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# INVESTIGATING THE EFFECTS OF THE INTERACTIONS BETWEEN FLOOR DIAPHRAGMS AND SEGMENTED CROSS-LAMINATED TIMBER SHEAR WALLS

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**ABSTRACT:** This paper presents a study on the effects of the interactions between floor diaphragms and segmented cross-laminated timber shear walls. An analytical model of a beam on a bed of elastic springs is used to describe such an interaction for the case of two- and three-panel segmented shear walls. The analytical model is, firstly, validated by comparing its results with those of a numerical model developed in SAP2000 and, secondly, used for the definition of an equivalent spring which simulates the effects of the floor-wall interaction in a simpler yet reliable way. In the study, it is shown that the equivalent spring can be used to take into account the effects of the floor-wall interactions for the cases of two- and three-panel segmented shear walls. The internal actions of the floor element as well as the increase of rocking stiffness due to such interaction are discussed in the final part of the study.

**KEYWORDS:** Cross-Laminated Timber, Multi-Panel Shear Walls, Floor-Wall Interaction.

## 1 INTRODUCTION

Cross-laminated timber (CLT) is a structural typology which offers many advantages in terms of sustainability, speed of construction and structural performance. Given their excellent in- and out-of-plane structural properties, CLT panels are used in these structures as both vertical and horizontal load-bearing elements. The assemblage of the CLT panels is realized by means of mechanical joints, which permit the creation of structures with excellent lateral performance.

This assemblage represents a key aspect of the structural behaviour of CLT buildings, which, *inter alia*, entails structural interactions, such as those between perpendicular walls or vertical walls and horizontal floors. These structural interactions are usually not directly considered when the lateral performances of CLT buildings are assessed, as most of the attention is paid to the wall-base connections, namely, hold-downs and angle brackets. However, several experimental campaigns aimed at investigating the lateral behaviour of platform CLT buildings have revealed that these structural interactions modify the lateral response of the structure, leading to higher mechanical performance [1] or, ultimately, unexpected failure mechanisms [2].

This paper deals specifically with the problem of the interaction between floor diaphragms and segmented shear walls (see Figure 1). Among the several effects, the interaction between these structural components contributes to stiffening the lateral response of segmented shear walls and, under some conditions, can even lead to local failures of the floor panels.

In particular, the current paper provides a generalization of the methodology proposed by D'Arenzo et al. [3], in which an analytical model describing the interaction mechanism of two-panel segmented CLT shear walls and

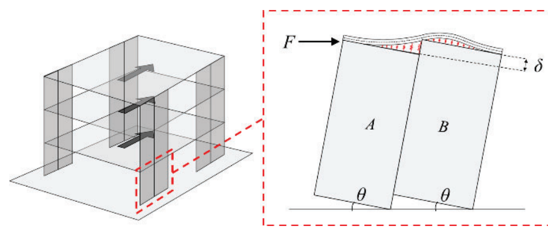


*Figure 1: Representation of the interaction between floor diaphragms and segmented CLT shear walls.*

CLT floors was presented. The analytical model, which is based on the theory of a beam on a bed of elastic springs, will be used in this study to describe the floor-to-wall interaction in the case of three-panel segmented walls. The reliability of the analytical model will be proved by means of numerical simulations and a parametric analysis, in which several wall geoemtries and connection stiffnesses are considered.

## 2 ANALYTICAL MODEL

The majority of the mechanical models available in the literature used for the prediction of the lateral response of CLT shear walls take into account the properties of the CLT wall segments and the wall base connections, yet do not consider the connections which are on the perimeter of the CLT panels, such as the connections between perpendicular walls or between floor and wall panels (see, for instance, Lukacs et al. [4]). In the model proposed by D'Arenzo et al. [3], the mechanical system is enlarged by considering the additional contribution of the floor elements and the floor-to-wall connections to account for

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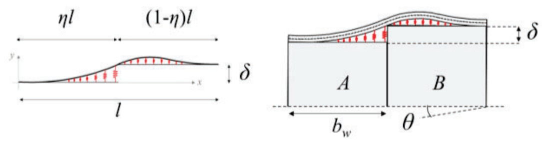


(a) (b)

***Figure 2****: Analytical model of floor-wall interaction.*

the effects of the interaction between segmented CLT shear walls and horizontal floor diaphragms. The effect of this interaction mainly contributes to reducing the slip between the vertical joints of the wall panels and, consequently, results in an increase of the rocking stiffness of the shear wall.

