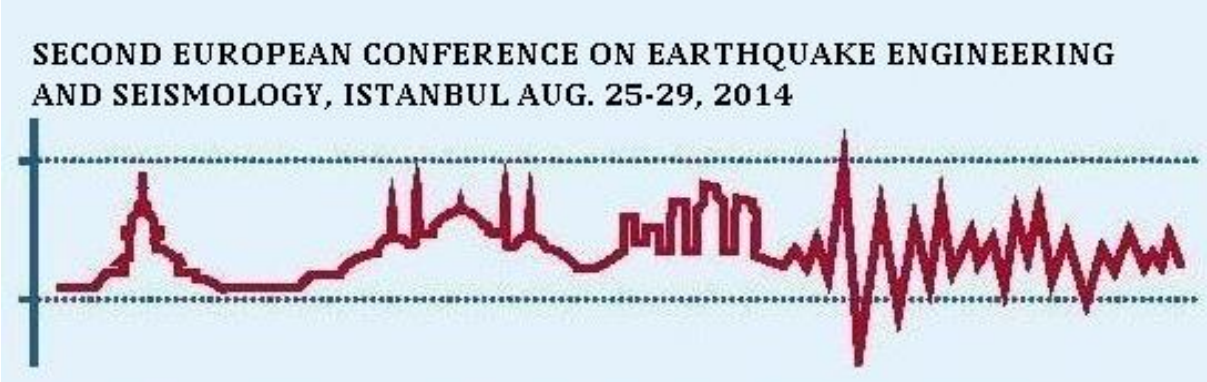

EFFECTS OF HYSTERESIS RULES ON SEISMIC BEAHVIOR OF SINGLE COLUMN RC BRIDGES BY NONLINEAR DYNAMIC ANALYSIS

Virote BOONYAPINYO[1] and Prakit CHOMCHUEN[2]

ABSTRACT

This study aims to investigate the effect of different hysteresis rules on the seismic behavior of the single column RC bridges, which typically used in Bangkok, Thailand. The incremental dynamic analysis of the equivalent single degree of freedom system (EDOF) was applied in this study. Three different bridge column heights, such as short, medium, and tall, were selected to be the case studies. The ground motions were generated corresponding with the design spectrum for the inner area of Bangkok. Two different lateral load pattern, such as first mode load pattern and uniform acceleration load pattern, were applied to the studied bridges and perform nonlinear static analysis to evaluate its lateral behaviors. Generated lateral behaviors with three different hysteresis models, such as kinematic model, takeda model, and pivot model, were defined to the ESDOF to generate the IDA curves of the studied bridges. The results were compared to the IDA curves generated by nonlinear time history analysis of multi degree of freedom of the studied bridges.

The study shows that the pivot hysteresis model and takeda hysteresis model result in similar IDA curves, but using ESDOF with the kinematic hysteresis model do not agree well for IDA curves with NTHA in some cases. The pivot model lead to slightly more accurate hysteresis than takeda model because the pivot model requires input parameters to generate the hysteresis. However, the necessary parameters for modelling hysteresis by takeda model are automatically generated by the program. Therefore, the takeda model will be the most practical hysteresis model.

INTRODUCTION

Nonlinear time history analyses (NTHA) of structure under monotonically scaling up considered ground motion until the response of structure shown collapse is the key concept of Incremental Dynamic Analysis (IDA) which has been compiled and proposed by Vamvastikos and Cornell (2002). The monotonic scalable ground motion intensity measure (IM) was plotted together with a damage measure (DM) called incremental dynamic analysis curve (IDA curve). The IDA curve contains the necessary information to assess the performance levels or limit-states of the structures which are important ingredients of Performance Based Earthquake Engineering (PBEE). The IDA attracts researchers and engineers to use as the tool for evaluating the seismic behavior of the structures, e.g. Mander et al. (2007), Vejdani-Noghreiyan and Shooshtari (2008), Tehrani and Mitchell (2012), Tehrani and Mitchell (2013), Alembagheri and Ghaemian (2013), and Nazari and Bargi (2014).

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To reduce the computational time and the price of using the series of Nonlinear Time History Analysis (NTHA) for generating each IDA curve of Multi-Degree-of-Freedom structures (MDOF), the concept of Equivalent Single-Degree-of-Freedom (ESDOF) have been proposed. The lateral behavior of MDOF under the lateral force is evaluated by the Nonlinear Static Analysis (NSA) and apply to be the equivalent behavior of the Single-Degree-of-Freedom system. Then, the lower computational price of NTHA of ESDOF will be performed to evaluate the IDA curve. 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 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. To account the higher mode effect, several advance IDAs have been proposed, e.g. Zafam and Mofid (2005), Han and Chopra (2006), Han et al (2010), and Zafam and Mofid (2011).

Using IDA by ESDOF for evaluating the seismic behavior of the bridge structures, Chomchuen and Boonyapinyo (2013) has found that the different lateral load patterns lead to significantly different shape of lateral behaviours and IDA curves. It is not only the shape of lateral behavior that strongly influences on the seismic behavior of the structures evaluated by ESDOF concept, but the selecting of the different hysteresis rule also important and it is not yet clearly explained. Therefore, this study will investigate the effect of different hysteresis rules for modelling the hysteresis behavior of the ESDOF on the seismic behavior of the single column RC bridges.

