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NUMERICAL STUDY OF THE SEISMIC RESPONSE OF AN URBAN OVERPASS SUPPORT SYSTEM

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ABSTRACT

A strategic urban overpass is to be built in the so-called transition and hill zones in Mexico City. The subsoil conditions at these zones typically consist on soft to stiff clay and medium to dense sand deposits, randomly interbedded by loose sand lenses, and underlain by rock formations that may outcrop in some areas. Several critical supports of this overpass are going to be instrumented with accelerometers, inclinometers and extensometers to assess their seismic performance during future earthquakes and to generate a database to calibrate soil-structure-interaction numerical models. This paper presents the seismic performance evaluation of one of these supports. The support foundation is a 3.6 by 4.6 m mat, structurally connected to four cast-in-place 0.80 m diameter piles. A finite elements model of the soil-foundation-structure system was developed. Initially, the model was calibrated analyzing the seismic response that an instrumented bridge support exhibited during the June 15th, 1999 Tehuacan (M_w=7) Earthquake. This bridge is located also within the surroundings of Mexico City, but at the lake zone, where highly compressible clays are found. The computed response was compared with the measured response in the free field, box foundation, and structure. Once the model prediction capabilities were established, the seismic response of the critical support of the urban overpass was evaluated for the design earthquake in terms of transfer functions and displacement time histories.

INTRODUCTION

Failures observed in bridges and vehicular overpasses during recent seismic events such as Loma Prieta, 1989; Northridge, 1994; Kobe, 1995; Kocaeli and Duzce, 1999; and Chi-Chi, 1999 earthquakes have clearly shown that the seismic behavior of these structures is far from being fully comprehended. Seeking to build both safe and economical structures, the engineer must be able to quantify accurately the input loading, to evaluate properly the soil behavior under this loading and to make reliable assessments of the soil-foundation system response, including potential ground motion incoherence and if it is the case, the possibility of surface rupture. Seismic loading acting upon a soil-foundation system results from the interplay of earthquake incoming waves with the structure-swaying-produced waves, which in some cases may lead to an increase on the structural spectral ordinates in the foundation response with respect to those observed in the free field (Mayoral *et al.* [2009]). The complex foundation vibration patterns that result from this interaction are difficult (if not impossible) to predict because they depend on many factors (that are interrelated) such as incoming wave-train characteristics, bridge-foundation vibration patterns, soil-foundation interaction, soil behavior (elastic/inelastic), site geological and geotechnical characteristics, and pre-earthquake foundation conditions (Romo *et al.* [2000]). Furthermore, in dense urban zones, such as Mexico City, the incoming wave patterns can be modified as compared with commonly assumed isolated single foundation structure conditions, due to their interaction with waves radiating away from nearby soil-foundation systems. Thus, modern urban bridge design has moved toward performance-based concepts, requiring that any minor damage the system may undergo during the design earthquake occurs first within the superstructure rather than the foundation. This framework implies that the foundations need to be analyzed considering the least conservative of the two following conditions: 1) loads and moments obtained considering a ductility factor, Q, and over-resistance factor, R, of one and two, respectively for the design response spectra (i.e. elastic forces) and 2) loads and forces transmitted by the column to the foundation based on the ultimate strength of the columns or the upper deck support system (e.g. frames, columns, shear walls). These innovative approaches demand more precise predictions of the structural response, using advanced numerical and analytical tools to conduct seismic soil-

structure interaction, SSSI, analyses, including an accurate estimation of the support beams displacements, in both the transversal and longitudinal components, to ensure that relative movements between them, will not trigger a separation of the central and the support beam (Fig. 1), reducing to minimum the probability of collapse of the upper deck. To guaranty a good estimation of the structure performance, it is necessary the calibration of numerical models, such as those developed with finite elements or finite differences, which allows the simulation of ground, foundation, and structure.

