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Seismic behavior of RC building by considering a model for shear wall-floor slab connections

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Abstract. Connections are the most important regions in a structural system especially for buildings in seismic zones. In R.C. structures due to large dimensions of members and lack of cognition of the stress distribution in a connection, reaching a comprehensive understanding of the connection behaviors becomes more complicated. The shear wall-to-floor slab connections in lateral load resisting systems have a potential weakness in transferring loads from slabs to shear walls which might change the path of load transformation to shear walls. This paper tries to investigate the effects of seismic load combinations on the behavior of slabs at their connection zones with the shear walls. These connection zones naturally are the most critical regions of the slabs in RC buildings. The investigation carried on in a simulated environment by considering three different structures with different shear wall layout. The final results of our study reveal that layout of shear walls in a building significantly affects the magnification of forces developed at the shear wall-floor slab connections.

Keywords: connection; floor slab; shear wall; FE method; seismic effects

1. Introduction

Understanding the behavior of shear wall-slab connections subjected to seismic loads is crucial for a building to safely pass any earthquake. In general, a shear wall-slab connection looks more like a beam-column connection which extended in one direction and one expects to observe almost the same behavior. Even though a relatively large number of experimental results are available on behavior of beam-column and slab-column connections but there is a lack of comprehensive tests on shear wall-floor slab connection.

Pantazopoulou and Imran (1992) reviewed few experimental tests carried out in Toronto and Lehigh universities in 80’s. They investigated those parameters that affect connection stiffness and shear resistance using experimental evidences and proposed a simple mechanical model. They concluded that the connections between floor slabs and shear walls constitute a potential weak link in structural lateral-force-resisting systems because of critical stress combinations that may

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develop in those regions during lateral sway. To avoid redistribution of forces from the walls to other elements which are not necessarily designed for lateral loads, the shear wall-floor slab connections must be designed to resist these stresses. Some of the existing experimental researches on behavior of shear wall-slab connections are listed in Table 1. All of these tests were arranged to examine the effects of seismic loads on the wall-slab connections.

2. Computer modeling

2.1 Definition of the problem

Table 1 Experimental researches on the RC wall-slab connections

University country (Year)	Researcher(s)	Characteristics of the specimen(s)	Type of loading	Observed behavior
Anna India (2011)	Greeshma *et al.*	Shear wall connected to slab in one face	Displacement controlled cyclic loading	Increasing ultimate strength observed in the specimen that crossed reinforcement used
Buet Bangladesh (1996)	Bari, Md. S.	Shear wall surrounded in three sides by slab	Vertical gravity and shear loads with increasing steps	The area around the wall nose is founded to be highly stressed, which is the critical area for punching failure too
Hokkaido Japan (1995)	Kudzys *et al.*	exterior and interior slab-wall connections	Vertical and reversal lateral out of plane forces	N/A
Torronto Canada (1990)	Imran, I.	Shear wall connected in both faces by slab	Monotonic and cyclic loading with and without gravity loads	Formation of flexural shear cracks in the slab at the connection with the wall
Lehigh U.S.A (1981)	Nakashima *et al.*	Coupled shear walls	Monotonically lateral forces and cyclic lateral load history	Formation of a major sliding crack, extended parallel to the wall at the junction boundary; development of a shear hinge & failure of the specimens by rapture of reinforcements in the critical region
Toronto Canada (1977)	Schwaighofer and Collins	Coupled in-line shear walls	vertical displacement by hydraulic jacks	Punching failure of the slab in the junction with the interior walls occurred; Punching failure of the exterior walls occurs after interior ones

For the safety of a structural system consists of shear walls, understanding the effects of seismic loads on shear wall-slab connections deserves some attention and cannot be neglected (see last column of Table 1 for more details). By assuming only gravity load combinations (GLC) in design process of slabs the seismic effects will be disregarded.

In this paper two different load combinations named GLC and SLC which defined by ACI 318 (2011) are used for analysis. Some effects of these load combinations on slabs and especially in regions with large stresses which specified by experiments (means floor slab-to-shear wall connections) are investigated. These two load combinations are as follows:

- GLC: 1.2DL + 1.6LL
- SLC: 1.2DL + LL ± EQ

To investigate the behavior of wall-slab connection three different structures as clearly defined in the following section are considered.

2.2 The structural plans and elements used for analysis

Three different structural plans with equal spans but different shear wall layouts are used (see Fig. 1). In Fig. 1(a), the shear wall layout is highly irregular and most of the shear walls are shifted to lower side of the plan. In the second plan shear walls are distributed over the plan but still irregular (Fig. 1(b)). The third plan shown in Fig. 1(c) is in the regular category so that the arrangement of shear walls provides central symmetry (center of mass, CM and center of rigidity, CR are coincided). All investigated models assumed to have 8 stories with 3 m in story height.

Each plan has 5 spans in X-direction and 4 spans in Y-direction. All spans are 4 m length in two directions for all models. CSI ETABS [8] program is used for analysis and design.