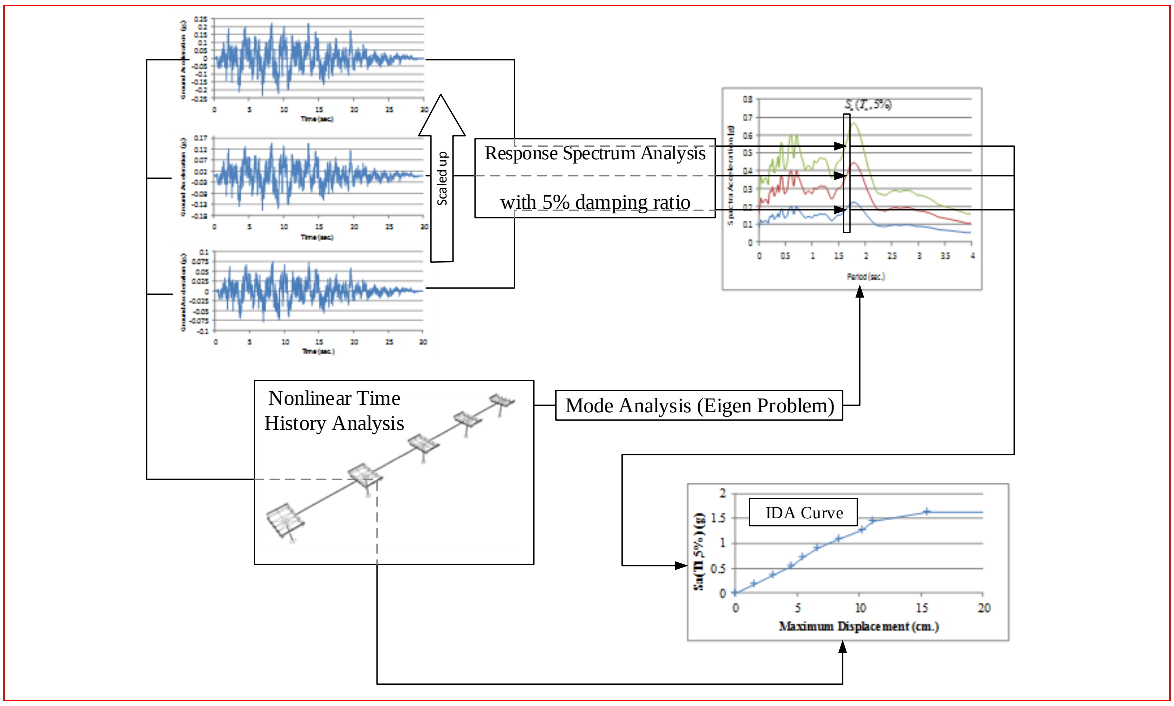


Figure 1. Concept of incremental dynamic analysis of single column bridges

IDA OF MULTI-DEGREE OF FREEDOM SYSTEM

The conventional IDA, the NTHA of MDOF analytical model under the set of monotonically increasing considered ground motions were performed to investigate the seismic behavior of the structure under considered ground motion until collapse. The scalars that can be used to reflect the intensity of ground motion called Intensity Measure (IM) were 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 were collected and observed at every result of NTHA of MDOF. Plot IM together with DM as shown in Fig.1 called incremental dynamic analysis curve (IDA curve). It indicates the seismic behavior of the structure, form under the small considered ground motion to the largest considered ground motion that make the structure collapse. Even if it is the useful method, the computational price and time are the most important drawback of using this method for evaluating the seismic performance of the structures.

IDA BY EQUAVALENT SINGLE DEGREE OF FREEDOM

To reduce the computational price and time of IDA of MDOF, the concept of IDA based on nonlinear static analysis has been considered (Vamvastikos 2005). The lateral behavior of the structure was generated by the nonlinear static analysis (NSA) under 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 was defined to the single degree of freedom system (SDOF) to be equivalent of MDOF structure. The SDOF which has the equivalent lateral behavior of MDOF was used to perform the IDA. The concept of ESDOF is shown in Fig.2. By using this method, the computational-time can be reduced effectively.

