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PARTICIPATION FACTORS OF THE THREE EARTHQUAKE COMPONENTS IN THE SEISMIC ELASTIC RESPONSE OF REGULAR BRIDGES

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ABSTRACT

This paper presents the results of a comparative study of the effects of the three seismic components in the elastic response of bridges. Analyses were performed using a small data base of ten earthquakes recorded in the zone of greater seismicity in Mexico. The selected records were applied to a regular reinforced concrete highway bridge model. The independent responses for each orthogonal direction were combined with different proposals, either available in the literature or recommended in different design codes. The results of elastic analyses show the variation in the displacement, shear and axial forces response for the different combinations, relative to the analytical one, obtained through a time history analysis.

Introduction

Nowadays there are several researches about the seismic behavior of bridges, because of inaccuracy of the predicted response. Papaleontiou and Rosset (1993) consider that part of the problem is due to an erroneous estimation of seismic displacements. Also, to some extent, to the laws of combination of seismic components and a lack of understanding of the effects that the vertical component has on this type of structures.

Analysis of the vertical component of the earthquake

In most design codes for bridges only the horizontal components of earthquakes are considered, leaving the analysis of the vertical component only to structures located near the failure zone, in many cases, without a detailed specification of how to perform its evaluation. In general, it has been determined that the ratio of vertical to horizontal movement (V/H) depends on the distance to the place where the earthquake was originated, and varies as a function of its period and magnitude. However, some codes estimate this ratio according to the initial proposal of Newmark *et al.* (Perea and Esteva 2004), that is, constant and equal to 2/3. As for the spectral ratio of the horizontal response to the vertical one, Bozorgnia *et al.* (1999) determined that this ratio is different for firm soils and rigid ones.

Authors like Papazoglou and Elnashai (1996) point out that, in the surroundings of the epicenters of

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moderate to strong earthquakes, the (V/H) rate is greater than unity, and therefore does not match the average value of 30% for the vertical component established by some design codes. On bridges, Saadeghvaziri and Foutch (1991) refer that important variations of the forces induced by the vertical movement of the earthquake on the bridge abutments are not considered in seismic codes, and thus may produce beating, which can be so high that the compression stresses are up to three times greater than those due to dead loads. In addition, the vertical movement could induce fluctuation of axial compression forces on the piers, produces instability, causes the transversal reinforcement to reach its fluency state, (especially in the zones where the reinforcement is poor), and affect the foundation elements (Elnashai *et al.,* 1996).

Combination factors of seismic components

The design codes solve the problem of predict the orientation of future seismic loads making separate analyses of each component in turn; to combine later on the responses so obtained using different rules. Most of them consider only the two horizontal components of movement in those combinations.

There are different proposals for the combination of unidirectional responses to obtain the maximum bi or three-directional responses. In the design codes, the common combination rule considers an orthogonal component applied in one direction and a percentage of the perpendicular one in the other direction and vice versa, whatever results more unfavorable in each case (see eq. 1).

$$\begin{aligned} R_{c1} &= R_X + \lambda R_Y \\ R_{c2} &= R_Y + \lambda R_X \end{aligned} \tag{1}$$

where R_{c1} and R_{c2} are the possible combinations of the bidirectional response, and R_X and R_Y are the response to the complete action of the earthquake in the orthogonal directions of the bridge; selecting *x* or *y* to produce the unfavorable condition. λ is the factor defining the contribution of the earthquake in the orthogonal direction. In some codes participation factors of 30% (see Caltrans-1990, UBC-1997, or the Mexico City regulations), 40% (in ASCE-1986 or ATC-32, 1996), or 50% for especial structures such as elevated tanks are considered. When the three components of the earthquake are used, the combinations are defined by eq. 2

$$\begin{aligned} R_{c_1} &= R_x + \lambda_1 R_Y + \lambda_2 R_Z \\ R_{c_2} &= \lambda_1 R_X + R_Y + \lambda_2 R_Z \\ R_{c_3} &= \lambda_1 R_X + \lambda_2 R_Y + R_Z \end{aligned} \tag{2}$$

where R_z is the response in the vertical direction, and λ_1, λ_2 are participation factors. In Eurocode-8 (2002), for bridges are proposed participation factors $\lambda = \lambda_2 = 30\%$ in the two perpendicular directions, although there is little research on the subject and further studies are needed.

