Correlation of Earthquake Ground Motion and the Response of Seismically Isolated Bridges

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Abstract—**The seismic response of bridges seismically isolated by lead-rubber bearings (LRB) to earthquake excitations of different magnitudes is presented in this thesis. The force-deformation behavior of LRB is considered as bilinear. The specific purpose of the study is to assess the effect of seismic isolation on the peak response of bridges subjected to different base accelerations ranging from 0.05g to 0.5g in the horizontal direction transversal to the bridge axis. Thus a certain level of efficiency can be shown in respect of the particular bearing modification. Furthermore, the effect of superstructure stiffness (i.e. pier stiffness, pier layout) on the isolator efficiency is investigated in depth. The seismic response in the finite element model of the continuous span isolated bridges is obtained by solving the governing equations of motion in the incremental form using an iterative step-by-step method. To study the effectiveness of LRB, the seismic response of isolated bridges is also compared with the response of corresponding nonisolated bridges (i.e. bridges without isolation devices). The important parameters included are the flexibility of the substructure and the ground acceleration. The results show that specific design of the isolation device has a significant effect on the seismic response of the isolated bridges. The efficiency of LRB in the bridges subjected to low seismic activity is considerably smaller due to the lead mostly remaining in the elastic range. The LRB can in this case therefore not contribute hysteretic damping which is, however, the great benefit of LRB from the design point of view. Stiffer structures tend to improve the isolator efficiency and show better performance in case of low damping.**

Keywords: seismic isolation, lead-rubber bearing, isolator efficiency

1 Introduction

The control of structures to improve their performance during earthquakes was first proposed more than a century ago. But it has only been in the last 30 years that structures have been successfully designed and built using earthquake protective systems. Today these systems range from simple passive devices to fully active systems. Major developments in the theory, hardware, design, specification, and installation of these systems have permitted significant applications to buildings, bridges, and industrial plants. Applications are now found in almost all of the seismically active countries of the world, but principally in Italy, Japan, New Zealand, and the United States. There are, however, limitations to the use of passive systems, which deserve further study and research. They include the uncertainty of response in the near field of an active fault, the non-optimal behavior of passive systems for both small and large earthquakes, and a lack of certainty about the ultimate limit states in unexpectedly large events. This study shall provide deeper understanding of the behavior of passive control devices for seismic excitations of different magnitude.

There have been several analytical studies in the past to demonstrate the effectiveness of seismic isolation for earthquake resistant design of bridges. Li (1989) developed a procedure for optimal design of bridge isolation systems with hysteretic dampers. He concluded that hysteretic damper act most effectively when mounted on a stiff supporting structure. Their effectiveness decreases with increasing flexibility of the supporting structure. Moreover is concluded that the larger the value of maximum allowed isolator displacements is used the more effective is the isolation system.

Iemura et. al. (1998) demonstrated that when seismic isolation systems are installed, a reasonable inelastic design method is required. A design procedure for isolated bridges is proposed using the relation between seismic isolation systems and piers with respect to their energy dissipation. Also parametric studies have been carried out to investigate the optimum characteristics of seismically isolated structures. The parameters mostly considered include time period of the superstructure and time period, damping, and yield characteristics of the isolator, and the ratio of predominant frequency of excitation to the frequency of the isolator.

Ghobarah (1988) studied the seismic response of single and two-span highway bridges with LRB modeled as bilinear spring. The influence of pa-rameters like the isolators stiffness, pier stiffness and pier eccentricity on the effectiveness of seismic isolation was investigated. Ghobarah and Ali (1988) studied the effect of deck stiffness on seismic responses.

Reinhorn et. al. (1998) examined the effects of variation

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of the ratio of isolator and pier yield char-acteristics on the response of isolated bridges. It has been recognized that, due to low redundancy and domination of the deck mode of vibration, isolated bridges are extremely sensitive to the characteristics of the ground motion.
Kawashima and Shoji (1998) presented an analysis on the interaction with emphasis on the yield force level of the isolator device. It was found that the post-yield stiffness and the yield force level of the device are important to predict the nonlinear response of the pier.
Koh et. al. (2000) developed a method to evaluate the cost effectiveness of seismic isolation for bridges in low and moderate seismic regions in order to calculate the minimum life-cycle costs of seismically isolated bridges under specific acceleration levels. The results show that seismic isolation is more cost effective in low and moderate seismic regions than in high seismic regions.

