SEISMIC PERFORMANCE ASSESSMENT OF STRUCTURAL WALLS SUPPORTING TOWERS IN TALL BUILDINGS

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Abstract

This paper examines the seismic shear demands on tower walls supporting tall buildings. It has been shown in earlier studies that the podium structure can impose different boundary conditions (restraints) on the tower walls. The resulting displacement incompatibility between connected walls imposes high in-plane strains in the slabs and beams connecting the tower wall above the podium interface level. Strutting (compatibility) forces have been shown to be the primary contributors to the occurrence of high internal force transfer between the floor slab and the walls. The scope of the study is extended to examine buildings featuring discontinued walls planted on a transfer plate at the podium interface level. A predictive model has been proposed in order that the additional shear force demands on the walls can be estimated. The model has been verified against results obtained from finite element analyses using 2D and 3D building models.

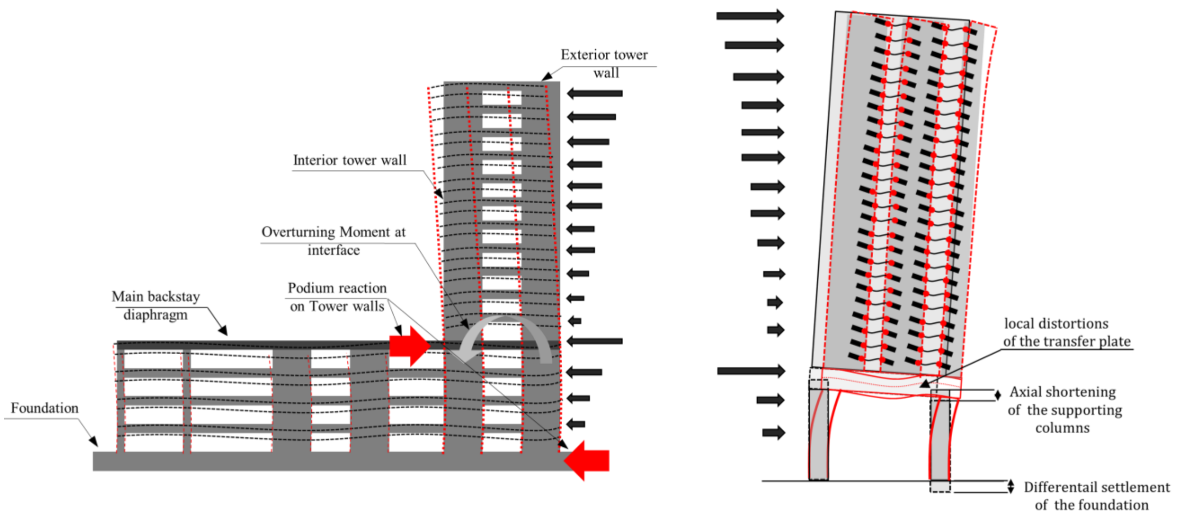
Keywords: Slab-wall interaction, compatibility forces, podium-tower buildings.

1 INTRODUCTION

Podiums are augmented floor areas at the lower level of medium-rise and tall buildings. This form of construction is favoured in metropolitan cities in regions of low-to-moderate seismicity as a building with this configuration can accommodate different functionalities (i.e. commercial space in the lower podium levels and residential/office space in the tower). The lateral load resisting system for such building structures comprises moment resisting frames and shear walls. As the tower of the building is positioned at an offset relative to the centre of the podium, high torsional demands can be imposed on the podium and high shear forces can be induced on the tower shear walls thereby jeopardising their structural integrity when subject to severe earthquake ground shaking (Elnashai and Soliman, 1995, Moehle, 1984, Wood, 1992, Yacoubian et al., 2017a, Mwafy and Khalifa, 2017). Recommendations against this form of construction have not been mandated in design codes in spite of potential undesirable behaviour in a rare seismic event (AS 3600, 2009, AS1170.4, 2007, ASCE 41-6, 2006).

In some cases, architectural requirements mandate discontinuities in some tower walls and columns. Transfer structures (plates or girders) are thus introduced at the interface level to restore the load path continuity between the upper (tower) and lower (podium) levels in the building. The resulting irregular lateral stiffness distribution up the height of the building can impose additional intricacies to the lateral response behaviour of the building especially when considerations are made for the local deformations of the transfer structure (Li, 2005, Su et al., 2002).

In both building configurations, the podium structure redistributes lateral loads from the tower to the supporting foundations. For the case of setback buildings, this is achieved by the reactive "backstay" forces that are developed in the interfacial diaphragm to resist overturning actions from the tower structure (refer Fig. 1a). On the other hand, distortions of the transfer plate together with the axial push-pull action of the supporting podium columns and structural walls partly contribute to the lateral load resistance of the building (Fig. 1b).



(a) Backstay actions in podium-tower sub-assemblage

(b) local and global deformations in transfer structures

Figure 1. Lateral load resisting mechanism in the investigated building types.

