Proposed Performance-Based Seismic Design Method for Assessing Vulnerability and Fragility of RC Buildings

Aung Mon*, Dr. Nang Su Le' Mya Thwin**

*Department of Civil Engineering, Mandalay Technological University, The Republic of the Union of Myanmar
**Professor in Department of Civil Engineering, Mandalay Technological University, The Republic of the Union of Myanmar

Abstract- The objective of this study is to develop a simplified seismic performance based design method which reduce the repetition cycles of nonlinear process. The peak ground accelerations are estimated using earthquake data along the Sagaing Fault for Mandalay city in Myanmar. The evaluated yield accelerations of twenty case studies of the seismic force resisting systems that complying with the performance objectives are documented as a data bank based on the current analysis and design procedure. Then, they are converted to a proposed yield acceleration equation using second-order polynomial regression. This method is also verified by the total of four buildings based on different number of stories. By introducing, this proposed yield acceleration at the initial stage of analysis and design as a simplified method, some repetitions of the nonlinear process can be reduced. Fragility curves are developed for performance based buildings on different peak ground accelerations and their damage probability is compared. The vulnerability of the buildings is estimated in terms of vulnerability index to assess the performance of the building.

Index Terms- A Simplified Performance-Based Seismic Design, Yield Acceleration, Second Order Polynomial Regression, Fragility curves, Vulnerability Index

I. INTRODUCTION

Performance Based Seismic Design (PBSD) has been considered as an essential part of earthquake engineering. New developments and methods for the application of PBSD methodology are needed because most existing PBSD approaches tend to provide guidance and tools for the evaluation of seismic performance of a building that has already been designed [8]. In other words, more research work is needed for development of initial design because there is no guideline provided in current PBSD practice [6]. Several approaches mainly provide a suitable design base shear that accounts for higher mode effects, system over strength, yield displacement, effective stiffness, viscous damping, effective period, or displacement ductility. Additionally, iteration during the design process is still required. Thus, practical methods based on these approaches are still under development and improvement [5]. Losses inflicted on modern buildings from recent earthquakes have shown the pressing need for investigation of the seismic safety of code-compliant buildings at various performance limit states. This need has stimulated significant research to develop methodologies for deriving fragility relationships, which are a key component in seismic loss assessment. The seismic vulnerability of a structure can be described as its susceptibility to damage by ground shaking of a given intensity. The methodologies are used to develop various tools such as vulnerability functions and fragility curves, from structural damages observed during earthquakes.

II. METHODOLOGY AND MODEL DEVELOPMENT

A. Seismic Hazard Analysis for Mandalay City Area

In considering earthquake hazard environment of Mandalay City, the probability of exceedance in 50 years is 50% for the operational earthquake level (MOE), 10% for the design basic level earthquake (DBE) and 2% for the maximum considered earthquake level (MCE) [4].

\[
T = \frac{1}{1 - (1 - p)^{1/n}}
\]

where, \( p \) = Probability of exceedance in 50 years
\( T \) = Return period
\( n \) = 50 years
Magnitude probability for Gutenberg-Richter law of Equation as follows [1].

\[
F_M(m_j) = \frac{1 - 10^{-b(M_m-M_{max})}}{1 - 10^{-b(M_{max}-M_{min})}}
\]

Where \( M_{max} \) the maximum and \( M_{min} \) the minimum earthquakes.

\[
P(M_m) = F_M(m_{j+1}) - F_M(m_j)
\]

Where \( m_j \) are the discrete set of magnitudes, ordered so that \( m_j < m_{j+1} \).

Estimation of peak ground acceleration is based on earthquake data from the Sagaing Fault [2].

\[
\ln(\text{PGA}) = -0.152 + 0.859\text{Mw} - 1.803\ln(\text{R}+25)
\]

where, PGA = Peak ground acceleration
\( \text{Mw} \) = Moment magnitude
\( \text{R} \) = Source Distance

