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第六届结构工程新进展国际论坛文集

Proceedings of the 6th International Forum on Advances in Structural Engineering (2014)

结构抗震、减震新技术与设计方法

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徐正安 任伟新 丁克伟 朱兆晴 王静峰 主编

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本书汇集11位国内外专家最新研究成果的专辑。为第六届结构工程新进展国际论坛特约报告文集。本届论坛的主题是“结构抗震、减震新技术与设计方法”。11位特邀报告主题涵盖了：大型复杂建筑结构、复杂多塔隔震结构设计、屈曲约束支撑减震技术、钢板剪力墙底层边界柱耐震设计、工程结构黏滞阻尼减振技术、城市高架桥抗震性能评估方法研究、约束钢管混凝土柱的低周疲劳性能、面向抗震设计的钢板件和钢截面分类、基于高性能计算的工程抗震与防灾、公路桥的地震分析、抗震工程在高压变电站中的应用。

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前　言　Preface

改革开放三十年以来，尤其是近15年，中国正在持续地从事着世界最大规模的土木工程建设，而这一建设高潮还会伴随着我国各项事业的发展和新型城镇化的推进而持续一段时间。持续的、大规模的工程建设，为结构工程技术的发展、学科的建设、人才的培养、队伍的壮健，特别是创新能力的提高，提供了历史性的机遇。正是基于这种时代背景，在建设部的支持下，由中国建筑工业出版社、同济大学《建筑钢结构进展》编辑部、香港理工大学《结构工程进展》（Advances in Structural Engineering）编委会联合主办，安徽省建筑设计研究院有限责任公司、合肥工业大学、安徽建筑大学联合承办的第六届"结构工程新进展国际论坛（The 6^{nd} International Forum on Advances in Structural Engineering）"在合肥举行。

本次论坛的主题是"结构抗震、减震新技术与设计方法"。近年来，大地震频发已引起人们的高度关注，目前，建筑工程的抗震、隔震和耗能减震控制技术已成为国际学术界和工程界关注的热点、研究的前沿。在本次论坛中我们荣幸地邀请了15位特邀报告人，他们的报告主题涵盖了近年来抗震、隔震、耗能减震的最新研究成果、设计方法、控制理论以及相应新型材料及构件的应用；阐述了在这些领域内的最新发展信息；同时也向与会者提供了一个与专家互动并获取宝贵经验的机会。

感谢特邀报告人，他们不仅在大会上做了精彩的主题报告，而且还奉献了精心准备的论文，使得本书顺利出版。

感谢论坛自由投稿作者以及参加本次论坛的所有代表，正是大家的积极参与配合，才使得本次论坛能够顺利进行。

感谢住建部执业资格注册中心、中国建筑工业出版社、同济大学《建筑钢结构进展》编辑部、香港理工大学《结构工程进展》编辑部对本次论坛的指导、支持和帮助。

感谢安徽省建筑设计研究院有限责任公司、合肥工业大学、安徽建筑大学对本次论坛成功主办的努力和付出。

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SEISMIC ASSESSMENT OF HIGHWAY BRIDGES SUBJECTED TO COMBINED HORIZONTAL AND VERTICAL MOTIONS

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ABSTRACT: A detailed evaluation of two typical bridge configurations is carried out to assess the suitability of linear response spectrum analysis to combined horizontal and vertical response spectra for predicting moment demands in the bridge girder. The process consists of first selecting the corresponding Caltrans ARS curve based on the earthquake magnitude and peak rock acceleration at the bridge site. This spectrum is applied in the longitudinal direction of the bridge. The vertical spectrum is built up using the procedures described in the report by Kunnath et al. (2008) . The moment demands of the bridge girder are then computed from elastic response spectrum analysis considering sufficient number of modes using the longitudinal and vertical spectra. These results are compared to demands estimated with time history simulations using nonlinear models of the bridge columns. It is concluded that response history analysis using the horizontal and vertical design spectra is a valid preliminary approach to estimate the effects of vertical ground motions on ordinary highway bridges. The estimates from response spectrum analysis are typically more conservative for multi-span bridges than for 2-span overcrossings.

