ANALYSIS OF ORDINARY BRIDGES CROSSING FAULT-RUPTURE ZONES

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ABSTRACT :

Rooted in structural dynamics, two procedures for estimating peak responses of linearly-elastic “ordinary” bridges crossing fault-rupture zones are presented: response spectrum analysis (RSA) procedure and a static analysis (SA) procedure. These procedures estimate the peak response by superposing peak values of quasi-static and dynamic responses. The peak quasi-static response in both procedures is computed by static analysis of the bridge with peak values of all support displacements applied simultaneously. In the RSA procedure, the peak dynamic response is estimated from the dynamic analysis including all significant modes, which is simplified in the SA procedure to static analysis of the bridge for appropriately selected forces. Both procedures are shown to provide estimates of peak response that are close to those from the “exact” response history analysis. Furthermore, it is shown that usually only one mode – the most dominant mode – is sufficient in the RSA procedure. These procedures utilize the “effective” influence vector that differs from that for the spatially-uniform excitation. Furthermore, the RSA procedure uses the response spectrum for the reference support excitation (including permanent offset of the ground due to fault rupture) that differs from the standard California Department of Transportation (CALTRANS) spectrum. The presented procedures idealize the spatially varying support excitation by *proportional multiple-support* (*PMS*) excitation in which motions at various supports of the bridge crossing a fault-rupture zone are proportional to the motion at a reference location.

KEYWORDS: Bridges, Dynamic Response, Earthquake Engineering, Ground Motion, Linear Analysis, Seismic Analysis

1. INTRODUCTION

Recent earthquakes have demonstrated the vulnerability of bridges that cross fault-rupture zones. Several bridges were seriously damaged as a result of rupture of causative faults in the 1999 Chi-Chi earthquake (EERI, 2001; Yen, 2002), 1999 Kocaeli earthquake (EERI, 2000), and 1999 Duzce earthquake (Ghasemi et al., 2000). While avoiding building bridges across faults may be the best practice, it may not always be possible to do so, especially in regions of high seismicity, such as California.

Bridges crossing fault-rupture zones will experience ground offset across the fault and hence spatially-varying ground motion. While site-specific seismological studies to define spatially-varying ground motions and rigorous response history analysis (RHA) are necessary for important bridges on “lifeline” routes, such investigations may be too onerous for “ordinary” bridges whose design is governed by the CALTRANS Seismic Design Criteria (SDC) (CALTRANS, 2006). “Ordinary” bridges are defined as normal weight concrete bridges with span lengths less than 90 m supported on the substructure by pin/rigid connections or conventional bearings. The bent caps of “ordinary” bridges terminate inside of the exterior girders, and their foundations consist of spread footings, piles, or pile shafts with underlying soil that is not susceptible to liquefaction, lateral spreading, or scour. For such structures, simplified procedures for estimating seismic demands are needed to facilitate their seismic evaluation and design. This paper presents simplified procedures – simpler than response history analysis (RHA) but rooted in structural dynamics theory – for estimating seismic demand of bridges crossing fault-rupture zones.

2. THEORETICAL BACKGROUND

A recently completed investigation by Goel and Chopra (2008) indicated that motions across various supports of a bridge in fault-rupture zone are essentially proportional to each other. Thus the displacement at support l is given by

$$u_{gl}(t) = \alpha_l u_g(t) \tag{2.1}$$

in which $u_g(t)$ is the displacement history of motion at a reference location, and α_l is the proportionality constant for the lth support. For such proportional multiple-support excitation, the total displacement of a linearly-elastic bridge at various structural degrees-of-freedom may be computed from

$$\mathbf{u}^t(t) = \mathbf{u}^s(t) + \mathbf{u}(t) = \boldsymbol{\iota}_{\text{eff}} u_g(t) + \sum_{n=1}^{N} \Gamma_n \boldsymbol{\phi}_n D_n(t) \tag{2.2}$$

