Evaluation of Seismic Behaviour of Old Reinforced Concrete Structures Based on Ductility Limit

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Abstract: The seismic design is basically used as a tool for the make sure the structural stabiality against liquefaction. However, the design philosophy behind the earthquake is “dissipation of energy without large deformation” in the structure. As long as Sri Lanka is considered moderate vulnerability country based on the records and investigate done within last five decades. But according to the current research status, a new plate boundary is found to be creating in south west of Sri Lanka due to splitting up of Indo Australia plate as intra-plate. Hence there is a potential of increasing seismic activities in Sri Lanka and need to consider the design requirement specially in the high rise buildings. Therefore, it’s essential to be assessed the current capacity of the building to resist seismic movements around and how frequency it occurs. Most of the building have been designed in Sri Lanka without considering the ductility demand required for the cyclic loading condition. Further, the energy dissipation capacity of structure is most critical and important for asses the building risk factor. By the way Eurocode 8(EC8) and) has emphasized the performance based design on the ductility using many more aggregate parameters such as confinement, R/F arrangement and geometry of the section etc. The seismic behaviour of the building can be assessed in several methods as push over analysis, time history analysis, ductility assessment etc. In this paper a ductility assessment has been carried out for a three storey reinforced concrete building in Sri Lanka to evaluate the capacity of the building in terms of global and local ductility demand according to Eurocode 8.

Keywords: Concrete building, Ductility, confinement, Seismic assessment

1. INTRODUCTION

The outer shell of the Earth is broken up into 12 major plates and a few smaller ones as per the latest information given. Sri Lanka is located within the tectonic plate known as “Indo Australia plate” in the Indian Ocean, about 24 km to the southeast of India. According to the historical records a very few number of earthquakes have been recorded within the country. However, historical records indicate that in 1615, an earthquake occurred near Colombo, Sri Lanka and over 200 deaths with many casualties had been recorded (Abaykoon 1998). It is observed that the damage caused by such earthquakes tend to be very high due to lack of preparedness against them special on building and and relevant infrastructure.
Hence, the risk of an earthquake cannot be simply overlooked due to its fewer occurrences and analyses the experience. By the way, it is important to do the investigation about the damage classes and its behaviour on particular areas.

Since Sri Lanka is located amid the large Indo Australia plate many believe that Sri Lanka is safe from earthquakes (See Figure 1 and Figure 2). However recent researchers have identified that the Indo Australian plate is splitting up and creating an intra-plate just about 400-500km from the southwest coast of Sri Lanka. Existence of this intra-plate approximately 400-500 km south of Sri Lanka will have potential impact on future geohazards in Sri Lanka especially earthquakes and minor tremors. The potential impact of this plate tectonic phenomenon on Sri Lanka will have to be further studied carefully. Given the lack of preparedness and the absence of pre-disaster mitigation measures, the occurrence of even a moderate earthquake close to urban centres such as Colombo may have disastrous consequences. Such a possibility exists and it would be wise for the building industry to take this into consideration. Therefore, the study of seismic effects on Sri Lanka has become an important matter, especially after the experience of 2004 tsunami, which was a low frequency high impact hazard.

The old structures in Sri Lanka can be categorised basically in two type which are reinforced frame with load bearing wall and masonry structures. However, it is true that, considerable steel structures are being constructed and industry rapidly moving to fast construction process with the industrial growth after the civil war, 2009. So, it is really important to carry out the dynamic analysis to understand the structural behaviour in order to make sure the capacity of the structure based on risk factor. However, the dynamic analysis still was taken into the design philosophy for the high rise building such as wind etc. But, due to increase of slight effects from earthquakes (Munasinghe & Curray 2013), structural designers to be concerned an impotency of seismic concern on building design in Sri Lanka. As a result, most of the high rise buildings now are being designed by taking account of seismic loading and it has also become a mandatory legal restrain to get the council approval for the high rise buildings.
This paper includes a ductility assessment carried out according to Eurocode 8 for the considered case of old building which has been designed without consideration of earthquake loads. The load analysis of the selected building is carried out by using the finite element analysis and moment curvature diagrams have been obtained to determine the available curvature ductility of the sections. Moment curvature analysis is carried out by using an open source software framework called “OpenSees”. The OpenSees allows to do the complex models for the non linear analysis which has flexibility to do the depth parametric study (Pradeep et al. 2013). But, most of the materials inside the OpenSees developed based on past researches constitution model.

