Development of seismic fragility curves for reinforced concrete tall buildings

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ABSTRACT: This study concentrates on the probabilistic risk assessment of reinforced concrete tall buildings subjected to ground motion excitation. In order to assess the vulnerability of tall buildings, twelve-, and twenty-four-story four-bay reinforced concrete moment resisting frames are studied. This evaluation is done through developing fragility curves. These fragility curves provide the probability of exceeding the multiple damage states for a given intensity (e.g. CAV) of ground motion excitation. To develop the fragility curves, a great number of nonlinear dynamic analyses are conducted in the open-source platform OpenSees. Since the PEER Guidelines for Performance-Based Seismic Design of tall buildings recommends to consider the effect of soil-structure interaction, SSI, in tall buildings using a response history substructure analysis of them, this study includes comparison between fragility curves of fixed base buildings and the ones derived from models considering the SSI effects to indicate the efficacy of that. The structural uncertainties are taken into account by generating and modeling random values of material properties using Monte Carlo simulation method. Former investigations show that considering uncertainty in soil properties such as friction angle, cohesion, density, shear modulus and Poisson's ratio makes a negligible difference in seismic demand, so we pass up this uncertainty. Four sets of seven earthquake accelerograms corresponding to type C & D soils (based on NEHRP classification) for far and near sources are selected to be used in the analysis. Finally the developed fragility curves can be used to assess the seismic performance of tall buildings and discern the effect of SSI, height of structures and soil type on seismic response of this type of structures. An increase in the probability of exceedance of different damage states is observed in the cases of considering SSI effects, type D soils and near-source accelerograms.

KEY WORDS: Tall buildings; Seismic fragility curves; Soil-Structure Interaction; Monte Carlo simulation; Damage index

1 INTRODUCTION

In past decades, several costly and destructive earthquakes have occurred and this phenomenon is going to recur. Trying to predict and reduce the risks and consequences of this natural disaster is the only thing that can be done. The earthquakes and fragility of structures due to these natural disasters cause risks. Both risk components have inherent uncertainties including structural and seismic uncertainties. Therefore it is necessary to assess the seismic performance and vulnerability of buildings for a given seismic parameter by employing these uncertainties. Tall buildings are specific types of buildings which are extensively used in seismically active regions, so it is necessary to conduct seismic risk assessments of these buildings. The vulnerability assessment is formerly done through developing fragility curves and relationships for some specific types of structures such as bridge piers, masonry and reinforced concrete structures, etc. This method is also applicable to the types of buildings we assess. Analytical fragility curves corresponding to a specific damage state define the probability of exceedance of the specified damage state for a given intensity of the seismic parameter of ground motion. In this investigation, fragility curves are obtained using nonlinear dynamic analysis procedures, specifically IDA method.

As cumulative absolute velocity (CAV) is found to be well associated with structural damage [1], it is selected as seismic parameter in this study. This parameter is calculated using equation (1):

\[ CAV = \int_0^T |a(t)| \, dt \]  

Tall buildings' seismic performance is influenced by several factors such as soil type, soil-structure interaction, near source ground motions.

Soil-Structure Interaction (SSI) has often a significant effect on dynamic behavior of the super structure resting on soft soils. According to Wolf [2], this effect is usually assessed in two separated parts: 1. the Kinematic interaction effect, 2. the inertial interaction effect. The first part modifies the input ground motion to the foundation as a result of rigidity of foundation against soil. The second part includes the effect of inertial loads created by vibration of structures which will cause a new motion in the foundation and accordingly deformation in the soil. This part will also influence the input ground motion to the foundation. The effect of the kinematic interaction versus inertial interaction on the structures with surface foundation is negligible [2], hence the kinematic interaction is not considered in this study. Figure 1 shows these effects schematically.

This study focuses on developing fragility curves for tall buildings and assessing the effect of SSI, height of structure
and near source ground motions on the vulnerability of such buildings.