<table>
<thead>
<tr>
<th>Earthquake Type</th>
<th>SE</th>
<th>DBE</th>
<th>MCE</th>
</tr>
</thead>
<tbody>
<tr>
<td>Return Period</td>
<td>72</td>
<td>475</td>
<td>2475</td>
</tr>
<tr>
<td>Moment magnitude</td>
<td>6.5</td>
<td>7.325</td>
<td>7.875</td>
</tr>
<tr>
<td>Acceleration at the base rock (g)</td>
<td>0.166</td>
<td>0.331</td>
<td>0.508</td>
</tr>
<tr>
<td>Amplification (Cg)</td>
<td>1.2</td>
<td>1.2</td>
<td>1.2</td>
</tr>
<tr>
<td>Acceleration at the</td>
<td>0.2</td>
<td>0.4</td>
<td>0.6</td>
</tr>
</tbody>
</table>

www.ijisrp.org
**B. Performance Criteria**

<table>
<thead>
<tr>
<th>Seismic Hazard Level</th>
<th>Performance Level</th>
<th>Probability/year</th>
<th>Critical Limit</th>
</tr>
</thead>
<tbody>
<tr>
<td>SE</td>
<td>IO</td>
<td>50%/50yr</td>
<td>1%</td>
</tr>
<tr>
<td>DBE</td>
<td>LS</td>
<td>10%/50yr</td>
<td>2%</td>
</tr>
<tr>
<td>MCE</td>
<td>CP</td>
<td>2%/50yr</td>
<td>4%</td>
</tr>
</tbody>
</table>

**E. Performance Based Seismic Design Of RC Buildings**

The reinforced concrete buildings are designed as on performance based seismic design procedure. Analytical results such as hinge formation maximum considered earthquake are shown from Fig. 2.

**F. Evaluated Yield Acceleration, \( S_{ay} \)**

The Evaluated \( S_{ay} \) for Four Group based on performance based seismic design of RC building. The minimum requirements of \( S_{ay} \) for Immediate Occupancy are shown in Table IV.

<table>
<thead>
<tr>
<th>No of Storey</th>
<th>3</th>
<th>5</th>
<th>7</th>
<th>9</th>
<th>12</th>
</tr>
</thead>
<tbody>
<tr>
<td>( T )</td>
<td>0.413</td>
<td>0.606</td>
<td>0.78</td>
<td>0.942</td>
<td>1.168</td>
</tr>
<tr>
<td>( S_{ay} ) for Group 1</td>
<td>0.672</td>
<td>0.475</td>
<td>0.431</td>
<td>0.348</td>
<td>0.283</td>
</tr>
<tr>
<td>( S_{ay} ) for Group 2</td>
<td>0.67</td>
<td>0.467</td>
<td>0.384</td>
<td>0.339</td>
<td>0.272</td>
</tr>
<tr>
<td>( S_{ay} ) for Group 3</td>
<td>0.658</td>
<td>0.461</td>
<td>0.375</td>
<td>0.332</td>
<td>0.251</td>
</tr>
<tr>
<td>( S_{ay} ) for Group 4</td>
<td>0.656</td>
<td>0.457</td>
<td>0.373</td>
<td>0.323</td>
<td>0.249</td>
</tr>
<tr>
<td>Average ( S_{ay} )</td>
<td>0.664</td>
<td>0.465</td>
<td>0.391</td>
<td>0.334</td>
<td>0.263</td>
</tr>
</tbody>
</table>

**G. Proposed Yield Acceleration, \( S_{ayT} \)**

The average \( S_{ay} \) and time period \( T \) are considered as a proposed yield acceleration for the proposed design method.
In this study, the $S_{ayT}$ are indicated in terms of the fundamental periods of buildings. Equations are derived using second-order polynomial regression based on the average evaluated yield acceleration. The second-order polynomial regression is shown in Equation 5.

$$S_{ayT} = a_0 + a_1T + a_2T^2$$ (5)

where, $S_{ayT}$= proposed yield acceleration

$T$ = fundamental period

$a_0$, $a_1$, and $a_2$ = coefficients

The coefficients can be determined by using the following set of equations.

$$\sum_{i=1}^{n} a_0 + \sum_{i=1}^{n} T_i a_1 + \sum_{i=1}^{n} T_i^2 a_2 = \sum_{i=1}^{n} S_{ayT_i}$$

where, $n$= number of set of data

$$R = \frac{\sum{(S_{ayT} - \bar{S}_{ayT})^2} + \sum{(S_{yy} - \bar{S}_{yy})^2}}{\sqrt{\sum{(S_{ayT} - \bar{S}_{ayT})^2} \sum{(S_{yy} - \bar{S}_{yy})^2}}}$$ (7)

where $\bar{S}_{ayT}$ and $\bar{S}_{yy}$ are the correlation coefficient

$S_{xx} = T^2 - n \bar{T}^2$

$S_{sy} = T \bar{S}_{ayT} - n \bar{T} \bar{S}_{ayT}$

$S_{yy} = S_{ayT}^2 - n \bar{S}_{ayT}^2$

The correlation coefficient $R$=0.956 indicates that the proposed yield acceleration is excellent fit.

