Multi-Mode Combination Load for Evaluating Seismic Performance of Single Column RC Bridges by Incremental Dynamic Analysis

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1. Introduction

Performing nonlinear time history analyses (NTHA) of structure under the set of monotonically increasing ground motions until the response of structure shows collapse is the key concept of Incremental Dynamic Analysis (IDA) which has been compiled and proposed by Vamvastikos and Cornell (2002). The result of this method contains necessary information for...
assessing the performance levels or limit-states of the structures which are important ingredients of Performance Based Earthquake Engineering (PBEE). Therefore, the IDA attracts the researchers and engineers to use it as the tool for evaluating the seismic behavior of the structures, e.g. Mander et al. (2007), Vejdani-Noghreiyani and Shooshtari (2008), Tehrani and Mitchell (2012), Tehrani and Mitchell (2013), Alembagheri and Ghaemian (2013), and Nazari and Bargi (2014). However, one disadvantage of this approach is the computational time because analysing the whole structures by the NTHA (Multi Degrees of Freedom system, MDOF) is time-consuming especially when highly nonlinear components were considered.

The concept of equivalent single-degree of freedom (ESDOF) has been investigated to mitigate disadvantage of using NTHA of MDOF in IDA. The basis of ESDOF originates from the concept of Nonlinear Static Analysis (NSA). The lateral behavior of MDOF under selected lateral force is generated by the Nonlinear Static Analysis (NSA) and applied to be the ESDOF system. Then, the lower computational price of NTHA of ESDOF will be performed to evaluate the IDA curve. Due to advantage of using ESDOF, Vamvastikos and Cornell (2005) proposed the empirical correlation between the standard pushover curves and IDA curves. FEMA P440A (2009) investigates the effect of stiffness and strength degradation on the seismic response of the structures by using concept of ESDOF. Because of using the result of the standard NSA to be the lateral behavior of the ESDOF, the drawback of the standard NSA has still remained in the IDA results, i.e. the effect of higher mode of vibration is not included in the standard NSA. Therefore, this method is improperly used for very flexible structures. Several advance IDAs have been proposed to account the higher mode effect, e.g. Zafam and Mofid (2005), Han and Chopra (2006), Han et al (2010), and Zafam and Mofid (2011).

There are several methods have been proposed in order to account the higher mode effect in NSA, e.g. Modal Pushover Analysis (MPA) (Chopra and Goel 2001), Energy-Based Pushover Analysis (EPA) (Hernandez-Montes et al 2004), and Adaptive Pushover Analysis (APA) (Antoniou and Pinho 2004). The MPA requires multiple analyses of NSA under various considered modal load patterns while EPA and APA require the experienced engineer who can interpret for using advanced computer programs. Therefore, there are not attractive proposition in engineering practice. Kunnath (2004) has proposed the method to include the higher mode effect in NSA by developing a new lateral load pattern using factored modal combinations. The higher modes contribution are accounted in this load pattern and only single-run NSA is required by this method.

Bridges are important structures and their seismic performance should be evaluated. Their structural behavior will vary between very stiff to very flexible structures depended on their
configurations. Due to the limitations of standard IDA by ESDOF, this paper aims to propose the seismic performance evaluation technique for the bridges based on IDA by ESDOF that account the higher mode effect but still retain the simplicity of the single-run analysis. Efficiency of proposed technique is investigated by comparison with the results of IDA of MDOF.

![Figure 1: Concept of incremental dynamic analysis of single column bridges.](image)

2. Concept of Seismic Performance Evaluation by Incremental Dynamic Analysis

2.1 IDA of Multi-Degree of Freedom System

In conventional IDA, the NTHA of MDOF analytical model are performed under the set of monotonically increasing ground motions in order to investigate the seismic behavior of the structure under considered ground motion until collapse. The scalars that can reflect the intensity of ground motion called Intensity Measure (IM) are collected at every step of scaling-up considered ground motion. The maximum response of the structure which can reflect the damage level of the structure called Damage Measure (DM) also are collected corresponding with IM.