2. METHODOLOGY

2.1 Determination of Curvature Ductility

Curvature ductility is key parameter to assess the damage level of the structures referring to the serviceability and ultimate limit state. Ductility ratio mainly depends on axial compression ratio, slenderness and the transvers R/F steel ratio (Watson et al. 1994). Axial compression ratio (Axial load ratio defined in Eq. 2) is a very important indicator to evaluate the ductility and the fragility of the RC structures during earthquakes in design process of reinforced concrete columns for seismicity. the special consideration should be given to plastic hinge regions occurrence to prevent collapsing of columns due to earthquakes. Ductility capacity of the members are very important in moment redistribution takes place in the joint where diagonal tension and compression take place.

Provision of adequate transverse reinforcement in the form of steel stirrups to confine the compressed concrete is an essential consideration of ductility in the areas where plastic hinge forms (Braga et al. 2006). Transverse reinforcements restrain shear failure of the sections and buckling of the longitudinal bars. Moreover, empirical studies have been shown that providing satisfactory transverse reinforcement to the confined concrete laterally increase the ductility and strength in concrete (Mander et al. 1988).

The purpose of this study is to evaluate the ductility capacity of the building under service loads, in a simple closed form manner trough the numerical method. The moment curvature analysis has been carried out with the use of an open source software framework (OpenSees 2009) on detailed reinforced concrete column sections and the resulted curvature ductility is compared with the curvature ductility demand using Eurocode 8. A moment curvature (See Figure 3) illustrates the flexural behaviour of a section beyond the elastic limit. Moment curvature has been obtained by increasing the axial strain up to the failure point for a given axial load by assuming no shear deformation is accounted. The moment curvature curve is a nonlinear curve, hence the yield curvature
and the ultimate curvature is identified by the equivalent bilinear curve obtained by the conventional methods (Mander et al. 1988). The curvature ductility of the columns have been determined according to the method proposed by Watson et al. 1994. The yield curvature $\phi_y$ is determined by extrapolating origin and the yield point with a straight line as shown in Figure 3. The yield point can be either when the steel in tension start to yield or extreme fiber in compression concrete reaches 0.002 strain, whichever take place first. The curvature ductility of a reinforced concrete section, $\mu_\phi$ is given by $\mu_\phi = \frac{\phi_u}{\phi_y}$, where $\phi_u$ is the curvature at ultimate when the concrete compression strain reaches a limiting value and $\phi_y$ is the curvature when the tension reinforcement first reaches the yield strength.

2.2 Global Ductility Demand: Eurocode 8

Eurocode 8 has classified two ductility classes which are high ductility class (DCH) and the medium ductility class (DCM) to evaluate the seismic behaviour of concrete buildings. According to Eurocode 8 (clause 5.2.3.4) for critical regions in primary structural elements, the curvature ductility factor should be at least equal to the following values.

$$\mu_\phi = \begin{cases} 2q_0 - 1 & \text{if } T_1 \geq T_c \\ 1 + 2(q_0 - 1) \frac{T_c}{T_1} & \text{if } T_1 < T_c \end{cases}$$

For this study two behaviour factors are taken by considering structural type of the building for DCH and DCM. According to Eurocode 8 (clause 5.2.2.2) for frame systems the $q_0$ for DCM and DCH are given as $3.0\frac{\alpha_u}{\alpha_1}$ and $4.0\frac{\alpha_u}{\alpha_1}$, where $\alpha_u/\alpha_1$ multiplication factor for multistorey frames is 1.3. Hence, the $q_0$ factor for DCM and DCH are obtained as 3.9 and 5.85 respectively.

As defined in the EC8 the curvature ductility values are multiplied by 1. According to the resulted curves in Figure 5, structural ductility demand for DCM and DCH are obtained as $\mu_\phi = 10$ and $\mu_\phi = 16$ respectively.