J. The Proposed Simplified Seismic Design Method

A proposed simplified seismic design method using $S_{ayT}$ is shown in Figure 4. The $S_{ayT}$ from the proposed Equation 6 is used in this method.

K. Algorithm of Proposed Seismic Method

1. Checking of design condition and obtaining the design requirement are required. Then, the structural model is developed.

2. Estimation of the base shear using R factor according to the UBC-97 and analyzing of the model are included. Then the member design is done based on moment, shear, and axial force from the analysis and the $S_{ayT}$is calculated. The required data for this equation can be obtained from the analysis.

3. Checking of the $S_{yy}$ with the $S_{ayT}$ from the proposed Equation 6 is required. If they are not nearly equal, go to the step-2 to determine new structural member set by adjusting R factor for the based shear. Then calculation of $S_{ayT}$ and comparison with the $S_{ayT}$ is done until they are almost equal.

4. The pushover analysis was done to check the three limit states such as IO, LS, and CP.

5. To check the IO level, the minimum required $S_{ad}$ from the performance point at under SE is obtained and with the evaluated $S_{ay}$ calculated from the step-3 is compared. To check the LS and CP levels, the story drifts are calculated using the structural deformations under DBE and MCE. Then, they are compared with the maximum drift limitations. If one of the three limit states check is not acceptable, it is required to repeat the step-4 by increasing stiffness to upgrade yield acceleration until they are acceptable.
L. Range of Applicability
1. Special concrete moment resisting frame with fundamental period $0.413 \leq T \leq 1.168$ seconds.
2. Irregular structure located in seismic zone 4.
3. The $L/B$ ratio must be between 1.25 to 2.

III. Verification
The four verification examples are considered according to various number of stories and plans.

A. Verification for Different Numbers of Storey
The geometries of the selected buildings are shown from Figure 5, 7, 9 and 11. Analytical results such as hinge formation, performance point at the maximum considered earthquake are shown from Figure 6, 8, 10 and 12.

Verification Example 1, (135’x85’x55’), $L/B=1.59$

Verification Example 2, (114’x72’x77’), $L/B=1.58$

Verification Example 3, (115’x70’x99’), $L/B=1.64$

Verification Example 4, (114’x87’x121’), $L/B=1.31$
The performance level of structure in Maximum Considered Earthquake is shown in Fig. 12. This spectrum curve points out the performance point at Life Safety level, the spectral acceleration 0.344g and spectral displacement 12.564in. It is occurred between step 12 and 13.

**B. Summary of Analytical Results for Different Number of Storey**

The evaluated $S_{ay}$ based on the analytical results of each case study are shown in Tables V. The minimum requirements of for IO and maximum interstory drift limitations for LS and CP are also checked.

<table>
<thead>
<tr>
<th>No of Storey</th>
<th>4</th>
<th>6</th>
<th>8</th>
<th>10</th>
</tr>
</thead>
<tbody>
<tr>
<td>Period, T</td>
<td>0.606</td>
<td>0.78</td>
<td>0.942</td>
<td>1.09</td>
</tr>
<tr>
<td>Max Interstory drift% at SE%</td>
<td>0.71</td>
<td>0.83</td>
<td>0.74</td>
<td>0.86</td>
</tr>
<tr>
<td>Allowable Interstory drift,%</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>Check for IO</td>
<td>Pass</td>
<td>Pass</td>
<td>Pass</td>
<td>Pass</td>
</tr>
<tr>
<td>Max Interstory drift at DE,%</td>
<td>1.25</td>
<td>1.27</td>
<td>1.42</td>
<td>1.39</td>
</tr>
<tr>
<td>Allowable Interstory drift,%</td>
<td>2</td>
<td>2</td>
<td>2</td>
<td>2</td>
</tr>
<tr>
<td>Check for LS</td>
<td>Pass</td>
<td>Pass</td>
<td>Pass</td>
<td>Pass</td>
</tr>
<tr>
<td>Max Interstory drift at MCE,%</td>
<td>1.68</td>
<td>1.79</td>
<td>2.28</td>
<td>1.95</td>
</tr>
<tr>
<td>Allowable Interstory drift,%</td>
<td>4</td>
<td>4</td>
<td>4</td>
<td>4</td>
</tr>
<tr>
<td>Check for CP</td>
<td>Pass</td>
<td>Pass</td>
<td>Pass</td>
<td>Pass</td>
</tr>
</tbody>
</table>