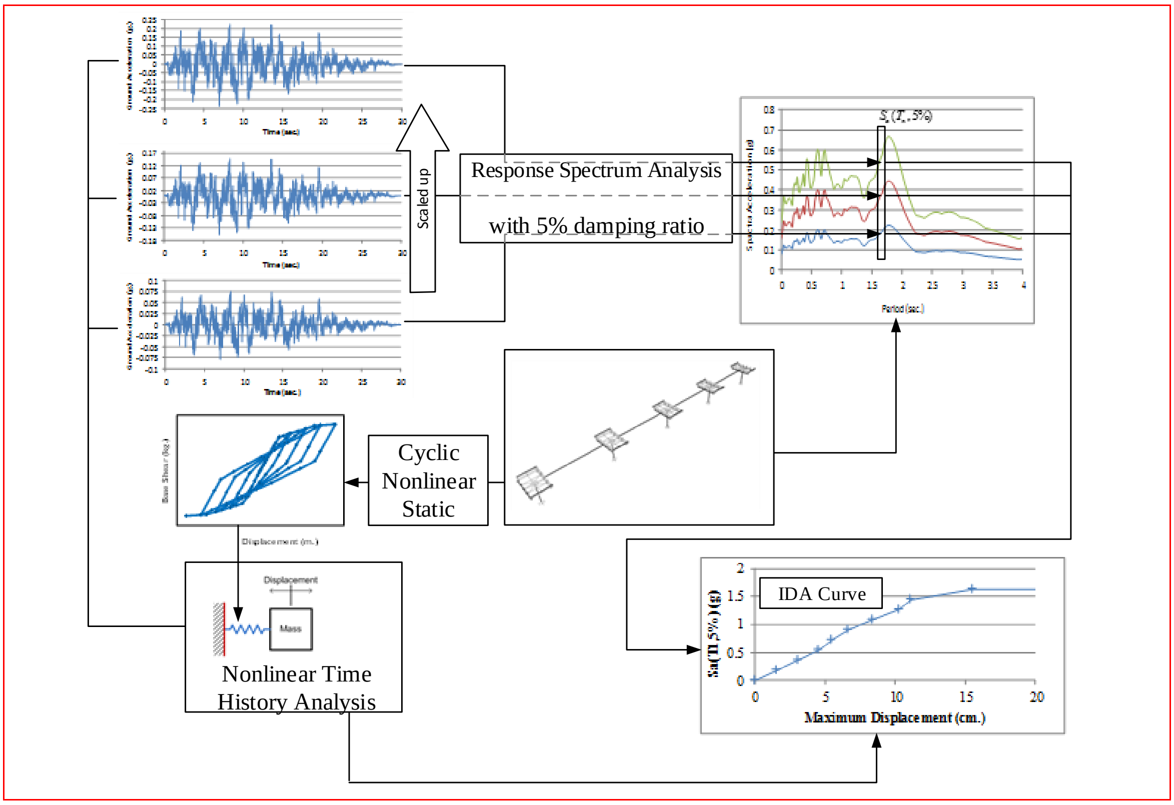


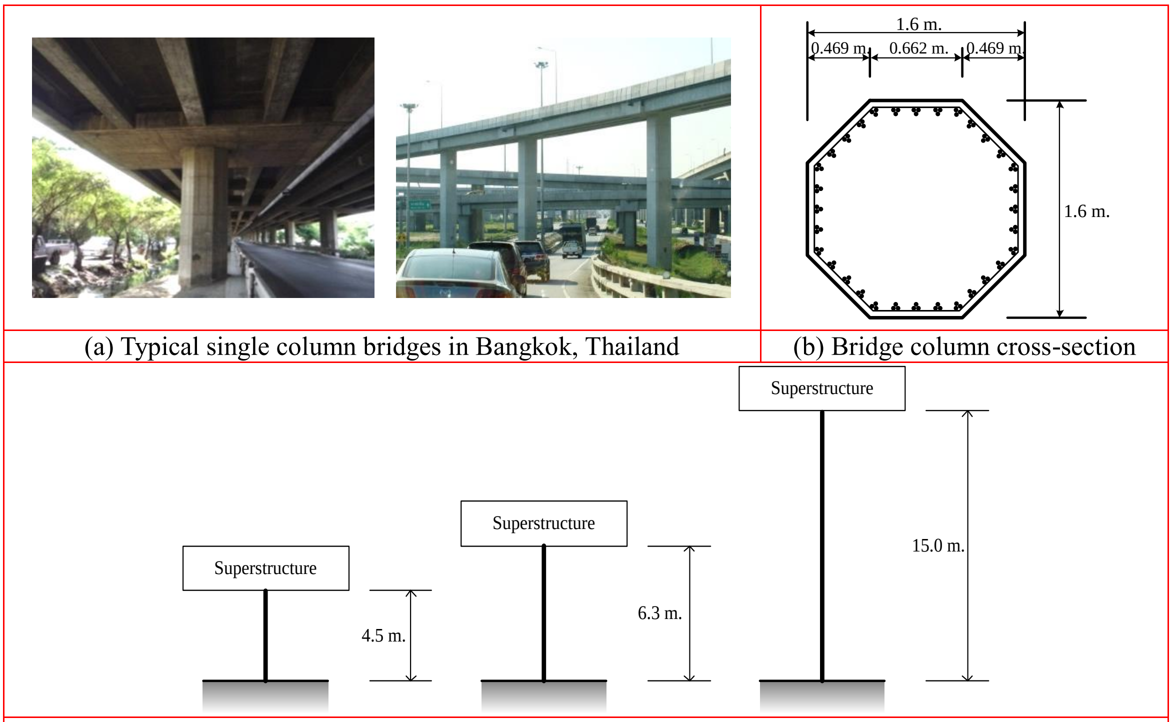
Figure 2. Concept of incremental dynamic analysis of single column bridges

CASE STUDIES OF SINGLE-COLUMN REINFORCED CONCRETE BRIDGES

The single column reinforced concrete bridges were widely used in Thailand. The typical configuration of the bridges is shown in Fig.3(a). This figure also shows that this bridge type was used with various column heights depend on the required function. Therefore, three different column heights with same all other configurations were used to be the case studies in this paper.

