16th World Conference on Earthquake, 16WCEE 2017
Santiago Chile, January 9th to 13th 2017
Paper N° 678 (Abstract ID)
Registration Code: S-Y1461886161

Wall-Slab-Gravity Columns Interaction in Multi-Storey Shear wall Buildings

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Abstract

In multi-story shear wall buildings with gravity columns and slabs, structural walls serve as the main lateral force resisting elements while frames are designed to resist only gravity loads which will have to ensure deformation compatibility with the lateral resisting system. It is common practice to assess the seismic performance of a whole structural system based on the seismic behavior of isolated structural walls mainly because slabs and gravity columns have low elastic lateral stiffness compared to structural walls and conducting 3D analysis of shear wall building system with floor slab and gravity system is tedious and demanding. This study aims to investigate the effect of strength and stiffness of slabs on seismic performance of the whole system and the structural wall itself. A parametric study with varying out-of-plane bending stiffness of slabs around the structural walls is conducted to assess the contribution of the slab-wall-gravity column interaction in the overall response of the whole system. These results demonstrate that concrete slabs increase the lateral strength of multi-storey shear wall buildings. However, this increase in lateral strength of the whole system is equivalent to the increase in shear forces on structural walls which may affect their failure modes or drift capacity.

Keywords: Seismic Performance; Shear Wall Buildings; Structural Wall; Shear Demand, Gravity Column, Slab

1. Introduction

Effect of roofing and flooring systems (such as cast in-situ slabs, beams or precast hollow cores) on seismic performance of multi-storey shear wall buildings should not be overlooked not only in seismic performance assessment of structural wall building systems but also in interpreting the results of experimental tests on structural walls. It is worthy to mention that the coupling effect of concrete slabs with structural walls (wall-slab interaction) can be noticeable in terms of increasing elastic stiffness and lateral strength of multi-storey shear wall buildings (Priestly 1995). On the other hand, design codes enforce formation of plastic hinge mechanism at the base of a structural wall by employing capacity design concept. The shear demand equivalent to the moment capacity of the base section can be used to design the structural wall for required shear strength. Some allowance should be also given for higher mode effects, flexural over strength and tension shift. Although framing actions between structural walls, slabs and gravity columns can increase the post-elastic lateral strength of the whole system, their implication on above explained capacity design of structural walls needs careful attention. This 3D spatial interaction will be discussed in more detail in the following sections.

Seismic performance assessment of multi-storey shear wall buildings with monolithic floor slabs and gravity columns can take large shear force (from what an isolated wall is expected) because of available over strength in the whole system. This benefit is not so obvious before significant yielding of the whole structure, but very prominent after effective yielding of structural walls as a lateral force resistant elements in the building.

A few studies have been conducted to explore the effect of a floor (such as in-situ slabs, beams or other roofing systems) stiffness or strength on the seismic behavior of multi-storey shear wall buildings. The effect of slabs on overall system behavior is more pronounced when structural walls enters into nonlinear response phase. Aktan et al. (1984) conducted earthquake simulation test on a 1/5 scale seven storey dual (wall-frame) system building to investigate the wall-floor-frame interaction following the full-scale test of the prototype structure in Japan (1984). The significance of the spatial interaction of structural walls with adjacent frames and slabs was highlighted in the above experiments. This interaction may also alter the structural wall lateral behavior itself. Recently shake-table test of a full-scale 7-storey building slice conducted under unidirectional earthquake motion at the University of California (Panagiotou (2010)).The experimental response demonstrated importance of three-dimensional interaction effects between rectangular wall, slotted slabs and gravity columns.

Therefore, the effect of wall-floor-gravity system interaction needs more investigation to highlight their effects on strength, stiffness and ductility of whole building especially when overall deformation pattern of the whole system is in the post-elastic range. This study intends to highlight the importance of slab, wall and gravity columns interaction and explains how this can affect the capacity design philosophy adopted in structural wall design or assessment. It is obvious that the analytical model with only isolated structural walls cannot capture the interaction effect explicitly. Moreover, linear analytical models are also not able to predict the abovementioned interaction and care must be taken to capture this spatial interaction in nonlinear analytical models. An appropriate nonlinear model is employed in this paper to conduct nonlinear analyses of different building systems with structural wall, slab and gravity columns. The nonlinear finite element model employed is found to predict the post-elastic behavior of the system reasonably well.

2. Effect of Slab and Gravity Columns on Wall Building Response

Gravity system and in-situ slabs in multi-storey shear wall buildings can increase the overturning moment capacity (due to interaction possibility) of the whole building. This interaction leads to not only increase in overall capacity of the whole building but also increase in shear demand induced at the base of structural walls. As soon as the wall edge start yielding, the upward movement of wall edges in tension side needs to keep deformation compatibility with adjacent slabs or beams connected to the wall in each storey of the building. This upward movement triggers out of plane stiffness of slabs or other flooring systems. The above deformation compatibility actions can provide extra lateral over strength for the whole system. Consequently, Increase in lateral strength of the whole system is equivalent to augment the shear demand at structural walls. This may raise some issues especially when isolated structural wall assessment or design is conducted without taking the effect of wall, slab and gravity columns interaction into account in nonlinear analytical models.

