Comparisons of Current Seismic Assessment Methods for Non-Seismic Designed Reinforced Concrete Bridges

P. Chomchuen
Mahanakorn University of Technology, Thailand

V. Boonyapinyo
Thammasat University, Rangsit Campus, Thailand

SUMMARY:
This study uses various methods, i.e., Capacity Spectrum Method, Inelastic Demand Diagram, Yield Point Spectra, and Equivalent Single Degree of Freedom with cyclic pushover analysis in evaluating the seismic response of low-, medium-, and high-rise single leg column bridges to investigate the accuracy of each method compared to the Nonlinear Time History Analysis (NTHA). The results show that all studied bridge is within elastic under the simulated earthquake ground motion generated corresponding to the design spectrum for Bangkok. The responses obtained from all assessment methods are conservative compared to NTHA. To study the efficiency of all assessment methods in evaluating the inelastic seismic responses of the bridge, the considered earthquake ground motion is scaled up to 5 time design spectrum for Bangkok. The results show that all assessment methods can be used for evaluating the inelastic seismic responses of the bridge, but ESDOF is the most accuracy compared to NTHA.

Keywords: Seismic Assessment, Nonlinear Static, Cyclic Nonlinear Static, Reinforced Concrete Bridge

1. INTRODUCTION

Bangkok, the capital of Thailand, is at moderate risk of distant earthquake due to the ability of soft soil to amplify ground motion about 3-4 times. In addition, before the enforcement of seismic load for building in the Ministerial Law in 2007, many existing reinforced concrete buildings and bridges in Bangkok may have been designed without consideration for seismic loading. In 2005, Federal Highway Administration (FHWA, 2006) has published the seismic retrofitting manual for highway structures. The six evaluation methods are described in this manual, which are all based, to varying degrees, on capacity-demand principles. As same as other seismic standards, the Nonlinear Time History Analysis (NTHA) is the most reliable method, but was suggested to use for irregular complex or major important bridges only because it is time consumed method. More practical method also suggested in this manual such as Capacity Demand Method (CDM). Capacity of the structures obtained from Nonlinear Static Analysis (NSA) is compared to the demand from considered earthquakes to evaluate the seismic performance of the structures. The several capacity demand based method for evaluating the seismic responses of the structures, especially for buildings, have been proposed by several standards and researchers, e.g., Capacity Spectrum Method (CSM) (ATC-40, 1996), Inelastic Demand Diagram Method (IDDM) (Chopra and Goel, 1999), N2 Methos (N2) (Fajfar, 1999), and Yield Point Spectra (YPS) (Aschheim, 1999).

The Department of Public Works and Town & Country Planning (DPT) has announced the seismic resistance design standard in 2009 (DPT, 2009). In this standard, Bangkok is a one region which will be affected by the earthquake. Therefore, it is not only the new structures but also the existing importance structures should be evaluated the seismic responses. This paper studies the application of four capacity demand based methods such as CSM, IDDM, N2, and YPS in evaluating the seismic response of the typical configuration of the structure of oldest expressway in Bangkok, Thailand under the design spectrum specified in the seismic resistance design standard of the Department of Public

Works and Town & Country Planning. This study also investigates the efficiency of the simple seismic evaluation method such as equivalent single-degree-of-freedom in evaluating the seismic responses of the studied bridges when its hysteresis behavior was modeled same as the full multi-degree of freedom structure obtained from cyclic pushover. This method called Equivalent Single Degree of Freedom with Cyclic Pushover (ESDOF).

2. CASE STUDY OF TYPICAL CONFIGURATION REINFORCED CONCRETE BRIDGES

2.1. Studied Bridges Configurations

The regular reinforced concrete bridges which used in the part of the expressway phase 1 since 1976 in Bangkok, Thailand, as show in Fig. 1, were chosen to be the case studies in this study. Three different bridge's column heights, i.e., 4.5 m., 6.3 m., and 15.0 m. as shown in Fig. 2 with 25 m. span length, 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.

