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Modified generalized pushover analysis for estimating longitudinal seismic demands of bridges with elevated pile foundation systems

Abstract

In longitudinal multi-mode pushover analysis of bridges with elevated pile foundation systems, the inelastic contributions of the second mode cannot be neglected. Generalized pushover analysis cannot be applied directly in this condition. A modified generalized pushover procedure is developed for estimating seismic demands of bridges with elevated pile foundation systems. Modified generalized pushover procedure, modal pushover analysis and incremental dynamic analysis of a bridge with elevated pile foundation systems are conducted. The results show that the modified generalized pushover procedure can provide reasonable estimations of moments and predict more accurate plastic hinge rotations compared with modal pushover analysis.

Keywords

Pushover analysis; Modified GPA; bridges with elevated pile foundation system; Contributions of higher modes; MPA.

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1 INTRODUCTION

For seismic evaluation of structures, nonlinear time-history analysis (NL-THA) could be used. However, NL-THA is time-consuming. As an efficient and economic method for seismic performance evaluation of structures, pushover analysis is favored by current structural engineers (Akhaveissy, 2012; Forcael, 2014).

Pushover analysis in many cases will provide much more relevant information than an elastic static or even dynamic analysis, while in some other cases it will provide misleading results (Krawinkler, 1996). Traditional pushover analysis procedures are conducted with common lateral force patterns, such as first mode, inverted triangular, uniform, etc (Applied Technology Council, 2005). The procedures are applicable for regular structures which vibrate primarily in the fundamental mode. However, they are not suitable for irregular structures, in which the contributions of higher modes are significant (Krawinkler and Seneviratna, 1998).

Multi-mode pushover analysis procedures have been proposed to consider contributions of higher modes. To include the influence of higher modes, Chopra and Goel proposed modal pushover analysis procedure (MPA) (Chopra and Goel, 2002). In the procedure, a pushover analysis is conducted for each mode separately, and then total seismic responses are computed by combining the responses due to each modal load. MPA produces the same results as response spectrum analysis (RSA) when a structure vibrates in linear elastic range. Even when a structure responds well into inelastic range, MPA is capable of providing good estimates of displacement demands and identifying locations of plastic hinges. Moreover, implementation of MPA is simple. To save computing effort, Chopra *et al* improved MPA to develop modified MPA(MMPA) (Chopra, 2004). In the MMPA procedure, response contributions of higher modes are computed by assuming a structure being elastic. SRSS combination rule is adopted by both MPA and MMPA to combine modal responses. There are several shortcomings of combining inelastic modal responses by SRSS, especially when internal forces are calculated (Papanikolaou and Elnashai, 2005).

Adaptive pushover procedures have been proposed to capture the changing properties of structures in pushover analysis (Antoniou and Pinho, 2004; Elnashai, 2001; Gupta and Kunnath, 2000). When a structure vibrates into inelastic range, its dynamic properties will change with time. In an adaptive pushover procedure, the lateral force pattern is updated according to the time-variant stiffness distribution of the structure. Antoniou and Pinho proposed a displacement-based adaptive pushover procedure (Antoniou and Pinho, 2004). In the procedure, a set of updated lateral displacements, rather than force, are imposed on the structure. Pushover procedures with adaptive lateral force patterns can provide more accurate dynamic response evaluations of structures. However, they are conceptually complicate and computationally demanding for routine application in structural engineering practice.

Based on MPA, Kalkan and Kunnath proposed an adaptive modal combination procedure (AMC) (Kalkan and Kunnath, 2006). AMC is the same as MPA with the exception being that the modes are updated at each step of modal pushover analysis. AMC can reasonably predict critical demand parameters, such as roof displacement and inter-story drifts, for both far-fault and near-fault seismic records. However, the AMC procedure is quite complicated, and SRSS combination rule is still adopted.

Benchmark values to evaluate accuracy of pushover results are usually provided by incremental dynamic analysis (IDA) (Vamvatsikos and Cornell, 2002). To obtain more accurate predictions through pushover analysis, it is necessary to analyze the relationships between pushover analysis and IDA. Both pushover analysis and IDA use principles of equilibrium and compatibility with the difference being that IDA equilibrium includes damping and inertia effects. The variable is current level of displacement or force in pushover analysis, while the variable is time in dynamic analysis (Papanikolaou and Elnashai, 2005). Maximum values of various response parameters generally happen at different instants in NL-THA. Hence, values of various response parameters are obtained at various instants in dynamic analysis. Consequently, they are in different equilibriums. However, values of response parameters of a structure are all obtained from a single equilibrium in a single-run pushover analysis. It is impossible to replicate all the inelastic response parameters of a dynamic analysis with a single-run pushover analysis.

To compute the inelastic response parameters with several different equilibriums (Sucuoglu and Gunay, 2011), Sucuoğlu and Günay proposed generalized pushover analysis (GPA) procedure. Maximum value of a response parameter can be expressed by modal expansion in time domain analysis. Meanwhile, it can also be expressed by quadratic combination (SRSS) of the related spectral modal responses. Constructing the equilibrium between the two expressions for a response parameter, a pair of GPA parameters (generalized force vector and target deformation demand) can be computed from RSA results of the structure. A generalized force vector corresponds to the inertial force vector of the structure at the instant when a specific response parameter reaches its maximum value in dynamic analysis. A target deformation demand corresponds to the maximum deformation in dynamic analysis of the structure. Generalized force vectors are applied to the structure separately in an incremental form until target deformation demands are attained. Maximum value of any response parameter is obtained from the envelope of the GPA results. GPA procedure is successful in estimating maximum deformations and member forces with reference to IDA results.

All the pushover procedures aforementioned are proposed for buildings. Paraskeva and Kappos have extended MPA for seismic assessment of bridges (Paraskeva, 2006). The adaption included "the selection of the appropriate control point, the way a pushover curve is bilinearized before being transformed into a capacity curve, the use of the capacity spectrum for defining the earthquake demand for each mode and then combining modal responses, and the number of modes that should be considered in the case of bridges". Biao Wei et al. have applied equal displacement rule to continuous bridges with long periods (Wei, 2014). Their work is mainly on applying MPA in the transverse direction of a curved bridge.

Elevated pile foundation systems (Figure 1) are widely used for deep water bridges. The system includes a group of long piles, a high-rise cap and a pier. When subjected to longitudinal excitation, superstructure of a straight bridge can be simplified to a lumped mass and the whole bridge can be simplified as a model shown in Figure 1. In the paper, all static and dynamic analyses are applied to the simplified model of the whole bridge.