A typical multi-storey shear wall building with gravity columns and flat plate floor slabs has been used in this study (Figure 1). Even in the linear elastic analysis, as the out-of-plane stiffness of the diaphragm (slab) and axial stiffness of the columns (no flexural stiffness) increase the moments in the walls can decrease. Conversely, when there is negligible out of plane stiffness to the diaphragm, the moments in the walls would not be affected much (Figure 1). This interaction is an important source of over strength in the whole building when plastic hinge forms at the base of the wall. Further parametric studies are required to investigate this over strength in multi-storey shear wall buildings.

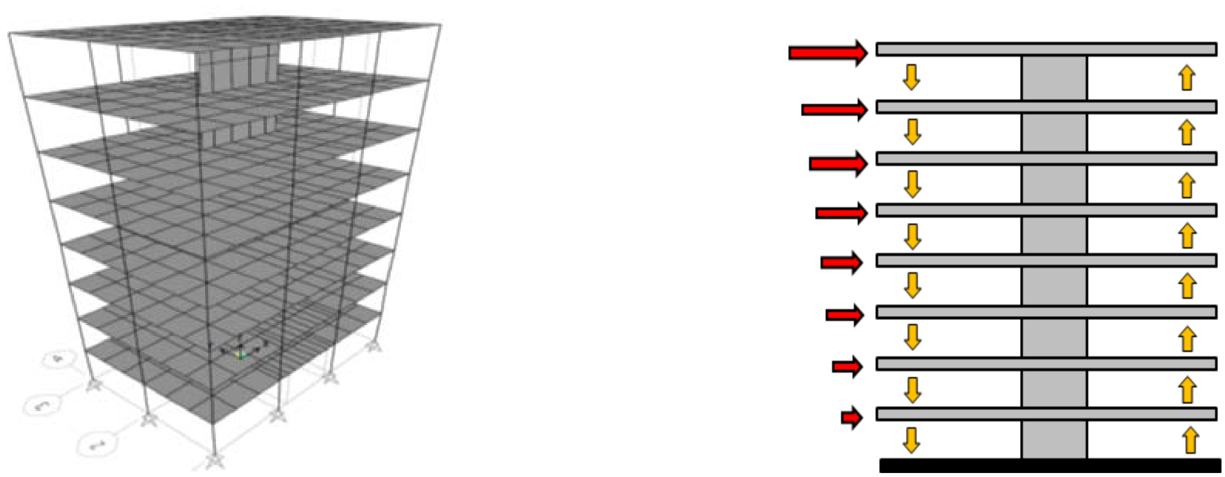


Figure 1. Typical Multi-story Shear Wall building with Slabs and Gravity Columns

In this study, the plastic mechanism of walls in a coupled system (structural wall, slabs and gravity columns) is expected to comprise only plastic hinges at the base of each wall. All other elements are assumed to remain elastic. Based on the probable flexural strength of the base sections of all walls of the system, the total overturning moment that can be sustained by these walls, M_{ow}, can be calculated. The total probable overturning moment capacity of the coupled system including gravity columns and slabs at the base is thus:

$M_{os} = M_{ow} + M_{oc} = M_{ow} + N_c*d$ as shown in Figure 2, where N_c is the additional axial force in gravity columns which is induced by framing action between a wall and d is the distance between the resultant tensile or compressive axial force induced on gravity columns under lateral action.

The framing actions induced by wall and slab interaction result in some over strength in the whole system. Therefore, the effective height of the wall system or shear span ratio as a center of lateral actions can be calculated as:

$h_{eff} = M_{ow}/V_b + N*d/V_b$

However, the effective height (center of lateral actions) of structural walls or the shear span ratio, which is used as a key parameter in the seismic assessment of shear wall buildings (NZSEE-2006), is commonly estimated as:

$h_{eff} = M_{ow}/V_b$

As it is apparent, this calculation neglects the effect of any probable framing action between the wall, slabs and gravity columns. It may assume very low elastic contribution of slabs and gravity columns in overall system behavior.

