Effect of Suppressed DOF on a Rigid RC Bridge Response under Strong Earthquake Motion

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Abstract: NLTHA (nonlinear time history analysis) is impractical for widespread use by the professional engineer because it requires long and inefficient computational time involving complexities when six DOF (degree of freedom) per node is applied. The NLTHA nowadays is predicted by MPA (modal pushover analysis). In this method, effects of higher modes on the dynamic response are considered to estimate seismic demands for structures. In this study, the effect of the reduction of number of DOF is analyzed using 3D NLTHA together with MPA of a rigid connection RC bridge under large earthquake motion. The results are compared with the 6 DOF NLTHA in terms of response of the structure and CPU time to obtain the most efficient computational effort. Result of NLTHA showed that the computational time of the structure both for 4 DOF (without two lateral torsional effects) and 3 DOF (without two lateral torsional and vertical displacements) was reduced significantly compared to the structure using 6 DOF. The reduction of computational time was close to fifty percent both for 4 and 3 DOF’s. When the maximum responses between NLTHA and MPA are compared, it is found that the differences are insignificant.

Key words: Rigid connection, RC multi span bridge, DOF, NLTHA, MPA.

1. Introduction

The NSP (nonlinear static procedure) has become a standard method in structural engineering practice for performance-based seismic evaluation of structures. Currently the most popular NSP is the CSM (capacity spectrum method). Research on the reliability of CSM has been carried out and reported in several scientific meetings [1–5]. However, the results are mostly limited to structures with dominant first mode contribution.

The other NSP is MPA (modal pushover analysis) proposed by Chopra and Goel [6]. The MPA method is a modification of the pushover analysis by incorporating the effects of higher mode responses to estimate the most probable maximum responses under seismic excitation to a structure. This method has shown being capable to predict the behavior of the building [7–9] and bridge [10, 11] structures under strong earthquake load.

In this paper, the study of MPA is extended for a structure of multi-span concrete bridge. The structure is modeled in SAP2000 [12].

2. Nonlinear Time History Analysis and Modal Pushover

The governing equilibrium equations of the MDOF (multi degree of freedom) system to horizontal earthquake ground motion \( \ddot{u}_g(t) \) are as follows:

\[
m \ddot{u} + c \dot{u} + ku = -m \ddot{u}_g(t)
\]

where, \( \mathbf{u} \) is the vector of \( N \) degree lateral displacements relative to the ground, \( \mathbf{m} \), \( \mathbf{c} \) and \( \mathbf{k} \) are the mass, damping, and lateral stiffness matrices of the system, each element of the influence vector \( \mathbf{l} \) is equal to unity.

In an inelastic system, the relations between lateral forces \( \mathbf{f} \) and the lateral displacement \( \mathbf{u} \) are not single valued, but depend on the history of the displacements:

\[
\mathbf{f}_s = f_s(\mathbf{u}, \text{sign} \dot{\mathbf{u}})
\]

With this generalization for inelastic systems, Eq. (1) becomes:
The standard approach is to solve directly these coupled equations, leading to the “exact” NLTHA results.

In developing MPA for inelastic structures, Eq. (3) is transformed to the modal coordinates of the corresponding linear system. Although it is not proper because modal analysis is not valid for inelastic system, it can be assumed that the initial state of inelastic condition, the inelastic system has the same properties (e.g., stiffness, mass and damping) with the elastic system [6]. Expanding the displacements of the inelastic system in terms of the natural vibration modes of the corresponding linear system [7]:

$$u(t) \cong \sum_{n=1}^{N} \phi_i(t) q_n(t)$$

where, \(\phi_i\) and \(q_n(t)\) are the nth natural vibration mode of the structure, and the modal coordinate respectively. Substituting Eq. (4) in Eq. (3), pre multi-plying by \(\phi_i^T\) and using the mass and damping orthogonality property of modes gives [7]:

$$q_n + 2\zeta_n \omega_n q_n + \frac{F_{in}}{M_n} = -\Gamma_n \ddot{u}_g(t) \quad n = 1, 2, 3, \ldots N \quad (5)$$

where,

$$\Gamma_n = \frac{L_n}{M_n}, \quad L_n = \phi_i^T m \phi_i, \quad M_n = \phi_i^T m \phi_i \quad (6)$$

in which \(\omega_n\) is the natural circular frequency and \(\zeta_n\) is the damping ratio for the nth mode. The solution \(q_n(t)\) can readily be obtained by comparing Eq. (5) to the equation of motion for the nth mode elastic SDOF (single degree of freedom) system subjected to \(\ddot{u}_g(t)\):

$$\ddot{D}_n + 2\zeta_n \omega_n D_n + \omega_n^2 D_n = \ddot{u}_g(t) \quad (7)$$

