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AMPLIFICATION OF BASE SHEAR WITH VERTICAL SHAKING INTENSITY IN BRIDGES WITH SPHERICAL SLIDING BEARINGS

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Abstract

Seismic isolation is an effective way to mitigate the effects of an earthquake on a structure by lengthening the natural period of the system and reducing the seismic force, or base shear demand. In structures isolated with spherical sliding bearings, the structure rests on an articulated slider that moves on a spherical sliding surface during earthquake excitation. Movement of the bearing generates a lateral resisting force that is proportional to the normal or axial force (determined by the weight of the structure above) through the principles of friction. When the structure is subjected to vertical ground shaking alongside lateral ground shaking, the axial force on the bearing varies according to the vertical ground acceleration. As a result, the seismic base shear increases when the vertical component of ground shaking is explicitly considered. This effect was experimentally observed in a large scale shake table test of a 5-story building with triple friction pendulum bearings, conducted at E-Defense in 2011, and subsequently validated through computational simulations.

The authors have systematically investigated the effect of vertical shaking in highway bridges isolated with triple pendulum bearings. Based on a simplified theory, an approximate methodology to predict the amplification of base shear with vertical ground shaking intensity in such bridges has been developed. Compared to nonlinear response history analysis, the approximate method is accurate for low to moderate intensity vertical motions, but becomes unconservative for higher intensity vertical motions. As the vertical shaking intensity approaches 1g, dynamic effects may cause uplift and subsequent impact of the isolation bearings that are not accounted for in the simplified methodology.

In this study, statistical trends for amplified base shear as a function of ground motion intensity are evaluated. Seismically-isolated highway bridge models are subjected to nonlinear response history analysis under a suite of ground motions, whereby the intensity of horizontal shaking is held constant while the vertical shaking is increased. The objective is to identify the threshold intensity of vertical shaking such that the relation between vertical ground intensity and base shear amplification factor becomes nonlinear, and the simplified method to predict amplified base shear is no longer reliable.

The simulated and estimated base shear coefficients as a function of vertical shaking intensity PGA_V for 11 ground motions were compared in model bridges with varying isolation system parameters. Although the simplified method was conservative in some motions over the entire range of PGA_V, in a number of motions – as anticipated – the base shear coefficient varied nonlinearly with PGA_V at higher intensities, and the simplified method was no longer reliable. As a preliminary recommendation, the simplified method should be applied only for PGA_V up to 1g. For vertical ground motions with PGA_V exceeding 1g, 3D response history analysis should be used to accurately predict the base shear coefficient for design.

Keywords: seismic isolation; spherical sliding bearings; friction pendulum system; vertical ground shaking

1. Introduction

Seismic isolation is an effective way to mitigate the effects of an earthquake on a structure by lengthening the natural period of the system and reducing the seismic force, or base shear demand. In structures isolated with spherical sliding bearings, the structure rests on an articulated slider that moves on a spherical sliding surface during earthquake excitation. Movement of the bearing generates a lateral resisting force that is proportional to the normal or axial force (determined by the weight of the structure above) through the principles of friction. When the structure is subjected to vertical ground shaking alongside lateral ground shaking, the axial force on the bearing varies according to the vertical ground acceleration. As a result, the seismic base shear increases when the vertical component of ground shaking is explicitly considered.

The amplification of base shear due to vertical ground shaking has typically been neglected in design of structures isolated with spherical sliding bearings, and until recently was not well understood. Most experimental studies have not concluded any significant effect of vertical motion on horizontal base shear or other lateral responses [1-5]. However, amplification of base shear due to vertical shaking was observed in a large scale shake table test of a 5-story building with triple pendulum bearings (TPBs), conducted at E-Defense in 2011 [6-7]. A variety of ground motions with different vertical intensity were imposed, and the vertical shaking was observed to introduce a high frequency component to the base shear that was proportional to the vertical ground acceleration, which also caused an amplification of the higher modes. The observed effects were subsequently validated through computational simulations [7]. A few other computational studies have supported the claim that vertical acceleration in spherical sliding bearings and other friction bearings amplifies the horizontal base shear [8-10] and excites higher modes [11-12].

These effects are anticipated to have direct significance in the design of isolated structures, where the amplified base shear in the isolation devices will increase the demands to other structural components. Design methods or simplified methods that can account for amplification of seismic base shear due to vertical shaking are needed. One study [13] performed a statistical evaluation to broadly quantify the effects of vertical shaking on the isolation system base shear using a rigid block on single pendulum bearing model, but the dependence of the base shear amplification on vertical shaking intensity was not directly considered. The authors [14] have investigated the effect of vertical shaking in highway bridges isolated with TPBs, and developed an approximate method to estimate the amplified base shear with vertical ground shaking intensity. The method has not yet been fully vetted for a systematic and broad variation of peak vertical acceleration. In particular, the method is suspected to become uniformly unconservative at some level of high intensity vertical shaking due to the possibility of uplift and subsequent impact of isolation bearings. The objective of this paper is to identify the limits of application of the simplified method through a systematic variation of vertical ground shaking intensity.