Details of Studied Bridges

Structural system of the bridges consists of the superstructure, reinforced concrete top-slab, and single octagon reinforced concrete column (Fig.3(b)). Three different bridge's column heights, i.e., 4.5 m., 6.3 m., and 15.0 m., as shown in Fig.3(c), are used to investigate the effect of column flexibility on the seismic performance of the bridges and efficiency of evaluation methods in evaluating the seismic performance of the bridges with different column flexibilities.



(a) Typical single column bridges in Bangkok, Thailand

(b) Bridge column cross-section

(c) Bridges with three different column heights

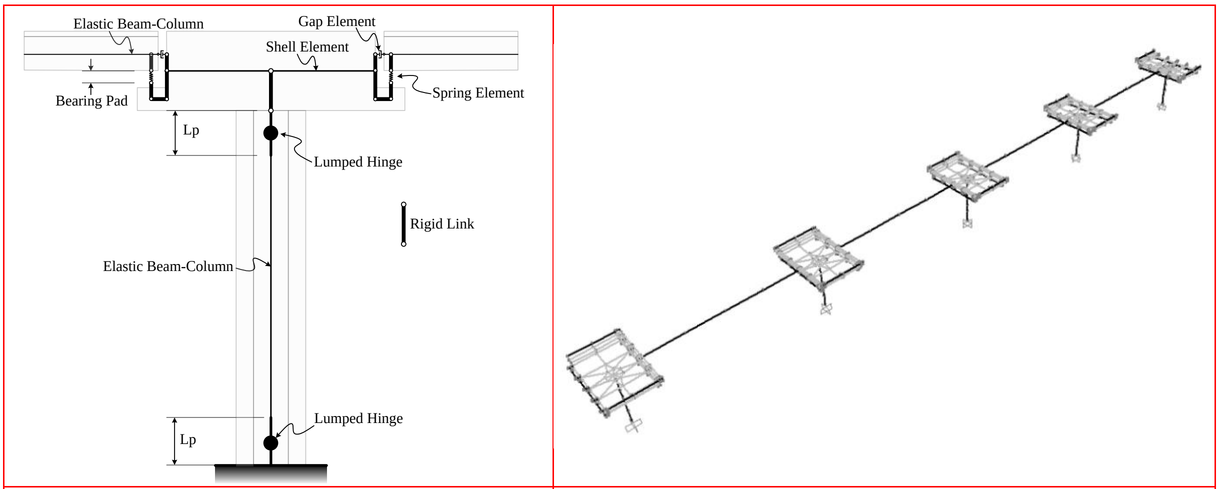
Figure 3. General configurations of studied single column reinforced concrete bridges

Analytical Model of Studied Bridges

The concept of the analytical model of studied bridges is concluded and shown in Fig.4. The superstructure is assumed to be elastic and modeled by lumped single elastic beam-column elements. Four elements per span are used in this study. The translational mass of the superstructure is automatically calculated and lumped to the nodes of the beam-column element. Rotational mass moment of inertia, which affect to the dynamic properties of the bridges, especially in transverse direction, is also calculated and defined to the nodes of the superstructure elements accordance with Aviram et al. (2008).

The substructure is also modeled by the elastic beam-column element. Inelastic behavior of the studied bridges is modeled by the lumped plastic hinge technique for nonlinear static analysis. The inelastic behavior of the plastic length member is lumped to a point at the center of the element as shown in Fig.4(a). The inelastic behavior which should be defined to the lumped plastic hinge is the Moment-Curvature ($M-\phi$) relationship of the cross-section of bridge's column. For nonlinear dynamic analysis, the non-linear spring is used to model the inelastic behavior of the plastic hinge length instead. Top of the column is rigidly connected to the 1.33 m. thickness cast-in-place reinforced concrete slab as shown in Fig.4(a). It is modeled by elastic shell element. Mass of the top-slab is automatically lumped to the nodes. Because the nodes are distributed along the slab area, the translational mass may produce the torsional rotation of the top slab already. Then, torsional mass is not defined to the top slab.

Bearing system of the studied bridges is modeled by the elastic six degree-of-freedom spring element. The stiffness of each degree of freedom is calculated from the beam theory (Yazdani et al. 2000). The boundary conditions at the bottom of the bridge's columns were assumed to be fixed supports. The analytical model in the computer program is shown in Fig.4(b).

