COLLAPSE ASSESSMENT OF VULNERABLE REINFORCED CONCRETE STRUCTURES UNDER EARTHQUAKES

C. Song* and S. Pujol**

ABSTRACT

There are approximately 17,000 reinforced concrete (RC) buildings built before 1980 in high-seismicity regions in California. These structures were not built to meet modern requirements for building performance during an earthquake. Assessing all these buildings in detail to determine which need to be fixed will have a prohibitive cost. Efficient screening methods are needed to identify the most vulnerable structures in this large inventory of older buildings.

This study focused on low- to mid-rise RC frames (with 6 or fewer stories) without shear walls. The ability of six seismic vulnerability indicators (SVI) and nonlinear dynamic analysis (NDA) to rank existing RC structures with respect to their seismic vulnerability was studied. The SVIs include: 1) column index, 2) ratio of moment capacities of columns and beams, 3) ratio of column shear capacity to plastic shear demand, 4) axial load ratio for 1st-story columns, 5) ratio of building initial fundamental period to number of stories, 6) ratio of building base-shear demand to base-shear strength. The six SVIs were first evaluated using numerical analysis of hypothetical frames. The SVIs and NDA were then evaluated against field observations collected from 1992 Erzincan, Turkey, and 2015 Kathmandu, Nepal earthquakes.

The evaluation described shows column index (CI) produced the best correlation between observed frequency of damage and estimated vulnerability. At the same time, CI is the index that requires the least amount of both a) information about the structures, and b) computational effort. It was also observed that buildings with lower CI have higher frequency of severe damage. This suggests retrofit work should start in the buildings with lowest CI.

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*Graduate Engineer, Walter P. Moore and Associates, Inc., 707 Wilshire Boulevard, Suite 2100, Los Angeles, CA 90017 (email: csong@walterpmoore.com)
**Professor, Lyles School of Civil Engineering, Purdue University, 550 W Stadium Ave, West Lafayette, IN 47907

Collapse Assessment of Vulnerable Reinforced Concrete Structures under Earthquakes

C. Song\textsuperscript{1} and S. Pujol\textsuperscript{2}

ABSTRACT

There are approximately 17,000 reinforced concrete (RC) buildings built before 1980 in high-seismicity regions in California. These structures were not built to meet modern requirements for building performance during an earthquake. Assessing all these buildings in detail to determine which need to be fixed will have a prohibitive cost. Efficient screening methods are needed to identify the most vulnerable structures in this large inventory of older buildings. This study focused on low- to mid-rise RC frames (with 6 or fewer stories) without shear walls. The ability of six seismic vulnerability indicators (SVI) and nonlinear dynamic analysis (NDA) to rank existing RC structures with respect to their seismic vulnerability was studied. The SVIs include: 1) column index, 2) ratio of moment capacities of columns and beams, 3) ratio of column shear capacity to plastic shear demand, 4) axial load ratio for 1st-story columns, 5) ratio of building initial fundamental period to number of stories, 6) ratio of building base-shear demand to base-shear strength. The six SVIs were first evaluated using numerical analysis of hypothetical frames. The SVIs and NDA were then evaluated against field observation collected from 1992 Erzincan, Turkey, and 2015 Kathmandu, Nepal earthquakes. The evaluation described shows column index (CI) produced the best correlation between observed frequency of damage and estimated vulnerability. At the same time, CI is the index that requires the least amount of both a) information about the structures, and b) computational effort. It was also observed that buildings with lower CI have higher frequency of severe damage. This suggests retrofit work should start in the buildings with lowest CI.

Introduction

Reinforced concrete buildings constructed before 1980 often show poor seismic performance and their vulnerability has become a rising concern. These buildings, also referred to as “older concrete buildings”\textsuperscript{7} (ATC, 2011), were not designed to have ductile performance because of limitations in building codes used before 1980. Surveys (Anagnos et al., 2008, Comartin et al., 2011) showed that there are approximately 17,000 pre-1980 concrete buildings in counties of California with highest seismicity and 1,600 pre-1980 concrete buildings in the city of Los Angeles. It should be noted that not all these buildings are hazardous. To retrofit all older concrete buildings is not necessary and will have a prohibitive cost. Identifying the most vulnerable structures in a large inventory is an urgent challenge.

