Estimation of Seismic Demands in Bridges Crossing Fault Rupture Zones

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ABSTRACT
This paper presents an analytical procedure for estimating seismic demands of a bridge crossing a strike-slip fault that is capable of producing motions with permanent offset. In this procedure for linear elastic systems, seismic demands are computed by adding the results from: (1) quasi-static analysis for all support displacements applied simultaneously, and (2) dynamic analysis for reference ground acceleration using the "effective" influence vector. A unique feature of this procedure is that even though the bridge is subjected to multi-support excitation, its dynamic response can be estimated by procedures similar to standard procedures for systems subjected to uniform ground motion.

Analysis of a linear elastic, three-span symmetric bridge subjected to transverse support motions resulting from a fault rupture between its two bents shows that only the torsional modes are excited even though the bridge is subjected to transverse motions; in contrast, only transverse modes are excited for symmetric bridge subjected to uniform transverse support motions. Furthermore, it is found that deck displacement at abutment 1 is dominated by the quasi-static part, whereas column drift is dominated by the dynamic part. Therefore, no generalization should be made about ignoring either quasi-static or dynamic parts in estimating seismic demands.

Finally, responses of bridge with nonlinear shear keys obtained from nonlinear response history analysis are compared with bridges without shear keys and bridges with elastic shear keys. A simple tri-linear model for shear-key behavior is used to "mimic" the experimental results. It is shown that response of bridges with nonlinear shear keys may be significantly different compared to the bridge either without shear keys or with elastic shear keys. However, the cases without shear keys and with elastic shear keys may be used to obtain bounding values on the response of bridges with nonlinear shear keys. Since different shear key condition may provide upper bound for different seismic demands, both cases – without shear keys and with elastic shear keys – must be considered to establish upper bounds on all seismic demands. The current Caltrans practice of ignoring shear keys may not always provide conservative estimates of all seismic demands.

INTRODUCTION

Bridges crossing active earthquake faults should be designed for a fault rupture offset and near-fault ground motions, which may include a forward directivity pulse and/or a fling displacement. While site-specific seismological studies to define spatially varying (or multiple-support) ground motions, including fault rupture, and rigorous nonlinear response history analysis (RHA) are required for important bridges designated as "lifeline" structures, such investigations are too onerous for "ordinary" bridges. For such structures, simplified procedures for estimating seismic demands are needed to facilitate their seismic evaluation and design. As a first step towards developing simplified procedure, this paper examines the dynamic response of bridges located across a strike-slip fault capable of producing a magnitude 6.5 earthquake that results in permanent offset between two sides of the fault.

Presented first is an analytical procedure for estimating seismic demands of linear elastic bridges crossing a strike-slip fault that is capable of producing motions with permanent offset. The dynamic response of linear elastic bridges is examined next to identify the modes that may be excited during selected ground motions, and examine the contribution of various parts in the analytical procedure. Finally, response of bridges responding beyond the linear elastic range is examined. In particular, seismic demands in bridges with nonlinear shear keys are compared with two extreme cases: bridges without shear keys and bridges with elastic shear keys. The shear key is modeled by a simple tri-linear force-deformation relationship that "mimics" the experimental observations.

CONSIDERED BRIDGE AND GROUND MOTIONS

A three-span bridge with single-column bents with fault located between bent 2 and bent 3 is considered (Figure 1). The deck is modeled by elastic beam elements. The columns are modeled by elastic beam elements for linear analysis and by nonlinear fiber-section beam-column elements for nonlinear analysis. For first part of the paper, the transverse restraint provided by shear keys at the abutment is ignored. However, linear and nonlinear shear keys are included in the later part of the paper.

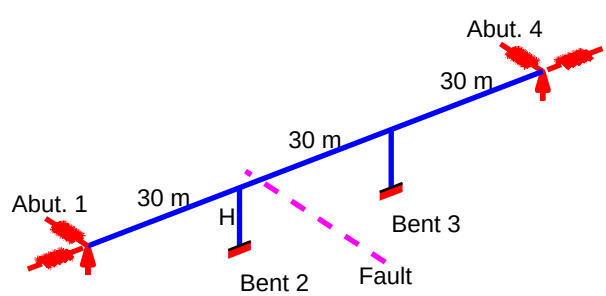

Figure 1. Selected three-span symmetric bridge with single-column bents.

