SOIL STRUCTURE INTERACTION EFFECTS ON BASE ISOLATED BRIDGES

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ABSTRACT :

Base isolation has already been used extensively for buildings in the United States, Japan, New Zealand, Italy and Chile among other countries. Its extension to bridges was a logical step. A large number of papers have been written over the last 15 to 20 years to investigate the adequacy of various types of isolation pads, their material properties and their behavior under different types of loads, to compare the seismic response of bridges with or without isolation pads, to study the effect of the relative stiffness of the pads compared to the stiffness of the structure, to assess the importance of soil structure interaction effects, and to develop analytical models and simplified design procedures. There are still, however, a number of questions that have not been fully addressed, particularly in relation to the nonlinear behavior of the isolation pads, and the combined effects of soil structure interaction. This work investigates more fully the combined effects of nonlinear behavior of the isolation pads and the inertial soil structure interaction effects on the seismic response of a collection of representative base isolated bridges on different types of soil.

KEYWORDS: Base isolation, Bridges, Soil structure interaction, Dynamics, Earthquakes, Nonlinear behavior.

1. INTRODUCTION

Bridges are vital structures in the infrastructure of any country for the transportation of persons and goods. Their operability after a major disaster, such as an earthquake, is essential. To improve the performance of bridges under seismic loads, several countries (United States, Japan, New Zealand, Italy, and Chile) have developed and implemented energy dissipation devices. This technology is a promising alternative for new and existing bridges that may be subjected to earthquakes. This is the case in Mexico with approximately 200 km of bridges, from which only one, the Infiernillo bridge, has been designed with base isolation. Yet several bridges were damaged during recent earthquakes such as the 7.9 surface magnitude Manzanillo earthquake of 1995, although current regulations require the consideration of seismic loads in their design.

To evaluate the dynamic response of bridges with base isolation it is necessary to use realistic and accurate models but the models commonly used in research and proposed in design codes (Ciampoli and Pinto, 1995, Spyrakos and Vlassis, 2002, Turkington et al., 1989, AASHTO, NZMWD, JPWRI, CALTRANS) introduce a number of approximations (two degrees of freedom systems or plane frames instead of a full 3D model, equivalent linearization techniques to simulate the nonlinear response of the isolation pads). The use of plane frames and two degrees of freedom systems can be appropriate for preliminary studies or to explore general trends (Olmos and Roesset, 2008) but will not reproduce accurately in general the behavior of an actual bridge. Codes recommend the use of simplified procedures to account for the nonlinear behavior of the isolators in practical design based on an iterative equivalent linearization (Hwang and Sheng, 1994; Hwang et al., 1994 & 1996; AASHTO; CALTRANS; NZMWD; and JPWRI). This methodology can be of value for a preliminary design but only for the types of seismic motions considered in their formulation. A number of studies have considered soil structure interaction (inertial interaction) effects on base isolated bridges using simplified models (Dicleli et al., 2004, Jangid, 2002, Ciampoli and Pinto, 1995, and Vlassis and Spyrakos, 2001). A major question is how these effects interact with the nonlinear behavior of the isolators for realistic bridge foundations.

A collection of 36 bridges were designed using present seismic design recommendations in Mexico for three different types of soil (stiff, medium and soft), and used to study the effects of the nonlinear behavior of the base isolation and the combined effects with the inertial soil structure interaction (only the bridges designed for the medium and soft soil were considered for the combined effects since SSI effects would be negligible for the first case). The bridges where then analyzed under three earthquakes, two of them representative of medium soil and one of soft soil.

2. PARAMETRIC STUDIES

The bridges considered were typical reinforced concrete (RC) bridges in Mexico, with 2 and 5 spans, span lengths of 20, 40 and 60 m, and pier heights of 10 and 30 m in seismic zone D (potentially more dangerous for structures located in Mexico). The combination of the geometric parameters led to 12 different bridges that were designed for each soil type (36 total cases). The response parameters studied were the relative displacements of the deck and the top of the piers, the absolute accelerations, the seismic forces in the piers, and the ductility demands for the isolation pads. The three earthquakes used were the Manzanillo, the SCT and the El Centro motions. In the text each bridge is identified with a number and a letter referring to the number of spans, the span lengths, and the pier height. For example, the bridge referred to as 2S20L10H represents a 2-span bridge with 20 m spans and 10 m pier height. All the bridges were considered to be RC except those with 60 m spans that had steel plate girders. All of the bridges had RC circular piers, RC slabs and RC bent caps. The diaphragms used in each of the bridges were RC or steel sections depending on the girders. The modulus of elasticity, shear modulus and Poisson ratio were 2.5E10 Pa, 1.03E10 Pa and 0.2 for the concrete, and 2E11 Pa, 7.7E10 Pa and 0.3 for the steel. Figure 1 shows a schematic plan and elevation of the 2-span bridges.

