MODAL PUSHOVER ANALYSIS OF MULTI-LAYER OVERPASS BRIDGE

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ABSTRACT

To evaluate the seismic performance of the multi-layer overpass bridge structure easily and accurately, this paper performed a modal pushover analysis (MPA) on the multi-layer overpass bridge structure and verified the accuracy of its results. A 3-layer overpass bridge was used as the research object, and the structural calculation model was established by the finite element software SPA2000. Modes with a modal mass participation ratio greater than 1% should be determined first, and the cumulative modal mass participation ratios greater than 90%. Then a pushover analysis was performed for each mode to be considered, and structural performance points were obtained by the capability spectrum method. Seismic response results corresponding to structural performance points can be obtained by the formula calculation. Finally, response results of pushover analysis corresponding to each mode were combined by the square root of the sum of the squares (SRSS) method to obtain the total response result. At the same time, the nonlinear time history analysis (NL-THA) results of the bridge structure were obtained by inputting seismic motions into the structural finite element calculation model. The applicability of MPA on multi-layer overpass bridge structures was evaluated by comparing response results obtained from MPA and NL-THA. The results show that MPA can reduce the computational effort compared to NL-THA. The MPA and NL-THA deviations are less than 20% and 25% in the longitudinal and transverse directions, respectively. The results of MPA can effectively evaluate the seismic performance of the multi-layer overpass bridge.

Keywords: Multi-layer overpass bridge, Seismic performance assessment, Modal pushover analysis, Nonlinear time history analysis, Response results.

1. INTRODUCTION

As the speed of urban construction increases, the overpass bridge structures are being used more and more frequently in urban transportation construction worldwide due to their unique advantage of higher utilisation of space (Zhang, et al., 2017). Multi-layer overpass girders are overpass girders with more than 2 layers of main girders. Compared with ordinary single-layer bridges, multi-layer overpass girders have advantages of occupying less space and having superior capacity. They have the characteristics of the stepped distribution of axial force along with the height of frame piers, more potential plastic hinge regions, and complex forces, which will make the seismic design complicated (Zong, 2014). After the collapse of the Cypress double-layer overpass bridge under the Loma Prieta earthquake in 1989 (Mazzoni et al., 2001), scholars have explored and studied the response characteristics and seismic performance of multi-layer overpass bridge structures under earthquake actions. Priestley et al. (1993a, 1993b) pointed out the seismic deficiencies of the Cypress double-layer overpass bridge based on the pseudo-static testing and proposed a strengthening solution. Kunnath et al. (1995) detailed the damage assessment method for general double-deck viaducts using the Cypress viaduct as a relying project. Alali et al. (2013) performed a parametric analysis and obtained that the lateral stiffness of the structure, soil-structure interaction, and reinforcement rate of the main reinforcement have important effects on the lateral seismic response of the double-layer overpass bridge. However, the seismic research data for multi-layer overpasses bridges is still relatively small, and the existing research methods and specifications are not fully applicable to them, and once damage occurs, the damage is large and difficult to repair.

The pushover analysis method was first proposed by Freeman et al. (1978), and subsequently became an important analytical method in the field of seismic performance assessment of structures (Krawinkler et al., 1998). The standard pushover analysis assumes that the response of a structure is controlled by its fundamental mode and is mainly applicable to regular short-period structures. For structures with long periods, the effect of higher-order modes needs to be considered. Chopra et al. (2002) conducted a pushover analysis study considering the effect of higher-order modes based on the idea of mode decomposition as the theoretical basis, performed an MPA on the frame structure, and achieved more satisfactory results compared with rigorous non-linear response history analysis, which verified the superiority of MPA. Paraskeva et al. (2006) applied the MPA method in bridge structure, compared it with the results obtained from nonlinear time history, and achieved ideal results, which promoted the development of the pushover analysis method in the field of bridge engineering. The comparison between MPA results and NL-THA results verifies the applicability of MPA to bridge structures. Wei (2011) verified the adaptability of MPA for three irregular continuous bridges. However, there are few results on MPA of multi-layer overpass bridges.

In this paper, a finite element model of a 3-layer overpass bridge is established using SPA2000, and fundamental mode-based pushover analysis (FPA), MPA, and NL-THA are performed. The comparison of the analytical results illustrates the necessity of considering higher-order vibration modes and the accuracy of MPA for the seismic performance assessment of multi-layer overpass bridges. It can provide a reference basis for the seismic performance evaluation of this type of bridge structure.

2. MODAL PUSHOVER ANALYSIS METHOD

The MPA method has two basic assumptions:

1) The coupling between the individual vibration modes after yielding of the structure is neglected.
2) The total seismic response of the structure is obtained by combining the response of each vibration mode in a specific method e.g. SRSS method, complete quadratic combination (CQC) method.