Another combination rule for maximum bi or three-directional responses commonly used in codes is the square root of the sum of the squares, called SRSS and defined by:

$$\begin{aligned} R_c &= \sqrt{R_X{}^2 + R_Y{}^2} \\ R_c &= \sqrt{R_X{}^2 + R_Y{}^2 + R_Z{}^2} \end{aligned} \tag{3}$$

Tena and Pérez (2006) studied the bi-directional combination factors for displacements and accelerations for building with seismic isolation, subject to simulated seismic action characteristic of the zone of greater seismicity in Mexico and for firm soils. By means of this study they proposed ratios between the bi-directional, R_{BD}, and unidirectional, R_{UD}, responses. The relation obtained is expressed in eq. 4

$$R_{BD} = \lambda R_{UD} \tag{4}$$

where λ is a coefficient, function of the structure period. Tena and Pérez defined a mean ($\bar{x}$) and a mean plus a standard deviation ($\bar{x}+s$) of λ values for the simulated records and for different structural periods. Due to the fact that the study was focused on applications in isolated structures, it considers only fundamental periods greater than 1 s, although the results were extrapolated.

The present work is a preliminary study that intends to compare the maximum responses obtained with different combination rules in a regular reinforced concrete bridge. For that purpose, the combination rules considered are: a) eqs. 1 and 2, with percentages values of 0.3 and 0.4 in λ; b) the SRSS rule (eq. 3); and c) the Tena and Pérez rules (eq. 4). The responses are combined for two and three directions, with the idea of also evaluating the influence of the vertical component on this type of structures.

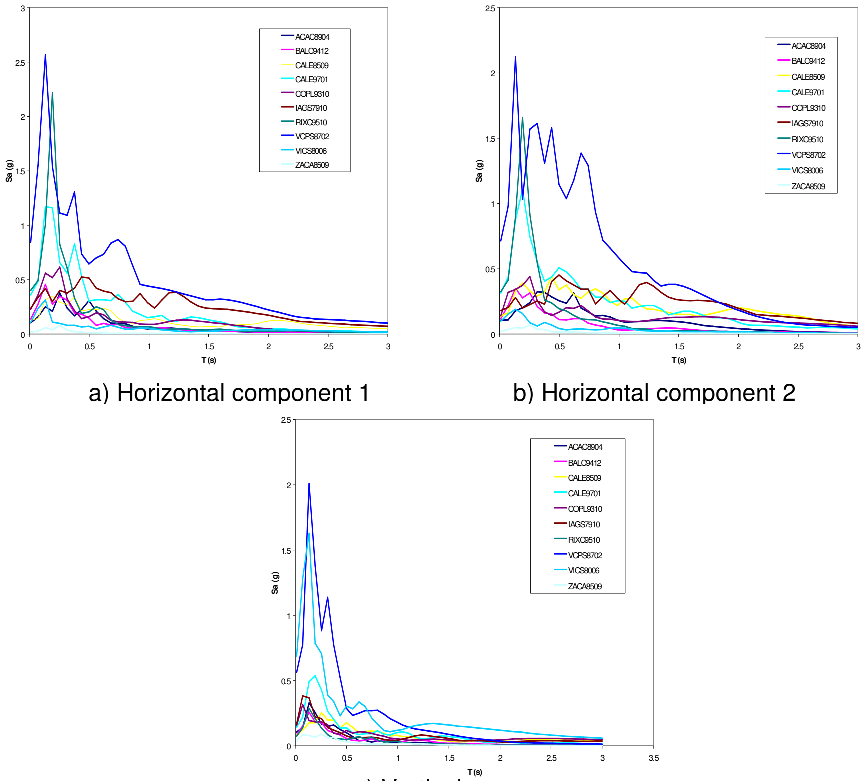
Used Accelerograms

To evaluate different proposals for the combination of the three independent components of the earthquake, ten records were selected, characterized by having important accelerations in some of its components. Such records were taken from the database of strong earthquakes in Mexico, from 1960 to 1999 (BMDSF 2000). The accelerograms considered were processed using a baseline correction and filter process with common techniques. Table 1 shows the general characteristics of the earthquakes used; specifically the maximum horizontal acceleration (AH) and maximum vertical acceleration (AV) of each record are shown. The three first records shown in table 1 are signals with especially high vertical accelerations. In fig. 1 the pseudo-acceleration spectrum of each used earthquake are shown, displayed for its three components.