The shear anomalies generated by virtue of the differential podium restraints on individual tower walls in podium-tower structure are not well understood (Yacoubian et al., 2017a). The structural walls closer to the centre of stiffness of the podium structure (referred herein as the interior wall) are subject to higher restraint at and below the interface level than the exterior wall (see Fig. 1a). Similarly, structural walls planted on the transfer plate are subject to different base rotations imposed by the local distortion of the transfer plate. In both cases, the interference of the podium on the lateral response of the tower walls results in incompatible displacements of connected structural walls. Compatibility (in-plane) forces are shown to be developed in the connecting floor slabs and beams to restore displacement incompatibility between the walls. Rutenberg (2004) and Bayer et al. (2014) first examined the evolution of these compatibility forces in floor slabs spanning between structural walls. Gardiner et al. (2008) and Bull (2004) further examined incompatibility issues resulting from abrupt stiffness variations up the height of the building and dual frame-wall interaction. Their work highlighted the detrimental increase in transfer (in-plane) forces when the structure undergoes inelastic response behaviour.

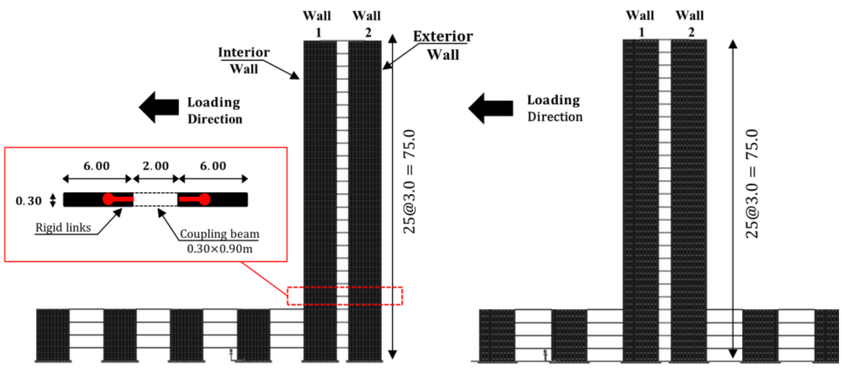
In this paper, the effects of podium interferences on the structural walls are first examined by way of analyses on 2D planar tower-podium sub-assemblage building models. Attention is cast on anomalies in shear distributions between tower walls in setback buildings (Section 2.1) and in buildings featuring transfer plates (Section 2.2). The findings are also verified through the analysis of a 3D finite element model of a case study building (Section 3).

2 ANALYSES ON 2D PLANAR PODIUM-TOWER SUB ASSEMBLAGES

In the first part of this study, the trends of the lateral response of 2D planar podium-tower sub-assemblage models of the building are examined by taking into account the interferences of the podium structure. The displacement response behaviour and shear force distribution of connected tower walls above and below the podium level are investigated for setback buildings (Section 2.1) and buildings featuring a transfer plate (Section 2.2).

2.1 Podium interference in setback structures

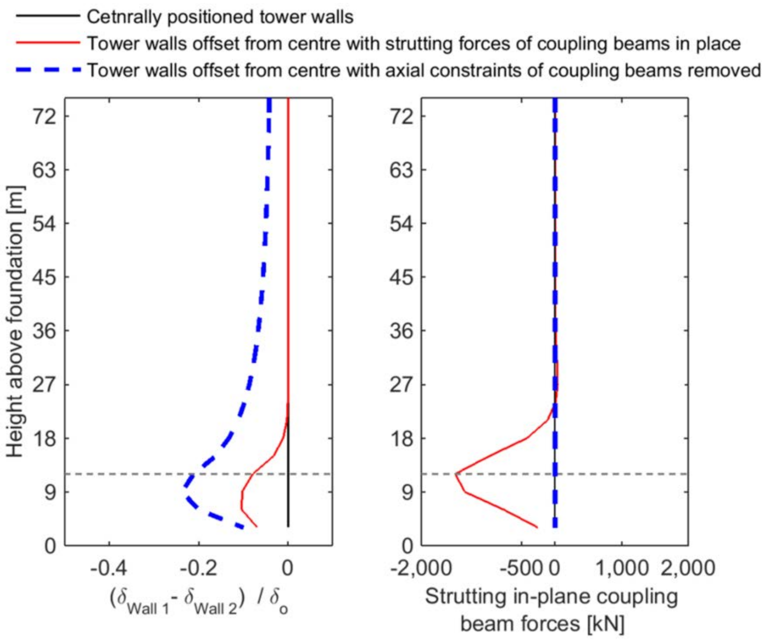
The 2D model shown in Fig. 2a represents the primary load resisting system of a building wherein the tower is not symmetrically positioned on the supporting podium structure. To emphasise on the effects of the geometric configuration models with the tower centrally positioned were also analysed in parallel (Fig. 2b). The models were employed in equivalent lateral load analyses based on the Australian design spectrum defined in the AS 1170.4 (2007).
The occurrence of significant strutting (in-plane) forces in the interconnecting beams between the interior and exterior walls up the height of the sub-assemblage model (Fig. 2a and 2b) are first illustrated in Figure 3.