in which $\mathbf{u}^s$ is the quasi-static response, $\mathbf{u}$ is the dynamic response, $\boldsymbol{\iota}_{\text{eff}}$ is the "effective" influence vector defined as the vector of displacements at all structural degrees of freedom due to simultaneous static application of all support displacements with value equal to α_l at the lth support, $\Gamma_n = \boldsymbol{\phi}_n^T \mathbf{m} \boldsymbol{\iota}_{\text{eff}} / \boldsymbol{\phi}_n^T \mathbf{m} \boldsymbol{\phi}_n$, $\mathbf{m}$ is the mass matrix, $\boldsymbol{\phi}_n$ is the nth mode shape of the bridge, and $D_n(t)$ is the deformation response of the nth-mode single-degree-of-freedom (SDF) system subjected to the reference ground motion $\ddot{u}_g(t)$ governed by

$$\ddot{D}_n + 2\zeta_n \omega_n \dot{D}_n + \omega_n^2 D_n = -\ddot{u}_g(t) \tag{2.3}$$

where ω_n and ζ_n are the frequency and damping ratio, respectively, of the nth-mode SDF system.

Equation (2.2) implies that total response of a linearly-elastic bridge may be computed by superposition of the quasi-static response, $\mathbf{u}^s$, and the dynamic response, $\mathbf{u}$. For linearly-elastic systems, it has been demonstrated (Goel and Chopra, 2008) that the peak values of the total responses, $\mathbf{u}_o^t$ and r_o^t, of bridges in fault-rupture zones may be estimated to a useful degree of accuracy by adding peak values of the quasi-static response, $\mathbf{u}_o^s$ and r_o^s, and dynamic response, $\mathbf{u}_o$ and r_o:

$$\mathbf{u}_o^t \simeq \mathbf{u}_o^s + \mathbf{u}_o \qquad r_o^t \simeq r_o^s + r_o \tag{2.4}$$

Such superposition of peak quasi-static and dynamic responses was found to be reasonable because for motions in fault-rupture zones, the peak value of the dynamic part of the response generally occurs during time phase after the quasi-static part of the response reaches, and maintains its peak value (Goel and Chopra, 2008).

The peak value of the quasi-static response, r_o^s, is due to static application of the peak values of ground displacements, $\alpha_l u_{go}$, simultaneously at all supports where u_{go} is the peak value of the ground displacement at the reference support. Presented next are two procedures to determine the peak value of dynamic response: response spectrum analysis and static analysis.

2.1. Response Spectrum Analysis (RSA) Procedure

The peak dynamic response is estimated by using SRSS or CQC rule, as appropriate, to combine the peak modal responses. Although not rigorously valid for ground motions in close proximity to the causative fault, these modal combinations will be used and their accuracy evaluated. The peak value of the dynamic response, r_o, is computed by implementing the following steps:

1. Compute the vibration periods, T_n, and mode shapes, ϕ_n, of the bridge.
2. Identify the significant modes that need to be considered in the dynamic analysis based on the modal contribution factors as follows:
   - 2.1. Compute the "effective" influence vector, $\boldsymbol{\iota}_{\text{eff}}$, the vector of displacements in the structural DOF obtained by static analysis of the bridge due to support displacements α_l, applied simultaneously in the appropriate direction: fault parallel or fault normal.
   - 2.2. Compute the static response, r^{st}, by static analysis of the bridge due to forces $\mathbf{m}\boldsymbol{\iota}_{\text{eff}}$ applied at the structural DOF.
   - 2.3. Compute the modal static response, r_n^{st}, from static analysis of the bridge due to forces $\mathbf{s}_n = \Gamma_n \mathbf{m} \boldsymbol{\phi}_n$ applied at the structural DOF, where $\Gamma_n = \boldsymbol{\phi}_n^T \mathbf{m} \boldsymbol{\iota}_{\text{eff}} \big/ \boldsymbol{\phi}_n^T \mathbf{m} \boldsymbol{\phi}_n$.
   - 2.4. Compute the modal contribution factor for the *n*th mode, $\bar{r}_n = r_n^{\text{st}} / r^{\text{st}}$ (Chopra 2007: Section 12:10).
   - 2.5. Repeat Steps 2.3 and 2.4 for all modes.
   - 2.6. Select the number of significant modes, *J*, such that the error in the static value of response quantity *r*, $e_J = 1 - \sum_{n=1}^{J} \bar{r}_n$, is less than acceptable value, e.g., 0.05.
3. Compute the peak value of the *n*th mode dynamic response, $r_{no} = r_n^{\text{st}} A_n$ in which r_n^{st} is the modal static response (Step 2.3) and A_n is the ordinate of the pseudo-acceleration spectrum for the reference support acceleration $\ddot{u}_g(t)$ corresponding to the *n*th-mode SDF system.
4. Repeat Step 3 for all significant modes identified in Step 2.
5. Combine the peak modal response by SRSS or CQC modal combination rule, as appropriate, to obtain the peak dynamic response, r_o.