Mass and stiffness distribution of the simplified model vary greatly in the vertical direction. Besides the first mode, second mode of elevated pile foundation system is easy to enter inelastic range even when subjected to moderate earthquake. In such a case, systematic errors of target deformation demands will occur if the second mode is treated as linear elastic in GPA procedure. Therefore, it should be confirmed that second mode, as well as the first mode, should be treated as inelastic in GPA procedure of the simplified model. GPA procedure proposed by Sucuoğlu and Günay considered inelastic contributions of the first mode only and couldn't be directly applied to the simplified model. Uncoupled modal response history analysis (UMRHA) (Chopra, 2007) is a convenient method to compute the GPA parameters, when inelastic contributions of more than one mode needs to be included. Modified GPA is developed based on this idea.

Typically, first two or three modes will be enough (Chopra and Goel, 2002). Modified GPA is able to obtain the generalized force vectors with only the first two or three modes.

In this paper, modified GPA is introduced and verified through a simplified model of a bridge with elevated pile foundation systems. Principle and basic steps of the modified GPA are expressed at first. Modified GPA for the elevated pile foundation system is then verified. The necessity of including inelastic contributions of the second mode is illustrated. Modified GPA, MPA and IDA of the elevated pile foundation system are conducted with an ensemble of 9 ground motions. The response parameters predicted by modified GPA are compared with MPA results, as well as IDA results. Conclusions are presented accordingly at last.

2 MODIFIED GPA

GPA uses response spectrum analysis (RSA) results to determine the GPA parameters. Through RSA, GPA provides a smart way to capture the GPA parameters. Consequently, people don't have to find out the exact instants when response parameters reach their maximum values individually.

Modified GPA uses UMRHA method to compute generalized pushover parameters. Having the capability of showing values of a response parameter in every step in time domain analysis, it is convenient to find out the instant when a response parameter reaches its maximum value in UMRHA procedure. Once all the instants are known, the GPA parameters can be computed by combining contributions of the uncoupled modes.

Two main characteristics of modified GPA are: (1) The GPA parameters are directly determined through UMRHA; (2) Inelastic contributions of higher modes are convenient to be considered.

2.1 The UMRHA-method to Determine the GPA Parameters

The differential equations governing seismic response of a structure can be uncoupled to several equations of effective SDOF systems (Chopra, 2007):

$$\ddot{q}_n + 2\zeta_n\omega_n\dot{q}_n + \frac{F_{sn}}{M_n} = -\Gamma_n\ddot{u}_g(t) \tag{1}$$

Here, $L_n = \varphi_n^T m\iota$; $M_n = \varphi_n^T m\varphi_n$; ω_n is natural vibration frequency; ζ_n is damping ratio for the nth mode; $\Gamma_n = L_n / M_n$ is nth mode participation factor; φ_n is nth natural vibration mode of the structure; $F_{sn} = F_{sn}(q_n, sign\,\dot{q}_n) = \varphi_n^T f_s(u_n, sign\,\dot{u}_n)$ is resisting force.

The resisting force depends on all modal coordinates $q_n(t)$, implying coupling of modal coordinates because of yielding of the structure. Neglecting contribution of the other modes to the nth mode resisting force F_{sn}, F_{sn} now depends only on the nth-mode coordinate q_n.

Eq. 1 now can be transformed to:

$$\ddot{D}_n + 2\zeta_n\omega_n\dot{D}_n + \frac{F_{sn}}{L_n} = -\ddot{u}_g(t) \tag{2}$$

where $F_{sn} = F_{sn}(D_n, sign\,\dot{D}_n) = \varphi_n^T f_s(D_n, sign\,\dot{D}_n)$; $q_n = \Gamma_n D_n(t)$.

Eq. 2 may be interpreted as the governing equation for the nth-mode inelastic SDOF system. The F_{sn}/L_n - D_n relation is approximated by a bilinear curve, which can be derived from the nth-mode pushover analysis of a structure.

$D_n(t)$ and $A_n(t)$ can be obtained by solving Eq. 2.

$$u(t) = \sum_{n=1}^{N} u_n(t) = \sum_{n=1}^{N} \Gamma_n \varphi_n D_n(t) \tag{3}$$

$$f(t) = \sum_{n=1}^{N} f_n(t) = \sum_{n=1}^{N} \Gamma_n m \varphi_n A_n(t) \tag{4}$$

$$\Delta_j(t) = \sum_{n=1}^{N} \Delta_{j,n}(t) = \sum_{n=1}^{N} \Gamma_n D_n(t)(\varphi_{n,j} - \varphi_{n,j-1}) \tag{5}$$

The nonlinear response histories of floor displacements $u(t)$ and interstory drifts $\Delta_j(t)$ can be obtained from Eq. 3 and Eq. 5. The instants $t_{\max}$, when the target deformation demands reach their maximum values, can be obtained easily. Then, $D_n(t_{\max})$ and $A_n(t_{\max})$ can be obtained. Bring $A_n(t_{\max})$ into Eq. 4, corresponding generalized force vectors can be computed. Compared with GPA in literature (Sucuoglu and gunay, 2011), modified GPA is straightforward and easy to implement.

Main errors of the UMRHA-method arise from neglecting the coupling of equations between modal responses. When a structure is subjected to weak excitation $P_{eff,n}(t)$, the response is in the nth-mode only, and all the other modes make no contribution. When a structure is subjected to strong excitation $P_{eff,n}(t)$, the other modes start responding to $P_{eff,n}(t)$ after the instant when the structure first yields. However, their contributions to the response are small (Chopra and Goel, 2011).

2.2 Main steps of the modified GPA

Main steps of the modified GPA can be summarized as following:

1. Eigenvalue analysis. Natural frequencies ω_n, natural periods T_n, modal vectors φ_n and the modal participation factors Γ_n are determined.

2. Pushover analysis. For the nth-mode, pushover analysis with the lateral force vector of $s_n^* = m\varphi_n$ is conducted. The base-shear--roof-displacement (V_{bn} - u_{rn}) pushover curve is obtained.

3. Obtain F_{sn}/L_n - D_n relationship. Pushover curves are idealized as bilinear curves. The idealized pushover curves are converted to the F_{sn}/L_n - D_n relationships.