2. Ground Motion Selection and Scaling

This study was conducted using a suite of ground motions with components in two horizontal directions and vertical. The shaking intensity, or scale factor, was fixed in the horizontal direction and varied in the vertical direction. The selected suite consisted of 11 recorded ground motions corresponding to shallow crustal earthquakes in active tectonic regions that incorporate near-fault effects [15]. The motions were amplitude scaled to match a 5%-damped target acceleration response spectrum that was developed from the 2008 NGA ground-motion prediction equations [e.g. 16-18] assuming a moment magnitude of 6.7 and a reverse-fault mechanism. Prospective motions were rotated to the direction that maximized the spectral acceleration at a period T = 1 sec, and amplitude scaled to minimize the sum-square-of-the-error relative to the target spectrum. Using the code SigmaSpectra [19], individual motions were selected such that the root mean square error of the suite of motions relative to the target spectrum was minimized [15]. After the suite of motions was selected, scale factors for individual motions were further increased or decreased while keeping the average scaling factors constant, so that the suite as a whole also matched the target standard deviation. The best scaling factor for each record were determined by the Centroid Method as illustrated in [20].

Since vertical motions were not considered in development of the ground motion suite [15], a similar procedure was adopted herein to scale the vertical motions. First, the target vertical spectral acceleration $S_{a,V}$ was developed based on NEHRP recommended seismic provisions [21], and then modified to better match the shape of the target horizontal spectrum. The vertical components were amplitude scaled to best fit the target vertical spectrum, and then scale factors for individual scale factors were similarly increased or decreased to represent the standard deviation.

Table 1 lists the ground motions selected by this method and their relevant statistics, including the final scaled peak ground acceleration (PGA) in each direction: transverse PGA_T, longitudinal PGA_L, and vertical PGA_V. The ground motions have been divided into three groups based on PGA_V intensity ranges: Group 1 = 0.8g and above (High Intensity), Group 2 = 0.5g to 0.7g (Moderate Intensity), and Group 3 = 0.2g to 0.4g (Low Intensity). Figs. 1(a) and 1(b) show acceleration spectra for individual scaled motions plotted against the target spectrum for horizontal and vertical components, respectively. In each plot, the Group 1, Group 2 and Group 3 spectra are identified by color. By inspection, the spectral intensities of the vertical motions (per group) are roughly proportional to those of the horizontal motions. Furthermore, Group 2, Group 1 and Group 3 motions correlate well to target median μ, median plus one standard deviation ($\mu + \sigma$) and minus one standard deviation ($\mu - \sigma$) horizontal target spectra, respectively (Fig. 1(a)). Further details about the ground motion selection and scaling procedure are provided in [14]. For some analyses reported here, vertical ground motion intensity PGA_V is varied widely from 0 to 1.5g.

Table 1 - Selected ground motions and relevant characteristics

Group #	**NGA #**	**Code**	**Event**	**Station**	**Scaled PGA**		
					PGA_V (g)	**PGA_T (g)**	**PGA_L (g)**
Group 1	77	SFPU	1971 San Fernando, CA	Pacoima Dam (upper left)	0.817	0.843	0.856
	825	CAM	1992 Cape Mendocino, CA	Cape Mendocino	1.10	1.116	0.776
	1051	NPA	1994 Northridge, CA	Pacoima Dam (upper left)	1.683	1.678	1.36
Group 2	763	LPG	1989 Loma Prieta, CA	Gilroy - Gavilan Coll.	0.62	0.47	0.43
	879	LAL	1992 Landers, CA	Lucerne	0.614	0.73	0.793
	1050	NPD	1994 Northridge, CA	Pacoima Dam (downstr)	0.597	0.544	0.568
	3473	CT78	1999 Chi-Chi-06, Taiwan	TCU078	0.694	0.336	0.49
	292	IIS	1980 Iprinia, Italy	Sturno	0.543	0.27	0.382
Group 3	285	IIB	1980 Iprinia, Italy	Bagnoli Irpinio	0.38	0.266	0.39
	1148	KCL	1999 Kocaeli, Turkey	Arcelik	0.272	0.424	0.271
	1486	CT46	1999 Chi-Chi, Taiwan	TCU046	0.262	0.28	0.234

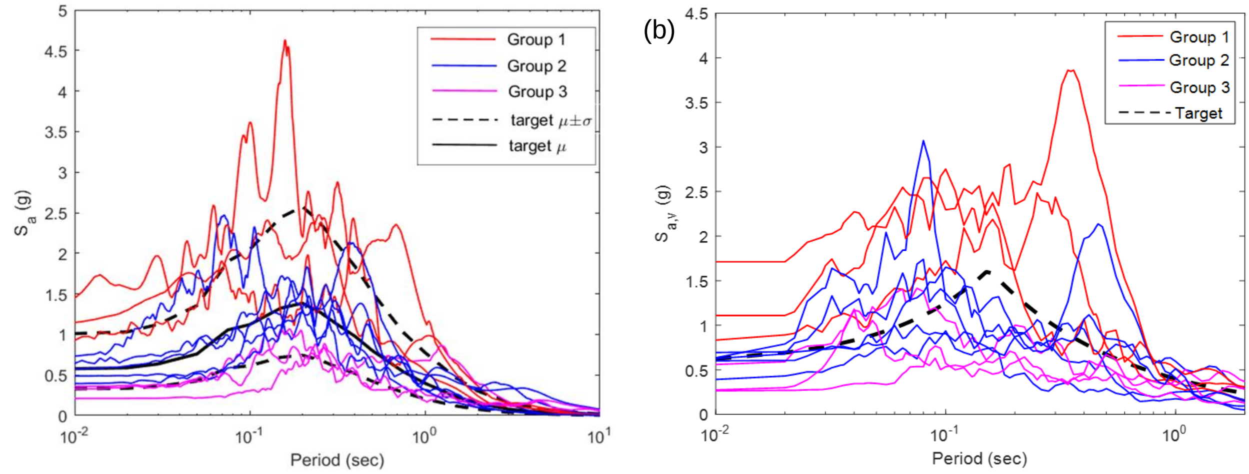

Fig. 1 – Spectral acceleration of scaled motions relative to the target spectra: (a) rotated horizontal component, (b) vertical component

3. Bridge Parameter Selection and Modeling

3.1 Description of Bridge Considered

The bridge analyzed in this study is a multi-span continuous concrete box girder bridge, representative of commonly occurring bridges and a preferred bridge archetype for use with seismic isolation in the state of California. Mojidra and Ryan [14] showed that the trend of base shear amplification due to vertical shaking was relatively insensitive to superstructure and substructure parameter variations, justifying the selection of a single bridge model for the study conducted herein.

