Seismic Performance Assessment of Existing Mid-Rise RC Buildings in Pakistan

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Abstract

Recent disastrous earthquakes reveal the vulnerability of the existing reinforced concrete (RC) buildings in Pakistan. The increased level of awareness about possible seismic hazard raises serious concerns about the structural performance of RC buildings. Most of the RC buildings in Pakistan are not in compliance with the prevalent stringent seismic requirements. In the current study, a 13-story RC shear wall building, located in the capital city of Pakistan, is considered as case study to evaluate the structural performance of existing buildings. The case study building is categorized as mid-rise RC building. Nonlinear response history analysis (NLRHA), as per ASCE-41-06, is used to evaluate the seismic performance of the case study building. The result shows that the case study building will be severely damaged In the case of an event of an earthquake. This study concluded that more studies are needed to access the seismic performance of the existing RC buildings in Pakistan so that suitable retrofitting measures can be devised.

Keywords: ***Seismic Evaluation, NLRHA procedure, RC Shear walls***

1. INTRODUCTION

Natural disasters have always been threatening to human civilization. Earthquake is one of the most devastating natural hazards. Pakistan geographically situated in a region of vigorous seismic activities. Pakistan has a long history of seismic events essentially because of the interaction of the plates in the Karakoram Range. In recent times Pakistan has faced many major earthquakes, 80,000 people died in Kashmir earthquake 2005 and nearly 2000 causalities were reported in Peshawar earthquake in 2014. These events of ground shaking is related with complex plate boundary conditions, which encompasses Pakistan. Indian plate and Eurasian plate are moving towards each other at 3cm and 1.3cm per year respectively. This opposite movement of plates has cracked Indian plate into many slices.
In a developing country like Pakistan, we are lacking far behind in technical skills and expertise to understand the seismic activities and accordingly designed safer structures, the situation is much worse than we thought. Recent earthquake raised serious concerns about the structural performance of reinforced concrete (RC) building in Pakistan.
After the Kashmir earthquake in October 2005, Government of Pakistan directed national engineering services of Pakistan (NESPAK) to develop new seismic codes for the country to save the buildings during earthquakes. Due to the delay in the process of developing the new codes, Earthquake Reconstruction and Rehabilitation Authority (ERRA) started using Uniform building codes (UBC-97, 1997). Later on, building codes of Pakistan (BCP, 2007) was published in 2007. Although this building code named as Building code of Pakistan, due to lack of ground motion filed data and experimental lab data, it is almost similar to the UBC 1997.
Recent earthquake raised serious concerns about the structural performance of reinforced concrete (RC) building in Pakistan. Severe damage was reported in the mid-rise buildings in Rawalpindi/Islamabad region. The incident of margalla tower collapse in prominent. So, there is a serious need for assessment of existing RC buildings in Pakistan.

2. BUILDING INFORMATION

AWT plaza is a thirteen story building, and it is located on the Mall road in Rawalpindi. The building was built in the 1980s. The building consists of two parts separated by 1-inch wide seismic joint. One part of the building is thirteen-story, while the other part is a single story. The thirteen story part of the building has a footprint of about 140’ x 120’; the building’s area decreases at each floor after the fifth level, making it an irregular structure in planar, as well as, vertical sense. The total height of the building is about 156ft (47.5m); its story height is 12 ft. (3.66m), and columns of the building are spaced at 20 ft. (6m) for the selected floor plan. Reinforced concrete walls have been employed in the building to cater to the strength and stiffness requirement of the structure against lateral loads. The building has reinforced concrete frames which enables it to transfer gravity loads from floors to the foundation. The floor system consists of reinforced concrete beams between columns and, predominantly, six inches thick reinforced concrete slab; the foundation of the building mainly consists of 53 inches thick mat for 13 story part.