2.2 Static Analysis Procedure

The static analysis procedure is developed by recognizing that: (1) In many cases, the individual modal responses in the RSA procedure tend to attain their peak values at essentially the same time (Goel and Chopra, 2008) indicating that the algebraic sum of the peak responses (instead of CQC or SRSS combinations) should provide a reasonable estimate of the peak value of the combined response; and (2) for most bridges, the value of A may be conservatively approximated by $A \simeq 2.5\ddot{u}_{go}$ and the peak dynamic response may be conservatively estimated by static analysis of the bridge due to lateral forces $= 2.5\mathbf{m}\boldsymbol{\iota}_{\text{eff}}\ddot{u}_{go}$. Thus, the peak value of the dynamic response, r_o, in the static analysis procedure is computed by implementing the following steps:

1. Compute the effective influence vector, $\boldsymbol{\iota}_{\text{eff}}$, as the vector of displacements in the structural DOF obtained by static analysis of the bridge due to support displacements α_l, applied simultaneously.
2. Estimate the dynamic response r_o by static analysis of the bridge due to lateral forces $= 2.5\,\mathbf{m}\boldsymbol{\iota}_{\text{eff}}\ddot{u}_{go}$.

3. STRUCTURAL SYSTEMS, MODLEING, AND RESPONSE QUANTITIES

The structural systems considered in this investigation were as follows: (1) a three-span symmetric bridge (Figure 1a); (2) a three-span unsymmetric bridge (Figure 1b); (3) a four-span symmetric bridge (Figure 1c); and a four-span unsymmetric bridge (Figure 1d). These bridges are supported on abutments at the two ends and intermediate single-column bents. The span lengths and bent heights are shown in Figure 1. The bases of columns in the bents were fixed (restraint for all six degrees-of-freedom). The deck, which is expected to accommodate two traffic lanes, was selected as a multi-cell box girder, and columns selected were circular sections with helical transverse (or hoop) steel and longitudinal steel arranged at their periphery. The area of longitudinal steel was selected as 3% of the gross columns area and hoop steel was selected as 1% of the column volume to represent well-confined columns. The fault was assumed to be located between bents 2 and 3 of the selected bridges (Figure 1). The selected bridge systems were analyzed using the structural analysis software Open System for Earthquakes Engineering Simulation (OpenSees) (McKenna and Fenves, 2001). The modeling details are available in Goel and Chopra (2008).

The four selected bridges (three-span symmetric, three-span unsymmetric, four-span symmetric, and four-span unsymmetric) and two shear-key cases (without shear keys and with elastic shear keys) provide a total of eight bridge configurations: (1) Three-span symmetric bridge without shear keys; (2) Three-span symmetric bridge with elastic shear keys; (3) Three-span unsymmetric bridge without shear keys; (4) Three-span unsymmetric bridge with elastic shear keys; (5) Four-span symmetric bridge without shear keys; (6) Four-span symmetric bridge with elastic shear keys; (7) Four-span unsymmetric bridge without shear keys; and (8) Four-span unsymmetric bridge with elastic shear keys. Seismic responses of these eight bridges, referred to as Bridges 1 to 8, are examined in this paper.

