New earthquake loss assessment framework of RC frame using component-performance-based methodology

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Abstract. New earthquake loss assessment framework of reinforced concrete (or RC) frame using component-performance-based methodology was studied. The damage indicator of structural component angle was measured using elasto-plastic rotation, and the damage-to-loss model was proposed on the basis of the deformation indicator of structural component. Expected earthquake loss of 3-storey RC frame was discussed. The expected earthquake loss included collapse loss, demolition loss and repair loss. The results indicate that ductile RC frame is more likely to be demolished than collapse. The demolition share of total earthquake loss might be more prominent than repair share and collapse share in ductile RC frame.

1. Introduction
Code-conforming RC frame was enough to resist collapse in Ramirez et al[1]. However, massive economic loss was induced in past earthquake. Therefore, earthquake loss needed to be considered. The first-generation performance-based earthquake engineering (or PBEE) evaluated seismic performance mainly based on deformations of structure and component, e.g. FEMA 273[2]. However, the structural seismic performance was described qualitatively. Thus, a new-generation PBEE (or PEER PBEE) was proposed in FEMA P58[3]. Ramirez et al[1] studied expected earthquake loss of RC frame using the PEER PBEE framework. A damage to loss model was proposed in Martins et al[4]. Earthquake loss of old RC frame was studied in O’Reilly et al[5].

Structural macro deformation, e.g. inter-storey drift ratio (or IDR), was considered as the damage index of structural component in most researches mentioned above. However, the IDR was not consistent with the actual structure damage in Ji et al[6]. Although code-conforming RC frame performed well in the matter of resisting collapse, the RC frame may have to be demolished due to severe damage in Ramirez and Miranda[1]. In this paper, the earthquake loss assessment framework using component-performance-based methodology (or ELAFCPM) was presented, which used the component deformation to represent the damage index of structural component. The earthquake loss caused by structural collapse and structural demolition was considered simultaneously.

2. Introduction of ELAFCPM
The ELAFCPM was presented herein, which included four parts: hazard analysis, structural analysis, component damage analysis, and loss analysis.
2.1. Seismic hazard analysis

Seismic hazard curve was proposed as follows in FEMA P58[3].

\[ \lambda(im) = P(IM > im) = k_i m^{-k} \]  

(1)

\( \lambda \) was annual exceedance frequency, and \( im \) was peak ground acceleration (or PGA).

2.2. Structural analysis

Incremental dynamic analysis (or IDA) was used. The dynamic instability was taken as collapse prevention. Moreover, the column failure was considered as the structure need to be demolished in Ji et al[6], which corresponds to the loss of vertical bearing capacity.

2.3. Component damage analysis

2.3.1. Component performance level and acceptance criteria. Fragility groups of RC frame could be divided into structural components, nonstructural components and rugged components.

  **Structural components.** The structural components include slab, beam, column and joint in RC frame. The beam-column joint was assumed to be rigid, and the loss of beam and column were considered according to Martins et al[4]. The slab was taken as rugged component based on FEMA P58[3]. The acceptance criteria of component performance level in FEMA 273[2] was too conservative to most buildings, therefore, the modified acceptable criteria was used in Cui[7]. Moreover, seven performance levels and six performance limits were introduced based on damage extent of components themselves. The performance limits \( P_1 \) and \( P_6 \) correspond to the yield point and failure point, respectively. The \( P_1, P_3, P_5 \) and \( P_6 \) were consistent with the Light damage (DS1), Moderate damage (DS3), Significant damage (DS5), Collapse (DS6) in Ramirez and Miranda[8].

  **Non-structural components and rugged components.** The non-structural components and rugged components were used in FEMA P58[3].