3. COLLECTION OF BUILDING DATA

The data regarding the structural system and sizes of structural members are taken from structural drawings. A visual inspection of the building was also made, and the size of shear walls, columns, and spacing between columns was verified with structural drawings. The strength of rebar is also taken from the structural drawings. The strength of concrete for different structural members is not mentioned in the structural drawings, only the class of concrete is mentioned, without reference to any structural member. The strength of concrete for foundation, beams, and slabs was assumed, while the strength of columns and shear walls was taken from the results of non-destructive tests.

3.1 Cross-section of columns

The size of columns has been taken from structural drawings and was confirmed with measurements while doing a visual inspection of the building. The detail of reinforcement of the column at a particular location could not be determined from structural drawings because of non-availability of the column layout plan. The reinforcement of a column at a particular location was assumed with the help of results of the ETABS model of the building and the data of column cross-sections on structural drawings. Table 1 shows the cross-section of columns.

3.2 Cross-section of Shear Walls

All shear walls in the building are 8 inches thick and have the same amount of flexural and shear reinforcement. Table 2 shows the detail of transverse and longitudinal reinforcement of the walls.

Table 1: Reinforcement detail of columns

Column ID	**Size (inches)**	**Longitudinal reinforcement (in^2)**	**Transverse reinforcement**
C1	24 x 24	6 at all levels	#3@8'' c/c ; 0.33 in^2 in each direction
C2	24 x 24	6 (1-3 levels); 7 (remaining levels)	#3@8'' c/c ; 0.33 in^2 in each direction
C3	24 x 24	12.64 (1-3 levels); 7 (remaining levels)	#3@8'' c/c ; 0.33 in^2 in each direction
C4	40 x 8	13 (1st level) 5.5 (remaining levels)	#3@8'' c/c ; 0.22 in^2 parallel to short direction 0.44 in2 parallel to long direction
C5	Triangular 39 x 31 x 47	22 at all levels	#3@8'' c/c ; 0.44 in^2 in both directions
C6	24 x 24	26.6 (1-2 levels) 12.6 (3rd level) 6 (remaining levels)	#3@8'' c/c ; 0.11 x3 in^2 in each direction

Table 2: Reinforcement detail in shear walls

Level	**Longitudinal reinforcement**	**Transverse reinforcement**	**thickness**
1-2	#5 @ 8 in c/c	#12 @ 8 in c/c	8 in
3-4	#4 @ 8 in c/c	#3 @ 8 in c/c	8 in
5-till end	#3 @ 8 in c/c	#3 @ 8 in c/c	8 in

Table 3: Material strengths

Member	**Concrete, *f'c* (ksi)**	**Main bars, *fy* (Ksi)**
Slab	3	60
Beams	3	60
Columns	4	60
Shear Walls	4.5	60
Foundation	3	60

4. NONLINEAR SEISMIC EVALUATION

4.1 Performance Objective

The main objective is to evaluate the structural performance of the existing building under gravity and seismic loadings. For seismic loading, the building shall be checked to satisfy Basic Safety Objectives, with a goal to provide a low risk to life safety for any seismic event likely to affect the building. “Life Safety” performance level shall be checked against 475-year return period earthquake the earthquake that has a 10% probability of exceedance in 50 years, also Known as Design basis earthquake (DBE), and “Collapse Prevention” performance level shall be checked against the 2475-year return period earthquake the earthquake that has a 2% probability of exceedance, also known as Maximum considered earthquake (MCE).

4.2 Seismic Loads

Uniform Hazard spectra used in the Pakistan Building Code-2007 (BCP, 2007) resulting from a probabilistic seismic hazard is used in this evaluation. Effective viscous damping of 5% of critical damping in considered in both 475-year (DBE) and 2475-year (MCE) return period earthquakes.

4.3 Expected Seismic Hazard and ground motion Selection

As required by the Building Code (TBI-2010, 2010), seven accelerogram sets were used for Nonlinear Response History Analysis (NLRHA). Keeping in mind this, deaggregation analysis is performed to identify the sources of the expected seismic hazard. This will help to select the suitable ground motion for AWT building. Results of deaggregation analysis, are shown in Figure 1 and Figure 2. The result of geographic deaggregated seismic hazard map for AWT Plaza shows that the 0-50Km seismic source dominates the seismic hazard (M6.60 at a distance of 14 km) and the Main Boundary Thrust is the single fault which shows a little contribution (M7.78 at a distance of 13 km). Based upon these results following ground motions were selected from the PEER NGA data base, Table 4.