KEYWORDS: Nonlinear time history; response spectrum analysis; vertical ground motions

INTRODUCTION

In a recent study by Kunnath et al. (2007) investigating the effect of vertical ground motions on the seismic response of ordinary highway bridges, strong vertical accelerations have been found to have significant effects on (i) the axial force demand in columns; (ii) moment demands at the face of the bent cap, and (iii) moment demands at the middle of the span. The last issue was identified as the primary issue to be considered for the analysis and design of typical short-span column supported bridge configurations. This is because, in the absence of vertical effects, the design of the mid-span section is governed by positive moments whereas strong vertical motions can cause significant negative moments in the mid-span that can yield the top reinforcement.

The effects of vertical ground motions on structural response have also been investigated by many researchers in the past. Saadeghvaziri and Foutch (1991) conducted one of the early studies on the effect of vertical ground motions and found that the energy-dissipating capacity of bridge columns was reduced and the section shear capacity was influenced by the variation of axial forces due to vertical excitations. Broekhuizen (1996) and Yu et al. (1997) investigated the response of several overpasses on the SR 14/15 interchange after the 1994 Northridge earthquake. The study by Broekhuizen indicated that the vertical accelerations could significantly increase tensile stresses in the deck while the latter study found increases of about 20% in axial force demand and only a marginal change in the longitudinal moment when vertical motions were considered in the evaluation. The evaluation of 60 prestressed box-girder bridges by Gloyd (1997) indicated that the dynamic response from vertical acceleration was larger than dead load effects. Papazoglou and Elnashai (1996) reported analytical and field evidence of the damaging effect of vertical ground motions on both building and highway bridge structures. They state that strong vertical motions induced significant fluctuations in axial forces in vertical elements leading to a reduction of the column shear capacity. In certain cases, compression failure of columns was also reported to be likely. Numerical simulations were carried out to confirm these observations. Later Elnashai and Papazoglou (1997) and Collier and Elnashai (2001) worked on simplified procedures to combine vertical and horizontal ground motions. Both papers focus on near-fault ground motions that have been recorded within 15 km of the causative fault since these ground motions were observed to possess significant vertical components. Moreover, it was suggested to limit the damping ratio of elements susceptible to vertical effects to 2% because vertical ground motions are associated with higher frequency oscillations. Secondly, there are limited hysteretic energy dissipation mechanisms for vertical inelastic response than in the case of transverse response. Button et al. (2002) examined several parameters including ground motion and structural system characteristics. However, most of their studies were limited to linear response spectrum and linear dynamic analyses. Finally, Veletzos et. al. (2006) carried out a combined experimental-analytical investigation on the seismic response of precast segmental bridge superstructures. Among other issues, they also examined the effects of vertical ground motions. Their numerical analyses indicated that the prestressing tendons above the piers of one of the bridge structures yielded under positive bending. The median positive bending rotations were found to increase by as much as 400% due to vertical ground motions. Lee (2012) carried out an experimental and analytical investigation of reinforced concrete columns subjected to horizontal and vertical ground motions. In the experiments conducted on the UC-Berkeley shaking table, the shear behavior of two 1/4- geometrical scale specimens was examined under combined vertical and horizontal components. The experimental results confirmed that vertical accelerations can induce tensile strains which result in shear strength degradation of RC bridge columns.

For ordinary standard bridges constructed on sites where the peak rock acceleration is expected to be more than 0. 6g, SDC-2006 (Caltrans 2006) requires consideration of vertical effects but does not require analysis of the structure under combined horizontal and vertical components of the ground motion. Instead, it stipulates the check of the nominal capacity of the structure designed considering horizontal effects only under an equivalent vertical load with a magnitude of 25% of the dead load (DL) of the structure applied separately in the upward and downward directions to account for vertical effect.

This study is a direct extension of the research completed by Kunnath et al. (2008) which concluded that if the expected peak rock acceleration is less than 0.4g, the vertical components of ground motions may be ignored and the design can proceed in accordance with existing SDC (Caltrans 2006) guidelines. If the expected peak rock acceleration is higher than 0.4g, Kunnath et al (2008) recommend a 3D linear response spectrum analysis as an initial first step to estimating seismic demands, particularly the mid-span moments, from vertical ground motions.