Table 1. Characteristics of the accelerograms.

RECORD	M	R	H	AH(g)	AV(g)	SOIL	SITE
VCPS870207	5.4	6	6	1.45	0.69	Volcanic rock	Mexicali Valley, Baja California
IAGS791015	6.6	3	10	0.36	0.91	Sediment (alluvial)	Mexicali Valley, Baja California
VICS800609	6.1	10	12	0.98	1.01	Sediment (alluvial)	Mexicali Valley, Baja California
COPL931025	6.6	7	19	0.28	0.13	Rock	Copala, Guerrero
CALE850919	8.1	21	15	0.14	0.09	Rock	Campos Cove, Michoacán
CALE970111	6.9	30	16	0.4	0.42	Rock	Campos Cove, Michoacán
BALC941210	6.3	38	20	0.27	0.19	Rock	EL Balcón, Guerrero
ACAC890425	6.9	56	15	0.12	0.11	Sand, slime, clay	Acapulco, Guerrero
ZACA850919	8.1	84	15	0.27	0.15	Compact clay	Zacatula, Michoacán
RIXC951021	6.5	54	98	0.45	0.12	Limestone	Tuxtla Gutierrez, Chiapas

M: Magnitude; R: Epicentral distance in Km; H: Focal depth in Km; AH, AV: Maximum acceleration of the ground in horizontal and vertical direction; BMDSF earthquake code: SSSSYYMMDD (SSSS: station, YY: year; MM: month; DD: day).



a) Horizontal component 1 b) Horizontal component 2

c) Vertical

Figure 1. Pseudo-acceleration response spectrum of the earthquakes in the database (ξ=5%).

Bridge Model

To verify the results obtained with different combination laws, an analysis model of a reinforced concrete bridge was used. This structure, taken from the literature (Priestley *et al*. 1996), was designed for gravitational and seismic loads, not considering a moving live load. The bridge selected consists of hollow rectangular piers and box girder section with the geometric properties represented on table 2. The structure has four 50 m spans and three piers of 14 m height.

In the bridge, the compression stress of the concrete (f´c) is 27500 KPa, and the fluency stress of the steel is f_y*=420 MPa*, for all the elements. Young's modulus was estimated according to the Mexico City code for Class 1 concretes, in which $E = 4400\sqrt{f´c}$. In table 2, geometric characteristics of girder and pier elements are shown, while a drawing of the general geometric of bridge and cross sections of girder and pier elements are presented in fig. 2.

The abutment stiffness was modeled as elastic springs elements in three directions. The vertical spring stiffness was assumed infinitely rigid. The two orthogonal ones were considered in two cases. The first with almost real values evaluated as it is show in eq. 5, as recommended by Caltrans code (Priestley *et al*. 1996):

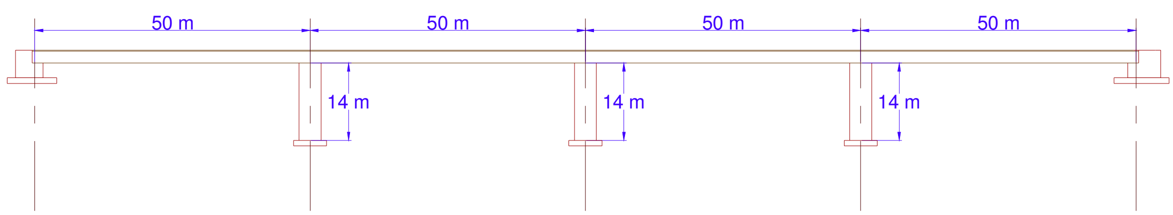
$$k_a = k_s + k_p = (115\,\text{kN/mm/m})B + (7\ \text{kN/mm})n_p \tag{5}$$

where k_a is the abutment initial stiffness, k_s and k_p are abutment piles and the backwall soil stiffness, respectively; B is the effective abutment width in the analysis direction and n_p is the number of piles. Considering real values for B and n_p, the k_a stiffness was evaluated. For the second option, the stiffness longitudinal spring of the abutment was selected such as to allow longitudinal mode shapes.