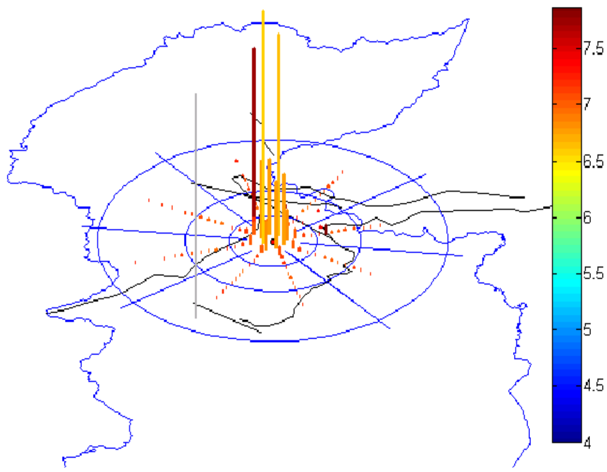

Figure 1: Deaggregation analysis for the earthqua with a 10% probability of exceedance in 50 year

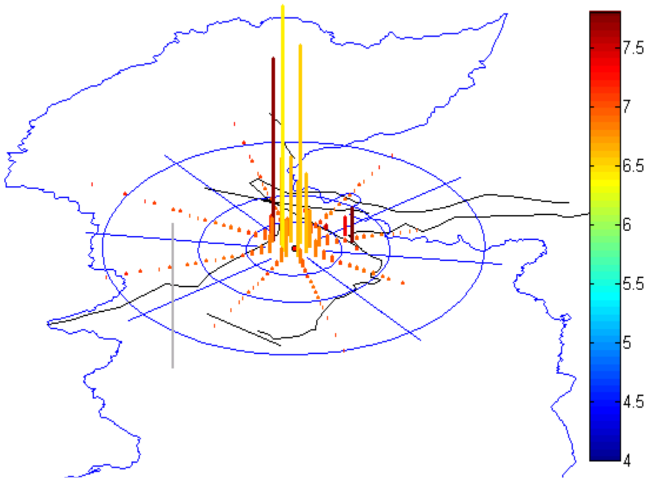

Figure 2: Deaggregation analysis for the earthquake with 2% probability of exceedance in

Table 4: Selected ground motions

No	Earthquake Event	Year	M_w	PGA (g)	Duration (sec)
1	Loma Prieta	1989	6.93	0.14	60
2	Loma Prieta	1989	6.93	0.21	59.88
3	Chi-Chi_ Taiwan	1999	7.62	0.27	80
4	Chi-Chi_ Taiwan	1999	7.62	0.15	70
5	Chi-Chi_ Taiwan	1992	7.62	0.13	70
6	Cape Mendocino	1999	7.01	0.33	28
7	Iwate_ Japan	2008	6.9	0.35	47

4.3 Acceptance Criteria

4.3.1 Adequacy of Components against seismic + gravity loads

The response of the components is checked based on the type of action, namely force control, and deformation control. For force control actions, the expected strength of the component should be less than the demand due to gravity and seismic forces. The expected strength of the component is calculated according to the procedures of ACI-318. The response of components is checked against “Life Safety” performance level under 475-year return period earthquakes while “Collapse Prevention” performance level will be checked under 2475-year return period earthquakes. The axial strain in shear walls is compared against the maximum allowed to check their adequacy against flexural-demand.

4.3.2 Story drift

Story drifts shall not exceed the following limits (ATC-40, 1996) table 11-2.

