SEISMIC RESPONSE EVALUATION OF FLAT PLATE FLOOR SYSTEM AND ITS COUPLING BEAM USING NON-LINEAR DYNAMIC ANALYSIS

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ABSTRACT

Flat plate slab systems are popular all over the world because of its aesthetics and improved facility to run utility services though they possess poor mechanisms against seismic forces. In a view to improving the lateral force-resisting capability, coupled shear-wall is often preferred with the flat plate system. The behavior of coupled shear walls as well as the global structure is largely influenced by the coupling beams, particularly against earthquake-induced ground motions. In this study, a 15 storied building of a flat slab structural system containing coupled shear-wall is investigated. A series of nonlinear time history analyses have been conducted for seven different earthquake ground motions which are matched with the seismic zone IV (Sylhet) of Bangladesh. The results are observed in terms of storey displacements, inter-storey drift, and the hysteretic responses of the coupling beams at different hinges. The results are compared for maximum considered earthquake (MCE), design basis earthquake (DBE), and service level earthquake (SLE). Parametric studies are also conducted to observe the influences of rebar grades, span to depth ratios, and concrete strengths of the coupling beams. The outcome of this study may appraise the role of coupling beams and their applicability in flat plate slab system.

Keywords: *Non-linear Dynamic Analysis, Flat Plate Slab System, Coupling Beam, Hysteretic Response, Storey Drift.*

1. INTRODUCTION

Generally, core shear wall systems are widely used in tall structures in a view to resisting the major amount of shear force by the wall and taking the higher flexural rigidity into account to reduce structural deflection(White and Dolan 1995). Alternative approaches such as dampers and base isolators are also widely used to mitigate seismic forces on the structures (Ahmed, Tasnim et al. 2016, Farzana and Ahmed 2020, Mahmud and Ahmed 2020). However, ordinary frames such as structures with a flat slab or flat plate slab system may take the advantage of core shear wall expecting that they will take the major lateral forces and control the deflections.

Shear walls are usually used as a primary lateral load resisting system in the high-rise building and sometimes they come in the form of two or several walls connected with a deep short beam due to the presence of voids to accommodate openings for windows or doors. This combination of walls and coupling beams is called pier-spandrel condition. Therefore, an RC coupling beam with a certain reinforcement layout is used as a means of increasing the wall resistance and a way of energy dissipation. This system which consists of walls connected at different levels with coupling beams acts as a frame. Those spandrels are responsible for coupling the lateral load resistance of adjacent walls so they are well known as coupling beams. A lot of researchers over the past decades found that this system provides high lateral stiffness and strength through better energy dissipation, less wall rotation, as well as a reduction in inter-story drift, peak roof drift, and base shear while ensuring a more evenly distributed heightwise deflection profile (Paulay and Santhakumar 1976). Application hybrid rebars such as stainless steels, shape memory alloy and HS rebars in RC components also improve the performance in terms of high deformability and low residual deformations (Islam, Billah et al. 2020, Ahmed, Habib et al. 2021).

In coupled shear wall systems, excessive shear forces are induced in the coupling beams. As a result, in such systems, the coupling beam and the joint of wall-coupling may yield first. The critical concern about the coupling beam is ductility demand. To have such ductility, the coupling beams are required to be properly detailed with significantly complicated reinforcement arrangement and insignificant strength degradation during ground motion (Bagheri and Oh 2018).

Fig. 1 presents a schematic illustration of how the level of coupling affects the axial stresses on the wall cross-section. In the case of no coupling, the overturning moment will be resisted through walls flexure strength. If there is enough coupling between the walls, end moments will be induced and the accumulated shear in the coupling beams will contribute to resisting a portion of the total overturning moment (Tabassum and Ahmed 2018, Nofal, Elsayed et al. 2020).

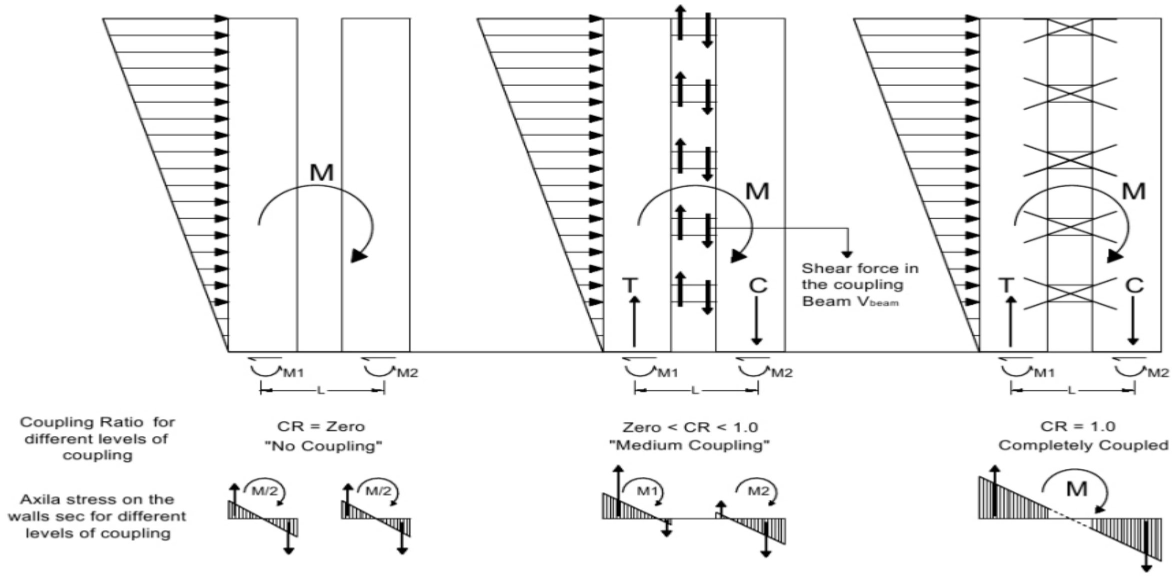


Fig. 1- Schematic representation of the different levels of coupling and their accompanied wall state of stresses.