Comparing Eqs. (5) and (7) gives:

$$q_n(t) = -\Gamma_n D_n \quad (8)$$

and substituting in Eq. (4) the displacement is shown by means of the following equation:

$$u_n(t) = \Gamma_n \phi_i D_n(t) \quad (9)$$

Displacement that defined in Eq. (9) performed for each mode in order to the contribution of mass participation factor (\(\alpha\)) > 90%. The results from the displacement of each mode are then combined according to rules of SRSS (square root of sum of square) as follows:

$$r_n \approx \left( \sum_{n=1}^{N} r_{n\alpha} \right)^{1/2} \quad (10)$$

3. Description of Bridge Structure and Modeling

A concrete bridge with integral (rigid connection) and continues system will be evaluated in this study as seen on Fig. 1. The bridge is an extended version of the actual Cisomang Bridge [13, 14]. This bridge is located in the city of Purwakarta, West Java, Indonesia. The geometric configuration of the bridge is as follows: 154.8 m in length and 26.9 m in width and it consists of 4 spans each of which consisting of 8 girders with a span length of 35.6 m (see Fig. 2). The concrete compressive strength \((f'c)\) of the girder is 41.5 MPa. The girders and the piers are connected using rigid connection throughout the pier heads. The girders and the pier heads are modeled using beam-column elements in SAP2000.

The pier of the bridge is assumed to have the same height of 42.8 m. The piers are rectangular hollow (see Fig. 3) reinforced concrete members \((f'c = 30\) MPa and steel yield strength, \(f_y = 400\) MPa). The frames of the bridge consisting of piers and girders are developed using beam-column elements.

The moment-curvature capacity of the plastic hinge is defined based on the stress-strain relationship of the section taking account for the confinement effect of transverse reinforcement. The moment-curvature

Fig. 1  Layout of the bridge configuration.
The bridge piers are supported by 16 bored piles with diameter 1,000 mm connected with 21.9×7.6×2.0 m pile cap. The pile cap is modeled using shell element in SAP2000 nonlinear program. The combination of the bored pile and the pile-cap forming the substructure is designed to comply with the upper structure’s assumption of the level of fixity at the upper level of pile-cap (fixed base).

4. Ground Motion

The seismic loading is in accordance with the response spectra of Indonesian seismic hazard map (SNI-1726-2002) [16] for zone six with 500-year return periods where the soil is medium properties. Soil classification is determined using N-SPT and the mean shear wave velocity ($V_s$) together with the un-drained strength ($S_u$) in the first 30 m soil layer complying with the current Indonesian Seismic Code, SNI 03-1726-2002 [16]. SIMQKE [17] program is used in order to obtain the AEM (artificial earthquake motion) which is matched to the target spectra (see Fig. 5). SIMQKE [17] generated 1,235 S-waves with different phase angles, frequencies, and amplitudes. A response spectrum from the code is used in this analysis and no
specific seismic hazard assessment and site effect is developed because it is not required by the Authority.

5. Results and Discussion

Under the NLTHA, the number of DOF (see Fig. 6) is varied to obtain that effective and sufficiently to implement the analysis. The reduction of the number of DOF can be done by having minor contribution to the response of the structure to simplify the dynamic analysis. In this study, suppressing the number of DOF is conducted on the nodal connection between the pier and the girder of the bridge.

When the maximum lateral deformation occurred at the girder, plastic hinges developed at all piers with different curvatures while no plastic hinges developed at girders as expected in the design stage (see Figs. 7–8). However, at the end of the motion, both girders and piers underwent plastic hinges with different curvatures as shown on Figs. 9–10. The phenomena indicate the ductile near collapse performance of the bridge. The ductile collapse mechanism is due to the failure of bending moments both for pies and girders. No shear failure is observed because the shear strength was designed stronger the flexural strength as required by the code. The similar behavior is observed both for 3 and 4 DOF’s structures.

The results of NLTHA presented in Fig. 11 shows that the structure with 4 and 3 DOF gives fairly accurate results when compared with the structure of 6 DOF. Therefore, the system is effective in predicting the performance of structures against earthquake loads. The inelastic lateral displacement errors that occur in both systems are less than 13.0 percents. The errors are within the range of 2.40 to 12.40 percents and 0.10 to 6.30 percents for the earthquake motion along the longitudinal and transverse direction, respectively.
The contribution of the first mode only in the NLTHA is defined as Modal NLTHA (M-NLTHA). The M-NLTHA is conducted in the structures with 6, 4, and 3 DOF as shown in Fig. 12. It can be seen that the vector of the deflection shape shows that the first mode of transversal direction is dominant. The mode indicates that the maximum deflection occurred at the middle and zeros deflections at the abutments. The NLTHA using first mode only is not quite accurate in predicting the performance of structures when compared with the NLTHA which involves the contribution of all modes. The inelastic lateral displacement errors occurred in the third systems are within the range of 11.95 to 21.30 percents and 18.10 to 29.60 percents for the earthquake motion along the longitudinal and transverse direction respectively.

The dynamic properties of the bridge for linearly elastic vibration are shown in Table 1. Based on the dynamic properties then n modes are selected to achieve modal mass participating factor ($\alpha$) larger than 90%. The modal mass participating factor parameter is used as the basis of the MPA.

The non-linear stiffness characteristic of the structure obtained from pushover analysis using equivalent SDOF [6] is then subjected to a strong earthquake motion. The results of the deflection
Table 1  Elastic dynamic properties of the bridge.