The steps of the MPA method are shown in Figure 1, and detailed as follows:

Step 1: Establish a structural calculation model, perform modal analysis on the structure, and obtain natural periods, T_n and modes Φ_n for the structures. The modal mass participation ratios can also be obtained.

<u>Step 2</u>: The modes with greater contribution to the structure are selected for pushover analysis. To ensure the reasonableness of modes selection, the selection of nodes can be carried out by two principles: the modal mass participation ratios of the selected nodes are greater than 1%, and the cumulative modal mass participation ratios of selected nodes is greater than 90%.

<u>Step 3</u>: The pushover analysis is carried out separately by the force distribution, $S_n^* = \mathrm{M}\Phi_n$, where M is the mass matrix of the structure, and obtain pushover curve (the base shear vs displacement of the monitoring point) for n th mode.

<u>Step 4</u>: The pushover curve is transformed into a capacity curve using Equation (1), and the design spectrum curve in the specification is transformed into a demand spectrum curve using Equation (2).

$$S_a = \frac{V_{bn}}{M_n^*} \qquad S_d = \frac{u_{rn}}{\Gamma_n \Phi_{rn}} \tag{1}$$

Wherein S_a is the spectral accelerations, S_d is spectral displacements, V_{bn} is the shear base forces, u_{rn} is the displacement of the monitoring point, M_n^* is effective modal mass for n th mode, Γ_n is the modal mass participation ratio for n th mode, and Φ_{rn} is the displacement of the monitoring point for n th mode.

$$S_d = \frac{T^2}{4\pi^2} S_a g \tag{2}$$

Wherein T is the periods in the design spectrum curve.

<u>Step 5</u>: Plotting the capability spectrum curve and the demand spectrum curve in the same coordinate system (S_d vs S_a) and the intersection point of the two is the performance point. The capability spectrum method recommended in ATC-40 (1996) is used to find the performance point of the structure, and then the seismic response of the structure is obtained.

<u>Step 6</u>: Steps 3-5 are repeated to obtain the response values for each selected mode.

<u>Step 7</u>: The total seismic response of the structure is obtained by combining the response values obtained at each mode by certain rules, e.g. the SRSS combination rule, or the CQC rule.

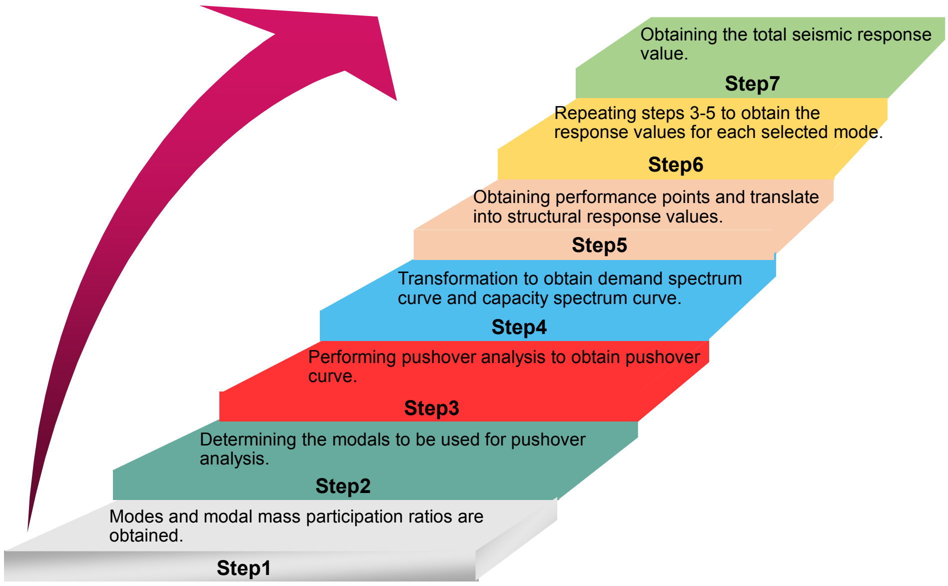


Figure 1: Research steps of the MPA method

3. FINITE ELEMENT MODELING AND MODAL ANALYSIS

3.1 Description of Relying on Bridge Engineering

A 3-layer frame multi-layer overpass bridge in Xi'an, China was selected as the relying on bridge engineering. The bridge adopts a simply supported structural system in the longitudinal direction and a frame pier with 3 layers in the transverse direction. The span arrangement of the bridge is 22+22+22 m. The bridge pier markings are specified from small mileage to large mileage as P1, P2, P3, and P4. The overall layout of the framed 3-layered overpass bridge and the cross-section of the main components are shown in Figure 2. The bridge site category is Class II (GB 50011 - 2010, (MHURDPRC, 2010)), which corresponds to the second Class in Table 9 of the South African Code (TMH7, 1981). The characteristic period is 0.35 s, the seismic intensity of Xi'an is 8 degrees, and the peak ground acceleration is 0.20 g.