In this paper, the methodology used by Mayoral *et al.*, 2009, is applied to model the seismic soil-structure-interaction of one of the more critical supports of a strategic urban overpass to be built in the so-called transition and hill zones in the North-East part of Mexico City. Initially, the model was calibrated analyzing the seismic response that an instrumented bridge support exhibited during the June 15th, 1999 Tehuacan (M_w=7) Earthquake. This bridge is located also within the surroundings of Mexico City, but at the lake zone, where highly compressible clays are found. The bridge worked as a deck in a surface subway station and was built 12 years ago in the so-called Lake Zone in Mexico City, known by its difficult subsoil conditions. Since then, pile loads, soil-raft contact pressures and the overall response of the foundation system have been recorded. Within this period several major earthquakes have occurred. Thus, an extensive data base of accelerations, pore pressures, and load histories have been gathered. Finite element models of the soil-foundation-structure system were developed using the program SASSI2000. The computed responses were obtained in the free field, raft foundation, and support beam, in terms of acceleration response spectra. They were in good agreement with the measured responses. Once the model and analysis approach prediction capabilities were established, the seismic performance evaluation of one of the most critical supports of the urban overpass was conducted.

PROJECT DESCRIPTION

A vehicular overpass 23 km long is to be built in the North-East region of Mexico City, and it will cross the so-called transition and hill zones (Fig. 2), as described by the Mexico City building code. The overpass consists of an upper deck resting on top of central and support beams (Fig. 3) that are structurally tied to the columns, which, in turn, are monolithically attached to a rectangular raft foundation 3.6 by 4.6 m^2, connected to four 0.8 m diameter, cast-in situ, concrete piles. For the particular support analyzed, the pile and column lengths are 35 m and 8.4 m, respectively (Fig. 3). The raft foundation is 1.70 m thick, as depicted in figure 4. The area surrounding the foundation and below it, up to a depth of 1.7m, was improved using a concrete filling. Table 1 shows the concrete strengths at 28 days (f'_c) of the concrete used in each structural member. Thus, it can be considered a total effective foundation depth of 4.15 m for the raft foundation. The separation between piles is 2.30 m and 3.30 m in the transversal and longitudinal direction, respectively. The reinforcement steel yield strength, f_y, was 412,020 kPa. The unit weight was 23.5 kN/m^3.

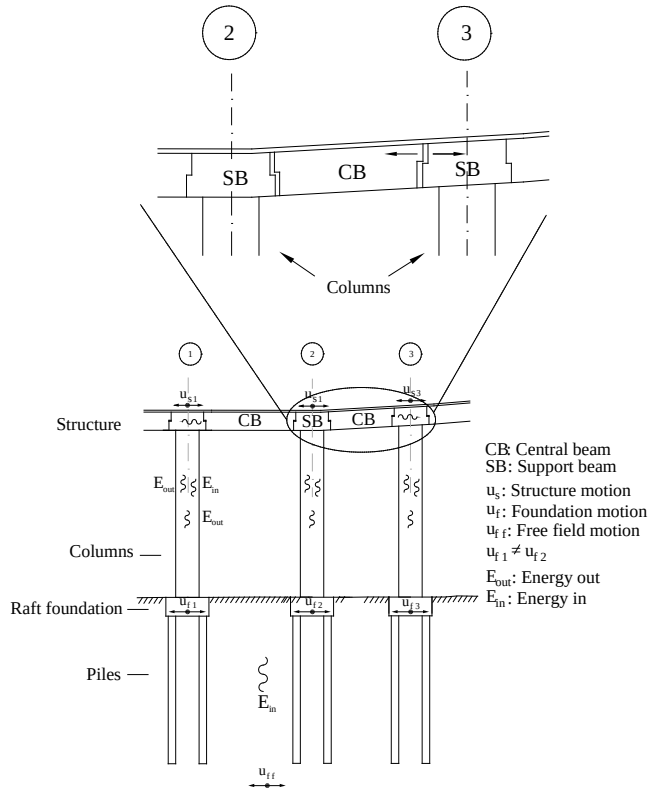


Fig. 1. Effect of relative movements of the bridge supports on the deck.