In the present study, a detailed evaluation of two typical bridge configurations is carried out to assess the suitability of linear response spectrum analysis to combined horizontal and vertical response spectra for predicting moment demands in the girder. The process consists of first selecting the corresponding Caltrans ARS curve based on the earthquake magnitude and peak rock acceleration at the bridge site. This spectrum is applied in the longitudinal direction of the bridge (longitudinal spectrum) . The vertical spectrum is built up using the procedures described in the report by Kunnath et. al. (2008) . The moment demands of the bridge girder are then computed from elastic response spectrum analysis considering sufficient number of modes using the longitudinal and vertical spectra. These results are compared to demands estimated with time history simulations using nonlinear models of the bridge columns.

MODELING OF BRIDGES

Two types of bridges will be considered in the study: single bent, two span overpasses and multi-span single frame bridges. A segment of an existing bridge in California, the Camino DelNorte Bridge, is selected as the prototype of an overpass bridge, whereas the Amador Creek Bridge is selected as the prototype multi-span column supported. Several configurations of each bridge are generated from the base configuration of each system without violating the specifications in SDC-2006 (Caltrans 2006) on allowed dimensional and balanced stiffness requirements to cover a practical range of fundamental periods

Modeling of a typical overcrossing

With the objective of selecting a typical ordinary standard bridge that was representative of a reinforced concrete over-crossing designed according to post-Northridge Caltrans specification, we selected a portion of the widening project of Camino Del Norte Bridge. The selected system is a single bent reinforced concrete bridge with two spans of 31.0 and 30.5 meters in length. The single bent is composed of two octagonal columns with spiral reinforcement. Line model is used in the numerical simulations for two-column bent over-crossings. The nonlinear simulations will be performed using the open-source software, OpenSEES (2009).

The typical two-column overcrossing and section detailing are shown in Figure 1. The column height and superstructure spans are identified in Figure 2. The column diameter is 1.68m. The column is reinforced with 25 #36mm bars and transverse reinforcement consisted of #25mm bars at a spacing of 0.1 m. Material properties are based on 27.6 MPa compressive strength with an ultimate strain of 0.006 concrete (unconfined) and 413.69 MPa steel for both longitudinal and spiral reinforcement. The confined concrete is modeled using Mander's model. A bilinear model with a post-yield stiffness of 1.33% of the initial stiffness was used to model the reinforcing steel. Each pier is modeled using a force-based nonlinear beam column element with fixed hinge lengths at element ends. The inelastic behavior of the hinge region is simulated with a discretized fiber section model. Using a fiber section is of critical importance in the present evaluation since it enables the consideration of the variations in the column moment capacity due to changes in the axial force in the columns as a result of vertical ground motions. The simulation model used in the nonlinear time history analyses is displayed in Figure 2.

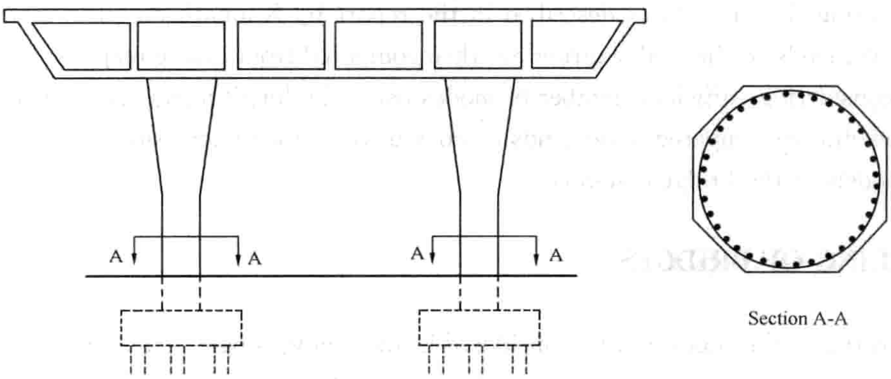

Figure 1 Configuration of two-column bent

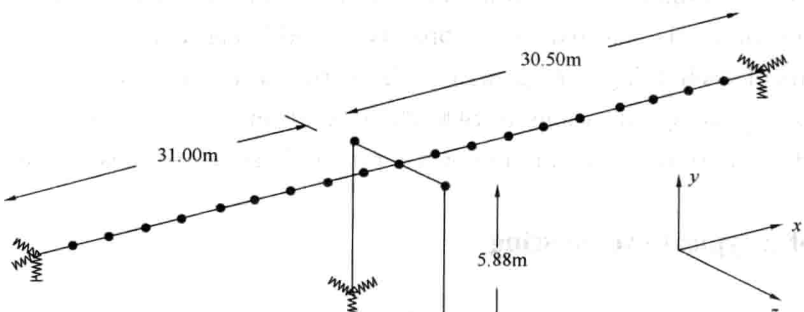

Figure 2 Simulation model of two-column bent

Following the SDC-2006 (Caltrans 2006) guideline which uses a capacity design approach to limit the inelastic behavior to the column element, the superstructure is designed and modeled to remain elastic under seismic motion. Based on this assumption, elastic elements are uses to model bridge girders. The elastic properties that used for the girder are summarized in Table 1.