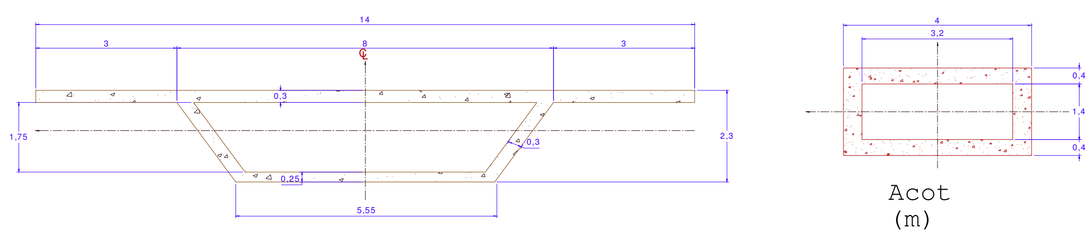
The piers were modeled with infinite stiffness at their base (soil-structure interaction effect was not considered), and with displacement connectivity at the girder union, assuming free rotation. In addition, to correctly capture the vertical modes of the bridge, the girder elements were discretized every 3.2 m, with concentrated mass. The girder nodes were constrains to equal longitudinal displacement.

Table 2. Geometric characteristics of the bridge elements.

Characteristic		Girder	Pier
Area	A (m^2)	6.8527	4.32
Moment of Inertia	Ix (m^4)	85.8023	7.9104
Moment of Inertia	Iy (m^4)	4.9577	2.8176
Section module	Sx (m^3)	12.2573	3.9552
Section module	Sy (m^3)	3.2046	2.5615
Radius of gyration	Rx (m)	3.5385	1.3532
Radius of gyration	Ry (m)	0.8506	0.8076



a) Front view of the bridge



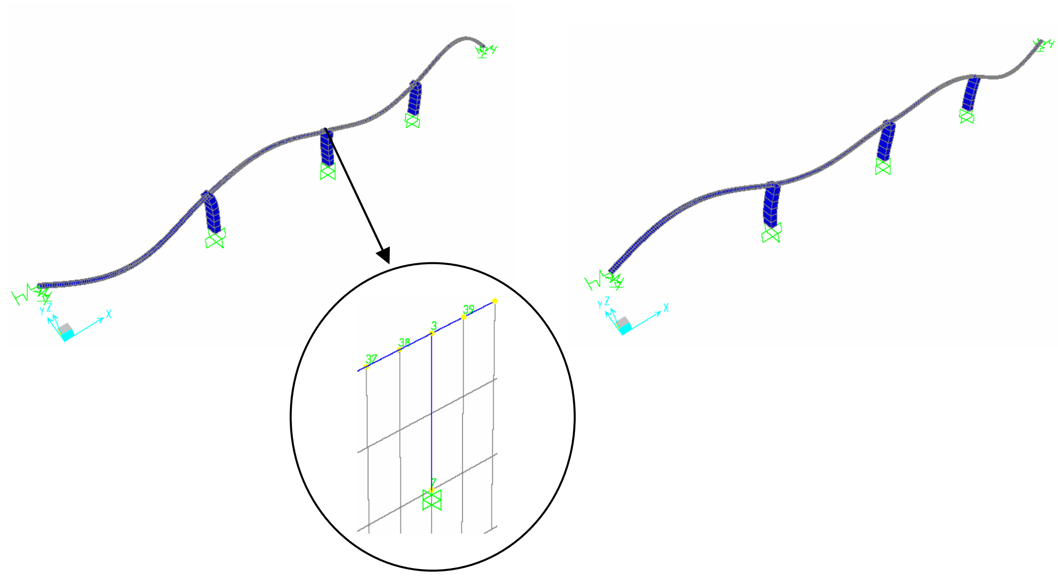
b) Girder (left) and pier (right) cross sections

Figure 2. Geometry of the bridge under study

The first four fundamental periods of the bridge and its orientation are shown in table 3 for the bridge with real and flexible abutments. In fig. 3, schemes of the models of the bridge in its first fundamental period are presented, also for real and flexible abutments stiffness.