2.1. Structural Model

The bridges were modeled as 3D structures and the analyses were carried out with the nonlinear SAP2000 program. The members (girders, diaphragms, bent caps, and piers) were modeled as beam elements but the RC slab was modeled with a mesh of rectangular thin shell (plate bending and stretching) finite elements. The energy dissipation devices, located at each of the beam supports with the same properties in the longitudinal and the transverse directions were modeled with nonlinear link elements. Their properties were computed to produce a shift in the longitudinal natural period of the bridges by a factor of two to four without exceeding an allowable deformation (AASHTO, CALTRANS, JPWRI, and Priestley, 1996). The abutments were not included in the model. The pier supports were first considered fixed, and then the flexibility of the foundations was accounted for using constant springs and dashpots in two orthogonal horizontal directions and the respective rotations. The maximum seismic responses for these two support conditions were compared to assess the inertial SSI effects.

2.2. Dynamic Stiffness of Pile Foundations

The foundations of the 36 bridges were designed according to present practice in Mexico. RC pile groups with end bearing piles were selected in all cases. This type of foundations is the most commonly used in México where it is recommended that piles lay on a hard soil stratum. The properties for each soil type were defined using representative values for medium and soft soils. The hard soil (type I) was not included in this part of the

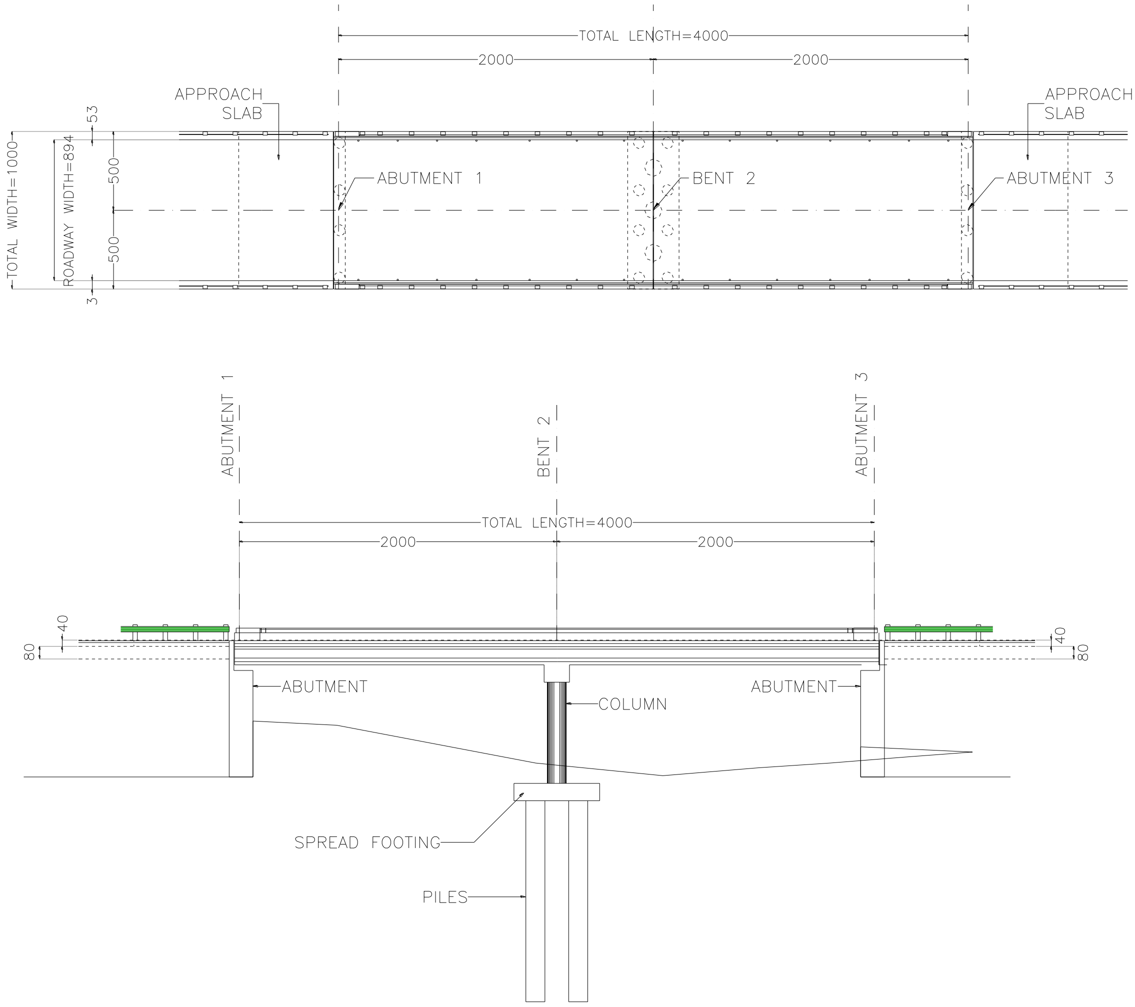

Figure 1 Plan and elevation views of the 2-span bridge model

study because the effects in this case were anticipated to be negligible. The soft soil (type III) was assumed to represent clays with 25kPa shear capacity whereas the medium soil (type II) corresponded to sands with 75 kPa and 6250 kPa of shear and axial capacities, respectively. A safety factor of 3 was considered in the foundation design. The dynamic stiffness of the foundations was evaluated with a program for dynamic analysis of pile groups in a layered medium. The soil was assumed to have a mass density of 17 kN/m^3, a Poisson's ratio of 0.25, a damping ratio of 0.05. The shear wave velocity was 100 m/s for the soft soil and 250 m/s for the medium soil. The results obtained from the program are the values of K_{real} and $K_{imaginary}$ as functions of frequency. $K_{dynamic}$ is defined in Eqn (2.1) as function of C, K_{real} ,and $K_{imaginary}$:

$$K_{dynamic} = K_{real} + iK_{imaginary} = K_{real} + i\Omega C \tag{2.1}$$

where K_{real} is the stiffness of an equivalent spring, C is the constant of and equivalent dashpot, Ω is the excitation frequency, and i is the imaginary number. The constant value of C is the result of dividing $K_{imaginary}$ by the frequency Ω. The dynamic stiffness for the pile groups in the medium soil had a real part nearly constant and a very small imaginary part. On the other hand, the stiffness for the soft soil showed to have more frequency dependence and larger values of the imaginary part (more important contributions of the geometric damping) for high frequencies, but these effects were still small for the range of small frequencies of interest here (for base isolated bridges). A preliminary estimate of the potential importance of soil structure interaction effects can be obtained from the stiffness ratio in Eqn. (2.2):

$$k_{ratio} = \frac{k_{str}}{k_x} + \frac{k_{str}\, h^2}{k_\phi} \tag{2.2}$$

k_{str} represents the equivalent stiffness of the structure, k_x is the foundation stiffness in the horizontal direction (x), k_ϕ is the rocking stiffness around the perpendicular axis (y), and h is the pier height. This ratio was generally very small, the only exception occurring when the foundation consisted of only one row of piles running in the transverse direction. In this case the comparison with the case of fixed base would not be appropriate since any experienced designer would consider the base pinned even if neglecting SSI.

This study considered only the effect of the flexibility of the foundation on the dynamic response of the bridges (inertial soil structure interaction). The effects of the foundation on the seismic motions without any structure, known as kinematic interaction, were neglected. These effects can lead to a reduction of the motion amplitudes for high frequencies but for the range of frequencies of interest in relation to the base isolated bridges they were estimated to be very small.