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Plotting IM together with DM as shown in Figure 1 is called incremental dynamic analysis curve (IDA curve). It indicates the seismic behavior of the structure from under the small ground motion to the largest one that makes the structure collapse. Even if IDA of MDOF is a useful method, this method requires high performance computer, huge data storage and long computational time. There is the most important drawback of using this method for evaluating the seismic performance of the structures.

2.2 IDA by ESDOF based on Nonlinear Static Analysis

The concept of IDA based on nonlinear static analysis has been considered in order to reduce the computational price and time of IDA of MDOF (Vamvastikos and Cornell 2005). The lateral behavior of the structure is generated by the nonlinear static analysis (NSA) under a suitable lateral load pattern. The cyclic nonlinear static analysis will be performed if the hysteresis behavior of the structures is required. Then, the generated lateral behavior is defined for the single degree of freedom system (SDOF) to be equivalent to that of the MDOF structure. The mass of SDOF is defined by a value that makes the fundamental dynamic properties of SDOF equal to the MDOF. An SDOF system that has lateral behavior equivalent to that of an MDOF system is used to perform the IDA. The concept of ESDOF is shown in Figure 2. By using this method, the computational time can be reduced.

![Figure 2: Concept of incremental dynamic analysis by ESDOF based on nonlinear static analysis.](image)
2.2.1 Conventional ESDOF based on Standard Lateral Load Pattern

The lateral load pattern selection is the important issue of the NSA. Different load patterns give different NSA results and lead to different lateral behaviors of ESDOF (Chomchuen and Boonyapinyo, 2013). The lateral load patterns based on the seismic rehabilitation standard such as FEMA 356 are typically used. Either fundamental mode load pattern or uniform distribution mass load pattern can be selected according to this standard. The result of NSA under selected load pattern is lateral behavior of ESDOF.

2.2.2 Higher mode effect consideration in ESDOF

The higher mode effect is an important role in evaluating seismic performance of flexible structures by concept of ESDOF. FEMA 356 suggests that the higher mode effect can be accounted by either the concept of response spectrum analysis or adaptive load pattern.

For the load pattern of the concept of response spectrum analysis, the load distribution calculated by combining modal responses from a response spectrum analysis of the structure. For the concept of adaptive load pattern, the load distribution is adapted every time when the structure changes the properties, e.g. the yield point occurs.

Mofid et al. (2005) proposes the concept of combining Modal Pushover Analysis (MPA) and IDA to evaluate the seismic performance of the structures called Modal Incremental Dynamic Analysis (MIDA). Modal pushover analyses of considered modes are performed in order to obtain the lateral capacity of the structure under each mode of vibration. The obtained lateral capacities are defined to each single degree of freedom systems (SDOFs) to be the ESDOF. The NTHA of each ESDOF is performed in order to obtain the maximum displacement to be target displacement. Then, the MDOF structure is pushed to the obtained target displacement. The results of each mode are combined by modal combination rule, i.e. square root of sum of square (SRSS). From this concept, the higher mode effect is considered. Even if the computation time is reduced and the higher mode effect is accounted by this method, MIDA requests more time to setup modal ESDOFs and to interpret the huge results of each analysis.

2.2.3 Proposed Multi-Modes Combination Load Pattern

Multi-Modes Combination load pattern (MMC) is proposed in this study in order to account the higher mode effect, but retain the simplicity of single-run NSA and NTHA of ESDOF. The distribution of load can be generated by the simple combining of mode shapes which weighted by...
its modal participating mass ratios as follows:

\[ F_j = \sum_{n=1}^{N} \Gamma_n \phi_{nj} \]  

(1)

Where \( F_j \) is the force at the j degree of freedom, \( \Gamma_n \) is the modal participating mass ratio of \( n \) mode, \( \phi_{nj} \) is the mode shape value at j degree of freedom of n mode which are each normalized with respect to the mass matrix such that \( \phi^T_n M \phi_n = 1 \). \( N \) is the number of consideration modes.