The motions used in this investigation are the simulated motions, generated by the seismological group of the research team, for a magnitude 6.5 strike-slip event resulting in fault rupture between two bents of the bridge (Figure 2). These motions indicate that: (1) the spatial variation in the ground motion on either side of the fault is minimal, and (2) the motion reverses algebraic sign at supports located on two opposite sides of the fault. Therefore, motion at supports of this bridge may be approximated as

$$\ddot{u}_{gl}(t) = \alpha_l \ddot{u}_g(t) \tag{1}$$

in which $\alpha_1 = \alpha_2 = 1; \alpha_3 = \alpha_4 = -1$ and $\ddot{u}_g(t)$ is the acceleration at a reference support selected to be at abutment 1.

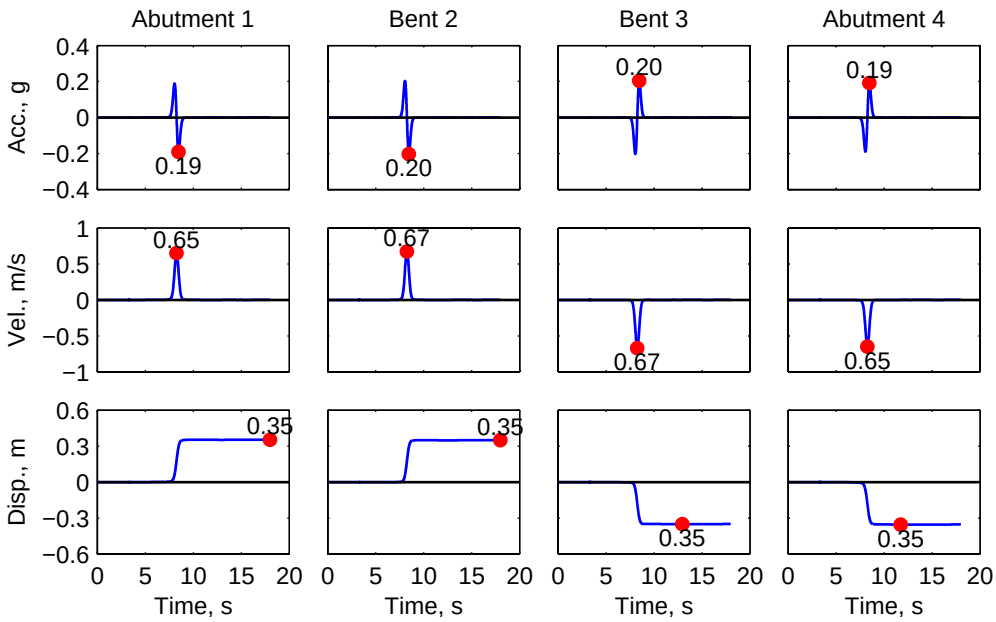

Figure 2. Motions at four supports of the selected bridge during a magnitude 6.5 event on a strike-slip fault.

THEORETICAL BACKGROUND: LINEAR SYSTEMS

Equations governing the motions of a structure subjected to multiple-support excitation are formulated by separating the displacement at the structure in two parts: (1) $\mathbf{u}^s$ the quasi-static displacement due to static application of the displacement $\mathbf{u}_g$ imposed at the supports, and (2) $\mathbf{u}$ the dynamic displacements. It can be shown that, for the excitation considered, $\mathbf{u}$ are governed by

$$\mathbf{m}\ddot{\mathbf{u}} + \mathbf{c}\dot{\mathbf{u}} + \mathbf{k}\mathbf{u} = -\mathbf{m}\boldsymbol{\iota}\ddot{u}_g(t) \tag{2}$$

