Economic losses due to earthquake - induced structural damages in RC SMRF structures

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Economic losses due to earthquake - induced structural damages in RC SMRF structures

Muhammad Rashid¹ and Naveed Ahmad²*

Abstract: This study presents the seismic performance assessment of reinforced concrete frame structures designed to modern buildings codes, for calculating the economic losses due to earthquake-induced structural damages. The structures investigated in the present research considered four (3, 5, 8, and 10 storeys) prototype special moment resisting frame (SMRF) structures designed to Uniform Building Code–97/Building Code of Pakistan. Quasi-static cyclic tests were carried out herein on special moment resisting beams in order to develop damage scale and beam reparability cost ratio. The considered structures were analyzed in a calibrated FE based software SeismoStruct using incremental dynamic analysis procedure employing a set of seven natural design spectrum compatible ground motion records. Damage to structural components was identified for each intensity level and integrated over the whole structure, with the required repair cost, to calculate the structure repair cost ratio (RCR). The structure RCR is correlated with the seismic intensity to develop seismic vulnerability curves, which can be used for the economic loss estimation (direct repairability cost) of SMRF structures given the seismic intensity.

Subjects: Structural Engineering; Georisk & Hazards; Computer Aided Design (CAD)

Keywords: RC SMRF; structural damage; repair cost ratio; economic loss; seismic vulnerability, UBC-97, BCP-2007; SeismoStruct

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PUBLIC INTEREST STATEMENT
Observations from recent worldwide earthquakes have demonstrated that structures designed to modern codes have been successful, in many cases, in protecting occupant’s lives during large damaging earthquakes. However, the economic losses due to incurred damages in structures were always enormous, surprising the clients about their structure seismic performance. This has been alarmed structural designers to look for alternative measures that can quantify the economic losses for a preliminary design scheme for future earthquakes. This paper presents a framework for the vulnerability assessment and economic loss estimation of structure with application to reinforced concrete frame structures designed to UBC-97/BCP-2007. This research demonstrates that economic losses of code design structures in a design base earthquake are significant, consequently, point to the importance of incorporating performance-based seismic assessment methodology in structural design practices.
1. Introduction

Earthquakes bring about extensive economic losses, massive casualties, and occupancy & business interruption, not only in the developing parts of the world but also in the developed and scientifically well-established (particularly in earthquake engineering) parts of the world. Recent and past observations have shown that earthquakes happen to be one of the costliest natural disasters in the history of human kind (Guin & Saxena, 2002). The 2011 Tohoku, Japan and Christchurch, New Zealand earthquakes are the recent examples of the potential of earthquakes to cause huge economic losses, which are regarded as some of the highest economic losses to insurance industry (Chang-Richards, Vargo, & Seville, 2013; Kam, Pampanin, Dhakal, Gavin, & Roeder, 2010; Nanto, Cooper, Donnelly, & Johnson, 2011).

Pakistan is a region with a high seismic risk, due to the prevailing earthquake hazard (moderate to severe) and high seismic vulnerability of structures and infrastructures (Ahmad, Ali, Crowley, & Pinho, 2014; Ali, Khan, Rahman, & Reinhorn, 2011). The seismic vulnerability of the reinforced concrete building stock of Pakistan was visible in the recent 2005 Kashmir earthquake due to the lack of seismic design nature of structures and poor quality of construction (Bothara & Hicyilmas, 2008; Naseer, Khan, Hussain, & Ali, 2010; Rossetto & Peiris, 2009). It is worth mentioning that this region is capable of triggering one or more future large earthquakes up to or even greater than magnitude 8, which was manifested in the 2015 Nepal earthquake that shares the same Himalayan belt (Avouac, Ayoub, Leprince, Konca, & Helmberger, 2006; Bilham, 2004). Recent worldwide earthquakes have brought the attention of many researchers to assess the seismic performance of existing and new designed structures and consequently to introduce measures to mitigate future seismic risk (Bossio, Fabbrocino, Lignola, Prata, & Manfredi, 2015; Formisano, Di Feo, Grippa, & Florio, 2010; Formisano & Mazzolani, 2015; Indirli, Kouris, Formisano, Borg, & Mazzolani, 2013; Pampanin, 2009, 2012, among others).