2.3.2. Performance level to loss model. The loss analysis was presented herein. The expected total repair cost \( E(RC|im) \) of RC element “j” was presented as follow.

\[ E(RC_j|im) = \sum_{i=1}^{m} P(L_i|im) \cdot E(C_i|L_i) \]  

(2)

\( m \) was the number of performance levels. \( P(L_i|im) \) and \( E(C_i|L_i) \) denote the probability and expected repair cost of performance level \( L_i \). The median of \( E(C_i|L_6) \) in Table 1 was derived based on Ramirez and Miranda[8]. Similarly, the loss functions of non-structural components could be obtained. The expected total repair cost \( E(C_{rep}|im) \) can be derived.

\[ E(C_{rep}|im) = \sum_{i=1}^{mele} E(RC_i|im) + \sum_{j=1}^{nele} E(NONRC_j|im) \]  

(3)

\( mele \) and \( nele \) were the number of structural components and non-structural components respectively.

The expected total repair cost can be normalized by the building replacement cost \( C_{rpl} \).

\[ R(C_{rep}|im) = E(C_{rep}|im) / C_{rpl} \]  

(4)

Once the structure was in collapse state or demolition.

\[ E(C_{rep}|im) = C_{rpl} \]  

(5)

Table 1. Loss analysis of structural components

| Structural component | Beam | Column |
|----------------------|------|--------|
| Performance limit    | <\( P_1 \) | \( P_1 \) | \( P_2 \) | \( P_3 \) | \( P_4 \) | \( P_5 \) | \( P_6 \) | <\( P_1 \) | \( P_2 \) | \( P_3 \) | \( P_4 \) | \( P_5 \) | \( P_6 \) |
| Performance level    | \( L_1 \) | \( L_2 \) | \( L_3 \) | \( L_4 \) | \( L_5 \) | \( L_6 \) | \( L_7 \) | \( L_1 \) | \( L_2 \) | \( L_3 \) | \( L_4 \) | \( L_5 \) | \( L_6 \) | \( L_7 \) |
| \( E(C_i|L_6) \)    | 0.0   | 0.1   | 0.2   | 0.3   | 0.4   | 1.0   | 1.0   | 0.0   | 0.1   | 0.3   | 0.5   | 1.2   | 2.0   | --    |

*The column failure is considered as the structure has to be demolished herein.*
2.4. Loss analysis
The expected total loss $E(\text{loss}|\text{im})$ of RC frame could be established under $IM=\text{im}$.

$$E(\text{loss}|\text{im}) = P(\text{cp}|\text{im})C_{\text{rpl}} + P(\text{de}|\text{im})C_{\text{del}} + P(\text{rep}|\text{im})E(C_{\text{rep}}|\text{im})$$  \hspace{1cm} (6)

$P(\text{cp}|\text{im})$, $P(\text{de}|\text{im})$ and $P(\text{rep}|\text{im})$ were the probabilities of structure collapse, structure demolition and structure repair, respectively. It was assumed that $C_{\text{rpl}}=C_{\text{del}}$ in Ramirez and Miranda[1]. Furthermore,

$$P(\text{cp}|\text{im}) + P(\text{de}|\text{im}) + P(\text{rep}|\text{im}) = 1$$  \hspace{1cm} (7)

The expected total loss ratio $R(\text{loss}|\text{im})$ can be derived.

$$R(\text{loss}|\text{im}) = \frac{E(\text{loss}|\text{im})}{C_{\text{rpl}}}$$  \hspace{1cm} (8)

The expected annual loss $E(\text{loss})$ could be derived over the seismic hazard curve

$$E(\text{loss}) = \int_0^\infty E(\text{loss}|\text{im})d\lambda(\text{im})$$  \hspace{1cm} (9)

The expected annual loss ratio $R(\text{loss})$ was also obtained.

$$R(\text{loss}) = \frac{E(\text{loss})}{C_{\text{rpl}}}$$  \hspace{1cm} (10)

3. Case study

3.1. Architecture archetype
3-storey RC frame was depicted in Fig. 1(a). Seismic hazard curve was presented in Fig. 1(b). Linear load was 30.5kN/m on beams. In addition, the building cost distribution of fragility groups each floor was provided in Table 2 based on Ramirez and Miranda [8], and the building cost was $640/m^2.

