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Superstructure irregular effect on seismic bridge behavior

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ABSTRACT

Evaluations of different irregular conditions in bridges subjected to seismic load, considering systems with length relations of extreme girder to central girder of 1:0.25, 1:0.5, 1:0.75, 1:1.25, 1:1.50, 1:1.75 and 1:2, and structures with curvatures of 30º, 60º, 90º, 120º, 150º and 180º; is analyzed in this paper. Seismic responses of irregular and regular (relation of lengths 1:1 and curvature of 0º) systems were obtained for maximum displacements and mechanics elements. The seismic loads used were a database of 53 seismic signals, recorded in stations located in one of the most hazardous zones of México. Elastic and nonlinear analyses were accomplished and the normalized differences between regular and irregular models were evaluated. This parameter was organized by quartiles to define trends of irregular bridge behavior.

1. INTRODUCTION

Irregular structures have a more complicate behavior, thus their analyses; inspection and maintenance need more attention. Bridge irregularity is due to skew and curvature superstructures, systems with variations in girders strength, or piers with not uniform height or resistance (substructure irregularity). In addition, bridges are considered irregular systems when they present important stiffness variations between superstructure and substructure elements. Some authors, as Isakovic and Fischinger (2005), consider a bridge as irregular, if it has important contribution of higher modes to its elastic response.

In literature there are different studies about the behavior of bridges with irregular substructures. In them, it is defined that demands of pier deformations are greater variables, then the highest columns work principally to flexion and the shortest to shear (Moehle and Eberhard, 2000). Some problems in bridges with piers of different heights

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are: irregular deformation demands in piers, concentration of seismic shear loads in the shortest elements by stiffness variation, changes in ductility demands in piers; torsion problems, and more participation of higher modes in bridge response (Kappos *et al*. 2005; Isakovic and Fischinger, 2008). These conclusions were verified in some works. For example, experimental analysis in ELSA laboratory compared the behavior of two bridges, one regular and symmetric and the other irregular, with different length of the three piers. The conclusion in this study was than absorbed energy in irregular system was concentrate in the shortest pier, with more than 70% of dissipation of the total energy (Tehrani and Mitchell, 2010). For bridges with irregular conditions there are not extensive studios to define their seismic behavior.

1.1 Bridge irregularity in design codes

AASTHO code (American Association of State Highway Transportation Officials, 2007) considers that an irregular bridge have a curvature angle not greater than 90º, maximum thresholds of lengths between adjacent spans, and stiffness relations between piers bents minors than certain limits, in function of the total number of spans. For multiple-span curved bridges; the irregular condition is assigned when the curvature angle is greater than 20º.

Transportation Department of California (Caltrans) indicates that an equivalent static analysis is accomplished when bridge is regular and ordinary, while specify dynamics spectral analysis for not ordinary and important irregular systems. In this code, the irregular conditions are assigned of a subjective form, considering curved structures, with more than one story, with variable sections, of not balanced mass, with variations in pier stiffness or skew systems. Eurocode (Escamilla *et al*., 2011) classified bridge regularity in function of a ductility factor, so more ductility is related to more irregular behavior of the structure. However, studies of irregular bridges (Tehrani and Michell, 2010), show that the recommended design procedure by Eurocode could generates smaller levels of security of this structures, with ductility demands greater than the expected ones.

In summary, the irregular condition of a bridge is used to define the specific analysis method to design the system. When the structure is classified as irregular, more rigorous and elaborated analysis methods are recommended. However, bridge irregular classification is very simple and subjective, and could generate systems with erroneous security levels.

1.2 Irregular indices

Recently, some indices have been proposed to classified irregular behavior of bridges. The objective of these indices is to predict if a bridge will have a behavior as it was defined in design process (Isakovic and Fischinger, 2000). The named bridge irregular indices are classified in elastic and nonlinear. The first ones consider that a bridge could be classified as irregular only with an elastic conduct, so its formulation is based only on elastic parameters. The nonlinear irregular indices define that no regular condition of a structure is only present when the system have inelastic performance. In the evaluation of these indices, accumulated damage is analyzed, generally using normalized transversal deformation. In Escamilla *et al*. (2011) a comparison of the obtained values of some irregular indices for bridges with variations in the piers length is presented. Results show that the obtained values for four irregular indices for the same structure are different, so these indices are not an absolute evaluation of bridge irregular condition.

Then, irregular indices could be a tool to characterize the irregularity of bridges. However, some aspects could be studied to facilitate the practical application of these indices. For example, what does it mean an irregular index of 0.45?, some times denominated as a mean irregularity or semi-irregularity (Pinho *et al*., 2007). Nowadays, it is not clear what does it mean.

1.3 Irregularity in inspection and maintenance programs

To define maintenance programs in bridge structures, regular inspection most to be accomplished. When a numerous group of bridges are evaluated, preliminary inspection methods are used, in which the structure capacity is classified by means of a vulnerability index. Some preliminary inspection methods have been proposed for bridges, although it is common, in most of them, to consider the bridge irregularity as an important parameter to define seismic behavior. For example, in the Kim method, the seismic vulnerability of bridges is defined pointing 12 parameters, some of them related to superstructure and structure irregularity. In table 1 the parameters, vulnerability categories and its weight of Kim method are showed, highlighting the parameters that evaluate substructure and superstructure irregular conditions.