(a) Tower walls offset from podium centre (b) Centred tower walls

Figure 2. Example podium-tower sub-assemblages

The difference in tower wall displacements ($\delta_{Wall\,1} - \delta_{Wall\,2}$) up the height of the building are normalised with respect to the storey displacement of coupled walls that are not connected to a podium structure (δ_o). The podium is shown to have a higher restraint on the interior wall (closer to the centre of stiffness of the podium) compared with the exterior wall. This is shown when comparing relative wall displacement trends up the height of the building. This wall displacement incompatibility is not manifested in the case where the tower walls are centred with respect to the podium. The direct result of the observed displacement incompatibility is the generation of substantial strutting (compatibility) forces in the beams connecting the walls (see Fig. 3b). These forces are shown to peak at the interfacial zone between the podium and the tower. Significant in-plane force demands on the connecting beams are also observed few storeys above the podium level. Interestingly similar trends were not found in the case where the tower walls are centrally positioned (Fig. 2b).



(a) Comparison between wall displacement up the height of the building

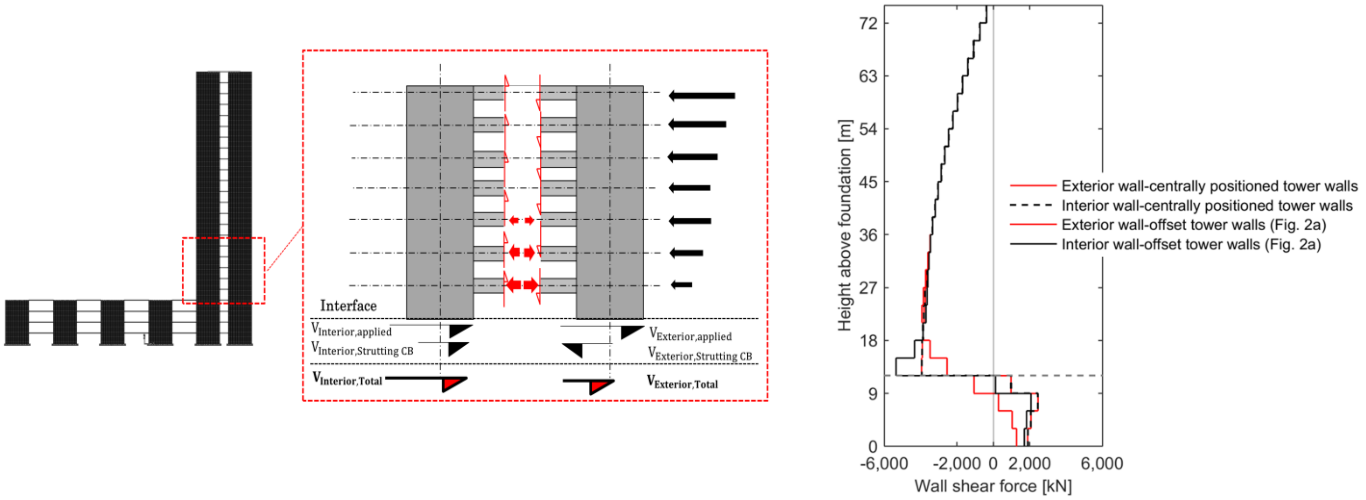
(b) Strutting force profile in the coupling beams between the walls

Figure 3. Results of the 2D planar model of setback buildings

The observed anomaly can also be illustrated by analysing a podium-tower sub-assemblage model with the axial constraint of the connecting beams is removed (set to a value close to zero). With reference to Fig. 3, when these axial restraints are removed, the ingression of the exterior wall becomes more adverse. This is also shown in Fig 3a (blue dashed line) where the incompatible wall displacements ($\Delta r < 1$) are maintained up the height of the building. Comparing Figs. 3a and 3b it can be shown that both strutting forces and incompatible displacements are concurrent. The latter highlights the significance of the role of connecting beams in restoring the wall displacement incompatibility in setback buildings.

The internal strutting in-plane forces shown in Fig. 3b are shown to result in local shear force redistributions between the connected tower walls when the building is subjected to lateral loading. Specifically, the shear intensity of the interior wall is increased locally (beyond equilibrium requirement) while the shear intensity is reduced in the exterior wall (see Fig 4b). The mechanism of this force transfer between the connecting slab and the wall is schematically shown in Fig. 4a.

The location of maximum shear was found to be above the podium level, which is contrary to earlier reports by Bevan-Pitchard et al. (1983) and Rad et al. (2009), where high shear demands were only found below the base in tower walls supported by sub-grade structures and perimeter foundation walls. These strutting (in-plane) forces in the beams are also shown to offset bending moment demands between connected walls (see Fig. 5). Accordingly the M/V (moment-to-shear) ratio of the interior wall is significantly reduced when compared to the connected exterior wall (refer Fig. 5).