The end conditions both at the abutments and at the bottom of the columns are modeled using spring elements to simulate the flexibility of the soil-pile-foundation system. SDC-2006 (Caltrans 2006) provides guidelines to determine these spring constants. The abutment stiffness in the longitudinal direction was computed as $K = K\_i\ \omega h/5.5$, where Ki is the initial stiffness of the abutment and is taken to be equal to 0.69 kN/m per unit length of the abutment, ω and h are the width and height of the diaphragm abutment. For the abutment stiffness in the transverse direction and foundation stiffness in both translational directions, an empirical value, based on recommendations in the Caltrans guidelines, equal to 4.55 kN/m per pile is used.

Elastic properties of the box girder of Camino Del Norte Bridge **Table 1**

Parameter	Value
Area, A	7.32 m^2
Moment of Inertia, I_x	3.51 m^4
Moment of Inertia, I_y	141.92 m^4
Torsional constant, J	1.50 m^4

Based on preliminary dynamic analysis, the fundamental periods of the base configuration of Camino Del Norte Bridge were determined to be 0.55, 0.32 and 0.19 seconds in the longitudinal, transverse and vertical directions, respectively. In order to cover a wide range of fundamental periods, especially in the vertical direction, the base bridge configuration was modified to develop the additional bridge configurations. Care is taken not to violate the limits imposed by SDC-2006 (Caltrans 2006) on the geometry and dimensional restrictions for Ordinary Standard Bridges. Accordingly, only the span lengths of the bridge are modified from the original values, which in turn alter the mass of the bridge, and correspondingly the fundamental period of the bridge in all three directions.

The selected Camino Del Norte Bridge has been modeled in OpenSEES and SAP2000. In OpenSEES, material properties and modeling procedure are described as above, while in SAP2000 a line model with linear material properties is used. Similar period and model properties are found in both models. The span lengths and bridge periods in longitudinal, transverse, and vertical directions of final configurations are summarized in Table 2.

Properties and periods of highway overcrossings considered in the study **Table 2**

Configuration	Original	Config1	Config2	Config3	Config4	Config5
Left span (m)	31.0	20.9	41.0	46.0	51.0	36.0

续表

Configuration	Original	Config1	Config2	Config3	Config4	Config5
Right span (m)	30. 5	20. 5	40. 5	45. 5	50. 5	35. 5
T_L (s)	0. 55	0. 46	0. 64	0. 68	0. 75	0. 59
T_T (s)	0. 32	0. 27	0. 38	0. 39	0. 45	0. 34
T_V (s)	0. 19	0. 13	0. 30	0. 36	0. 43	0. 24

The moment curvature relationship of bridge column for each configuration is shown in Figure 3. The loading case is gravity only. The differences among each curves origins from the change of weight of superstructure. Configuration 4 and configuration 1 are the cases that have longest and shortest span length, respectively. Due to different gravity load that carried by bridge column, column section of configuration 4 shows highest moment capacity compared to other configurations at same curvature, while configuration 1 gives the lowest moment capacity.

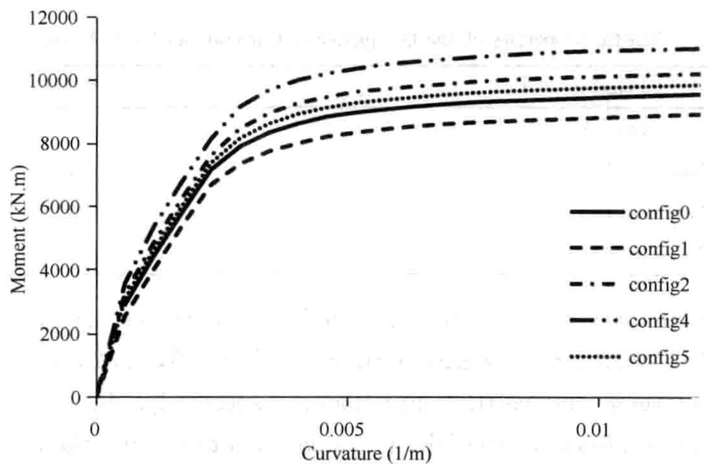

Figure 3 Moment curvature relationships of the column sections of the Camino Del Norte Bridge under gravity loads