Current design guidelines, which are based on a strength-based design approach, often result in beams with unrealistic details. Previous studies have also demonstrated that coupled core walls behave differently from what is assumed in a traditional strength-based design approach. Many researchers used linear elastoplastic analysis to investigate the partial or complete collapse mechanism of beams coupled with shear wall system [e.g. (Paulay 1970), (Lu and Chen 2005)]. Discrete lumped mass plasticity models have also been used to model the coupling beam as an element with a concentrated shear or flexure stiffness spring according to the predominated behavior in the regions under consideration and arms to simulate the rigid portions e.g. (Harries, Mitchell et al. 1998), (Kwan and Chan 2000). Even though the design procedure is similar to that used with moment frames, the existing force-based seismic design practice has some limitations in considering response modification factors, design overstrength, deflection amplification factors, ductility, etc.

To solve these problems and to increase energy dissipating capacities and reduce damage to the structure, this study presents an investigation of the seismic behavior of coupling beam with the coupled shear wall-frame system, in which energy dissipation hinge properties are located at the middle portion of the linked beam. Critical responses of the prototype structures at different limit states under representative ground motions are compared in order to evaluate the adequacy of the performance-based design method. Such parametric study is particularly well suited for use in regions of moderate to high seismic risk.

This study aims to investigate the response of a flat plat structural system containing a coupled shear wall and gravity columns through non-linear time history analysis. In view to understanding the response of the coupling beam of the shear wall, this study also observed the hysteretic response of the coupling beam at different floor levels. A parametric study has also been conducted to obtain the influence rebar and concrete strength, and length to height ratio of the coupling beam on their cyclic responses.

2. METHODOLOGY

A 3D finite element model of the RC structure with flat plate system containing columns and core shear wall with coupling beam has been developed using commercial building design software ETABS of CSI Inc. The plan view of the structure with dimensions is shown in Figure 2. The basic dimensions are kept as 22m in the x-direction, 22m in the y-direction, 4.25m of bottom story height, and 3m of typical floor height. Both geometric and material non-linearity are considered in the analysis procedure. The basic materials properties used in this building are presented in Table-1.

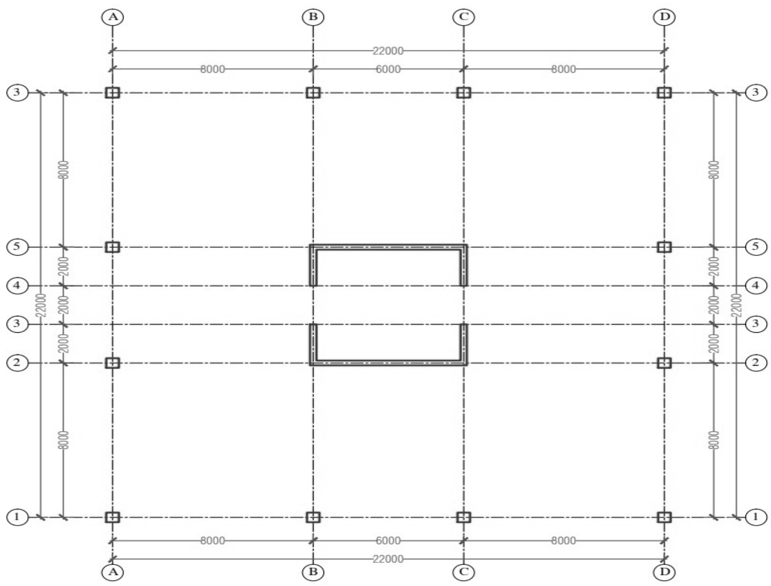


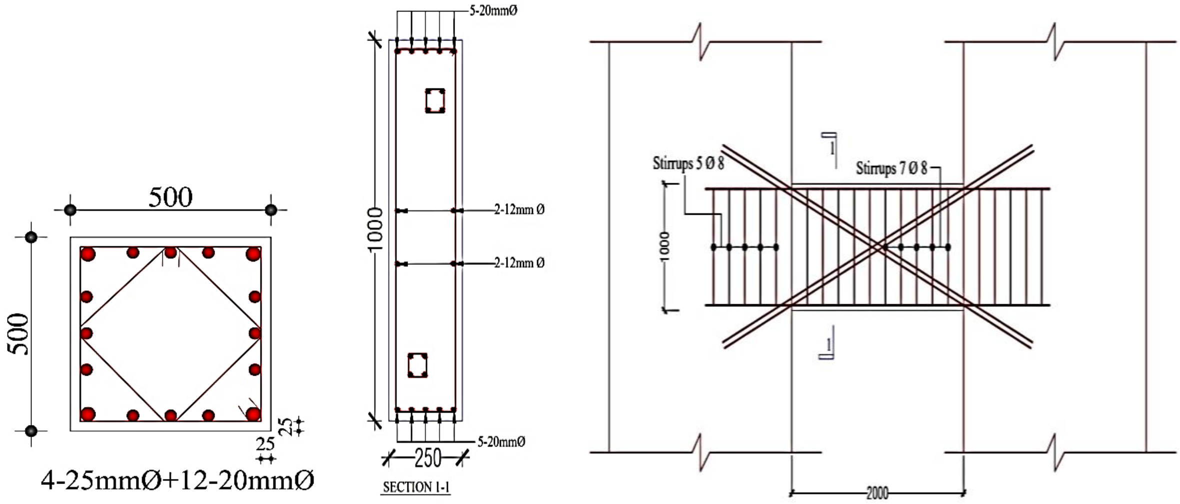
Fig. 2 - Floor Plan of Structure (all dimensions in mm)

