

A static analysis-based method for estimating the maximum inelastic seismic response of upper-deck steel arch bridges

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ABSTRACT: We have previously proposed a static analysis-based method for estimating the maximum inelastic seismic response of upper-deck steel arch bridges. The method employs free vibration analysis, response spectrum method and equal energy assumption for the estimation of maximum response. Correction functions are proposed to improve the accuracy of estimates by the equal energy assumption. In this paper, the outline of the proposed method is introduced and its validity is shown through the numerical evaluations.

1 INTRODUCTION

Japanese seismic design code for highway bridges (JRA 2002) was revised after the Hyogo-ken Nanbu earthquake and a new design ground motion type called the Level 2 ground motion was introduced. The inelastic response demand of all structures is specified to be obtained for the verification of the design against this new ground motion. Steel arch bridges are no exceptions. Nonlinear dynamic response analysis became compulsory to obtain the inelastic seismic demand for them. This greatly complicates the design process for the steel arch bridges, which are conventionally treated as structures for which earthquake loading is not predominant. There is a desire for a method of seismic evaluation that does not rely on dynamic response analysis.

In our previous research (Cetinkaya 2006), we have developed a method for the estimation of maximum inelastic out-of-plane response of upper-deck steel arch bridges that does not require dynamic response analysis. The method combines pushover analysis with the response spectrum method by using the equal energy assumption (Veletsos 1960) with some correction functions to improve the estimation accuracy. Although seismic deficiencies under longitudinal excitations in steel arch bridges are minor (Usami 2004), a simplified approach for the in-plane response, which can be an additional tool for the evaluation of the overall seismic performance, is also necessary. For this purpose, the applicability of the method to the maximum in-plane response estimation is also studied (Cetinkaya 2009) by carrying out numerical examinations on the same bridge models studied previously. Consequently, it is found that the method can be applied to the estimation of maximum inelastic in-plane response with a reasonable accuracy by only changing the pushover analysis procedure.

This paper presents the outline of the method and its numerical evaluation results.

2 PROPOSED METHOD

The basic application steps of the proposed method to the maximum inelastic response estimation are listed below.

- (1) Establish a finite element (FE) model for the upper-deck steel arch bridge under investigation;
- (2) Perform eigenvalue analysis to acquire the predominant vibration modes;

(3) Obtain the force-displacement relationship of the structure as well as the yield displacement δ_y by performing elasto-plastic pushover analysis using the modal force distribution obtained in Step 2 for the out-of-plane response estimation (Cetinkaya 2006), or using an incremental displacement load pattern placed at the mid point of the stiffening girder in the longitudinal direction for the in-plane response estimation (Cetinkaya 2009);

(4) Obtain the maximum response from the response spectrum specified in the JRA code (JRA 2002) for Level 2 ground motion depending on the corresponding ground condition and modal damping ratio. Calculate the corresponding elastic strain energy;

(5) Estimate the maximum inelastic response displacement δ_{SP} SP by applying the equal energy assumption to the force-displacement curve obtained in step 3 and the maximum strain energy obtained in Step 4. Calculate the estimated ductility factor μ_E , ($\mu_E = SP/y$);

(6) Calculate the value of the correction function $f(\mu_E)$ either for the average estimation (equation (1)) or the lower bound estimation (equation (2)).

$$f(\mu_E) = 1/(0.1843\mu_E + 0.8159), (0 < f(\mu_E) \leq 1) \quad (1)$$

$$f(\mu_E) = 1/(0.1700\mu_E + 0.7050), (0 < f(\mu_E) \leq 1) \quad (2)$$

$$\delta'_{SP} = f(\mu_E) \times \delta_{SP} \quad (3)$$

The average estimation correction function is for the optimum estimation whereas the lower bound estimation correction function guarantees that the estimated maximum response is greater than or equal to the actual inelastic response in most cases. Correction is only necessary when the value of the correction function is less than 1. Correction is carried out by simply multiplying the estimated maximum inelastic response by the correction function of the desired type as shown in equation (3).

3 NUMERICAL EVALUATION (Cetinkaya 2006, 2009)

3.1 Analyzed models

The applicability of the method is examined numerically by studying six upper-deck steel arch bridge models. The models differ in their arch-rise to span ratio and arch rib spacing, as shown in Table 1. These two structural parameters are given variations over a wide coherent range in order to obtain a pattern representing the behavior of general upper-deck steel arch bridges and also to examine their influence on the applicability of the method.