Figure 1. Typical configuration of oldest expressway in Bangkok, Thailand

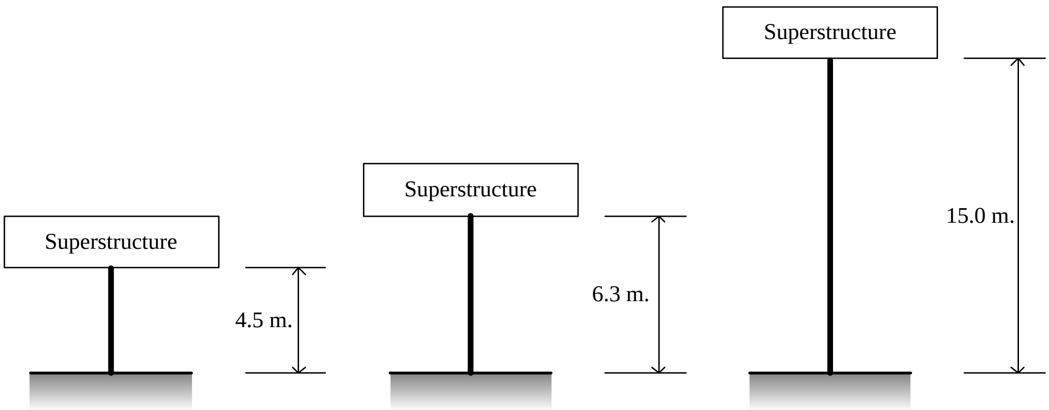


Figure 2. The case study of three different bridge's column heights

Superstructure of the studied bridges is the 18 cm. thickness reinforced concrete slab placed on the top of five pre-stressed concrete I-girders. Substructure of the studied bridges is the octagon reinforced concrete column with 1.33 m. thickness top slab. The cross-section of the column is 1.60 x 1.60 m as shown in Fig. 3. The connecting system between substructure and superstructure is bearing pads.

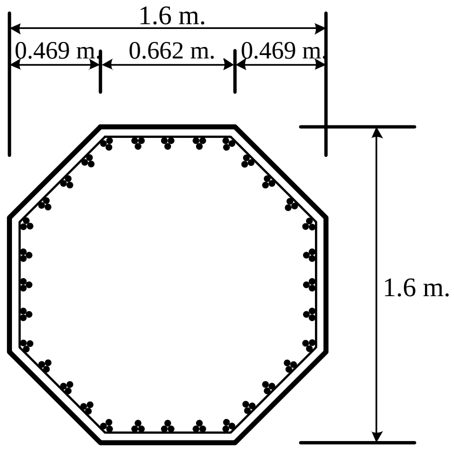


Figure 3. Column's cross-section of studied bridges

2.2. Analytical Model of Studied bridges

The analytical model of studied bridges is 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 beam-column element. Torsional mass, which affect to the dynamic properties of the bridges especially in transverse direction, is also calculated and defined to the nodes of elements.

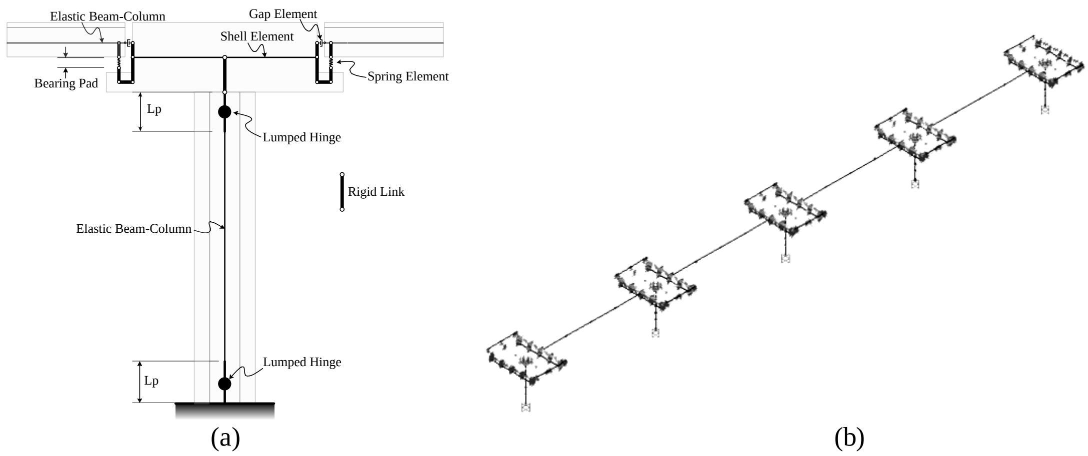


Figure 4. Analytical model of studied bridges: (a) Detail of each component modeling, and (b) Analytical Model