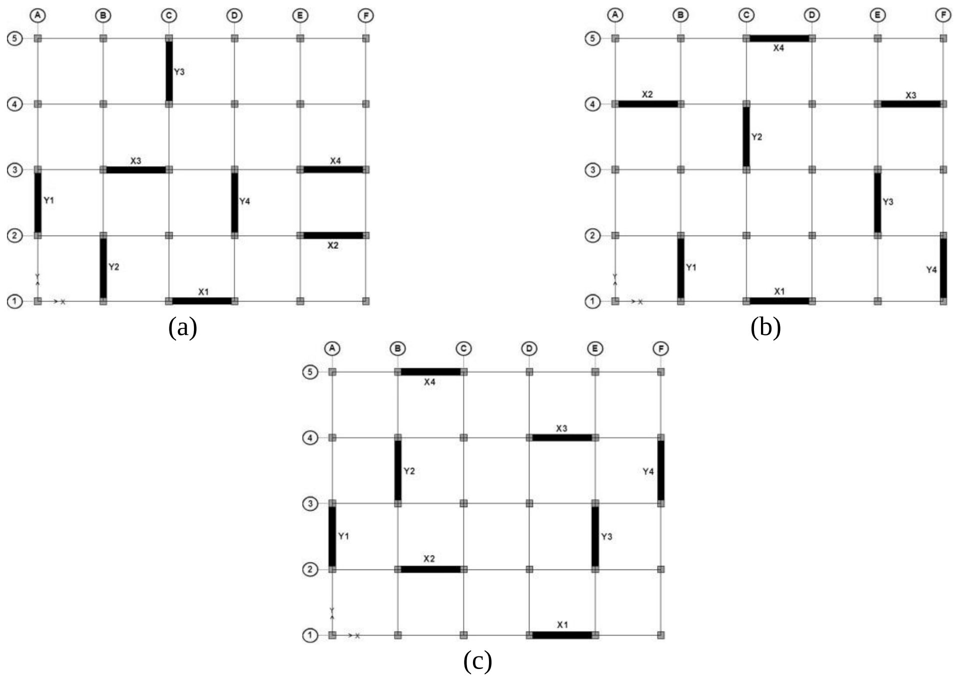

Fig. 1 Three considered shear wall layouts

The process that was followed in this paper can be summarized in three phases: (1) Analysis and design of structural members (with coarse meshing). (2) A more accurate analysis (with fine meshing). (3) Extracting and comparing the results from step (2). Each step explained in details in the following sections.

We will focus on the development of forces and cracks in the slab next to the wall; therefore correct modeling of these members such that to be able to extract slab behavior is crucial. To model slabs in ETABS it is a common practice to not considering their flexural stiffness by defining slabs as rigid diaphragms. It means that the out of plane stiffness of the slab is ignored. Recent researches revealed that this hypothesis is incorrect and provides not reliable results for steel distribution.

The floor stiffness reduces the total displacement of the stories and natural periods of the building. Employing inappropriate elements to model floor slabs yields unsafe and uneconomical design for slabs and shear walls. Therefore, we use “shell elements” to model the slabs rather than defining them as rigid diaphragms.

A shell element is a three-or four-node finite element formulation with 6 DOF’s per node which combines membrane and plate-bending behaviors. Employing the full shell behavior (membrane plus plate action) is recommended for all three dimensional structures by CSI Analysis Reference Manual as well.

3. Phase (1): analysis and design

In order to design structural members and determining steel ratio for each member of three considered structures with plans (a), (b) and (c) shown in Fig. 1, simple models have been constructed using Shell Elements for slabs and shear walls. In this phase, to grasp overall behavior of each structure while saving time, the models developed with coarse meshing, means each span for the slabs and shear walls divided to three equal parts. Each structure was analyzed using the design response spectrums presented in ASCE/SEI 7-10. The shear wall-slab connections considered as straight lines and did not model realistically. ETABS assumes shear walls intercept the slabs by straight lines although in reality their interfaces are planes. This assumption is a major drawback of commercial soft wares. In the following section this problem will be solved by a proposed innovative method.

4. Phase (2): more accurate analysis

4.1 Modeling the slab-wall connections

Reliable results to derive final conclusions need ac-curate and close to real modeling behavior. After fixing the major members’ section geometries (beams and columns) from the first phase, it is time to re-build the models with more accuracy. As mentioned in the previous section, ETABS is not able to explicitly consider the thickness of the plane members (like slabs and shear walls) in the analysis and this could affect the results.

To improve the accuracy and quality of the final results a new method in modeling of the slab-wall connections is proposed. In this method the connection of slab-to-wall (named panel zone; see Fig. 2(a)) is divided into four segments, two vertical segments that each one has the

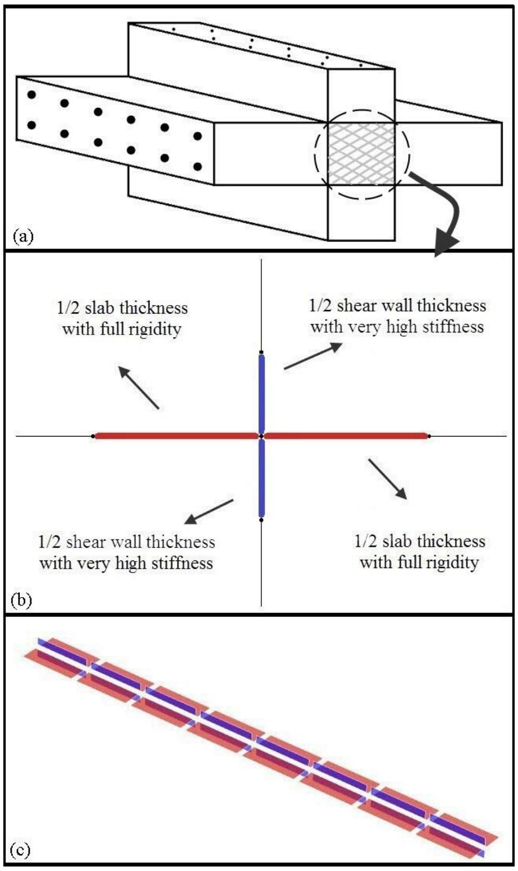

Fig. 2 Proposed model for slab-wall connection, (a) Shear wall-slab intersection region (panel zone), (b) Proposed model for the panel zone, (c) Shear wall-slab connection in 3D (one span with eight equal segments)