Model 1 shown in Figure 1 is used as the template from which the other five parametric models are generated. This bridge was adopted by the JSSC committee as a representative model for investigations of nonlinear behavior during major earthquakes (Usami 2003). The parametric models are generated by using the JSP-15W preliminary design software for steel arch bridges (JIP 2003). Models 2-4 are generated from Model 1 by changing only the arch rise. Models 5 and 6 are generated from Model 1 by changing only the spacing between the two arch ribs. The generation process is carried out carefully, in order to ensure that the newly generated models remain within realistic limits. The selected arch-rise to span ratios can be found in existing steel arch bridges. The template Model 1 and newly generated Models 2-4 carry two-lane traffic. The distance between the arch ribs is widened in order to carry a three-lane deck in Model 5, and a four-lane deck in Model 6. In this way, realistic steel arch bridge models are generated for numerical analysis. Models 1, 2, 3 and 4 constitute a pattern demonstrating the effect of arch-rise to span ratio, whereas Models 1, 5 and 6 are a series demonstrating the effect of arch rib spacing on the applicability of the method. Fig.1 also gives the cross sections of the main structural elements of the template model. A box-type section is used for the arch rib and side column, whereas an I-section is adopted for the stiffening girder. The figure shows the cross section of the arch rib near the support and that of the stiffening girder in the span center. The side columns have a uniform box section. The other five

generated models have cross sections of the same shape based on dimensions given by the preliminary design software. The models are analyzed by using MARC nonlinear finite element (FE) analysis software (MSC Software 2005).

Please refer to the reference (Cetinkaya 2006, 2009) for the detail of the models and analysis conditions.

Table 1 : Structural parameters of the analyzed models

Model	Span Length (m)	Arch Rise (m)	Arch Rise Span Length	Arch Rib Spacing (m)
Model 1	114	16.87	0.15	6.0
Model 2	114	22.80	0.20	6.0
Model 3	114	34.20	0.30	6.0
Model 4	114	45.60	0.40	6.0
Model 5	114	16.87	0.15	9.5
Model 6	114	16.87	0.15	13

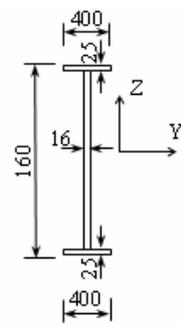
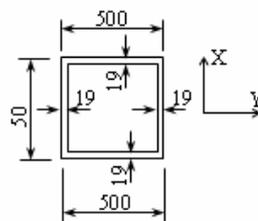
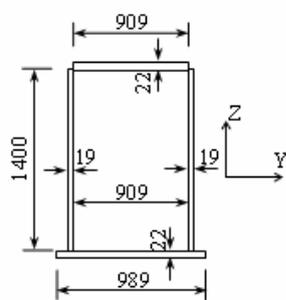
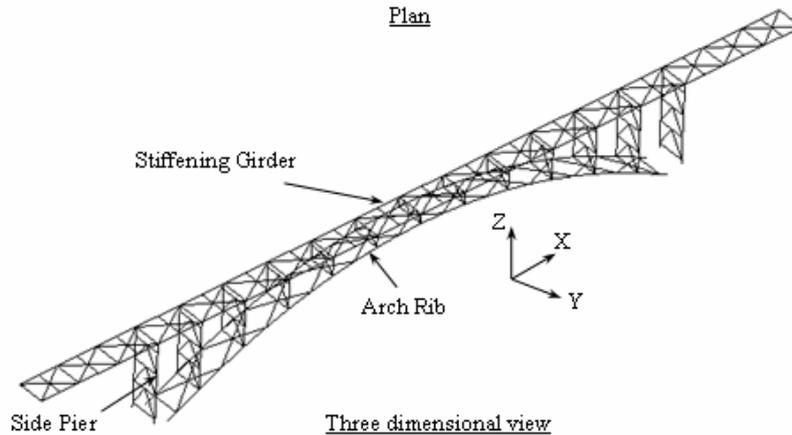
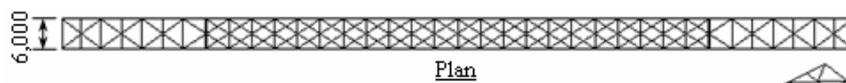
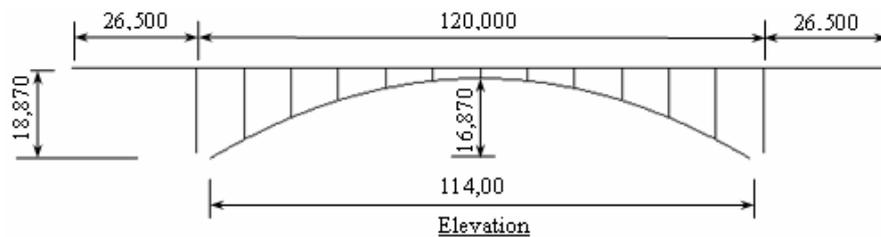