Modeling of multi-span single frame bridges

For the case of a prototype bridge that represents a typical multi-span bridge, the Amador Creek Bridge which is a multi-frame pre-stressed concrete bridge built by Caltrans according to post-Northridge design practice was selected. It is a three-bent, four span bridge with a total length of 208. 8 m. The elevation view and column details are shown in Figure 4. The elastic properties of the superstructure are presented in Table 3. The spring properties used to model footings have been summarized in Table 4, and the structural dynamic properties are summarized in Table 5. The moment curvature relationship of bridge column for each configuration is shown in Figure 5.

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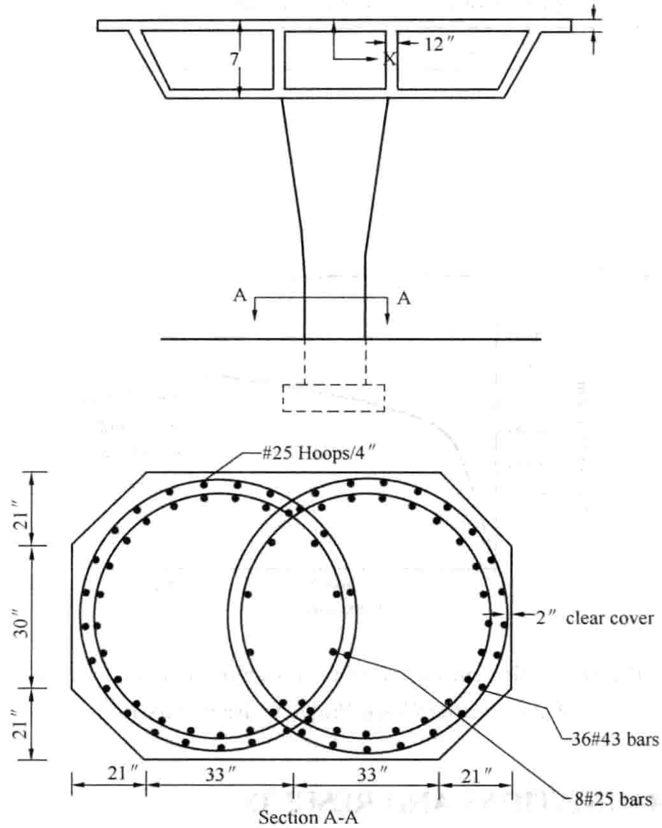

Figure 4 Elevation view and column details of Amador Creek Bridge

Elastic properties of the Amador Creek Bridge superstructure **Table 3**

Parameter	Value
Area, A	6.73m^2
I_x	4.56m^4
I_y	73.70m^4
J	30.52m^4

Elastic properties of springs to model footings **Table 4**

Spring Direction	Value
Translation, x	5.18×10^6kN/m
Translation, y	6.01×10^6kN/m
Translation, z	4.99×10^6kN/m
Rotation, x	1.05×10^8kN · m/rad
Rotation, y	1.16×10^8kN · m/rad
Rotation, z	5.30×10^7kN · m/rad

Dynamic properties of selected multi-span bridges **Table 5**

Configuration	Config1	Config2	Config3
Side span (m)	40.5	34.0	25.0
Middle span (m)	54.0	45.0	35.0
T_L (s)	2.50	2.29	1.98
T_T (s)	2.33	2.12	1.77
T_V (s)	0.52	0.37	0.23

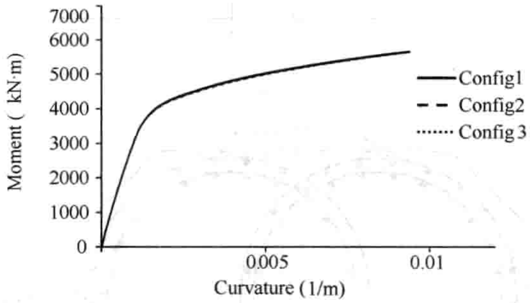

Figure 5 Moment-curvature relationships of column sections of the Amador Creek Bridge under gravity loads