This interaction can be described by means of a mechanical model of a beam on a bed of springs, in which the beam represents the floor element and the springs represent the floor-to-wall connections. Figure 2 depicts the mechanical problem of this interaction, while Eq. (1) represents the differential equation governing the problem.

$$w^{IV}(x) + 4\lambda^4 w(x) = 0 \quad (1)$$

In Eq. (1), $w(x)$ represents the transversal displacement of the beam and $\lambda$ is an elastic parameter, which takes into account the elastic properties of the system, i.e., the bending stiffness of the beam $EI$ and the stiffness per unit length of the springs $k$, see Eq. (2).

$$\lambda = \sqrt[4]{\frac{k}{4EI}} \quad (2)$$

The solution to the differential Eq. (1) can be written as piecewise function defined over the two segments of length $\eta l$ and $(1-\eta)l$, see Figure 2 and Eq. (3).

$$w(x) = \begin{cases} w_A(x) = w_{g,A}(x); & 0 \le x \le \eta l \\ w_B(x) = w_{g,B}(x) + \delta; & \eta l \le x \le l \end{cases} \quad (3)$$

In Eq. (3), the functions $w_{g,A}(x)$ and $w_{g,B}(x)$ represent exponential functions solution of Eq. (1), which depend on constants to be defined in accordance to the boundary conditions of the system. $\delta$ represents the slip between the two wall panels, which triggers the resisting mechanism of the floor-wall interaction.

The boundary conditions of the system depicted in Figure 2 are reported in Table 1 and take into account the rotation, the bending moment, and the shear of the floor-beam, expressed as derivative of the transversal displacement.

It should be noted that the mechanical behaviour of the floor-to-wall connections is different for tensile and compressive forces. While the stiffness of the floor-to-wall connection is engaged in case of tensile forces, a contact mechanism takes place in case of compressive forces. This implies a nonlinearity of the mechanical behaviour of the floor-to-wall connections, which, however, cannot be described with the linear elastic model presented in this study. The solution adopted by D'Arenzo et al. [3] was to consider a linear behaviour of the floor-to-wall connections and to check downstream that the solution found is consistent with the real mechanical behaviour of the system.

***Table 1:*** *Boundary conditions of the analytical model of floor-wall interaction.*

| $x = 0$ | $x = \eta l$ | $x = l$ |
|---|---|---|
| $w''_A(0) = 0$ | $w'_A(\eta l) = w'_B(\eta l)$ | $w''_B(0) = 0$ |
| $w'''_B(0) = 0$ | $w''_A(\eta l) = w''_B(\eta l)$ | $w'''_B(0) = 0$ |
| | $w'_A(\eta l) = \delta$ | |
| | $w'_B(\eta l) = \delta$ | |

## 3 EQUIVALENT SPRING

In D'Arenzo et al. [3], the expression of an equivalent spring to be placed between the vertical joints of two-panel segmented shear walls was presented, to provide a simple strategy to consider the effects of the floor-to-wall interaction (Figure 3). In this study, the equivalent spring will be validated for the case of three-panel segmented shear walls.

The mechanical system consisting of the floor and floor-to-wall connections provides stiffness that reduces the slip between the wall panels. This stiffness can be represented by a vertical equivalent spring in the joint of two wall panels, and can be calculated as the ratio between the shear force in the floor beam, $V(b_w)$, and the slip of the vertical joint of the wall, $\delta$. This equivalent spring reproduces the effect of the floor-wall interaction for the rocking behavior of the segmented shear wall, based on the elastic properties of the floor beam and floor-to-wall connections, and independent of the kinematic behaviour of the segmented shear wall.

The stiffness of the equivalent spring, $k_{f\text{-}w,eq}$, can be determined using the polynomial function in Eq. (4), which depends on the bending stiffness of the floor beam, $EI$, the elastic parameter, $\lambda$, and the length of structural system, $l$ (see Figure 2 and Figure 3).