4. Compute response history. The deformation history $D_n(t)$ and pseudo-acceleration history $A_n(t)$ of the nth-mode inelastic SDOF system are computed with force-deformation relationships developed in step 3.

5. Determine target displacement demands and corresponding $t_{\max}$. Through the deformation histories computed in step 4, response histories of displacement demands are obtained with Eq. 3 or Eq. 5. Target displacement demands and corresponding $t_{\max}$ are directly captured.

6. Calculate generalized force vectors. Using the pseudo-accelerations $A_n(t_{\max})$ computed in step 5, generalized force vectors are determined with Eq. 4.

7. Once the GPA parameters are obtained, GPA can be conducted. Envelope of the member deformations and the internal forces are taken as the maximum seismic response values.

3 CASE STUDY

Modified GPA is verified through a simplified model of a bridge with elevated pile foundation systems, which has been used in the literature (Wancheng and Jun, 2008). The necessity of including inelastic contributions of the second mode is illustrated. Both modified GPA and MPA of the simplified model are conducted. To provide benchmark results, IDA of the simplified model is also conducted. The system is analyzed with 9 different ground motion components.

3.1 Simplified model of a bridge with elevated pile foundation systems

Simplified model of the elevated pile foundation system is shown in Figure 1. Superstructure is simplified as a mass and fixed to the top of pier. The pile cap is modeled as a mass and fixed to top of the piles and bottom of the pier. Equivalent cantilever pile model is adopted to represent the interaction between piles and soil (Chen, 1997). Piles are assumed to be embedded in ground with a predetermined depth. It should be noted that research on the interaction between piles and soil is still performed (Kim, 2011).

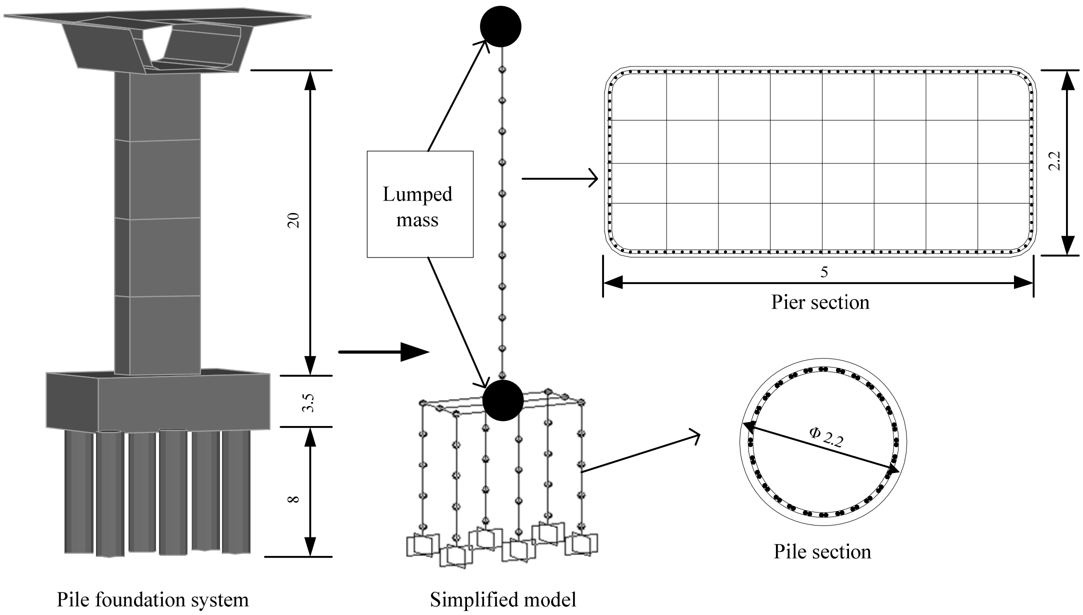


Figure 1: Elevated pile foundation system of bridges (m).

Sections of piles and pier are shown in Figure 1 too. C30 (piles and cap), C40 (pier) concrete and steel characteristic strengths are 20.1 MPa, 26.8 MPa and 335 MPa respectively. Simplified mass of the bridge superstructure and the pile cap are 3000 ton and 6000 ton, respectively. Effective length of piles, depth of pile cap and pier height are 8 m, 3.5 m and 20 m respectively. Distance from the simplified mass of superstructure to the top of the pier is 1.8 m. Distances between the center line of piles are 4.5 m in both longitudinal and transverse direction of the bridge.

Simplified model of the bridge is constructed and analyzed by the open source software OpenSees (Mazzoni, 2007). Piles and pier are simulated by the dispBeamColumn element, which is a distributed-plasticity, displacement-based beam-column element. Element length of piles and pier is settled to 2 m and each element includes 5 integration points since accurate determination of local response quantities requires a finer finite element mesh. Fiber sections with unconfined and confined concrete materials presented by Mander et al. are assigned to pier and piles. Plastic hinge rotations are integrated at integration points where curvature of the section exceeds elastic range (Usually at ends of pier and piles). Masses of superstructure and pile cap are fixed to structure elements by rigid link. P-Delta effects are not included. Figure 2 shows the elastic modal shape of the elevated pile foundation system. Damping ratio of 0.05 is assigned to the simplified model through periods of the first and the second mode 2.457 s, 0.512 s, respectively. Modified GPA, MPA and IDA are conducted in the longitudinal direction of the bridge. Contributions of the first three modes are considered in the pushover analyses.

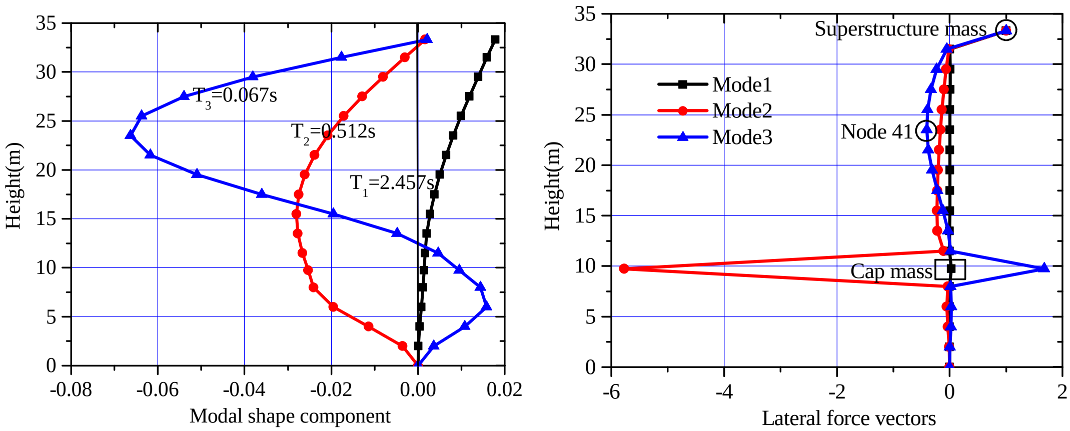


Figure 2: Elastic modal shapes and lateral force vectors of the elevated pile foundation system.