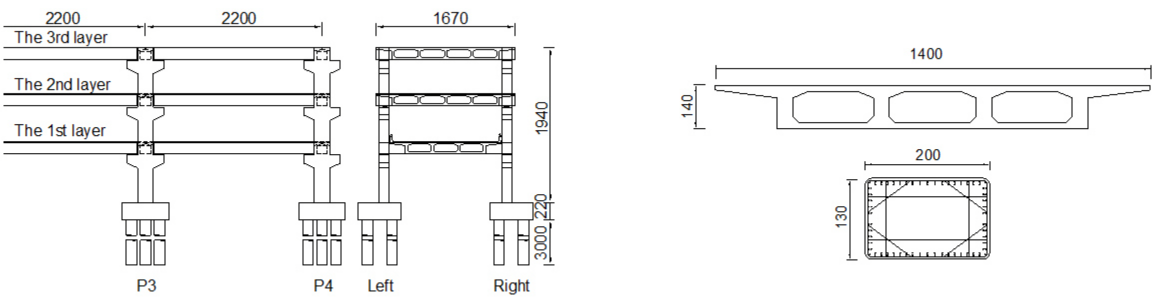


Figure 2: Overall layout of the framed 3-layer overpass bridge (unit: cm)

3.2 Finite Element Modeling

The finite element analysis software SPA2000 was used to build the full bridge model of the frame type 3-layer overpass girder. The potential plastic hinge regions of piers were simulated by fiber plastic unit, the bridge deck load is simulated by line load applied on the main girders, and the girders and piers were simulated by beam unit. The member units were reasonably divided. The force deformation behavior of the bearings was accurately defined, and the spring stiffness matrix model was used to consider the pile-soil interaction. The prototype bridge and finite element analysis model of the framed 3-layered overpass bridge are shown in Figure 3.

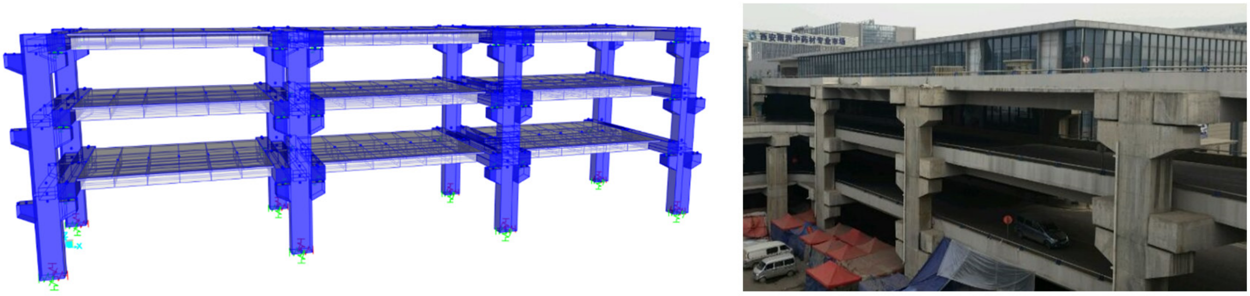


Figure 3: Prototype and finite element model of the framed 3-layer overpass bridge

3.3 Ground Motions Input

The design spectrums of E1 (63% probability of exceedance in 50 years) earthquake action and E2 (2% probability of exceedance in 50 years) earthquake action were determined according to Code (GJJ 166-2011, (MHURDPRC, 2011)) for seismic design of urban bridges. E1 and E2 seismic design intensities correspond to South African code (TMH7, 1981) Class viii seismic intensities (Peak ground acceleration of 0.1g) and Class ix seismic intensities (Peak ground acceleration of 0.3g), respectively. The software was used to randomly generate five artificial seismic time history waves corresponding to the E1 and E2 design spectra. The spectrums of five artificial seismic time history waves fitted well with the design spectrum, which can effectively represent the ground vibration level of the site under the predetermined exceedance probability. The design spectrums and spectrums of five artificial seismic time history waves are shown in Figure 4.