NUMERICAL SIMULATIONS AND RESULTS

In order to investigate the effectiveness of simplified elastic response spectrum analysis (RSA) to estimate moment demands in the girder for the typical ordinary bridges, the response parameters of interest computed by RSA will be compared with results from nonlinear time history analysis. The moment demands at the mid-span and interior supports are identified as the main response parameters of interest. For the response spectrum analysis, the following three cases will be considered: (1) the use of the ARS design spectrum and the developed vertical spectrum by Kunnath et al. (2008); (2) Intensity scaled (to match the spectral acceleration of the ARS spectra at the fundamental period) response spectrum of each selected ground motion used in the nonlinear time-history (NTH) simulations, record by record, both horizontal and vertical components; (3) Mean horizontal and vertical spectrum of the response spectra of the NTH ground motion set. In addition to comparing moment demands in the bridge girder, the probability density distribution of the girder moment demands estimated by both methods will also be compared.

Ground motions used in simulation study

A site in San Francisco was selected as the location for all the bridge configurations investigated in the study. The Caltrans ARS spectrum for the site was developed to be applied in the longitudinal direction of the bridge. Caltrans ARS Online (version 2.1.05) (2013) is the web-based tool that calculates both deterministic and probabilistic acceleration response spectra for any location in California based on criteria provided in Appendix B of Caltrans Seismic Design Criteria (2006). In the report by Kunnath et al. (2008), a procedure was introduced to construct a vertical response spectrum based on site characteristics and horizontal spectrum. The V/H ratio (ratio of spectral accelerations at each period) is calculated using Equation 1. The coefficients used for this equation are summarized in Kunnath et al. (2008) and listed in Table 6.

$$\ln(V/H) = b_1 + b_2 \times (M-6) + b_3 \times \ln(D+5) + b_4 + b_5 \times (\ln(PGA_{rock} + 0.05)) \times \ln(vs_{30}) + \sigma \quad (1)$$

In the above expression, PGA_{rock} is taken as the horizontal peak ground spectral acceleration in g, M is the maximum moment magnitude of the site, and D is the closest rupture distance to the rupture plane in km. In this study, the standard deviation for the selected ground motions is assigned as $\sigma = 0.5$.

List of parameters for V/H model **Table 6**

Period (sec)	*b1*	*b2*	*b3*	*b4*	*b5*
0.01	0.644	−0.039	−0.15	−0.073	0.0135
0.02	0.534	−0.058	−0.118	−0.046	0.024
0.03	1.185	−0.079	−0.118	−0.115	0.003
0.05	2.135	−0.076	−0.168	−0.241	0.0219
0.1	1.89	0.03	−0.111	−0.245	0.023
0.15	1.63	0.064	−0.191	−0.248	−0.003
0.2	0.488	0.048	−0.144	−0.11	−0.002
0.3	−1.03	0.051	−0.083	0.059	−0.012
0.4	−1.536	0.041	−0.068	0.107	−0.022
0.5	−2.264	0.033	−0.006	0.191	−0.02
0.75	−3	0.05	−0.015	0.287	−0.041
1	−2.83	0.053	−0.068	0.292	−0.042
1.5	−3.29	0.094	−0.116	0.398	−0.047
2	−3.39	0.103	−0.113	0.434	−0.04
3	−2.86	0.217	−0.092	0.338	−0.038

The site characteristics are as follows: shear wave velocity $V_{S30} = 500$ m/s and nearest rupture distance = 14.32 km. Using the data in the web-based tool Caltrans ARS Online (version 2.1.05) (2013), the horizontal spectrum of site is generated and is shown in Figure 6.

Based on the specifications set forth in the Caltrans Seismic Design Criteria (2006), the Design Spectrum (DS) is defined as the greater of:

- Probabilistic spectrum based on a 5% in 50 years probability of exceedance (or 975-year return period);
- Deterministic spectrum based on the largest median response resulting from the maximum rupture (corresponding to M_{max}) of any fault in the vicinity of the bridge site;
- Statewide minimum spectrum defined as the median spectrum generated by a magnitude 6.5 earthquake on a strike-slip fault located 12 kilometers from the bridge site.