2 Method of calculation

Since the inelastic behavior of the bearings is of interest there is a need to consider the appropriate bearing reaction to seismic load at all times of the earthquake duration. For the purpose of an inelastic dynamic analysis the time history method is chosen. It provides a realistic measure of response because the inelastic model accounts for the redistribution of internal actions due to the nonlinear force displacement behavior of the components [2]. Nonlinear damping is considered as well as nonlinear stiffness and load deformation behavior of members. For nonlinear dynamic analysis a step-by-step integration procedure is the most powerful method. The most important assumption is that acceleration varies linearly while the properties of the system such as damping and stiffness remain constant during the time interval. In the procedure a nonlinear system is approximated as a series of linear systems and the response is calculated for a series of small intervals of time Δt and equilibrium is established at the beginning and end of each interval. The accuracy of the method highly depends on the length of the time increment t. It should be small enough to consider the change of loading p(t), nonlinear damping and stiffness properties, and the natural period of vibration.
The characteristics of an SDOF system are its participating forces, namely the spring and damping forces, forces acting on mass of the system, and arbitrary applied loading. The force equilibrium can be shown as:

$$f_i(t) + f_d(t) + f_s(t) = p(t) \tag{1}$$

where $f_i(t)$ is the force acting on the mass, $f_s(t)$ is the spring force and $f_d(t)$ is the damping force. The incremental equations of motion for time t can be shown as:

$$m\Delta\ddot{u}(t) + c(t)\Delta\dot{u}(t) + k(t)\Delta u(t) = \Delta p(t) \tag{2}$$

Current damping $f_d(t)$, elastic forces $f_s(t)$ are then computed using the initial velocity $\dot{u}(t)$, displacement values $u(t)$, nonlinear properties of the system, damping $c(t)$, and stiffness $k(t)$ for that interval. At the beginning of each time increment new structure properties are calculated based on the current deformed state. The complete response is then calculated by using the displacement and velocity values computed at the end of each time step as the initial conditions for the next time interval and repeating until the desired time.
In general terms, such formulations are described by the following:

$$\begin{bmatrix} K & K_R \\ K_R^{T"} & K_{RR} \end{bmatrix} \begin{Bmatrix} U \\ U_R \end{Bmatrix} = \begin{Bmatrix} F \\ F_R \end{Bmatrix} \tag{3}$$

The subscript R represents the reaction forces. The top half of equation (3) is used to solve for $\{U\}$:

$$\{U\} = -[K]^{-1}[K_R]\{U_R\} + [K]^{-1}\{F\} \tag{4}$$

The reaction forces $\{F_R\}$ are computed from the bottom half of equation (3) as

$$\{F_R\} = [K_R]^{T"}\{U\} + \{K_{RR}\}\{U_R\} \tag{5}$$

Equation (5) must be in equilibrium with equation (4).

3 Model development

3.1 Model configurations

To systematically analyze the effect of both regular and irregular longitudinal section geometry (i.e. symmetry and asymmetry) as well as the influence of the substructure stiffness and to compare those results with nonisolated bridges six different bridge models have been developed. All bridges are simple six-span continuous deck bridges with reinforced concrete piers and prestressed concrete box girders. The isolation concept is realized by using isola-tion devices on the top of the pier caps. All bridges are fixed at the pier bottoms and all movements are restrained at the abutments. The types of bridges are:

1. Regular Bridge. Symmetrical longitudinal section geometry, harmonically varying pier lengths like the V-shape as it can be seen in figure 1 (Type 1)
2. Regular Bridge with double pier stiffness. Basically same properties like type 1, but all piers have double moment of inertia for both longitudinal and transverse direction (Type 2)
3. Regular Bridge non-isolated. Basically same properties like type 1, but there are no isolation devices between the pier cap and the bridge deck. These connections are all fixed (Type 3)

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	Deck	Pier	Mod Pier
Area (m^2)	6.88	4.16	4.16
Mom. of Inertia x/z (m^4)	87.24	7.39	14.78
Mom. of Inertia y (m^4)	5.26	0.67	1.34
Max. dimension (m)	14x2.3	4x2.2	4.9x2.5
Mass density (kg/m^3)	2500	2500	2500