in which $\mathbf{u}$ is a N×1 vector representing the dynamic displacement in the N superstructure degrees-of-freedom (DOF); $\mathbf{m}, \mathbf{c}, \mathbf{k}$ are N×N matrices representing the mass, damping, and stiffness corresponding to structural DOF; $\ddot{u}_g$ is the acceleration at a reference support; and $\boldsymbol{\iota}$, the "effective" influence vector, is a vector of displacements at all structural degrees of freedom due to simultaneous static application of unit displacement at all supports in the appropriate direction. The total response is then given by

$$\mathbf{u}^t(t) = \mathbf{u}^s(t) + \mathbf{u}(t) = \boldsymbol{\iota} u_g(t) + \sum_{n=1}^{N} \Gamma_n \boldsymbol{\phi}_n D_n(t) \tag{3}$$

in which $D_n(t)$ is the displacement response of an nth mode SDF system subjected to ground motion $\ddot{u}_g(t)$, $\boldsymbol{\phi}_n$ is the nth mode shape, and $\Gamma_n = \left(\boldsymbol{\phi}_n^T \mathbf{m}\boldsymbol{\iota} / \boldsymbol{\phi}_n^T \mathbf{m}\boldsymbol{\phi}_n\right)$ is the nth mode participation factor. Therefore, analysis of bridges crossing fault rupture zones becomes a two step process: (1) static analysis for all support displacements applied simultaneously to obtain the quasi-static response as a function of time; and (2) dynamic analysis to reference ground acceleration $\ddot{u}_g(t)$ using the "effective" influence vector.

However, peak values of the total displacements of linear elastic bridges may be estimated by adding peak absolute values of the quasi-static displacement, $\mathbf{u}_o^s$, and dynamic displacements, $\mathbf{u}_o$

$$\mathbf{u}_o^t \simeq \mathbf{u}_o^s + \mathbf{u}_o \,. \tag{4}$$

The peak absolute value of the dynamic displacements may be estimated by standard Response Spectrum Analysis (RSA) procedure using the response spectrum for the reference acceleration $\ddot{u}_g(t)$, which should include the effects of permanent offset resulting from fault rupture. Note that the standard ARS spectrum specified in Caltrans design guidelines is not appropriate.

It is useful to note that the dynamic part of the response of a bridge which is subjected to multi-support excitation resulting from fault rupture with permanent offset may be estimated by using standard RSA procedure developed for systems subjected to uniform ground motion. This simplification is achieved because the support motions conform to equation 1. However, the procedure presented in this paper differs from the standard procedures for uniform excitation because the "effective" influence vector must be used as opposed to $\boldsymbol{\iota} = \mathbf{1}$, where $\mathbf{1}$ is a vector with all elements equal to unity, used for uniform transverse support motions.

MODES EXCITED BY THE SELCTED SUPPORT MOTIONS

Typically, RSA of linear elastic bridges includes only those modes that contribute significantly to the response. In order to identify the significant modes, the *modal contribution factor* (MCF) defined by Chopra (2001: Section 12.10) is examined next for two seismic demands: (a) column drift in bent 2, and (2) deck displacement, relative to the ground, at abutment 1.

The first six mode shapes of the linear elastic bridge are shown in Figure 3. The 1st and 4th modes of the bridge are 1st and 2nd transverse modes. The 2nd and 6th modes of the bridge are 1st and 2nd torsional modes. The 3rd mode is coupled vertical-longitudinal and 5th mode is predominantly vertical mode.

The deflected shape of the bridge associated with the effective influence vector for the selected support motions is shown in Figure 4. This deflected shape exhibits significant torsional motion of the bridge indicating that torsional modes may be excited by the motions resulting from fault rupture between bent 2 and bent 3. This assertion is confirmed by the MCFs presented in Table 1 which indicate that the largest contribution to both deck displacement at abutment 1 and column drift is due to the 2nd mode, which is 1st torsional of the selected bridge (see Figure 3). Although 6th mode, which is the 2nd torsional mode of the bridge (see Figure 3), is also excited, its

contribution is relatively small. The MCFs for all other modes, including the 1st and 2nd transverse modes are zero indicating that these modes are not excited by the ground motion.