Pushover curves and their bilinear fits of the first 3 modes are shown in Figure 3. F_{sn}/L_n - D_n relationships of single degree of freedom(SDOF) systems corresponding to the first three modes are obtained from these bilinear fits.

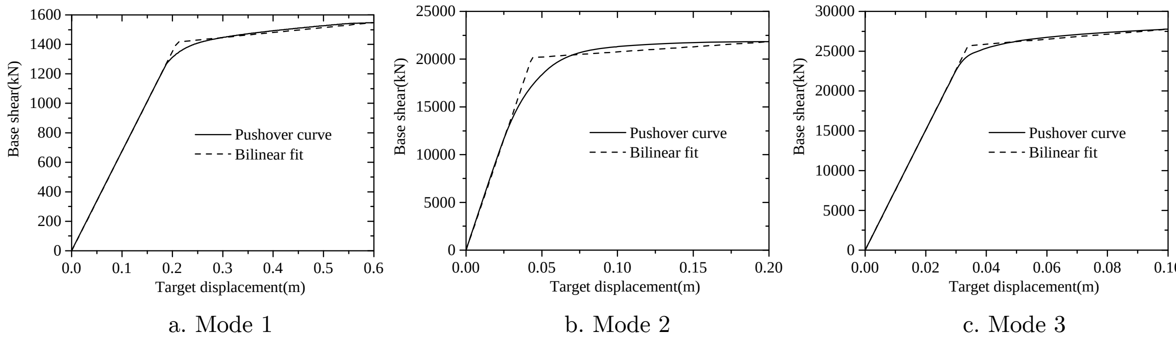


Figure 3: Pushover curves and bilinear fits of the first three modes.

Modal properties of the first 3 modes are shown in Table1. These properties can be used to compute GPA parameters in MGPA procedure.

	T_n	Δ_{Pier}	Δ_{Pile}	D_{roof}	Γ_n	Fsny	Fsno	Urny	Urno
Mode 1	2.457	0.017105	0.000768	0.017873	62.19	1415.8	1547.5	0.2096	0.6
Mode 2	0.512	-0.01391	0.012335	-0.00158	76.74	20165.2	21835.5	0.0434	0.2
Mode 3	0.067	-8.6E-05	-0.00261	-0.0027	-4.12	25693.9	27770.6	0.034	0.1

Table 1: Modal properties of the first 3 modes.

An ensemble of 9 earthquake ground motion records was selected for the case study, including both far-fault and near-fault ground motion records. Ensemble of the records is shown in Table 2. Spectral accelerations of the records are shown in Figure 4. In the records, the spectral acceleration achieves peak value at around the 2**nd** modal period (0.512 s). As a result, significant second mode effects are affected by the records. The ground motion records are downloaded from the PEER strong motion database and the COSMOS Virtual Data Center.

Short name	Year	Earthquake	Moment mag.	Mech.[a]	Recording station	Dist.[b] (km)	Site Class[c]	Comp.	PGA (g)	PGV (cm/s)
Far-fault ground motions										
Kern	1952	Kern county	7.5	TH/REV	Taft	36.2	D	111	0.18	17.50
Bigbear	1992	Big Bear	6.4	SS	Desert Hot Spr. (New Fire Stn.)	40.1	D	090	0.23	19.14
Moorpark	1994	Northridge	6.7	TH	Moorpark (Ventura Fire Stn.)	26.4	D	180	0.29	20.97
Near-fault ground motions										
Erzincan	1992	Erzincan	6.7	SS	Erzincan	2.0	C	EW	0.50	64.32
Cape	1992	Cape Mendocino	7.1	TH	Petrolia, General Store	15.9	C	090	0.66	90.16
Loma	1989	Loma Prieta	7.0	OB	Los Gatos Parent Center	3.5	C	000	0.56	94.81
Sylmar	1994	Northridge	6.7	TH	Sylmar Olive View Hospital	6.4	D	360	0.84	170.37
Kobe	1995	Kobe	6.9	SS	JMA	0.6	C	000	0.82	81.62
El	1940	El Centro	6.9	SS	117(UGUS)	12.2	D	180	0.35	33.45

[a]Faulting mechanism: TH-thrust; REV=reverse; SS=strike-slip; and OB=oblique.
[b]Closest distance to fault.
[c]NEHRP site classifications: [C for Vs (shear-wave velocity)=360-760 m/s], (D for Vs=180-360m/s).

Table 2: Ground Motion Ensemble.

3.2 Necessity of Including Inelastic Contributions of the Second Mode

For a simplified model of a bridge with elevated pile foundation systems, the second mode may also enter inelastic range even in moderate earthquake. If the second mode is treated as linear elastic, systematic error will occur in results of target displacement demands.

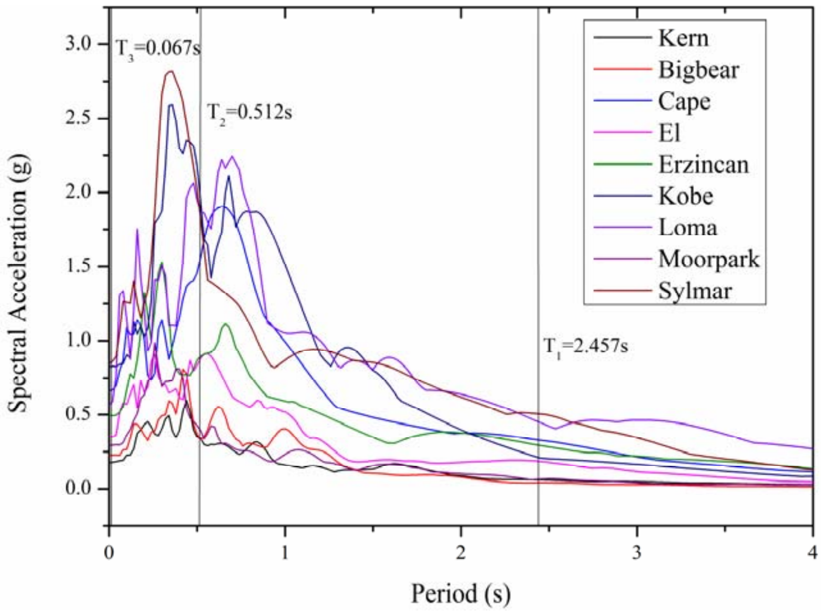


Figure 4: Acceleration spectra of the selected ground motion records.