Figure 1 : Model 1

3.2 Pushover analysis

The applicability of the method greatly depends on selecting appropriate load pattern for the pushover analysis that will deform the structure similar to maximum dynamic response. Therefore, it is necessary to verify that dynamic behavior is sufficiently well represented by pushover analysis with the loading pattern described in Chapter 2.

(1) Out-of-plane response

The displacement distribution obtained by pushover analysis in which a modal force distribution is used as a lateral loading pattern is compared with that obtained from the nonlinear dynamic response. This comparison is carried out for each model using the most severe dynamic excitation. The displacement distribution obtained in the dynamic response analysis at the time increment representing the maximum response at the reference point is compared with the distribution given by pushover analysis at the static force increment corresponding to the same reference point displacement. The comparisons are given in Fig.2 for the stiffening girder and the arch rib of models 1 and 3. These two models are chosen since they showed relatively larger difference between pushover and dynamic response analyses. From this figure, it can be concluded that pushover analysis carried out using a modal force distribution based on the first out-of-plane vibration mode with an effective mass ratio of more than 60% suitably accounts for dynamic behavior, matching the findings of Lu et al. (2004).

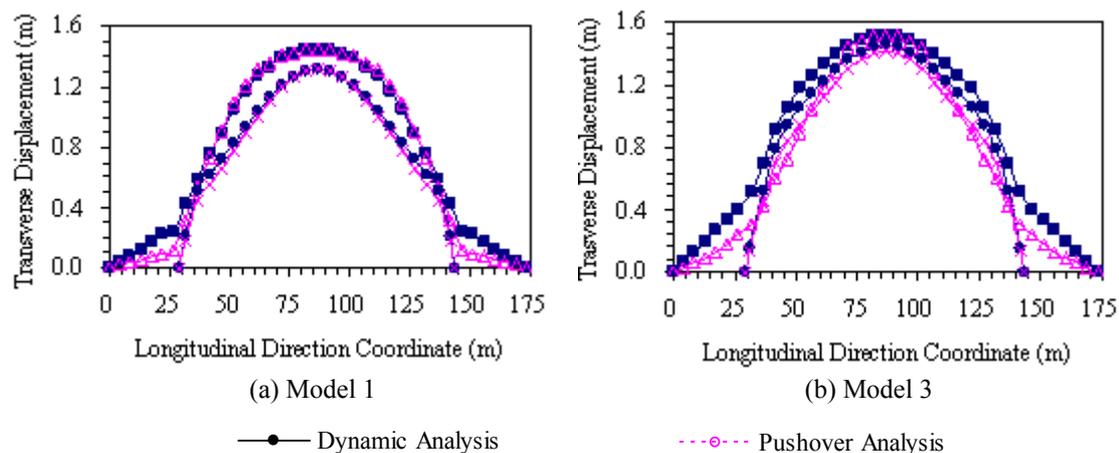


Figure 2 : Transverse displacement distributions for pushover and dynamic response analyses

(2) In-plane response

In the out-of-plane response estimation, the load pattern proportional to the product of the eigenvector of the dominant single mode and the distribution of the concentrated mass was adopted (Cetinkaya 2006). The same approach was applied to the in-plane pushover analysis by adopting a modal force distribution from the single dominant mode in the longitudinal direction (1st in-plane mode). Vertical component of this mode was also taken into account since vertical displacement is significant in the longitudinal excitations. However, analysis revealed that the deformed shape of the pushover analysis is significantly different from the displacement distribution of the dynamic response when such a load pattern is employed.