<table>
<thead>
<tr>
<th>Mode</th>
<th>Period (Sec)</th>
<th>α (Modal mass participating factor)</th>
<th>Frequency (cyc/sec)</th>
<th>CircFreq (rad/sec)</th>
<th>Eigenvalue (rad²/sec²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2.44</td>
<td>0.77570</td>
<td>0.00000</td>
<td>0.41</td>
<td>2.57</td>
</tr>
<tr>
<td>2</td>
<td>0.89</td>
<td>0.00000</td>
<td>0.78593</td>
<td>1.12</td>
<td>7.05</td>
</tr>
<tr>
<td>3</td>
<td>0.56</td>
<td>0.17190</td>
<td>0.00000</td>
<td>1.79</td>
<td>11.26</td>
</tr>
<tr>
<td>4</td>
<td>0.55</td>
<td>0.00000</td>
<td>0.00000</td>
<td>1.81</td>
<td>11.40</td>
</tr>
<tr>
<td>5</td>
<td>0.54</td>
<td>0.01679</td>
<td>0.00000</td>
<td>1.84</td>
<td>11.57</td>
</tr>
<tr>
<td>6</td>
<td>0.53</td>
<td>0.00000</td>
<td>0.00000</td>
<td>1.89</td>
<td>11.89</td>
</tr>
<tr>
<td>7</td>
<td>0.53</td>
<td>0.00000</td>
<td>0.14445</td>
<td>1.90</td>
<td>11.93</td>
</tr>
<tr>
<td>8</td>
<td>0.52</td>
<td>0.00000</td>
<td>0.00975</td>
<td>1.93</td>
<td>12.10</td>
</tr>
<tr>
<td>Σ</td>
<td>96.44%</td>
<td>94.01%</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Fig. 13  Pier top displacement from MPA first mode only (SDOI) and NLTHA (MDOI): (a) Longitudinal direction; (b) transverse longitudinal.

Fig. 14  Pier top displacement from MPA (longitudinal direction).

time history is plotted and it is compared with the actual deflection of piers or girders resulted for NLTHA both for longitudinal and transverse directions as seen in Fig. 13. The results show that RHA (response history analysis) of MPA using first mode only and NLTHA show in a good agreement. The results of MPA presented in Figs. 14 and 15 showing that the MPA is in a good agreement with NLTHA in predicting the peak displacement at top of the pier. However, the method slightly underestimates in predicting inelastic lateral displacement of pier of bridge as shown in Table 2.
Fig. 15  Pier top displacement from MPA (transverse direction).

Table 2  Inelastic lateral displacement errors.

<table>
<thead>
<tr>
<th>Pier</th>
<th>U1 (longitudinal)</th>
<th>U2 (transverse)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>U_{Mode 1}</td>
<td>U_{Mode 1+2}</td>
</tr>
<tr>
<td>1</td>
<td>-11.43%</td>
<td>-0.96%</td>
</tr>
<tr>
<td>2</td>
<td>-12.80%</td>
<td>-2.49%</td>
</tr>
<tr>
<td>3</td>
<td>-14.27%</td>
<td>-4.12%</td>
</tr>
</tbody>
</table>

6. Conclusions

Based on the analysis NLTHA, M-NLTHA, and MPA, it can be concluded as follows:

The reduction of the number of DOF is appropriate to simplify the dynamic analysis. The results of NLTHA indicate the structure with 4 and 3 DOF gives fairly accurate results. In addition, result of NLTHA show that at least 3 DOF is used to obtain the most efficient for computational time as it reduces to half time as compared with the structure of 6 DOF.

Analysis is conducted using computer with specification as follows: Processor Intel Pentium M 1.73 GHz and 2 GB RAM. The computational times of NLTHA with 6 DOF required about 4.0 hours CPU time while NLTHA with 4 and 3 DOF’s took about 2.0 hours only. Therefore suppressing the 6 DOF to 4 or 3 DOF’s reduces the CPU time significantly.

The NLTHA with first mode only (M-NLTHA) is not quite accurate in predicting seismic demands of the structure when compared with NLTHA that involves the contribution of all modes. This is because modal mass participating for first mode of the system is less than 90 percents (only about 78 percent).

The MPA procedure has shown quite effective in estimating deformation demands. The number of modes used to conduct MPA affects the accuracy of the analysis. Therefore, the higher number of modes the better result is obtained in predicting the performance of structures under strong earthquake motion.

The limitations of developing AEM are as follows: (1) estimating the duration of the earthquake which is related to the amount of energy input imparted the structure, and (2) prediction of the shape of the amplitude’s envelope of an earthquake. If anyone or both limitations are different then the artificial seismic acceleration motion generated will produce different energy input even though they may result in the same response spectra. The energy input may have an influence to the collapse mechanism and inelastic response of a structure.

The limitation of the use of pushover analysis is that the modal shapes both for elastic and inelastic responses are constant. In fact they should be different.
However, this is the basic assumption in MPA where the modal shape of the elastic and the inelastic responses remain the same.

The MPA is applied to predict the maximum response only using the modal combination rules. The MPA method cannot be used to obtain the time history response as NLTHA.

References