Target interstory drift demands determined by GPA, Modified GPA and MPA are shown in Figure 5 with scaled accelerogrames (PGA range from $0.1g$ to $0.6g$). The second mode is treated as linear elastic in GPA, while it is treated as inelastic in modified GPA and MPA. Maximum values of target interstory drifts determined by IDA are also presented to show benchmark values.

GPA results for piles clearly deviate from IDA results under Cape, El, Erzincan, Kobe, Loma and Sylmar. It is revealed that second mode shouldn't be considered as elastic for the elevated pile foundation system. Otherwise, systematic errors will occur. Compared with MPA results, MGPA results is more close to IDA results under Cape, El and Sylmar.

3.3 Implementation of the Modified GPA Procedure

As an example, MPGA procedures of the simplified model subjected to El (0.6g) are shown as follows.

Modal properties and pushover curves of the simplified model have been shown in Figure 2 and Table 1. Each mode is simplified to an effective SDOF system. When these SDOF systems are subjected to El (0.6g), their deformation history $D_n(t)$ can be obtained by linear time-history analysis. Pier drift and piles drift computed by Eq. (5) are shown in Figure 6.

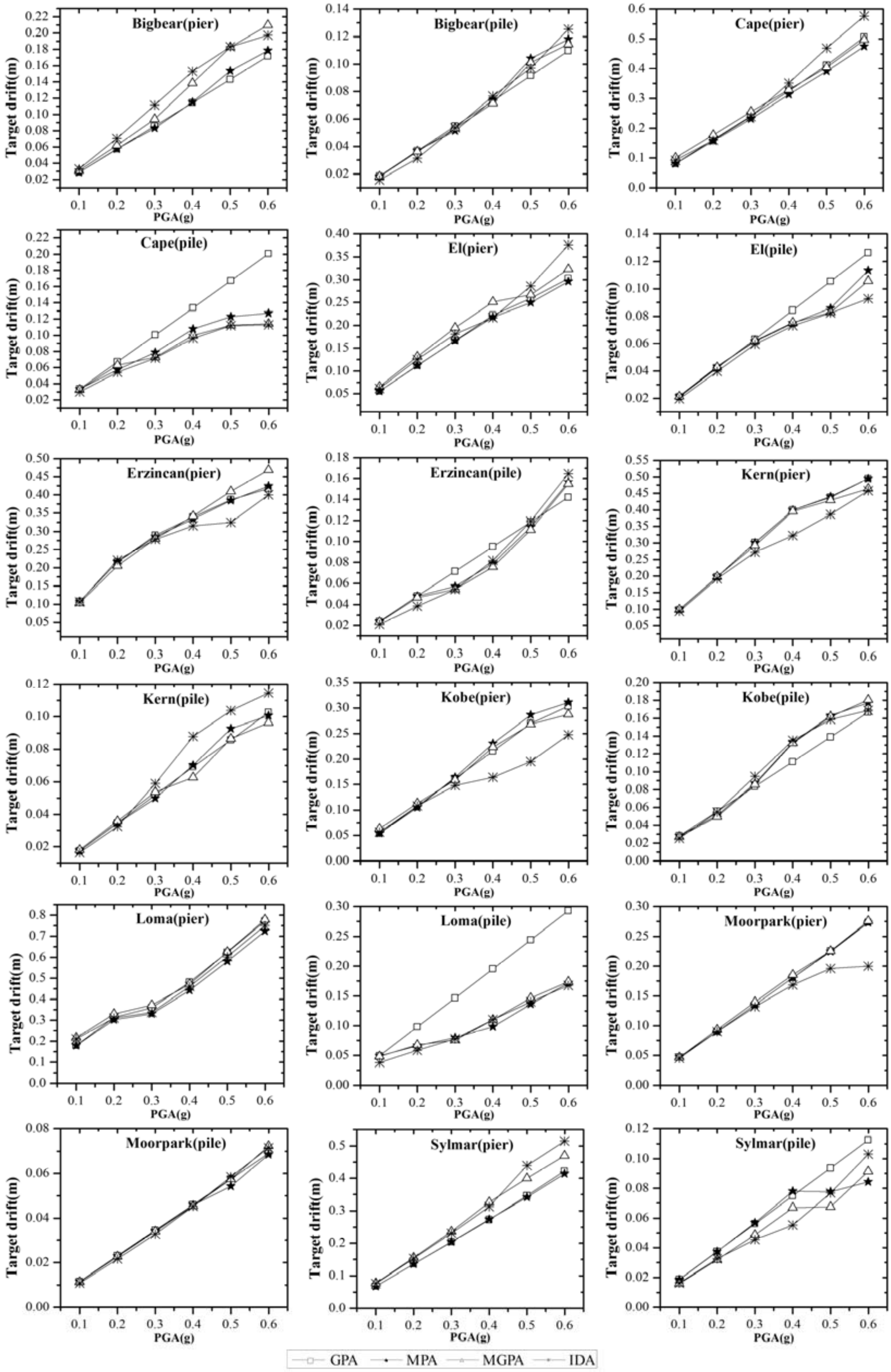
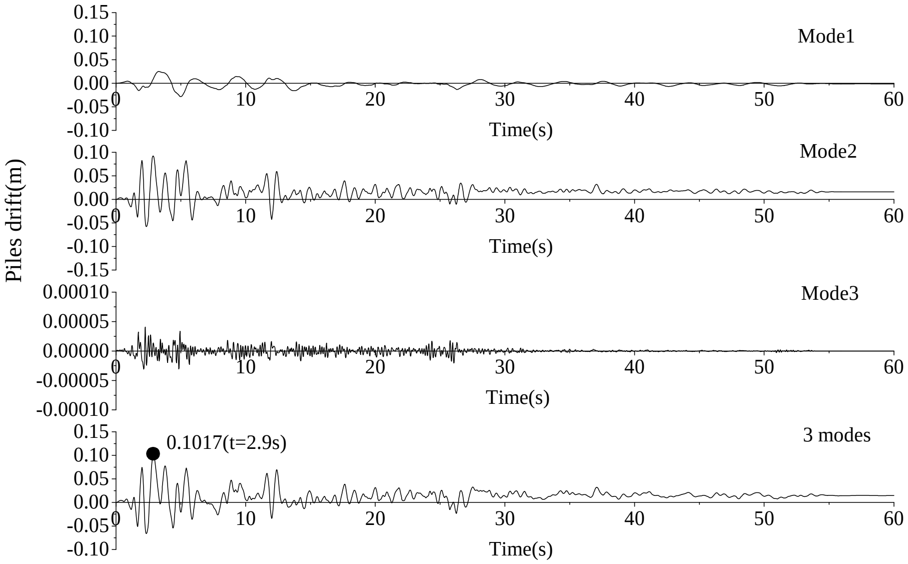
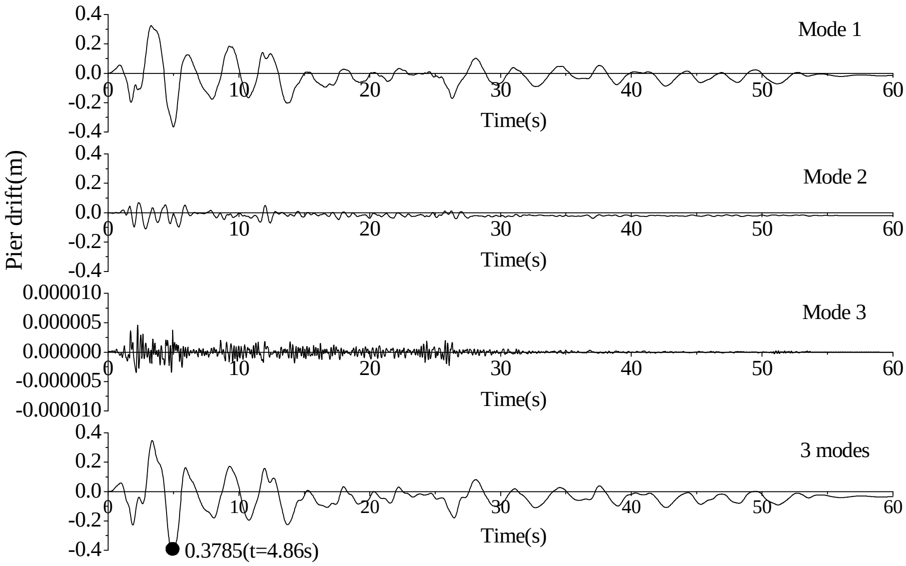