The bridge is a three-span bridge with two-column bents, and isolated with TPBs. The bridge dimensions and details are as depicted in Fig. 2. The intermediate span length is 120 ft and the approach spans are 100 ft. The 5 ft deep bent cap is set on 5 ft diameter columns, which have clear height of 20 ft and center-to-center spacing of 25 ft. The superstructure is a 45 ft wide and 4.8 ft deep three-cell box girder, with 8.875 in. deck thickness, 7 in. soffit thickness, 12 in. wall thickness, and 11.75 ft center-to-center spacing between walls. The bridge is assumed to be isolated with TPBs at the column tops and at the abutments.

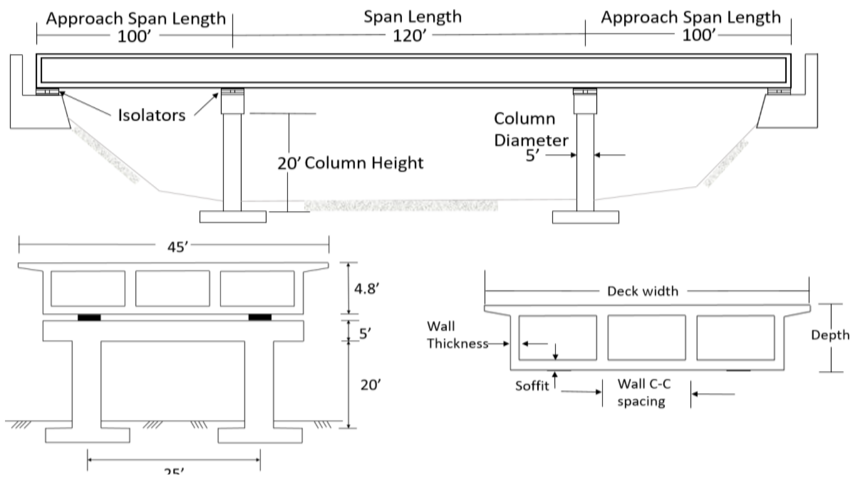

Fig. 2 – Considered bridge dimensions and details

Isolation system parameter variations were applied to the bridge to investigate the influence of friction coefficient and isolation period on bridge response to combined horizontal and vertical ground motions. Table 2 lists the isolator parameter variations considered, where the symbols μ_2 and T_2 correspond to friction coefficient and pendulum period associated with sliding on the outer sliding surfaces. The TPB model is described more fully in Section 3.2.

Table 2 – Isolation parameter variation applied to the bridge model

Iso System No.	μ_2	T_2 (sec)
1	0.04	2
2	0.06	2
3	0.08	2
4	0.04	3.5
5	0.06	3.5
6	0.08	3.5
7	0.04	5
8	0.06	5
9	0.08	5

3.2 Bridge Modeling Details

A three-dimensional computational model of the isolated bridge was developed in OpenSees using a spine model approach (Fig. 3). The superstructure was was modeled using elastic beam–column elements with uncracked section properties for prestressed concrete, and the contribution of steel was neglected. The superstructure was divided into multiple elements per span, and tributary mass – computed from unit weight and volume of concrete – was lumped at the nodes. Superstructure nodes were connected to isolators with rigid links. Cap–beam elements were also connected to the base of the isolators and the column top nodes via rigid links. Cap beams and columns were modeled as elastic frame elements, and columns were fixed at the base. Abutments were modeled such that isolators can displace freely at abutment ends without restriction.

The bridge in this study was isolated with TPBs, although the observations regarding the influence of vertical shaking are expected to apply generally to any spherical sliding bearing. Fig. 4 presents a general section view of a TPB, which consists of an inner slider, two articulated sliders, and two main concave surfaces. The radii of these curved surfaces R_1, R_2, R_3, and the friction coefficients of the sliding interfaces, μ_1, μ_2, μ_3, determine the hysteretic response of the TPBs. Parameters d_1, d_2, and d_3 represent displacement limits of the pendulum mechanisms, and h_1, h_2, and h_3 are slider heights. Different pendulum mechanisms, determined by sliding on one or more of the sliding surfaces, are engaged in the TPB under different levels of shaking intensity. A general normalized backbone curve, or force vs. displacement of the TPB, is presented in Fig. 5(a). Each segment of the backbone curve represents a different stage of sliding; these stages are well-documented in [22-23]. Also note that, f_1^* is the horizontal force that is normalized by the instantaneous acting axial force, not the constant weight of the superstructure. The effective radii of the spherical surfaces, which determine the normalized stiffnesses on the backbone curve, are: $L_1 = R_1 - h_1$; $L_2 = R_2 - h_2$; $L_3 = R_3 - h_3$.