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. The inelastic behavior of the plastic length member is lumped to a point at the center of element as shown in Figure 4. 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. Top of the column is rigidly connected to the 1.33 m. thickness cast-in-place reinforced concrete slab as shown in Figure 6. It is modeled by elastic shell element. Mass of 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 elastic six degree-of-freedom spring element. The stiffness of each degree of freedom is calculated by the beam theory (Yazdani, Eddy, and Cai, 2000). The boundary conditions at the bottom of the bridge's columns were assumed to be fixed supports.

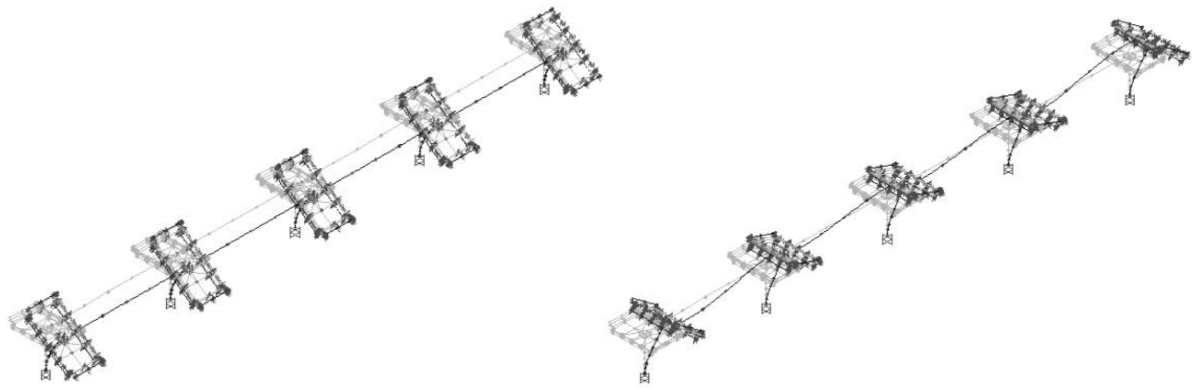
2.3. Dynamic Properties of the Studied Bridges

Dynamic properties of three different bridge's column heights are investigated in this study by modal analysis. The first transverse mode shape and first longitudinal mode shape of studied bridge 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 longitudinal direction is stiffer than transverse direction. The frequencies of the 6.3 m. column height were compared to the field test data. The frequency of the bridges in transverse direction and longitudinal direction from field test is 1.60-2.00 Hz and 2.00-2.80 Hz, respectively. It shows that the frequencies of the analytical model are in range of the field test data.

Table 1 Dynamic properties of three studied bridges

Column Height (m.)	Transverse Direction		Longitudinal Direction	
	Period (sec.)	Frequency (Hz)	Period (sec.)	Frequency (Hz)
4.5	0.450	2.224	0.272	3.679
6.3	0.610	1.640	0.358	2.796
15.0	1.746	0.573	0.980	1.020

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 effect to the flexural rigidity of the members and should be considered in the seismic evaluation. This study also considered the effect of the cracked section in seismic performance evaluating process.



(a) (b)

Figure 5. Mode shape of the studied bridges in (a) Transverse direction, and (b) Longitudinal direction

3. SEISMIC EVALUATION METHODS

Four capacity demand methods and one proposed ESDOFCP were used to evaluate the seismic responses of three bridges of single leg column, i.e., low-, medium, and high-rise bridges in this study. The results were compared to NTHA to investigate the accuracy of each method. The details of each method are described following.