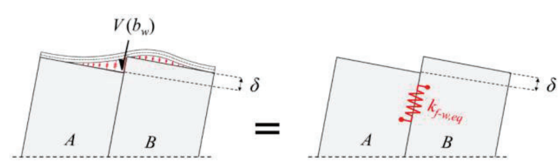


***Figure 3:*** *Representation of the equivalent spring.*

$$k_{f-w,eq} = \frac{V(b_w)}{\delta} = EI \cdot (p_{00} + p_{10}l + p_{01}\lambda + p_{11}\, l\, \lambda + p_{20}l^2 + p_{02}\lambda^2) \quad (4)$$

***Table 2:*** *Polynomial coefficients for the calculation of the equivalent spring stiffness (Eq. (4)).*

| $\lambda\ [m^{-1}]$ | $p_{00}$ | $p_{10}$ | $p_{01}$ | $p_{11}$ | $p_{20}$ | $p_{02}$ |
|---|---|---|---|---|---|---|
| $0.5 \leq \lambda \leq 0.7$ | 0.73100 | 0.05060 | -4.28300 | 0.21595 | -0.01507 | 5.19500 |
| $0.7 \leq \lambda \leq 1.2$ | 2.84650 | 0.12635 | -10.2850 | 0.32880 | -0.03467 | 8.91500 |
| $1.2 \leq \lambda \leq 2.0$ | 8.02500 | 1.41600 | -22.6350 | 0.06530 | -0.14895 | 14.3100 |

The polynomial coefficients, $p_{ij}$, are listed in Table 2 for three different ranges of the elastic parameter, $\lambda$. Eq. (4) and the coefficients in Table 2 can be used to calculate the stiffness of the equivalent spring, $k_{f-w,eq}$, in $[kN/m]$ by using the bending stiffness, $EI$, in $[kNm^2]$, the system length, $l$, in $[m]$ and the elastic parameter, $\lambda$, in $[m^{-1}]$.

## 4 METHODOLOGY

To predict the floor-wall interaction for three-panel shear wall sytems, the analytical model is used to describe the shear force of the floor element by considering the three-panel system as the superimposition of two two-panel systems which share the central panel, see Figure 4. In particular, the analytical model is used to predict the shear force of the floor-beam in the region above the two vertical joints of the shear wall, as this represents the shear force value relevant for the definition of the equivalent spring. If the analytical model predict with sufficient accuracy the shear force in the region above the vertical joints of the shear wall, the equivalent spring can then be used for the case of three-panel shear wall systems.

The methodology used in this study involved comparison between the results obtained from the analytical model presented in the previous section and the numerical models presented in the next section. The numerical models were distinguished in deatailed models and simplified models. In the deatailed models, the floor-wall interactions was considered by modelling the floor and the floor-to-wall connections, while in the simplified models, the floor-wall interactions was considered through the equivalent spring.

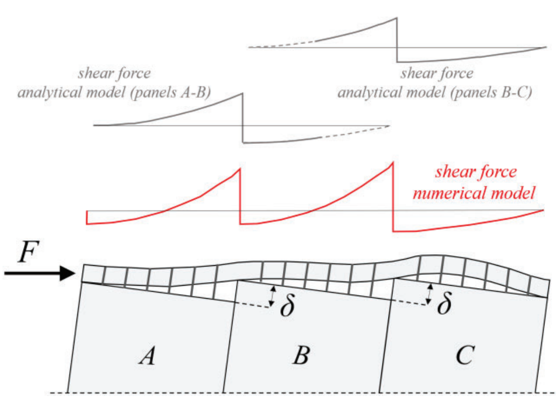


***Figure 4:*** *Representation of the methodology adopted to use the analytical model for three-panel shear walls.*

Two types of analysis were considered: (i) local analysis, at floor level, to evaluate the capability of the analytical model to describe the internal actions of the floor, and (ii) global analysis, at wall level, to evaluate the capability of the equivalent spring to simulate the effect of the floor-wall interaction in three-panel shear wall systems. In the first analysis, the comparison is conducted by directly comparing the shear force functions of the floor, as shown in Figure 4. In the second analysis, the comparison is conducted by comparing the rocking stiffnesses of the shear walls, calculated as the ratio between the lateral force applied on the top of the wall, $F$, and the rocking displacement, $\Delta_{rock}$, namely the lateral displacmenet on the top of the wall.