In México, the Secretary of Communications and Transports uses a preliminary inspection method, named SIPUMEX, to classified bridge current conditions. Results of this procedure are used to assign restricted economic resources to maintenance tasks. In this method, superstructure irregularity is not considered, and substructure irregularity is evaluated analyzing pier length variations.

The simplified form to consider the bridge irregularity in design codes and the insufficient reliability of simplified preliminary inspection methods to reflect the influence of irregularity parameter, defined the importance of more studies. Then, a parametric study of the seismic behavior of different bridge with superstructure irregularity is achieved in this work, considering variations in girders length and curved bridges.

2. BRIDGE MODELS

To analyze the influence of irregular superstructure conditions a bridge model available in literature (Priestley et al., 1996) was employed; the same used by Gómez and Salas (2012) to study substructure irregular conditions. This bridge has four spans of 50 m and three piers of 14 m, as it is illustrated in Figure 1a. Deck is composed by a unicellular box section and piers have box transversal section, with dimensions indicated in Figure 1b. For this model, two variations were applied to evaluate irregularity by girder length variations, one with four spans and another with five spans (see Figure 2), to evaluate if the distance of the irregular span might have influence of the bridge behavior.

Table 1 Parameters, vulnerability categories and weighs of Kim method

Parameters	Categories	W
Peak ground acceleration	1: *PGA* <0.1g; 2: 0.1g < *PGA* < 0.2g; 3: 0.2g < *PGA* < 0.3g; 4: *PGA* > 0.3g	0.141
Design specifications	1: after 1981; 2: 1972-1980; 3: 1940-1971; 4: before 1940	0.456
Type of superstructure	1: cable-stayed, suspension, single span; 2: arc, monolithic girder, trusses; 3: continuous girder, trusses; 4: simple-supported girder and trusses, multiple spans, elevated structures	0.114
Shape of the superstructure	1: straight; 2: skewed 20 - 45° or curved 40 - 90°; 3: skewed 45 - 60° or curved 90 - 180°; 4: > skewed 60° or curved 180°	0.437
Internal hinges	1: none; 2: yes, with cable restrainers or seat length 12 "; 3: yes, with 6" < seat length < 12"; 4: yes, with seat length < 6"	0.089
Type of pier	1: monolithic multi-pier bent or solid; 2: pinned multi-pier bent; 3: monolithic single pier; 4: pinned single pier	0.029
Type of foundation	1: single pier shaft; 2: spread footing; 3: piled footing; 4: pile bent	0.024
Material of the subsest.	1: steel; 2: ductile concrete; 3: no-ductile concrete; 4: timber, masonry, other old materials	0.034
Structural irregularity	1: none; 2: any heights of 2 piers ≥ 1.25 times; 3: any adjacent heights of 2 piers ≠ more than 1.25 times; 4: any adjacent pier heights ≠ more than 1.5 times	0.278
Soil condition	1, 2, 3 o 4 for different soil types	0.188
Liquefaction	1: LSI* < 5; 2: 5 < LSI < 25; 3: 25 < LSI < 100; 4: LSI > 100	0.932
Seat length	1: good; 2: fair; 3: poor; 4: very poor	0.512
*LSI = Youd and Perkins factor that characterize the effect of liquefaction [6]		

Superstructure irregularity was evaluated considering changes in the length of spans and curved structures. For the first option, ratios of extreme and central girders lengths of 1:1, 1:0.25, 1:0.5, 1:0.75, 1:1.25, 1:1.5, 1:1.75 and 1:2 were studied, being the ratio 1:1, the one for the regular bridge. For the second option, the bridge models, based on the one from Figure 1a, were elaborated with curvature angles of 30°, 60°, 90°, 120°, 150° and 180°, as it is observed in Figure 3. Maybe some of the irregular models are not very accurate, but they complement the study of bridges with superstructure irregularity.

All bridges were modeled in SAP (2000) program. In these models, piers are entirely supported and abutments were considered with three elastic strings. The lateral stiffness of the elastic strings was defined using simplified expressions defined in Priestly *et al*. (1996). Abutments were assumed extremely strong in vertical direction.

To ponder the connection between piers and girder elements, three types of bridges were assumed: continuous, simple-supported and monolithic. To model monolithic bridge, rigid pier-girder connection was assumed, while for continuous bridges the longitudinal moment was free type. For simple-supported models, fixed and roller supports were located in extremes of beam elements. In the last one, the longitudinal rotation was unrestraint, meanwhile for the other support, the longitudinal moment and rotation is free.