3.1. Nonlinear Time History Analysis

The time history analysis is the method which generates the structural responses directly from dynamic equilibrium. The structural responses have been show in form of time history responses. In addition to consider the inelastic behavior of the structural system, the equilibrium equations governing the lateral displacements of the structural systems due to ground acceleration $\ddot{u}_g(t)$ are

$$m\ddot{u} + c\dot{u} + f_s(u, \text{sign}(\dot{u})) = -m\iota\ddot{u}_g(t) \tag{3.1}$$

Where m and c are the mass and damping matrices, ι is the influence vector, and $f_s(u, \text{sign}(\dot{u}))$ describes the inelastic lateral force deformation relation.

3.2. Capacity Spectrum Method

The capacity spectrum method used in this study is the procedure B of ATC-40 (ATC-40, 1996). The evaluation of seismic performance of structures is necessary to determine the ductility and total viscous damping. Typically, the capacity curve will intersect the demand curves corresponding to several viscous damping ratios. Each point on the capacity curve can be associated with an equivalent viscous damping ratio and natural period. The point at which the capacity curve intersects a demand curve associated with the same viscous damping ratio is the performance point which defines the spectral displacement demand.

3.3. Inelastic Demand Diagram Method

The inelastic response spectrums for elasto-plastic SDOF systems which different ductility factors (μ) were generated and used for the evaluation. Chopra and Goel (1999) presented the procedure for evaluating seismic performance by graphical method with iteration. The capacity spectrum presented in from of the relationship between spectral acceleration and spectral displacement can be obtained from the pushover analysis as shown in Eqn. 3.2 and 3.3:

$$S_a = \frac{V}{\alpha_1 W} \tag{3.2}$$

$$S_d = \frac{\Delta_{roof}}{PF_1 \phi_{1roof}} \tag{3.3}$$

Where:
V = The base shear from pushover analysis
W = The building dead weight
α_1 = The modal mass coefficient for the first natural mode
Δ_{roof} = The lateral roof displacement
PF_1 = Modal participation factor for the first natural mode
ϕ_{1roof} = The amplitude of the first natural mode of the building
S_d and S_a = Spectral displacement and spectral acceleration estimated by the inelastic static response

When both capacity spectrum and demand diagrams are plotted in the Acceleration- Displacement format, the yielding branch of the capacity diagram intersects the demand curves for several ductility factor (μ) values. The intersection between capacity and demand spectrum having the same ductility factor is the performance point. The system ductility is estimated by the ratio of inelastic maximum displacement at the performance point to yield displacement in capacity spectrum.

3.4. N2 Method

Fajfar (1999) presented the intersection of the radial line corresponding to the elastic period of the idealized bilinear system, T^* with the elastic demand spectrum, S_{ae} defines the acceleration demand required for elastic behavior and the corresponding elastic displacement demand. The yield acceleration, S_{ay} represents both the acceleration demand and the capacity of inelastic system. The reduction factor, R_μ can be determined as the ratio between the accelerations corresponding to the elastic and inelastic system:

$$R_\mu = \frac{S_{ae}(T^*)}{S_{ay}} \tag{3.4}$$

If the elastic period, T^*, is larger than or equal to Tc; the transition period where the constant acceleration segment of the response spectrum passes to the constant velocity segment of the spectrum, the inelastic displacement demand, S_d is equal to the elastic displacement demand S_{de}. The ductility demand, define as $\mu = S_d/D_y^*$ is equal to R_μ:

$$S_d = S_{de}(T^*) \qquad T^* \geq T_c \tag{3.5}$$

$$\mu = R_\mu \tag{3.6}$$

If the elastic period, T^*, is smaller than Tc, the ductility demand can be calculated from Eqn. 3.7:

$$\mu = (R_\mu - 1)\frac{T_c}{T^*} + 1 \qquad T^* < T_c \tag{3.7}$$

The displacement demand can be determined:

$$S_d = \mu D_y^* \tag{3.8}$$

The intersection of the radial line corresponding to the elastic stiffness of the idealized bilinear system and the elastic demand spectrum defines the strength required for elastic behavior and the corresponding elastic displacement demand.