A representative normalized backbone curve for one of the isolation system parameter variations used in this study (Iso System No. 6 in Table 2) is shown in Fig. 5(b). The parameters were selected in such a way that sliding stages 2 through 4 collapse into a single stage. Specifically, the design parameters of the upper and lower concave plates were selected such that effective lengths $L_2 = L_3$, friction coefficients $\mu_2 = \mu_3$, and displacement limits $d_2 = d_3$. Based on these assumptions, the normalized stiffnesses for the first and section stage of sliding are: $k_1 = 1/(2L_1)$, and $k_2 = 1/(2L_2)$. The periods associated with individual sliding stages on the normalized force-displacement curve are defined by:

$$T_i = 2\pi\sqrt{\frac{1}{k_i g}} = 2\pi\sqrt{\frac{2L_i}{g}} \tag{1}$$

where k_i is the normalized stiffness = $1/(2L_i)$, and g is the gravitational constant. For the normalized backbone curve shown in Fig. 5(b), L_1 = 3.7 in. corresponding to T_1 = 0.88 sec, and L_2 = 60 in. corresponding to T_2 = 3.5 sec. The friction coefficients were selected as $\mu_1 = 0.02$ and μ_2 = 0.08. A vertical stiffness was assigned corresponding to a vertical period of 0.03 sec, as TPB bearings are very stiff in the vertical direction.

The TPBs were modeled using the *TripleFrictionPendulum* element [24] in OpenSees. This element is capable of capturing vertical–horizontal coupling and bidirectional coupling in the two horizontal directions. Past analytical studies using this element have replicated the response observed in the full-scale experiment at E-defense shake table in Japan.

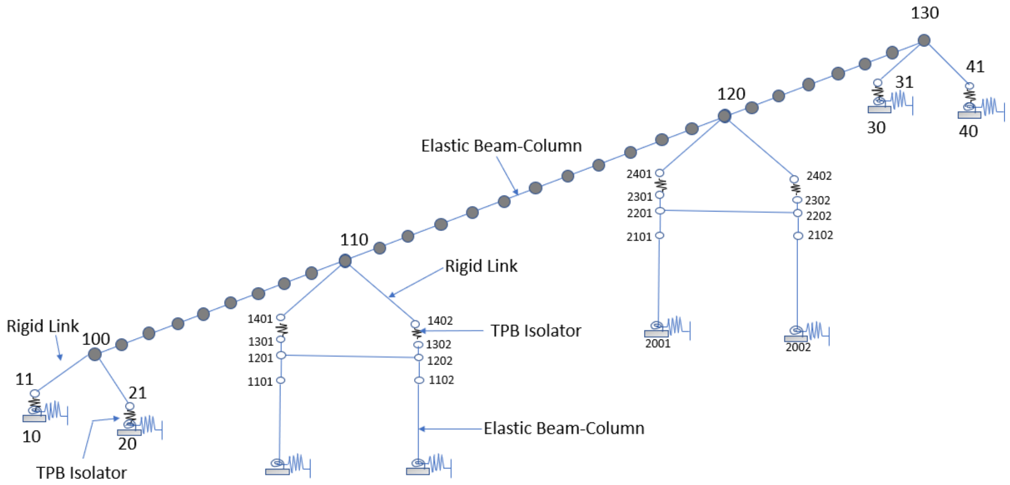

Fig. 3 – Overview of the bridge model

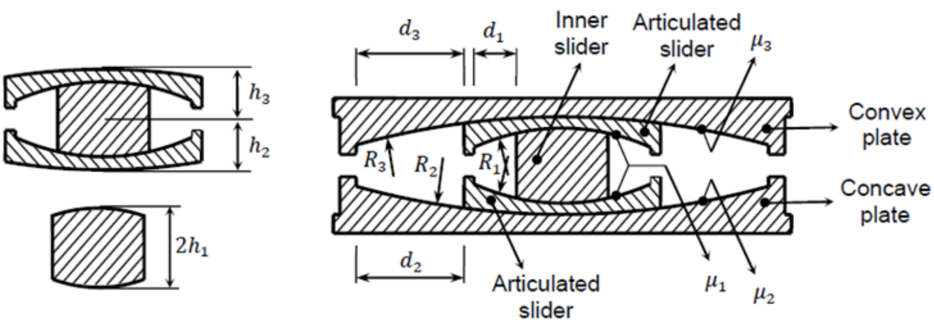

Fig. 4 – Section view of TPB

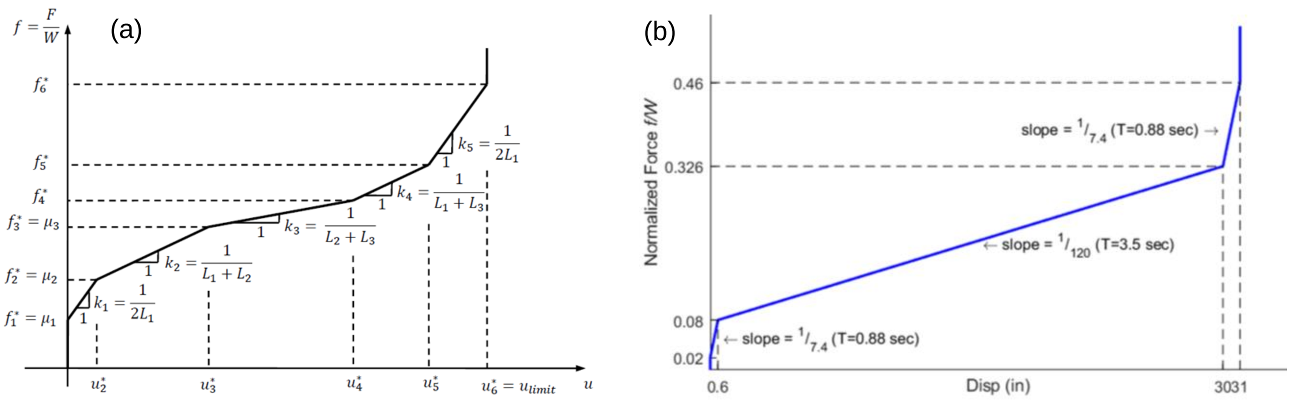

Fig. 5 – Force vs. displacement backbone curve: (a) generalized, (b) Iso. System No. 6