Table 1 - Material Property

Material	Property	Values
Concrete	Compressive Strength (MPa)	35
	Modulus of Elasticity (GPa)	34
Steel Reinforcement	Yield Strength (MPa)	355
	Tensile Strength (MPa)	510
	Modulus of Elasticity (GPa)	21

2.1 Section and Material Properties

All the column sections were chosen the same all over the building height for simplification. The density of concrete and brick walls are 24 kN/m^3 and 20 kN/m^3, respectively. The slab thickness was kept uniform to be 200mm for all floors as material properties, a standard 35 MPa concrete and 60-grade steel are used in the analysis process as shown in Table 1. The load patterns and combinations of the preliminary force-based analysis are presented in Tables 2 and 3, respectively. There are variations of column and shear wall sizes of the building is shown in Table-4. Other seismic design parameters are given in Table -4. Considering an OMRF (R=3 as per ASCE 7-10) at the Sylhet zone (Z=0.36g) and soil type E (ASCE -7) of Bangladesh, the PMM ratio has been checked for the concrete columns before going into the non-linear analysis of the structure. The capacity of structural columns and shear walls was kept within the limit. The non-linear behavior of the material is presented in figure 4. The kinematic hardening model is considered for hysteretic responses of the rebar.



(a) Column (b) Coupling beam

Fig. 3- Detailing of Columns and Coupling Beams.

Table 2 – Loads

Load Patterns	Descriptions	Roof
DL	1.5 kN/m^2	1.5 kN/m^2
LL	5 kN/m^2	2 kN/m^2
PW	5.5 kN/m^2	0.25 kN/m^2
UTILITY	1 kN/m^2	0 kN/m^2

Table 3 - Load Combinations

Combinations	Descriptions
1	1.4DL[a]
2	1.2DL+1.6LL[b]
3	1.0DL+1.0LL+1.0Ex[c]
4	1.0DL+1.0LL+1.0Ey[d]

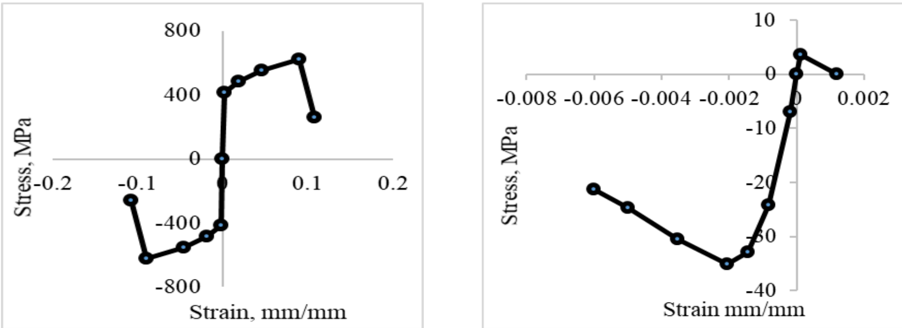
[a] Dead Load; [b] Live Load; [c] Earthquake Load in x-Direction; [d] Earthquake Load in y-Direction

The non-liner sectional properties in terms of the moment-curvature relationship of the column are presented in Fig.5. It is a representation of the bilinear moment-curvature relationship in non-linear analysis. As the moment to capacity ratio increases, the curves seem to fall as it cannot take much moment as the load increases that much after this failure will occur. In Figure, a moment-curvature diagram of the section is given. The section cannot carry more moment than approximately 10000 kN-m. A section undergoes fiber to fiber redistribution when these moment values are reached. If the curvature of the element is greater than approximately 0.05 m-1, the section capacity reduces to approximately 75kNm, in other words, plastic hinges are formed.

Table 4 –Sizes of columns and Shear wall and seismic design parameters

Storey ID	Floor Level	Description	Column Size (mm)	Shear Wall Thickness	Seismic Design Parameter
		Corner Column	600 x 600	-	Ss= 1 S1= 0.3
15 storey	up to roof	(2% Reinforcement)		350	Site Class: C Risk Category: III
		Other Column Sizes Size (mm x mm)	750 x 750	-	SDC: E I= 1 R= 3, Cd= 3

Note: SDC = Seismic Design Category



(a) (b)

Fig. 4 - Material non-linearity; (a) Steel and (b) Concrete.

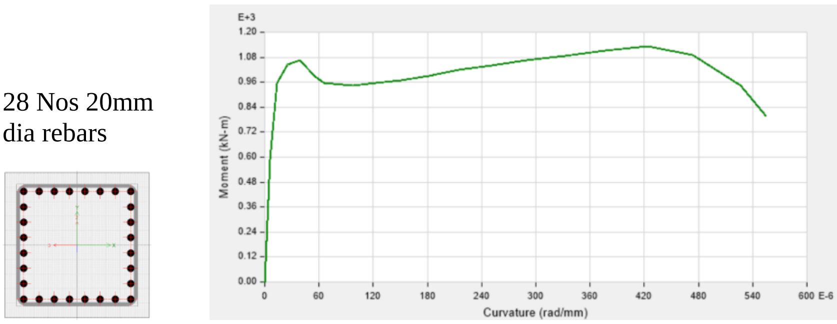


Fig. 5 - Moment curvature relationship of column section.