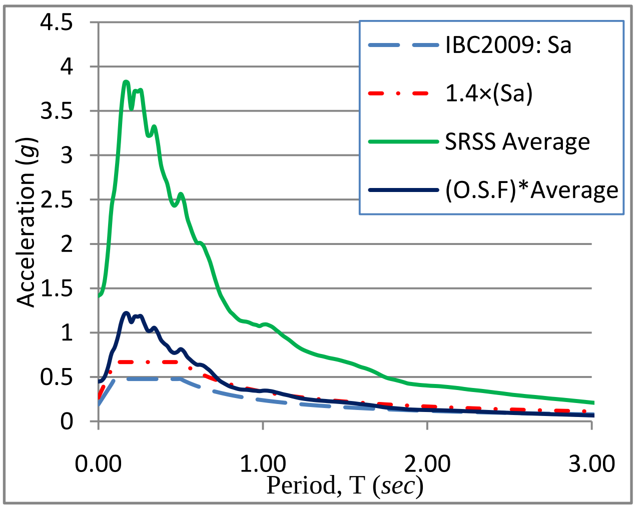

Fig. 3 Comparison of the standard spectrum of IBC2009 with the SRSS average of the 7 paired accelerograms to determine the O.S.F.

length equals to half of the connecting shear wall thickness and two horizontal segments each one has the length equals to half of the corresponding slab thickness. Because the slabs and the shear walls have significant stiffness in their own planes (relative to other members) their connection (slab-wall panel zone) has rigid behavior. So the horizontal segments modeled by rigid slabs and vertical segments modeled by introducing higher order stiffness coefficients. Fig. 2(b) clearly demonstrates this proposed method. It should be mentioned that other regions of the slabs and the shear walls were modeled as usual. To reach more accuracy in this phase, fine meshing (0.5 m×0.5 m mesh sizes) for plane members used. It means each span is divided into eight equal segments. Fig. 2(c) shows the entire connection in 3-D (other details are deleted for clarity). For modeling the panel zone using our proposed method, one of the horizontal or vertical segments should be assigned with zero weight concrete to avoid considering weight of the panel zone twice.

4.2 Type of analysis

After rebuilding all of the models with known sections and applying the aforementioned changes, the modified structures must be analyzed. Since a time history (T.H.) analysis is the most accurate analysis method accepted by all design specifications we will adopt T.H. analysis. T.H. analysis can be carried out by either: (a) using three pairs of accelerograms and getting maximum responses (each pair contains two accelerograms in two perpendicular directions); or (b) using seven pairs of accelerograms and computing the average for a specific response. In this research both methods are employed and the results compared to each other. But first the accelerograms must be normalized according to a specific algorithm presented in IBC 2009. The overall scale factor (O.S.F) calculated by this algorithm for 7 selected pair accelerograms is 0.317 (see Fig. 3). Table 2 contains some information about these 7 paired accelerograms which selected from Peer Ground Motion Database (PGMD) website. The last column of Table 2 shows the final scale factors which should be multiplied by the accelerograms to be use in ETABS for time history analysis.

Table 2 PGMD earthquakes and their information contents

	Earthquake	Year	Direction	P.G.A	Significant duration (Sec)	Final scale factor
1	Chi Chi, Taiwan	1999	1	0.30	30.2	1.052
			2	0.64	22.1	0.497
2	Friuli, Italy	1976	1	0.35	4.2	0.904
			2	0.31	4.9	1.009
3	Hector Mine, USA	1999	1	0.27	11.7	1.196
			2	0.34	9.7	0.943
4	Loma Prieta, USA	1989	1	0.45	9.5	0.705
			2	0.39	9.7	0.805
5	Manjil, Iran	1990	1	0.51	28.9	0.617
			2	0.50	30.6	0.640
6	Northridge,USA	1994	1	0.28	11.3	1.143
			2	0.47	10.2	0.670
7	San Fernando, USA	1971	1	0.37	10.7	0.868
			2	0.28	11.9	1.123

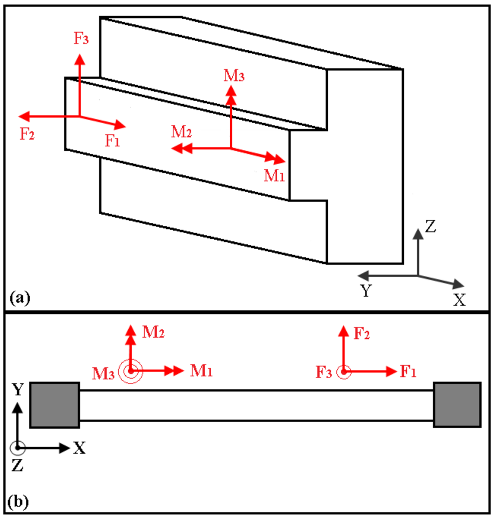

Fig. 4 Sign conventions used for forces and moments in the section cuts in the slab near the shear wall-slab connections (compatible with the global coordinate system of ETABS) (a) 3-D view, (b) Top view