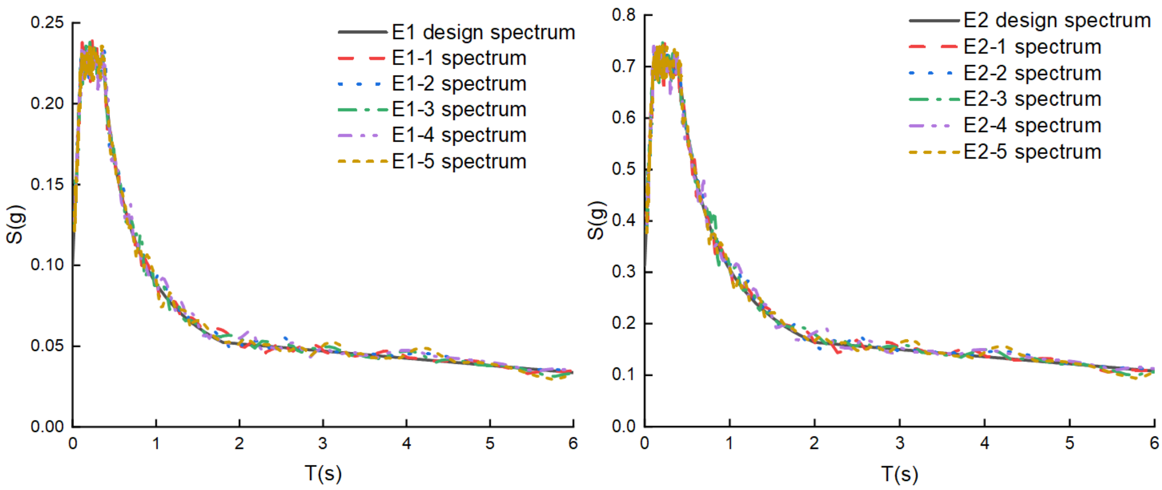


Figure 4: Design spectrum and spectrums of five seismic time history waves

3.4 Selection Modes Dynamic Characteristics

According to the results of dynamic characteristics analysis of the overpass bridge, the cumulative modal mass participation ratios of 91.20% were selected for the mode 1, mode 7, mode 22, mode 61, mode 86, mode 10, and mode 35 in the longitudinal direction, and the cumulative modal mass participation ratios of 92.81% were selected for the mode 3, mode 13, mode 30, mode 20, mode 6, mode 49, mode 36, and mode 32 in the transversal direction. The specific modal mass participation ratios are shown in Table 1.

Table 1: Modal mass participation ratios

Longitudinal direction			Transversal direction		
Mode		Cumulative	Mode		Cumulative
1	66.38%	66.38%	3	64.94%	64.94%
7	12.35%	78.73%	13	8.28%	73.22%
22	4.58%	83.31%	30	4.46%	77.68%
61	3.04%	86.35%	20	4.16%	81.84%
86	1.84%	88.19%	6	3.04%	84.88%
10	1.54%	89.73%	49	2.94%	87.82%
35	1.47%	91.20%	36	2.82%	90.64%
			32	2.17%	92.81%

4. ANALYSIS

4.1 Fundamental Mode-Based Pushover Analysis

FPA can be used as a reference for the MPA to reflect the effect of higher order mode on the framed 3-layered overpass bridge. The pushover curve of FPA of the framed 3-layer overpass bridge is shown in Figure 5. From the figure, the pushover curve obtained by using the FPA shows the development of the structural response of the framed 3-layer overpass bridge structure from the elastic phase to the ductility phase, which can roughly simulate the inertia force distribution of the framed 3-layer overpass bridge under seismic action in both directions.

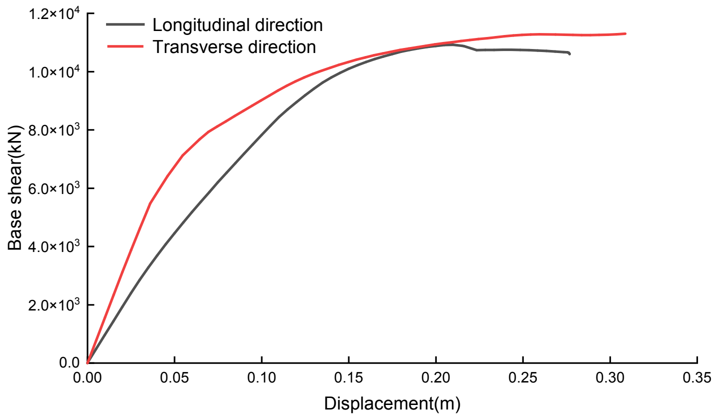


Figure 5: Base shear vs displacement curves of FPA in both directions

4.2 Analysis of Calculation Results in the Longitudinal Direction

In order to better consider the effect of higher order modes on the frame 3- layer overpass bridge for pushover analysis, MPA is performed of the bridge in the longitudinal direction. According to the mode selection analysis in Section 3, mode 1, mode 7, mode 22, mode 61, mode 86, mode 10 and mode 35 are selected for pushover analysis in the longitudinal direction.