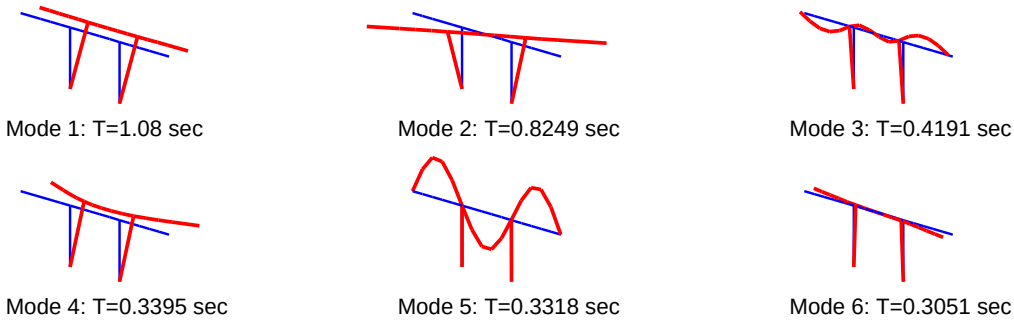

Figure 3. First six linear elastic mode shapes of the selected bridge.

The results presented so far indicate that transverse motions associated with the fault rupture between two bents of a symmetric bridge excite only the torsional modes of the bridge. Note that torsional modes would not be excited if the bridge is subjected to uniform transverse support motions; only transverse modes would be excited in the latter case.

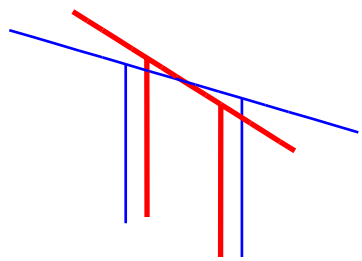

Figure 4. Deflected shape of the selected bridge associated with the effective influence vector corresponding to motions due to fault rupture between bent 2 and bent 3.

Table 1. Modal Contribution Factors for deck displacement at abutment 1 and column drift of the selected bridge and support motions.

Mode No.	**Period, T_n (sec)**	**Modal Contribution Factors**	
		Deck Disp. at Abutment	**Column Drift**
1	1.0800	0.000	0.000
2	0.8249	0.996	0.995
3	0.4191	0.000	0.000
4	0.3395	0.000	0.000
5	0.3318	0.000	0.000
6	0.3051	0.004	0.005
	Total	**1.0**	**1.0**

RELATIVE IMPORTANCE OF QUASI-STATIC AND DYNAMIC RESPONSE

As mentioned previously in equation 3, the total displacements of the linearly elastic bridge subjected to multiple-support motions consist of the quasi-static and dynamic displacements. The

contributions of these two displacements to the total displacements are examined next for linear elastic bridges. For this purpose, peak values of total seismic demands and their contributions from the quasi-static and dynamic parts, computed by linear RHA, are examined for a range of longitudinal steel ratio, which is defined as the longitudinal reinforcement in columns of the bridge as a percentage of the gross column cross section.

The results presented in Figure 5 indicate that the column drift in the selected bridge is dominated by the dynamic part, with the column drift almost entirely due to the dynamic part. The deck displacement at abutment 1, on the other hand, is dominated by the quasi-static part. However, its relative contribution increases with longitudinal reinforcement. These results also show that although the contribution of the dynamic part is small and decreases with longitudinal steel ratio, it can not be completely ignored in estimating the deck displacement at abutment. While the column drift for the selected can be computed by considering only the dynamic part, results for other bridges and fault locations, not shown here for brevity, indicate that quasi-static part may be significant for column drift and dynamic part may be comparable to the quasi-static part for deck displacement at abutment. Therefore, no generalization should be made about ignoring either quasi-static or dynamic parts in estimating seismic demands.