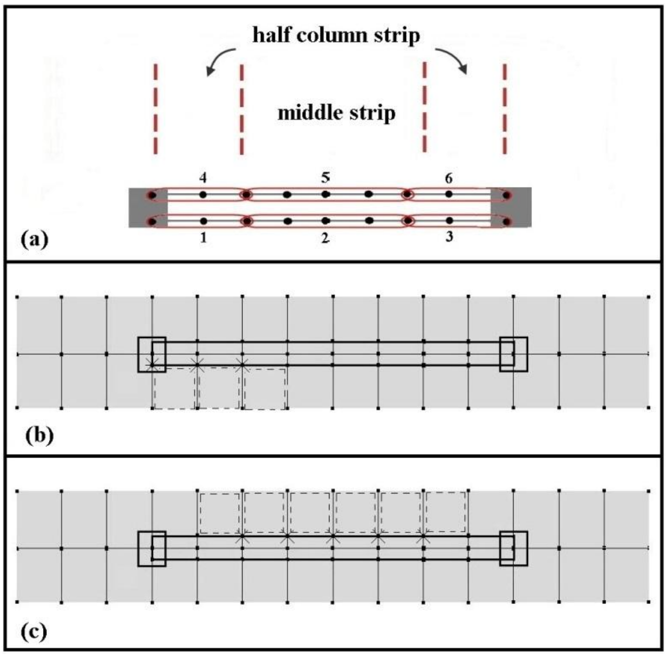

Fig. 5 Locations of section cuts in the slab around a shear wall-slab connection: (a) schematic view, (b) an example of ETABS model (section cut No.1 and its corresponding components), (c) section cut No.5 and its corresponding components

4.3 Section cut tool in ETABS

The main objective of this paper is to determine forces and moments transferred from the slabs to shear walls through their connections. This goal is achieved by employing “Section Cut” tool provided in ETABS. Each section cut contains some nodes and their corresponding shell elements. After each analysis, three forces (F1, F2 and F3) and three moments (M1, M2 and M3) computed at the center of each section cut and reported. The sign convention for these forces and moments are compatible with the global coordinate system of ETABS (see Fig. 4).

We propose to select locations of the section cuts in comply with the design process of a slab. In design process, a slab in each span is divided into three segments; the half middle length of a span called middle strip and two ¼ lengths on each side of the middle strip called column strips (see Fig. 5(a)). So, six section cuts around each shear wall-slab connection created as shown in Fig. 5(a). Two sample section cuts presented in Figs. 5(b) and 5(c) that depict the real section cuts and their components in ETABS models used in this paper. Section cuts No.1, 3, 4 and 6 refer to column strips and section cuts No.2 and 5 refer to middle strips.

5. Sensitivity analysis: time step in T.H. analysis

An influential parameter that significantly affects the accuracy of results computed by T.H. analysis is ΔT. A small ΔT increases the computational cost while a large ΔT decreases accuracy of the results. Therefore, determining an appropriate magnitude for time step is important. A suitable time step is found by try and adjusts method, and the results are presented in Table 3. Three amounts for ΔT were considered: 0.005, 0.02 and 0.1 seconds. The maximum difference accepted between the results was 5%. The results revealed that ΔT=0.02 seconds is relatively appropriate for T.H. analysis (see Table 3).

6. Phase (3): extracting final results

As mentioned before for T.H. analysis we have investigated two methods in this paper; (a)

Table 3 Results of analyzes using different time steps (ΔT) (time history analysis, Plan No.1)

Parameters	ΔT (Seconds)		Difference (%)	ΔT (Seconds)		Difference (%)
	0.02	0.005		0.02	0.1	
Base Shear (Ton)	431.101	439.796	2	431.101	404.896	6.1
Top Story Drift X	0.00134	0.00134	0	0.00134	0.00125	6.3
F3 (S3-X1-5)*	1.69857	1.70714	0.5	1.69857	1.62857	4.1
M1(S3-X1-5)*	3.58136	3.60093	0.5	3.58136	3.40443	4.9

*Section cut No. 5 for shear wall X1 in story 3 identified as: S3-X1-5

Table 4 Earthquake groups used in T.H. analysis

Group No.	Earthquakes		
G1	Chi Chi	Manjil	Northridge
G2	Hectormine	Loma Prieta	San Fernando
G3	Loma Prieta	Manjil	San Fernando

using three pairs of earthquake's accelerograms and getting the maximum responses; (b) using seven pairs of earthquake's accelerograms (Table 2) and computing the average responses. We selected three different groups of three earthquake pairs provided in Table 2 to obviate the unreasonable effects of one or more severe earthquakes solitarily in the final results. These triple groups of earthquakes are listed in Table 4.

6.1 Combined effects of shear and torsion

ACI318-11 considers the simultaneous effects of shear and torsion by satisfying the following provision (ACI 318M-11 Eqs.11-18).

$$\sqrt{\left(\frac{V_u}{b_w d}\right)^2 + \left(\frac{T_u P_h}{1.7A_{oh}^2}\right)^2} \le \phi\left(\frac{V_c}{b_w d} + 0.66\sqrt{f_c^{'}}\right) \tag{1}$$

where

$$V_c = 0.17\left(1 + \frac{N_u}{14A_g}\right)\sqrt{f_c^{'}}\, b_w d \tag{2}$$

In this equations Nu refers to F2 and Vu and Tu refer to F3 and M2, respectively in the defined section cuts. So, we rewrite Eq. (1) as a shear demand-capacity ratio (shear D.C. ratio) and denote it by αshear which is a dimensionless quantity as follows.

$$\alpha_{shear} = \frac{\sqrt{\left(\frac{V_u}{b_w d}\right)^2 + \left(\frac{T_u P_h}{1.7A_{oh}^2}\right)^2}}{\phi\left(\frac{V_c}{b_w d} + 0.66\sqrt{f_c^{'}}\right)} \le 1 \tag{3}$$

In this formula T_u is the torsional moment at the considered section and its associated term in Eq. (3) reflects the torsional moment effects. The term $0.66\sqrt{f_c^{'}}$ represents the shear reinforcement effects. At last, b_w and d are width and depth of the connection respectively also T_u and A_{oh} are perimeter and area enclosed by centerline of the outermost closed transverse torsional reinforcement respectively. In the following section, we will define three different scenarios for Eq. (3). In each scenario one or more of the terms in Eq. (3) omitted in order to investigate different cases could be considered in design mode. These scenarios are the reflection of the design philosophy followed by the designer.