4. Simplified Theory to Estimate Base Shear Amplification

The authors have previously presented a simplified method to estimate the amplification of base shear due to the vertical motion [14]. The horizontal force in a system of a spherical sliding bearings, expressed as system base shear V_b, or alternatively base shear coefficient V_b/W, can be approximated as:

$$V_b = \frac{W}{R_{eff}} u + \mu W \qquad \text{or} \qquad \frac{V_b}{W} = \frac{u}{R_{eff}} + \mu \tag{2}$$

where R_{eff} is effective radius of curvature and μ = friction coefficient corresponding to the dominant sliding mechanism, and u is the isolator displacement. The equation is exact for a single pendulum bearing with a single sliding mechanism, and is a good approximation for multi-spherical bearings when the response is dominated by the outer pendulum mechanism. The base shear estimate of Eq. (2) can apply in either the longitudinal or transverse direction, and is maximized at the peak displacement u_o. This estimate is hereafter referred to as $V_{b,2D}/W$, the peak base shear coefficient in 2D shaking without vertical input.

As mentioned previously, the horizontal base shear in a spherical sliding bearing isolation system is dependent on the vertical force acting on isolators. When vertical acceleration is considered, the system base shear is determined from the instantaneous normal force N on the isolators rather than the static weight:

$$V_b = \frac{N}{R_{eff}} u + \mu N \tag{3}$$

The system normal force N varies in time according to:

$$N = W + m\ddot{u}_z^t \tag{4}$$

where m = the total mass and $\ddot{u}_z^t$ is the vertical acceleration at the isolators. Accordingly, the component of the base shear due to vertical shaking (absolute or normalized by the weight) can be estimated as:

$$V_{b,V} = m\ddot{u}_{zo}^t \left(\frac{u_o}{R_{eff}} + \mu \right) \qquad \text{or} \qquad \frac{V_{b,V}}{W} = \ddot{u}_{zo}^t (g) \left(\frac{u_o}{R_{eff}} + \mu \right) \tag{5}$$

where $V_{b,V}$ = horizontal force due to vertical acceleration only, and $\ddot{u}_{zo}^t$ is peak vertical acceleration over time. In the normalized form, $\ddot{u}_{zo}^t$ is given in units of g. Eq. (5) indicates that an additional component of base shear develops that is proportional to the vertical acceleration.

Mojidra and Ryan [14] proposed that vertical acceleration at the isolators be approximated as the PGA_V multiplied by an amplification factor that represents the amplification of vertical acceleration from the ground to the bridge superstructure. Several alternative amplification factors were investigated, and the proposed theory with an amplification factor of 1 was found to best fit the data generated from 2D and 3D response history analysis of the bridge to the ground motion suite. Hence, Eq. (5) is revised to

$$\frac{V_{b,V}}{W} = PGA_V \left(\frac{u_o}{R_{eff}} + \mu \right) \tag{6}$$

and, in turn, the total base shear coefficient for 3D shaking, $V_{b,3D}/W$, can be estimated as:

$$\frac{V_{b,3D}}{W} = \frac{V_{b,2D}}{W} + \frac{V_{b,V}}{W} \tag{7}$$

The estimation is expected to be exact if the peak horizontal displacement and the peak vertical acceleration occur at the same time, and the dynamic amplification factor is exactly known. The conclusion that an amplification factor of 1 best fits the data seems to suggest that no dynamic amplification occurs in the vertical direction, and the peak vertical acceleration equals PGA_V. However, the peak lateral base shear $V_{b,2D}$ and peak vertical base shear $V_{b,V}$ may not occur at the same time. The amplification factor of 1 could be justified if the dynamic amplification and phase lag effects approximately cancel out.

As mentioned above, the approximate method for estimating base shear in a 3D motion was evaluated by comparing the base shear coefficient determined using the approximate method [Eq. (7)] to the base shear coefficient determined from numerical simulation using the suite of ground motions. To apply the approximation, $V_{b,2D}/W$ in Eq. (7) is also taken as the numerically simulated value. The simulated and estimated base shear coefficients are shown in Fig. 6 for three select ground motions. These ground motions represent high, medium and low intensity PGA_V. The general trend in base shear coefficient for each motion and longitudinal or transverse direction are as follows. The isolation period T_2 = 2 sec for Iso. System No. 1-3, 3.5 sec for Iso. System No. 4-6 and 5 sec for Iso. System No. 7-9. Therefore, the actual base shear coefficient decreases for each subgroup as the period increases. The base shear coefficient also increases slightly over each subgroup as the friction coefficient increases (e.g μ_2 increases from 0.04 to 0.08 over Iso. System No. 1-3). The amplification of base shear (increase from 2D to 3D) tends to be proportional to the intensity of the base shear. Therefore, the shorter period systems with higher base shear coefficients also see greater amplification of base shear. For the high intensity motion SFPU, amplification of base shear coefficient from 2D to 3D is significant for all isolation system parameters [Fig. 6(a) and (d)]. For the moderate and low intensity motions (LPG and IIB), base shear amplification is noticeable for the short period T_2 = 2 sec systems 1-3, but relatively small for other periods [Fig. 6 (b), (c), (e), (f)].