## 5 NUMERICAL MODELS

In this section the numerical models are presented. Three different numerical models were considered: (i) shear walls with floor-wall interactions considered through a detailed modelling of the floor and the floor-to-wall connections (Det-Model), (ii) shear walls with floor-wall interactions considered through the equivalent spring (EqSp-Model), and (iii) shear walls without floor-wall interactions (NoFl-Model). The last numerical model is considered for purpose of comparison, to evaluate the effects of the floor-to-wall interaction on the rocking stiffness of the shear wall. An illustration of the Det-Model for a three-panel shear wall system can be seen in Figure 5.

The same modelling strategy was used to model the wall segments and the base connections of the numerical three models considered. The wall panels were modeled as shell elements with orthotropic elastic material. To simulate the rocking behaviour of the shear walls, the external corner of the shell element under compression was hinged while the internal corner was restrained with a gap element. This allowed for free uplift of the internal corner while preventing displacement in the opposite direction to simulate the wall-to-foundation contact. The sliding mechanism of the shear-wall was disregarded as it has no influence on the rocking behaviour of the shear-wall. The external corner under tension was restrained with a vertical spring that represents the vertical stiffness of the hold down or the weighted vertical stiffness of all wall base connections. 2-joint links with high compression stiffness, no tension stiffness, and linear elastic stiffness in the transversal direction were placed between the two shell elements to prevent wall panel interpenetration and simulate the wall-to-wall connection flexibility in the transversal direction.


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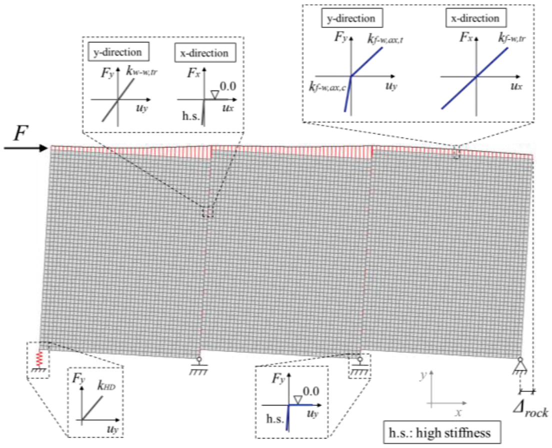


***Figure 5:*** *Numerical model of floor-wall interaction (Det-Model).*

In the Det-Model, the floor was modeled as a linear elastic beam with an orthotropic material. The connections between the floor and the walls were modeled to account for their different behavior under tensile and compressive forces. While for tensile forces the behaviour of the floor-to-wall connection depends on the withdrawal stiffness of the connections, for compressive loads, the floor-to-wall contact takes place. This different behaviour for tensile and compressive forces was modelled with a 2-joint link with multilinear behaviour. For tensile forces the behaviour was assumed with stiffness of the floor-to-wall connection, while for compressive forces an equivalent stiffness that simulates the floor-to-wall contact was considered. Additionally, the horizontal stiffness of the floor-to-wall connection was considered in these links.

In the EqSp-Model, one equivalent spring was placed in the vertical joints between the shell elements representing the wall panels, in addition to the wall-to-wall connections.

A horizontal force was applied to the top of the wall panel, in all numerical models. Nonlinear static analyses were conducted to account for the multilinear behavior of the links simulating hold down, wall-to-wall, and wall-to-floor connections.

## 6 PARAMETRIC ANALYSIS

The analytical model is validated through a parametric analysis using common geometrical and mechanical parameters used in construction. The analysis considers two symmetric systems, each consisting of two or three wall panels of equal width anchored at both ends with two hold downs and an overlying horizontal floor with evenly spaced floor-to-wall connections, as depicted in Figure 6. For the validation, three wall panel aspect ratios were utilized, each with a height equal to $h = 2.5\ m$ and widths of $b_w = \{1.5 - 2.0 - 2.5\}\ m$. A 5-layer panel with a total thickness of $t_w = 120\ mm$ and individual layer thicknesses of **30**-20-**20**-20-**30** mm (with the thickness of the vertically arranged layers in bold) was considered in the analysis. Three different floor panels typologies with