![Image](image1)

Figure 1. Soil – Structure Interaction (SSI). (a) free-field motion. (b) Kinematic interaction effect. (c) Inertial interaction effect [3]

2 MODEL DESCRIPTION AND ANALYSIS

2.1 Description of buildings

In this study, twelve- and twenty-four-story four-bay RC moment resisting frame buildings were designed according to Iranian seismic code (Standard 2800). All of the bay widths are 6 meter. These buildings are regular and symmetric in plan and height, so as a representative, one of the interior frames is picked out and analyzed for each building. 12-, 24-story buildings have been designed considering a response reduction factor, R, of 7 and 10 corresponding to intermediate and special moment frames, respectively. The 24-story building is designed based on response spectrum analysis due to the Code requirements. The cross sections of all columns are quadrangular and the beams are rectangular with an aspect ratio of 1.5.

In order to conduct dynamic nonlinear response history analysis, the structures are modeled in the open source program, OpenSees. The beam and columns are modeled using nonlinear beam-column elements defined in program and the P-Δ effect has been considered for columns. The concrete behavior is modeled using Concrete02 material which considers tensile strength and linear tension softening. As depicted in Figure 2, stiffness degradation, strength loss and pinching are considered in the hysteretic behavior of Concrete02. The confinement effect due to transverse reinforcement is considered according to formulation developed by Mander et al. Steel behavior is modeled using Steel02, a uniaxial Giuffre-Menegotto-Pinto model. The hysteretic behavior of steel02 is shown in Figure 3. Beam and columns’ sections are taken as fiber sections, in which the section is divided into smaller segments and yield a resultant behavior. In order to consider the increase in strength and strain values due to confinement, the core and cover materials are defined separately.

![Image](image2)

Figure 2. Hysteretic behavior of Concrete02 [4]

![Image](image3)

Figure 3. Hysteretic behavior of Steel02 [4]

2.1.1 Compressive stress-Strain behavior of confined concrete

The confinement effect due to transverse reinforcement is not considered automatically in this platform, thus it is necessary to model this effect manually. Among different models considering confinement, it is found that Mander et al. model is suitable to be utilized [5]. The stress-strain model is illustrated in Figure 4.

Parameters shown in Figure 4 are calculated using equations (2) through (5) and Figure 5 for rectangular sections.
2.2 Soil – Structure Interaction (SSI)

As mentioned before, SSI is assessed in two different parts: (1) Inertial interaction (2) Kinematic interaction. Former investigations show that the inertial interaction is more effective on the seismic performance of the buildings than kinematic interaction in surface foundations. According to Wolf [2], there are two different approaches that can evaluate the effect of SSI. These methods can be categorized into direct and substructure categories. In the direct approach, both soil and structure are modeled as a complete system and free-field motion is applied to the artificial boundaries defined by respective methods. In the substructure method, soil and structure are partitioned into two portions. At first a force-displacement relationship (dynamic stiffness) is considered for the soil and it is physically modeled by some spring and dampers. The spring and dampers’ characteristics are dependent on the excitation frequency. Second, the structure resting on these spring and dampers as support, will be modeled and finally the ground motion excitation will be applied to them.

In this study, the foundation on the surface of a homogeneous half-space soil is modeled using the cone model presented by Wolf and equivalent spring and dashpots' characteristics are calculated. The cone models are divided into two parts, translational and rotational cones. The translational cone model considers vertical and horizontal motion, and the rotational cone model considers rocking and torsion [6]. Figure 6 shows the corresponding lumped-parameter models used here:

![Diagram](image1)

Figure 4. Stress-strain model proposed for monotonic loading of confined and unconfined concrete

\[
\varepsilon_{cc} = \varepsilon_{cc0} \left[1 + 5 \left(\frac{f'_{cc}}{f_{cc0}} - 1\right)\right]
\]

(2)

\[
\varepsilon_{cu} = 0.004 + \frac{1.4\rho_x f_y h \varepsilon_{sm}}{f'_{cc}}
\]

(3)

\[
f'_{ck} = k_e \rho_x f_y h , \quad f_{ky} = k_e \rho_y f_y h
\]

(4)

\[
k_e = \left(1 - \sum_{i=1}^{n} \left(\frac{w_i}{h_i/d_i}\right)^2\right) \left(1 - \frac{s'}{2d_c}\right) \left(1 - \frac{s'}{2d_c}\right)
\]

(5)

Where \(f'_{cc}\) and \(\varepsilon_{cc}\) are the confined concrete strength and corresponding strain, \(k_e\) is the confinement effectiveness coefficient, \(f'_{ck}\) is the effective lateral confining stress on concrete, \(w_i\) is the ith distance between longitudinal bars, \(\rho_{cc}\) is the ratio of longitudinal bars’ area to the core area of the section and \(s'\) is the distance between transverse bars.