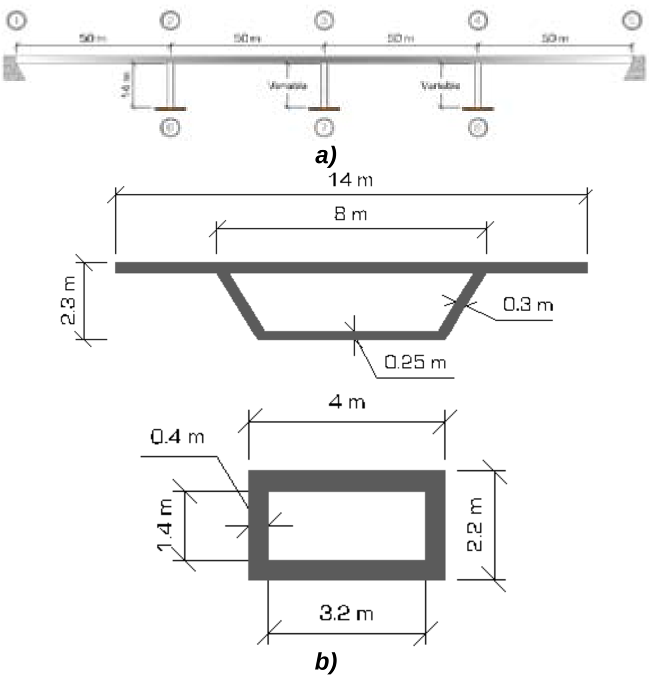


Fig. 1 Regular model, a) elevation, b) girder and pier transversal section

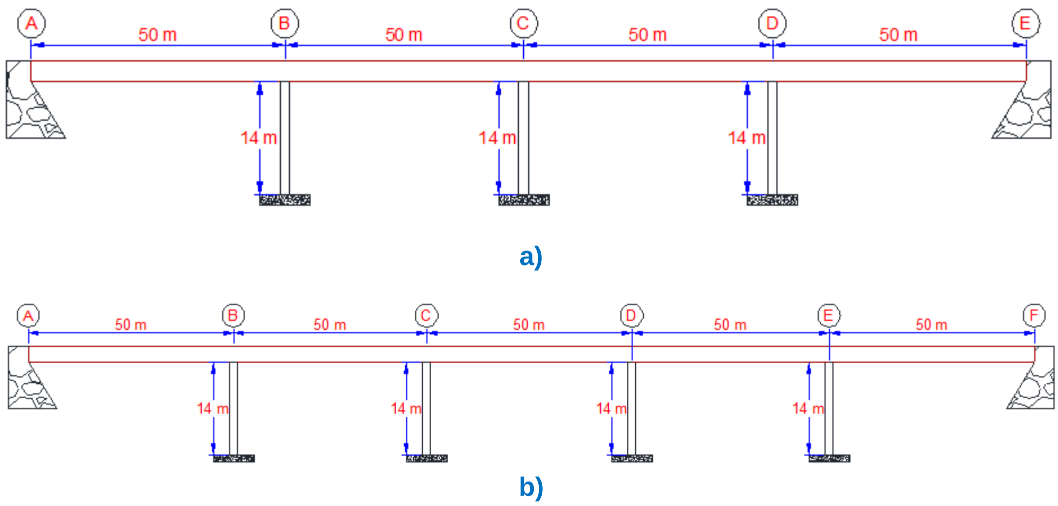


Fig. 2 Regular models with four (a) and five (b) spans

To represent the influence of diverse superstructure irregular conditions, normalized difference parameter between regular and irregular responses is used. Normalized difference is defined as:

$$D_{if} = \left\lfloor \frac{R_{ir} - R_r}{R_r} \right\rfloor \tag{1}$$

where D_{if} = percentage of normalized difference of regular and irregular response, R_{ir} = maximum response of irregular bridge and R_r = maximum response of regular bridge. Mean and standard deviation of D_{if} variable were also evaluated. Regular and irregular responses are maximum displacements and maximum mechanic elements, but this paper only presents, because of the space limit, the results of maximum displacements of central nodes (node C in Figure 2).