The objective of this study was to evaluate screening methods that are simple and efficient

\textsuperscript{1} Graduate Engineer, Walter P. Moore and Associates, Inc., 707 Wilshire Boulevard, Suite 2100, Los Angeles, CA 90017 (email: csong@walterpmoore.com)

\textsuperscript{2} Professor, Lyles School of Civil Engineering, Purdue University, 550 W Stadium Ave, West Lafayette, IN 47907

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so that they can be used to assess a large inventory of buildings quickly. Further inspections and evaluations can be done to buildings ranked as being more vulnerable. Seven screening methods including six seismic vulnerability indicators (SVIs) and nonlinear dynamic analysis (NDA) were investigated. The selected methods were used to rank low- to mid-rise buildings (with 6 or fewer stories) with reinforced concrete frames. Buildings with shear walls were not considered.

Selected Evaluation Methods

Column Index

Column index, first introduced by Hassan and Sozen (1997), is defined as the ratio of effective column cross-sectional area at base to total floor area including all stories:

\[ \text{CI} = \frac{A_{ce}}{A_{ft}} \times 100\% \]  

where \( A_{ce} \) is the effective cross-sectional area of columns at base (50 percent of total column cross-sectional area at base); \( A_{ft} \) is the total floor area including all stories. The column index is of interest because it is simple and requires minimum building information and computation effort. It was observed to have good correlation with field observations (Hassan and Sozen, 1997, O’Brien et al., 2011, Zhou et al., 2013). Because this index does not capture all building details that may affect seismic response, engineers may be reluctant to use it. However, it provides a quick measure of both building strength and stiffness, and the field data suggested that it may be an efficient and promising screening method.

Ratio of Moment Capacity of Columns to Beams

The ratio of column-to-beam moment capacity is related to the design concept of “strong-column weak-beam” for moment frame structures resisting seismic loads. The ratio of column-to-beam moment capacity is expressed as \( \frac{\sum M_c}{\sum M_b} \) in this study. \( \sum M_c \) and \( \sum M_b \) are the sums of moment capacities of columns and beams at a joint. The value of \( \frac{\sum M_c}{\sum M_b} \) assigned to a building is the “minimum floor mean value”: the value of a floor is taken as the mean of all interior joints. For multiple-story buildings, the value of a building is the minimum of the mean values of each floor except the roof. It should be noted that the values of \( \frac{\sum M_c}{\sum M_b} \) for multi-story and one-story buildings cannot be compared directly. Joints at roof level have different numbers of connecting columns and beams than joints at intermediate levels.

Ratio of Column Shear Capacity to Plastic Shear Demand

The ratio of column shear capacity to plastic shear demand is related to the vulnerability of a column to shear failure. This indicator is expressed as \( V_n/V_p \) in this study, where \( V_n \) is column nominal shear strength and \( V_p \) is column plastic shear demand. The column nominal shear strength \( V_n \) is calculated following ACI guidelines (ACI 318-14, Eqn. 22.5.6.1). The column plastic shear demand \( V_p \) is estimated as \( 2 M_p/h \), where \( h \) is the clear column height and \( M_p \) is column plastic moment capacity. \( M_p \) was estimated as 1.2 times column yield moment capacity \( M_y \). The \( V_n/V_p \) value of a building is taken as the “minimum story mean value”.
Axial Load Ratio for First-story Columns

The axial load ratio for first-story columns is expressed as $P / f'_c A_g$, where $P$ is axial load from gravity loads; $f'_c$ is concrete compressive strength; $A_g$ is column gross cross-sectional area. Axial loads were estimated by multiplying tributary floor area and an assumed distributed weight. The axial load ratio assigned to a building was taken as the maximum $P / f'_c A_g$ of all columns in the first-story.