According to the steps in Section 2, MPA of the frame 3-layer overpass bridge was performed in the longitudinal direction, and capacity curves for seven longitudinal modes and design spectrums of E1and E2 are plotted as shown in Figure 6. From the figure, performance intersection points of the seven mode capability spectrums and the E1 design demand spectrum are in the elastic phase. Performance intersection points of the capability spectrum of modes 1 and 22 with the E2 design requirement spectrum are in the inelastic phase, and performance intersection points of the other five mode capability spectrums with the E2 design demand spectrum are in the elastic phase. The values of the performance intersection point in the inelastic phase can be calculated by step 5 in Section 2. The values of the performance intersection points corresponding to seven modes are shown in Table 2.

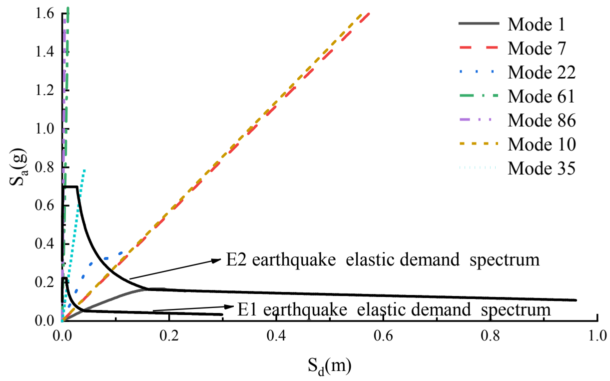
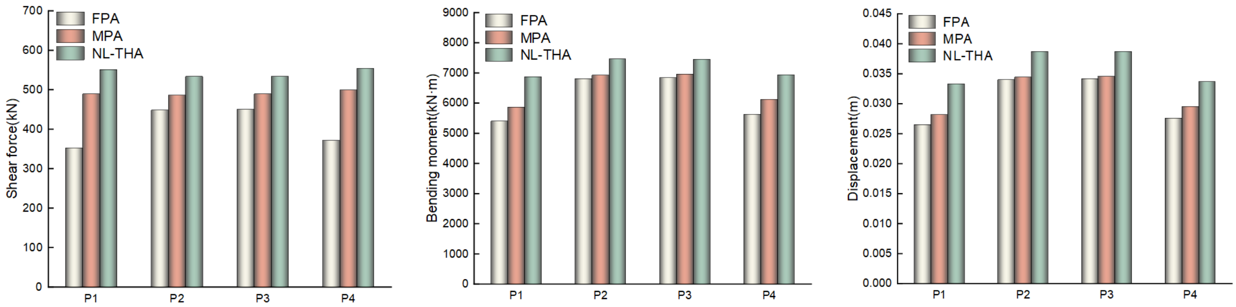


Figure 6: Capacity curves for seven longitudinal modes and the design spectrums

Table 2: Performance intersection point values for seven longitudinal modes

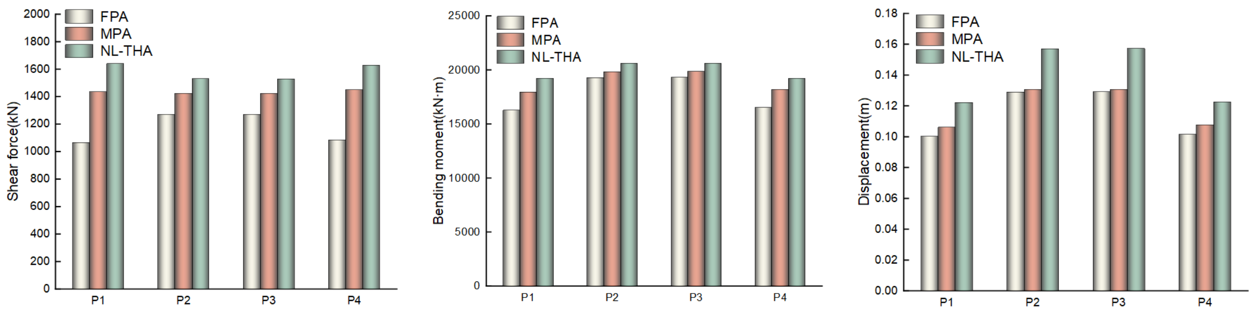
Seismic category	Performance Points	M1	M7	M22	M61	M86	M10	M35
E1	S_d (m)	0.039	0.027	0.018	0.0018	0.00065	0.027	0.0093
	S_a (g)	0.053	0.075	0.106	0.223	0.223	0.076	0.181
E2	S_d (m)	0.131	0.093	0.056	0.0053	0.002	0.092	0.032
	S_a (g)	0.150	0.259	0.304	0.719	0.697	0.262	0.622