Table 1: Geometrical and Physical Properties of Structural Members

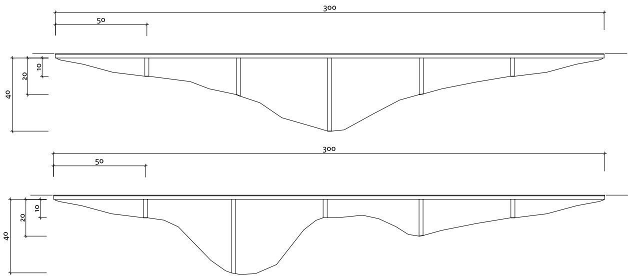

Figure 1: Longitudinal section of regular and irregular bridge.

4. Irregular Bridge. Asymmetrical longitudinal sec-tion geometry, unproportionally varying pier lengths as it can be seen in figure 1 (Type 4)

5. Irregular Bridge with double pier stiffness. Basically same properties like type 4, but all piers have double moment of inertia for both longitudinal and transverse direction. (Type 5)

6. Irregular Bridge non-isolated. Basically same properties like type 4, but there are no isolation devices between the pier cap and the bridge deck. These connections are all fixed (Type 6).

The substructure of all bridges consists of rigid abutments and hollow reinforced concrete piers. The superstructure comprises the reinforced concrete box girder and all installations above. The main properties chosen for the structural members are listed in tabular 1.

3.2 Force-Deformation Behavior of Bearings

For the present study, the lead rubber bearing consisting of alternating layers of steel shims and rubber are considered as the isolation devices.
The LRB is very stiff in the vertical direction and flexible in the horizontal direction (due to the presence of steel shims in rubber). The horizontal flexibility and damping characteristics of the bearing provide the desired isolation effects in the system (Skinner et al. 1980; Tyler and Robinson 1984; Constantinou and Tadjbakhsh 1985).
For the numerical analysis the vertical stiffness of the bearings is regarded as infinitely high, i.e. there is no relative vertical deformation between the pier cap and the deck. For the representation of the horizontal bearing stiffnesses a bilinear material law is chosen for each

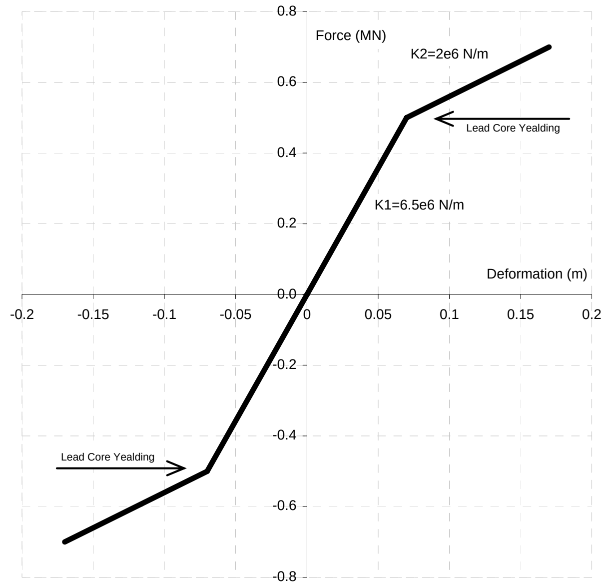

Figure 2: Defined material law (excl. viscous damping)

system standing for a discrete separation of the pre- and post-yield state. The damping ratio for rubber bearings is usually in the range of 5 to 8% and is assumed to be 7% in the analysis. The damping ratio of the global structure for an undamped system is typically set to 5%. For this study 1.5% global damping is used, because unlike typical undamped structures isolated structures are designed to remain in the elastic range and it is assumed that they will dissipate less energy within the structure during the earthquake. The lead yield force of the LRB is chosen by a set ratio yield force/deck weight of around 0.1 to be 0.5 MN. This yield force level might be considered as higher than normally used but it was chosen to ensure an elastic response for at least most members at lower earthquake forces. Figure 2 shows the defined material law for both bearing types used in the numerical simulation.