**Table VI**

<table>
<thead>
<tr>
<th>No. of Story</th>
<th>4</th>
<th>6</th>
<th>8</th>
<th>10</th>
</tr>
</thead>
<tbody>
<tr>
<td>Period, T</td>
<td>0.606</td>
<td>0.78</td>
<td>0.942</td>
<td>1.094</td>
</tr>
<tr>
<td>Proposed $S_{ay}$</td>
<td>0.492</td>
<td>0.385</td>
<td>0.317</td>
<td>0.28</td>
</tr>
<tr>
<td>Evaluated $S_{ay}$</td>
<td>0.512</td>
<td>0.411</td>
<td>0.332</td>
<td>0.295</td>
</tr>
<tr>
<td>Proposed $S_{ay}$/Evaluated $S_{ay}$</td>
<td>0.96</td>
<td>0.937</td>
<td>0.955</td>
<td>0.949</td>
</tr>
</tbody>
</table>

The four verification examples for different numbers of storey are selected. They are irregular buildings with L/B ratios between 1.3 and 1.64. From the results, it is concluded that the Proposed $S_{ay}$/Evaluated $S_{ay}$ ratio must be between 0.9 to 1.

**C. Comparison of Repetition Cycles for Different Numbers of Storey**

As a result, it is found that the number of repetitions required for the analysis and design process to satisfy the three limit states are five times for Four storeyed building, six times for the Six and Eight storeyed buildings, and eight times for Ten storeyed building when the general design procedure is used.

However, when the simplified approach is used, the repetitions are decreased to one time for Four, Six and Eight storeyed buildings, and two times for the Ten storeyed building. The comparison of required numbers of repetition for the both methods is shown in Fig. 13.

![Fig. 13 Comparison of Repetition Cycles](image)

In summary, the number of repetitions required for the simplified method is less than that of using the general procedure. In addition, L/B ratio is small effect on the proposed method as long as the building is irregular.

**D. Seismic Vulnerability Assessment of Verification Examples**

The vulnerability index is a measure of the damage in a building obtained from the pushover analysis. It is defined as a scaled linear combination (weighted average) of performance measures of the hinges in the components, and is calculated from the performance levels of the components at the performance point or at the point of termination of the pushover analysis. The vulnerability index of a building is assessed with the expression as follows [9].

$$VI_{bldg} = \frac{1.5 \sum_{i} N_{i}^{d} x_{i} + \sum_{j} N_{j}^{d} x_{j}}{\sum_{i} N_{i} + \sum_{j} N_{j}}$$  \hspace{1cm} (8)

Where $N_{i}^{d}$ and $N_{j}^{d}$ are the numbers of hinges in columns and beams, respectively, for the $i^{th}$ and $j^{th}$ performance range. A weightage factor ($x_{i}$) is assigned for columns and ($x_{j}$) is assigned for beams to each performance range, the weightage factor is shown in Table VII.

$V_{bldg}$ is a measure of the overall vulnerability of the building. A high value of $V_{bldg}$ reflects poor performance of the building. However, this index may not reflect a soft storey mechanism.
A storey vulnerability index (VI\textsubscript{storey}) defined to quantify the possibility of a soft/weak storey with the formation of flexural hinges. For each storey, VI\textsubscript{storey} is defined as

\[ VI_{\text{storey}} = \sum_{i=1}^{c} \frac{W_i}{\sum_{k=1}^{c} W_k} \cdot X_i \tag{9} \]

A Storey Vulnerability Index based on Maximum Considered Earthquake (Verification Example I)

<table>
<thead>
<tr>
<th>Serial Number</th>
<th>Performance Range</th>
<th>Weight Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>O</td>
<td>0.125</td>
</tr>
<tr>
<td>2</td>
<td>IO</td>
<td>0.375</td>
</tr>
<tr>
<td>3</td>
<td>LS</td>
<td>0.625</td>
</tr>
<tr>
<td>4</td>
<td>CP</td>
<td>0.875</td>
</tr>
<tr>
<td>5</td>
<td>C</td>
<td>1.000</td>
</tr>
</tbody>
</table>

A storey vulnerability index of zero indicate that most of the hinges are formed in beams rather than in columns.

A Storey Vulnerability Index based on Maximum Considered Earthquake (Verification Example II)

From the results, it is found that, the storey 3 are more vulnerable than other storey. The value of storey vulnerability index are different due to their configurations.

A Storey Vulnerability Index based on Maximum Considered Earthquake (Verification Example III)

From this table it is apparent that, storey 5 are more vulnerable than other storey.

The storey vulnerability index of zero indicate that most of the hinges are formed in beams rather than in columns.

A Storey Vulnerability Index based on Maximum Considered Earthquake (Verification Example IV)

From this table, the storey vulnerability index of zero indicate that most of the hinges are formed in beams rather than in columns. These are strong column and weak beam design.