As per the EC8 the minimum local curvature ductility requirement for a primary column section is considered to be satisfied if,

$$\alpha_\omega_{wd} \geq 30\mu_\phi v_d \cdot \frac{b_c}{b_o} \cdot \frac{\epsilon_{sy,d}}{\mu_\phi} - 0.035$$  \hspace{1cm} (1)

Where,

- $\alpha$ – confinement effectiveness factor ($\alpha = \alpha_n \cdot \alpha_s$)
- $b_o$ – width of confined core
- $\epsilon_{sy,d}$ – design value of tension steel strain at yield
- $v_d$ – normalized design axial force
- $\omega_{wd}$ – mechanical volumetric ratio of confining hoops within the critical regions
- $b_c$ – gross cross sectional width
- $\mu_\phi$ – curvature ductility factor

In here $\alpha$ for rectangular sections are obtained by,

$$\alpha_n = 1 - \sum_n \frac{b_i^2}{6b_o h_o}$$

$$\alpha_s = (1 - s/2b_o)(1 - s/2h_o)$$

Where $n$ is total nos. of longitudinal bars in the section and $b_i$ is distance between consecutive engaged bars.

The normalized design axial force (axial force ratio) is given by

$$v_d = \frac{N_{Ed}}{A_{cf cd}}$$  \hspace{1cm} (2)
Where \( N_{Ed} \) is the axial force from the analysis for the seismic situation design, \( A_c \) is the area of section of concrete member and \( f_{cd} \) is the design value of concrete compressive strength. The mechanical volumetric ratio of transverse reinforcement is given by the following equation.

\[
\omega_{wd} = \frac{2A_s f_{yd}}{b_0 f_{cd}}
\]

Where \( A_s \) is the area of transverse reinforcement, \( b_0 \) is the width of confined concrete core, \( f_{cd} \) is the concreted compressive strength and \( f_{yd} \) is the design yield strength of steel.

### 3. DETAILS OF THE STRUCTURE: CASE STUDY

The ductility assessment has been performed to a selected school building. SAP2000 has been used to model the building, to get the axial load of the columns and moment curvature. Curvature ductility of the columns have been obtained by the use of Open Sees (See Figure 4) respect to axial load values.

![Selected school building](image)

The selected school building is a three storied building which 27m long and 7.9m wide. It has total of 9 bays with 3m span along its length. The frame of the building has been designed according to the BS 8110 and done the construction using reinforced concrete. Loading of the building and design have been done according to BS 6399 and BS8110. The considered parameter for the Open Sees analysis have been shown in Table 1.

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Range</th>
</tr>
</thead>
<tbody>
<tr>
<td>( P/f'_c A_g )</td>
<td>0.2-0.7</td>
</tr>
<tr>
<td>( f'_c ) (MPa)</td>
<td>30</td>
</tr>
<tr>
<td>( \rho_{t,m} )</td>
<td>0.1-0.4</td>
</tr>
<tr>
<td>( c/h )</td>
<td>0.02</td>
</tr>
<tr>
<td>( f_{yd} )</td>
<td>460MPa</td>
</tr>
<tr>
<td>( \omega_{wd} )</td>
<td>0-0.25</td>
</tr>
</tbody>
</table>

Note: The design parameters: Density of the concrete – 24 kN/m³, Minimum imposed floor load of a class room – 3 kN/m², Minimum imposed floor load of stairs and landings – 4 kN/m², uniformly distributed load due to finishes – 1 kN/m². In order to check whether the columns are satisfying the local ductility demand the calculation to be done as shown in Figure 5. Hence the curvature ductility value for axial load factor 0.2 to 0.5 were calculated from Eq. 1 as follows.

\[
\mu_{\phi} = \frac{(\alpha \omega_{wd} + 0.035)b_0}{30v_a \cdot f_{syd} \cdot b_c}
\]

By substituting values, the theoretical values of curvature ductility demand for columns have been determined and shown in Table 2.
Table 2 Local ductility demand in column according to the EC 8