6.1.1 Scenario No.1

In this scenario Eq. (3) is used with all terms. This means that the designer has considered effects of the torsion in the slab design and embedded extra shear reinforcement in the shear wall-slab connection, although in practice this assumption is not applicable. In Table 5 and Fig. (6) to facilitate the comparison the computed values for α_{shear} due to SLC's are normalized by corresponding values of α_{shear} for GLC.

Table 5 Values of relative αshear for Scenario No. 1

Type of analysis			Model No.1		Model No.2		Model No.3	
			αshear (SLC/GLC)	Diff. (%)	αshear (SLC/GLC)	Diff. (%)	αshear (SLC/GLC)	Diff. (%)
SLC (T.H.)	7 Pairs		1.61	60.78	1.97	96.60	1.49	49.12
	3 Pairs	G1	2.35	135.10	2.55	154.71	1.75	74.93
		G2	2.21	120.81	2.93	193.14	1.79	78.90
		G3	2.35	135.10	2.07	106.86	1.73	72.74

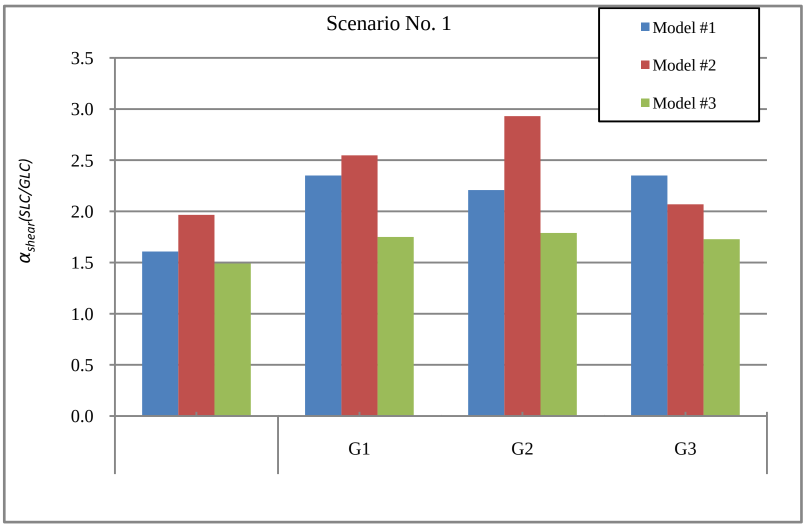

Fig. 6 Relative values of αshear (SLC/GLC) for three models in Scenario No. 1

Table 6 Values of relative α_{shear} for Scenario No. 2

Type of analysis			Model No.1		Model No.2		Model No.3	
			αshear (SLC/GLC)	Diff. (%)	αshear (SLC/GLC)	Diff. (%)	αshear (SLC/GLC)	Diff. (%)
SLC (T.H.)	7 Pairs		1.59	58.87	1.96	95.96	1.49	49.23
	3 Pairs	G1	2.31	131.18	2.51	150.88	1.75	74.74
		G2	2.18	118.49	2.88	187.95	1.79	78.68
		G3	2.31	131.18	2.03	103.12	1.73	72.53

6.1.2 Scenario No.2

In this scenario, term $0.66\sqrt{f_c^{'}}$ is omitted from Eq. (3) so it converted to Eqs. (4) or (5) in dimensionless form which means it is assumed that the slab does not require any shear reinforcement in the design process. This assumption is very close to reality because in the

Table 7 Values of relative α_{shear} for Scenario No. 3

Type of analysis			Model No.1		Model No.2		Model No.3	
			αshear (SLC/GLC)	Diff. (%)	αshear (SLC/GLC)	Diff. (%)	αshear (SLC/GLC)	Diff. (%)
SLC (T.H.)	7 Pairs		1.42	41.60	1.01	0.88	0.90	-9.84
	3 Pairs	G1	2.12	111.83	1.25	25.01	1.04	4.32
		G2	2.00	100.30	1.29	29.26	1.07	7.30
		G3	2.12	111.83	1.07	7.19	1.03	2.83

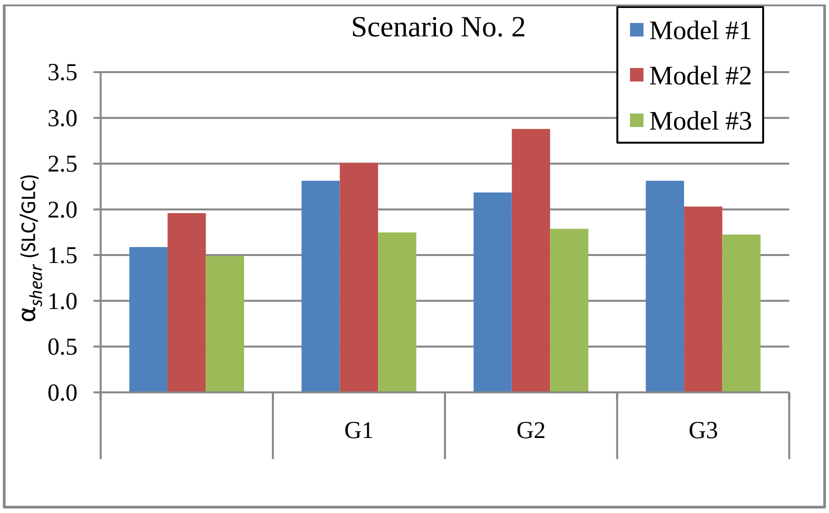

Fig. 7 Relative values of αshear (SLC/GLC) for three models in Scenario No. 2

common design process of the slabs almost always shear reinforcement will not be provided. In Tables 6 and 7 and Fig. 7 the relative values of αshear computed for various models are presented. The results shows the same trends as mentioned for scenario No. 1.