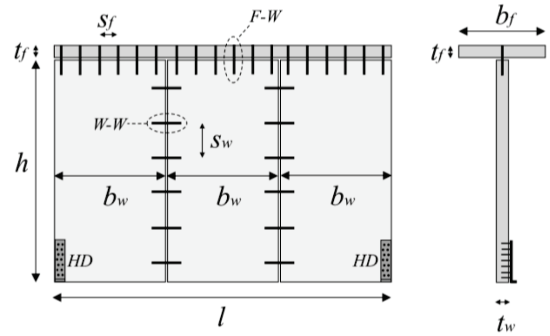


***Figure 6:*** *Reference system used for the parametric analysis.*

total thickness equal to $t_f = \{140 - 180 - 220\}\ mm$ and layer thicknesses as shown in Table 3 were analysed. The floor width was not chosen a priori as it depends on the effective section that contributes to internal bending actions, as per composite section beam theory. The effective width was calculated using the methodology developed by Masoudnia et al. [21], which demonstrates that the effective width is dependent on the wooden layer structure of the CLT panel. The values obtained for the three floor-panels were $b_f = \{1006 - 479 - 636\}\ mm$.

***Table 3:*** *Thickness (in mm) of the floor panels used in the parametric analysis.*

| ID | $t_f$ | $t_1$ | $t_2$ | $t_3$ | $t_4$ | $t_5$ | $t_6$ | $t_7$ |
|---|---|---|---|---|---|---|---|---|
| 1 | 140 | **30** | 25 | **30** | 25 | **30** | - | - |
| 2 | 180 | **30** | **30** | 20 | **20** | 20 | **30** | **30** |
| 3 | 220 | **30** | **30** | 35 | **30** | 35 | **30** | **30** |

The floor-to-wall connection spacing and withdrawal stiffness were selected to provide a broad range of stiffness per unit length $k$ in the analytical model. The withdrawal stiffness of the screws was set at $k_{with} = 4kN/mm$, as reported in the experimental study of Gavric et al. [5]. This value is representative of floor-to-wall connections with partially threaded screws 10×260 mm. The screw spacing in the floor-to-wall connection was set at $s_f = \{900 - 300 - 100\}\ mm$. The shear stiffness of the floor-to-wall connections was set at $k_{sh} = 4kN/mm$, based on the experimental results from Gavric et al. [5]. The hold down stiffness was chosen as $k_{HD} = 10\ kN/mm$, as reported by Hummel et al. [6]. The CLT panels of both floor and walls were assumed to be made of wooden boards with an elastic modulus in the longitudinal direction of $E_0 = 11.6\ GPa$.

### 6.1 MECHANICAL PROPERTIES OF THE ANALYTICAL MODEL

The stiffness per unit length $k$ of the analytical model was determined using Eq. (5), based on the withdrawal stiffness and the spacing of the floor-to-wall connections. The values presented in the previous section were utilized to calculate the stiffnesses per unit length as $k = \{4.4 - 13.3 - 40.0\}\ N/mm^2$.

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$$k = \frac{k_{with}}{s_f} \tag{5}$$

The bending stiffness of the floor-beam was calculated based on the layered structure of the floor panel, as described by Blaß and Fellmoser [7], excluding the effect of the wooden boards in the transverse direction. Using the geometrical and elastic properties of the floor outlined in the previous section, the following bending stiffnesses were adopted for the floor beam in the analytical model: $EI = \{2197.7 - 2606.5 - 5947.6\}\ kNm^2$.

### 6.2 MECHANICAL PROPERTIES OF THE NUMERICAL MODEL

The wall panels were modeled as shell elements and their elastic properties were assigned based on the layered structure of CLT panels. Equivalent longitudinal and transversal elastic moduli of $E_{eq,l} = 7.7\ GPa$ and $E_{eq,t} = 3.9\ GPa$ were assigned to the orthotropic shell elements, based on the methodology proposed by Blaß and Fellmoser [7]. Equivalent shear modulus of $G_{eq} = 0.5\ GPa$ was assigned using the methodology proposed by Brandner et al. [8]. The floor-to-wall connections were modeled as links with spacing $a_f = 50mm$. The tensile stiffness of these links was calculated using Eq. (6), based on the stiffness per unit length of the analytical model and the spacing of the links in the numerical model.

$$k_{f-w,ax,t} = k \cdot a_f \tag{6}$$

The axial stiffnesses of the links simulating the floor-to-wall connections were determined using the values in the previous section, resulting in $k_{f-w,ax,t} = \{0.22 - 0.67 - 2.00\}\ kN/mm$.