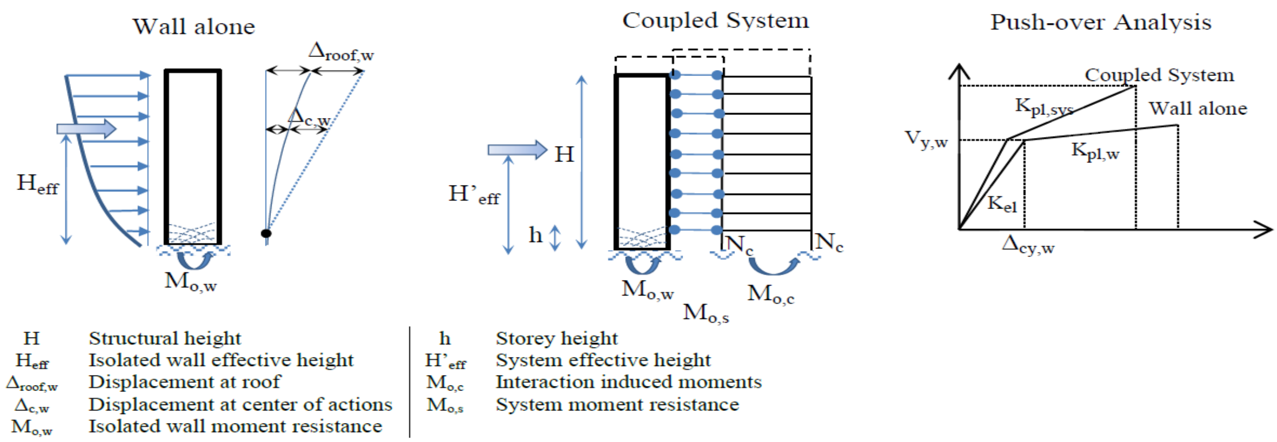


Figure 2. Schematic Illustration of Wall-Slab-Gravity System Interaction

For a given applied force pattern over the system height, tensile edge elongation or compression edge shortening of structural walls in post-elastic response trigger out of plane stiffness of slabs (or roofing system). This interaction induces significant additional axial forces (N_c) in the gravity columns, which can develop extra moment capacity in the system. As there is no shear resistant element in the different stories of the whole system except the wall sections, any additional shear force is required to be resisted by structural walls themselves. The implication is that, shear force demand at the structural wall in different stories will be increased. However, the amount of induced axial forces in gravity columns are more pronounced when wall elements start yielding (post elastic-range).

In this study, in order to address the net effect of framing action due to slabs, it is assumed that the gravity columns just have axial stiffness with very small flexural stiffness. Thus; they do not contribute to the shear strength of the whole system. In a ductile structure, roofing system (or floor slabs) will almost always be required to remain elastic, so that they can sustain their function of transferring forces to the main lateral-resisting structure, and tying the building together. Therefore, diaphragms (slabs) should in principle have the strength to sustain the maximum forces that may be induced in them for a chosen yielding mechanism within the rest of the structure.

On the other hand, in structural walls with relatively low shear span ratio (M/VL_w<3) estimating shear demand accurately is critical to calculate the shear capacity of structural walls. Therefore, it is obvious that any increase in shear demand not only may decrease the shear capacity of structural walls but also alter their failure modes or deformation capacities.

3. Design of Prototype System

The layout for the eight storey prototype building used herein is shown in Figure 3, with an assumed inter-storey height of 4.00 m. The prototype building has 18m x 30m dimensions in the plan with two rectangular shape structural walls in both directions (although not displayed in the Figure, in the perpendicular direction too, the same number of structural walls resist the lateral forces) .

This building is designed based on NZS 3101-2006 and NZS 1170.5-2004 provisions for soil class C. Lateral load resistance in the Y direction provided exclusively by two shear walls of 6m length and 400mm thickness. This building comprises monolithic floor slab connected to the structural walls through starter bars. The connections are assumed to be strong enough transfer the inertia force from the floor to the walls without any damage. Gravity columns have circular cross section with 500 mm diameter with 1.5 percent reinforcement. The building is regular and any possibility of torsional response being induced after one of the walls has yielded is ignored in the model. The detailed specification of this typical multi-storey shear wall building is presented in Figure 4.

It is worthy to mention that the fundamental period of this building was estimated to be 1.06 sec by eigenvalue analysis and it changes slightly by the inclusion of slab and gravity columns in the model.