Table 3. Bridge periods.

Real abutment stiffness			Flexible abutment stiffness		
Mode	**Period**	**Orientation**	**Mode**	**Period**	**Orientation**
1	0.481	Vertical	1	0.519	Longitudinal
2	0.462	Vertical	2	0.462	Vertical
3	0.449	Transversal	3	0.448	Transversal
4	0.367	Transversal	4	0.427	Longitudinal



a) With real abutment stiffness b) With flexible abutment stiffness

Figure 3. Fundamental periods of the bridge models

The responses of the bridge model with the action of accelerograms of fig. 1 were combined orthogonally, as in the case of displacements, or collinearly, as in axial forces, as described by Valdés (2005). Responses so combined were compared with the exact ones and errors between the two responses were defined as follows:

$$Error = \frac{R_{com} - R_e}{R_e} \tag{6}$$

where R_{com} is the response obtained with some of the proposed combination rules and R_e is the exact response. This former response is determined by the combination of history responses, time to time, for the three orthogonal directions of the bridge. In orthogonally or collineary form, for bidirectional analyses, the exact response is defined as

$$R_e = \left[\sqrt{R_X^2(t) + R_Y^2(t)}\right]_{\max} \qquad R_e = \left[R_X(t) + R_Y(t)\right]_{\max} \tag{7}$$

in which R_e is the exact response and $R_X(t)$ and $R_Y(t)$ are the history of the responses in the two orthogonal direction of the structure. In Eq. 6 we observe that a negative error means that the combination rule used underestimates the structure response, whereas a positive error is due to an overestimation.

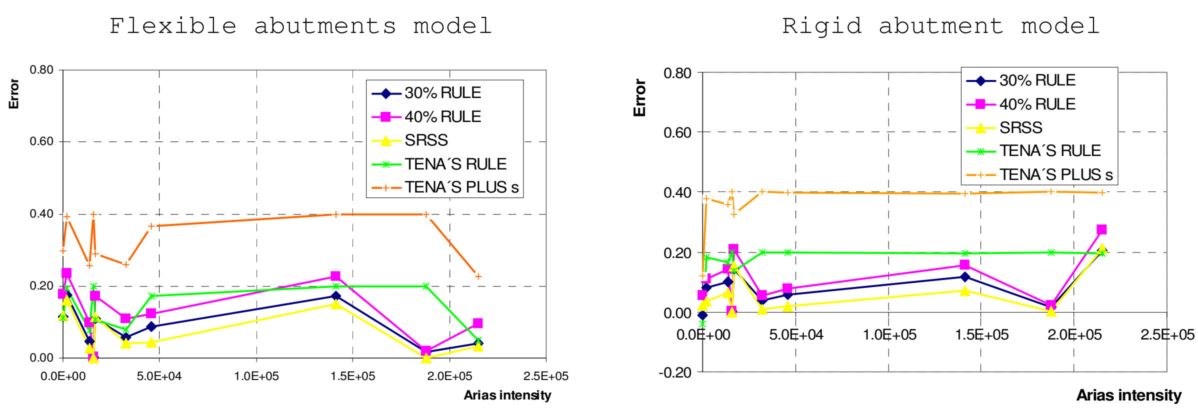
The fundamental periods of the bridge under study are 0.481 s and 0.591 s, thus, to apply the combination proposal of Tena and Perez (2006), statistical values of λ were extrapolated, considering a mean of 1.2 and a standard deviation of 0.2. Therefore, the bidirectional and three-directional responses was estimated for one mean and for a mean plus one standard deviation.