Ratio of Building Initial Fundamental Period to Number of Stories

The ratio of building initial fundamental period to number of stories, expressed as $T / N$, is of interest as it provides an approximate estimate of building drift ratio demand for a given ground motion (Lepage, 1997). $T$ is building initial fundamental period computed assuming uncracked sections, and $N$ is number of stories. An exact method for prediction of drift may be out of reach today because it has to start with inexact ground-motion information. The method proposed by Lepage (1997) suggested that for a given ground motion intensity, building inter-story drift ratio (SDR) associated with nonlinear response can be estimated as proportional to the ratio of $T / N$. Therefore $T / N$ could be used as a vulnerability indicator.

Ratio of Building Base-shear Demand to Base-shear Strength ($R$ factor)

The ratio of building base-shear demand to base-shear strength was estimated as:

$$R = \frac{S_a(T) \times (W_b / g)}{V_{max}}$$  \hspace{1cm} (2)

where $S_a(T)$ is linear spectral acceleration for the building initial fundamental period; $W_b$ is the building weight; and $V_{max}$ is building base-shear strength calculated from analysis. This ratio is referred to as the $R$ factor because it is analogous to the response modification coefficient $R$ defined in most building codes. In evaluating building vulnerability, the $R$ factor is expected to produce a measure of ductility demand caused by ground motion. Buildings with higher $R$ require higher ductility to resist ground motions. Older concrete buildings often do not have adequate detailing, so their ductility can be limited.

Nonlinear Dynamic Analysis (NDA)

NDA was also evaluated as a method to estimate building seismic vulnerability. If both building details and resources to cover computational costs are available, NDA is considered by many engineers a reliable method for seismic evaluation. There is no widely accepted method for conducting a nonlinear dynamic analysis for seismic evaluation. The numerical analyses in this study were conducted using the software OpenSees (McKenna et al., 2000). Details of the analysis process and modeling assumptions are described in the numerical study sections below.

Numerical Study on Hypothetical Frames

Nonlinear dynamic analysis (NDA) was used as an initial reference to evaluate the SVIs described
The details of hypothetical frames, numerical models, and dynamic analysis procedures are described in the sections below.

**Numerical Models of Hypothetical Frames**

The hypothetical frames used in this study were variations of a prototype building designed for the Seattle area (ATC, 2011). The prototype building had six stories and a rectangular plan with five bays in each direction. It consisted of reinforced concrete frames in each direction without shear walls and masonry infills. Table 1 lists the properties of all hypothetical frames considered. Bay lengths of 12 and 30 ft may seem unrealistic, but they were considered to provide a wide range of building configurations and SVI values.

<table>
<thead>
<tr>
<th>Model ID</th>
<th>Number of Stories</th>
<th>Bay Length, (ft)</th>
<th>Column Size, (in)</th>
<th>Beam Size (width x height), (in)</th>
<th>( \rho_c )</th>
<th>( \rho_{sh} )</th>
<th>( \rho_b )</th>
</tr>
</thead>
<tbody>
<tr>
<td>S3-1</td>
<td>3</td>
<td>12</td>
<td>18 x 18</td>
<td>18 x 20</td>
<td>0.1%</td>
<td>0.6% (top)</td>
<td></td>
</tr>
<tr>
<td>... S3-60</td>
<td></td>
<td>15</td>
<td>18 x 20</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>S6-1</td>
<td>6</td>
<td>20</td>
<td>24 x 24</td>
<td>18 x 28</td>
<td>1.5%</td>
<td>0.4% (bottom)</td>
<td></td>
</tr>
<tr>
<td>... S6-60</td>
<td></td>
<td>25</td>
<td>20</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

\( \rho_c \): column reinforcement ratio; \( \rho_{sh} \): column transverse reinforcement ratio; \( \rho_b \): beam reinforcement ratio.

Numerical models of the hypothetical frames were constructed for NDA to be conducted using the OpenSees software package (McKenna et al., 2000). The models had linear line elements representing columns and beams, and nonlinear spring elements for plastic hinges at column/beam ends. Columns were assumed to be fixed at ground level. Beam-column joints were assumed rigid, and joint failures were not considered. Building mass was assigned at beam-column joints as lumped mass. P-Delta effect was included. For beam plastic hinge regions, one rotational spring element was used for nonlinear flexural behavior. For column plastic hinge regions, three spring elements were used at column top for nonlinear flexural, shear, and axial behaviors. One spring element was used at column bottom for nonlinear flexural behavior.