<table>
<thead>
<tr>
<th>Axial load factor ($v_d$)</th>
<th>Local curvature ductility demand ($\mu_\phi$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.2</td>
<td>6.26</td>
</tr>
<tr>
<td>0.3</td>
<td>4.17</td>
</tr>
<tr>
<td>0.4</td>
<td>3.13</td>
</tr>
<tr>
<td>0.5</td>
<td>2.50</td>
</tr>
</tbody>
</table>

4. RESULTS AND DISCUSSION

As per the EC8, curvature ductility demand for Medium Ductility Class and High Ductility Class are considered 10 and 16 respectively. The estimated ductility values by numerical analysis of both C1 and C2 columns are lower than the ductility of 10(C1 and C2 are the internal and corner column of the building respectively). Hence the required ductility demands for DCM or DCH haven’t been achieved in the members to meet the limits. Estimated ductility and theoretical ductility demand values shown in Figure 7.
According to the graph both curvature ductility curves of C1 and C2 columns are above the demand curve. Hence the columns satisfy the local ductility demand specified in EC8. The theoretical ductility demand in the both cases (T10, T12) have not been satisfied.

Mechanical volumetric ratio of confining hoops $\omega_{wd}$ has a direct impact on the curvature ductility of a member. The $\omega_{wd}$ of columns with both 12mm and 16mm main diameter bars is equals to:

$$\omega_{wd} = \frac{2A_{s}f_{yd}}{s \cdot b_{pf} c_{d}} = \frac{2 \left( \frac{\pi}{4} \right) \left( \frac{400}{1.15} \right)}{125 \cdot 252 \cdot \left( \frac{30}{1.5} \right)} = 0.056$$
From equation (3) the required amount of volumetric ratio of confining hoops for the DCM and DCH can be determined according to EC8 (Refer Table 3 and Figure 6).

\[ \omega_{wd} = \frac{30 \mu \phi_v \cdot \varepsilon_{sy,d} \cdot \left( \frac{b_c}{b_o} \right) - 0.035}{\alpha} \]

<table>
<thead>
<tr>
<th>Axial load factor</th>
<th>Current value of ( \omega_{wd} )</th>
<th>Required ( \omega_{wd} ) for DCM</th>
<th>Required ( \omega_{wd} ) for DCH</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.2</td>
<td>0.0555</td>
<td>0.1914</td>
<td>0.3449</td>
</tr>
<tr>
<td>0.3</td>
<td>0.0555</td>
<td>0.3193</td>
<td>0.5497</td>
</tr>
<tr>
<td>0.4</td>
<td>0.0555</td>
<td>0.4473</td>
<td>0.7544</td>
</tr>
<tr>
<td>0.5</td>
<td>0.0555</td>
<td>0.5753</td>
<td>0.9592</td>
</tr>
</tbody>
</table>

Figure 7 \( \omega_{wd} \) vs Axial load factor

The Figure 7 shows the requirement of confinement for the ductility limitation which could be fulfilled only local demands. According to that, the significant value of transvers reinforcement should be provided in the primary section in order to satisfy the requirement. But, the numerical analysis (OpenSess) indications have been shown that the provided confinement in the satisfy the local demand. However, the global demands (DCM-10, DCH-16) haven’t meet and this is the compulsory requirement to fulfil the demands.

5. CONCLUSIONS

Basically, Ductility limitation need to be considered in terms of global and local demand as per the EC 8. But, the limitation for the global ductility is compulsory for the all primary section on the RC structure (depend on vulnerability) and including beams as well. However, if the beams are designed under the EC 2 avoiding the over reinforcement in flexure aspect, the section are accepted for the local ductility. As per the results obtained from the above analysis, it is showing that the local demand for the section have been achieved numerically. But theoretically, it is needed more confinement in the primary region to meet the ductility demand as per the EC 8. Simply, the demand and limitation shown in EC 8 are comparatively high with numerical results. However, none of these results were not met global ductility demand of 10 and 12 respectively. The results showing that the studied building is not satisfied the resistivity required for the moderate seismic condition. It is required to ensure the safety level of the public buildings for the moderate earthquake condition. This research could be extended to all the school buildings and hospital to get an overall understanding of the structure condition in Sri Lanka. By the way, the mandatory policy should be implemented for the new project at the design stage to consider the requirement.
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OpenSees, Open source software frame work, Berkley.