The base shear force and monitoring point displacement values in the longitudinal direction are obtained by converting the performance intersection points values (S_d, S_a) of the capacity spectrums and design demand spectrums. At the same time, we can obtain various response values for the structure, and the final calculated result values of MPA are obtained by the SRSS combination method. This paper mainly analyzes the calculation results through 3 aspects of pier bottom shear force, pier bottom bending moment, and pier top displacement, the calculation results are shown in Figures 7 and 8. To more directly compare the difference between the values of the calculated results of the three methods, the deviation of the calculated results of the three methods is shown in Figure 9. As can be seen from figures, the bottom shear force, bottom bending moment and top displacement of bridge piers obtained by the MPA method are closer to those calculated by the NL-THA method than the results of the FPA method. The shear force deviations between FPA and NL-THA in two design stages are more than 30%, The response value deviations between MPA and NL-THA in two design stages are less than 20%, mainly because the calculation results obtained by MPA consider the influence of higher modes. It shows that the MPA method can be used to evaluate effectively the longitudinal seismic response of bridge structures, whereas the FPA method introduces large errors in predicting the longitudinal seismic response of bridge structures.



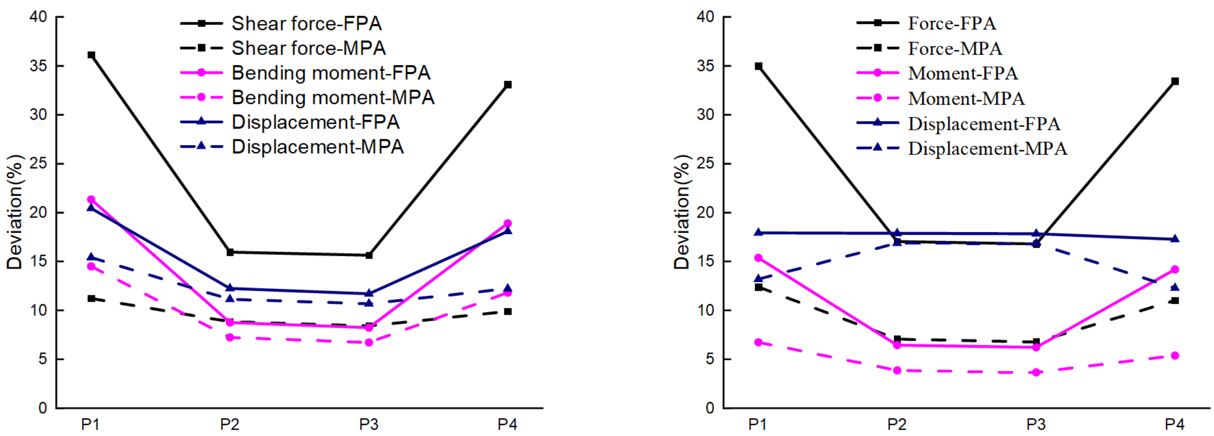
(a) Shear force **(b) Bending Moment** **(c) Displacement**

Figure 7: Response to the E1 earthquake (longitudinal direction)



(a) Shear force **(b) Bending Moment** **(c) Displacement**

Figure 8: Response to the E2 earthquake (longitudinal direction)



(a) E1 **(b) E2**

Figure 9: Comparison of results from different methods for the longitudinal direction

4.3 Analysis of Calculation Results in the Transversal Direction

According to the mode selection analysis in Section 3, mode 3, mode 13, mode 30, mode 20, mode 6, mode 49, mode 36, and mode 32 are selected for pushover analysis in the transversal direction.

Capacity spectrum curves for eight longitudinal modes and design spectrums of E1 and E2 are plotted as shown in Figure 10. From the figure, performance intersection points of eight mode capability spectrums and the E1 design demand spectrum are in the elastic phase. The performance intersection point of the capability spectrum of mode 3 and the E2 design demand spectrum is in the inelastic phase, and performance intersection points of the other seven mode capability spectrums and the E2 design demand spectrum are in the elastic phase. The values of the performance intersection point in the inelastic phase can be calculated by step 5 in Section 2. The values of the performance intersection points corresponding to eight modes are shown in Table 3.