Thus, design spectrum is chosen as the probabilistic spectrum based on the USGS spectrum that corresponds to anexceedance probability of 5% in 50 years, with near fault factor applied. For the design spectrum used in this paper, the maximum magnitude (M_{max}) is 7.9 and peak rock acceleration in vertical direction is 0.58g which is larger than 0.4g. Hence a full 3D elastic bridge model is required for the corresponding response spectra analysis, based on SDC regulations (2006).

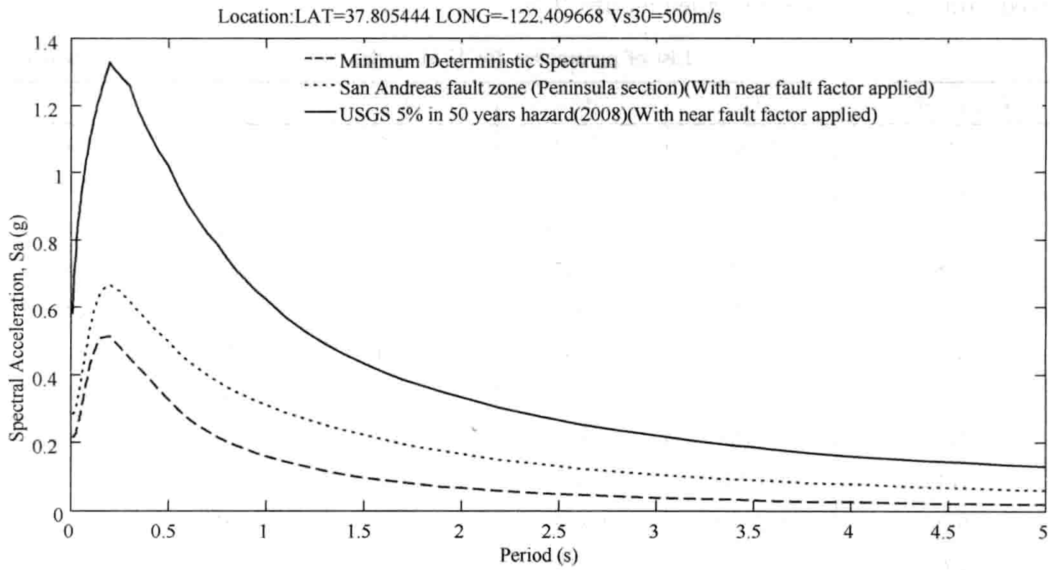

Figure 6 Site-specific spectra for selected bridge location

Twenty ground motions (see Table 7) with peak ground acceleration (PGA) of one or both horizontal components larger than 0.25g and relatively high vertical-to-horizontal PGA ratios are selected from PEER NGA database (2011). The ground motions are scaled to match the spectrum acceleration value on the design spectrum (the Caltrans ARS spectrum, see Figure 6) at the fundamental longitudinal period of bridge models, for each of the configurations considered in the study. For consistency, the same scale factor is applied to both horizontal and vertical directions of a ground acceleration record. The mean horizontal spectra of the twenty scaled ground motions for configuration 0 of both the model of Camino Del Norte Bridge and Amador Creek Bridge are displayed in Figure 7 and

8 respectively. The "target" spectrum – the Caltrans ARS design spectrum and the developed vertical spectrum are also plotted.

Selected ground motions for the study **Table 7**

No.	EQ name	Station	V/H ratio
1	Gazli, USSR	Karakyr	1.76
2	Imperial Valley-06	El Centro Array #5	1.03
3	Coalinga-05	Oil City	0.66
4	Nahanni, Canada	Site 1	1.90
5	N. Palm Springs	North Palm Springs	0.63
6	N. Palm Springs	Whitewater Trout Farm	0.77
7	Baja California	Cerro Prieto	0.42
8	LomaPrieta	Capitola	1.02
9	LomaPrieta	LGPC	0.92
10	Cape Mendocino	Cape Mendocino	0.50
11	Landers	Lucerne	1.04
12	Northridge-01	Jensen Filter Plant	0.81
13	Northridge-01	Jensen Filter Plant Generator	0.81
14	Northridge-01	Pacoima Dam (upper left)	0.78
15	Northridge-01	Rinaldi Receiving Sta	1.01
16	Northridge-01	Sylmar - ConverterSta	0.65
17	Chi-Chi, Taiwan	TCU071	0.69
18	Northridge-06	Rinaldi Receiving Sta	0.92
19	Chi-Chi, Taiwan-06	TCU079	0.75
20	Chi-Chi, Taiwan-06	TCU080	0.89