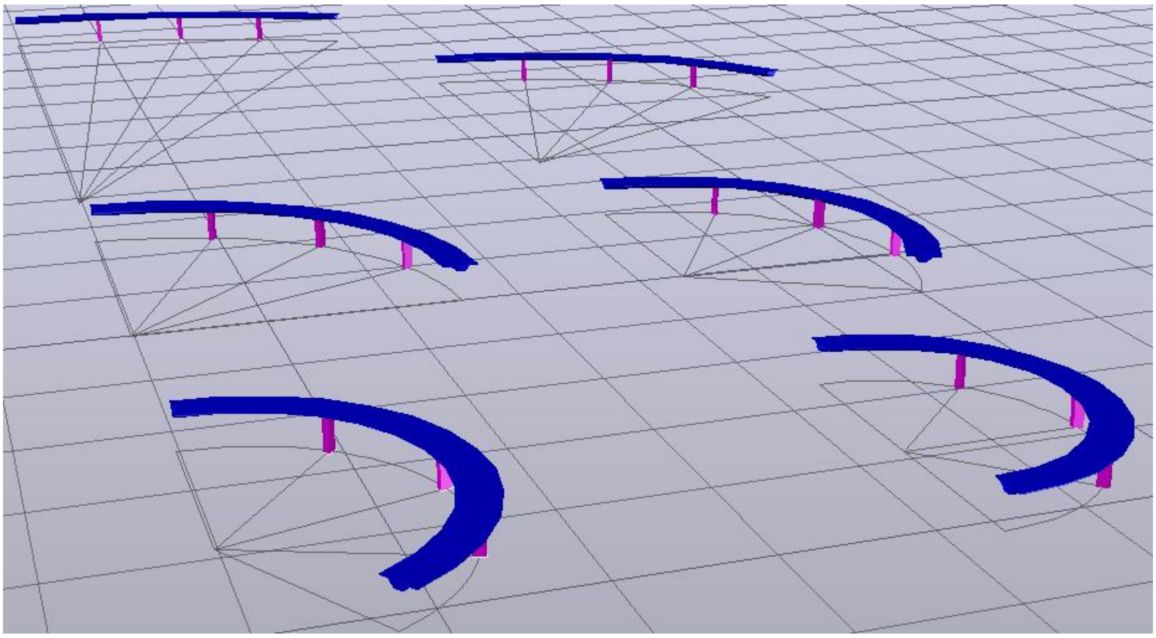
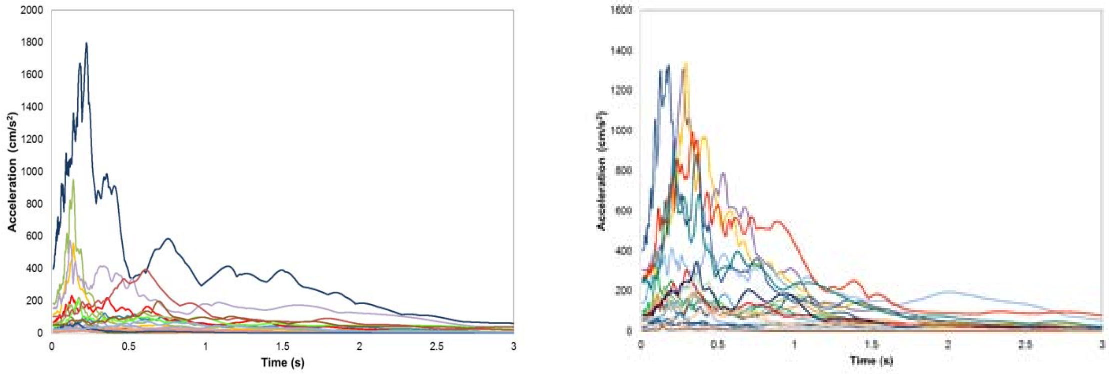


Fig. 3 Curved models, with curvature angles of 30° to 180°

3. EARTHQUAKE LOAD

Seismic load was applied to all bridge models. This load was defined by 53 seismic signals in three components; obtain from the Strong Ground Motions Mexican Database (BMSF, 2000). The selection of signals was function of their magnitude, peak ground acceleration or velocity, and the location of seismic stations, in the Mexican states of Colima, Guerrero y Michoacán. Only one of the two horizontal components of the signals, the one with greater amplitude, was applied in bridge transversal direction, in order to evaluate the most critical option. Vertical component was not applied, although some studies indicate that it has influence for some relations between dynamic bridge and excitation characteristics. The Figure 4 shows the response spectrums for the horizontal component with greater amplitude, for a 5% of the damping ration. As it can be observed in this Figure, there are an important variety of records; nevertheless, most of them have fundamental periods minor than 0.5 s.



a) Colima y Guerrero states b) Michoacán state

Fig. 4 Elastic spectrums regular model, a) elevation, b) girder and pier transversal section

4. RESULTS FOR BRIDGES WITH VARIATIONS IN GIRDERS LOENGHT

Dynamic properties of models were defined by means of a modal analysis. The fundamental periods of regular and irregular models are presented in tables 1 and 2, for models with four and five spans, respectively. In these tables, it is observed, the change of the relation of extreme and central lengths generate important variations (greater than the ones achieved for bridges with substructure irregularity, as showed by Gómez and Salas, 2012). For example, for a four-span, simple-supported system, an irregular condition of 1:2 produces a variation of fundamental period of 230%, compared with the fundamental period of the regular bridge. With these differences it is difficult to say if the response variations are due by the change in bridge dynamic properties of by irregular conditions. Then, for the irregular bridge with relation 1:2 an additional model was elaborated, with the modification of the mass to produce a bridge with a fundamental period similar to the period of the regular model. Results for this additional model show that the normalized differences of the maximum transversal displacements are similar (minor than 10%) to the original model. So, it is concluded that the responses differences are related principally to bridge irregular conditions and to original models utilized.

Normalized responses are very variable, then values obtained with Equation 1 were organized by quartiles, to define responses trends. In Figure 5 the normalized differences of maximum displacements of node C (Figure 2) for monolithic bridges with four spans are showed. As it can be observed, for greater change in the length of the extreme span, related with the length of central span; the dispersion of the results is greater. Also, there is more dispersion of the results when the extreme length is reduced, than when it is increased. Then, it can be say that bridges with extreme spans length greater than the central spans have more variations in their responses, and thus, minor prediction in design process and more vulnerability.