The compressive stiffness of the links simulating the floor-to-wall connections, $k_{f-w,ax,c}$, was calculated using Eq. (7) as described by Schickhofer and Ringhofer [9] and Blaß and Görlacher [10].

$$k_{f-w,ax,c} = \frac{E_{90} \cdot A_{ef}}{t_f} = \frac{E_{90} \cdot l_{ef} \cdot a_f}{t_f} \tag{7}$$

The equation considers the effective area under compression in the floor element, $A_{ef}$, and the elastic modulus perpendicular to the grain of the floor, $E_{90}$. The effective area was determined by multiplying an effective length, $l_{ef}$, by the spacing of the link elements, $a_f$. The effective length was calculated based on a stress distribution line with an inclination of 1:3, according to Eurocode 5 [11] specifications. The resulting compression stiffnesses for the links simulating the floor-to-wall contact were $k_{f-w,ax,c} = \{45.7 - 40.0 - 36.4\}\ kN/mm$.

The transversal stiffness of the floor-to-wall connections was computed using Eq. (8), which considers the different spacings of the numerical model, $a_f$, and the reference system, $s_f$.

$$k_{f-w,tr} = \frac{a_f}{s_f} \cdot k_{sh} \tag{8}$$

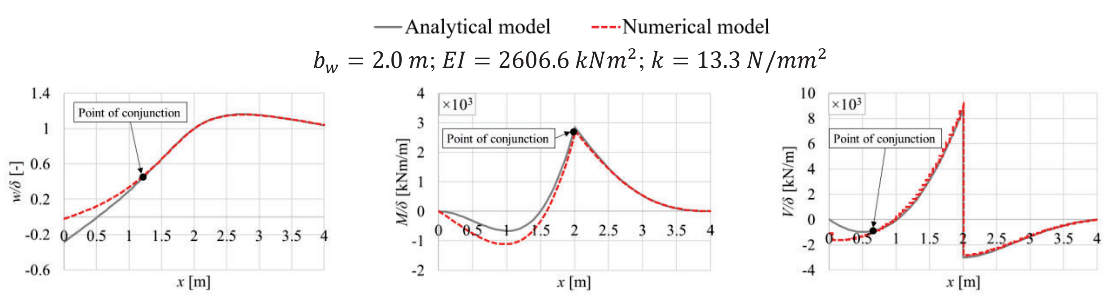
Based on the values presented in previous sections, the following transversal stiffnesses were utilized for the floor-to-wall connections: $k_{f-w,tr} = \{0.22 - 0.67 - 2.00\}\ kN/mm$. The high stiffness of the wall-to-wall connections and gap element was set to $10^6 kN/mm$.

## 7 RESULTS AND DISCUSSION

### 7.1 ANALYSIS AT FLOOR LEVEL

The validation of the analytical model is carried out by comparing the displacement, the bending moment, and the shear force of the floor-beam obtained from analytical and numerical models. Figure 7 shows this comparison for the case of systems with two-panel shear walls with width $b_w = 2.0\ m$, floor bending stiffness $EI = 2606.6\ kNm^2$, and floor-to-wall connection stiffness per unit length $k = 13.3\ N/mm^2$. It can be observed that the analytical and numerical model results converge starting from a point of conjunction, located in the first half of the floor length.

For the case of three-panel shear walls, the analytical model is used to describe the shear force of the floor element. This is done by considering the three-panel system as the composition of two two-panel systems, as discussed in section 4. Figure 8 shows the comparison between the shear force obtained from the analytical and numerical model for the case of three-panel shear walls with width $b_w = \{1.5 - 2.0 - 2.5\}\ m$, floor bending




(a) (b) (c)