Reinforced concrete special moment resisting frame (SMRF) is the most prevalent structural system in the modern building stock of Pakistan. These structures are primarily designed to the Uniform Building Code-97 (UBC, 1997), which is adopted in the Building Code of Pakistan–seismic provisions (BCP-07, 2007). With the development and modernization of major cities of Pakistan such as Karachi, Islamabad, Lahore and Peshawar, the construction of multi-story reinforced concrete moment resisting frames is showing a rampant growth. Modern seismic design codes (ACI-318; NZS-3101, 2006; EC-8, 2004) design buildings with the objective to protect occupants and control damage in design level earthquakes and, presume, to avoid collapse in very rare earthquakes. The satisfactory or acceptable level of performance of a building designed to modern seismic design codes is only defined qualitatively and the performance of the building in a future earthquake can only be qualitatively described. Assuming that these buildings perform as intended by the building code: saves lives, avoids collapse and sustains damage in a design level earthquake, the unaware client has to bear repercussions like repair costs, building closure and in the worst case, the building might not even be repairable and must be demolished, which is a state of absolute loss for the owner as experienced in the recent earthquakes 2010 Chile and 2011 Christchurch (Kam, Pampanin, & Elwood, 2011; Westenenk et al., 2013). This points to the importance of vulnerability assessment of code-designed structures to quantify the structural risk and losses that can help guide the decision makers on seismic risk mitigation.

The advent of Performance Based Earthquake Engineering (PBEE) has made possible the performance-based assessment of buildings for future expected earthquakes in a probabilistic manner (Hamburger & Moehle, 2000; Mahin, 2016; Moehle & Deierlein, 2004). It is a comprehensive methodology for seismic performance assessment of structures, which encompasses hazard analysis for seismic hazard characterization, structural analysis for response evaluation, damage analysis of components for the expected demand and loss estimation for the incurred damages (and the required repairability) in components and structure. The current generation of PBEE allows assessing the seismic performance of structures, quantifying it in reliable measures relevant to the needs of stakeholders e.g. damageability, repair cost, casualties, business downtime, etc. (FEMA P-58, 2012).
The present research adopts the PBEE methodology for seismic performance assessment and direct economic loss estimation of SMRF structures (3, 5, 8, 10 storeys) designed to UBC-97/BCP-2007 for structural damageability and the subsequent repairability cost. The considered structures are analyzed using incremental dynamic nonlinear time history analysis for a suite of seven natural accelerograms compatible to the design spectrum extracted from the PEER NGA database. The economic losses due to direct structural damages are calculated and correlated with the seismic intensity to derive seismic vulnerability curves.

2. Description of structure models

Four reinforced concrete SMRF structures of different heights, representing low to medium to high-rise buildings, have been considered for seismic performance analysis and subsequent economic losses. The selected frames have been designed for seismic Zone 2B of BCP-2007 for a soil type S_0 (as per NEHRP soil classification). The specifications of ACI-318 (ACI-318, 2008) have been used for the design and detailing of structural beam-column members and joints. The selected frames used for analysis are 2D frames extracted from recently designed real moment resisting frames, as shown in Figure 1.

Inelastic numerical models were prepared in finite element based software SeismoStruct (SeismoSoft, 2015), calibrated and recently employed for many applied research related to earthquake engineering and risk mitigation (Formisano & Mazzolani, 2015; Pinho, 2007). Figure 2 shows the idealization and inelastic modelling of considered frame; the inelastic behavior of beam-column members was modeled using force-based plastic hinge elements. Fiber-based elements can simulate the spread of flexure inelasticity along the plastic hinge length of the structural member and can also account for member axial force and moment interaction. Shear behavior of members is not considered, due to the fact that SMRF members respond primarily in flexure and shear response of these members is essentially elastic. Additional mechanism, like bar-slip as noticed for the considered RC SMRF beam members (Ahmad, Rashid, & Waqas, 2016), was modeled using moment-rotation spring at the beam-joint interface. The joints are modeled using elastic beam-column frame elements, due to the fact that these primarily respond elastically when detailed as per the code specifications.