Table 2 Fundamental periods of bridges with variations in extreme girder length. Models with four spans

Models	**Monolithic T(s)**		**Simple-supported T(s)**		**Continuo T(s)**	
	1	**2**	**1**	**2**	**1**	**2**
Regular	0.456	0.438	0.700	0.700	0.608	0.522
Irregular 1:0.25	0.390	0.356	0.700	0.700	0.428	0.390
Irregular 1:0.50	0.406	0.364	0.700	0.700	0.457	0.406
Irregular 1:0.75	0.415	0.376	0.700	0.700	0.493	0.415
Irregular 1:1.25	0.668	0.657	0.906	0.906	0.785	0.734
Irregular 1:1.50	0.937	0.927	1.303	1.303	1.071	1.023
Irregular 1:1.75	1.256	1.243	1.773	1.773	1.416	1.369
Irregular 1:2	1.622	1.608	2.315	2.315	1.811	1.751

Table 3 Fundamental periods of bridges with variations in extreme girder length. Models with five spans

Models	**Monolithic T(s)**		**Simple-supported T(s)**		**Continuo T(s)**	
	1	**2**	**1**	**2**	**1**	**2**
Regular	0.449	0.447	0.701	0.701	0.605	0.544
Irregular 1:0.25	0.414	0.377	0.701	0.701	0.484	0.414
Irregular 1:0.50	0.420	0.381	0.701	0.701	0.501	0.420
Irregular 1:0.75	0.423	0.387	0.701	0.701	0.521	0.423
Irregular 1:1.25	0.583	0.488	0.906	0.906	0.845	0.760
Irregular 1:1.50	0.934	0.930	1.303	1.303	1.170	1.048
Irregular 1:1.75	1.252	1.247	1.773	1.773	1.555	1.389
Irregular 1:2	1.618	1.612	2.315	2.315	1.995	1.780

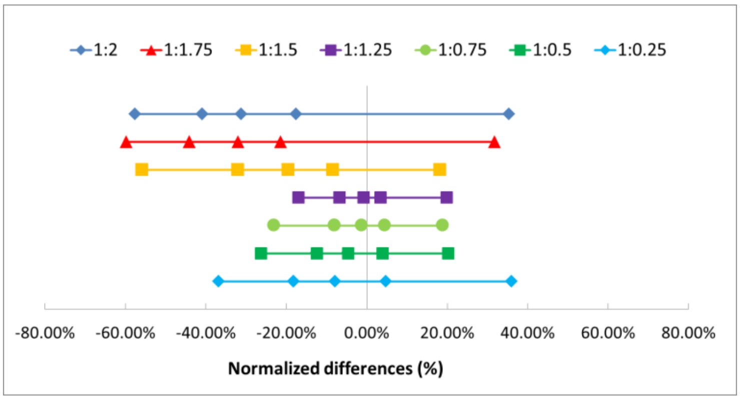


Fig. 5 Normalized displacements for continuous bridges with four spans

Figure 6 show the same results than in figure 5, but for monolithic bridges with five spans. In this figure is observed similar tendencies of the ones described for Figure 5, however the dispersions are greater for the same relations for the models with four spans. For example, for the models with relation 1:2, maximum normalized displacements are of 65%, 15% greater than the normalized displacements for the same bridge type with four spans. Then, it can be assumed that while more distance are between central bridge and the spans with different length, more dispersion is presented in the results. More analyses are necessary to validate these comments.

For continuous and simple-supported bridges normalized differences by quartiles are showed in Figures 7 to 10, for systems with four and five spans. Similar trends are observed in these figures. Normalized responses of monolithic, continuous and simple-supported systems have responses with slight variations, so it is possible to indicate that bridge type does not have greater influence.