***Figure 7:*** *Displacement, bending moment, and shear force function of the floor element for the two-panel shear walls.*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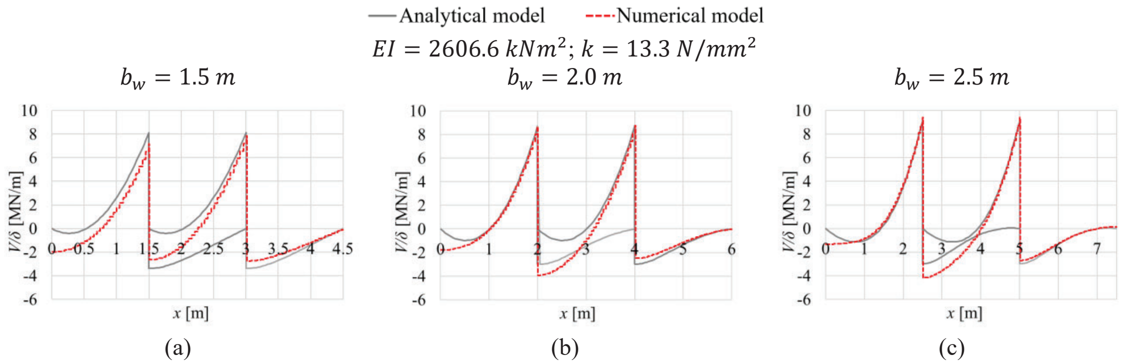


***Figure 8:*** *Shear force function of the floor element for the three-panel shear walls.*

stiffness $EI = 2606.6\ kNm^2$, and floor-to-wall connection stiffenss per unit of length $k = 13.3\ N/mm^2$. It can be observed that the trend of the shear force for the three-panel system is similar to that of the two-panel system. In particular, the analytical and numerical models show a good agreement in the region above the vertical joints of the shear walls, with identical values for the cases with $b_w = 2.0\ m$ and $b_w = 2.5\ m$, and reasonably close values for the cases with $b_w = 1.5\ m$. This allows the equivalent spring defined for two-panel shear walls to be used for three-panel shear walls as well.

### 7.2 ANALYSIS AT WALL LEVEL

In this section the effect of the floor-wall interaction is evaluated at wall level. The results of the three numerical models described in section 5 are compared, (i) to evaluate the impact of the the floor-wall interaction on the lateral behavior of the shear wall, and (ii) to validate the equivalent spring in case of three-panel shear wall systems. In particular, the rocking stiffness of the models that consideres the floor-wall interaction (Det-Model and EqSp-Model) are compared with the rocking stiffness of the models without floor-wall interaction (NoFl-Model). Figure 9 shows the comparison of the dimensioneless rocking stiffness of the systems with width $b_w = \{1.5 - 2.0 - 2.5\}\ m$, floor-to-wall connection stiffenss per unit length $k = \{4.4 - 13.3 - 40.0\}\ N/mm^2$, and floor bending stiffness $EI = 2606.6\ kNm^2$. The dimensionless rocking stiffness, $\widetilde{K}_{rock}$, was calculated as the ratio between the rocking stiffness of the models that consideres the floor-wall interaction and the rocking stiffness of the model without floor-wall interaction.

In general, the graphs show that the floor-wall interaction increases the rocking stiffness of the shear walls. The increase of rocking stiffness is primarily governed by the withdrawal stiffness of the floor-to-wall connections. For instance, for the systems with panel width $b_w = 2.0\ m$, the increase of rocking stiffness associated to the stiffness per unit length $k = \{4.4 - 13.3 - 40.0\}\ N/mm^2$ are equal to 24.8 %, 48.5 %, and 84.9 %, respectively. The panel width also affects the increase of rocking stiffness; for a floor-to-wall connection stiffness of $k = 13.3\ N/mm^2$, the increase of rocking stiffness associated to the panel widths $b_w = \{1.5 - 2.0 - 2.5\}\ m$ is equal to 43.8 %, 48.5 %, and 50.8 %, respectively.