Because of lack of space we will only show the evaluation errors defined for the different combinations in one node, which is the one located at the bridge girder at the union with the intermediate pier, as is illustrated in fig. 3. In fig. 4 such errors are plotted for maximum displacements, with the three types of bidirectional combinations, $R_X + R_Y$, $R_X + R_Z$, and $R_Y + R_Z$, and the three-directional combination $R_X + R_Y + R_Z$, respectively. The *x, y* and *z* orthogonal directions of the bridge are show the bridge schemes of fig. 3. In each case the errors obtained are related to Arias' intensity of the earthquakes used, defined as

$$I_a = \frac{\pi}{g}\int_0^{\infty} \left[a(t)\right]^2 \, dt \tag{8}$$

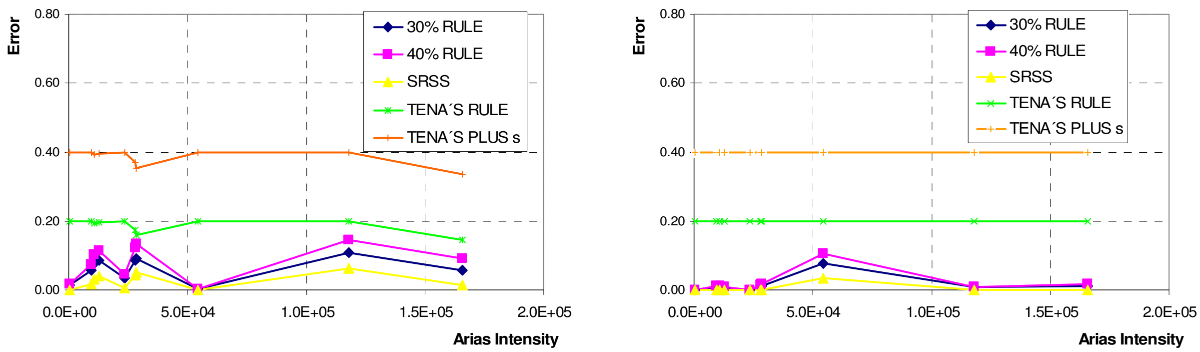
where I_a is the Arias´s intensity, g is the gravity acceleration and $a(t)$ is the acceleration record. The Arias intensity is considered as a representative parameter of the earthquake, due to the fact that its evaluation includes aspects of duration, amplitude and frequency content, as opposed to the maximum acceleration of the earthquake (the most common parameter), that represents only its amplitude.

The maximum bidirectional displacements with the SRSS and 30% rules estimate the least errors in displacements, as it can be observed in Fig. 4. The 40% rule is more conservative and the Tena´s rules, for a mean and a mean plus a standard deviation values of λ estimate error about 20% and 40%. Tena´s rules were defined for buildings structures and special soil conditions. It is necessary more studies to future applications in bridges.