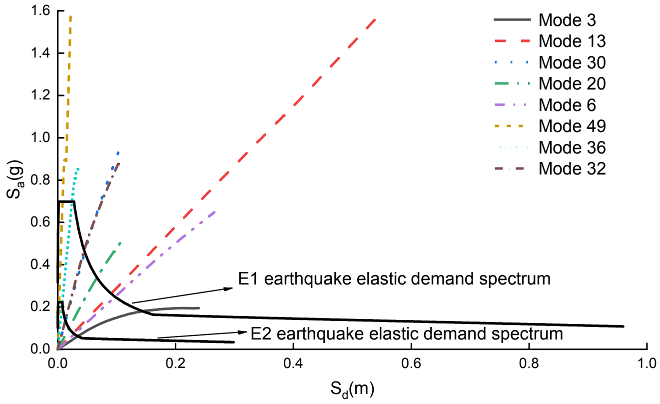


Figure 10: Capacity curves for eight transversal modes and the design spectrums

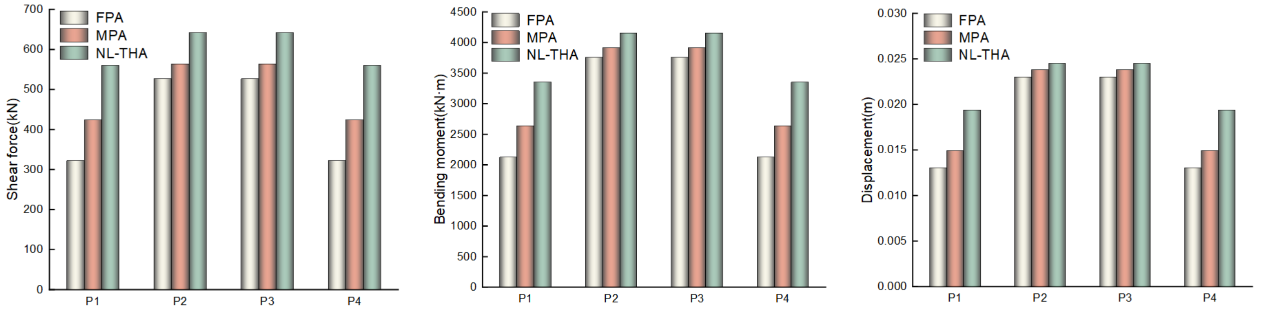
Table 3: Performance intersection points values for eight transversal modes

Seismic category	Performance Points	M3	M13	M30	M20	M6	M49	M36	M32
E1	S_d (m)	0.034	0.026	0.013	0.019	0.028	0.0026	0.077	0.013
	S_a (g)	0.062	0.077	0.134	0.100	0.072	0.223	0.223	0.138
E2	S_d (m)	0.108	0.091	0.046	0.064	0.098	0.008	0.024	0.056
	S_a (g)	0.157	0.264	0.457	0.329	0.249	0.697	0.692	0.620

The base shear force and monitoring point displacement values in the transversal direction are also obtained by converting the performance intersection point values (S_d , S_a) of the capacity spectrums and design demand spectrums. At the same time, we can obtain pier bottom shear force, pier bottom bending moment, and pier top displacement response values for the structure, and the final calculated result values of MPA are obtained by the SRSS combination method, the calculation results are shown in Figures 11 and 12. The difference between the calculation results of the three methods is shown in Figure 13. The deviations in the transversal direction are greater than the deviations in the longitudinal direction. Deviations in response values are less than 25% for MPA and NL-THA and greater than 30% for FPA and NL-THA in both design phases. As with the longitudinal direction calculation results, the MPA calculation results are closer to those of NL-THA than those of FPA, mainly because the calculation results obtained by MPA take into account the effect of higher-order modes on the structure. The deviations in the response values for Piers 2 and 3 are larger than for Piers 1 and 4, mainly because MPA cannot take into account the changes in the stiffness matrix during plastic development and the influence of boundary conditions. From the overall effect, the MPA method can make a better prediction of the transverse seismic response of the bridge.