2.3. Seismic Excitation

Three earthquakes were used to study the seismic responses of the bridges. Two of them are from Mexico: the SCT 1985 Mexico City and the 1995 Manzanillo earthquakes, and another from the USA: the 1940 El Centro record. The first record was selected because it is the one that caused devastating damage in Mexico City, and the second is the most recent high intensity earthquake that also caused considerable material losses. These two records have different characteristics: the first ground motion is almost harmonic and was recorded in soft soil; the second has high frequency content and was recorded close to the epicenter on hard soil. The third earthquake was selected because it has somewhat similar characteristics to the Manzanillo record and has been used in the majority of the studies reported in the literature. To study the inertial SSI effects, the bridges designed to be on soil type II were subjected to the Manzanillo and the El Centro ground motions, and those designed on soil type III to the SCT earthquake. Figure 2 shows the acceleration time history of each earthquake and their pseudo acceleration and relative displacement response spectra for 5% damping.

3. INERTIAL SSI EFFECTS ON BASE ISOLATED BRIDGES

The seismic responses of the base isolated bridges on fixed and on flexible supports are compared in this part of the paper. Since the bridges are symmetric about two horizontal axes, the central and the extreme piers have very similar seismic demands and the results are presented here for only one pier. In addition due to the lack of space they are only presented for the longitudinal direction although the seismic responses in the transverse direction are discussed. The responses shown are the maximum shear forces on the piers (V_{max}), the maximum relative displacements (U_{max}) and absolute accelerations (A_{max}) of the deck, and the maximum ductility demands (μ) of the isolation pads. In general the results showed that the inertial SSI effects were almost negligible in the longitudinal direction, for both soil types. The effects were slightly more significant in the transverse direction, particularly for bridges located on soil type II; in spite of this, their significance is pretty similar to that found for the longitudinal direction. The results shown in figures 3 to 5 correspond to U_{max} and A_{max} at deck level, to V_{max} on the piers and, to the ductility μ of the isolator, for the SCT, Manzanillo and El Centro ground motions, respectively. Figures 4 and 5 show the base isolator's hysteretic loops for the bridges under the Manzanillo earthquake, for the longitudinal and transverse directions, respectively.