![Diagram](image2)

Figure 5. Confined strength determination from lateral confining stresses for rectangular sections [5]

![Diagram](image3)

Figure 6. Cone model and corresponding lumped-parameter models for foundation on surface of homogeneous half-space. (a) Truncated semi-infinite cone. (b) Spring-dashpot-mass model for translational degree of freedom. (c) Spring-dashpot-mass model for rotational degree of freedom [6]
As a two-dimensional analysis is conducted in this study, only the horizontal and rocking degrees of freedom have been considered. Formulas used to determine the related spring and dashpot values are presented in Table 1.

<table>
<thead>
<tr>
<th>Motion</th>
<th>Horizontal</th>
<th>Rocking</th>
</tr>
</thead>
<tbody>
<tr>
<td>Equivalent radius</td>
<td>$r_0$ = $\sqrt{\frac{A_0}{\pi}}$</td>
<td>$r_0$ = $\sqrt{\frac{4I_0}{\pi}}$</td>
</tr>
<tr>
<td>Aspect ratio</td>
<td>$\frac{S_0}{r_0} = \frac{\pi}{8} (2 - \nu) \left( \frac{9\pi (1 - \nu) v^2}{2v_0^2} \right)$</td>
<td></td>
</tr>
<tr>
<td>Poisson's ratio</td>
<td>all $\nu$</td>
<td>$\nu \leq \frac{1}{3}$, $\frac{1}{3} &lt; \nu$</td>
</tr>
<tr>
<td>Wave velocity</td>
<td>$v_s$</td>
<td>$v_p$</td>
</tr>
<tr>
<td>Trapped mass</td>
<td>$\Delta M$</td>
<td>$\Delta M_{\varphi}$</td>
</tr>
<tr>
<td>$\varphi$</td>
<td>$0$</td>
<td>$-\frac{1}{3} \rho_0 r_0$</td>
</tr>
<tr>
<td>Lumped parameter model</td>
<td>$K = \rho v_0^2 A_0 z_0$</td>
<td>$K_\varphi = 3\rho v_0^2 l_0 z_0$</td>
</tr>
<tr>
<td></td>
<td>$C = \rho v_0 A_0$</td>
<td>$C_\varphi = \rho v l_0$</td>
</tr>
</tbody>
</table>

Where $\rho$ is the mass density, $A_0$ is the area of foundation and $I_0$ is calculated using $\pi r_0^2/2$.

In this study, it is assumed that $\rho$ is 2000 kg/m$^3$ and Poisson’s ratio is 1/3. The wave velocity is obtained from utilized accelerograms’ characteristics.

These spring and dashpots are modeled in OpenSees using zero length elements with respective characteristics and the excitation is applied to them.

### 2.3 Uncertainty

According to Wen et al. (2003) [7], uncertainties are classified into two categories: (1) Aleatory uncertainties, such as inherent variations in material properties and wind loads, these uncertainties cannot be reduced through collecting additional information (2) Epistemic uncertainties, due to lack of knowledge and incorrect modeling. These uncertainties can be reduced by obtaining more information.

In this study, two types of uncertainties are considered: (1) structural uncertainty including inherent variations in concrete compressive strength and modulus of elasticity, steel tensile yield strength and damping ratio, (2) seismic uncertainty due to the difference in the event magnitude, site to source distance, path attenuation and site effects.

#### 2.3.1 Structural uncertainty

Monte Carlo simulation method is used to generate random values corresponding to each structural parameter based on their probability distribution functions.