Therefore, an alternative load pattern, which is an incremental displacement load applied at the mid point of the stiffening girder from the both sides, is adopted for the pushover analysis due to its simplicity being likely to simulate dynamic response at its ultimate stage. To check the validity of this loading pattern, the displacement distribution obtained by pushover analysis is compared with that obtained from the dynamic response analysis. The similar comparisons to Fig.2 are given in Fig.3 only for the stiffening girder in each model since similar shapes are also observed for the arch ribs. The reference point is the node at the 1/4 span on the stiffening girder since the maximum vertical displacement is observed at this node during dynamic response analysis. These comparisons demonstrate that the displacement distributions agree well with each other suggesting that the employed load pattern is sufficiently accurate to account for the in-plane dynamic behavior.

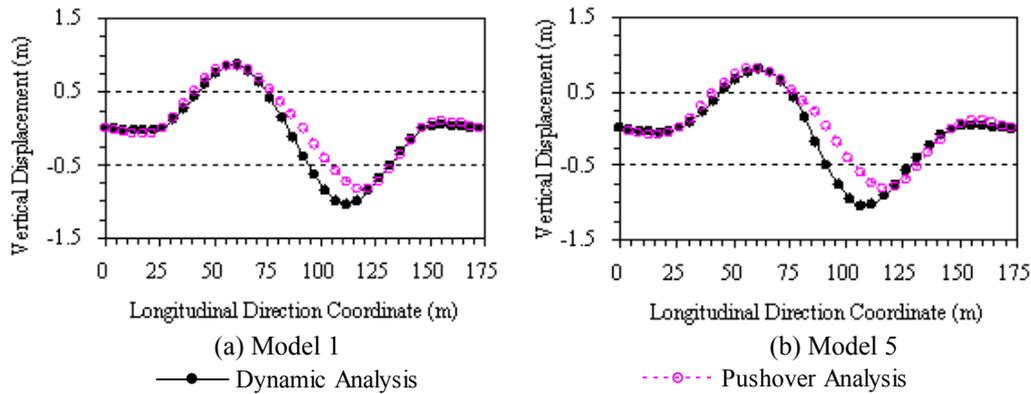


Figure 3 : Vertical displacement distributions for pushover and dynamic response analyses

3.3 Estimation accuracy

(1) Out-of-plane response

In order to illustrate the accuracy of the proposed method, the estimated maximum nonlinear response δ^{SP} is compared with the actual maximum dynamic response δ^{DP} calculated directly by nonlinear dynamic response analysis. This comparison is shown for the average estimation in Fig.4 for ground conditions I and II. The estimation error range is around 20% for the individual ground motions and 15% for the average response displacements. The lower bound estimation is studied only for average response displacements since it is meaningful only for a design procedure in which the average of three response displacements should be taken according to JRA code (JRA 2002), and the error in this case is found to be less than 20% as shown in Fig.5. Judging from these figures, it is considered that the proposed method is applicable to the preliminary design of upper-deck steel arch bridges as a simple way of predicting their maximum response.

For further confirmation of its validity, the proposed method is applied to the same models using a different set of ground motions. These are ground motions not considered during the development of the correction functions. Type I ground motions for ground conditions I and II, amplified by factors of 1.5, 2 and 5, are employed as the input ground motions in this examination. The estimation obtained, δ^{SP} , are compared with the actual dynamic response, δ^{DP} , in Fig.6. The results for average response displacements are within the error range of

20%. It can be also seen that fairly good estimation results are obtained for individual ground motions. These findings verify the proposed method for type I ground motions in addition to type II. However, it should be noted that the lower bound estimation results, which are supposed to fall on the safe side, are given as less than the actual response in a few cases.

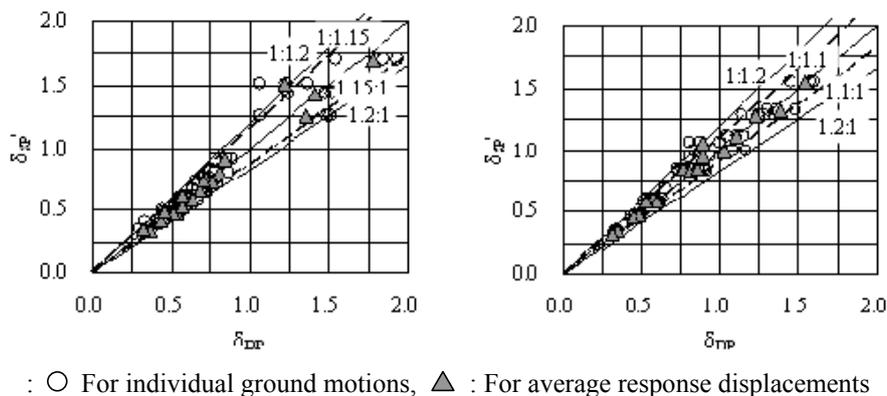


Figure 4 : Average estimation results with the proposed method: (a) Ground Condition I, (b) Ground Condition II