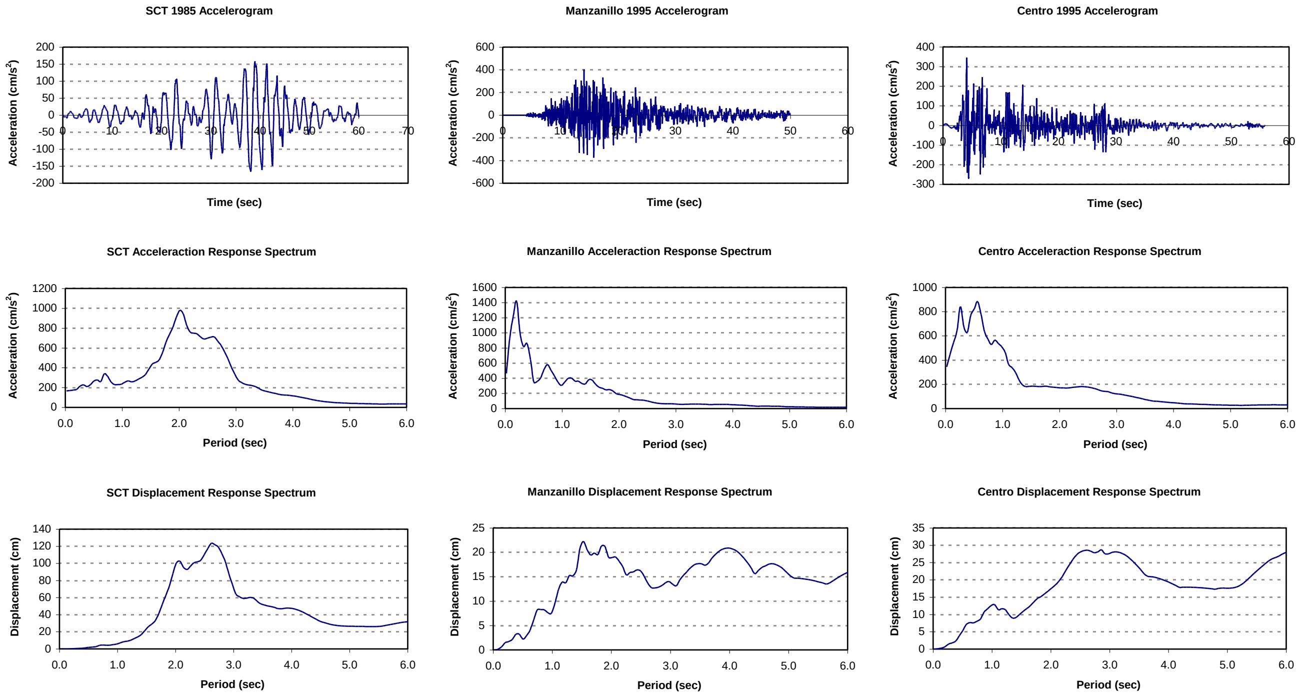

Figure 2 Acceleration time histories, pseudo acceleration and displacement response spectra for the SCT, Manzanillo and El Centro earthquakes

As can be seen in figures 3 to 5, the maximum responses (U_{max}, A_{max}, μ and V_{max}) in the longitudinal direction, were affected very little by the flexibility of the foundation. The responses experienced small changes, increases and decreases, smaller than 5%. The same tendencies were found under the three ground motions. In the transverse direction, there were little more noticeable changes in the responses, particularly for bridges located on soil type II. The effects on the transverse seismic responses of the bridges under the SCT and the El Centro were larger than in the longitudinal direction, but still less than 10%. In general, the inertial SSI effects have a tendency to decrease the response although there were a few cases that showed small increases (for bridges with 30 m pier height on soft soil and with 10 m pier height on medium soil).

Figures 6 and 7 show the cyclic behavior (hysteresis loops) of the isolators for the fixed and flexible bases when the bridges were subjected to the Manzanillo ground motion. The figures show that the nonlinear behavior in the longitudinal direction is not affected by the flexible foundations whereas for the transverse direction the isolator's hysteretic loops exhibited smaller reductions on the loops' areas for the flexible foundations (figure 7), indicating a smaller loss of energy through nonlinear behavior. The results agree with the reductions on the U_{max} found for the isolated bridges on flexible base when subjected in their transverse direction to a dynamic motion.

4. CONCLUSIONS

In this work we studied the combined effects on the nonlinear behavior of the isolator pads and inertial soil structure interaction on the seismic responses (relative displacements, absolute accelerations, shear forces and ductility demands) of base isolated bridges under ground motions recorded on similar soils. The results showed that for typical foundations designed according to present codes with the required factors of safety the inertial SSI had very small effects on the nonlinear responses of the bridges, particularly in the longitudinal direction. Larger effects had been reported in the literature for cases where the foundations were very narrow surface strip footings with very small rotational stiffness around the transverse axis. The same situation would be encountered if one had only one row of piles under the cap. In these cases however the results should be compared to those obtained assuming hinges at the base of the piers. This is what most experienced designers would assume in their analyses. Studying 2 degree of freedom systems (Olmos and Roesset, 2008) with larger values of the stiffness ratio between the structure and the foundation it was found that the small increases and decreases in the responses (in general lower than 10%) were mostly due to the foundations rocking. This is particularly so for tall piers.