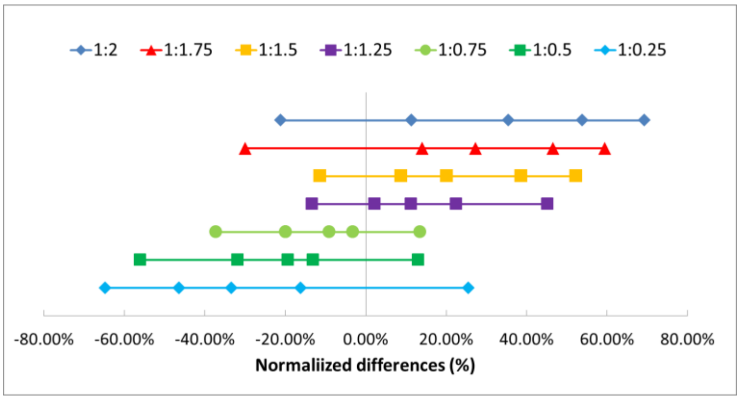


Fig. 6 Normalized displacements for continuous bridges with five spans

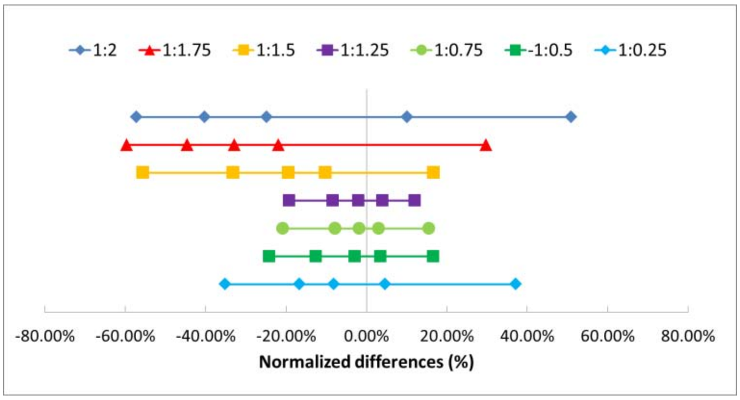


Fig. 7 Normalized displacements for monolithic bridges with four spans

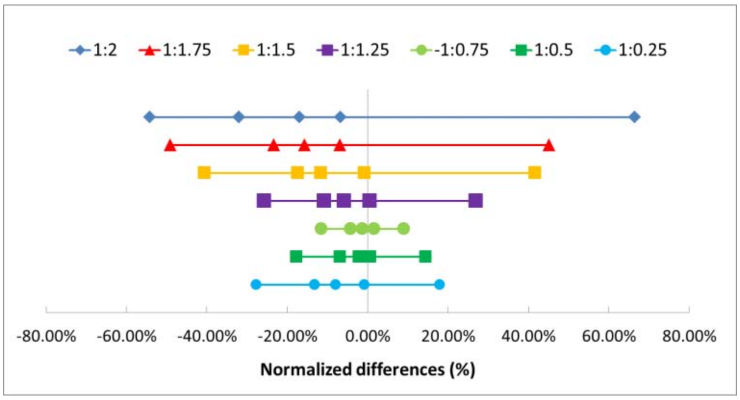


Fig. 8 Normalized displacements for monolithic bridges with five spans

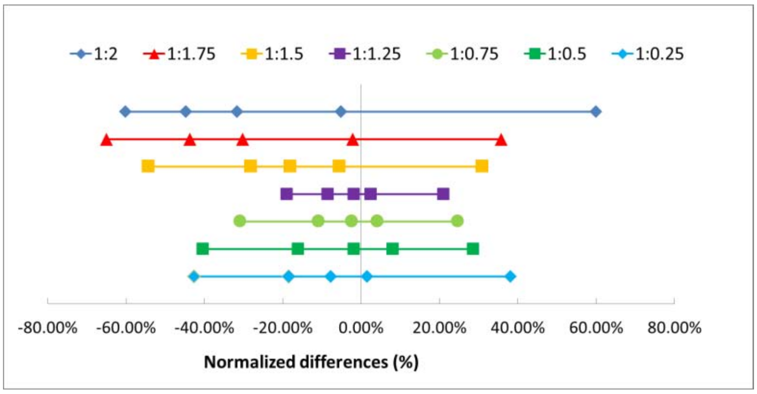


Fig. 9 Normalized displacements for simple supported bridges with four spans

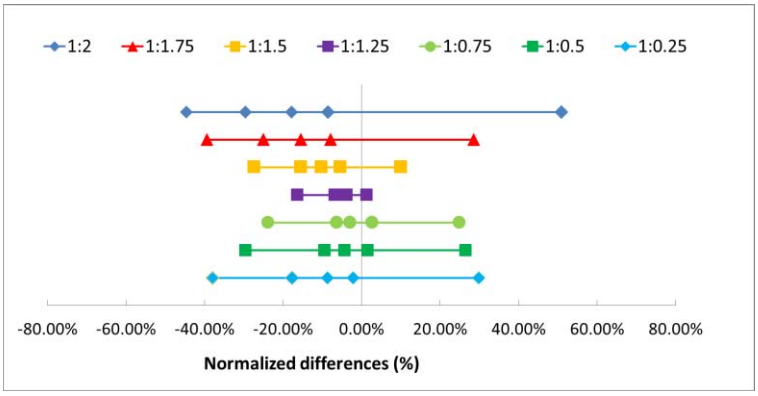


Fig. 10 Normalized displacements for simple-supported bridges with five spans

5. RESULTS FOR CURVED BRIDGES

For curved bridges only monolithic and continuous bridges were analyzed, because it was determined that for small curvature angles, simple supported systems present tension forces in the connection of deck and bearing.

Fundamental periods of curved models are indicated in Tables 4 and 5, for bridges with four and five spans, respectively. In these tables, compared with Tables 2 and 3, lesser variations between regular and irregular bridges fundamentals periods, are observed.