Figure 5: Target interstory drift demands.

It can be seen that t_{max} is 2.9 s for maximum piles drift and 4.86 s for maximum pier drift. $D_n(t)$ for these instants are shown in Table 3. It can also be seen that target interstory drift of piles and pier are 0.1017m and 0.3785m respectively.



a. Piles drift



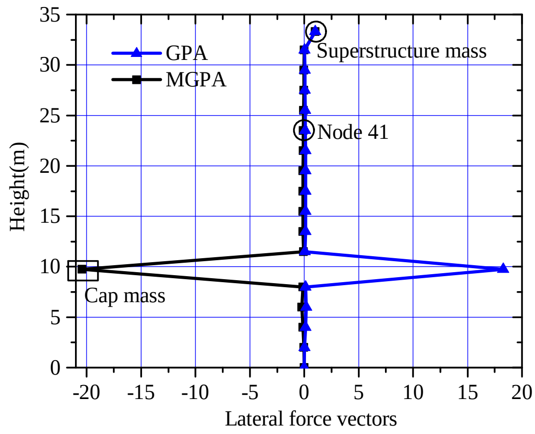
b. Pier drift

Figure 6: Pier and piles drift contributions.

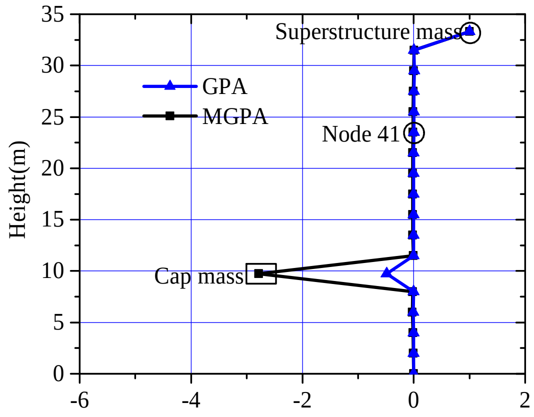
t_{max}	$D_1(t)$	$D_2(t)$	$D_3(t)$
2.9	0.11875	0.09999	0.00175
4.86	-0.28672	0.04182	-0.00430

Table 3: Ground Motion Ensemble.

General force vectors are computed from Eq. (4) and shown in Figure 7. General force vectors that computed from GPA are also shown in Figure 7. It turns out that they are consistent with each other.



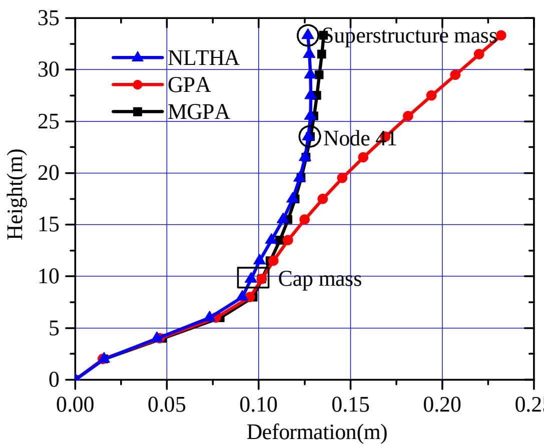
a. Piles



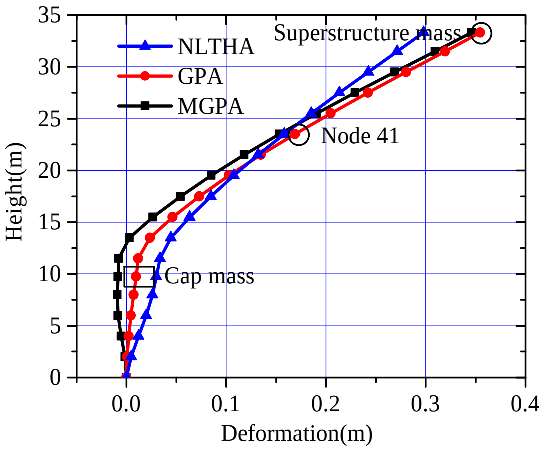
b. Pier

Figure 7: Lateral force vectors of pier and piles.