$$\sqrt{\left(\frac{F_3}{b_w d}\right)^2 + \left(\frac{M_2 P_h}{A_{oh}^2}\right)^2} \leq \phi\left(\frac{V_c}{b_w d}\right) \tag{4}$$

$$\alpha_{shear} = \frac{\sqrt{\left(\frac{F_3}{b_w d}\right)^2 + \left(\frac{M_2 P_h}{A_{oh}^2}\right)^2}}{\phi\left(\frac{V_c}{b_w d}\right)} \leq 1 \tag{5}$$

In this scenario SLC's magnifies the values of αshear at least 50% in model No. 3 and at most 200% for model No. 2 with respect to GLC.

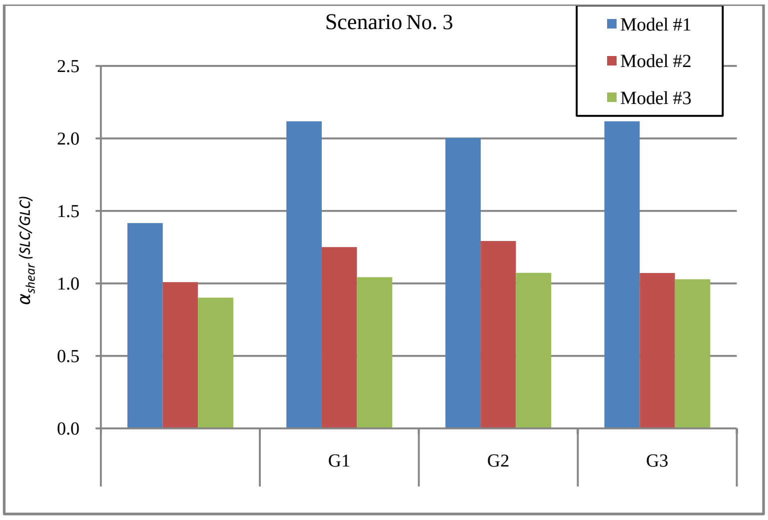

Fig. 8 Relative values of αshear (SLC/GLC) for three models in Scenario No.3

6.1.3 Scenario No.3

If the designer neglects both effects of torsional moment $(T_u P_h / 1.7A_{oh}^2)^2$ and shear reinforcement, $0.66\sqrt{f_c'}$ from both sides of Eq. (3) the equation converted to a simple shear D.C ratio as in Eq. (6).

$$F_3 \le \phi V_C \rightarrow \alpha_{shear} = \frac{F_3}{\phi V_c} \le 1 \tag{6}$$

In this scenario the results shows that 7 pairs T.H. analysis is conservative compares to results from GLC for model No. 2 and 3 and more regularity in the layout of shear walls decreases the difference between SLC and GLC results, Fig. 8.

6.2 Reinforcement ratios

In this section we compare the reinforcement ratios (ρ) required for different load combinations. After analyzing and deriving the final forces and moments at the shear wall-slab connections for each model, the slabs designed by "direct design method" and corresponding reinforcement ratios determined. In Tables 8 to 13 amounts of ρ for each model classified by column strip and middle strip presented. Since the slab thickness in all stories is the same the maximum value of the bending moment (M1 in each section cut) considered for each model.

In model No. 1 there are at most 80% increase in the reinforcement ratios in column strips and 38% in middle strips by using SLC instead of GLC. The reinforcement ratios especially for middle strips exceed ACI specification.

The same trend as defined for model No. 1 can be observed for Model No. 2 but in middle strips the reinforcement rations are slightly higher.

As can be seen in Tables 12 and 13 the differences between reinforcement ratios due to SLC's and GLC decreased significantly for as for 7 pair analysis the R.R. values are fewer than GLC.

Table 8 Reinforcement ratios in column strip for model No. 1

Column strip	Max M1 (kN-m)	ρ	Difference (%)
GLC	68	0.0055	-
SLC-7Pairs	76	0.0061	12.2
SLC-3Pairs-G1	117	0.0098	79.8
SLC-3Pairs-G2	101	0.0083	52.3
SLC-3Pairs-G3	117	0.0098	79.8

Table 9 Reinforcement ratios in middle strip for model No. 1

Middle strip	Max M1 (kN-m)	ρ	Difference (%)
GLC	120	0.0096	-
SLC-7Pairs	125	0.0100	4.4
SLC-3Pairs-G1	162	0.0132	37.5
SLC-3Pairs-G2	153	0.0123	28.8
SLC-3Pairs-G3	162	0.0132	37.5

Table 10 Reinforcement ratios in column strip for model No. 2

Column strip	Max M1 (kN-m)	ρ	Difference (%)
GLC	55	0.0044	-
SLC-7Pairs	70	0.0056	27.3
SLC-3Pairs-G1	91	0.0075	70.7
SLC-3Pairs-G2	89	0.0073	65.9
SLC-3Pairs-G3	76	0.0061	39.4