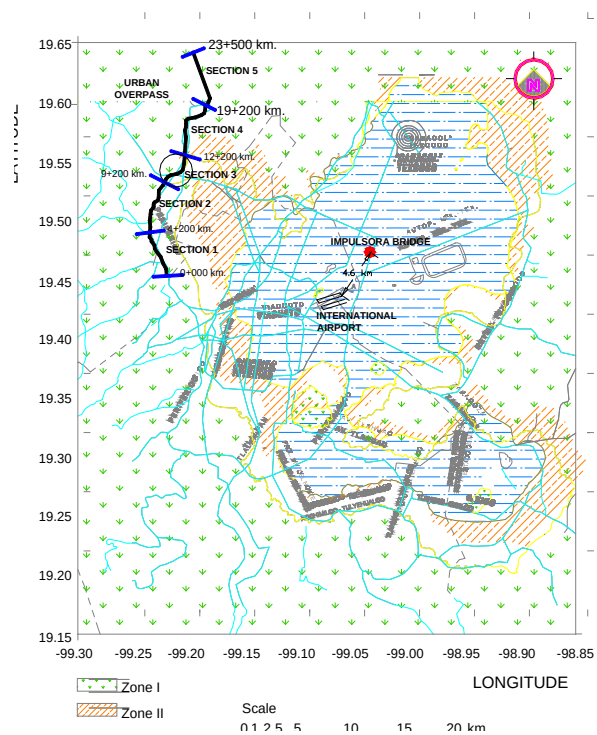


Fig. 2. Project location

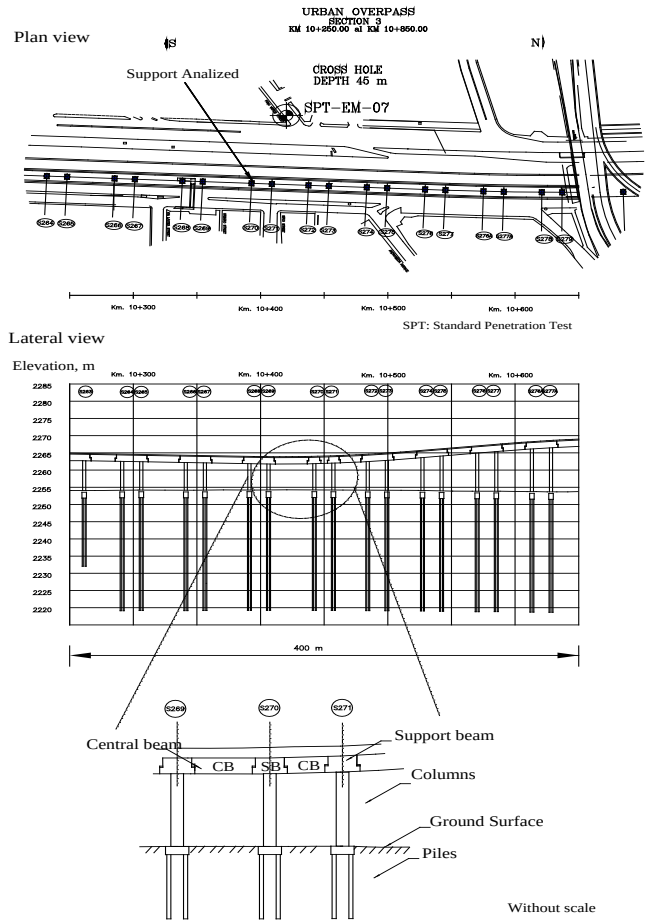


Fig. 3. Support analyzed

GEOLOGYC SETTINGS

The transition zone is characterized by abrupt stratigraphical changes, thus both soft to stiff clays and medium to dense sand deposits, randomly interbedded by loose sand lenses can be found. The hill zone is comprised by well cemented pumice-type tuffs (i.e. cemented silty sands and sandy silts), piroclastic materials, interbedded by alluvial sands of medium to dense relative density. From the geologic stand point, this area is underlain by the Tarango andesitic rock formation, which may outcrop in some areas. It is common to find caves, some of them associated to mining activity. The oldest deposits of the Tarango formation consist of yellow tuffs, which in some regions reached thicknesses larger than 50 m.