The simplified bridge model are pushed with the lateral force vectors to corresponding target interstory drifts. Deformation shapes of the model are shown in Figure 8. When interstory drifts of pier and piles are maximum in nonlinear time history analysis (NLTHA), deformation shapes of the model are recorded and shown in Figure 8 too.



a. Piles



b. Pier

Figure 8: Deformation shapes of pier and piles.

In general pushover analyses for piles, Figure 8 shows that GPA can't predict deformation shape of the simplified model, and that MGPA predicts well. In general pushover analyses for pier, both GPA and MGPA provide similar deformation shape as NLTHA. Generally speaking, MGPA vectors are more reasonable.

3.4 Validation of Modified GPA Procedure

Modified GPA, MPA and IDA of the elevated pile foundation system are conducted due to an ensemble of 9 ground motion records. The records are scaled with PGAs ranging from 0.1g to 0.6g. Contributions of the first three modes are considered. Inelastic contributions of the first two modes are considered. Responses of the modified GPA and MPA of the simplified model are compared with each other, as well as the benchmark responses derived from IDA.

Maximum moments of pier and piles

Figure 9 shows the maximum moments of pier and piles computed by modified GPA, MPA and IDA with scaled accelerations (PGA range from 0.1g to 0.6g).

When the elevated pile foundation system vibrates in linear elastic or early inelastic range, the moments predicted by modified GPA for pier are well consonant with the IDA results and the moments predicted by MPA are a little conservative under the ground motions Cape, El, Erzincan, Kern, Kobe, Loma and Sylmar. In further inelastic range, the moments predicted by the MPA for pier are well consonant with the IDA results.

The moments predicted by modified GPA and MPA for piles are almost the same as each other. They match the IDA results well in both linear elastic range and inelastic range.

Plastic hinge rotations of pier and piles

Plastic hinge rotations of pier and piles are illustrated in Figure 10 with scaled accelerations (PGA range from $0.1g$ to $0.6g$). Except for plastic hinge rotations in ground motions of Kobe and Moorpark, reasonable plastic rotations are predicted by modified GPA. Most of them are more accurate than the plastic hinge rotations predicted by MPA. It is revealed that modified GPA provides better seismic evaluation of plastic hinge rotations than MPA.

Modified GPA is able to track the formation of plastic hinges, while MPA is not. For example, when the elevated pile foundation system is subjected to the scaled Bigbear (0.5g and 0.6g), El (0.3g), Kobe (0.4g) and Sylmar (0.3g), the plastic hinge formed in the pier is not captured by MPA, but it is successfully captured by modified GPA.

4 SUMMARY AND CONCLUSIONS

Based on the GPA procedure, modified GPA is proposed for seismic performance evaluation of bridges with elevated pile foundation systems. Principle and basic steps of the modified GPA are developed at first. Then, a bridge with elevated pile foundation systems was chosen for case study. The necessity of considering inelastic contributions of the second mode in multi-mode pushover analysis is illustrated. Modified GPA, MPA and IDA of the simplified model of the bridge are conducted. At last, maximum moments and plastic rotations of the system computed by the three methods are compared with each other. Main conclusions of this study are summarized as following:

1. In longitudinal multi-mode pushover analysis of the bridge, the second mode can't be treated as linear elastic. Otherwise, systematic error will occur when target deformations are computed.

2. Modified GPA can be applied successfully for seismic performance evaluation of the bridge. Inelastic contributions of the second mode can be conveniently considered by Modified GPA.