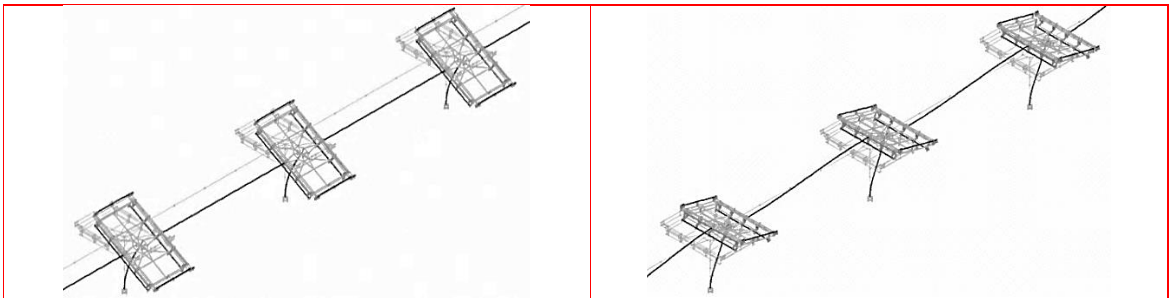


(a) Details of bridge component modeling | (b) Analytical model of the bridge in computer program

Figure 4. Details of the analytical model of the studied bridges

Dynamic properties of the studied bridges

Dynamic properties of three different bridge's column heights are investigated in this study. The first transverse mode shape and first longitudinal mode shape of studied bridges are shown in Fig.5(a) and Fig.5(b), respectively. The periods and frequencies of all bridges are shown in Table 1. The dynamic properties show that the shorter bridge's column is stiffer than the longer bridge's column and the bridge's behavior in the longitudinal direction is stiffer than transverse direction. The frequencies of the bridge with 6.3 m. column height were compared to the field test data. The frequency of the bridges in the transverse direction and longitudinal direction of field tests is 1.60-2.00 Hz and 2.00-2.80 Hz, respectively. It shows that the frequencies of the analytical model are in the range of the field test data.



(a) Transverse mode shape | (b) Longitudinal mode shape

Figure 5. Fundamental mode of vibration of the studied bridges in transverse and longitudinal direction

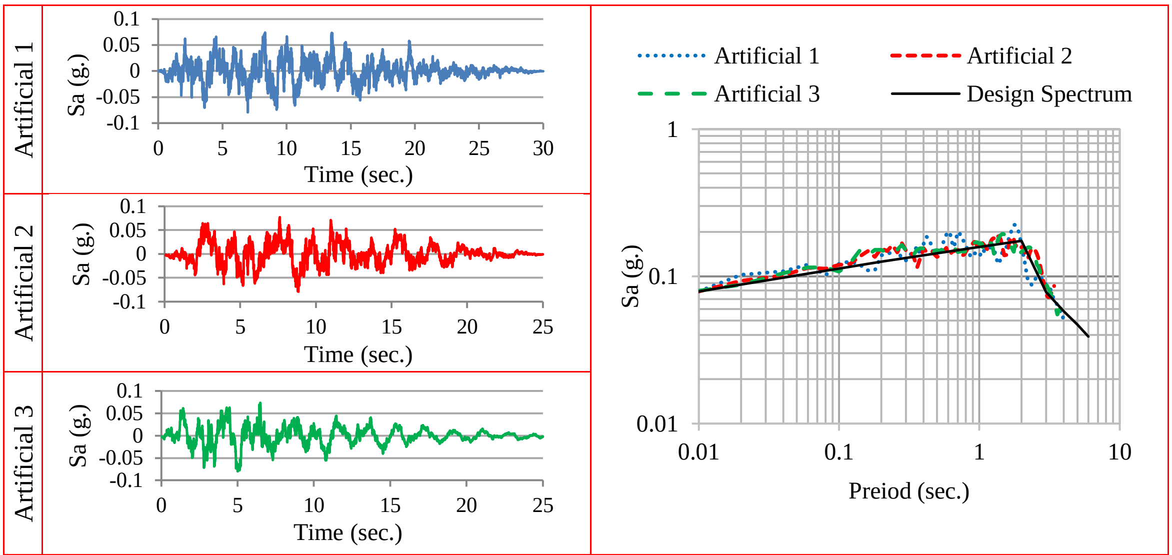
Table 1. Fundamental dynamic properties of vibration of three studied bridges

Column Height (m.)	Transverse Direction			Longitudinal Direction		
	Period (sec.)	Frequency (Hz.)	Modal Participating Mass Ratio (%)	Period (sec.)	Frequency (Hz.)	Modal Participating Mass Ratio (%)
4.5	0.450	2.224	53.3	0.272	3.679	66.2
6.3	0.610	1.640	68.3	0.358	2.796	87.1
15.0	1.746	0.573	86.6	0.980	1.020	92.8

Table 1 also contains the modal participating mass ratios of the studied bridges. Considering the transverse direction which is the weakest direction, the fundamental mode dominates about 86% of total mass for the studied bridge with tall column. The influence of the domination of fundamental mode of vibration decreases when the bridge's height decreases. It should be noted that the bridge with short column (stiff bridge) should consider the higher mode effect.