2.2 Hinge Properties

Beam and column elements were modeled as nonlinear frame elements with lumped plasticity. In this study, user-defined hinge properties were implemented. The plastic hinge locations were assumed and defined on the two ends of the column and beam elements. The hinge properties and seismic performance criteria have been assigned in accordance with FEMA 356 (ASCE 2007) guidelines. Since shear behavior is usually not coupled with bending, it is desirable to maintain shear demand below shear capacity to ensure ductile flexural behavior as the governed action.

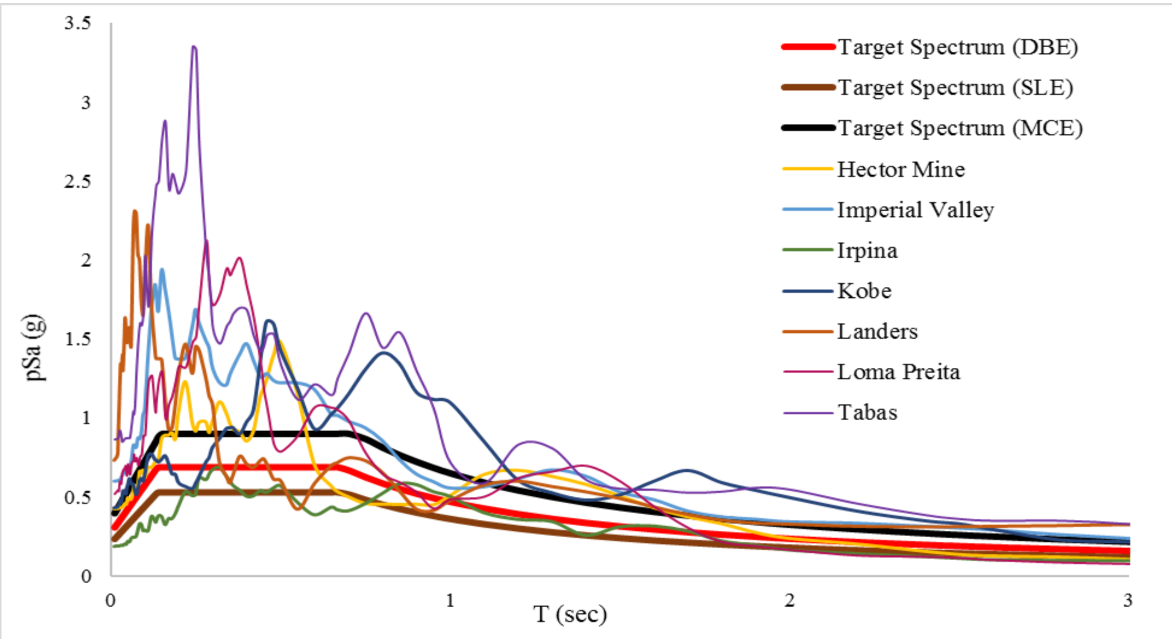
2.3 Ground Motions

The seven pairs of ground motions (both X and Y Direction) are considered in this study as presented in Table-5. These are recorded earthquakes and will be scaled to approximately 1.35g to represent an MCE earthquake. The peak displacement from NLTH does not correspond to ultimate displacement from pushover analysis. For nonlinear analysis, FNA usually uses the Ritz vector which is customized with P-Delta based on the gravity loads. It is important to use P-delta as it is critical in properly determining stability behavior. A series of nonlinear time history analyses have been conducted for seven different earthquake ground motions which are matched with the seismic zone IV (Sylhet) of Bangladesh. All the ground motions were matched with the site response spectra for maximum considered earthquake (MCE), design basis earthquake (DBE), and service level earthquake (SLE) as shown in Fig.6.

Table 5 - Earthquake GMs used in the study

NGA No.	Event	Year	Magnitude	Component	PGA(g)	Scale factor
1101	Kobe-Japan	1995	6.90	FP	0.275	3.64
				FN	0.326	3.07
184	Imperial Valley-06	1979	6.53	FP	0.353	2.83
				FN	0.481	2.08
289	Irpina-Italy-01	1980	6.90	FP	0.226	4.42
				FN	0.236	4.24
1787	Hector Mine	1999	7.13	FP	0.265	3.77
				FN	0.327	3.06
143	Tabas-Iran	1978	7.35	FP	0.852	1.17
				FN	0.859	1.16
879	Landers	1992	7.28	FP	0.719	1.39
				FN	0.779	1.28
752	Loma Preita	1989	6.93	FP	0.511	1.96
				FN	0.437	2.29

Note: FP = Fault-Parallel; FN = Fault-Normal; NGA No. = sequential number in PEER strong GM database; PGA = Peak Ground Acceleration.