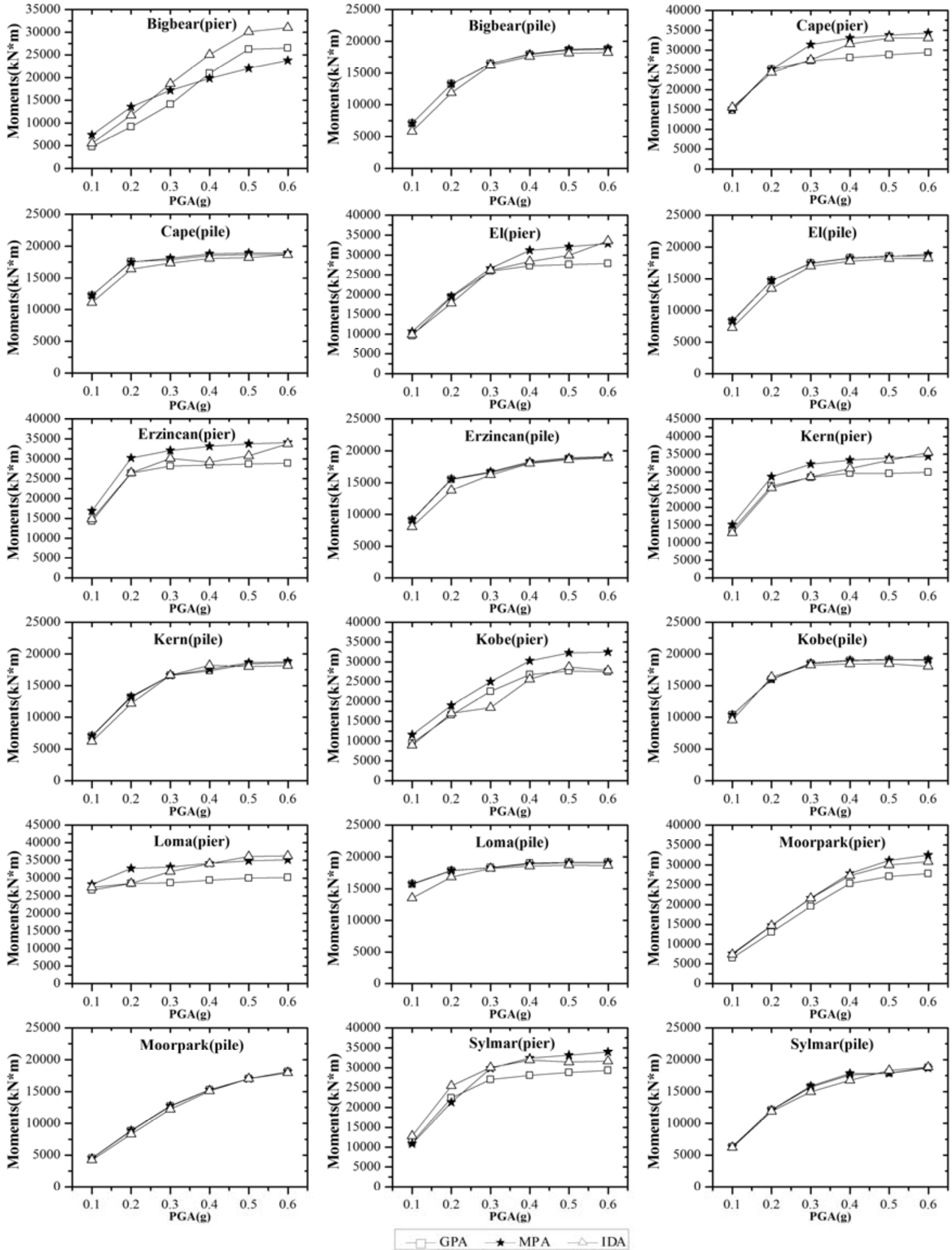


Figure 9: Biggest moments in the pier and piles.

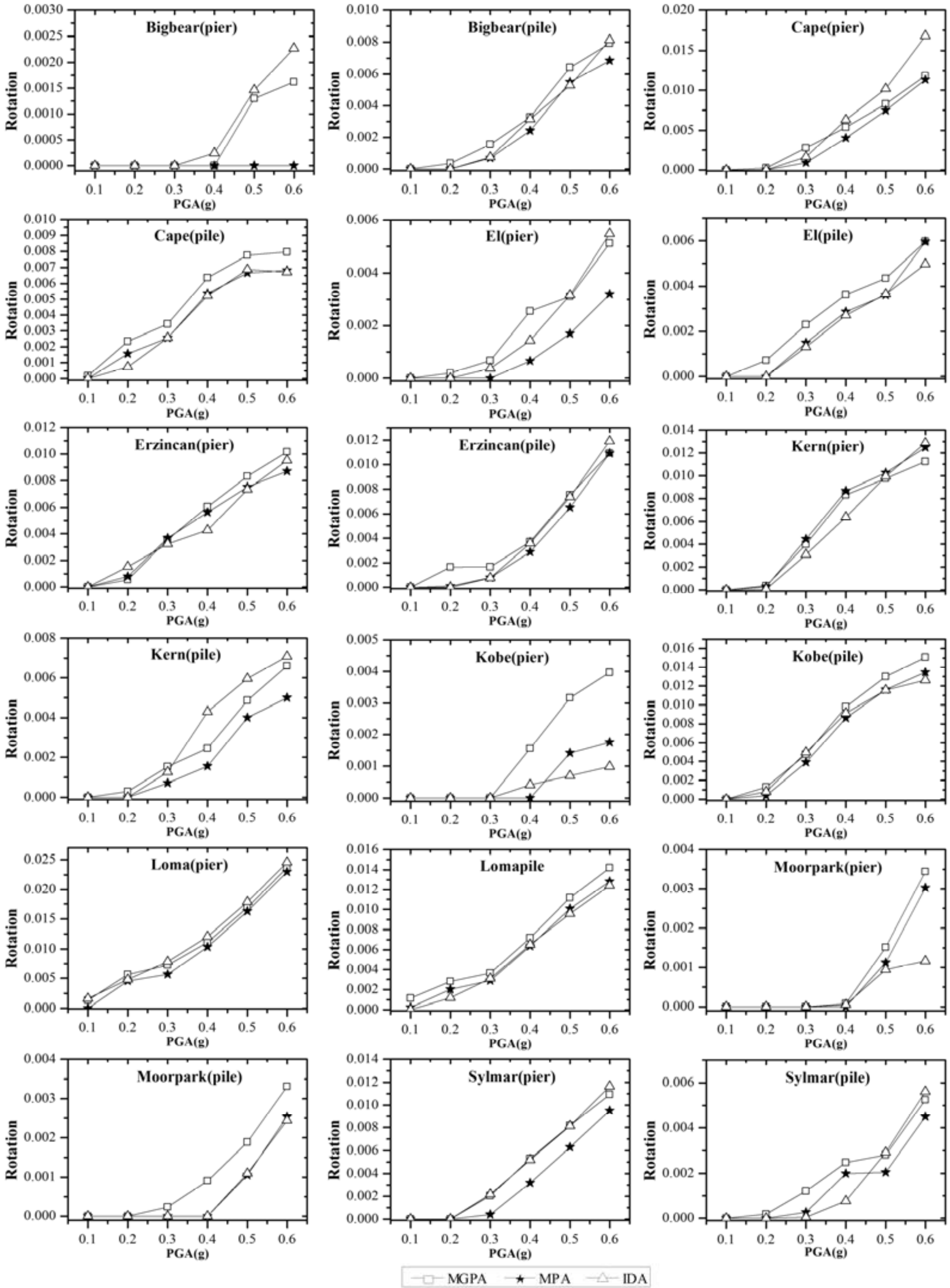


Figure 10: Rotations of the pier and piles.

3. When the bridge vibrates in linear elastic or early inelastic range, maximum moments predicted by modified GPA are more reasonable than moments predicted by MPA. When the bridge enters further inelastic range, more reasonable moments are provided by MPA. Since piles and

piers of bridges are usually designed as capacity-protected components, they vibrate primarily in linear elastic or early inelastic range. In this respect, modified GPA is more attractive for longitudinal seismic performance evaluation of bridges with elevated pile foundation systems.

4. Compared with MPA results, most of plastic hinge rotations predicted by modified GPA are closer to IDA results. Locations of plastic hinges are predicted precisely by Modified GPA, while some locations are not captured by MPA.

5. It should be noted that MGPA procedure is proposed for pushover analyses of bridges with elevated pile foundation systems in longitudinal direction, in which the effective SDOF system of second-mode also enters inelastic range. Basically, MGPA can also applied to bridges or structures in which higher modes don't enter inelastic range.

Limitations

Since MGPA uses the UMRHA method to calculate generalized pushover parameters, its accuracy is better when a structure vibrates in early inelastic range. When a structure enters heavy inelastic range, accuracy of MGPA results should be used with caution, and NL-THA may be used.

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