3.3 Finite Element Model

For the analysis of the six bridge types under several dynamic loads, a simple finite element model was developed to represent all desired features. The software of choice was ANSYS Release 10.0. For both piers and the deck, 3D beam elements were used. Each node has in total 6 degrees of freedom, one translation in each direction and one rotation around every axis.
The bilinear isolation systems are represented by a special element that is a combination of a spring-slider and damper in parallel. The initial stiffness is the sum of K1 and K2 when both springs are still active. Once a certain force in spring one is reached, no higher force can be taken and sliding occurs caused by any additional force. For higher forces to take than the limit force of spring one, the remaining stiffness is hence only the stiffness of spring two. These spring elements do not have masses

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and in their initial state they do not have a length. Both nodes the one representing the pier cap and the one standing for the deck are coincident and linked by these elements. 31 Elements and 32 Nodes were used in total with a numerical extent of 143 degrees of freedom (including the restrains at the abutments and the pier bases). The calculation includes geometrical nonlinearity effects but no physical nonlinearities except the bilinear lead rubber bearings.

3.4 Ground Accelerations

Four different magnitudes of ground accelerations are considered to analyze the correlation of earth-quake ground motion and the response of the bridges, namely $0.5g$, $0.2g$, $0.1g$ and $0.05g$. These ground motions are artificially generated time-history data based on acceleration spectra from Eurocode 8. With respect to the commonly higher natural period of isolated structures, acceleration spectra basing on ground type C were chosen. Ground type C shifts the acceleration spectrum to higher time periods, which are actually closer to the natural period of isolated structures and thus may increase the seismic response considerably. For each earthquake magnitude three different earthquake excitations were generated to ensure a certain maximum of the three curves with random peaks, respectively. Although all three generated excitations per magnitude base on the same spectrum, peaks occur with a difference of up to 40%. Figure 4 shows one generated excitation of a respective magnitude. In order to simplify the load scenario and specify the extent of load combina-tions, the acceleration is limited to the horizontal direction transversal to the bridge axis.

4 Results

In the following diagrams the maximum of the three response values of the respective criteria is plotted versus the ground acceleration. The abscissa always represents the level of peak ground acceleration (PGA).

4.1 Modal Analysis

To obtain general information on the performance of the six bridges, a modal analysis was carried out. Since the method does not consider nonlinear stiffnesses, the post-yield stiffness is used in the LRB material definition in that specific analysis. The first 10 eigenfrequencies of all bridge types are presented in figure 3.
It can be observed that there is a clear shift in terms of frequency between isolated and nonisolated bridge types. The first eigenfrequency of the nonisolated regular bridge is $2.05Hz$, while the natural frequency of the isolated bridge is, at $0.41Hz$, only a fifth, which proves the desired effect of massive stiffness reduction. Table 2 gives an overview of the first five natural frequencies for all bridges.

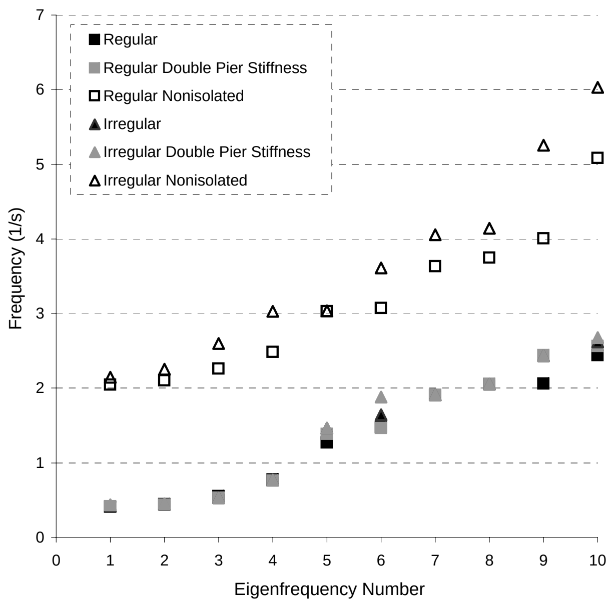

Figure 3: Distribution of eigenfrequencies

Freq No	Type1	Type2	Type3	Type4	Type5	Type6
1	0.41	0.42	2.05	0.42	0.43	2.15
2	0.44	0.44	2.11	0.44	0.45	2.26
3	0.55	0.53	2.26	0.53	0.53	2.60
4	0.78	0.76	2.49	0.77	0.77	3.03
5	1.27	1.39	3.03	1.46	1.47	3.04