(a) Effect of strutting forces on wall shear forces

(b) Shear force distribution in tower walls

Figure 4. Shear force distribution in in walls above and below the podium

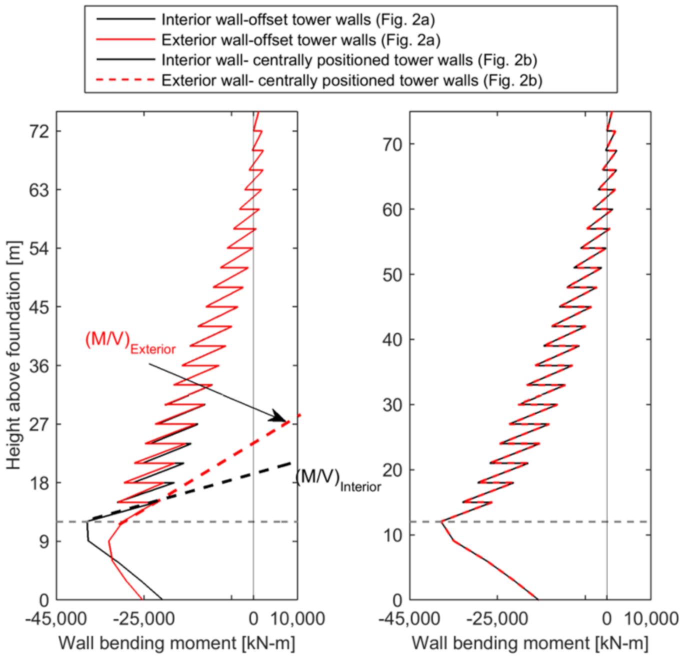


Figure 5. M/V ratio and bending moment distribution in shear walls in podium-tower sub-assemblage

Studies on buildings featuring podiums at various portions of the building's height have been undertaken to quantify the extent of podium interferences on the shear response behaviour of tower walls (Yacoubian et al., 2017a). The study showed that the most adverse scenario with respect to the asymmetric shear distribution (defined as the ratio of the shear intensity of the interior wall to the shear intensity of the exterior wall) occurs when the podium is about 1/4-1/3 of the height of the building (refer Fig. 6).

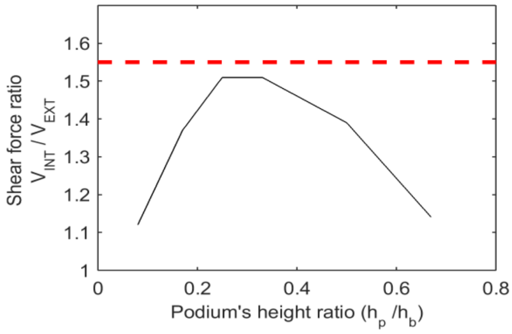


Figure 6. Correlation of the maximum wall shear force ratio with podium height ratio

2.2 Podium interference in transfer structures

The scope of podium interference on the response behaviour of the tower walls is extended herein to buildings featuring transfer plates at the interface level between the tower and the podium.

The 2D building model shown in Fig. 7 comprises of stiff podium columns in the lower levels which support a 1500mm thick transfer plate. The tower walls (annotated by wall 1, 2 and 3) are planted at the transfer floor level. The floor slabs connecting the tower walls are modelled as equivalent frame elements with an effective width (b_{eff}) assigned based on recommendations given by Grossman (1997) and PEER/ATC guideline (2010). The lateral loading profile was similar to the one adopted in the Section 2.1. To The response behaviour of the building was compared to a control model with a rigid transfer plate in order that the effects of transfer plate flexibility can be highlighted. The displacement ratio (Δ_r) is defined as the ratio of the storey lateral displacement of walls 1 (δ_1) and 3 (δ_3) in the original model to the storey displacement (δ_o) of the control model.

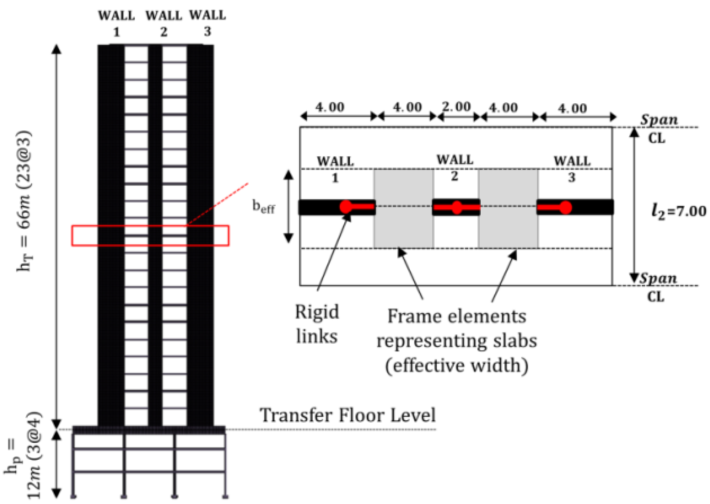


Figure 7. 2D model of the building featuring a transfer plate

Incompatible wall displacements ($\Delta_r \neq 1$) imposed by the flexibility of the transfer plate (by way of rotations at the base of the walls) are shown in Fig. 8. This trend extends to approximately 10% of the tower's height (above the transfer floor level) beyond which the displacement ratios tend to unity (suggesting compatible wall displacements are achieved above this level). Similar to observations reported in Section 2.1, the incompatible wall displacements resulted in the generation of compatibility forces in the floor slabs connecting the walls (see Fig. 8). The mechanism is also illustrated by the analysis of a hypothetical building model with the in-plane stiffness of the connecting floor slabs set to approximately zero value (axial constraints removed). The displacement ratios for wall 1 in both models (original and hypothetical) are obtained following the procedure described earlier (plotted in Fig. 8). When the axial restraints of the floor slabs are removed displacement incompatibilities ($\Delta_r \neq 1$) are shown to extend the entire height of the tower.