3.5. Yield Point Spectra

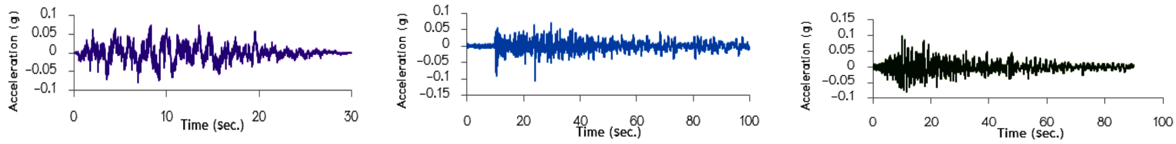
The YPS (Aschheim, 1999) has been proposed as a simple alternative to capacity spectrum method. The seismic response of the structure can be estimated by the product of yield ductility ratio and the yield displacement. The yield ductility ratio is the ductility ratio of the ESDOF system which gives the inelastic demand intersects with the capacity spectrum at the yield point.

3.6. Equivalent Single-Degree-of-Freedom with Cyclic Pushover

The seismic response of the Multi-Degree-of-Freedom (MDOF) system can be simulated by the Equivalent Single-of-Freedom (ESDOF) system at the concerning point (Monitoring point). The inelastic lateral force displacement relationship of the MDOF and its hysteresis behavior at the concerning point can be obtained by the cyclic pushover analysis. The nonlinear time history analysis of ESDOF is performed to evaluate the seismic response of the concerning point.

4. CONSIDERED GROUND MOTIONS

FHWA(2006) suggests that the maximum response for the three ground motions should be used for evaluating performance. This study use three artificial ground motions, Fig. 6, generated corresponding to the design spectrum for Bangkok area specified in seismic resistance design standard of Thailand (DPT, 2009) in evaluating seismic performance of the studied bridges. The design spectrum for Bangkok area is shown in Fig. 7.



(a) (b) (c)

Figure 6. Artificial ground motions generated corresponding with the design spectrum for inner area of Bangkok: (a) Simulated, (b) CABAJA, and (c) GULFCA

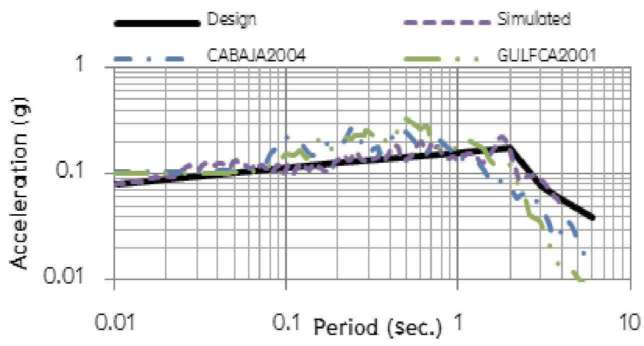


Figure 7. Comparison of spectrums of artificial ground motions and design spectrum

The three artificial ground motions, which use in this study, are shown in Fig. 6. Details of each ground motion are summarized in Table 2. The response spectrums of the artificial ground motions were generated and compared to the design spectrum as shown in Fig. 7.

Table 2 Details of the considered artificial ground motions

Ground Motion	Event	Year	Station	Magnitude	Distance (km.)	Simulation Method
Simulated	-	-	-	-	-	SIMQKE
CABAJA	CA/BAJA border area	2002	Calipatria Fire Station	5.31	89.2	PEER Online Database
GULFCA	Gulf of California	2001	Seeley School	5.7	107.8	PEER Online Database

Table 3 Seismic responses of three studied bridges evaluated by NTHA under 3 artificial ground motions corresponding with the design spectrum

Column Height (m.)	Ground motion	Transverse Direction		Longitudinal Direction	
		Displ. (m.)	Base Shear (kg.)	Displ. (m.)	Base Shear (kg.)
4.5	Simulated	0.004056	177,100.0	0.001204	205,700.0
	CABAJA	0.006042	265,400.0	0.002015	316,300.0
	GULFCA	0.00708	306,200.0	0.002442	391,200.0
6.3	Simulated	0.01051	216,000.0	0.003781	277,800.0
	CABAJA	0.01162	217,600.0	0.005516	365,800.0
	GULFCA	0.01178	237,500.0	0.005305	352,300.0
15.0	Simulated	0.07846	174,800.0	0.02392	188,100.0
	CABAJA	0.04392	90,720.0	0.01964	150,300.0
	GULFCA	0.05471	119,200.0	0.02981	228,400.0