The structural systems considered in this investigation do not necessarily represent "actual" bridges. They are selected to demonstrate the accuracy of the presented procedures for varying parametric conditions: number of spans (three-span versus four-span bridges); asymmetry in bridge geometry (symmetric versus asymmetric bridges); and shear-key condition (elastic shear keys versus no shear keys). It is expected that if the presented procedures provide reasonable estimates of seismic responses for a range of structural systems considered in this investigation, they would provide accurate results for actual "ordinary" bridges as well.

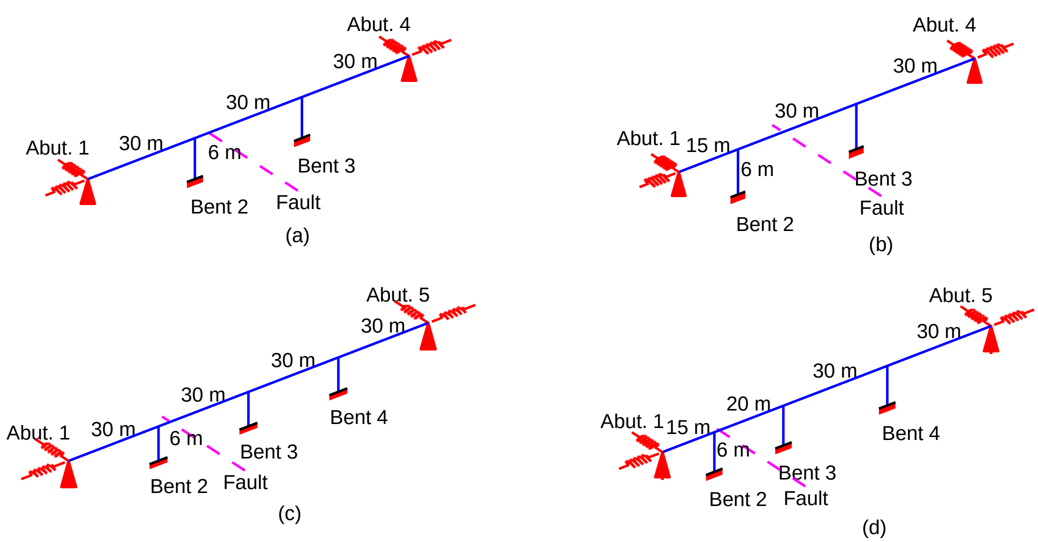

Figure 1. Bridges considered: (a) three-span symmetric bridge, (b) three-span unsymmetric bridge; (c) four-span symmetric bridge; (4) four-span unsymmetric bridge.

The response quantities considered in this investigation are the column drift and deck displacement at the abutment. The column drift, which indicates deformation demand in the column, is defined as the displacement at top of the column relative to its base displacement. The deck displacement at the abutment, which is used to estimate the relative displacement of the deck from the abutment, is defined as the displacement of the deck at the abutment relative to the displacement at the top of the abutment.

4. GROUND MOTIONS

Ground motions are needed at bridge supports in close proximity to the fault (Figure 1). Unfortunately, to date, ground motions have never been recorded at such fine spacing in close proximity to the causative fault. For this investigation, motions were simulated at stations spaced 15 m apart (Figure 2) due to a magnitude 6.5 earthquake in the fault-normal, fault-parallel, and vertical directions across a fault rupture zone (Dreger et al. 2007). Resulting from this simulation, the fault-parallel component of ground acceleration and displacement at stations 1-6 on a vertical strike-slip fault (fault with dip = 90°, rake = 180°) are presented in Figure 3. Motions on other types of faults and in other directions are not presented here for brevity; they may be found elsewhere (Goel and Chopra, 2008; Dreger et al. 2007). These simulated motions are considered for analyzing the selected bridges.