The MMC is used to perform the displacement-based nonlinear static analysis by the mean of SAP2000 in this paper. Therefore, the distribution of load is the most important because SAP2000 monotonically increases the magnitude of load until the displacement of the monitor point reaches the specified value by kept the proportion of load as same as the specified load distribution. This method seems the same concept of the method of modal combination proposed by Kunnath (2004) for the force-based nonlinear static analysis. But, this study eliminates the few parameters in calculating force magnitude procedure to make it more simple, but still accurate to use for the displacement-based nonlinear static analysis.

3. Details of single-column reinforced concrete bridges

The single column bridge configuration is typically used in Thailand as shown in Figure 3(a). Therefore, this study uses this type of structure as case studies both to assess their seismic performance and to observe the application of a proposed technique for evaluating the seismic behavior of this kind of structures at the same time. Because they are widely used, bridges with three different column heights, as shown in Figure 3(b), are considered in order to investigate their effects on the seismic behavior. Details of the studied bridges are described as follows.

3.1 General configurations and analytical model

The studied bridges are reinforced concrete single column bridge. A specification describing a bridge in detail is referred to as its as-built drawing. The design strength of concrete are 240 and 350 kilogram-force per square centimeter (kg/cm² (ksc)) for in-situ reinforced concrete and precast pre-stressed concrete beam, respectively. A grade 40 deformed bar was used for reinforcement.

3.1.1 Superstructure

The superstructure of the studied bridges is an 18-centimeter-thick reinforced concrete slab placed on the top of five pre-stressed concrete I-girders as shown in Figure 4. It is assumed to be
elastic and is modeled as lumped single elastic beam-column elements. According to the bridges modeling for nonlinear analysis proposed by Aviram (2008), four elements per span are used in this study. The translational mass of the superstructure is automatically calculated and lumped to the nodes of beam-column element. Torsional mass, which affects the dynamic properties of the bridges especially in transverse direction, is also calculated and defined to the nodes of the elements.

(a) Typical single column bridges in Bangkok, Thailand.

(b) Bridges with three different column heights

Figure 3: General configurations of studied single column reinforced concrete bridges.

3.1.2 Bearing system

The bearing system of the studied bridges is an elastomeric bearing pad system as shown in Figure 5(a). It is modeled as an elastic spring element with six degrees of freedom. Three of the six degrees of freedom of spring element are shown in Figure 5(b). The stiffness in each degree of freedom is calculated according to the beam theory suggested by Yazdani et al. (2000). The shear modulus of the rubber depends on its hardness. A report by the Thammasat University Research and Consultancy Institute (TU-RAC) about an inspection of the bearing pads of an expressway in Bangkok shows that the hardness of the bearing pad’s rubber is about 60 Shore A scale.
(2010) suggest that an appropriate shear modulus of the rubber with 60 Shore A scale is 0.9 MPa. Although the effective compressive modulus of elasticity of the new bearing pads can be calculated directly from specified shear modulus according to the method in AASHTO (2010), Yazdani et al. (2000) have shown that aging and cold temperature elastomers may experience significant stiffening. The increase in stiffness may be as high as 50 times of the original stiffness.

![Diagram of bridge components](image)

(a) Bearing pad of the bridge  
(b) Three of the six degree of freedom spring element

**Figure 5:** Bearing system modelling.

### 3.1.3 Substructure

The substructure of the studied bridges is an octagon reinforced concrete column with a top slab. The cross-section of the column is 1.60 x 1.60 m, see Figure 6(a). For nonlinear static analysis, the inelastic behavior is modeled using the lumped hinge technique. The lumped axial-bidirectional moment interaction hinge (P-M-M) is used in this study. Moment-curvature of the specified column cross-section is evaluated and shown in Figure 6(c). The column reaches the limit when the extreme fiber of the cross-section reaches the ultimate compression strain. When the column is subjected to an axial load, the moment-curvature is adjusted automatically by considering the interaction diagram shown in Figure 6(b).