5. SEISMIC RESPONSES OF STUDIED BRIDGES

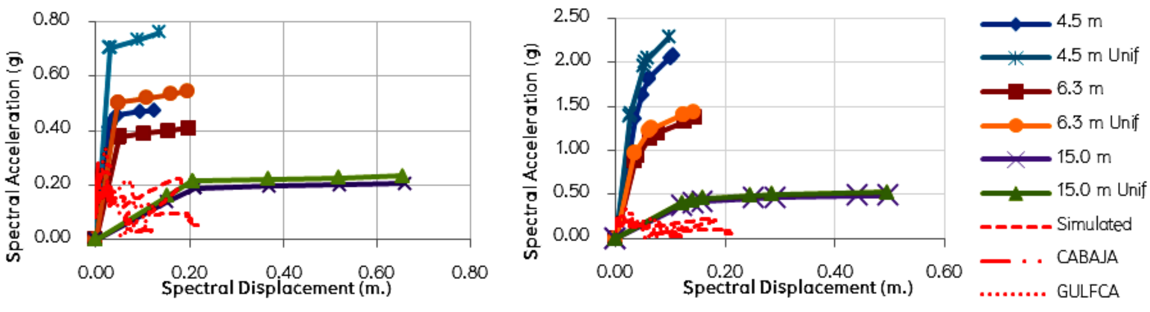
5.1. Nonlinear Time History Analysis

The NTHA is used to evaluate the seismic responses of the studied bridges in this study. The results of

NTHA were shown in from of the maximum displacement and maximum base shear of the studied bridges under each considered ground motion and were summarized in Table 3.

5.2. Capacity Demand Method

The combination of the capacity diagram of the studied bridges, from 1st mode load pattern and uniform load pattern, and elastic seismic demand diagrams corresponding to the design spectrum for inner area of Bangkok, as shown in Fig. 8, shows that the responses of the all studied bridges are in the elastic range under all considered ground motions. Then, the four capacity demand based methods, i.e., CSM, IDDM, N2, and YPS, give the same seismic responses which are the intersection between the capacity diagram and elastic demand diagram.

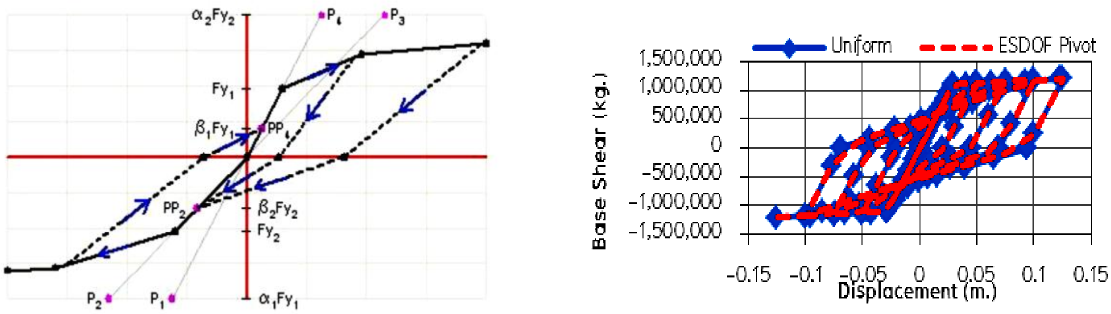


(a) (b)

Figure 8. Comparison among capacity diagram of three bridges and elastic demand diagram of the design spectrum (a) Transverse direction and (b) Longitudinal direction

5.3. Equivalent Single-Degree-of-Freedom with Cyclic Pushover

The hysteretic behavior of MDOF is modeled by the pivot hysteretic model and defined to the spring of the ESDOF in this proposed method. The parameters which use to model the pivot hysteresis are shown in Fig. 9(a). The comparison of the hysteretic behavior of MDOF and ESDOF with pivot hysteresis of the studied bridge with 6.3 m. column height is shown in Fig. 9(b).