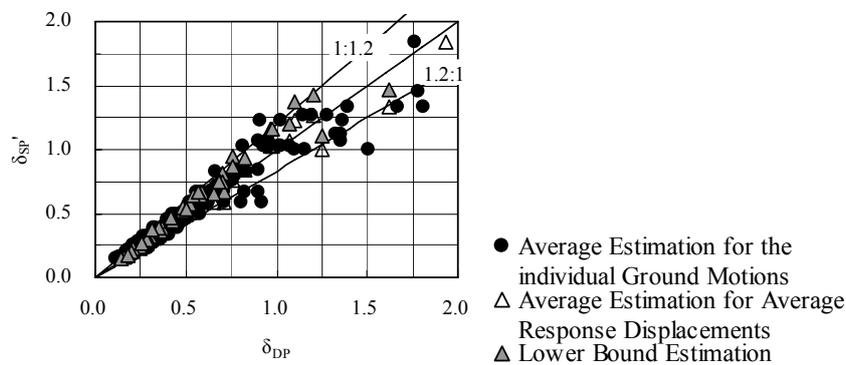
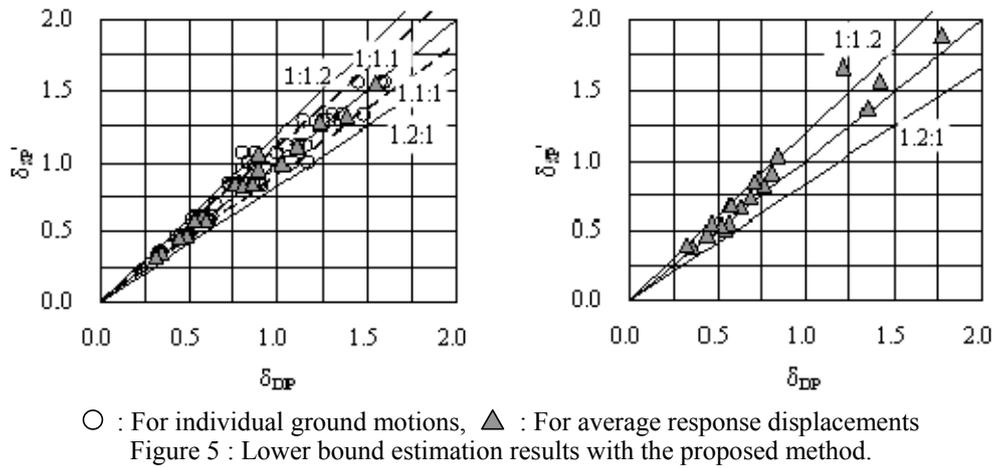


Figure 6 : Estimation accuracy for the type I ground motions

(2) In-plane response

The validity of the method for the in-plane response is also illustrated through the numerical examples by comparing the maximum nonlinear response δ^SP estimated by the method with the actual dynamic response δDP calculated directly by nonlinear dynamic response analysis. This comparison is shown for average and lower bound estimations in Fig.7(a) and (b), respectively. The average estimation leads to an error around $\pm 20\%$ for the individual ground motions and $\pm 15\%$ for the average response displacements. The lower bound estimation is studied only for average response displacements due to the same reason as the out-of-plane response. The error in this case is found to be less than 20% as shown in Fig.7(b). When these results are compared with the yields of out-of-plane direction, it can be recognized that the estimation accuracies of the method for in-plane and out-of-plane responses are almost on a level. Within this error range, it is considered that proposed method can be used for the preliminary design of upper-deck steel arch bridges as a simple way of predicting the maximum in-plane response as well.

For further confirmation, the proposed method is applied to the same models by using different set of ground motions. Type I ground motions for ground conditions I and II, amplified by factors of 1.5, 2 and 5 are employed like it was done for the out-of-plane direction evaluations. The estimation obtained, δ^SP , are compared with the actual dynamic response, δDP , in Fig.8. Fairly good estimation results are obtained for average estimations with the estimation error less than $\pm 20\%$. Lower bound estimation also leads to an error of less than 20%. However, it should be noted that some of the estimation results are less than the actual results, although the safe side estimation should be achieved with the lower bound estimation.