The flexural spring element has a single degree of freedom allowing “in-plane” rotation. The load-unload “rules” used were proposed by Clough (1966) and coded by Ibarra et al (2005). The shear and axial springs were modeled “limit-state materials” developed by Elwood (2004). Limit states of the spring elements were defined by equations developed by Elwood and Moehle (2003). Column shear and axial-load capacity are expressed in Eq. 3 and 4 below. Failure of the shear or axial spring elements is assumed to occur when the shear or axial-load demand reaches the limiting capacity estimated from Eq. 3 or 4.

\[
V_n = 500A_g\sqrt{f'c} \left( 0.03 + 4\rho_t - \frac{\Delta}{t} - \frac{1}{40A_gf'c} \right) 
\]  

(3)
\begin{equation}
P_n = \left( \frac{(1+\tan \theta)^2}{25A/L} - \tan \theta \right) \frac{A_{sh}f_{ytd}}{s} \tan \theta \tag{4}
\end{equation}

where \( V_n \) is shear capacity; \( \frac{A}{L} \) is inter-story drift ratio; \( \rho_t \) is transverse reinforcement area ratio \((A_v/bs)\); \( P \) is axial load on the column; \( A_g \) is gross cross-sectional area; \( f'c \) is concrete compressive strength in psi; \( P_n \) is axial load capacity; \( \theta \) is shear crack angle, which is approximated as 65\(^\circ\); \( \frac{\Delta}{L} \) is inter-story drift ratio at axial failure; \( A_{sh} \) is transverse reinforcement area; \( f_{yt} \) is yield stress of transverse reinforcement; \( d \) is effective depth of column cross-section; \( s \) is spacing of transverse reinforcement.

**Ground Motions and Dynamic Analysis**

The models of hypothetical frames were analyzed using the Far-Field ground motion record set recommended in FEMA P-695 (FEMA, 2009). This record set contains 44 ground motions obtained during events with magnitudes from M6.5 to M7.6 with an average magnitude of M7.0.

In conducting dynamic analysis, each frame model was excited using the selected set of 44 ground motions. To have a larger variety in the ground motion intensity, each ground motion was scaled by factors of 1, 1.5, 2, 2.5, and 3. This made a total of 220 ground motion records \((44 \times 5)\) to apply on a single frame model. Furthermore, records with PGV larger than 120 cm/s \((47 \text{ in/s})\) or less than 40 cm/s \((16 \text{ in/s})\) were excluded in analysis, because earthquakes with PGV > 120 cm/s are rare and earthquakes with PGV < 40 cm/s cause sparse damage.

The frame response was analyzed at each time step. An analysis case was stopped when it reached: a) the end of the ground motion record, or b) conditions referred to as “vulnerable to severe damage.” The condition of “vulnerable to severe damage” is defined as when one of the following was met: 1) One or more columns in a frame reached the shear limit defined by Eq. 3; 2) One or more columns in a frame reached the axial limit defined by Eq. 4.

For each hypothetical frame, a term called “frequency of severe damage” was used as a measure of frame toughness. It is the frequency of cases considered as “vulnerable to severe damage” (as described in section above) against the total analysis cases of a frame. The frequencies of severe damage for all hypothetical frames are plotted against their SVI values in Figure 1. The results indicates column index and \( P/f'cA_g \) had good correlation with the frequency of severe damage. For column index, the frequency of severe damage increased dramatically for cases with column index less than 0.25%. \( T/N \) and \( R \) also showed correlation with the frequency of severe damage, but the data had larger scatter. No clear relationships can be drawn between the other two SVIs \( \sum M_c/\sum M_p \) and \( V_n/V_p \) and the frequency of severe damage from the analysis results. Note that the absolute values of the frequency of severe damage here may have little meaning because they were derived from numerical analyses. But the relationships between the damage frequency and SVIs can provide a perspective on the relevance of each SVI.
Field Data

In the case of earthquake engineering, field data are the best source of information to evaluate methods to estimate seismic vulnerability. Evaluations of SVIs and NDA using two sets of field data form the most critical part of this study. Data and drawings from 79 buildings collected from 1992 Erzincan, Turkey, and 2015 Kathmandu, Nepal earthquakes were available and used in this study. The data from 1992 Turkey earthquake were made available by the METU (1993) and AIJ-JSCE-BU (1993) teams. The data from 2015 Nepal earthquake was collected by Shah et al. (2015).