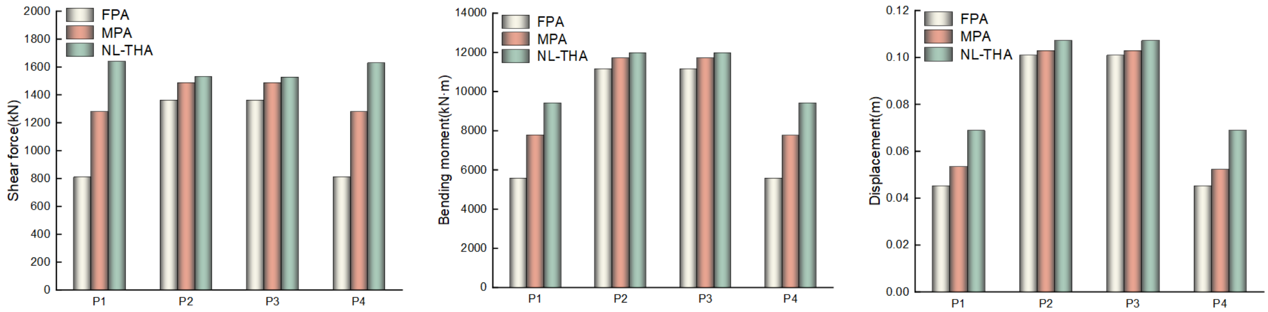
It should be noted that some limitations of the MPA analysis method are mainly as follows:

- The coupling between the modes of each order after the structure enters inelastic period is ignored.
- The combination of the response values of each order of mode using the SRSS or CQC method to obtain the total response value is not based on a rigorous theory.
- In the future, we can develop the MPA method from the above aspects, and then more accurately assess the seismic performance of the structure.



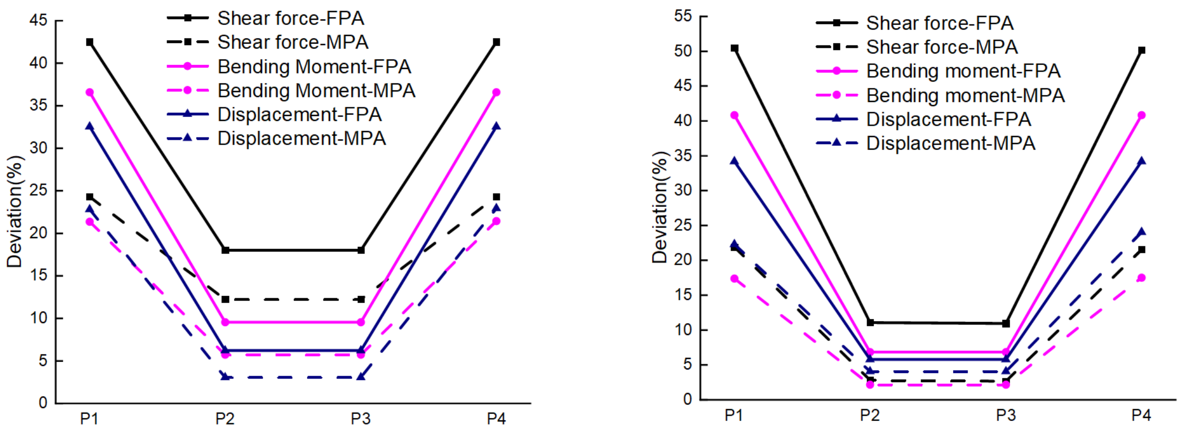
(a) Shear force **(b) Bending Moment** **(c) Displacement**

Figure 11: Response to the E1 earthquake (transversal direction)



(a) Shear force **(b) Bending Moment** **(c) Displacement**

Figure 12: Response to the E2 earthquake (transversal direction)



(a) E1 **(b) E2**

Figure 13: Comparison of results from different methods for the transversal direction

5. CONCLUSIONS

This paper verifies the rationality of using the MPA to evaluate the seismic performance of multi-layer overpass bridges. By comparing the calculated results of FPA, MPA and NL-THA methods, the following conclusions can be drawn:

- The pushover analysis can reflect the development of the structure from the elastic phase to the elastic-plastic phase in both directions.
- Compared to the FPA, the MPA is closer to the NL-THA in estimating the seismic demand of multi-layer overpass bridges. The MPA method can well take into account the influence of other order modes with higher mass participation ratios on the structure. Prediction of seismic response of multi- layer overpass bridges using the FPA method is inaccurate.
- The deviations between the MPA and the NL-THA are less than 20% and 25% in the longitudinal and transverse directions, respectively, and MPA can make an effective evaluation of the required response amount for the seismic design of multi-layer overpass bridges. MPA can improve the computational efficiency compared with the NL-THA and can be used for the preliminary seismic design of such bridges.

6. DECLARATION OF COMPETING INTEREST

The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

7. ACKNOWLEDGEMENTS

This work was supported by the National Natural Science Foundation of China (grant numbers 51978062); Key Research and Development Project of Shaanxi Province Fund (grant numbers 2019KW-051); Shaanxi Innovative Talents Promotion Plan-Science and Technology Innovation Team Fund (grant numbers 2018TD-040); Natural Science Basic Research Program of Shaanxi-Joint Fund Program (grant numbers 2021JLM-47).

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