}$	#1	50	40	30	20	15	8
	#2	50	30	20	15	10	5
$k_{VJ,i,j}$	#1 #2	1.2	1.2	1.2	1.2	1.2	1.2
$k_{AB,i,j}$	#1 #2	60	60	60	60	60	60
$k_{FC,i,j}$	#1 #2	40	40	40	40	40	40
$n_{VJ,i,j}$	#1 #2	18	15	12	10	8	5
$n_{AB,i,j}$	#1 #2	2	2	2	1	1	1
$n_{FC,i,j}$	#1 #2	2	2	2	1	1	1

The contribution of TS to ISD was assessed through $\xi_{ISD,AS}$), where building #1, with a stiffer TS, exhibited slightly lower ISD values, ranging from 86% to 91%, compared to building #2, which ranged from 100% to 102%. The deviation between the 2nd and 5th levels was significantly lower, with a maximum of 27%, due to the uniform stiffness transition between TS. In contrast, greater differences were observed between the 5th and 6th levels (122%–145%), resulting from the lower bending moment at the base of the 6th level, while other levels exhibited a more consistent bending moment transition.

Table 6 presents the contribution of rocking and wall panels bending deformations to the total ISD. The contribution of bending was limited to 4.8%-18.6% of the total ISD, satisfying the 30% requirement specified by [1]. Rocking behaviour, however, accounted for 23.5% to 77.1% of the total ISD, which is desirable in seismic regions due to the engagement of connections. Even in cases with lower rocking deformation, the rotation which accumulates along the building height and consist of both rocking and bending deformations, was the primary contribution to the total ISD.

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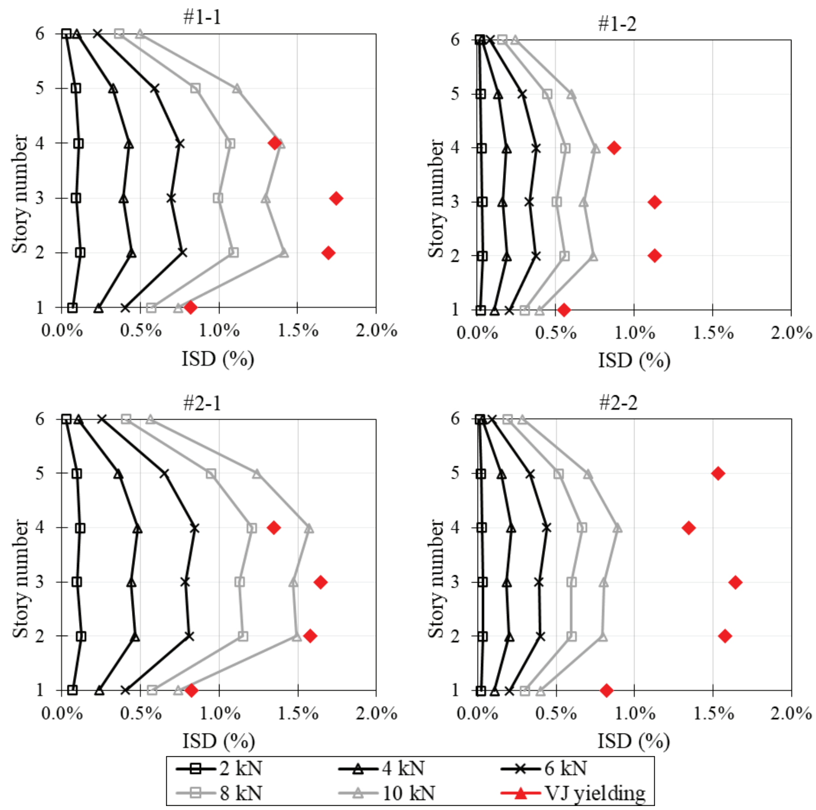

Figure 5: ISD for different top lateral load at each storey [12]

Table 5: Analytical results

Properties	Building	Level					
		#1	#2	#3	#4	#5	#6
IDS at $F_{1,6}$=10 kN Unit: [%]	#1-1	0.74%	1.41%	1.30%	1.39%	1.11%	0.50%
	#1-2	0.40%	0.74%	0.68%	0.75%	0.60%	0.24%
	#2-1	0.74%	1.49%	1.47%	1.57%	1.24%	0.56%
	#2-2	0.40%	0.80%	0.81%	0.89%	0.70%	0.29%
ISD at VJ yielding Unit: [%]	#1-1	0.82%	1.70%	1.75%	1.35%	-	-
	#1-2	0.56%	1.13%	1.13%	0.87%	-	-
	#2-1	0.82%	1.58%	1.64%	1.35%	-	-
	#2-2	0.56%	1.02%	0.91%	0.69%	1.53%	-
$\xi_{ISD,S}$ Unit: [kN]	#1-1	85%	91%	91%	85%	85%	108%
	#1-2						
	#2-1	85%	86%	81%	76%	77%	93%
	#2-2						
$\xi_{ISD,AS}$ Unit: [kN]	#1-1	91%	9%	7%	25%	124%	-
	#1-2	86%	9%	11%	25%	148%	-
	#2-1	102%	1%	7%	27%	122%	-
	#2-2	100%	1%	11%	27%	145%	-

Table 6: Contribution of rocking and bending to total ISD

Walls	Deflection	Level					
		#1	#2	#3	#4	#5	#6
#1-1	Rocking	77.1%	37.5%	36.9%	47.1%	23.5%	27.0%
	Bending	18.6%	9.2%	8.9%	7.0%	6.3%	7.1%
#1-2	Rocking	76.7%	38.2%	37.7%	48.9%	23.6%	27.5%
	Bending	14.7%	7.4%	7.2%	5.4%	4.8%	5.6%
#2-1	Rocking	77.1%	40.8%	39.1%	46.9%	24.7%	28.0%
	Bending	18.6%	8.7%	7.9%	6.2%	5.6%	6.2%
#2-2	Rocking	76.7%	42.8%	40.6%	48.6%	25.3%	29.0%
	Bending	14.7%	6.9%	6.0%	4.6%	4.1%	4.6%

6 – CONCLUSION

This study presents an analytical approach for estimating the lateral deflection of multi-storey platform-type CLT shear walls subjected to lateral loads. The findings offer valuable insights into connection design and strategies for achieving uniform ISD across building heights. The key conclusions are:

- The predicted elastic lateral displacements aligned well with observed values at both storeys. Deviations in elastic stiffness ranged from 2% to 15% for the first storey and 12% to 25% for the second storey. The larger variations in the second storey were primarily due to assumptions regarding TS properties. Additionally, comparisons of elastic stiffness between storeys highlighted the critical role of TS in maintaining uniform ISD along the building height.

- The panel segment aspect ratio had a significant influence on ISD, with deviations ranging from 81% to 108% when comparing 3:1 and 2:1 aspect ratios. Furthermore, maintaining a uniform TS stiffness across storeys helped reduce ISD variations between levels.

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