Table VIII: Vulnerability Index for Structural System

<table>
<thead>
<tr>
<th>Building</th>
<th>Vulnerability Index</th>
</tr>
</thead>
<tbody>
<tr>
<td>4-storey</td>
<td>0.36</td>
</tr>
<tr>
<td>6-storey</td>
<td>0.356</td>
</tr>
<tr>
<td>8-storey</td>
<td>0.313</td>
</tr>
<tr>
<td>10-storey</td>
<td>0.334</td>
</tr>
</tbody>
</table>

\( V_{\text{bldg}} \) is a measure of the overall vulnerability of the building. A high value of \( V_{\text{bldg}} \) reflects poor performance of the building. From this table it is apparent that, the buildings (Verification Examples III and IV) are more resistant than Verification Examples I and II under DBE and MCE.

E. Seismic Fragility Analysis of Verification Examples

Fragility curves describe the probability of damage to building. Building fragility curves are lognormal functions that describe the probability of reaching, or exceeding, structural and non-structural damage states, given median estimates of spectral response, for example spectral displacement. These curves take into account the variability and uncertainty associated with capacity curve properties, damage states and ground shaking.

Evaluation of Structural Fragilities [10]

\[ P[d/S_a] = \Phi \left( \frac{1}{\beta_{ds}} \ln \left( \frac{S_a}{S_{a,ds}} \right) \right) \tag{10} \]

Where,

\( P[d/S_a] \) = damage probability value, \( ds \)
\( S_{a,ds} \) = Median value of spectral acceleration at which the building reaches the threshold of damage state, \( ds \)
\( \beta_{ds} \) = Standard deviation of the natural logarithm of spectral acceleration for damage state, \( ds \)
\( \Phi \) = Standard normal cumulative distribution function.
\( S_a \) = Given peak spectral acceleration

Fragility Curve for Verification Example I
If the spectral acceleration 0.746g (PGA =0.6g) corresponding to a return period of 2475 years, the probabilities of slight, moderate and severe damage to the Verification Example I is 97%, 77% and 45% respectively.

**Fragility Curve for Verification Example II**

If the spectral acceleration 0.588g (PGA =0.6g) corresponding to a return period of 2475 years, the probabilities of slight, moderate and severe damage to the Verification Example II is 98%, 89% and 36% respectively.

**Fragility Curve for Verification Example III**

If the spectral acceleration 0.366g (PGA =0.6g) corresponding to a return period of 2475 years, the probabilities of slight, moderate and severe damage to the Verification Example III is 98%, 91% and 22% respectively.

**Fragility Curve for Verification Example IV**

If the spectral acceleration 0.352g (PGA =0.6g) corresponding to a return period of 2475 years, the probabilities of slight, moderate and severe damage to the Verification Example IV is 99%, 85% and 30% respectively. This table shows the probability of damage for SE(0.2g), DBE(0.4g) and MCE(0.6g).

IV. CONCLUSIONS

In this study, the proposed yield acceleration $S_{ayT}$ or $V_A$ that satisfy the basic safety objectives including acceptable story drift limits for LS, CP and minimum required $S_{ay}$ for IO. Therefore, the yield acceleration $S_{ay}$ is evaluated for concrete moment frames based on the total of twenty case studies. The yield acceleration, $S_{ay}$ decreases substantially with increased building height. The proposed simplified seismic performance based design method is developed by using evaluated yield acceleration, $S_{ayT}$. The repetition cycles of nonlinear analysis and design process can be reduced by using the proposed method.

A storey vulnerability index of zero indicates that most of the hinges are formed in beams rather than in columns. These are strong column and weak beam design. The yield mechanisms adopted in earthquake resistant design are strong column and weak beam. These buildings (Verification Examples III and IV)are more resistant than Verification Examples I and II under DBE and MCE.

The Fragility Curve are plotted considering Spectral Acceleration as a ground motion parameter. Fragility curves were developed for performance based seismic design of RC buildings and compared their damage states. It is observed that verification example III is seismically more resistant than other verification examples for severe damage states.

ACKNOWLEDGMENT

I would like to express heartfelt gratitude to Dr. Nilar Aye Professor and Head of Civil Engineering Department and all of my teachers at Mandalay Technological University for their encouragement and suggestions.

REFERENCES


AUTHORS

First Author – Aung Mon, Ph.D. candidate, Mandalay Technological University, Myanmar, aungmon.civil@gmail.com.

Second Author – Dr. Nang Su Le’ MyaThwin, Professor, Mandalay Technological University, Myanmar.