Table 4 Fundamental periods of curved bridges. Models with four spans

Models	**Monolithic T(s)**		**Continuo T(s)**	
	1	**2**	**1**	**2**
Regular	0.456	0.438	0.608	0.522
30°	0.498	0.416	0.603	0.495
60°	0.462	0.444	0.589	0.508
90°	0.470	0.451	0.600	0.516
120°	0.480	0.461	0.615	0.527
150°	0.493	0.473	0.635	0.542
180°	0.508	0.488	0.660	0.560

Table 5 Fundamental periods of curved bridges. Models with five spans

Models	**Monolithic T(s)**		**Continuo T(s)**	
	1	**2**	**1**	**2**
Regular	0.449	0.447	0.605	0.544
30°	0.451	0.448	0.582	0.527
60°	0.454	0.451	0.586	0.530
90°	0.458	0.456	0.593	0.536
120°	0.465	0.463	0.601	0.545
150°	0.472	0.472	0.612	0.555
180°	0.482	0.482	0.628	0.568

Figures 13 to 16 show the normalized differences of curved bridges, continues and monolithic types, for systems with four and five spans. In these figures it is observed that with a more curvature angle of the deck, related a right and regular bridge, results present more dispersion, until to a curvature angle of 90º. For example, for the monolithic models with four spans and a curvature angle of 30º, a maximum normalized displacement of 18% was defined. For a bridge with curvature angle of 90º, this value was of 68%. Also, from these figures, it is defined that there are minor dispersion in results of bridges with curvature angles greater than 90º. In bridges with five spans, similar dispersions were calculated, compared with models with four spans.

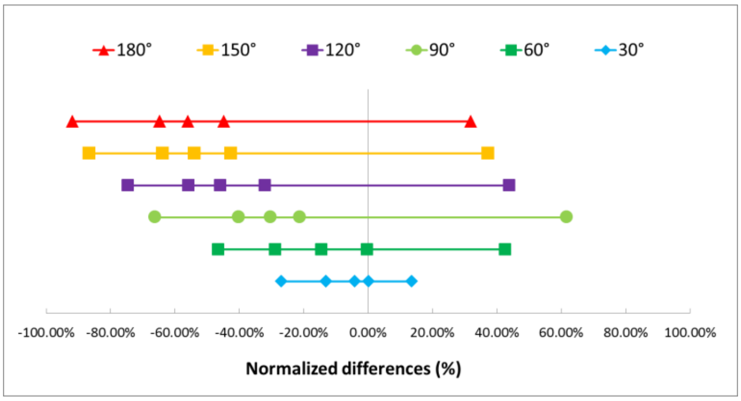


Fig. 11 Normalized displacements for curved continuous bridges with four spans

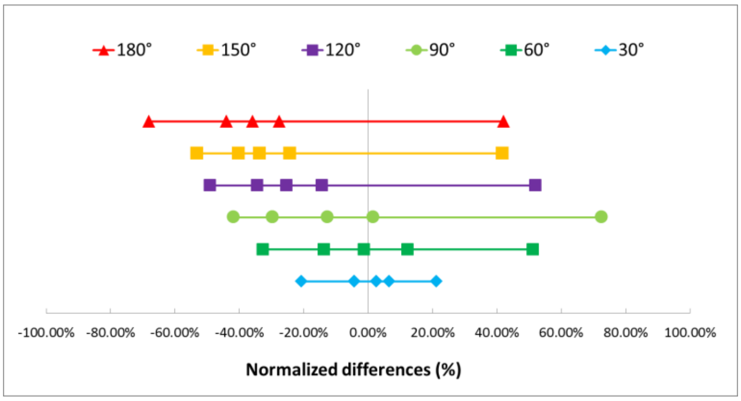


Fig. 12 Normalized displacements for curved continuous bridges with five spans

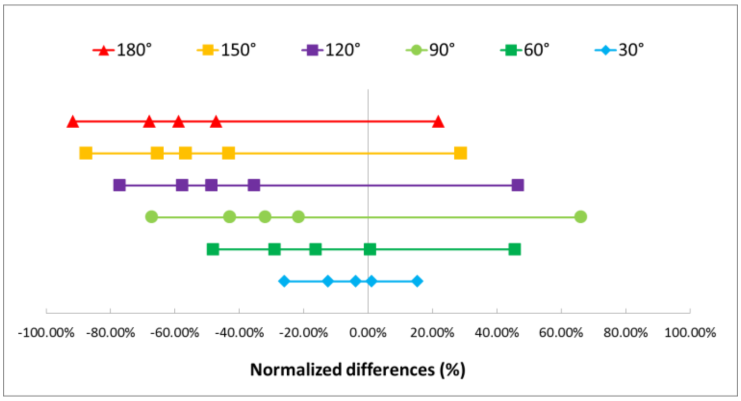


Fig. 13 Normalized displacements for curved monolithic bridges with four spans

In general, significant changes are not observed when the normalized differences of monolithic and continuous bridges are compared. Thus, these bridge types have not important influence in the response of irregular systems.