Interestingly similar observations were reported in Section 2.1 where the connecting beams have been shown to restore displacement compatibility between connected walls above the interface level. The consequent shear force redistributions between the walls are shown in Fig. 9.
A study conducted by the authors has shown high proportionality between the relative transfer plate rotation at the base of the connected walls (difference in the plate rotation) and the in-plane strutting strains in the connecting floor slabs (Yacoubian et al., 2017b). The two parameters have been also shown to exhibit displacement-controlled conditions and have been accordingly expressed proportionally to the maximum spectral displacement of the ground motion RSD_{max} (Yacoubian et al., 2017b). The peak rotation demand (PRD) has been introduced to quantify the maximum relative transfer plate rotation at the base of connected tower walls planted on a transfer plate (Eq. 1).

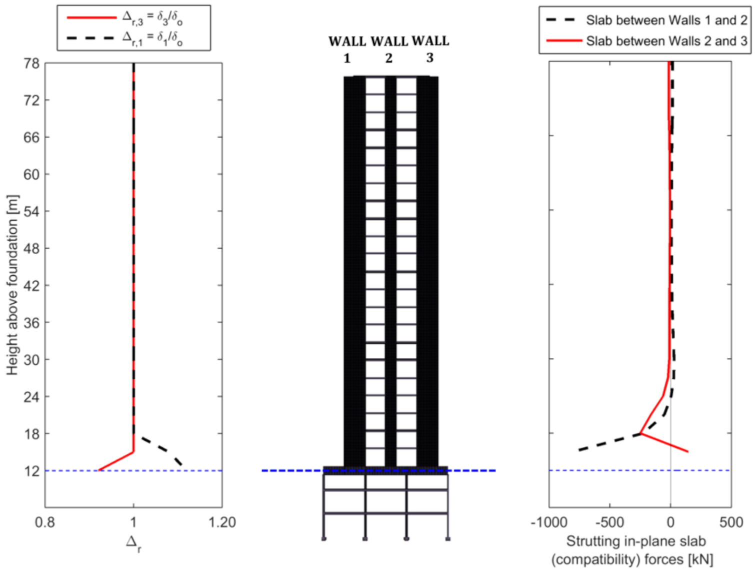


Figure 8. Displacement incompatibility between connected walls and the resulting strutting (compatibility) slab force distribution.

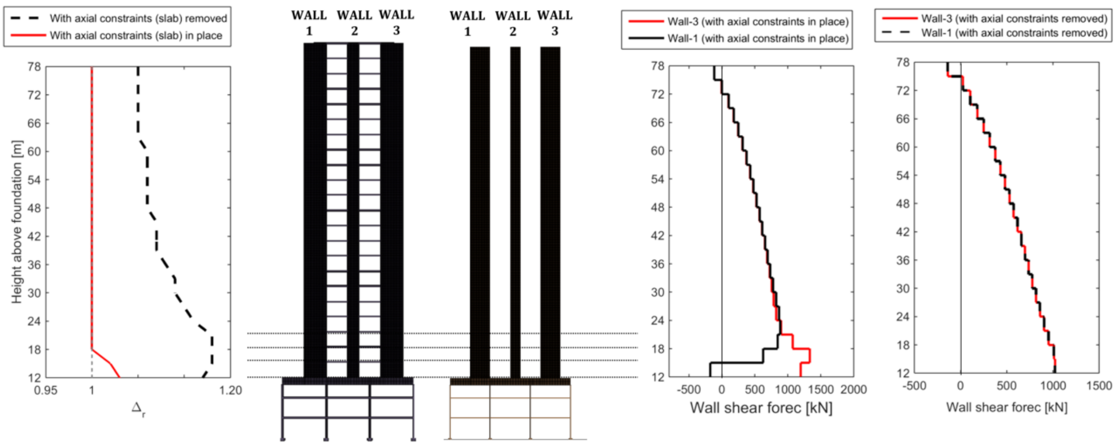


Figure 9. Comparison between the analysed sub-assemblage models with and without the connecting floor slabs.

$$\frac{\mathrm{PRD}}{\bar{\varphi}_{\mathrm{ave}}} = -0.2 \times \ln(\mathrm{b_r}) + 0.6 \qquad (1)$$

where $\bar{\varphi}_{ave}$ is the average drift measured at the effective height of the building corresponding to a displacement magnitude equal to the maximum spectral displacement of the ground motion RSD_{max} and b_r represents the ratio of the rotational and translational stiffness of the building (Yacoubian et al., 2017b). The *flexibility index (FI)* has been introduced to quantify the proportionality between the in-plane slab strains and the relative plate rotation (see Fig.10). For brevity, the reader is referred to earlier works for more details on the derivation of the parameter (Yacoubian et al., 2016, Yacoubian et al., 2017b). Importantly the flexibility index (slope of the line shown in Fig. 10) has been shown to be directly proportional to the ratio of the (flexural) rigidity of transfer plate and the supported wall (parameter α_r in Eq. 2 and Fig. 11).