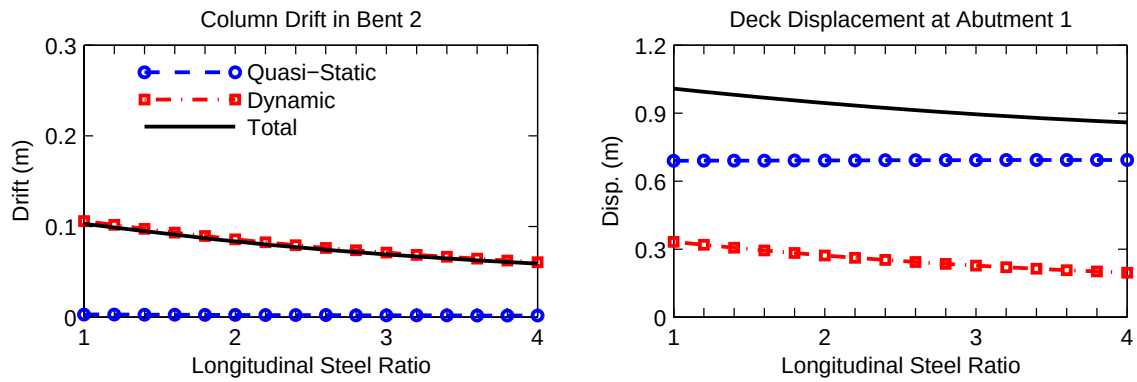

Figure 5. Relative contribution of quasi-static and dynamic parts to the total seismic demands.

SEISMIC DEMANDS IN NONLINEAR BRIDGES

Bridges subjected to motions resulting from fault rupture between their supports are expected to be deformed significantly beyond the linear elastic range. Therefore, response of the selected bridge responding in the nonlinear range is examined next. The response is computed by solving nonlinear equations of motion, not presented here for reason of brevity, for total displacements and the relative displacements, such as column drift and deck displacement considered in this investigation, are obtained by subtracting the ground displacements.

Since the bridge response can be significantly influenced by the transverse restraint provided by the shear keys at the abutments, three shear-key conditions are investigated: (1) bridges without shear keys; (2) bridges with elastic shear keys, and (3) bridges with nonlinear shear keys. The bridges without shear keys represent the current Caltrans design philosophy (CALTRANS, 2004): shear keys are designed as sacrificial elements to protect abutment walls and piles, implying that the shear keys should break-off before initiation of damage either in the piles or the abutment walls, during the Maximum Considered Earthquake (MCE). The bridges with elastic shear keys represent condition in which the shear key may be too strong and may not break-off

during the MCE. The bridges with nonlinear shear key consider the nonlinear behavior of external shear keys observed during recently conducted experiments (Bozorgzadeh et al., 2003).

For nonlinear shear keys, a simple tri-linear model (Figure 6) is used to "mimic" the experimental observations (Bozorgzadeh et al., 2003). The first yield is selected to occur at V_y = 1263 kN and the peak strength of the shear key (or break-off force) to occur at V_n = 1407 kN. The values of displacements at initiation of yielding and shear key break-off of 0.1 in (2.54 mm) and 1 inch (25.4 mm), respectively, observed during the experiments were modified to include the effects of pile flexibility which were not considered during the experiments. The displacement, u_m, at which the shear key breaks-off, i.e., it ceases to provide any resistance, is selected as 4 inch (101 .6 mm).

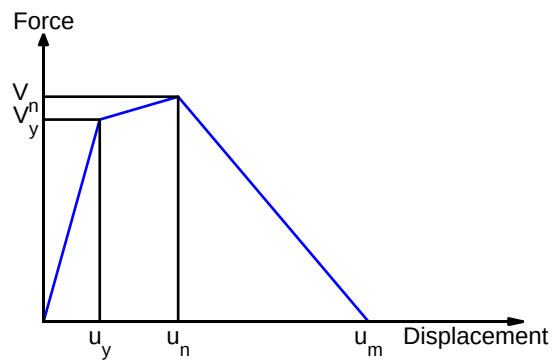

Figure 6. Nonlinear model for shear keys.