Due to a limitation of SAP2000, the plastic hinge length element with lumped hinge should be replaced by a nonlinear spring element to model the inelastic behavior of the bridges for nonlinear time history analyses. The moment-curvature of the column cross-section is evaluated taking into account the axial load and is defined to the nonlinear spring element. The fundamental dynamic properties of the nonlinear time history analytical model were compared to those of the nonlinear static analytical model to make sure that the dynamic properties of both models are the same.

The top of the column is rigidly connected to the 1.33-meter-thick cast-in-place reinforced concrete slab as shown in Figure 6(d). It is modelled as elastic thick shell element. This element is used to realistically model the mass distribution of the slab. Because of the high of thickness-to-width ratio of the slab, a thick element is used to account for the influence of shear deformation.
A gap element is used to model the gap between the superstructure and the substructure. The initial gap was set to 2 centimeters which is the same as the details shown in the as-built drawing. The stiffness of the gap element is set to about 100 times the axial stiffness of the superstructure in order to avoid rounded errors (CSI, 2011).

According to the modeling concepts described above, the inelastic analytical model of the bridges is shown in Figure 7(a). Figure 7(b) shows the analytical model of the studied bridges in the computer program.
4. Dynamic Properties of Studied Bridges

This study uses method of modal analysis to evaluate both mode shapes and modal frequencies of the studied bridges in order to compare with those results from field testing results.

The results show that the fundamental mode of vibration of the single column bridge is the oscillation of the structure in a direction perpendicular to the traffic direction called the transverse direction. The oscillation in a parallel traffic direction is called the longitudinal direction. The shapes of the fundamental mode of vibration in the transverse and longitudinal directions are shown in Figure 8(a) and 8(b), respectively. The dynamic properties of both directions of vibration are summarized in Table 1.

![Figure 8: Fundamental vibration mode of studied bridges in transverse and longitudinal direction](image)

**Table 1: Fundamental dynamic properties of vibration of three studied bridges**

<table>
<thead>
<tr>
<th>Column Height (m.)</th>
<th>Transverse Direction</th>
<th>Longitudinal Direction</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Period (sec.)</td>
<td>Frequency (Hz.)</td>
</tr>
<tr>
<td>4.5</td>
<td>0.450</td>
<td>2.224</td>
</tr>
<tr>
<td>6.3</td>
<td>0.610</td>
<td>1.640</td>
</tr>
<tr>
<td>15.0</td>
<td>1.746</td>
<td>0.573</td>
</tr>
</tbody>
</table>

To validate the analytical model and to calculate dynamic properties of the bridges, the calculated frequencies of a bridge with 6.3 meter column height were compared to those from field testing results. The frequencies of the bridges in the transverse and the longitudinal directions of the field test are 1.60-2.00 Hz and 2.00-2.80 Hz, respectively. These numbers show that the calculated frequencies are in the range of the field testing results. Therefore, the analytical model can be used to reliably evaluate the seismic performance.

The mass participation ratios of all bridges were also shown in Table 1. These results show that the vibration of a bridge with tall columns is dominated by the fundamental mode. Also, the higher modes may be more influential when the bridge column height decreases.
According to the Seismic Retrofitting Manual for Highway Bridges published by the Federal Highway Administration (FHWA) in 2006, the invisible crack in the structural member may affect the flexural rigidity of the members and should be considered in the seismic evaluation. This study also considered the effect of cracked section in the seismic performance evaluation process. Then, the flexural rigidity of the reinforced concrete column of all studied bridges was reduced to half of that of gross-section ($0.5E_{ig}$).

5. **Artificial Ground Motions**

In evaluating the seismic performance of bridge structures, the ground motion should be selected carefully because each ground motion form each earthquake has its own unique characteristic. According to the seismic retrofitting manual for highway bridges published by the Federal Highway Administration (FHWA 2006), the maximum response of the three ground motions should be used for evaluating performance. This study uses the design spectrum for the inner area of Bangkok, Thailand (DPT 2009) as the target spectrum as shown in Figure 9(a). The ground motions were generated by matching the response spectrum with the target spectrum. Three artificial ground motions generated by SeismoArif and referred to as A1, A2, and A3 were shown in Figure 9(b). According to the concept of IDA, the generated ground motions will be scaled up until the DM of the structures show collapse.