All the column and beam sections have been divided into unconfined and confined concrete. Mander, Priestley, and Park (1988) model was used to model the behavior of confined concrete. The effects of confinement on the enhancement of strength and ductility are automatically accounted for by the analysis software SeismoStruct. The cyclic behavior of longitudinal steel was simulated using the reinforced steel model of Giuffre-Menegotto-Pinto (1983). The considered forced-based element has end plastic hinge zones with a middle elastic portion. The plastic hinge length \( L_p \) was
assumed to be equal to half of the member depth for beams and full depth for columns, which was calibrated with the experimental quasi-static cyclic tests on full-scale beams (Ahmad et al., 2016). The lumped plasticity moment-rotation springs, used for the bar-slip modeling, were assigned with bilinear moment-rotation constitutive relationship, developed based on the experimental quasi-static cyclic tests carried out on full-scale special moment resisting beams at the Earthquake Engineering Center of UET Peshawar (Ahmad et al., 2016) (Figure 3).
3. Selection of ground motions for time history analysis

Seven ground motion records were extracted from the PEER NGA strong ground motions database. A magnitude of $M_w$ 6–7.5, source-to-site distance of 10–30 km, $V_{S30}$ 180–360 m/s were used in the search engine to obtain ground motions compatible with the site seismic hazard. The final ground motions included records taking into account regional and earthquake-to-earthquake variability (Figure 4). The ground motions were scaled and matched, using wavelet based approach in SeismoMatch (SeismoSoft, 2015), to the design spectrum corresponding to seismic Zone 2B and soil type S, for a period range of $0.2T$–$3T$ as suggested by Haselton, Whittaker, Baker, Bray, and Grant (2012), where $T$ corresponds to the first mode vibration period.

4. Development of damage scale

A damage scale was established for beam members based on the quasi-static cyclic tests performed on full-scale special moment resisting beams (Ahmad et al., 2016), which has been linked to the engineering demand parameter (EDP) for damage measure (DM) assessment i.e. identification of beam damage states during non-linear time history analysis for a given seismic intensity. Although, the columns in code designed SMRF structures primarily respond elastically, the damage scale developed by Bearman (2012) for columns was adopted herein.

Experimental investigation was carried out on the repair of damaged full-scale special moment resisting beams for various damage states, using epoxy injection and concrete patching, to derive the beam repair cost ratio (RCR); the ratio of repair cost to specimen construction cost. Figure 5 and

| Event No. | Event Name | Year | Station/Component | $M_w$ | PGA(g) |
|-----------|------------|------|-------------------|-------|--------|
| 1         | Kobe, Japan | 1995 | Abeno             | 6.90  | 0.327  |
| 2         | San Fernando, USA | 1971 | L.A. – Hollywood No 79 | 6.61  | 0.246  |
| 3         | Tabas, Iran | 1978 | Tabas             | 7.35  | 0.252  |
| 4         | Düzce, Turkey | 1999 | Bolu              | 7.14  | 0.418  |
| 5         | Victoria, Mexico | 1980 | Chihuahua        | 6.33  | 0.235  |
| 6         | Spil, Antwerp | 1988 | Chalonon         | 6.77  | 0.500  |
| 7         | L’Aquila, Italy | 1983 | Avazzano         | 6.30  | 0.256  |
Table 1 included the details of moment resisting beams considered herein for investigation. The longitudinal and stirrup reinforcement consisted of ASTM-A615 grade-60 bars. Normal weight concrete with a specified strength of 3,000 psi was used for all the beams. The concrete mix design ratio used was 1:2.13:3.61 with a water-to-cement ratio of 0.57. Standard cylinders of 12 in height and 6 in diameter were tested under compression loading to validate the mix design (Figure 6).