The graph also reports the kinematic behaviour associated to each system. It is possible to see that if the effect of the floor-wall connection is neglected, the shear walls deform with a couple panel (CP) kinamtic behaviour, namely with

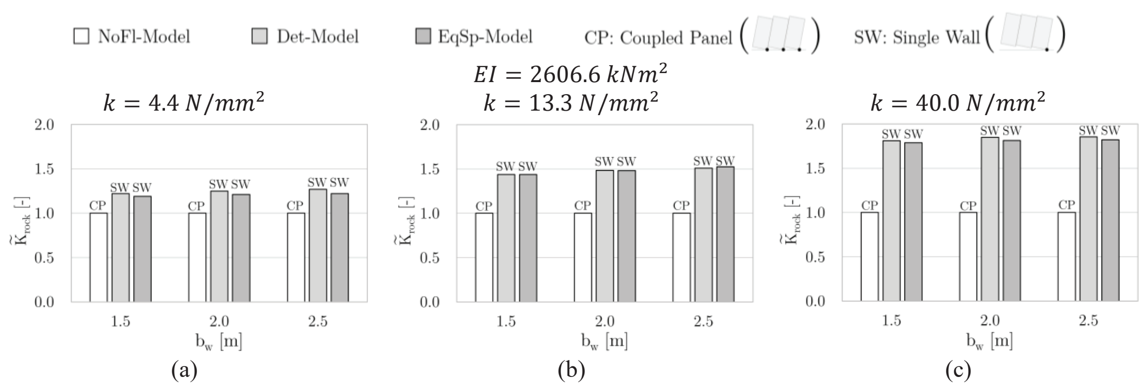


***Figure 9:*** *Dimensionless rocking stiffness of the three-panel shear walls with and without floor-wall interaction.*

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each wall panel being in contact with the foundation and the shear wall having multiple centre of rotations. On the other hand, when the floor-wall interaction is considered the kinamtic behaviour switchs to Single Wall (SW), meaning with only one panel in contact with the foundation and with the whole shear wall pivoting about one point. This is due to the fact that the floor-interaction fictitiously increases the overall stiffness of the wall panel vertical joints.

From Figure 9, it is also possible to observe the accuracy of the models with the equivalent springs (EqSp-Model). In fact, the values of rocking stiffness of these models are very close to those obtained from the models with detailed modelling of the floor-wall interaction. The largest discrepancy between these models is 3.8 % and is found in the case of system with $b_w = 2.5\,m$ and $k = 4.4\,N/mm^2$.

The results showed in Figure 9 reveal that the floor-wall interaction increases the rocking stiffness of segmented shear walls and may lead in some cases to modification of the kinematic behaviour, from coupled panel to single wall. This is particualry important to be considered in contexts of seismic design, when highly dissipative behaviours are expected from the vertical joints of segmented shear walls.

## 8 CONCLUSIONS

This paper presents a study on the effect of the floor-wall interaction on the rocking behaviour of multi-panel CLT shear walls. Previous research on two-panel wall systems conducted by D'Arenzo et al. [3] showed that the floor-wall interaction generates internal actions in the floor element and reduces the slip in the vertical joints of the wall panels. The internal actions can be described with an analytical model of beam on elastic springs, while the reduction of slip in the vertical joints of the wall panels can be simulated by means of an equivalent spring. In this study, the analytical model and the equivalent spring presented by D'Arenzo et al. [3] were used to describe the effects of the floor-wall interaction in the case of three-panel wall systems.

The results of this study showed that the analytical model can be used to describe the shear force of floor elements connected to three-panel shear wall systems. The comparison of the shear force obtained from the analytical and numerical models for three-panel systems showed a good agreement in the region above the vertical joints of the shear walls, with identical values for the cases with 2.0 m and 2.5 m panel width and reasonably close values for the case with 1.5 m panel width. This allowed the equivalent spring defined for two-panel systems to be used for three-panel systems.

At wall level, the results showed that the floor-wall interaction increases the rocking stiffness of the wall. It was found that the increases of rocking stiffness is primarily governed by the stiffness of the floor-to-wall connections and affected by the wall panel width. The increase of rocking stiffness was observed for all the systems, with the highest increase observed for the systems with the highest floor-to-wall connection stiffness and largest wall panels width.

These results demonstrate the significance of considering the interaction between floor diaphragms and segmented shear walls when evaluating the lateral response of multi-panel CLT shear walls. Nonetheless, further research is required to extend the findings of this study to the case of multi-panel systems with more than three panels both for linear elastic and nonlinear analyses.

The study provides valuable insights into the interaction between floor diaphragms and segmented shear walls in CLT buildings, demonstrating that these interactions can significantly modify the lateral response of CLT structures. The analytical model developed in this study provides a useful tool to account for the floor-to-wall interaction in the analysis of CLT buildings.

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