Table 2: First five natural frequencies (Hz) of all bridge types

4.2 Response Accelerations

Figure 4 shows the response acceleration of the applied, artificially generated earthquake ground motion for both the isolated and nonisolated regular bridge at $0.5g$ ground motion during the entire earthquake. It can be observed how different both reactions are, namely larger amplitudes and higher frequencies for the nonisolated structure.
The peak response accelerations of both systems are shown in figure 5. While for the LRB isolated regular bridge the deck above the long pier is most accelerated and the short and mid pier behave similarly, in the case of irregular pier layout the second short pier gives the largest response and the deck acceleration above the first short pier and the long pier is in the same range. For all bridge types, the deck above the abutments is the point with the maximum acceleration.
Figure 5 also shows the response acceleration of the two nonisolated bridge types for comparison. Due to the absence of any nonlinear material, the response here is solely linear. A disproportional increase of the response by the pier height can be observed for the regular structure. While the response acceleration increases from $0.7g$ to $1.0g$ by the doubling of the pier height from 10 to 20m, a peak response of $2.2g$ is measured for the 40m tall pier,

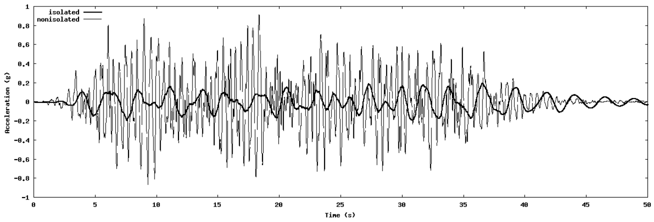

Figure 4: Deck Response Acceleration above mid pier during earthquake (regular bridge; 0.5g ground motion)

which means a further doubling of the pier height. This correlation of response acceleration and pier height is also found for the irregular bridge where both first and second short pier respond equally. The measured acceleration of the long pier, however, is much lower than the long pier acceleration of the regular bridge. Comparing the peak responses of the nonisolated and isolated bridges it can be stated that the response acceleration is re-duced significantly by using isolation devices.

4.3 Isolator Participation in Deflection

From the performance of the isolators during the range of ground motions, another conclusion can be made regarding the isolator efficiency. Consequently derived from the deflection results, the ratio of isolator and total deflection shall be a significant representation for the effect of the applied seismic isolation. These obtained relations can be observed in figure 6. The isolators deflection participation indicates the efficiency of a particular isolator for the given pier configuration. A common design principle for seismic isolation devices would be the adjustment of each single isolator to obtain a similar performance in terms of isolator-pier deflection ratio. However, the individual isolation behavior and thus the determination of diverse material laws is not regarded in this study. The diagrams rather demonstrate the significant low participation of the long pier isolator in all bridges. The isolator participation is almost not changing over the range of ground motion intensity. For lowest earthquake accelerations, the LRB participation is at least 62% of the total deflection while for higher ground motions around 90% of the deflections can be contributed. The shorter the piers and the stiffer the substructure the higher is the participation of the isolator in the deflection. While the isolators on the mid and short pier always range mostly far above 80% for all bridges, the long pier isolators participation increases to 90% (irregular with doubled pier stiffness).

5 Conclusion

The consideration of pier stiffness and pier layout revealed only a small influence on the resulting response. Higher substructure stiffness (i.e. double pier stiffness, irregular shape) leads to higher isolator efficiency through reduction of pier deflections. The increase of pier stiffness, however, also causes higher bending moments at the pier base and higher pier base shear forces. The irregular pier layout basically leads to a load concentration at a single column and is therefore even more suitable for the base-isolation concept.
The response curves reveal an explicit bilinear character, which is determined by the influence of hysteretic damping and bearing stiffness reduction after lead yielding. There is no noteworthy reduction of response during the pre-yield state of the lead. Since there is hysteretic damping in post-yield state of the lead in the isolation system, the response is reduced significantly and the isolator is even more efficient (i.e. reduction is higher) for larger magnitudes of ground motion. Apart from these general characteristics, the extent of most response values is heavily dependent on the yield force level that is defined by lead modification (i.e. geometry). The results show correlation of response curves for full ground motion scale and the material law definition, because a characteristic change occurs mostly at the respective yield force level.