The estimated base shear coefficients follow the trends observed in the simulated base shear coefficients. In other words, these estimated base shear coefficients vary with T_2 and μ_2 in a manner consistent with the simulated base shear coefficients (Fig. 6). Recall that the dependence of base shear amplification on these isolation system parameters is inherently captured through Eq. (7).

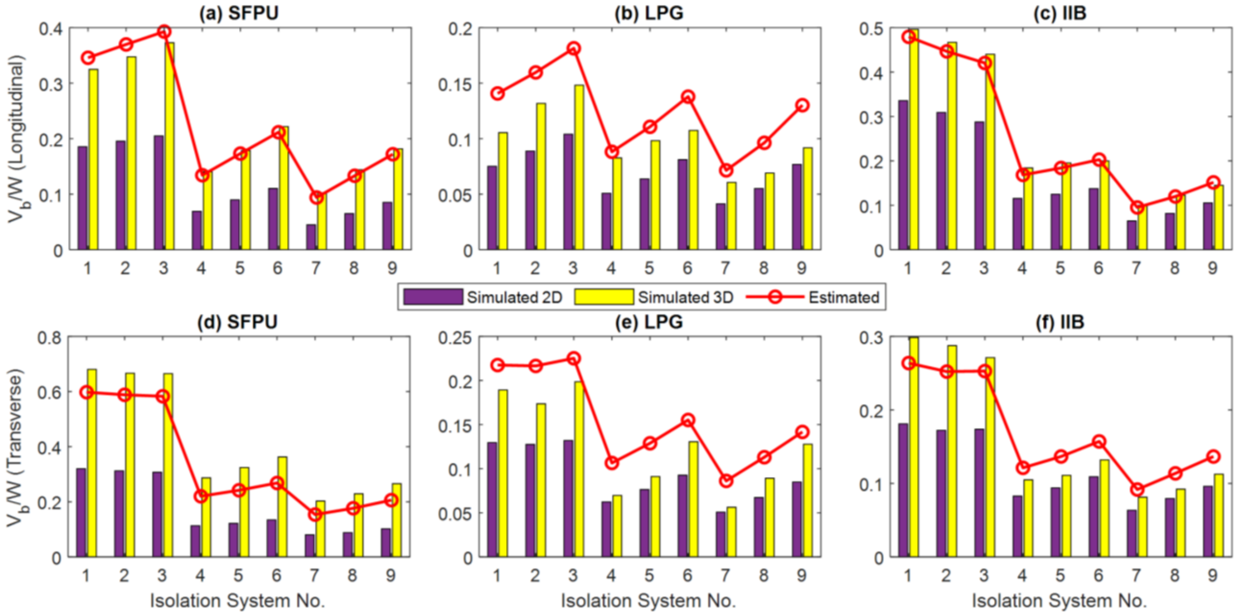

Fig. 6 – Simulated and estimated base shear coefficient V_b/W of bridge with isolation parameter variations to select motions: (a) – (c) longitudinal direction, (d) – (f) transverse direction

Mojidra and Ryan [14] also presented results for the error in the estimated relative to the simulated 3D base shear coefficient, where positive error indicates that the approximate method (Eq. (7)) overestimates or is conservative relative to the simulated base shear coefficient. The average error (over the suite of 11 ground motions and for an assumed amplification factor of 1) ranged from about 15-30% and was relatively insensitive to the isolation system parameters. However, a wide dispersion was observed for individual ground motions, with the error outliers ranging from about -30% to 90%. In these results, a tendency for the negative outliers to be associated with high intensity vertical ground motions was observed; for example, SFPU in the transverse direction (Fig. 6(d)). The approximate method is expected to be unconservative when the vertical shaking becomes so intense as to cause uplift and subsequent pounding of the bearings, which is expected to occur around 1*g*. In the next section, the effect of vertical shaking intensity is investigated more directly to determine limits on the applicability of the simplified method to estimate base shear amplification.

5. Influence of the Intensity of Vertical Shaking

The influence of vertical ground shaking intensity or PGA_V was evaluated directly by simulating the response of the bridge with different isolation parameters (Table 2) as the PGA_V was varied continuously between 0 and 1.5g. In these simulations, the scaled PGA_V for each motion in Table 1 was ignored, and instead the vertical component of each motion was scaled independently over the stated range. The simulated versus estimated base shear coefficient as a function of PGAV was evaluated for each isolation system number and each ground motion, in the transverse and longitudinal direction.