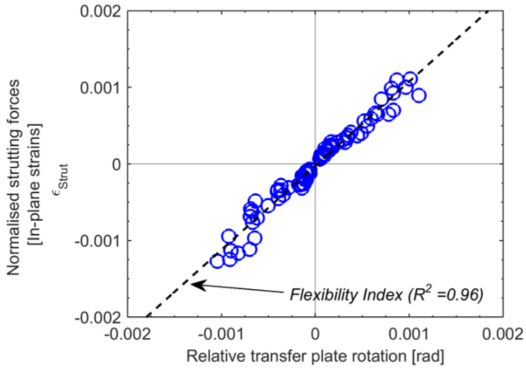


Figure 10. Proportionality of the relative transfer plate rotation and the in-plane strutting strains (definition of the parameter: flexibility index) (Yacoubian et al., 2017b)

$$\alpha_r = \sqrt{\frac{(E_c I)_{TP}}{(E_c I)_{Wall}}} \tag{2}$$

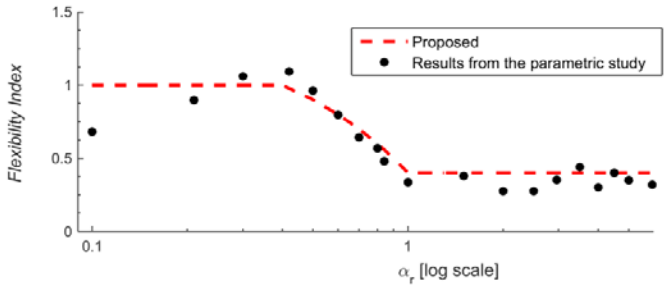


Figure 11. Variation of the flexibility index with α_r (transfer plate rigidity)

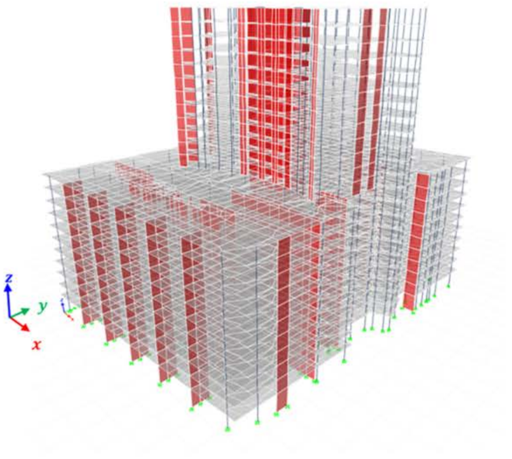
The newly introduced parameter (flexibility index) can provide conservative estimates of the magnitude of the compatibility forces generated in the slabs connecting tower walls (Yacoubian et al., 2017b). The proposed expression (Eq. 3) for estimating the maximum strutting forces in the slabs (F_{STRUT}) above the transfer plate incorporates the PRD, the Flexibility index (proportionality constant) and the effective in-plane properties of the connecting slabs (defined by the term $E_C A_{eff}$).

$$F_{STRUT} = FI \times PRD \times E_C A_{eff} \tag{3}$$

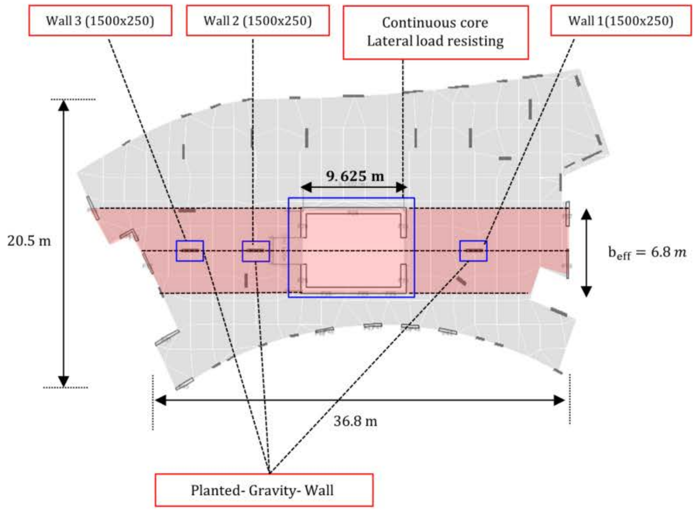
The magnitude of F_{STRUT} also represents the additional shear force demands that are transferred to the connected walls.

3 VERIFICATION STUDY ON 3D FE MODEL OF A CASE STUDY BUILDING

The 72.5m (24 storey) reinforced concrete building shown in Fig.12 has been employed in a dynamic time history analyses as part of the verification study. The building comprises of a podium (36.5m) and a tower (36m). A 600mm transfer plate is introduced at the interface between the podium and tower wherein some of the tower's gravity walls and columns are discontinued beyond the transfer floor level. The primary lateral load resisting system comprises of a continuous core spanning the full height of the building (see Fig. 12b). The numerical model of the building was constructed using the finite element program package ETABS (Habibullah, 1997). Artificial records were generated using SeismoArtif (SeismoSoft, 2007) to match the code spectrum recommended in the AS 1170.4 (2007) for three site classes A, C and D (refer to Table A-1 and Figs.A-1a-c in the Appendix). Details of the examined gravity walls (enclosed in a blue box in Fig.12b) are summarised in Table 1.