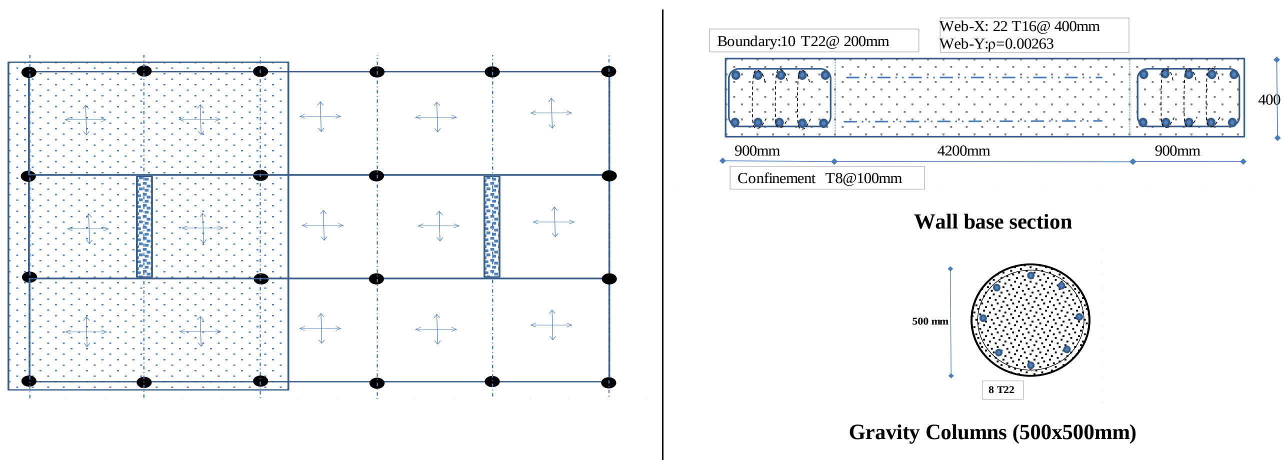


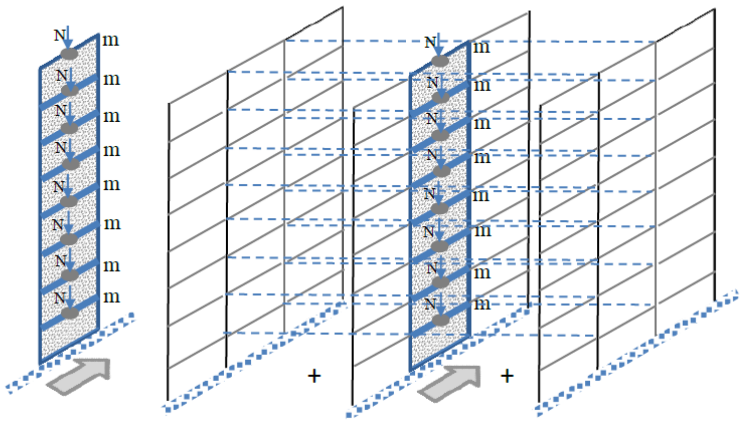
Figure 3. Prototype Building Plan

4. Analytical Modeling of the Isolated Wall

To estimate the global force displacement response of isolated structural walls by hand, moment curvature analysis is conducted and the moment curvature envelope is converted to the force displacement curve by assuming a plastic hinge length (Figure 5). The material models for confined and unconfined concrete in compression are based on the uniaxial hysteretic constitutive model developed by Mander et al. (1988) and any minor contribution of concrete in tension is neglected.

The stress-strain characteristics of reinforcement under monotonic loading are shown in Figure 6. Typical values are assumed in the analysis for the key modeling parameters of the model; such as strain hardening is assumed to start around ε_{sh} =0.088, the ultimate strain is assumed as ε_{su} =0.10 to 0.12, and the ratio of ultimate to yield stress is assumed as f_u/f_y =1.31. To account for the effects of cyclic loading, the moment curvature analysis is based on an ultimate strain limit of ε_s =0.6ε_{su}, as recommended by Priestley et al. (2007).

The prototype wall has relative large shear span ratio which implies that shear displacement contribution in displacement profile at the top is low and it can be neglected in calculating the total displacement. The shear capacity envelope is calculated based on the procedure proposed in Krolicki et al. (2011) and it is compared with the lateral force displacement curve to avoid any probability of post yield shear failure in the design of the prototype building.



Shear wall dimensions	32000 x 6000 x 400 mm
Typical floor height	4000 mm
F_c	30MPa wall and 40MPa Column
F_{yl} , F_{yh}	460MPa , 300MPa
Gravity columns dimension	D500 mm
Gravity load on each wall(Base)	$0.08\ f_cA_g$
Gravity load on each column (Base)	$0.5f_cA_g$-$0.13f_cA_g$
Soil Type	C
Near Fault Factor	1
Seismic weight on each wall(floor)	2100 kN
Slab Thickness(without beams)	150 mm
Zone factor	0.4
Return Period Factor	1.0
Structural Ductility	5
Soil Type	C

Figure 4. Prototype building specification and system interaction

Because, during ductile response of the system, walls are expected to remain essentially elastic above the plastic region at the base, their deformations will control that of the system response. The member deformation response is obtained by plastic hinge method proposed by Priestley et al. (1996). This method as shown in Figure 5 replaces the real curvature distribution with equivalent distribution in plastic hinge length to calculate the member displacement. Plastic hinge length needs to be assumed based on the available recommendation in literature. In this study, plastic hinge length is calculated as:

$L_P = k\,(L_{ss}) + 0.1 l_w + L_{sp} > 2L_{sp}$

$k = 0.2(f_u/f_y - 1) \leq 0.08$

and the strain penetration length recommended by Priestely et al. (2007) is given as:

$L_{sp} = 0.15 f_y d_{bl}$.