Figure 7 shows the time-histories of deck displacement at abutment 1 and column drift due to the selected motions obtained from nonlinear RHA. Also shown on the time history of nonlinear shear key case is the time instant when the shear key breaks-off. These results show that response of bridge with nonlinear shear keys can be significantly different compared to the bridge either without shear keys or with elastic shear keys. The response in bridge with nonlinear shear keys begins to diverge from the elastic shear keys case at about 3.5 sec, when the shear key begins to yield, with significant changes after the shear keys breaks-off. After the shear key breaks-off, the bridge begins to vibrate essentially in the same manner as the one without shear keys. The peak responses, however, are different because of different initial conditions at the instant of shear key break-off. Note that the response for the bridge with elastic shear keys does not exhibit oscillatory behavior because the vibration periods of modes that contribute significantly to the dynamic response are much shorter, about one-fifth, of the effective rise time of the ground displacement. However, the bridge without shear keys case and the elastic shear key case serve as the two bounding cases for the seismic demands in bridge with nonlinear shear keys: without shear keys and with elastic shear keys cases provide the upper and lower bounds, respectively, for deck displacement at abutment 1. But this trend reverses for column drift.

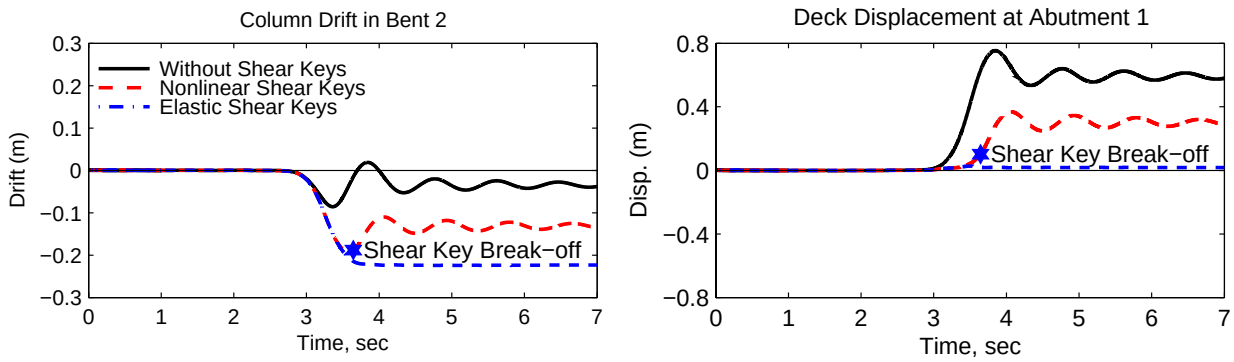

Figure 7. Time histories of column drift and deck displacement at abutment 1.

It is also useful to examine how the various seismic demands are affected by the shear key strength. For this purpose, the variations of seismic demands – column drift and deck displacement at abutment 1 – with the normalized shear key strength are examined next. The normalization of the shear key strength is with respect to the reference strength of the shear key defined previously. The normalized values of the shear key strength in the range of zero to two are considered. A value of normalized strength equal to zero leads to bridge without shear keys whereas normalized strength equal to one leads to bridge with the selected nonlinear shear keys. Much higher values of the normalized strength would lead to bridge with elastic shear keys.

Figure 8 presents the variation of peak seismic demands with the normalized shear keys strength. For reference purpose, the curves for peaks seismic demands in bridges without shear keys and with elastic shear keys are also included. These results show that peak values of the seismic demands depend significantly on the strength of the shear key. The seismic demands in the bridge with nonlinear shear keys approach those without shear keys for low values of normalized shear key strength, and approach those with elastic shear keys for high values of normalized shear key strength. Therefore, the cases without shear keys and with elastic shear keys provide bounding values on the response of bridges with nonlinear shear keys.