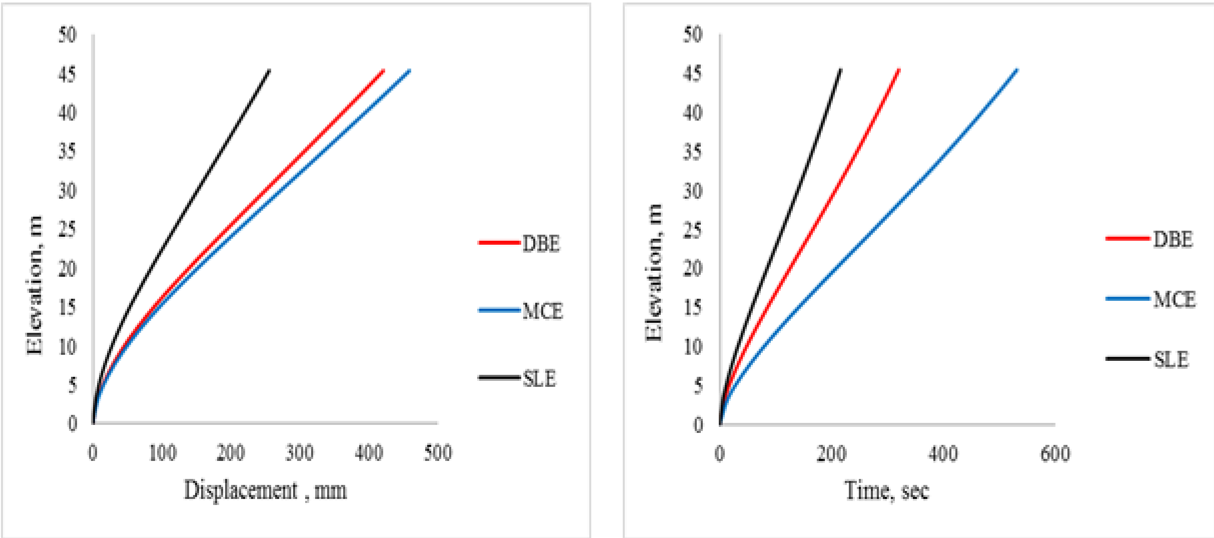


DBE = Design Basis Earthquake; SLE = Service Level Earthquake; MCE = Maximum Considerate Earthquake.

Fig. 6-Response spectra of earthquake GMs

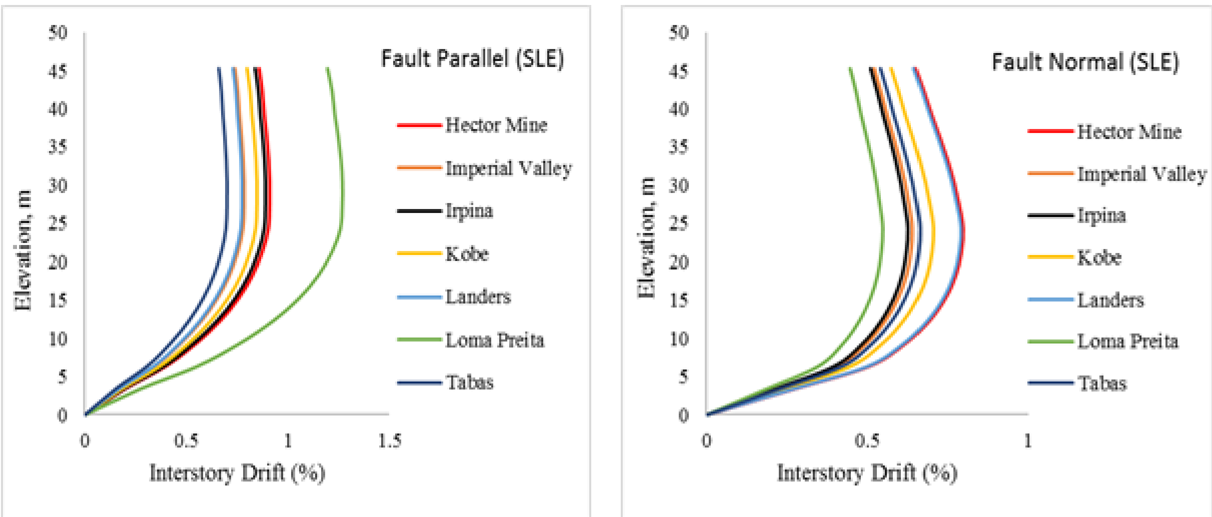
3. RESULTS AND DISCUSSIONS

Many coupled shear walls had been numerically analyzed in an attempt to know how each coupling beam parameter affects the overall behavior of the coupled shear wall system in terms of story displacements, inter-story drifts, and hinge responses. Besides, the impact of the coupling beams design and its reinforcement pattern on the overall system behavior. In this section, the numerical results have been discussed to know which parameter is effective in changing the system behavior. Results are discussed for Imperial Valley to get a clear idea of changes in the results of the buildings.
It can be observed from the NLT analysis that maximum displacement is found at the top story for both of the GMs directions along Fault Parallel and Fault Normal. In comparison among MCE, DBE, and SLE, maximum story displacement is observed for MCE expectedly and the value is around 500 mm for Imperial Valley. For DBE the maximum story displacement is reduced to around 400-500 mm and for SLE displacement is around 200-300 mm. The difference between the MCE and DBE is only 10-20% whereas the displacement at the service level earthquake is nearly half of that observed in the MCE.