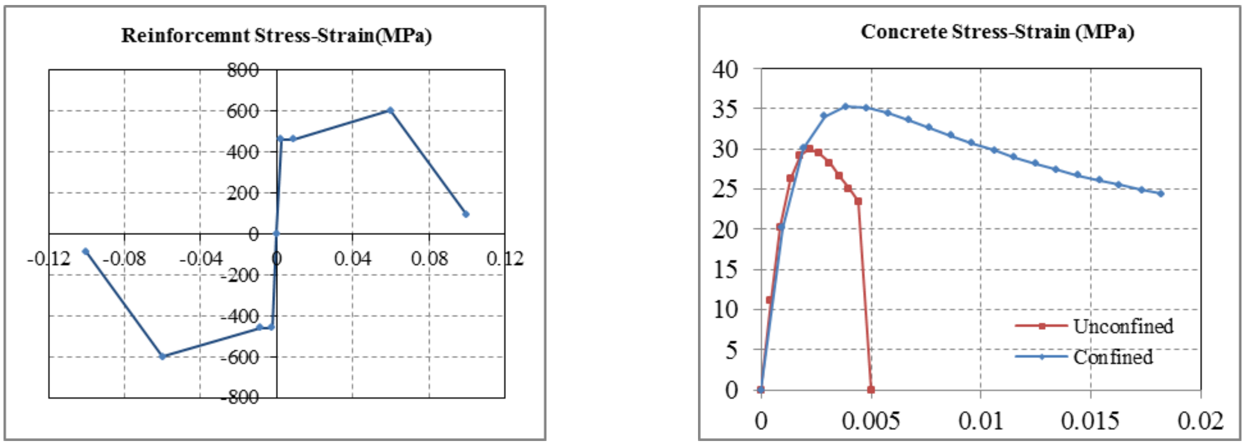
where l_w is the wall length, f_y is the yield strength and d_{bl} is the diameter of longitudinal reinforcement.

The member shear span L_{SS} is the distance from the point of maximum moment to the point of contra-flexure. Shear span can be calculated as the moment to shear ratio at the critical section; i.e. $L_{ss}=M/V=h_w$

Figure 5. Deformation response of an isolated wall

5. Finite Element Model

While slabs can provide significant in-plane stiffness, their out of plane stiffness governs the framing action between gravity columns and wall through slabs. Rigid diaphragm assumption is employed in the analysis which implies high in-plane stiffness for slabs. To diminish the effect of column flexural stiffness and strength contribution to the lateral capacity of the whole building, the columns are modeled as axial members with very low flexural stiffness (similar to axial springs). The initial axial forces in gravity columns due to gravity forces are high enough to prevent any net tension forces in these columns during analysis.



a) Rebar **b) Concrete**

Figure 6. Rebar and concrete backbone curve

Shell elements are employed in the nonlinear analyses using SAP2000. The shell element has six degrees of freedom at each node and an in-plane rotational degree of freedom. It should be mentioned that the shell element available in SAP2000 adopts a parabolic shape function to define the displacement field of the quadrilateral elements (Wilson (2002)).

The nonlinear multi-layer shell element available in this program can simulate the coupled in-plane/out-of-plane bending and the coupled in-plane bending/shear behavior of RC structural walls. The shell element is made up of many layers with different thickness. Different material properties can be assigned to various layers. This means that the reinforcement rebars are smeared into one layer or more. During the finite element calculation, the axial strain and curvature of the middle layer can be obtained in one element. Then according to the *plane section remains plane* assumption, the strains and the curvatures of the other layers can be calculated. And then the corresponding stresses are calculated through the constitutive relations of the material assigned to the layer.

Slabs can also be modeled using linear shell elements with different out-of-plane stiffness in both directions to reproduce the effect of stiffness of concrete slabs on the system response. Table 1 provides the assumed stiffness values for different elements used in the model to conduct nonlinear push-over analysis.

Table 1. Element Stiffness

Element	Axial stiffness	Flexural stiffness	In plane stiffness
Columns	E_cA_g	Very low	-
Slabs	-	$0.10E_cI$, $0.15E_cI$, $0.25E_cI$ In both directions	Rigid Diaphragm
Walls	Nonlinear Shell Element (Material Based)	Nonlinear Shell Element (Material Based)	Nonlinear Shell Element (Material Based)

Push-over analysis is the preferred tool for seismic performance evaluation of structures. It is allowed in most guidelines and codes because it is conceptually and computationally simple. Push-over analysis allows tracing the sequence of yielding and failure at member and structural levels as well as the progress of overall capacity curve of the structure. In a push-over analysis, a mathematical model of the building that includes all significant lateral force resisting members is subjected to a monotonically increasing invariant (or adaptive) lateral force (or displacement) pattern until a pre-determined target displacement is reached or the building undergoes a large enough strength degradation to indicate the failure of the system to sustain the gravity loads.

Conventional push-over analysis uses invariable force pattern during the analysis. Although some recent developments in push-over analysis have resulted in the introduction of modal push-over analysis (MPA) or adaptive displacement push-over (ADP) methods, they are not yet fully implemented in all commercially available software's. In this research, push-over analysis is conducted in two ways. In the first, the lateral force pattern was applied based on the first mode response of the structure, and the second approach employed linear force profile similar to the equivalent static force method. Displacement controlled push-over analysis is adopted to capture significant points of strength degradation in the system response. In the following sections, analysis results are illustrated only for the first method of push-over analysis.

6. Results

Moment-curvature analysis of the section is conducted with the prescribed stress-strain backbone curves for concrete and reinforcement, and the envelope curve is shown as solid line in Figure 7. Figure 7 also shows the calculated nominal yield curvature, ultimate curvature and hardening part based on bi-linearization procedure. The base section ultimate curvature capacity is estimated as 12.57 (1/Km). This is equivalent to ductility capacity of 23 based on proposed bi-linearization method (dashed line). The bi-linearized section response is adopted to obtain the force displacement curve of the structural wall. The analytical force displacement curve of the structural wall based on the section moment-curvature response is compared with the numerical force displacement curve obtained from push over analysis in Figure 8.