ARTIFICIAL GROUND MOTIONS

To evaluate the seismic performance of the 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 under three ground motions should be used for evaluating seismic performance of the bridge structures. This study uses the design spectrum for the inner area of Bangkok, Thailand (DPT 2009) to be the target spectrum as shown in Fig. 6(b). Because the concept of IDA that the considered ground motion will be scaled up until the DM of the structures show collapse, The ground motions were generated by matching the response spectrum with the target spectrum. Three artificial ground motions generated by SeismoArif program and referred as A1, A2, and A3 were shown in Fig. 6(a). The response spectrums of the artificial ground motions were compared to the design spectrum as shown in Fig.6(b).



(a) Three artificial ground motions

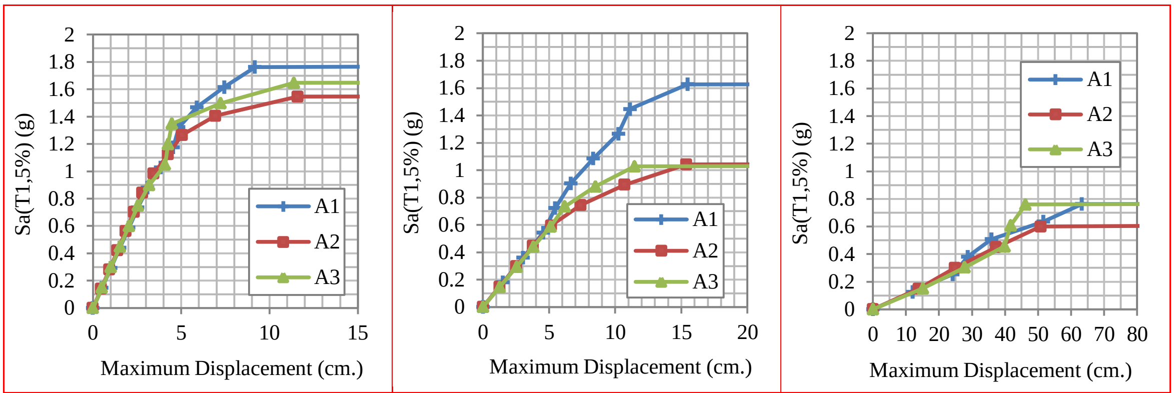
(b) Comparison of response spectrum of generated ground motions with the design spectrum

Figure 6. Artificial ground motions generated corresponding with the design spectrum for the inner area of Bangkok

INCREMENTAL DYNAMIC ANALYSIS OF MDOF STUDIED BRIDGES

To verify the effect of different hysteresis modeling on the seismic performance of the studied bridges, the IDA of MDOF in this study was achieved by SAP2000 program. The concept of this method is shown previously in Fig.1. Selected intensity measurement (IM) of this study is the spectrum acceleration at the fundamental mode of the structures with 5% damping ratio. Each IM of scaled ground motions were plotted together with the corresponding maximum transversal displacement of the top of the middle column to be the incremental dynamic analysis curve (IDA curve) as shown in Fig.7.

Fig.7 shows that the bridge with shorter column has more seismic performance than the longer column height. On the other word, when considered only the flexural failure, the performance of the bridges decreases when the height of bridge's column increases.



(a) 4.5 m. column height | (b) 6.3 m. column height | (c) 15.0 m. column height

Figure 7. IDA curves of three studied bridges under three artificial ground motions in transverse direction

NONLINEAR STATIC ANALYSIS OF STUDIED BRIDGES

Lateral Load Patterns

It is obviously that the lateral load pattern is an important factor that influences on the lateral capacity of the structures. FEMA356 (2000) have suggested that at least two lateral load patterns should be considered. These two load patterns are selected from two groups separately. The first group includes an inverted triangle and the fundamental modal shape load pattern. The second group includes a lateral load pattern that is proportional to mass and adaptive loading pattern.

This study uses two built-in lateral load patterns in SAP2000, i.e. 1st mode load pattern (1st) to be the load pattern from the first group and uniform acceleration load pattern (Unif) to be the mass proportional load pattern, to study the effect of load patterns on the lateral capacity of the bridges.

The 1st is the fundamental mode lateral load pattern. The lateral force at any degree-of-freedom is proportional to the product of the amplitude of the fundamental mode and the mass at that node.