![Comparison of response spectrum of generated ground motions with the design spectrum](image1)

![Three artificial ground motions](image2)

**Figure 9:** Artificial ground motions generated corresponding with the design spectrum for the inner area of Bangkok

*Corresponding authors (P. Chomchuen). Tel: +66-2-9883666 Ext.3219; cprakit@gmail.com, and (V.Boonyapinyo) bvirote@engr.tu.ac.th. © 2015. American Transactions on Engineering & Applied Sciences. Volume 4 No.3 ISSN 2229-1652 eISSN 2229-1660 Online Available at [http://TUENGR.COM/ATEAS/V04/139.pdf](http://TUENGR.COM/ATEAS/V04/139.pdf).
6. **Nonlinear Static Analysis of the Studied Bridges**

Nonlinear static analyses of the studied bridges were performed to generate its lateral behaviors. An important parameter that strongly affects the lateral behavior of the structures is the lateral load pattern (Chomchuen and Boonyapinyo 2013).

6.1 **Lateral Load Pattern**

6.1.1 **Standard-based Load Pattern**

In evaluating the seismic behavior of structures using methods based on the NSA, international standards, such as FEMA 356, suggest that at least two lateral load patterns should be considered. One of the load patterns can be selected from the following two categories.

The first category is the modal pattern. The distribution of vertical load can be selected from one of the following: The distribution proportional to the shape of the fundamental mode in the direction under consideration, and the distribution proportional to the story shear distribution calculated by combining modal responses from a response spectrum analysis of the building, including sufficient modes to capture at least 90% of the total building mass, and using the appropriate ground motion spectrum.

For the second pattern, the distribution can be selected from one of following: uniform distribution consisting of lateral forces at each level proportional to the total mass at each level (or uniform acceleration load distribution), and adaptive load distribution that changes as the structure is displaced.

This study uses two lateral load patterns selected from the two categories above in order to investigate the effects of different lateral load patterns on the seismic behavior of single-column RC bridges as evaluated using the concept of ESDOF IDA. The first load pattern distributes the load proportionally to the fundamental mode in the transversal direction. We refer to this load pattern as “First mode load pattern (1st)”. The second load pattern is the uniform distribution consisting of lateral forces at each level proportional to the total mass at each level. We refer to this load pattern as “Uniform acceleration load pattern (Unif)”.

6.1.2 **Proposed Multi-Modes Combination Load Pattern**

Concept of the proposed MMC load pattern was described in the previous section 2.2.3. The number of mode combination should be appropriately considered. For the MMC in this study, the higher modes were combined to capture at least 80% of the total mass for the studied bridges with
short and medium column height. For the studied bridge with tall column height, the combination of four lateral modes was also achieved. Mode shapes and participating mass ratios of higher mode were shown in Table 2.

Table 2: Mode shapes and participating mass ratios of three studied bridges in transverse direction for MMC load pattern.

<table>
<thead>
<tr>
<th>Mode Shape</th>
<th>Participating Mass Ratio (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>4.5 m column height</td>
</tr>
<tr>
<td>(First mode of column)</td>
<td>53.3</td>
</tr>
<tr>
<td></td>
<td>0.53</td>
</tr>
<tr>
<td></td>
<td>0.12</td>
</tr>
<tr>
<td>(Second mode of column)</td>
<td>28.4</td>
</tr>
</tbody>
</table>

6.2 Effect of Lateral Load Pattern on the Lateral Capacity of the Bridges

The lateral behavior of the studied bridge with 4.5, 6.3, and 15 meter column heights from three selected lateral load patterns is shown in Figure 10.

The modal participating mass ratios of the bridge with short, medium, and tall column are 53.3, 68.3, and 86.6 percent, respectively. For all three bridges, the maximum capacity obtained from Unif higher than those obtained from 1st. They were about 1.62, 1.33, and 1.13 times for the bridges with short, medium, and tall column respectively.