As illustrated in Figure 9, the base shear versus displacement at the roof of the structural wall in the whole system (including wall, slab and columns with very low flexural stiffness) and a system including wall only are almost identical until the point of significant yielding with coordinates (1046.5kN, 125mm) in push-over analysis; i.e. yielding of half of the reinforcement in the boundary zone. The nominal yielding force and its equivalent yielding displacement are obtained as (1223kN, 126mm) in this approach. The proximity of these two yielding points suggests that the employed finite element model is reliable. Table 2 compares the over strength induced by interaction between wall, slab and gravity columns in the whole system with the various assumption for bending stiffness of slabs (these values are indicated by the different value of EI in Figures). As Figure 9 illustrates, when slabs have no out of plane bending stiffness, it is obvious that no interaction exists between walls and gravity columns at all drift levels.

Different values recommended in available guidelines to estimate the out of plane stiffness of slabs to make some allowance for cracked concrete. However, the results indicate that even assuming only 0.10EI for bending stiffness of slabs cracked section (which is an extremely low value) in the two directions resulted in 23% and 40% increases in the overall base shear capacity of the whole system at 1 percent and 2 percent drift respectively compared to the isolated wall only (Figure 9). On the other hand, this considerable over strength in the system is equivalent to the same amount of increase in the base shear demand at the base of the structural wall. Currently, no allowance is made in codes to account for this effect when calculating shear force demand in design of structural walls.

To investigate the sensitivity of the results to the assumed stiffness of slabs, the value of 0.25EI (as recommended in ACI318-08) is used next for flexural stiffness of the slab cracked section in both directions. In this case, the interaction effect is more pronounced and the shear demand increase at the base of the wall is in order of 26% and 47 % at 1 percent and 2 percent drifts respectively (Figure 10). This interaction raises alarming concerns in terms of the extra shear force likely to be attracted by structural walls, which may cause the wall to fail in shear.

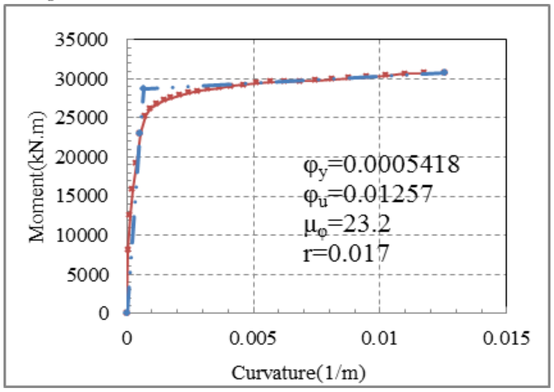


Figure 7. Moment-curvature response of the wall section

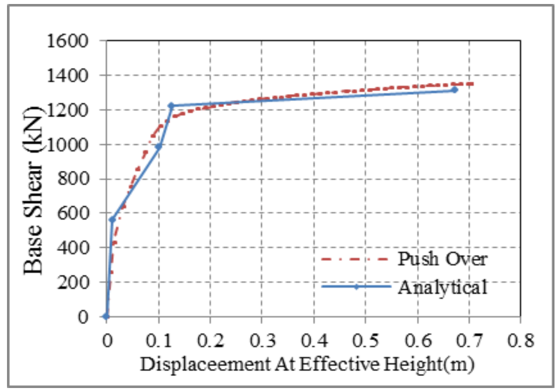


Figure 8. Force-displacement curve of the isolated wall

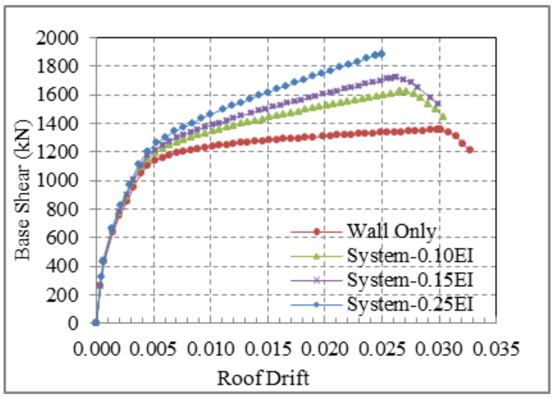


Figure 9. Base shear versus roof drift

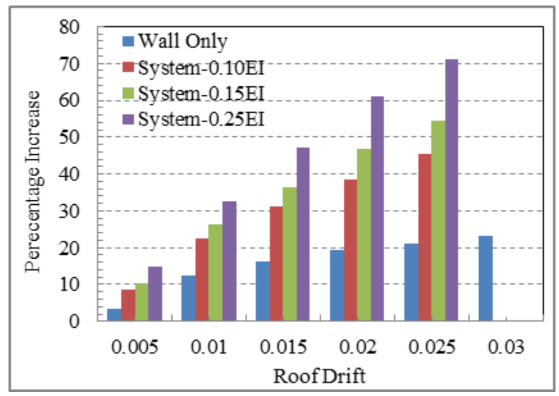


Figure 10. Increase in the system capacity

Table 2: Base shear at different drift levels(kN)