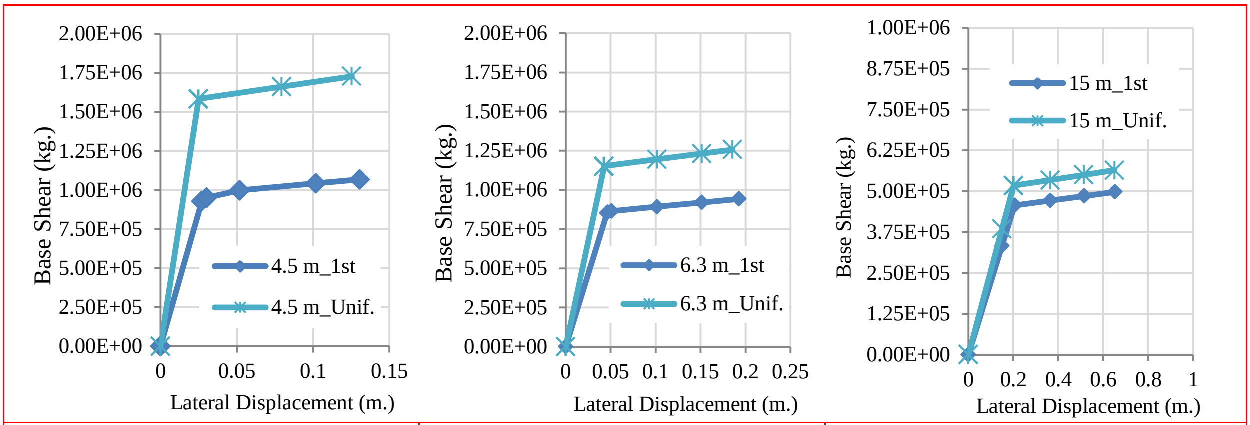
The pattern for the mass proportional load in this study is the Unif. The unit acceleration was automatically assigned to all degree-of-freedom corresponding with the defined direction (CSI 2011). The pattern of force which is the product of mass and unit acceleration (mass proportional distribution) is used to be the Unif.

Lateral Capacities of Studied Bridges

Applying two selected load patterns separately to the MDOF model of studied bridges, the NSA of all studied bridges was carried out by SAP2000. The relationship between the base-shear and lateral displacement of the top of middle column of every load increments were plotted together to be the lateral capacity of the bridges. The lateral behavior of the studied bridge with 4.5, 6.3, and 15 meter column height were shown in Figs.8(a)-(c), respectively.

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

The results shown that different lateral load pattern leads to the highly different lateral capacity of the bridge structures, especially the bridge with least fundamental modal participating mass ratio in the considered direction. The different of the lateral capacity of the bridges decrease when the modal participating mass ratio increases. It is the results of higher mode effect.

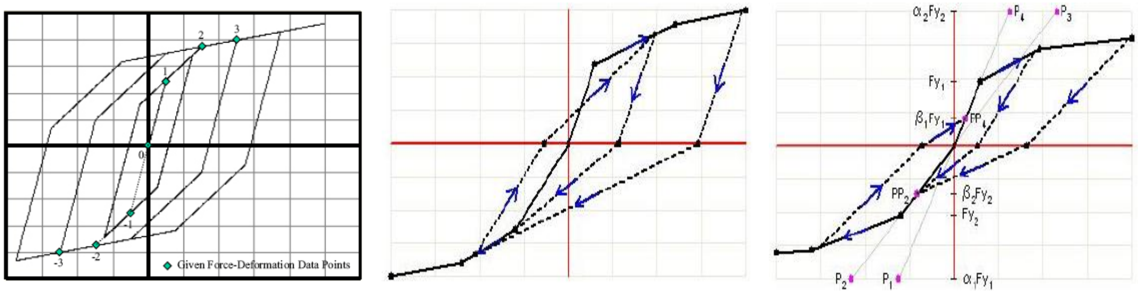


(a) 4.5 meters column height | (b) 6.3 meters column height | (c) 15.0 meters column height

Figure 8. Lateral capacity of three studied bridges in transverse direction

HYSTERESIS MODELING

This study uses three built-in hysteresis models in SAP2000, i.e. kinematic model, pivot model, and takeda model as shown in Figs.9(a)-(c), respectively, to observe the effect of hysteresis modeling on the IDA curve of the bridges.



(a) Kinematic model (b) Takeda model (c) Pivot model

Figure 9. Three different studied hysteresis rules (CSI 2011)