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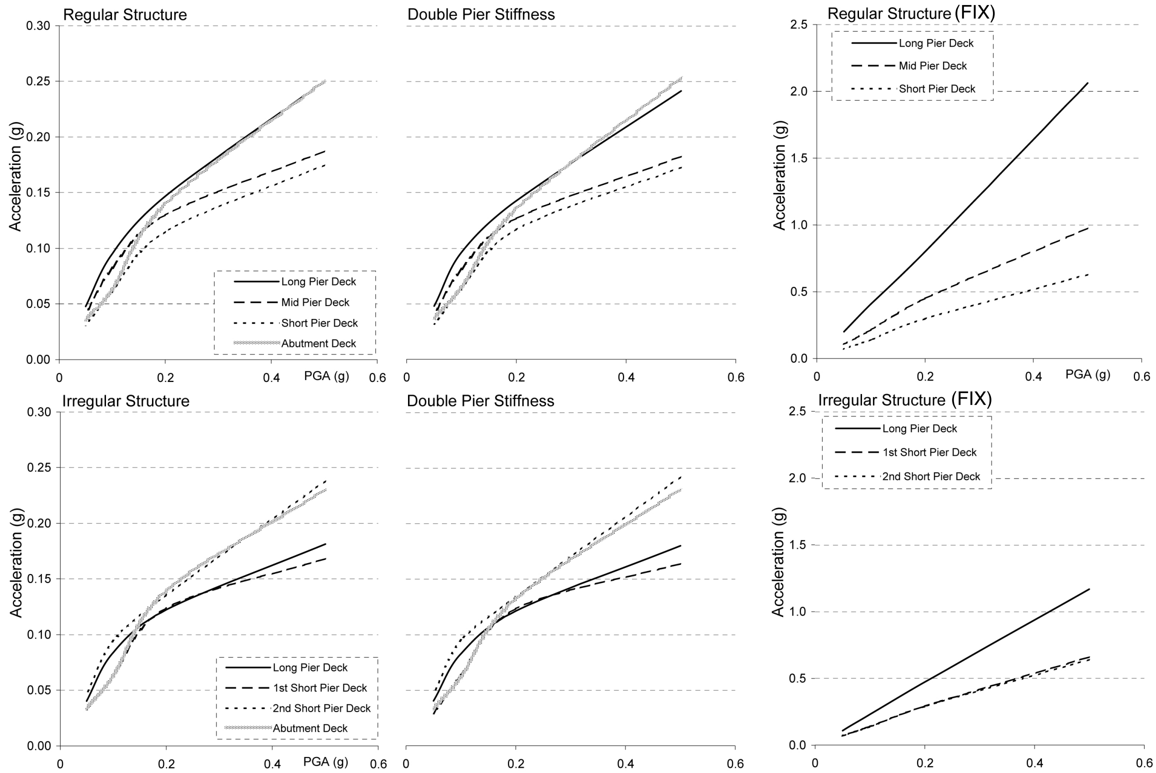

Figure 5: Response acceleration LRB isolated deck

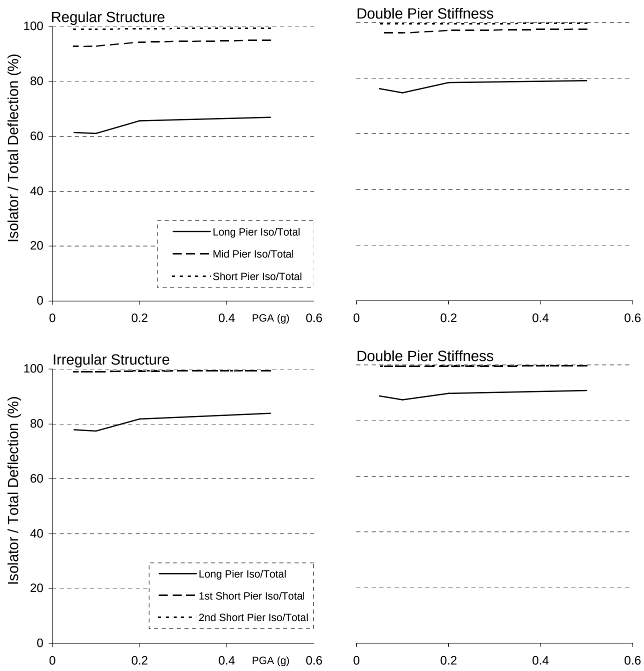

Figure 6: Isolator participation on total deflection for all isolated bridges