Buildings with nominally identical layouts and properties located within 500 m (0.3 miles) from one another were grouped into a single “case.” The damage level assigned to such “cases” was the most severe damage in the group. After this was done, a total of 18 cases were identified. The eighteen cases selected had 1 to 6 stories. They had reinforced concrete frames without structural walls. Table 2 listed the 18 cases of buildings with the calculated SVIs values and their damage levels after earthquakes. The damage in surveyed buildings was classified into four groups (METU, 1993): None: No obvious damage or crack was found related to earthquake demand;
Light: “Reinforcement exposed but not buckled. Fine flexural cracks in structural and nonstructural elements”; Moderate: “Reinforcement buckled near joint faces and/or inclined cracks in structural walls”; and Severe: “Structural failure of individual elements.” More details of the field data were discussed in the thesis by Song (2016).

Table 2. Building from field survey with observed damage levels and calculated SVIs values

<table>
<thead>
<tr>
<th>Building Name</th>
<th>Building ID</th>
<th>No. of Stories</th>
<th>Damage</th>
<th>CI (%)</th>
<th>$M_c/M_b$</th>
<th>$V_n/V_p$</th>
<th>$P/f_cA_y$</th>
<th>$T/N$</th>
<th>R</th>
</tr>
</thead>
<tbody>
<tr>
<td>Erzincan High School Gymnasium</td>
<td>B1-1</td>
<td>1</td>
<td>None</td>
<td>0.43</td>
<td>0.3</td>
<td>1.5</td>
<td>0.07</td>
<td>0.3</td>
<td>3.0</td>
</tr>
<tr>
<td>Erzincan High School Connecting Bldg</td>
<td>B1-2</td>
<td>1</td>
<td>None</td>
<td>0.42</td>
<td>0.6</td>
<td>2.2</td>
<td>0.09</td>
<td>0.4</td>
<td>5.8</td>
</tr>
<tr>
<td>Erzincan High School Cafeteria</td>
<td>B1-5</td>
<td>1</td>
<td>Light</td>
<td>0.45</td>
<td>0.1</td>
<td>1.9</td>
<td>0.08</td>
<td>0.4</td>
<td>4.1</td>
</tr>
<tr>
<td>Erzincan Committee High School Assembly Hall</td>
<td>B3-1</td>
<td>1</td>
<td>None</td>
<td>0.58</td>
<td>0.5</td>
<td>1.4</td>
<td>0.05</td>
<td>0.2</td>
<td>1.1</td>
</tr>
<tr>
<td>Vocational High School Assembly Hall</td>
<td>B6</td>
<td>1</td>
<td>Light</td>
<td>0.75</td>
<td>0.3</td>
<td>1.8</td>
<td>0.05</td>
<td>0.2</td>
<td>1.6</td>
</tr>
<tr>
<td>Vocational High School Cafeteria</td>
<td>B13</td>
<td>1</td>
<td>Medium</td>
<td>0.36</td>
<td>0.3</td>
<td>1.9</td>
<td>0.08</td>
<td>0.5</td>
<td>5.2</td>
</tr>
<tr>
<td>Police College Gymnasium</td>
<td>B14b-2</td>
<td>1</td>
<td>None</td>
<td>0.79</td>
<td>0.9</td>
<td>2.1</td>
<td>0.05</td>
<td>0.7</td>
<td>6.5</td>
</tr>
<tr>
<td>Erzincan Commercial High School CRB</td>
<td>B3-3</td>
<td>3</td>
<td>Severe</td>
<td>0.18</td>
<td>0.3</td>
<td>2.0</td>
<td>0.16</td>
<td>0.2</td>
<td>8.7</td>
</tr>
<tr>
<td>Vocational High School Electrical Shop</td>
<td>B5a</td>
<td>2</td>
<td>Moderate</td>
<td>0.23</td>
<td>0.3</td>
<td>1.8</td>
<td>0.24</td>
<td>0.3</td>
<td>7.3</td>
</tr>
<tr>
<td>Public Library</td>
<td>B12</td>
<td>3</td>
<td>Light</td>
<td>0.21</td>
<td>0.9</td>
<td>1.3</td>
<td>0.12</td>
<td>0.2</td>
<td>4.9</td>
</tr>
<tr>
<td>Dept. of Agriculture Office Bldg. 1</td>