Structure	**Effective Yield force**		**Base Shear at 1% drift**		**Base Shear at 1.5% drift**		**Base Shear at 2.5 % drift**	
Wall Only	1226.8	1046.4	1230.0	1237.6	1238.0	1279.4	1250.0	1332.5
	Analytical	Finite Element	Analytical	Finite Element	Analytical	Finite Element	Analytical	Finite Element
Wall + Slab + Gravity Column (0.10EI)	1090.9		1345.6		1441.9		1600.8	
Wall + Slab + Gravity Column (0.15EI)	1108.9		1387.7		1602.4		1700.0	
Wall + Slab + Gravity Column (0.25EI)	1134.2		1457.0		1618.1		1872.6	

Table 2 lists the base shear values at 4 different drift levels in all systems investigated in this paper. It is evident that the system over strength is drift-dependent. Although one can overlook this system over strength in the design thinking it is conservative rather than unsafe, it could be critical in the capacity design of structural walls.

To highlight the importance of axial forces induced in columns due to edge extension of the structural wall, Figure 11 compares the additional moment capacity of whole system offered by the axial force in gravity columns at different drift levels. For example, the base moment only due to the induced axial force in columns increased from 1.27MN.m (system with zero out of plane stiffness of slabs) to 8.22MN.m (547 percent) at 1.5% drift in the case of the system with 0.25EI flexural stiffness compared with the case without any interaction. However, this additional base moment amounts to 21 percent increase (from 31.25MN.m to 37.66MN.m) in the total base moment of the whole system. The contribution of gravity columns axial forces on the base moment of the system increased up to 33 percent (from 32.61kN to 43.19kN) at 2.5% drift. The most important implication of above framing action in design is to ensure that shear demand equivalent to this moment receive the required attention.

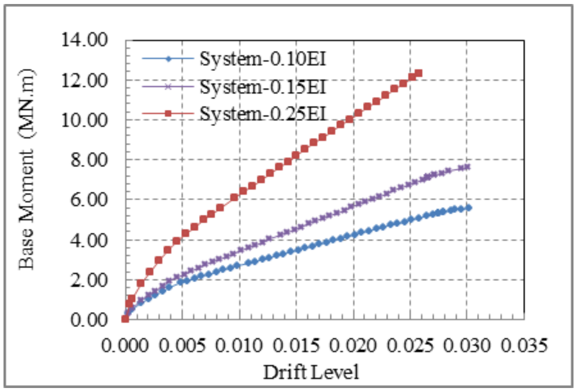


Figure 11. Base moment induced due to column axial forces

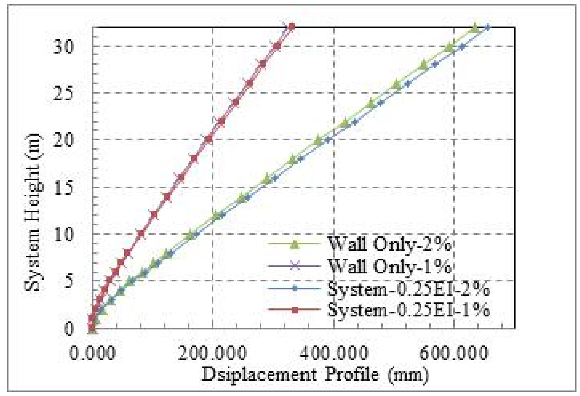


Figure 12. Displacement profile in different drifts

Inter-storey drift (IDR) profile and displacement profile of a building with nonlinear response are other key parameters in seismic performance evaluation. Deformation compatibility of gravity columns with the main lateral resisting system due to rigid diaphragm assumption, demands on drift-sensitive nonstructural elements and punching shear in column-slab connections should be evaluated based on the IDR profile. As Figure 12 shows, although ultimate displacement of the whole system undergoes a small change in structural ductility due to including slab and gravity columns interaction in the employed analytical model, the overall shape of displacement profiles at different drift levels are identical for an isolated wall and system.

Figure 13 and Figure 14 show the axial forces induced in one of the gravity columns and isolated wall respectively at 2% drift. It has been found that when interaction is included in the model axial force in gravity columns and individual wall augmented by 15% and 10% respectively compared to the system with no interaction. Thus, it is evident that one should ensure that gravity columns can accommodate this extra axial force due to wall-slab-column interaction.