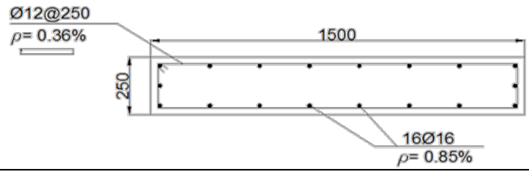
(a) 3D render of the FE model of the building



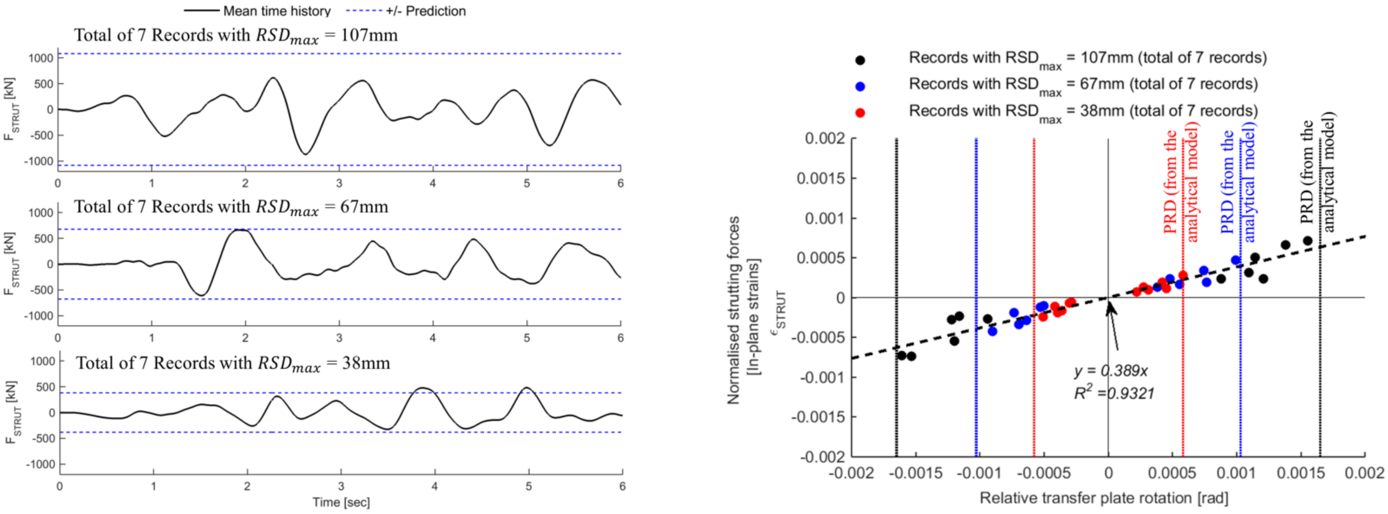
(b) Typical tower floor plan layout showing the planted gravity walls and the continuous core.

Figure 12. Case study building

Table 1: Design details of the examined tower walls

	$L \times t\ [mm]$	ρ_v[%	ρ_h[%]	f'_c [MPa]	Reinforcement Details
Tower gravity walls (wall 1,2 and 3)	1500×250	0.85%	0.36%	40	

The compatibility (strutting) forces in the slabs connecting wall 2 and the core were examined by integrating in-plane (shell) stresses along the length of the span. The product of the width of the column strip (defined by the extent of the negative-hogging- moment distribution) and the gross thickness of the slab have been used to calculate the in-plane strains in the floor slab. It is shown in Fig.13a that the predictive model (Eq. 3) based on the ground motion intensity (RSD_{max}) and the effective gross in-plane stiffness of the slab is capable of providing upper-bound estimates of these forces. Figure 13b plots the compatibility strains in the slab connecting wall 2 and the core along with the relative transfer plate rotation (at the base of wall2 and the core). The flexibility index was found to be 0.389 which is in good agreement with the value of 0.4 predicted using Fig. 11 (based on a value of $\alpha_r = 1.29$). The PRD calculated based on the analytical model (Eq. 1) is consistent with the maximum relative transfer plate rotation at the base of wall 2 (refer Fig. 13b). Shear force distributions up the height of wall 2 are shown in Fig. 14. The predicted value of the in-plane forces in the slabs are shown to result in high shear concentrations in the wall one storey above the transfer plate. The magnitude of the predicted forces (by employing Eq. 3) is also in good agreement with results obtained from the FE simulations (refer Fig. 14).