Due to certain limitations of the lab, the beams had to be tested in a vertical position. The beams cantilevered out of an anchorage block, which was anchored to the strong floor of the laboratory using bolts (Figure 7). The lateral load was applied at some distance from the free end of the beam (generally six inches) using a manual-controlled hydraulic actuator. The actuator was attached to the beam using a hinge assembly to allow the rotation of the actuator. A total of 3 LVDTs and one cable displacement sensor were used to record different response parameters of interest (see Figure 7). LVDT 1 and 2 were used to record flexural deformation of the beams whereas LVDT 3 was used to measure the fixed-end rotation resulting from the inelastic extension and slippage of longitudinal bars. The cable displacement sensor (string pot) was used to measure the beam tip displacement resulting from the application of the lateral load.

Two loading protocols were used as part of this study; standard loading history and constant amplitude loading history (Figure 8). The standard loading protocol was used for all specimens except B3K3-R-2.5. It consisted of a series of increasing displacement cycles with three cycles per each displacement increment. The constant amplitude loading protocol was used to study the influence of loading history on specimen response. The control point for the displacement amplitude was the point of application of load by the actuator.

All the tested specimens were observed initially with flexure cracking that spread along the length of beams up to 4.5–5 feet. The cracks widened with further increasing of laterally imposed displacement, however, a wider crack appeared at the beam-block interface that resulted in to fixed-end rotation. Spalling of cover concrete and core concrete disintegration was observed at the maximum lateral displacement demand (Figure 9).

The damage sustained by the beam specimens due to cyclic loadings is classified into two parts; cracking and spalling of the plastic hinge region and fixed-end rotation due to slip and inelastic extension of longitudinal bars. The cracks of plastic hinge region and the interface crack (due to fixed end rotation) were both injected with low viscosity epoxy, which is a general practice in the region

**Table 1. Reinforcement details of the considered specimens for test**

| Specimen ID   | Top/bottom longitudinal bars | Longitudinal reinforcement ratio | Stirrups dia | Stirrups spacing (in.) |
|---------------|-------------------------------|----------------------------------|--------------|------------------------|
| B3K3-R-5.5    | 3-#8                          | 1.26                             | #3           | 4                      |
| B3K3-R-2.5    | 3-#8                          | 1.26                             | #3           | 4                      |
| B3K3-R-3.5    | 3-#8                          | 1.26                             | #3           | 4                      |
| B3K2-R-5.5a   | 2-#8                          | 0.8                              | #3           | 4                      |
| B3K2-R-5.5b   | 2-#8                          | 0.8                              | #3           | 4                      |

Notes: Nomenclature: B = Beam, 3K = 3,000 psi concrete. 3 = # of longitudinal bars in one layer, R = Repair, 5.5 = Max. Applied Disp.
for rehabilitation of reinforced concrete structures. It is important to mention here that none of the specimens exhibited strength drop, as they could not be tested beyond certain displacement amplitude due to lab limitations. The repair cost is represented through RCR and is calculated as the ratio
of the cost of repair of damaged beams of various damage states to the construction cost of beam members. The repair cost includes only the material cost and no consideration is given to the labor and formwork costs. The associated drift limits states were defined in SeismoStruct as performance limit states for member damage identification and the required repair cost estimation.
5. Economic loss estimation
SeismoStruct software and the aforementioned inelastic modeling technique were first employed for test and validation against the experimental investigation. Figure 10 shows the comparison of the analysis carried out using SeismoStruct with that of the experimentally obtained response, which shows better performance of SeismoStruct.

The economic losses for the considered structures were calculated using the PBEE framework for seismic performance assessment. The structures were first analyzed using nonlinear incremental dynamic time history analysis procedures (Vamvatsikos & Cornell, 2002); the matched
accelerograms were scaled to various levels of seismic intensity for structural response analysis. The ground motion records PGA is used as an intensity measure with target levels 0.028, 0.112, 0.196, 0.28, 0.364, 0.448, 0.532, 0.616, 0.700, 0.784, 0.840, 0.980, 1.120 g. Beam-column chord rotation demand was used as the EDP, to identify the damage state of members and the required repair cost.