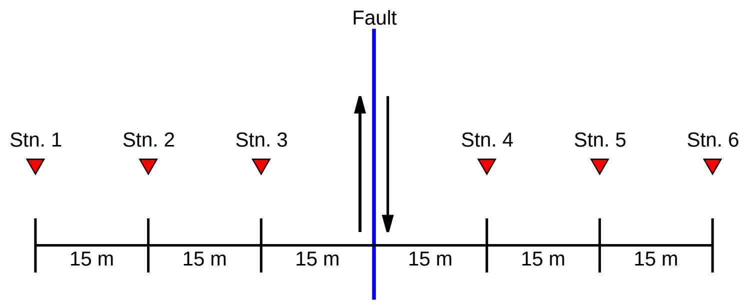

Figure 2. Location of stations across the fault where spatially-varying ground motions were simulated.

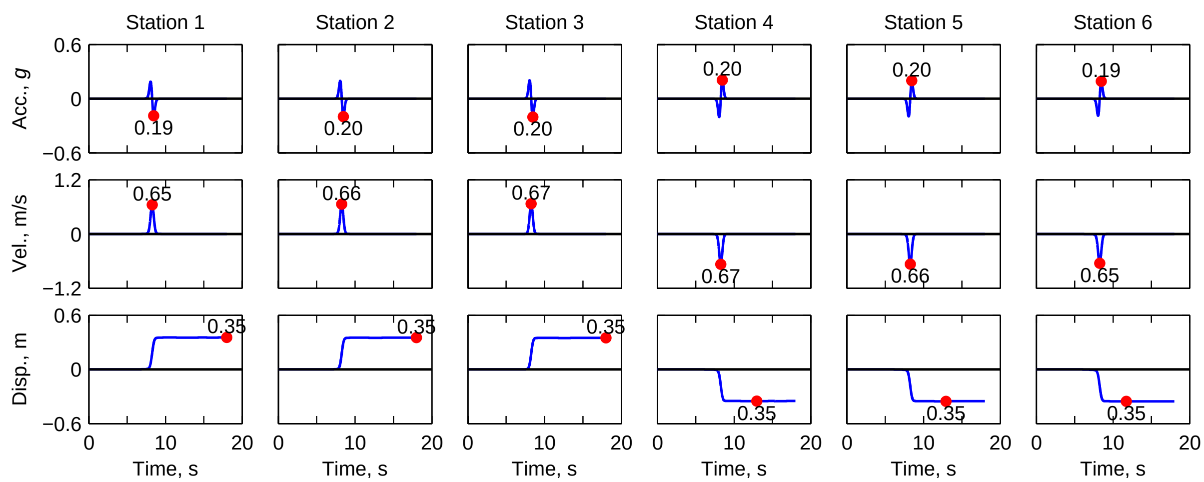

Figure 3. Displacement and acceleration in fault-parallel direction at six stations across a vertical strike-slip fault during magnitude 6.5 earthquake.

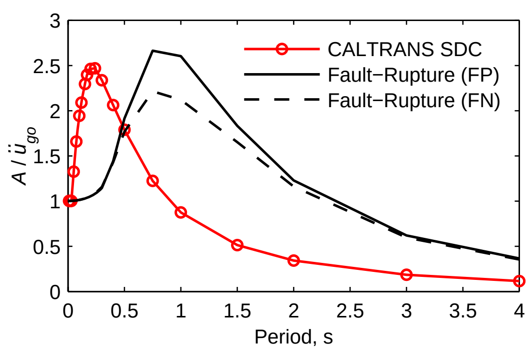

Figure 4. Comparison of normalized 5%-damped elastic response spectrum for ground motions in fault-rupture zone with the CALTRANS SDC spectrum.