(a) (b)

Figure 9. Hysteretic behavior of the 6.3 m. column height bridge (a) Pivot hysteresis model, (b) Hysteretic behavior of ESDOF compared with MDOF by uniform load pattern

5.4. Efficiency of Seismic Evaluation Methods

To investigate the efficiency of each seismic evaluation method, the displacements evaluated from each method (Di) were compared to those from nonlinear time history analysis (Dtime). The results are shown in Fig. 10. It is shown that: (1) the ESDOF shows the most accuracy displacements in both transverse and longitudinal direction; (2) four considered capacity demand based methods give the same seismic responses when the demand diagram intersects with capacity diagram in the elastic region; (3) the capacity demand method give the larger displacements compared with NTHA especially when uses the lateral capacity obtained from 1st mode load pattern; (4) the evaluated longitudinal displacements slightly different when lateral capacity obtained from both 1st mode load pattern and uniform load pattern were used.

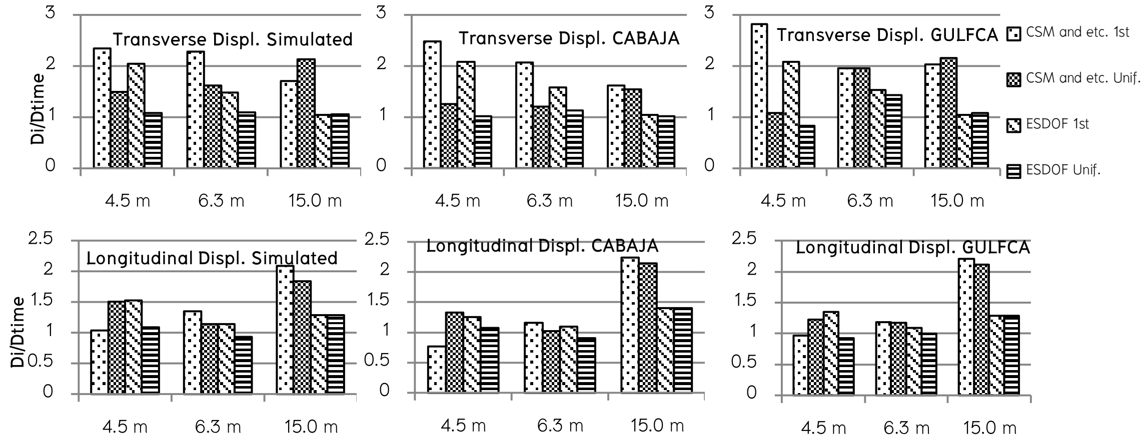


Figure 10. Comparison of the maximum transverse and longitudinal displacement of three studied bridges among various methods under 3 artificial ground motions corresponding with design spectrum

To investigate the accuracy of each evaluation method in evaluating the inelastic seismic responses of the studied bridges, the simulated ground motion is scaled up to five time of the design spectrum. The maximum displacement of studied bridges evaluated from each method were compared to the NTHA and shown in Fig. 11. The lateral capacities obtained from uniform load pattern were used in this evaluation.