Representative results are presented in Fig. 7 for the transverse direction and Fig. 8 for the longitudinal direction, for 6 of the 11 ground motions. Simulated versus estimated 3D base shear coefficient versus PGAV are shown for isolation systems with outer pendulum friction coefficient μ_2 = 0.08; and effective period T_2 = 2 sec (Iso. System No. 3), 3.5 sec (Iso. System No. 6), and 5 sec (Iso System No. 9). Four general trends in base shear coefficient with vertical intensity (PGA_V) were observed for individual systems and ground motions: Trend 1 – the simulated base shear coefficient displays significant nonlinear variation with intensity for higher PGA_V and the estimated base shear coefficient is accurate only for low intensities, Trend 2 – the simulated base shear coefficient varies linearly with intensity and the estimated base shear coefficient is accurate over the entire PGA_V range, Trend 3 – the simulated base shear coefficient displays significant nonlinear variation with intensity for higher PGA_V and yet the estimated base shear is conservative over most or all of the PGA_V range, and Trend 4 – the simulated base shear coefficient varies linearly with intensity and the estimated base shear coefficient is conservative over the entire PGA_V range. Ranked by qualitative observation and judgment, Trend 1 is exhibited for motions SFPU, IIS and NPD in the transverse direction [Fig. 7(a)-(c)], and SFPU in the longitudinal direction [Fig. 8(a)]. Trend 2 is exhibited for motions CT4 in the transverse direction [Fig. 7(d)] and no motions in the longitudinal direction. Trend 3 is exhibited for motion CAM [Fig. 7(f)] in the transverse direction and motions IIS, NPD, CT46 and CAM in the longitudinal direction [Fig. 8(b)-(d), 8(f)]. Trend 4 is exhibited for motion LAL in both the transverse direction [Fig. 7(e)] and the longitudinal direction [Fig. 8(e)]. Over the suite of 11 motions, Trend 1 was observed in 4 of 11 in the transverse direction and 3 of 11 in the longitudinal direction, Trend 2 was observed in 2 of 11 in the transverse direction and was absent in the longitudinal direction, Trend 3 was observed in 2 of 11 of in the transverse direction and 5 of 11 in the transverse direction, and Trend 4 was observed in 3 of 11 in both the transverse and longitudinal directions.

Based on prior observations of the effect of intensity, nonlinear variation of the simulated base shear coefficient at larger PGA_V intensities (Trend 1 or Trend 3) was expected to be the dominant trend for most or all ground motions, especially as the PGA_V surpassed 1g. This behavior was demonstrated very clearly for SFPU, NPD and CAM in the transverse direction [Fig. 7(a), 7(c), 7(f)]. Trend 1 was more dominant in the transverse direction and while Trend 3 was more dominant in the longitudinal direction because of a transverse-vertical coupling phenomenon that tended to amplified base shear more in the transverse direction, as was explained in [14]. The nonlinearity in base shear coefficient is believed to be caused by a bearing uplift-impact phenomenon that occurs for PGA_V around 1 g. The impact transmits high frequency large intensity vertical accelerations to the structure that tends to affect base shear in a more unpredictable way. Further investigation is necessary to understand the trends in different motions; however, the following effects are expected to contribute to both lack of nonlinear variation of base shear in some motions and conservative estimates of base shear across the range of vertical intensity. First, the vertical frequency content of the ground motion may be mismatched with that of the structural modes, and thus dynamic amplification is limited. Second, the timing of the PGA_V may be out of phase with the occurrence of the peak base shear, such that the peak vertical intensity, even if significant, may not directly impact the base shear.

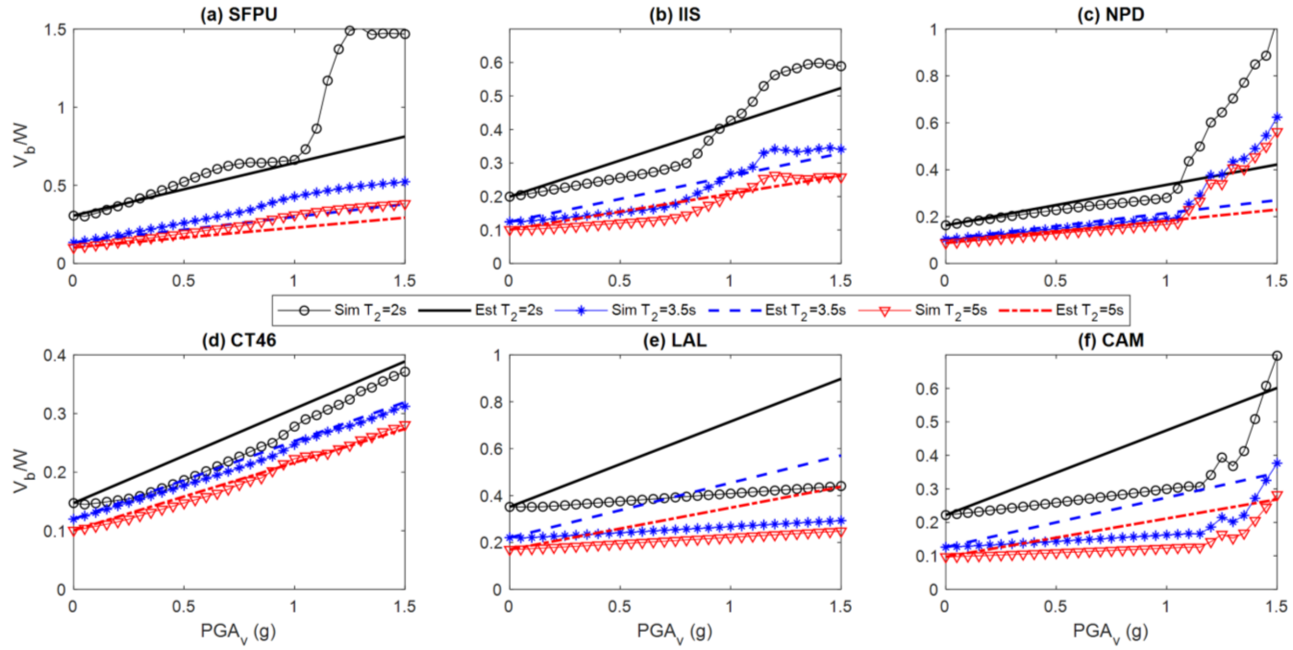

Fig. 7 – Simulated and estimated bridge transverse direction base shear coefficient V_b/W vs PGA_V of select systems for motions: (a) SFPU, (b) IIS, (c) NPD, (d) CT46, (e) LAL, and (f) CAM