Table 11 Reinforcement ratios in middle strip for model No. 2

Middle strip	Max M1 (kN-m)	ρ	Difference (%)
GLC	100	0.0079	
SLC-7Pairs	120	0.0095	20.8
SLC-3Pairs-G1	153	0.0123	56.4
SLC-3Pairs-G2	152	0.0122	55.2
SLC-3Pairs-G3	133	0.0107	35.4

Table 12 Reinforcement ratios in column strip for model No. 3

Column strip	Max M1 (kN-m)	ρ	Difference (%)
GLC	69	0.0056	-
SLC-7Pairs	66	0.0053	-5.1
SLC-3Pairs-G1	76	0.0062	10.9
SLC-3Pairs-G2	77	0.0062	11.6
SLC-3Pairs-G3	77	0.0062	11.6

Table 13 Reinforcement ratios in middle strip for model No. 3

Middle strip	Max M1 (kN-m)	ρ	Difference (%)
GLC	116	0.0092	-
SLC-7Pairs	105	0.0083	-10
SLC-3Pairs-G1	121	0.0096	4.6
SLC-3Pairs-G2	123	0.0098	7
SLC-3Pairs-G3	120	0.0095	3.6

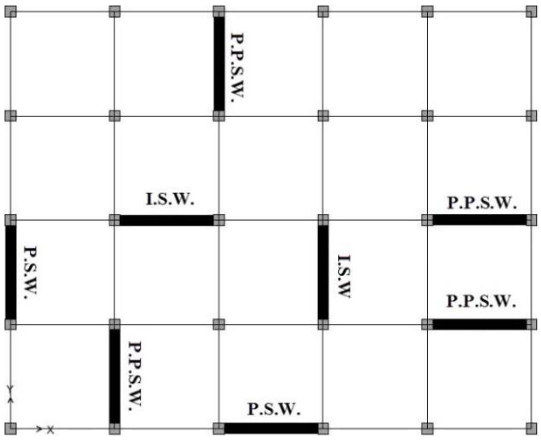

Fig. 9 Various shear wall positions

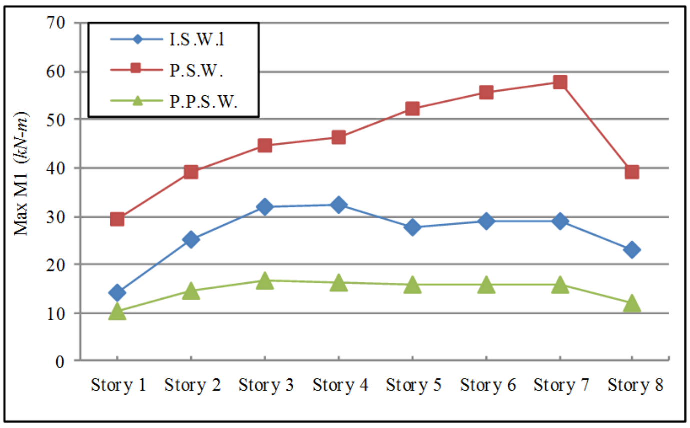

Fig. 10 Maximum values of M1 in 'SLC-3Pairs-G1' for model No. 1

6.3 Effect of shear walls location

Now we investigate how the location of a shear wall can affect the final forces and moments developed at the panel zone of a shear wall-slab connection. For this purpose all locations of shear walls in a plan categorized into three groups; (i) peripheral shear wall (P.S.W.), (ii) perpendicular to periphery shear wall (P.P.S.W.) and (iii) internal shear wall (I.S.W.) this positions depicted on