(a) 4.5 meters column height   (b) 6.3 meters column height    (c) 15.0 meters column height

Figure 10: Lateral capacity of three studied bridges in transverse direction.

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The results show that different lateral load patterns lead to dramatically different lateral capacities of the bridge structures, especially the bridge with the least modal participating mass ratio in the considered direction. The difference in lateral capacities of the bridges decreases when the modal participating mass ratio increases. These results are due to the effects of the higher mode.

Including higher mode effect by the mean of MMC in this study, the lateral load pattern is the combination of mode shapes with considering the weight of modal participating mass ratio until the modal participating mass ratio in the considered direction more than 80 percent. The results show that the lateral behavior of the bridge structures obtained from MMC stronger than those from 1st and slightly weaker than those from Unif.

7. Incremental Dynamic Analysis of Studied Bridges by ESDOF with Various Load Patterns

The IDA of ESDOFs are performed in order to investigate the effects of different lateral load patterns on the IDA curves obtained by the ESDOF. The IDA curves of ESODF are compared to the IDA curves of the MDOF and shown in Figure 11. The IDA curves obtained from the ESDOF with lateral capacity from 1st and Unif are referred to ESDOF_PT1 and ESDOF_PTU respectively. The hysteresis behavior of the ESDOF is the pivot hysteresis rule.

Figure 11 shows that The ESDOF_PTU shows slightly stiffer seismic behavior than the NTHA of MDOF while the ESDOF_PT1 shows much weaker seismic behavior than the NTHA of MDOF for the short column bridge. For the medium column bridge, the ESDOF_PTU gives moderately stiffer IDA curves while the ESDOF_PT1 shows moderately weaker one than NTHA. The different lateral load patterns insignificantly affected the IDA curve of the tall column bridge.

Figure 11 also shows that the MMC give the most accurate IDA among the three difference load patterns compared with the results of MDOF. It is implied that combining the mode shape for MMC until the modal participating mass ratio more than 80 percent (three modes for studied bridges) is enough for giving the acceptable seismic behavior. This figure also shows that the 1st and Unif can be used for evaluating the seismic behavior as the lower-bound and upper-bound aspect, respectively. It should be noted that the computational time for NTHA of MDOF and ESDOF with MMC are 60 minutes and only 4 minutes per load case, respectively.
The results show that lateral load patterns strongly affect the seismic behavior of the bridge structure, especially for bridges with low modal participating mass ratios in the fundamental mode as well as bridges with short columns. This effect decreases when the bridge column height increases. It may be concluded that this is result of higher mode effect.

Figure 11: Comparison of IDA curves of three studied bridges under three artificial ground motions obtained from NTHA of MDOF and ESDOF with three different lateral capacities in transverse direction.
8. Conclusions

This study investigated efficiency of proposed technique for evaluating seismic performance of the three studied bridges under three artificial ground motions by the mean of IDA by ESDOF. The results lead to the following conclusions.

• The use of the concept of ESDOF for evaluating the seismic performance of the studied bridges by the mean of IDA can reduce the computational time from 60 minutes per load case for MDOF to 4 minutes per load case for ESDOF (a reduction of about 93%).

• The effect of different lateral load patterns on the seismic behavior of the studied bridges decreases when the bridge column height increases. Therefore, either the first mode or the uniform acceleration lateral load patterns based on FEMA 356 can be used effectively in applying the concept of ESDOF for evaluating the seismic performance of the studied bridges that is dominated by the fundamental mode (participating mass ratio higher than 80% of the total mass as is the case for the 15- meters column height in this study).

• The seismic performance of the studied bridges with the participating mass ratio less than 80% of the total mass (4.5 meters and 6.3 meters column heights in this study) can be evaluated by the ESDOF with the load patterns in accordance with FEMA 356. However, the ESDOF with the first mode load pattern leads to an overestimate of the seismic responses while the uniform acceleration results in an underestimate of the seismic responses.

• The ESDOF with the proposed multi-modes combination load pattern with single-run NSA and IDA gives better accuracy of seismic performance than standard load patterns for all studied bridges, compared with MDOF.

9. Acknowledgements

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10. References


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