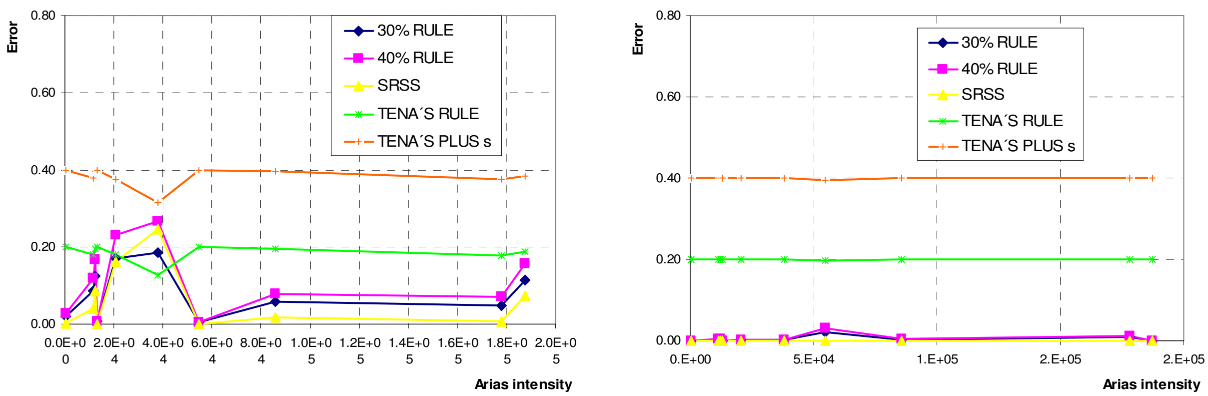


a) Combination $R_X + R_Y$

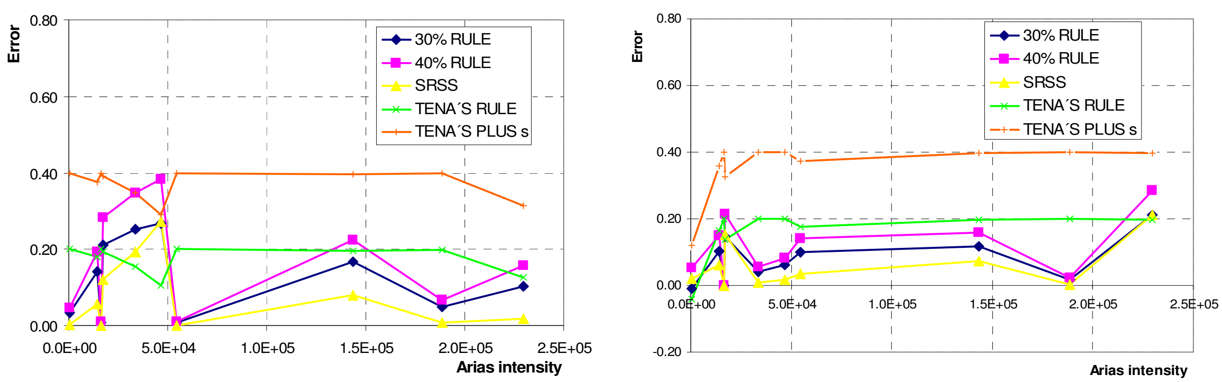
Flexible abutments model   Rigid abutment model



b) Combination $R_X + R_Z$



c) Combination $R_Y + R_Z$



d) Combination $R_X + R_Y + R_{Z\,Z}$

Figure 4. Maximum displacement errors *vs.* Arias' intensity

Final Comments

When a structure is designed it is difficult to predict the direction of action of the future seismic load. To face this problem, the design codes consider some ways of combining the orthogonal components of the earthquake acting independently. Some previous studies in buildings have shown that the different combinations vary compared to the exact response, in percentages that can be greater than 30%. There are few studies in respect to bridges, and more research is necessary in order to characterize more reliable combination factors.

In this paper, the errors obtained when estimating the combined responses to the independent orthogonal components are compared with the exact values. The analyzed responses include maximum displacements, axial, shearing forces and bending moments. Comparisons were made for a reinforced concrete highway bridges in elastic conditions, subject to a high acceleration seismic load in some of its components. As combination rules are considered five options used in design codes, such as 100% in one direction and 30% or 40% in the perpendicular, and the SRSS rule; or expressions recently proposed that take into account the dynamic characteristics of the structure and the condition of the surrounding soil. The errors obtained in elastic analyses are associated to the Arias' intensity of each one of the used records, due to the fact that this parameter integrates the greater variety of characteristics of a seismic movement in amplitude, duration and frequency content.

From the obtained results, we can deduct that the Tena´s rules are the most conservative in the definition of the maximum displacements, reaching maximum error values near to 40%. The SRSS and the rule of 100% in one direction and 30% in the perpendicular are the ones that better approaches the exact values. The Tena´s rule produces important errors because is defined for isolated buildings with important fundamental period, compared with the modal characteristics of the used bridge.

Although the error values are not the same, the conclusions of the applicability of the combination rules are similar for the bridge with real and flexible abutments. The rigid abutment model has small participation of the vertical component, as can be shown in figs. 4 b) and c), compared with the model with flexible abutments. If a three-directional combination is considered, the errors tend to be smaller.

The obtained results present similar tendencies with those reported for buildings in the literature, principally for the 30%, 40% and SRSS rules. Although not shown, the resulting errors when combining the maximum responses in axial forces and shearing force follow similar tendencies to those commented above for maximum displacements.

This study should be complemented with bridge models of irregular superstructure and substructure. It will also be important to complement these results including more records from different seismic zones in Mexico and in the World. Actually, two more combination rules are studied, the CQC3 method and Valdés procedure, which considers the random vibration theory to formulate expressions to a percentage combination rule for buildings. With these analyses, a more complete judgment could be made about the different combination rules proposed.

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