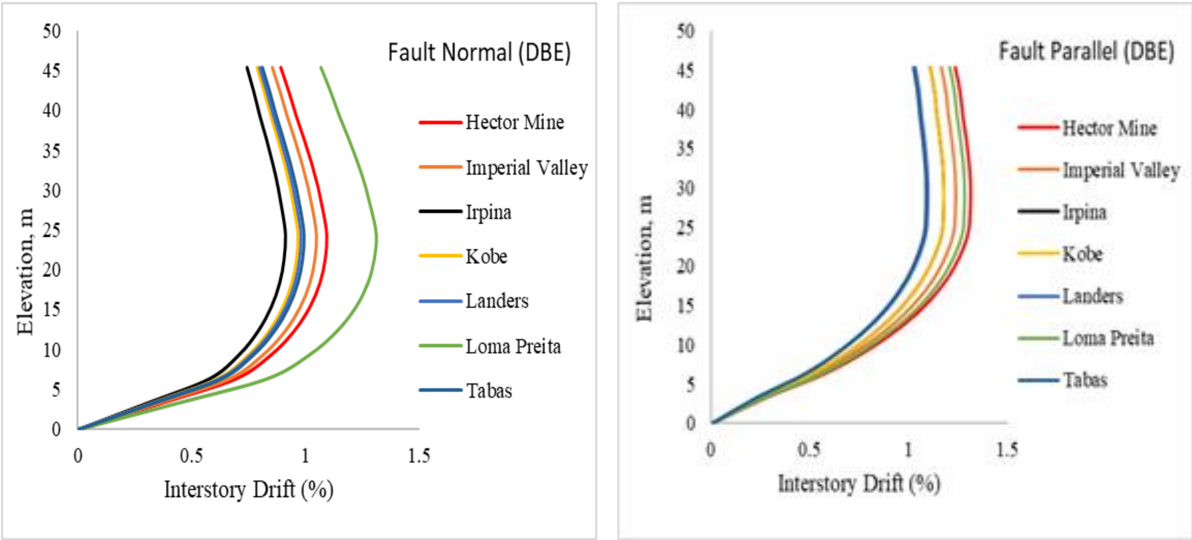
(a) (b)

Fig. 7- Maximum storey displacement (Imperial valley); (a)Fault Parallel (b)Fault Normal.



(a) (b)

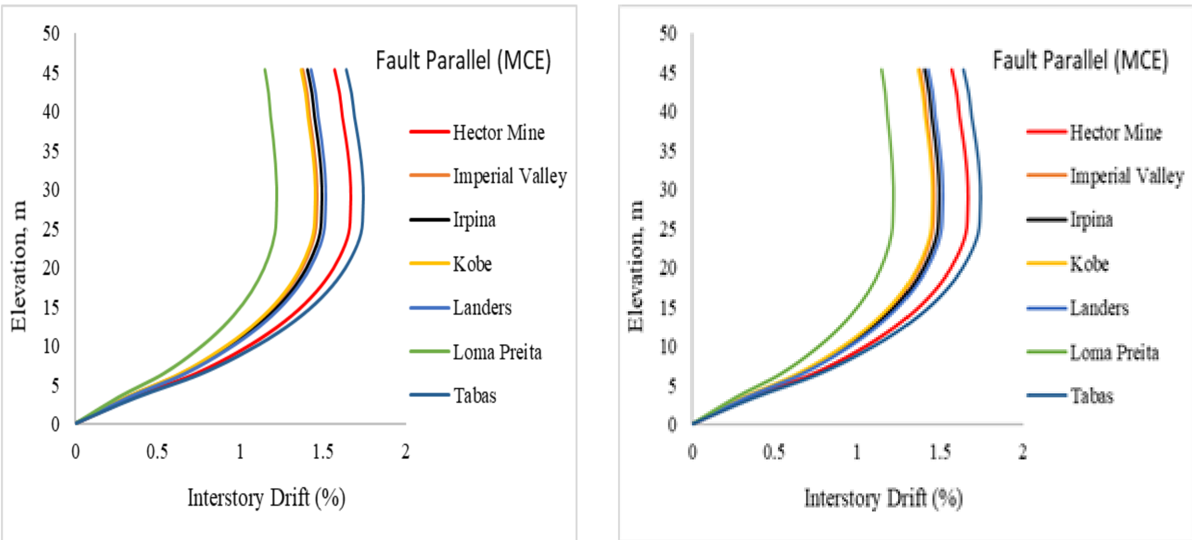
Fig. 8- Inter-story drift ratio under earthquake GMs (SLE); (a) Fault Parallel (b) Fault Normal



(a) (b)

Fig. 9- Inter-story drift ratio under earthquake GMs (DBE); (a) Fault Parallel (b) Fault Normal

The inter-story drift demand is computed from dynamic analyses for the ensemble of ground motion data and corresponding average values are depicted in Figs.8~10. The figures show that for the 15-storey structure the demand is maximum at 8th storey level. The result shows that the maximum inter-storey drift is observed at the 7th to 8th storey level (20-25m elevation). For MCE, the maximum storey drifts ratio for Loma Prieta and Hector mine are observed to be 1.4% and 1.75% for fault normal and fault parallel, respectively. For DBE maximum storey drift ratio is observed to 1.2 to 1.4 % for the same pair of ground motions.



(a) (b)

Fig. 10- Inter-story drift ratio under earthquake GMs (MCE); (a) Fault Parallel (b) Fault Normal.

The response coupling beam of the coupled shear wall at different storey levels has been presented in Fig.11. The hinge of the coupling beam shows that maximum plastic rotation is observed on the ground floor and the chord rotation decreases as the beams are located at the upper stories. The beam at the 8th storey and above shows a linear response where inelastic rotation is nearly zero. It is also evident that the coupling beam allows higher deformability and hence absorbs the earthquake-induced energy.