(a) Mean strutting forces in the slab connecting wall 2 and the continuous core

(b) flexibility index

Figure 13. Strutting force and relative transfer plate rotation (wall 2)

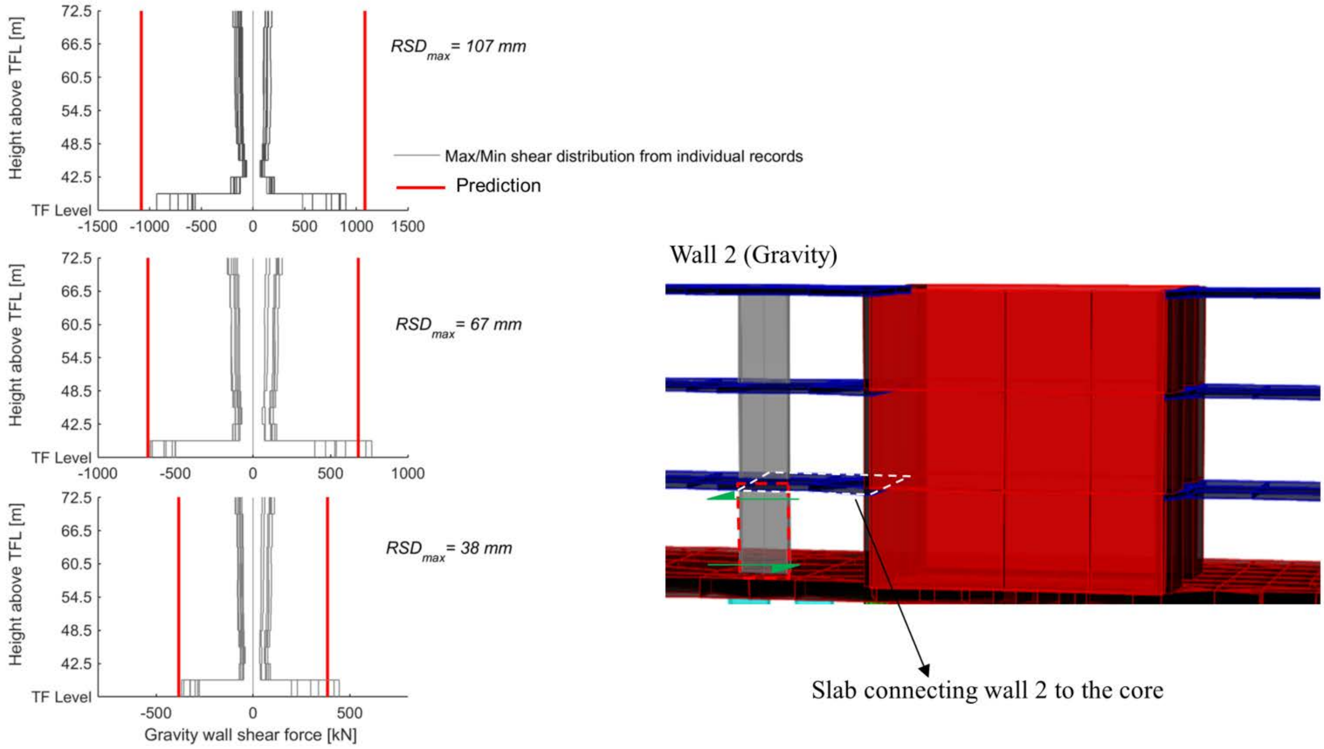


Figure 14. Shear force distributions on the planted wall (wall 2)

4 CONCLUSION

This paper sheds light on the unfavourable interference of the podium structure on the structural walls supporting the tower. It was found that podium can impose different boundary conditions on tower walls in relation to their proximity to the centre of the podium or the location along the supporting transfer plate. Slab-wall interactions in the form of strutting compatibility forces have been shown to be mobilised in the storeys immediately above interface level. These forces are the primary contributors to shear force distribution anomalies in the walls in the vicinity of the podium interface level. Analytical models have been proposed (and verified) to estimate these forces in order that a more accurate quantification of shear force demands can be established.

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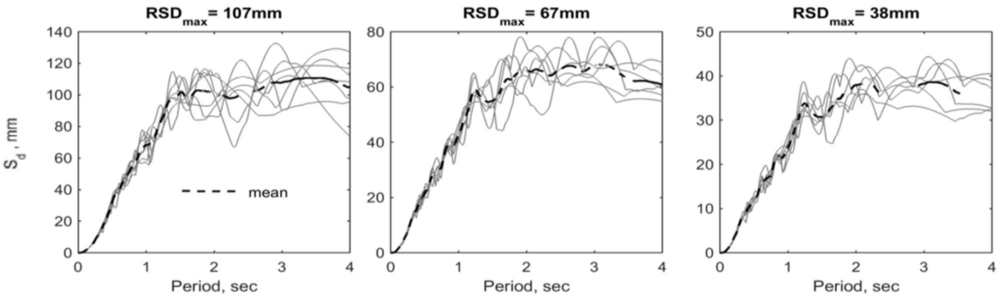
APPENDIX

Table A-1 Description of the accelerograms used in the study

Displacement spectrum	Record designation	Source
Figure A-1a	D-x	Synthetic code-compliant suite of records based on the response spectrum of the Australian Standard 1170.4(2007) for site class D (2% in 50 years)- SeismoArtif (SeismoSoft)

Figure A-1b	C-x	Synthetic code-compliant suite of records based on the response spectrum of the Australian Standard 1170.4 (2007) for site class C (2% in 50 years)- SeismoArtif (SeismoSoft)
Figure A-1c	A-x	Synthetic code-compliant suite of records based on the response spectrum of the Australian Standard 1170.4 (2007) for site class A (2% in 50 years)- SeismoArtif (SeismoSoft)



(a) D-x (b) C-x (c) -A-x

Figure A-1. Displacement spectra of records used in the study