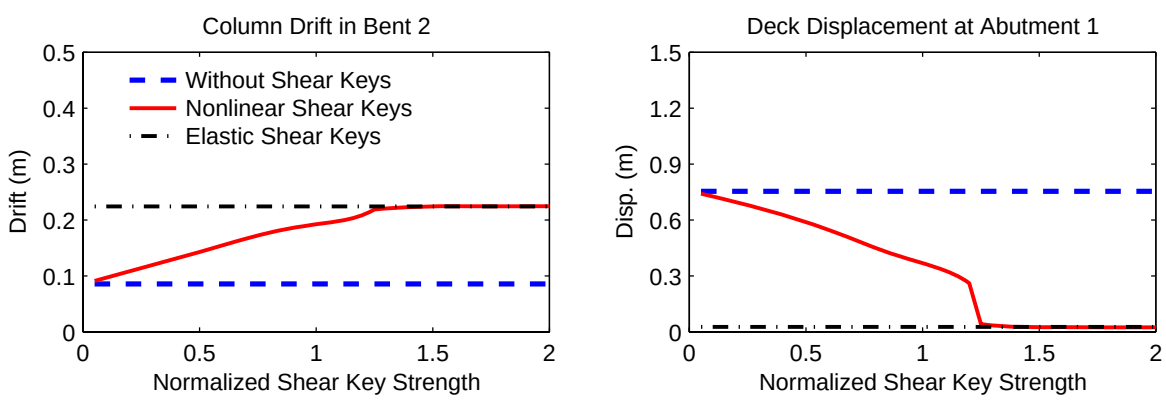

Figure 8. Variation of seismic demands with normalized shear key strength.

As mentioned previously, the Caltrans bridge design practice typically ignores the transverse restraint provided by the shear keys for the MCE ground motion. Therefore, seismic demands are estimated assuming the bridge has no shear keys. The results presented here clearly indicate that the case without shear keys may not be adequate in estimating upper bounds on all seismic demands. While the case without shear keys provides upper bound for deck displacement at

abutment 1, it did not provide the upper bound for column drift. The case with elastic shear keys provided upper bound estimate of column drifts. Therefore, both extreme cases – without shear keys and with elastic shear keys – must be considered to obtain estimates of upper bound of all seismic demands.

CONCLUSIONS

Presented in this paper is a procedure for analysis of bridges crossing strike-slip faults capable of producing motions with permanent offset in which seismic demands for linearly elastic bridges are computed by adding the results from: (1) static analysis for all support displacements applied simultaneously, and (2) dynamic analysis for reference ground acceleration using the "effective" influence vector. The dynamic displacements may be estimated by the standard RSA procedure using the "effective" influence vector and the response spectrum for the reference acceleration $\ddot{u}_g(t)$. However, response spectrum must include the effects the near-fault motions of Figure 2 with permanent offset resulting from fault rupture. Note that the standard ARS spectrum specified in Caltrans design guidelines is not appropriate for this case.

The dynamic response of a three-span symmetric bridge responding in the linearly elastic and nonlinear range computed for ground motions resulting from rupture on a strike-slip fault between its two bents has led to the following conclusions:

1. The selected support motions excite only the torsional modes in the symmetric bridge due to predominantly torsional nature of the "effective" influence vector. Note that only transverse modes are excited for symmetric bridge subjected to uniform transverse support motions.

2. No generalization should be made about ignoring either quasi-static or dynamic parts in estimating seismic demands because different parts may dominate different seismic demands or both parts may contribute significantly to seismic demands.

3. Response of bridges with nonlinear shear keys may be significantly different compared to the bridge either without shear keys or with elastic shear keys. However, the response of bridges with nonlinear shear keys is bounded by that of bridges without shear keys and with elastic shear keys provide.

4. To obtain an upper bound on seismic demands in bridges with nonlinear shear keys, both cases – without shear keys and with elastic shear keys – must be considered. The current Caltrans practice of ignoring shear keys may not always provide conservative estimates of all seismic demand.

While nonlinear RHA of bridges with nonlinear shear keys responding beyond the elastic limit provides the most "accurate" estimate of seismic demands, it may not always be suitable for design of "ordinary" bridges. Therefore, procedures simpler than the nonlinear RHA are desirable. The research is currently underway to develop simplified procedures that consider the nonlinear behavior of the bridge and provide improved estimates, compared to the current Caltrans practice, of seismic demands in bridges crossing fault rupture zones.

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KEYWORDS

Bridge Analysis, Fault Rupture, Fault Offset, Bridge Seismic Demand, Multi-Support Excitation