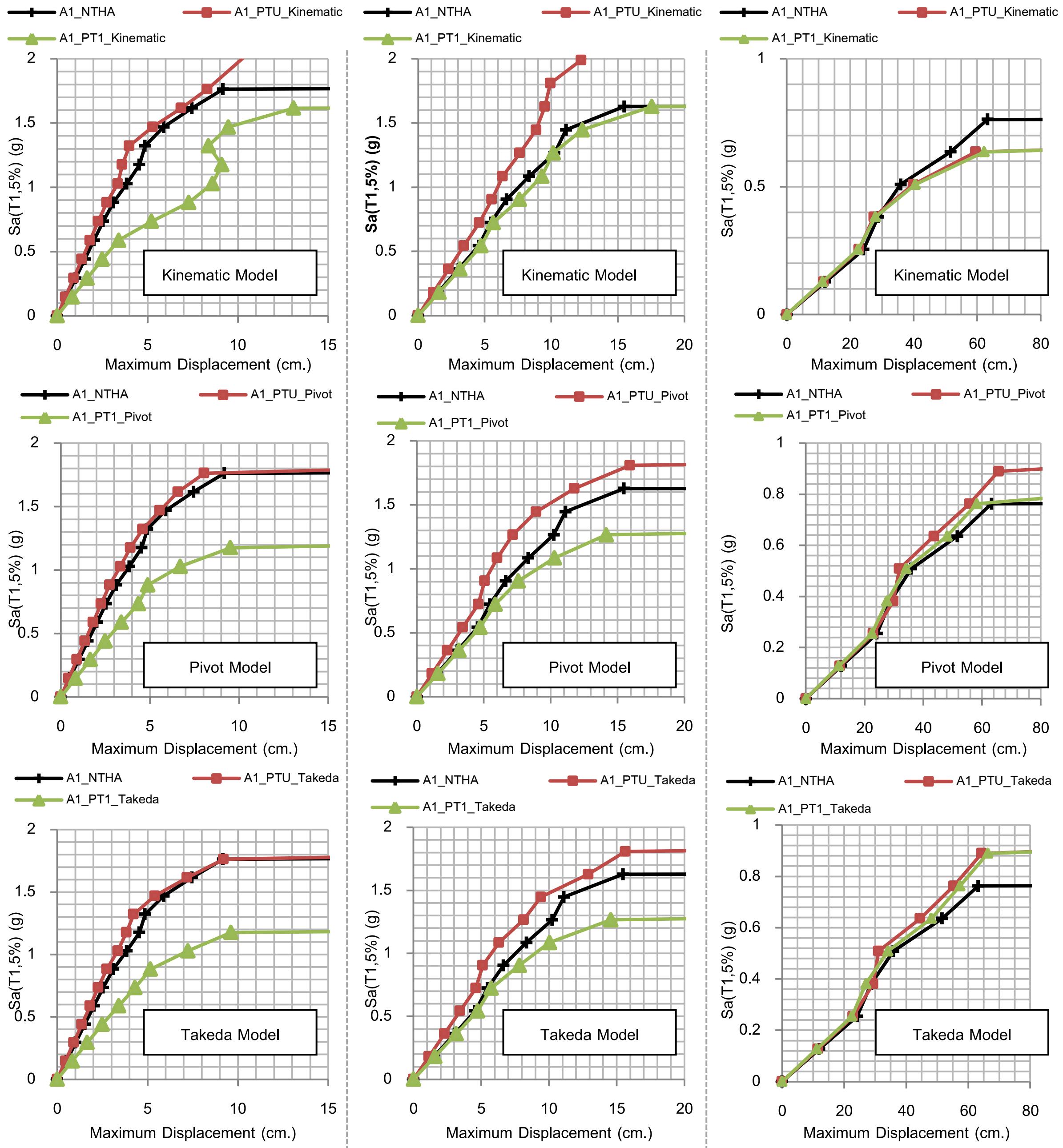
The kinematic model and takeda model are the built-in function which all required parameters were automatically generated by the program from the defined lateral capacity. However, SAP2000 requests user to input some parameters for modeling the pivot model. This study performed the cyclic nonlinear static analysis to evaluate these parameters required for modeling the pivot model.

EFFECT OF HYSTERESIS MODELING ON THE IDA CURVE

To investigate the effect of hysteresis modeling on the IDA curves of the studied bridges generated by ESDOF,the IDA curves generated by ESDOF with three different hysteresis models were compared with the IDA curves of MDOF. The results were shown in Fig.10(a)-(c) for studied bridges with 4.5 meter column, 6.3 meter column, and 15 meter column, respectively.

Fig.10 shows that the pivot hysteresis model and takeda hysteresis model result in similar IDA curves while the ESDOF with the kinematic hysteresis model do not agree well for IDA curves with NTHA of MDOF in some cases.

The pivot model leads to slightly more accurate hysteresis than takeda model because the required input parameters to generate the hysteresis were extracted from the results of the cyclic pushover analysis of MDOF of the studied bridges while the necessary parameters for modelling hysteresis by takeda model are automatically generated by the program.



(a) 4.5 meter column height (b) 6.3 meter column height (c) 15 meter column height

Figure 10. Comparison of IDA curve obtained from IDA by ESDOF with three various hysteresis models and NTHA of three studied bridges under A1 artificial ground motion

CONCLUSIONS

The effect of three different hysteresis models, such as kinematic model, takeda model, and pivot model, on the seismic behavior of the single column reinforced concrete bridges were investigated in this paper. The results lead to conclude that the suitable hysteresis model should be carefully selected when evaluates the seismic behavior of the structures like studied bridges by the concept of ESDOF.

The ESDOF with the kinematic hysteresis model do not agree well for IDA curves with NTHA of MDOF in some cases.

The ESDOF with the pivot hysteresis model shows the most accurate IDA curves with NTHA of MDOF in this study. However, the required input parameters to model the hysteresis must be extracted from the results of cyclic nonlinear static analysis.

The ESDOF with the takeda hysteresis model shows slightly less accurate IDA curves than the pivot hysteresis model compared with NTHA of MDOF. However, the necessary parameters for modelling hysteresis by takeda model are automatically generated by the program. Therefore, the takeda model will be the most practical hysteresis model.

ACKNOWLEDGMENTS

This work was supported by the National Research University Project of Thailand Office of Higher Education Commission and was also partially supported by the Thailand Research Fund (TRF) under the Royal Golden Jubilee Ph.D. Program contract number PHD/0296/2549.

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