![3-storey RC frame](a)

| Fragility group | Beam | Column | Partition | Partition-like | Window | Generic drift | Generic acceleration | Ceiling |
|-----------------|------|--------|-----------|----------------|--------|--------------|---------------------|--------|
| 1-storey        | 4.8  | 2.1    | 18.3      | 14.3           | 7.3    | 8.4          | 26.3                | 5.8    |
| 2-storey        | 5.0  | 2.2    | 18.5      | 13.9           | 6.3    | 7.8          | 27.1                | 6.5    |
| 3-storey        | 4.1  | 1.8    | 13.9      | 11.9           | 6.5    | 8.1          | 33.0                | 3.1    |

3.2. IDA results

3.2.1. IDA analysis. The structure in Fig. 1(a) was simulated in OpenSEES. In order to realize IDAs, 22 earthquake records were used and introduced in Qiao et al[9]. Rayleigh damping was 5%. The IDA curves were presented in Fig. 2(a), and the collapse points as well as demolition points were depicted
accordingly. Moreover, the damaged state distributions of structure collapse (CP), structure demolition (DEMO) and structure repair (REPAIR) were demonstrated in Fig. 2(b).

3.2.2. Loss analysis. Expected earthquake loss ratio of each PGA under each earthquake record can be calculated using Eq. (11), hence, the distributions of structural component (STR) loss, nonstructural component (NONSTR) loss and rugged component (RUGGED) loss were demonstrated in Fig. 3(a). The expected annual loss of STR, NONSTR and RUGGED was $1.06 \times 10^{-4}$, $4.96 \times 10^{-4}$ and $0.55 \times 10^{-4}$, respectively. Additionally, the earthquake loss ratio distributions of repair(REPAIR), demolition(DEMO) and collapse(CP) were presented in Fig. 3(b). The earthquake loss disaggregation different damage state was also derived in Table 3 under rare earthquake ($PGA=0.220g$) and very rare earthquake ($PGA=0.425g$).

Table 3. Earthquake loss disaggregation of different damage state

| Damage state           | Repair (%) | Demolition (%) | Collapse (%) |
|------------------------|------------|----------------|--------------|
| Rare earthquake (0.220g) | 37.1       | 40.0           | 22.9         |
| Very rare earthquake (0.425g) | 9.4        | 53.3           | 37.3         |

Figure 2. IDA curves

Figure 3. Expected earthquake loss analysis
4. Results and discussion
In Fig. 2(b), the cumulative collapse probability increases slowly with PGA less than 0.40g, however, the curve of demolition probability enhances sharply and the corresponding peak point was 54.6% with PGA of 0.40g. It could be deduced that structure is more likely to be demolished than collapse.

In Fig. 3(a), the histogram of non-structural loss was steeper than the structural loss, and the share of non-structural loss in expected annual loss ratio was 75.5%. That is to say, the non-structural loss was more remarkable to promote the increase in total loss. Further, the share of non-structural loss in total loss was dominating.

In Fig. 3(b), the loss caused by structure demolition improves more rapidly. The demolition percents were 1.08 times and 5.67 times of the repair percents under rare earthquake in Table 3, as well as 1.75 times and 1.43 times under very rare earthquake. It could be gleamed that the demolition percent may be more prominent than repair percent and collapse percent, especially less than the PGA of very rare earthquake. It can also be demonstrated in Ramirez and Miranda[1]. Therefore, the loss caused by demolition should be highlighted.

5. Conclusions
The ELAFCPM was presented for RC frame. The elasto-plastic rotation angle was taken as the damage index of structural component, and the RC frame has to be demolished in case of the bearing capacity loss in column. According to the damage index of structural component, the damage-to-loss model was introduced. Moreover, the earthquake loss caused by demolition was also considered. Expected earthquake loss of 3-storey frame was studied. Some conclusions could be drawn here.

Code-conforming RC frame structure is very difficult to reach collapse under rare earthquake. However, the frame structure is more likely to be demolished.

The non-structural loss is more remarkable to promote the increase in total earthquake loss, and the share of non-structural loss in total loss is dominating.

The loss percent suffering from demolition may be more prominent than repair percent and collapse percent, especially PGA less than very rare earthquake. Hence, the loss caused by demolition should be considered.

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