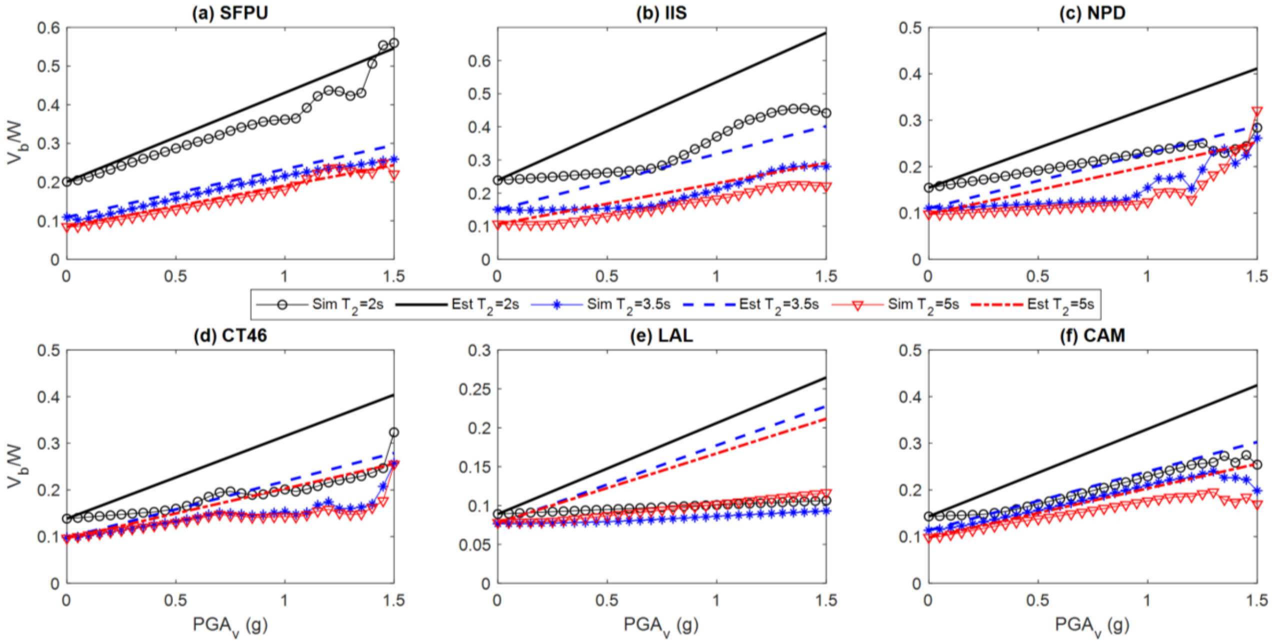

Fig. 8 – Simulated and estimated bridge longitudinal direction base shear coefficient V_b/W vs PGA_V of select systems for motions: (a) SFPU, (b) IIS, (c) NPD, (d) CT46, (e) LAL, and (f) CAM

The simplified method for estimating the base shear coefficient cannot predict the variation in trends that might occur with individual motions. To be conservative, the simplified method should be based on the Trend 1 behavior that leads to the greatest amplification of base shear with vertical shaking. This analysis confirms that the simplified method is not sufficiently conservative at large PGA_V for Trend 1 type motions.

For each model and ground motion considered, the PGA_V at which the simulated base shear coefficient started to vary nonlinearly with PGA_V – referred to hereafter as transition PGA_V – was determined by inspection. Significant judgment was exercised in the determination of transition PGA_V. Model/ground motion combinations that did not exhibit the nonlinear trend were excluded. The average transition PGA_V exhibited the following trends; transition PGA_V decreased with increasing friction coefficient μ_2 and decreased with decreasing period T_2. The lower transition PGA_V indicates that the system is more sensitive to the vertical intensity and more likely to exhibit irregular variations in peak friction coefficient. Thus, the systems with the highest friction coefficients and the lowest pendulum periods are more sensitive to the vertical intensity. However, the observed trends were not very systematic.

The average transition PGA_V over all models was 1.11g in the transverse direction and 1.05g in the longitudinal direction. It is recommended that a conservative transition PGA_V of 1g be implemented for establishing the limits of the simplified method to estimate base shear coefficient. That is, the simplified method should be applied only for PGA_V up to 1g, and 3D response history analysis should be used for PGA_V beyond 1g.

6. Summary and Conclusions

In this paper, a simplified method to estimate the amplification in the base shear coefficient due to 3D shaking in bridges isolated with TPBs has been further investigated. The simulated and estimated base shear coefficients as a function of vertical shaking intensity PGA_V for 11 ground motions were compared in model bridges with varying isolation system parameters. Although the simplified method was conservative in some motions over the entire range of PGA_V, in a number of motions – as anticipated – the base shear coefficient varied nonlinearly with PGA_V at higher intensities, and the simplified method was no longer reliable. As a preliminary recommendation, the simplified method should be applied only for PGA_V up to 1g. For vertical ground motions with PGA_V exceeding 1g, 3D response history analysis should be used to accurately predict the base shear coefficient for design.

7. Acknowledgments

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8. References

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