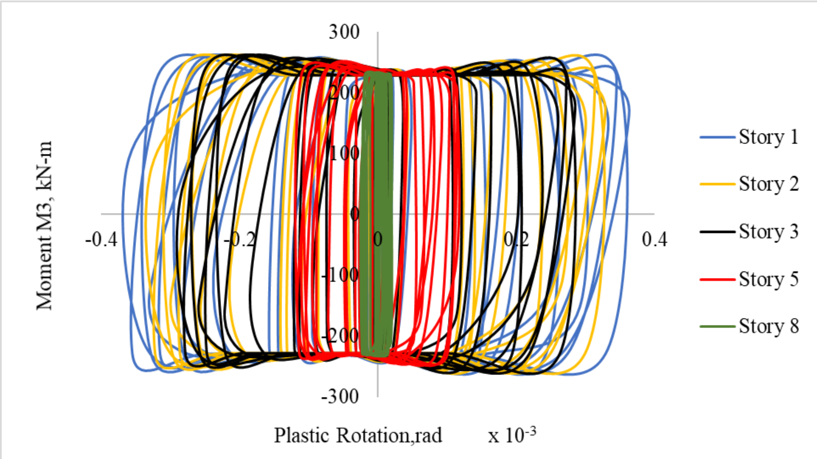


Fig. 11- Response of the coupling beam of coupled shear wall

4. Parametric Study

To investigate the influence of steel grade, length to height ratio, and concrete strength on the response of the coupling beam, a parametric study has been conducted for Imperial valley and Hector mine ground motions. It is observed from Figure 12 that as the steel grade increases the moment carrying capacity also increases keeping the same maximum chord rotation. As with the increase in steel grade the yield and ultimate strength also increase. As a result, coupling beam with 75 grade takes more moment in comparison to 40 and 60grade. Plastic rotation remains quite similar for all the steel grades. Both of the GMs imperial valley and hector mine show a similar response.

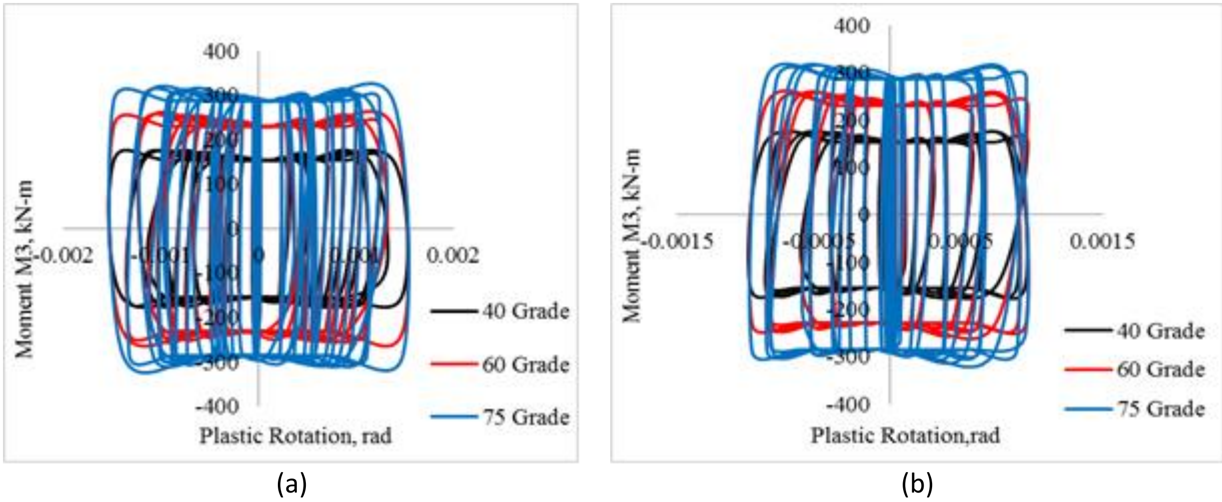
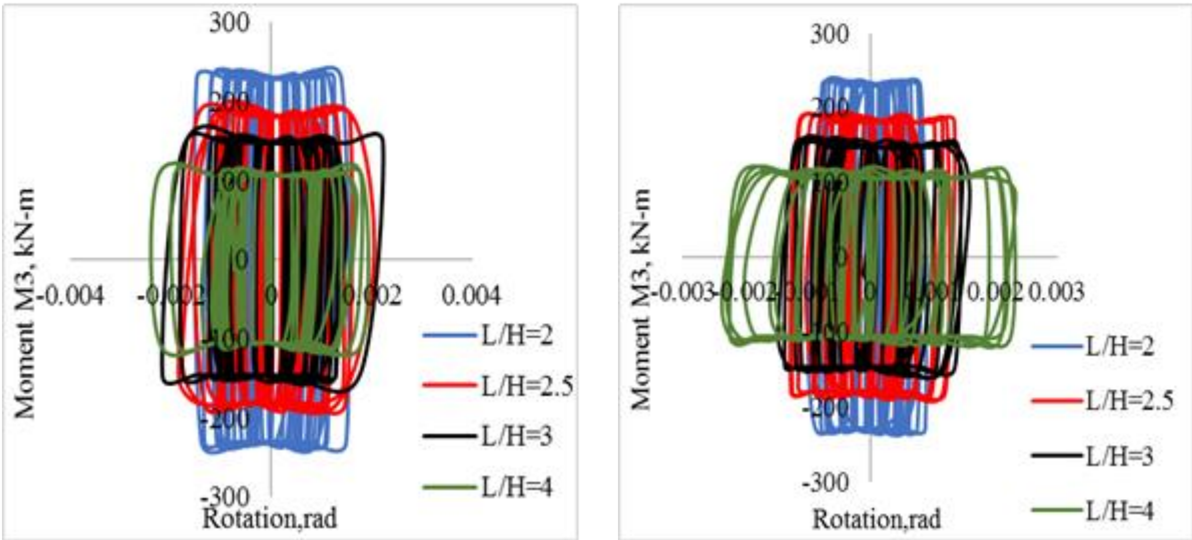


Fig. 12- Beam hinge responses under earthquake GMs for different grades of rebar: (a) Hector Mine; and (b) Imperial Valley.