#### 2.3.2 Seismic uncertainty

In this study, four sets of seven earthquake accelerograms corresponding to type C & D (based on NEHRP classification) soils and for far-field and near-source locations are selected to consider the seismic uncertainty. Three criterions are considered to select the records which are the event magnitude, distance to source and site soil type. Quantitatively, records with PGV greater than 15 cm/sec, magnitude greater than 6 on the Richter scale and 10 kilometers distance as a mediocrity for far-field and near-source records are opted. As a two-dimensional analysis is conducted, the components of records which have greater PGV values are used to analyze the structures. In order to prevent unnecessary calculations, the duration of strong ground motion is defined using Trifunac-Brady method, so the duration of records is shortened.

The earthquake accelerograms used to analyze the structures considering SSI effect are recorded on soil type D with shear wave velocities about 300 m/sec.

#### 2.4 Damage index

In order to quantify the vulnerability of reinforced concrete structures, different indices are presented by scholars. The damage indices are categorized into two groups: (1) Local damage indices (2) Global damage indices. Local damage indices specify damage measure in a member or at a joint during an earthquake, while global damage indices specify the overall damage measure of the structure based on the distribution and severity of the local damage. In order to consider the effect of both the repeated cycles of deformation and large deformation of elements, damage indices called combined damage indices are used as local damage indices. These damage indices are linear combinations of the normalized parameters mentioned before, deformation and...
repeated cycle of deformations. The effect of repeated cycles of deformation is considered by calculating the absorbed hysteretic energy. The combined damage index provided by Park and Ang is commonly used in former investigations [10], [11]. In this study, a slightly modified Park and Ang damage index is used which is defined using equation (6) [12]:

$$DI = \frac{\phi_m - \phi_y}{\phi_u - \phi_y} + \beta \frac{\int dE}{M_y \phi_u}$$

(6)

Where $\phi_m$ and $\int dE$ are obtained using nonlinear dynamic analysis outputs, $\beta$ is considered to be 0.15. $\phi_y$, $\phi_u$ and $M_y$ values are calculated using the moment-curvature analysis of the reinforced concrete section. In order to conduct this analysis, a zero-length element with a section equivalent to the defined cross section for beam and column elements is modeled. The model consists of two nodes and a zero length element. The defined element is schematically shown in Figure 7. The left part of the figure shows an edge view of the element. The utilized section is depicted in the right part of this figure. Node 1 is completely restrained, while the applied loads act on node 2. The existing compressive axial load is applied to the section during the moment curvature analysis [4].

The cross sections defined in modeling the structures are used as the cross section of the zero-length element. The beams' axial loads are considered to be zero and columns' axial loads are measured using design load combinations.

Figure 8 shows a sample moment-curvature analysis result for one of the beams used in modeling.

$\phi_y$ and $M_y$ Values obtained through this method has a good adaptation to values calculated using experimental relationships presented by Park and Paulay, but $\phi_u$ values are a little different. This difference is owing to utilizing fiber sections in this model and also simplifications done in experimental relationships.

Thereby, the damage index of elements can be calculated. Hence, the damage distribution in stories can be defined and finally the overall damage index is calculated according to absorbed energy in each story ($\lambda_i$). This method is formulated in equations (7), (8).

$$DI_{storey} = \sum(\lambda_i)_{element} (DI)_{element}$$

(7)

$$; (\lambda_i)_{element} = \frac{E_i}{\sum E_i}_{element}$$

$$DI_{overall} = \sum (\lambda_i)_{storey} (DI)_{storey}$$

(8)

$$; (\lambda_i)_{storey} = \frac{E_i}{\sum E_i}_{storey}$$

2.5 Damage states

Park & Ang proposed the following damage classification based on the tests done:
Table 2. Damage states presented for the above damage index