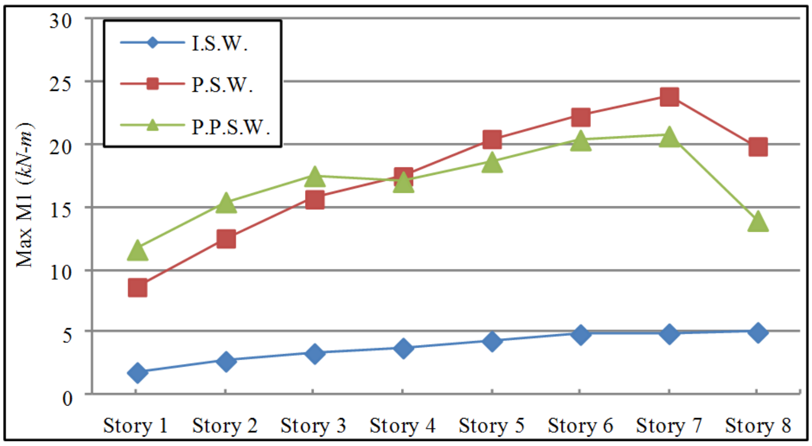

Fig. 11 Maximum values of M1 in 'SLC-3Pairs-G1' for model No. 2

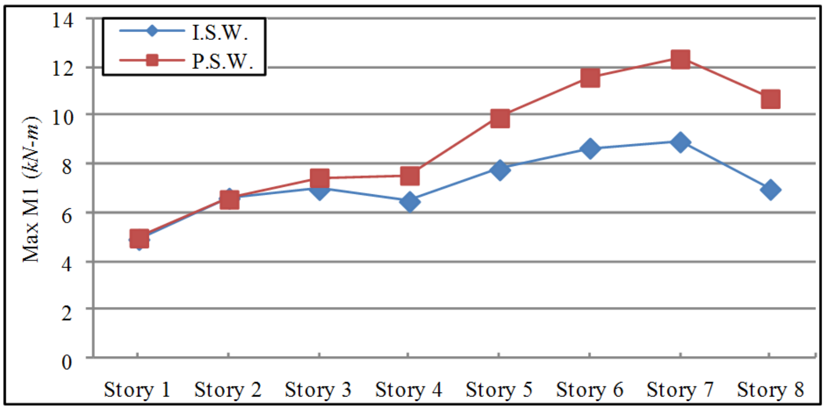

Fig. 12 Maximum values of M1 in 'SLC-3Pairs-G1' for model No. 3

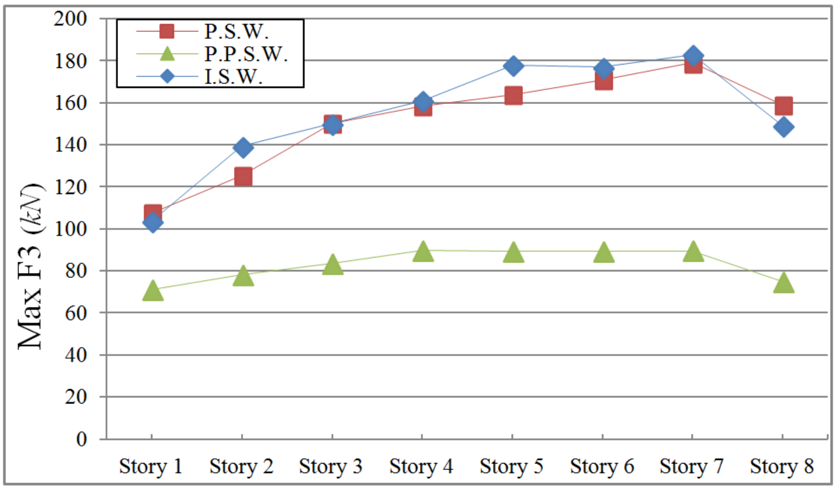

Fig. 13 Maximum values of F3 in 'SLC-3Pairs-G1' for model No. 1

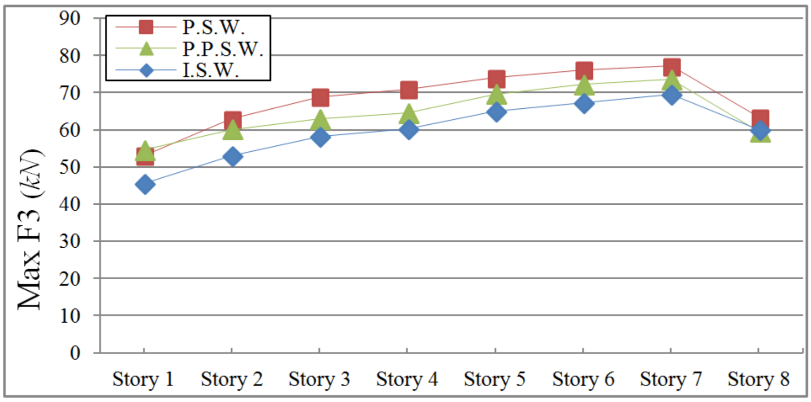

Fig. 14 Maximum values of F3 in ‘SLC-3Pairs-G1’ for model No. 2

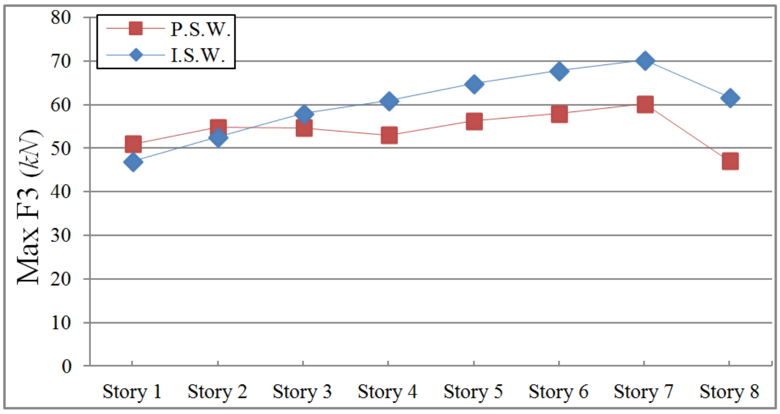

Fig. 15 Maximum values of F3 in ‘SLC-3Pairs-G1’ for model No. 3

plan No. 1 in Fig. 9. It’s should be mentioned that model No. 3 does not have any P.P.S.W. type.

Now we can compare the section cut forces for various positions by each load combination and analysis type.

In Fig. 10 through Fig. 12 the maximum values of bending moment (M1) compared for each model. In Fig. 13 through Fig. 15 maximum values of shear forces (F3) are compared (all of these values derived for ‘SLC-3Pairs-G1’ analysis).

As can be seen in Fig. 13 through Fig. 15 P.S.W. location type absorbs more moments than the other types of shear wall positions. In shear forces absorption the I.S.W. and P.S.W. are more critical.

7. Conclusions

To overcome the shortcoming of commercial soft wares in modeling planar members a new procedure is proposed for modeling the slab-to-wall panel zone.

To determine the effects of lateral loads on forces transferred through slabs to shear walls in their connection regions two parameters are investigated: (i) different load combinations; (ii) different shear wall layouts.

• The final results showed that those load combinations that include lateral forces (SLC) increase the forces and moments transferred between slabs and shear walls. In the common design process of the slabs the effects of magnified forces and moments due to earthquake loads are ignored, although the magnification of forces is not significant for a plan with regular layout of shear walls. But the location of a shear wall in the plan could be very influential. Results revealed that for shear walls placed on the exterior side of a plan (which are very prevalent in RC buildings because of architectural limitations) the magnifications of forces is more severe compared to the magnified forces in interior shear walls.

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