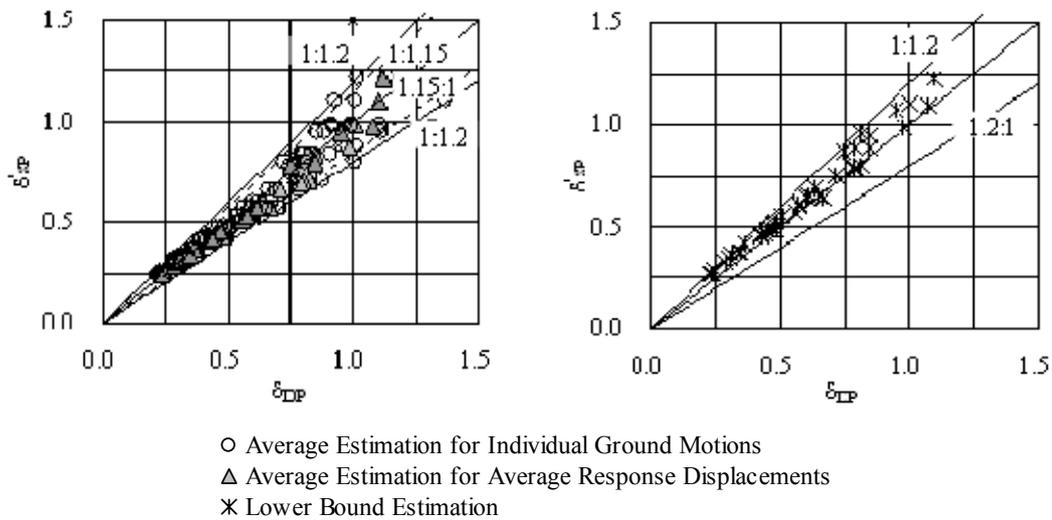


Figure 7 : In-plane response estimation results with the proposed method.

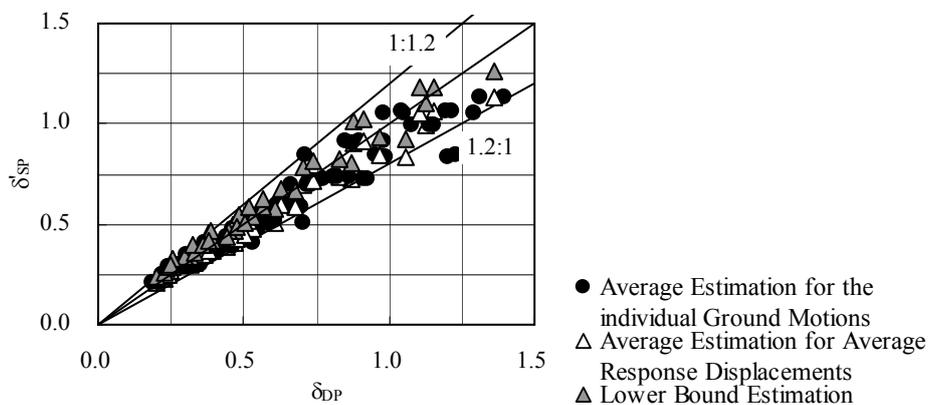


Figure 8 : Estimation accuracy for the type-I ground motions.

4 CONCLUDING REMARKS

This paper introduces the outline of a static analysis-based method for estimating the maximum inelastic seismic response of upper-deck steel arch bridges and shows its validity through numerical evaluation.

The method was established based on the numerical analysis results of parametric upper-deck steel arch bridge models. The equal-energy assumption was applied on the results of pushover analysis and response spectrum method to predict the inelastic response at the reference points where the maximum structural response is observed in the case of transverse and longitudinal Level 2 ground motion excitations. Certain correction functions were proposed in order to improve the estimation accuracy of the equal-energy assumption. Having improved the estimates of the maximum structural response, seismic demand of the whole system can be obtained from the pushover analysis results corresponding to the estimated maximum response at the reference point.

In the future, the proposed method will be applied to other upper-deck steel arch bridges and will be evaluated for its accuracy and efficiency compared to the conventional nonlinear dynamic response analysis.

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