The considered structures were analyzed for the PGA-based target seismic intensity levels. The analyzed structures were investigated; for each scale factor, the number of beams and columns in a particular damage state were identified to calculate the required repair costs, as per Table 2, and then integrated over the whole structure to compute the total structure repair cost. It is normalized by the total super structure cost to calculate the structure RCR.

\[
\text{RCR}/\text{IM} = \frac{\sum_{i} \sum_{j} (\text{Beam DS}_i \times \text{RCR}_i + \text{Column DS}_j \times \text{RCR}_j)}{\text{Total cost of super structure}}
\]

where DS is the attained damage state of the structural member and RCR is the corresponding required RCR for the damage. The structural RCR is correlated with the intensity measure PGA to develop the vulnerability curves. Figure 11 shows the derived vulnerability curves for the considered structures based on each of the individual ground motion record and the median curves. Furthermore, the mathematical functional form of fragility proposed by Kircher, Nassar, Kustu, and Holmes (1997) is used for fitting to the derived vulnerability curves to facilitate future usability.

\[
\text{RCR}/\text{IM} (\text{PGA} > \text{pga}) = \Phi \left( \frac{1}{\beta} \ln \frac{\text{PGA}}{\text{PGA}_{\text{50}}} \right)
\]

where \(\Phi\) is the standard normal cumulative distribution function, RCR/IM is the structural RCR given the intensity measure IM (PGA), \(\text{PGA}_{\text{50}}\) is the threshold intensity measure that has 50th percentile

| Description | Chord rotation (%) | Confined concrete crushing strain |
|-------------|--------------------|----------------------------------|
| Initial cracking | Moderate cracking | Severe cracking + spalling | Collapse |
| Beam | 0.78 | 2.6 | 4.7 |
| Column | 0.3 | 1 | 1.5 |
| Repair cost ratio (%) | Epoxy injection | Epoxy injection | Epoxy injection + concrete patching | Replacement |
| 0.3 | 0.56 | 0.87 | 1 |
probability, $\beta$ is the logarithmic standard deviation. The above equation is fitted to the derived vulnerability curves to obtain the values of $\text{PGA}$ and $\beta$ for each of the considered structures. Figure 12 shows the median vulnerability curves and fitting through the Equation (2).

The derived vulnerability curves were analyzed to obtain the economic losses of the considered structures for the design-basis earthquake ground motions. For this purpose, the design base earthquake ground motions 0.20g for Zone 2B (having 10% probability of exceedance in 50 years with 475 year return period) was considered as the intensity measure and the corresponding RCR was obtained from the relevant vulnerability curve. This gives RCR of 20.21% for 3 storey, 14.91% for 5 storey, 14.94% for 8 storey and 12.17% for 10 storey structures. The losses in actual buildings can be even more than this, as the indicated values of RCR represent the direct losses due to structural...
components damage only. In a real scenario, these structural losses will be accompanied by losses due to non-structural damages as well and the subsequent closure of building (business downtime) during repair that will amplify the losses.

6. Conclusions and recommendations

Seismic performance assessment of the considered SMRF structures have shown that buildings designed to modern codes may have considerable cost implications upon being subjected to future earthquakes, due to the resulting damage and subsequent repairability (i.e. epoxy injection and concrete patching, as normally practiced). The RCR of structures with low height are relatively more than the medium and high-rise structures (but does not differ very significantly), since the initial cost of medium and high-rise structures are more, the cost implications in medium and high-rise structures will be higher.

The resulting losses are significant, which are generally not known to the designers, and may surprise the owner following the design level earthquake, who is considering his building to be earthquake-proof. The owner may not regard the code-designed building as an earthquake-resistant and may blame the designer for consequences. This was also experienced during the recent 2010 Chile, 2010 Darfield and 2011 Christchurch earthquakes.

This points to the fact that there is a strong need to incorporate performance-based earthquake engineering in structural design practices to provide structural design with various performance objectives i.e. achieving earthquake resistant design with reduced implications of cost, and the same can be communicated to the owner in the design and planning phase of structures to arrive at a mutually agreed design.

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