<td>B16-1</td>
<td>4</td>
<td>Light</td>
<td>0.27</td>
<td>1.7</td>
<td>0.8</td>
<td>0.18</td>
<td>0.2</td>
<td>9.9</td>
</tr>
<tr>
<td>Fatih District Apartment Building</td>
<td>B-Fatih</td>
<td>4</td>
<td>Severe</td>
<td>0.19</td>
<td>0.4</td>
<td>1.7</td>
<td>0.18</td>
<td>0.2</td>
<td>13.1</td>
</tr>
<tr>
<td>Hotel</td>
<td>B-Hotel</td>
<td>6</td>
<td>Severe</td>
<td>0.17</td>
<td>1.5</td>
<td>1.2</td>
<td>0.22</td>
<td>0.1</td>
<td>11.7</td>
</tr>
<tr>
<td>Clinic</td>
<td>B-Clinic</td>
<td>4</td>
<td>Severe</td>
<td>0.21</td>
<td>1.1</td>
<td>1.3</td>
<td>0.21</td>
<td>0.2</td>
<td>11.3</td>
</tr>
<tr>
<td>Oriental Apartment Blocks G, and H</td>
<td>G, H</td>
<td>6</td>
<td>Severe</td>
<td>0.12</td>
<td>1.5</td>
<td>1.6</td>
<td>0.31</td>
<td>0.2</td>
<td>6.9</td>
</tr>
<tr>
<td>Oriental Apartment Block I</td>
<td>I</td>
<td>6</td>
<td>Severe</td>
<td>0.10</td>
<td>1.3</td>
<td>1.8</td>
<td>0.24</td>
<td>0.2</td>
<td>5.9</td>
</tr>
<tr>
<td>Oriental Apartment Blocks F, J, and K</td>
<td>F, J, K</td>
<td>6</td>
<td>Severe</td>
<td>0.10</td>
<td>1.3</td>
<td>1.8</td>
<td>0.24</td>
<td>0.2</td>
<td>5.6</td>
</tr>
<tr>
<td>Oriental Apartment Block L</td>
<td>L</td>
<td>6</td>
<td>Severe</td>
<td>0.09</td>
<td>1.4</td>
<td>1.6</td>
<td>0.29</td>
<td>0.2</td>
<td>7.3</td>
</tr>
</tbody>
</table>

**Evaluation of Selected Screening Methods**

The seven selected screening methods, six SVIs and NDA, were evaluated using the field data described in sections above. SVI values were computed to rank each surveyed building case in terms of their estimated seismic vulnerability. A limiting value of each SVI was chosen to
classify surveyed cases into two groups: a) “more vulnerable to severe damage,” or b) “less vulnerable to severe damage.” The chosen limits and reasons are listed in Table 3. Similarly, NDA was conducted to classify surveyed cases into group a) or b) using criteria defined in the numerical study section above. The evaluation results from SVIs and NDA were compared with building damage levels observed in the field. The observed damage levels from the field were grouped into two classes: “Severe” or “Non-severe” (Moderate, Light, and None) damage.

Each method was used separately to evaluate buildings. If the damage classification of a case obtained with a given SVI or NDA matched the damage observed in the field, the evaluation was marked as “Correct.” Otherwise it was marked as an “Error.” There were two types of errors: a) Type-I Error – A case was classified as “less vulnerable to severe damage,” but the observed damage was “Severe.” This error is more critical because it may lead to loss of human lives and property; b) Type-II Error – A case was classified as “more vulnerable to severe damage,” but the observed damage was “Non-severe.” This type of error may lead to unnecessary retrofit efforts and extra cost. The frequency of errors was used to study which method (among SVIs and NDA) provided the best evaluation results for the sample considered. The frequencies of both types of errors are plotted in a bar chart in Figure 2.