Load Case	Story Drift Limit
DBE Seismic (10%/50 yr)	2%
MCE Seismic (2%/50 yr)	$0.33\frac{V_i}{P_i} = \frac{0.33 \times 0.08}{100} = 2.7\%$

5. NONLINEAR MODELING OF THE AWT BUILDING

A nonlinear model of the AWT building is created to Perform 3D version 5.0 (Perform3D, CSI). Each structural wall is modeled by nonlinear fiber elements over the entire height since flexural cracking and yielding may occur at any location. The wall is divided into many horizontal layers, where each layer is modeled by a newly developed fiber model called Multi Vertical Line Element Model (MVLEM) (Orakcal & Wallace, 2012). This model is made of a large number of vertical concrete and steel fiber elements to simulate the combined axial and flexural behavior of the wall. It also has a horizontal shear spring to simulate the elastic shear response. A bilinear hysteretic model of non-degrading type is used for the steel fibers. The post-yield stiffness is set to 1.2 percent of the elastic stiffness. In making concrete fiber elements, the Mander’s stress-strain (Mander, Priestley, & Park, 2008) model for either confined or unconfined concrete is approximated by a tri-linear envelope. Each RC column is modeled by a combination of a linear elastic beam-column element with nonlinear plastic zones at its two ends. The un-cracked flexural rigidity is assigned to the linear element. The plastic zones are assumed to have a length of 0.5D, where D is the lesser cross-sectional dimension of the column. They are modeled by concrete and steel fibers in a similar manner to RC walls. By this way, the un-cracked (linear elastic), cracked, yielded, and post-yielded states of the column can be fully simulated(Najam & Warnitchai, 2018).

RC beam is modeled by a combination of a linear elastic beam-column element with nonlinear plastic zones at its two ends. The un-cracked flexural rigidity is assigned to the linear element. For nonlinear plastic hinge zone on both ends of beams, moment rotations relationships were developed using available construction detail(Najam, Warnitchai, Qureshi, & Mehmood, 2018).

The concrete slabs are assumed to remain elastic and are modeled by using rigid diaphragm floor constrains command. The mat foundation is treated as a rigid boundary, which is displaced horizontally by the input ground motion.

6. RESULTS OF NONLINEAR SEISMIC EVALUATION

6.1 Story Shears

Figure 3 presents the comparison of story shears obtained from the NLRHA procedure and from the code based Equivalent Lateral Static Force procedure. This comparison clearly indicates that there is significant shear amplification due to the negligence of higher mode contribution in the design procedure. Approximately a shear amplification factor of 2.8 and 3.6 was observed at the base of the structure. Usually, a shear amplification factor of 1.3 to 1.5 is anticipated to strength reduction factors and strain hardening. This shows that no shear amplification due to irregularities of the buildings are included in the design procedure.

6.2 Story drift

Story drift ratios are checked for MCE level ground shaking and found to be within the specified limit. Results are presented in Figure 4. Story drift ratios are lesser in the x-axis, which is obviously due to higher lateral stiffness contribution from RC walls in the x-axis direction.

6.3 Component responses

6.3.1 RC shear walls

Shear strength of RC walls is checked against the seismic shear demand obtained from both MCE and DBE level ground shaking. Results are presented in Figure 3. Five walls are estimated to fail in the shear mode of failure at several levels at MCE level seismic hazard, while 3 walls are estimated to fail in shear at DBE level seismic hazard.

The axial strain is an important seismic performance response parameter to assess the deformation related damage to a structure in an event of an earthquake. To avoid the crushing of RC shear walls, compression strain should be within the specified limit by ACI-318, which is 0.003.

6.3.2 RC Columns

Shear strength of columns is calculated based on ACI-318 (ACI-318-14, n.d.) equation and compared with the seismic shear demand obtained from both DBE and MCE level ground shaking. It was observed one of the RC columns, shown in the red in Figure 6, is expected to fail in the shear mode of failure under both MCE and DBE level at several floor levels. All other columns possess sufficient reserve capacity against the expected level of seismic hazard. Average demand to capacity ratio (D/C) is approximately 0.5. Flexural yielding occurs only in few columns at upper floor levels due to an increase in inter story drift ratios.