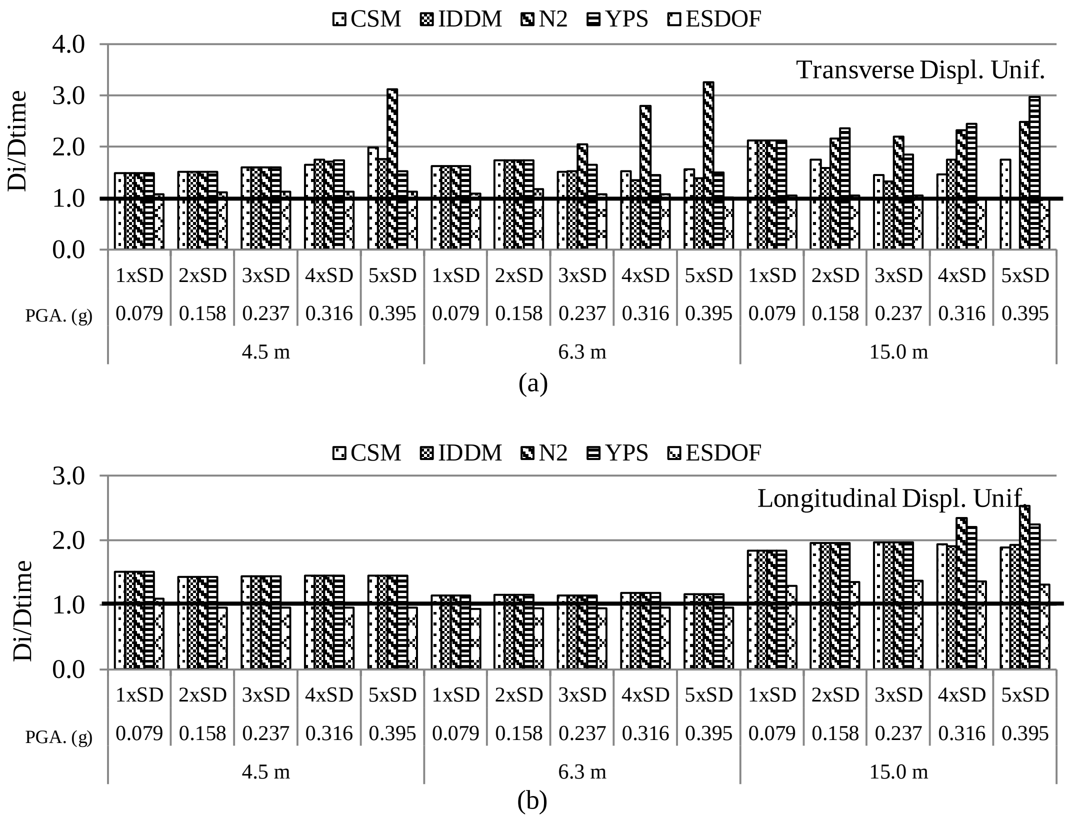


Figure 11. Comparison of the maximum displacement of three studied bridges among various methods under 1-5 times scaled ground motion in: (a) Transverse direction, and (b) Longitudinal direction

After the structures yielded, each seismic evaluation method has its own technique in evaluating the inelastic seismic responses. Fig. 11 shows that the studied bridges yield when the intensity of considered ground motion was scaled up. It also shows that: (1) the displacements evaluated by N2 show an over conservative compared to NTHA; (2) the YPS also shows some over conservative displacements after the structures yielded, especially in high-rise bridge; (3) the IDDM shows slightly more accuracy than CSM and moderately more accuracy than N2 and YPS, but the transverse displacement of high-rise bridge under the five times intensity of ground motion cannot be obtained by this method; (4) the ESDOF still be the most accuracy method in evaluating the displacements of the studied bridges in both transverse and longitudinal direction even if the intensity of considered ground motion was scaled up.

6. CONCLUSION

This study investigates the efficiency of several seismic evaluation methods for evaluating the seismic responses of the typical single leg column reinforced concrete bridges in Bangkok. The results lead to the following conclusions.

Even if each capacity demand based method has its own technique and limitations in evaluating the seismic responses of the structures, the actual hysteretic behavior of the structures is not exactly considered in all methods and may lead to the over conservative responses. This study shows that the simple nonlinear time history analysis of SDOF with hysteretic behavior obtained from cyclic pushover gives more accuracy responses than capacity demand based methods compared with NTHA. It means that the hysteretic behavior modeling of SDOF strongly influence on the evaluated responses. If the hysteretic behavior of the MDOF can be accurately captured and modeled to SDOF, the simple nonlinear time history analysis of SDOF can lead to more accuracy evaluated responses.

This study also shows that the flexibility of bridge's column directly influence to the dynamic properties of this kind of bridge. The short column bridge stiffens than the tall one. The evaluated responses of the studied bridges show that the bridge with shorter column resists the larger base shear force. Even if this is not always true because resonant responses depended on both character of structures and character of considered ground motions but it should seriously check not only flexural failure but also shear failure in the short bridge's columns.

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