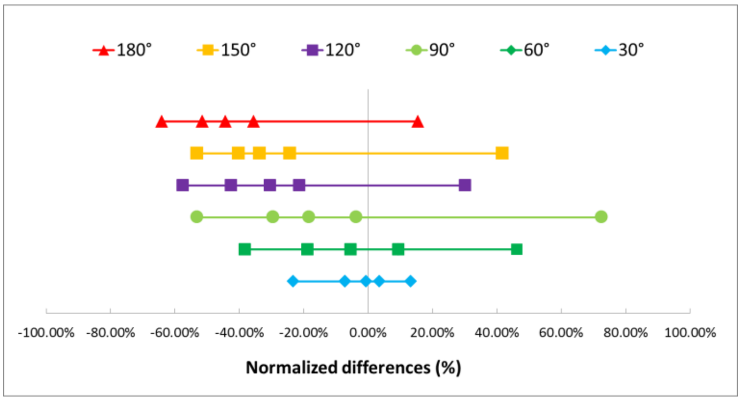


Fig. 14 Normalized displacements for curved monolithic bridges with five spans

6. SUMMARY OF THE ANALYSES

In Table 6, quadratic functions of the trend lines for the 100% quartile are presented, for irregular systems with variations of the extreme girder length and for all considered bridge types. Similar equations for curved bridges are indicated in Table 7.

Beginning with Tables 6 and 7, two parameters are proposed to characterize the superstructure vulnerability to seismic action, showed in Tables 8 and 9. In these tables vulnerability categories and their weights are also indicated. Parameters, vulnerability categories and weights, could substitute the ones highlight in Table 1 for Kim method. The proposed values were obtained by means of analytical studies.

Table 6 Quadratic functions of maximum normalized displacements

Models	**Quadratic functions, models with four spans**
Monolithic	$I_d = 43.99r_c^2 - 90.95r_c + 55.63$
Simple-supported	$I_d = 36.26r_c^2 - 71.68r_c + 55.22$
Continuo	$I_d = 24.46r_c^2 - 52.44r_c + 44.63$
	Quadratic functions, models with five spans
Monolithic	$I_d = 25.81r_c^2 - 29.47r_c + 22.02$
Simple-supported	$I_d = 44.81r_c^2 - 95.73r_c + 58.80$
Continuo	$I_d = 15.68r_c^2 - 3.35r_c + 17.30$
I_d = Maximum normalized displacement (%) r_c = Relation of lengths of extreme and central girder	

Table 7 Quadratic functions of maximum normalized displacements for curved bridges

Models	Quadratic functions, models with four spans
Monolithic	$I_d = -0.007c^2 + 1.38c - 15.55$
Continuo	$I_d = -0.005c^2 + 1.2c - 13.11$
	Quadratic functions, models with five spans
Monolithic	$I_d = -0.007c^2 + 1.44c - 18.31$
Continuo	$I_d = -0.005c^2 + 1.2c - 4.61$
I_d = Maximum normalized displacement (%) c = Curvature	

Table 8 Vulnerability categories for different relations of extreme and central lengths

Parameter	***Categories***
	1.0: Relation 1:1
Y_3 Superstructure irregularity	3.5: Relation 1:0.25
	2.5: Relation 1:0.5
	1.9: Relation 1:0.75
	2.8: Relation 1:1.25
	3.5: Relation 1:1.5
	4.5: Relation 1:1.75
	5.0: Relation 1:2

Table 9 Vulnerability categories for different curvature angles

Parameter	***Category***
Y_3 Superstructure irregularity	1.0*:* None
	1.9: 30°curvature angle
	4.0: 60° curvature angle
	5.0: 90° curvature angle
	4.2: 120° curvature angle
	3.5: 150° curvature angle
	2.9: 180° curvature angle

7. CONCLUSIONS

The analyses of highway bridge with different superstructure irregularity, are showed in this work, Irregular conditions were analyzed by changes of extreme girder length, in relation to central girder length, and of deck curvature angles. Elastic analyses of regular and irregular monolithic, continuous and simple-supported structures were performed. As seismic load, 53 signals were used, registered in stations located in Mexican Pacific coast. From the obtained results, normalized differences between regular and irregular structures were calculated. These normalized differences were organized by quartiles, to define trend lines.

Having as beginning the elastic analyses for bridges with variation of the extreme girder length, some conclusions are:

- To greater change in length of the extreme span, compared to the dimension of the central span, greater dispersion is defined in the normalized differences of maximum displacements.
- Greater dispersion and more differences in displacements were obtained for bridges with five spans, compared to the obtained with four-span bridges.
- Monolithic and simple-supported bridges have more differences in normalized displacements than continuous structures, although differences are small. So, these bridge typologies have minor influence in the normalized differences.
- For monolithic and continuous bridges with five spans, the previous comment is also valid, but dispersions are greater than the one of bridges with for spans and the same percentages of variation.
- Greater dispersions were calculated when the length of the extreme span was increased, compared with systems where the extreme span length was diminished.
- For curved bridges, greater differences in normalized displacements were determined for system with a curvature angle of 90º. For greater curvature angles, differences were reduced. Similar trend were calculated for bridges with four and five spans.
- Simple-supported curved bridges were not analyzed, because these structures have tension forces in the connection are between the deck and bearings.
- For curved bridges more statistical range are calculated, compared with structures where the extreme length was changed. This comment indicates more variability in the curved bridges responses, thus their seismic performance is lesser predictable in design process.

More studies are necessary to verify the aforementioned comments. For example, seismic accelerograms with other specific characteristics need to be evaluated. Also, other common highway bridge typology should be analyzed.

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