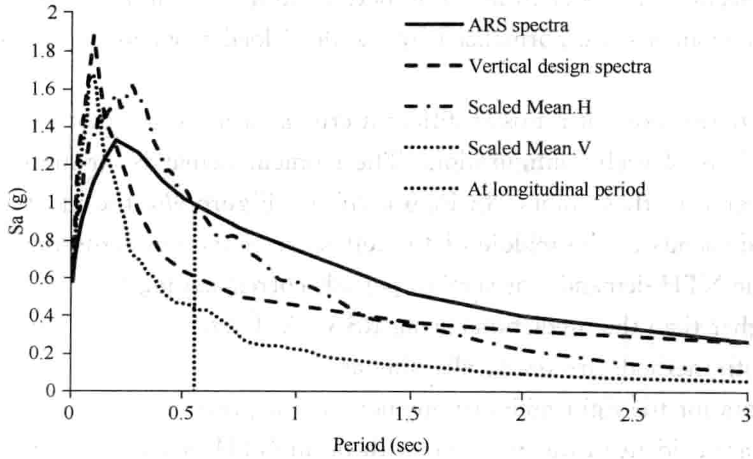

Figure 7 Scaled spectra for base configuration of 2-span overcrossings

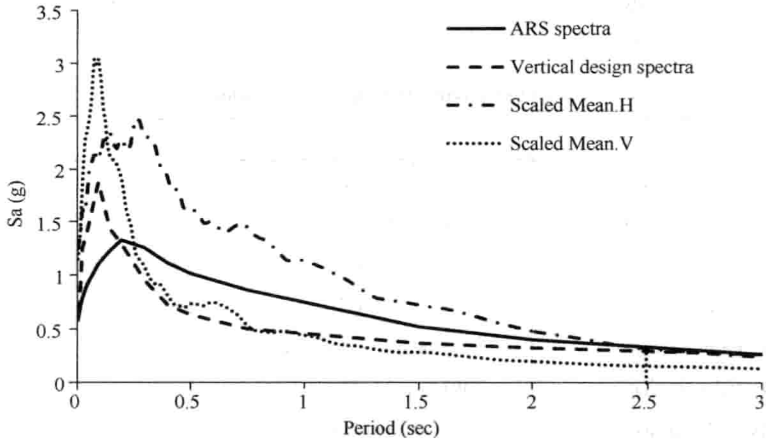

Figure 8 Scaled spectra for base configuration of multi-span bridge

Results of simulations for two-span overcrossing systems

The nonlinear time history (NTH) analysis of six configurations listed in Table 2 subjected to the 20 selected ground motions (Table 7) is accomplished using OpenSEES. For comparison, the response spectrum analysis (RSA) is performed using SAP2000 (Computers and Structures, 2012) . In the results presented in this section, the girder positive moment indicates tension on the bottom face. In general, in the absence of vertical motions, the peak mid-span moments are positive moments and the peak support (girder-column region) are negative moments. However, as demonstrated in the previous research by Kunnath et al. (2008), one of the significant effects of strong vertical motions is to induce large negative moments in the mid-span which may result in yielding of the top reinforcement. The ability of RSA to capture the amplification of the mid-span moments is examined in this section. In order to facilitate better interpretation of results, the maximum values of girder moments are normalized by the dead load moment and expressed as moment ratios.

In Figure 9, the moment ratios at different critical sections are plotted as a function of the vertical periods of each configuration. The moment demands computed by RSA and NTH are compared in these plots. In Figure 9*a* and Figure 9*b*, the maximum and minimum moment demands at the middle of the left span of the over-crossing are compared. The mean of the NTH demands for vertical periods corresponding to 0. 13 sec and 0. 19 sec are slightly higher than the predictions using RSA. At longer periods, the mean computed demands for both methods are practically similar.

Similar plots for the right mid-span moment are displayed in Figure 9*c* and Figure 9*d*. Similar trends are evident in this case too-with mean NTH demands being slightly higher than those predicted using RSA. These differences are not significant considering the fact

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