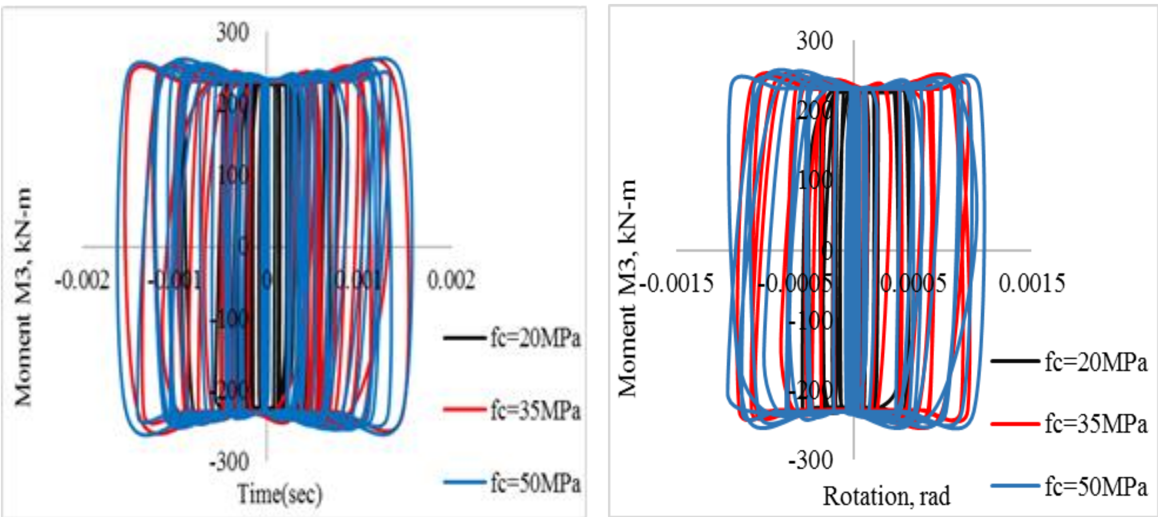
Fig.13 shows the length-to-height ratio dependency of the coupling beam. It is interesting to highlight that as the L/H ratio increases, the hysteretic response shapes flattens and cord rotation increases and the maximum moment of the plastic hinge decreases. This phenomenon can be described by the fact that as L/H increases the bending stiffness of the beam decreases and hence deformability increases therefore the hysteretic curve flattens as chor rotation increases. The higher plastic rotation indicates higher damage to the structural components. The structural designer must be careful in selecting the sizes of the coupling beam as it controls the deformation of the core shear wall system. Therefore, it is considered the most effective parameter that could change the behavior of the coupling beam dramatically.



(a) (b)

Fig. 13- Beam hinge responses under earthquake GMs for different ratios of span to depth: (a) Hector Mine; (b) Imperial Valley.

For the higher strength of concrete, earthquake energy absorbing capacity is higher. The influence of concrete strength is presented in Figure 14. It is observed from the figure that the role of concrete strength is not much significant in carrying the moment of the coupling beam particularly for the strength of 35MPa and above. In fact, the moment carrying capacity is largely dependent on the shape and reinforcing bars rather than the concrete strength as it can hardly take any tension. Both of the GMs show quite similar responses.



(a) (b)

Fig. 14- Beam hinge responses under earthquake GMs for different concrete strength: (a) Hector Mine; (b) Imperial Valley

5. CONCLUSIONS

In this study, a 15 storied commercial building containing a flat slab structural system with coupled shear-wall is investigated to observe the building behavior and also coupling beam responses. The key outcomes of this study are

(a) The maximum storey displacement is observed on the top floor. The difference of storey displacement between MCE and DBE is 10 to 20% whereas the difference s 50% between MCE and SLE.
(a) The maximum storey drift ratio is observed at the 8th floor of the 15 storey building which is nearly half of the total height of the building for both fault normal and fault parallel scenarios.
(b) The hinge response of the coupling beam shows that the plastic rotation of the beam decreases as it goes to the upper floors and so the beam at 1st storey level experiences maximum plastic rotation. The study suggests that the coupling beam at 1st storey level must be designed carefully. The related engineer and stakeholders must decide the design strategy whether minor damage is allowed in the coupling beam particularly at the first-floor level.
(c) The parameter dependency study of the coupling beam suggested that a higher grade of steel shows comparatively better performance as it can absorb more energy without increasing the chord rotation. In such cases, the ductility must be ensured by the manufacturer. The influence of the length-to-height ratio of the coupling beam is very much significant. The results show that with the increase of the L/H ratio of the coupling beam, the system becomes more flexible and experience higher cord rotation and lower moment capacity. The higher plastic rotation will result in high damages to the beam without increasing its capacity. The structural engineer must consider this situation in the design strategy to control the damages of the core shear wall as well as the global structure.
(d) The variation of concrete strength minorly influences the coupling beam responses. A noticeable influence is observed for the concrete strengths higher than 35 MPa.

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