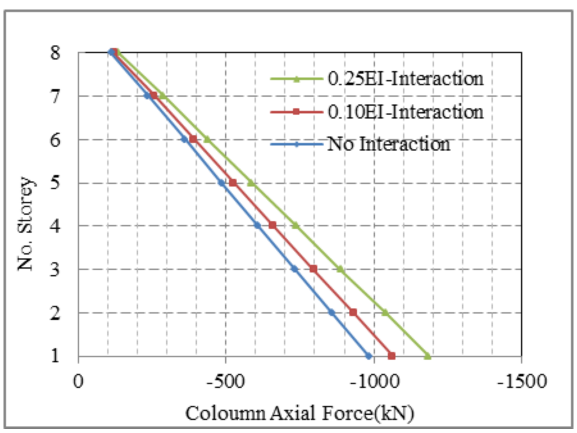


Figure 13. Axial force in a gravity column at 0.02 drifts

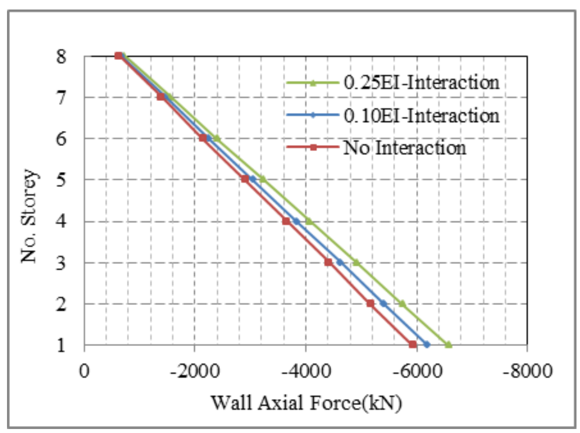


Figure 14. Axial Force in the Wall at 0.02 drift

Columns around the slabs perform as the boundary conditions of the whole system. The number of these columns along with their distance from the structural wall can change the amount of the system base shear. Due to space limitation, results of parametric investigation are presented only for a slab with flexural stiffness equal to 0.25EI at 2.5 percent drift. Figure 15 shows the variation in the increase in the normalized base shear of the system when the gravity columns distance in the X and Y direction changed respectively. The normalized base shear of the whole system reduced when the distance to the columns from the isolated wall increased. Figure 18 also illustrates the variation of normalized base shear of the whole system when the distance to columns changed in both directions simultaneously. It seems that the case with L_x=4 and L_y=4 offer 6 percent higher normalized base shear compare to case with L_x=4 and L_y=6. On the contrary, the case with L_X=8 and L_y=8 gives less normalized base shear compare with L_x=8 and L_y=6. The results are more sensitive to the number of gravity columns around the slabs.

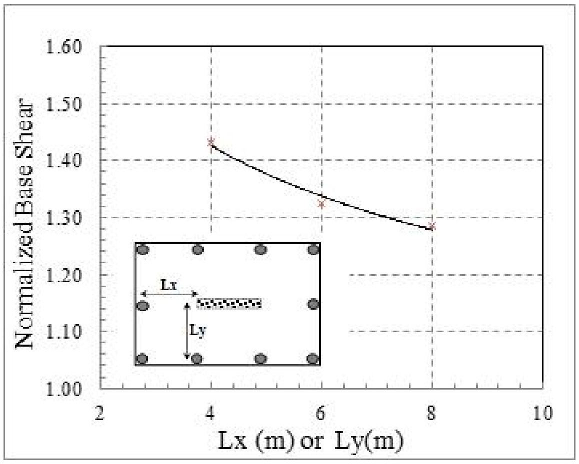


Figure 15. Normalizad base shear versus Lx **or** Ly

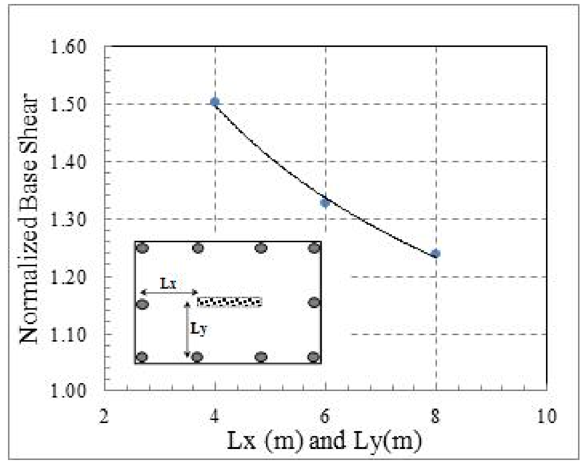


Figure 16. Normalizad base shear versus Lx **and** Ly

In a one-dimensional beam element, the shear force due to vertical displacement δ of one end while restraining other degrees of freedom is calculated as $V = 12EI/L^3 \times \delta$. In other words, axial force induced in gravity columns are best correlated with the inverse of the cube of the slab length. Note, however, that slabs deform in two directions simultaneously, not very similar to a beam element, a power law function has been chosen for regression analysis of results.

7. Conclusion

This paper has explored the effect of wall-slab-gravity system interaction on the overall behavior of shear wall building systems and isolated walls. The out of plane stiffness of slabs can induce some additional axial forces in gravity columns and this interaction can increase the moment capacity and the corresponding over strength of the whole structure. In capacity design philosophy, this over strength may affect the strength hierarchy of different failure modes of the structural walls mainly due to additional shear force demand induced in the different storeys of the structural walls. This system interaction effect requires additional allowance in base shear demand calculation and shear force envelope proposed for the structural walls.

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