Table 3. Evaluation criteria for SVIs

<table>
<thead>
<tr>
<th>SVI</th>
<th>Threshold</th>
<th>SVI &lt; Threshold</th>
<th>SVI &gt;= Threshold</th>
<th>Rationale to Choose the Threshold</th>
</tr>
</thead>
<tbody>
<tr>
<td>CI</td>
<td>0.25%</td>
<td>More Vulnerable</td>
<td>Less Vulnerable</td>
<td>Field evidence shows buildings with column index less than 0.25% were more vulnerable to severe damage</td>
</tr>
<tr>
<td>$M_c/M_b$</td>
<td>1.2</td>
<td>More Vulnerable</td>
<td>Less Vulnerable</td>
<td>Minimum design requirement of 1.2</td>
</tr>
<tr>
<td>$V_n/V_p$</td>
<td>1</td>
<td>More Vulnerable</td>
<td>Less Vulnerable</td>
<td>Estimated value to avoid vulnerability of shear failure in columns</td>
</tr>
<tr>
<td>$P/f'cA_g$</td>
<td>0.2</td>
<td>Less Vulnerable</td>
<td>More Vulnerable</td>
<td>Estimated value for the balance point of column axial-load and moment interaction</td>
</tr>
<tr>
<td>$T/N$</td>
<td>resulting 3% inter-story drift</td>
<td>Less Vulnerable</td>
<td>More Vulnerable</td>
<td>Estimated value resulting an limiting inter-story drift ratio of 3% (of story height) using the approximate method by Lepage (1997)</td>
</tr>
<tr>
<td>R</td>
<td>3</td>
<td>Less Vulnerable</td>
<td>More Vulnerable</td>
<td>Estimated ductility capacity for older concrete buildings</td>
</tr>
</tbody>
</table>

Figure 2. Errors in evaluation of all field buildings
Additional Evaluation of Column Index

Two databases (Sim et al., 2015, Shah et al., 2015) including nearly 750 buildings surveyed after seven earthquakes was used to reexamine column index (CI) and thresholds that can be used to classify buildings. Evaluation of other SVIs and NDA were not available with these field data due to lack of building details and material properties. Buildings in this database were classified into two groups according to their column index values with a limiting value of 0.25%. The ratio of number of buildings with severe damage to total number of buildings in each group was plotted in Figure 3. The building group with column index less than 0.25% had a frequency of severe damage larger than 50% (about twice that of buildings with CI larger than 0.25%).

![Figure 3. Frequency of severe damage for buildings within different column index range](image)

Conclusions

The objective of this study was to evaluate screening methods to rank existing reinforced concrete (RC) structures with respect to their seismic vulnerability. The screening methods are to be simple and efficient so that evaluating a large inventory of buildings does not have a prohibitive cost. Six seismic vulnerability indicators (SVI) and nonlinear dynamic analysis (NDA) were studied as potential methods to evaluate low- to mid-rise (one to six stories) concrete frame buildings without shear walls.

Two steps were used to evaluate the seven screening methods listed above (six SVIs and NDA). Numerical analysis of hypothetical frames was first conducted to study the six SVIs. The best correlation between the frame “frequency of severe damage” and their SVI values was obtained with column index (CI). The second step was to evaluate the six SVIs and NDA results against observations from 18 cases surveyed in Turkey and Nepal in 1992 and 2015. Among all six SVIs and NDA, column index (CI) provided the best correlation between estimates and observations. It was noted that ratio of building initial fundamental period to number of stories (T/N), ratio of building base-shear demand to base-shear strength (R factor), and nonlinear dynamic analysis (NDA) produced conservative results in estimating building damage. They succeeded in identifying buildings with severe damage in earthquakes, but also suggested similar high risks for buildings without severe damage. They also require much more building information and computational effort. It was also observed that buildings with lower column index have higher frequency of severe damage. This suggests retrofit work should start in the buildings with lowest column index.
References

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