Figure 4 shows the pseudo-acceleration response spectrum for the fault-parallel and fault-normal components of ground motion, with permanent offset, in very close proximity (say, roughly, 15 meters) to the causative fault. The pseudo-acceleration scale has been normalized by the peak ground acceleration ($A/\ddot{u}_{go}$). Also included is the CALTRANS SDC spectrum for peak ground acceleration (PGA) of 0.4g, soil type B, and earthquake magnitude

6.5±0.25. Although CALTRANS SDC provides spectrum for PGA values in the range of 0.1g to 0.6g for each earthquake magnitude and soil type, only one spectrum for 0.4g PGA is included here for clarity. Figure 4 shows that the spectrum for motions in the fault-rupture zones differs considerable from the CALTRANS SDC spectrum. Therefore, the CALTRANS SDC spectrum is inappropriate for analysis of bridges crossing fault-rupture zones.

5. SIGNIFICANT VIBRATION MODES

The modes of a bridge that are excited by motions in the fault-rupture zone may differ significantly from those excited by the spatially uniform ground motion. For this purpose, consider the first six mode shapes of a three-span symmetric bridge (Figure 5). Also consider the deflected shape of the bridge associated with the effective influence vector for multiple-support excitation in fault-rupture zones and spatially-uniform excitation for bridges on one side of the fault (Figure 6). The effective influence vectors exhibit significant torsional motion about the vertical axis for a bridge across a fault (Figure 6a), in contrast to the translational motion of the bridge on one side of the fault (Figure 6b). Therefore, predominantly torsional modes are excited for bridges crossing fault-rupture zones whereas predominantly transverse modes are excited in bridges subjected to spatially uniform excitation. A comprehensive discussion on this subject is available in Goel and Chopra (2008).

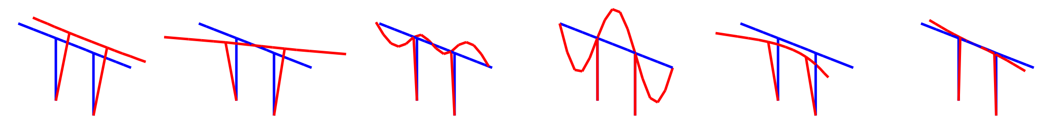

Figure 5. Vibration periods and mode shapes of a three-span symmetric bridge without shear keys.

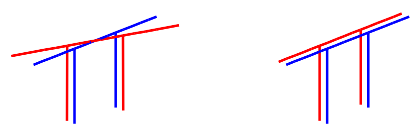

Figure 6. Deflected shapes associated with the "effective" influence vector of a three-span symmetric bridge: (a) excitation in fault-parallel direction; and (b) spatially-uniform excitation.

6. ACCURACY OF PROPOSED PROCEDURES

The procedures presented to estimate the peak response are based on two approximations: (1) superposing the peak values of quasi-static and dynamic responses; and (2) estimating the peak dynamic response by the RSA or the static analysis procedure. In this section, the combined errors due to both approximations are investigated by comparing the peak values of the responses determined by approximate procedures and by RHA, the "exact" procedure. For this purpose, presented are the transverse responses of bridges due to fault-parallel motions resulting from a rupture on a vertical strike-slip fault (Figure 7). Also included are results from the RSA procedure considering contribution of only the dominant mode, the mode with the largest modal contribution factor; these results are denoted as RSA:1-Mode.

The presented results show that both versions of RSA lead to estimates of seismic demands that are very close to those from the "exact" RHA procedure, indicating that the most-dominant mode contributes essentially all of the dynamic response of the selected systems.

The presented results also show that the static analysis procedure that avoids dynamic analysis also provides a reasonably good estimate of the seismic demand, which is slightly conservative in most cases. Such an overestimation is expected because the simplified procedure is based on an upper bound estimate of the pseudo-acceleration = $2.5\ddot{u}_{go}$. However, it underestimates the seismic demand slightly in a few cases, e.g., bent 2 drift for bridges 3 and 7 (Figure 7b) because these bridges have two nearly most-dominant modes and contribution of these two modes to some seismic demands tend to cancel out.