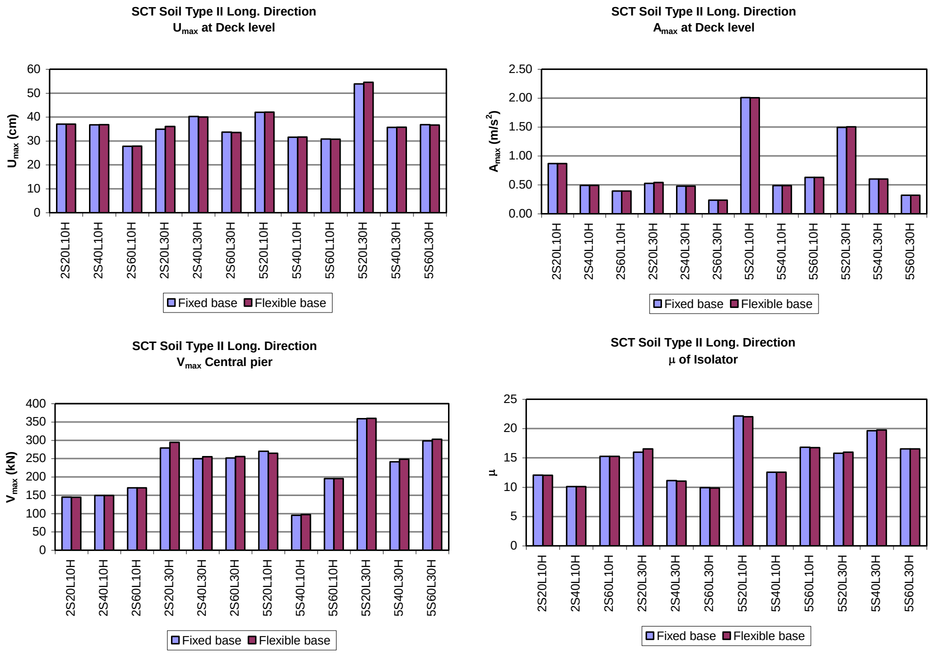

Figure 3 Maximum seismic responses in the longitudinal for studied bridges under the SCT ground motion, 1985

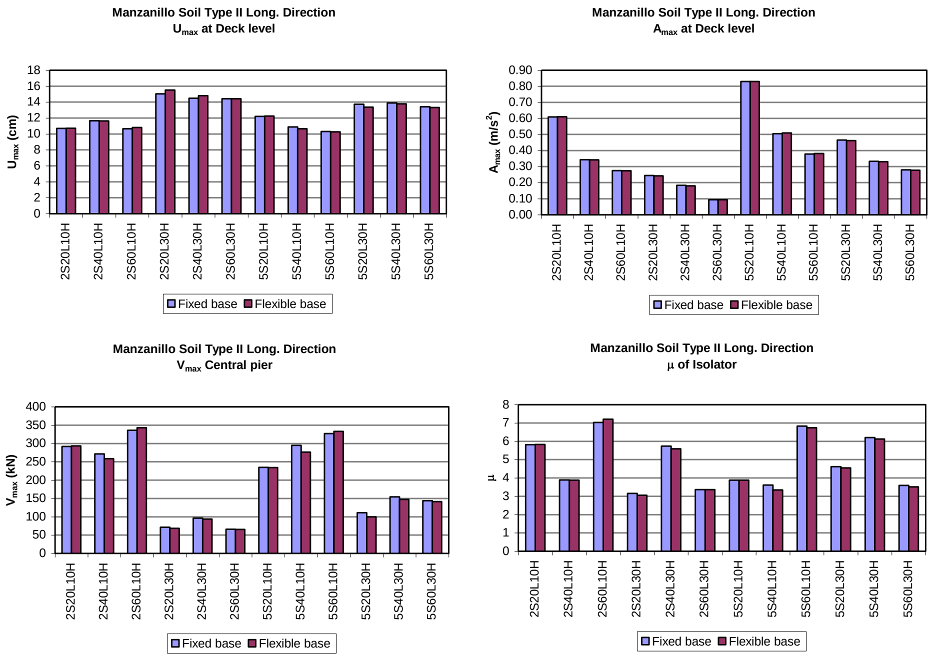

Figure 4 Maximum seismic responses in the longitudinal for studied bridges under the Manzanillo ground motion, 1995