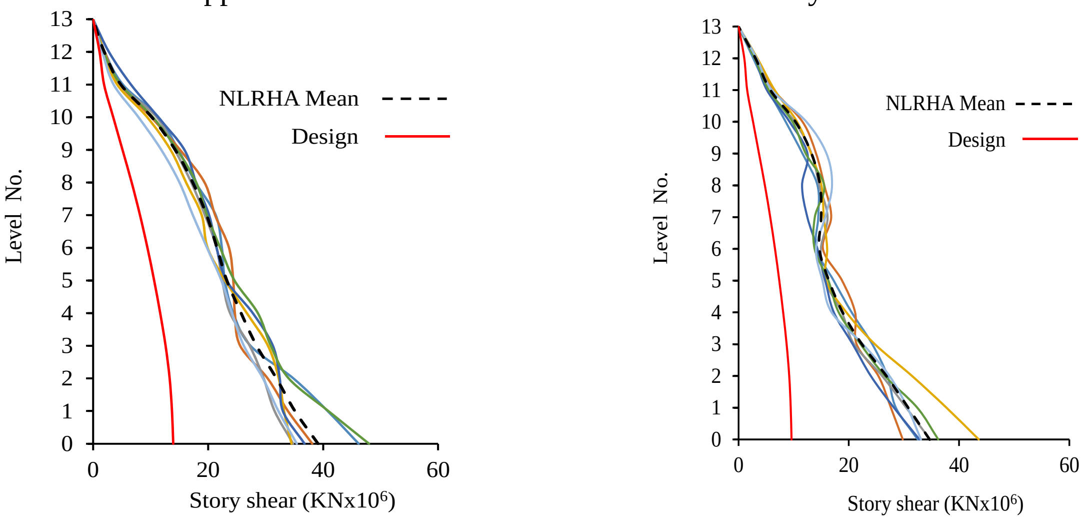

Figure 3: Story shear (a) X-axis (b) Y-**axis**

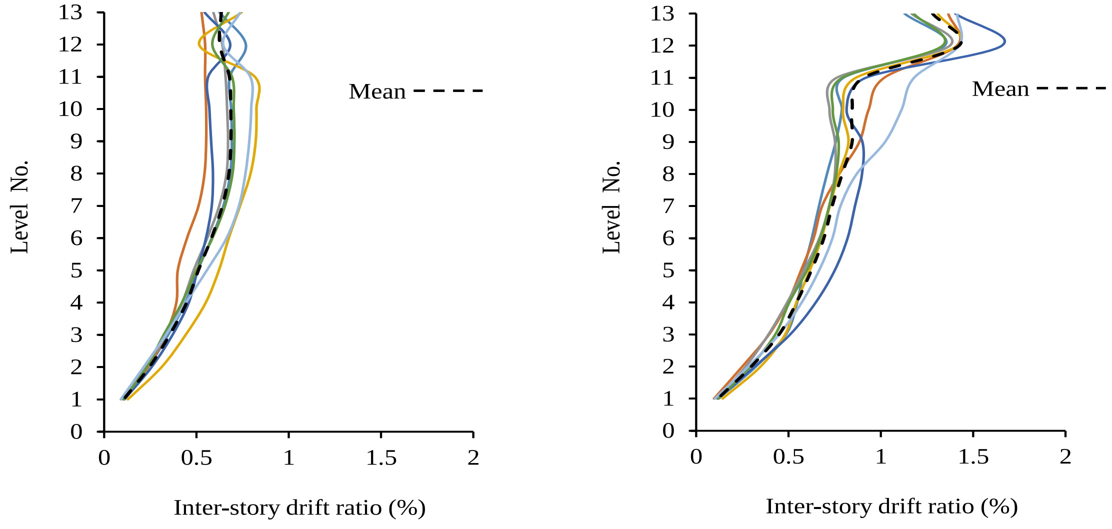

Figure 4: Inter-story drift ratio at MCE (a) X-axis (b) Y-axis

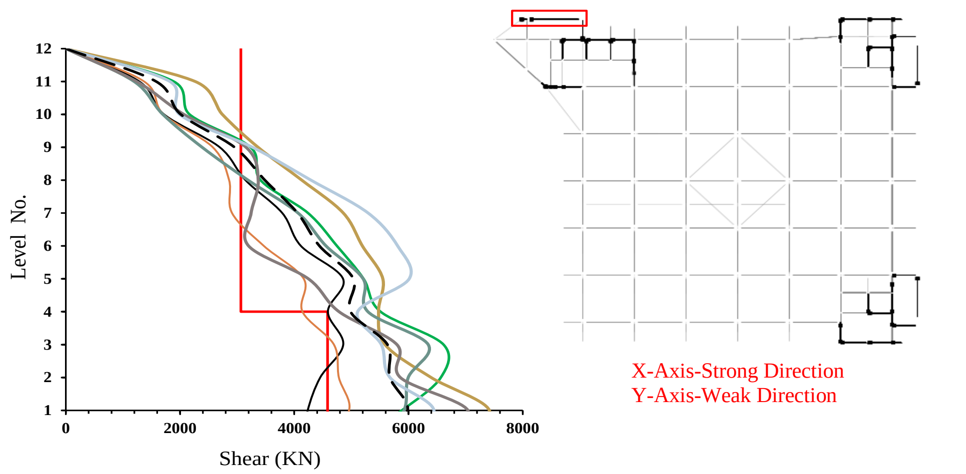

Figure 5: Shear demand versus shear capacity of RC walls at MCE level

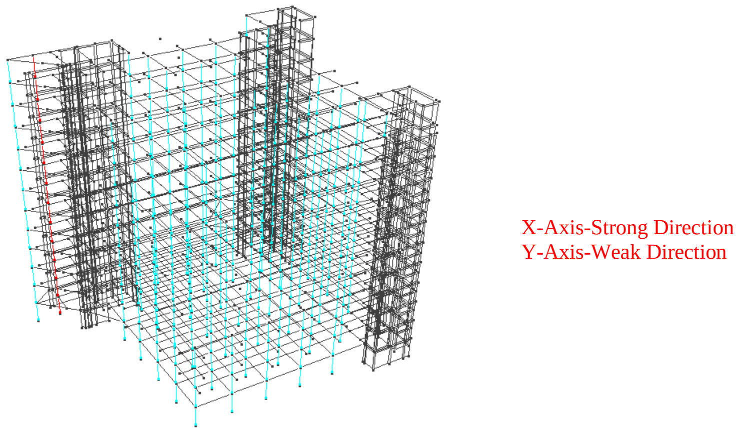

Figure 6: Shear failure in RC columns at MCE and DBE level

7. CONCLUSIONS

Following conclusions can be made based on the result of the nonlinear seismic evaluation.

- Structural walls are expected to fail in shear against MCE, as well as DBE, level seismic hazard.
- Structural walls are expected to behave favourably in flexure against both levels of seismic hazard.
- Beams should respond, to both levels of seismic hazards, favourably against force controlled, as deformation controlled, actions.
- One column is expected to fail in shear, as well as flexure, against both levels of seismic hazards. All other columns should respond favourably.
- The diaphragm is found to be adequate in transferring inertial forces to vertical members of the structure.
- The building should not be in serviceable condition after experiencing both levels of seismic hazards.
- The building poses a life safety hazard in the event of DBE level earthquake due to shear and flexural failure of one column; the failure of that column can lead to the partial collapse in that part of the building. The column is highlighted by an oval in figure 6.
- The building should not collapse under MCE level seismic forces as the drift ratio is under control, compression strains are within allowable limits and the bare frame can take at least 50% of the seismic force in both directions.

8. RECOMMENDATIONS

- It is recommended that the column, discussed in the previous section, be retrofitted to avoid a possible partial collapse in that part of the building.
- Pakistan building codes need to be developed and implemented efficiently and more studies are needed to assess the seismic performance of existing RC buildings in Pakistan So that suitable retrofitting measurements can be devised.

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