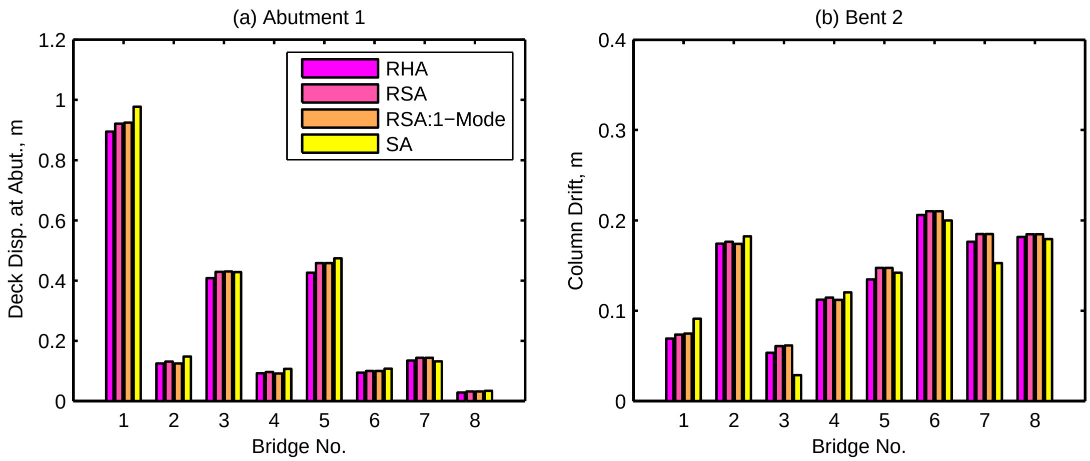

Figure 7. Comparison of seismic demands determined by three proposed procedures – RSA, RSA:1-Mode, and static analysis (SA) – with those from the "exact" RHA procedure: (a) deck displacement at abutment 1; and (b) column drift in bent 2. Results are for fault-parallel ground motions associated with a vertical strike-slip fault.

7. CONCLUSIONS

This investigation has led to development of two procedures – response spectrum analysis (RSA) procedure and static analysis procedure – for estimating peak responses of linearly-elastic "ordinary" bridges crossing fault-rupture zones. Although much simpler than response history analysis, these procedures provide estimates of peak seismic responses that are sufficiently "accurate" for most practical application.

In the presented procedures, the peak value of seismic response of the bridge is computed by superposition of peak values of quasi-static and dynamic parts of the response. The peak quasi-static response is computed by static analysis of the bridge with peak values of all support displacements applied simultaneously. Two procedures are presented for estimating the peak dynamic response. In the RSA procedure it is estimated directly from the response spectrum including all significant modes in the dynamic analysis. The static analysis procedure avoids computing the vibration periods of the bridge, and estimates the peak dynamic response by a much simpler static analysis of the bridge to appropriately selected forces. These procedures utilize the "effective" influence vector that differs from that for the spatially uniform excitation. Furthermore, the RSA procedure uses the response spectrum for ground motions expected in close proximity to the causative fault.

The natural vibration modes that are excited in bridges subjected to motions resulting from rupture on a fault passing under the bridge differ entirely from those excited in bridges on one side of the fault. Therefore, it is important to correctly identify the modes that need to be considered in the RSA procedure. For this purpose, the modal contribution factor concept is demonstrated to be useful.

It is recognized that bridges subjected to motions resulting from rupture of a fault passing under the bridge are unlikely to remain within the linear elastic range. Therefore, the research reported in this paper must be viewed as the basis for development of a procedure to estimate seismic demands in bridges responding beyond the linearly elastic range, a subject addressed elsewhere (Goel and Chopra, 2008).

8. ACKNOWLEDGMENT

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