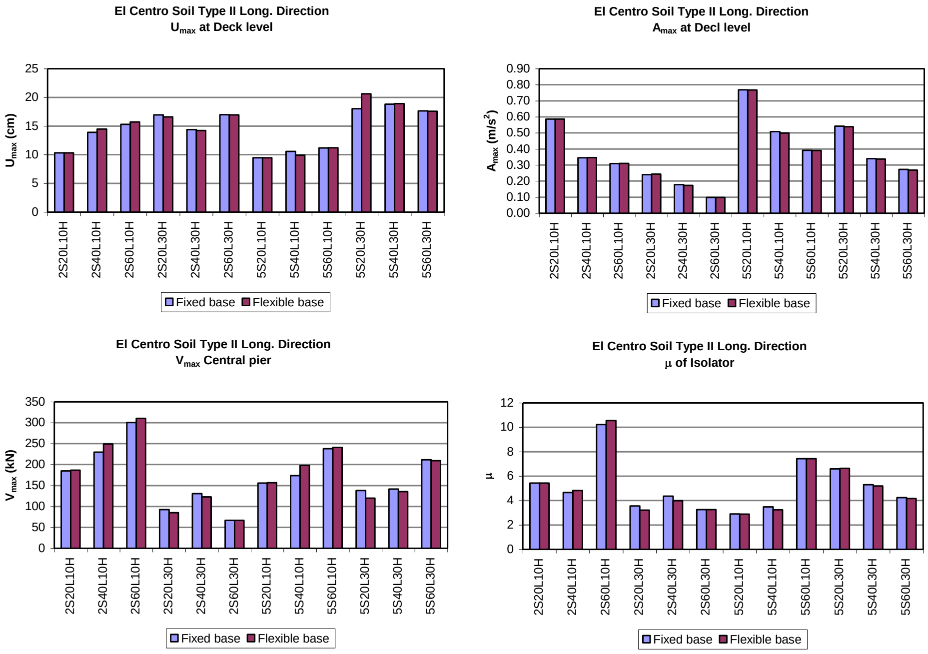

Figure 5 Maximum seismic responses in the longitudinal for studied bridges under the El Centro ground motion, 1940

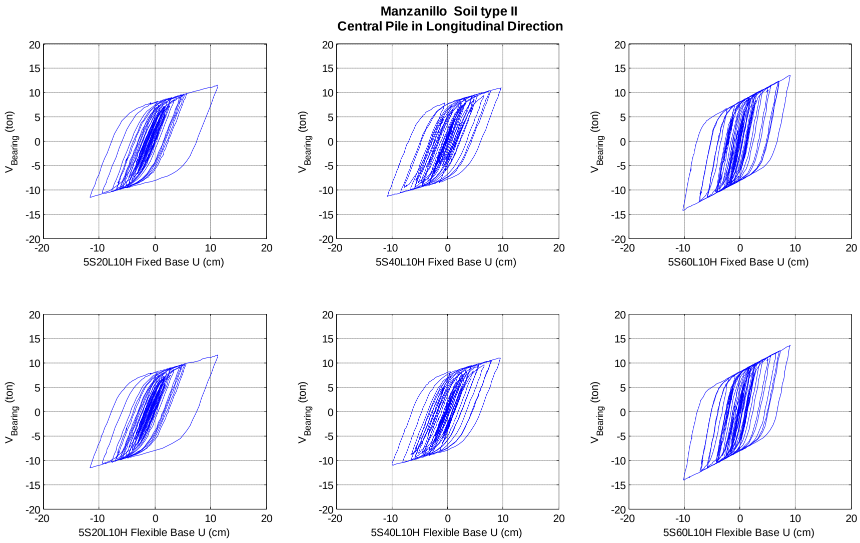

Figure 6 Isolator hysteretic behavior in the longitudinal direction for bridges on fixed base vs. flexible base (first and second rows, respectively)

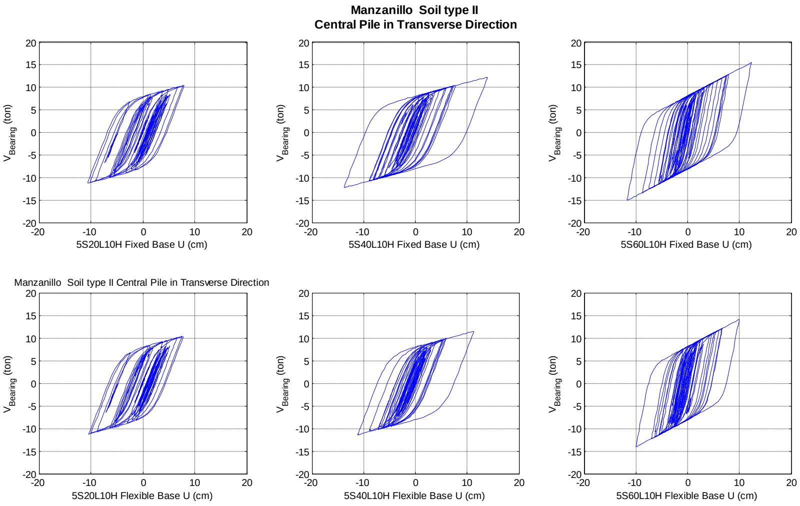

Figure 6 Isolator hysteretic behavior in the transverse direction for bridges on fixed base vs. flexible base (first and second rows, respectively)

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