<table>
<thead>
<tr>
<th>The overall damage index</th>
<th>Structure condition</th>
</tr>
</thead>
<tbody>
<tr>
<td>$0 &lt; DI &lt; 0.1$</td>
<td>No damage or localized minor cracking</td>
</tr>
<tr>
<td>$0.1 \leq DI &lt; 0.25$</td>
<td>Minor damage – light cracking throughout</td>
</tr>
<tr>
<td>$0.25 \leq DI &lt; 0.4$</td>
<td>Moderate damage – severe cracking, localized spalling</td>
</tr>
<tr>
<td>$0.4 \leq DI &lt; 1.0$</td>
<td>Severe damage – crushing of concrete, reinforcement exposed</td>
</tr>
<tr>
<td>$DI \geq 1$</td>
<td>Collapsed</td>
</tr>
</tbody>
</table>

The median values of the ranges defined in Table 2 are used to estimate the probability of exceedance of each damage state.

2.6 Fragility derivation

In order to assess the vulnerability of the buildings due to different ground motions, damage state definitions are applied to obtained damage indices. So the probability of exceedance of different damage states can be calculated for specified values of seismic parameter ($CAV$, in this study). $PF_{ij}$, the probability of exceeding the $i$th damage state for the occurrence of an earthquake with $CAV$ equal to $v_j$ can be determined using the equation (9):

$$PF_{ij} = \text{Prob} (DI \geq DI_i | CAV = v_j)$$
$$= 1 - \text{Prob} (DI < DI_i | CAV = v_j)$$
$$= 1 - F_{DI}(DI_i | CAV = v_j)$$

(9)

Where $DI_i$ is the damage index corresponding to $i$th damage state and $F_{DI}$ is probability distribution function of $DI$. In this study, it is assumed that the damage index has a lognormal probability distribution. The equation (10) expresses this distribution:

$$PF_{ij} = 1 - \phi \left( \frac{\ln(DI_i) - \ln(DI)}{\sigma_{\ln(DI)}} \mid CAV = v_j \right)$$

(10)

Where $\phi()$ is the standard normal distribution function, $\ln(DI)$ and $\sigma_{\ln(DI)}$ are logarithmic mean value and standard deviation of damage index, respectively. Thereby, Fragility curve for the $i$th damage state can be plotted using $PF_{ij}$ values for each level of $CAV$ [13].

3 RESULTS AND DISCUSSION

Through processing the results of the previous sections, fragility curves are developed for sample tall reinforced concrete moment resisting frames for near-source and far-field records, considering and not considering SSI.
By assessing the fragility curves in Figure 9 to Figure 15, it can be clearly observed that increase in height of these types of structures leads to decrease in vulnerability of the structures. The near-source earthquakes may cause more damage to structures than far-field earthquakes and this effect is more significant in soil type C versus soil type D and in 24 story building versus 12 story structure. The vulnerability of these types of structures resting on soil type D is greater than structures resting on soil type C. The soil type effect is more significant in 24-story building. It is also clear that SSI causes a little increase in the probability of exceedance of different damage states for 12- and 24-story building rested on the considered soil type D, however it is more effective on 24-story building. Both increase and decrease in vulnerability of tall buildings in case of considering SSI effect are observed for different ground motion records which shows that the results are dependent on the seismic input and soil condition. So we can say that the SSI effect on tall buildings is generally negligible, and that’s because the period lengthening in tall buildings is near unity (i.e., little or no period lengthening).

4 CONCLUSION

In this study, fragility curves for reinforced concrete tall moment resisting frames buildings were developed from great numbers of nonlinear dynamic analysis. The vulnerability of elements was specified using a slightly modified cumulative combined damage index provided by Park & Ang and thereby the damage measure of the structure was calculated. The evaluations of different damage indices’ values showed that seismic uncertainty has the most significant impact on the seismic behavior of the structures among all other uncertainties considered in this research. Hence, it is necessary to ponder this source of randomness in the risk assessment of the structures.

On the other hand, based on the obtained results, the effect of SSI was negligible for these types of building and soil conditions, nevertheless it is essential to consider this effect for other types of tall buildings such as buildings with other
lateral bearing system (e.g. buildings with a dual core wall system) and buildings with embedded foundations.

Finally the presented fragility curves and relationships are applicable to this specific type of tall buildings and additional investigations are needed to obtain general fragility relationships for other types of structures.

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