SUBSOIL CONDITIONS

The urban overpass is located in a nearly flat area. To characterize the geotechnical subsoil conditions found at the site where the support analyzed is located, a standard penetration test, SPT, boring was conducted along with selective undisturbed sampling. In addition, a piezocone was installed to obtain pore pressure distribution, and one cross-hole was performed to measure the shear wave velocity distribution with depth. The soil profile at the site (Fig. 5) is mainly comprised by a stiff clay layer at the top (i.e. undrained shear strength, s_u, of 50 kPa), intercalated with sand and sandy silts lenses, this layer extends down to 10 m. After this depth and up to 30 m, the number of dense sand lenses increases (i.e. number of blows corrected by energy and overburdening, $(N_1)_{60}$ is larger than 60). The water content of these materials ranged from 20 to 100 %. Underlying this layer and up to the maximum explored depth there is a very dense sandy silt ($(N_1)_{60}$ larger than 65).

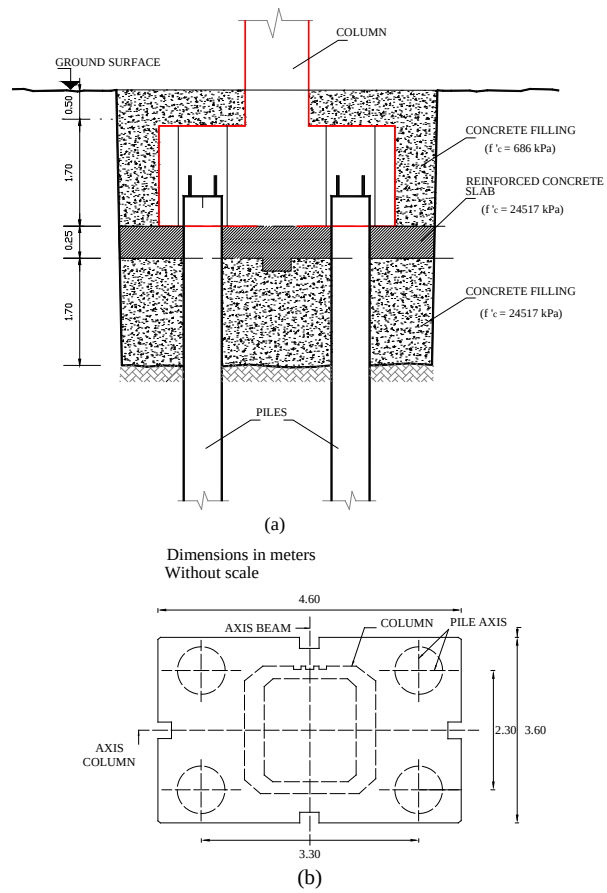


Fig. 4. Support foundation (a) elevation, (b) plan view

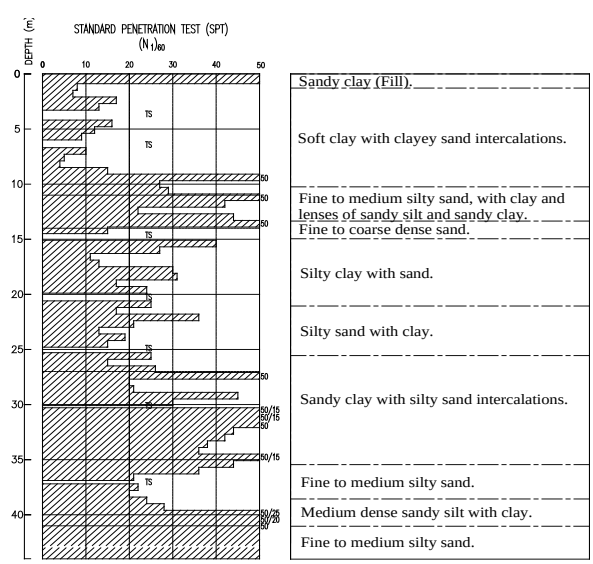


Fig. 5. Sub-surface conditions prevailing at the studied site

Shear wave velocity profile

As mentioned previously, the cross hole technique was used to determine in-situ values of shear wave velocity, V_s, (Romo *et al.* [2009]), and in turn, to define the small strain shear modulus G_{max} (for strain levels of 10^{-5} or less).

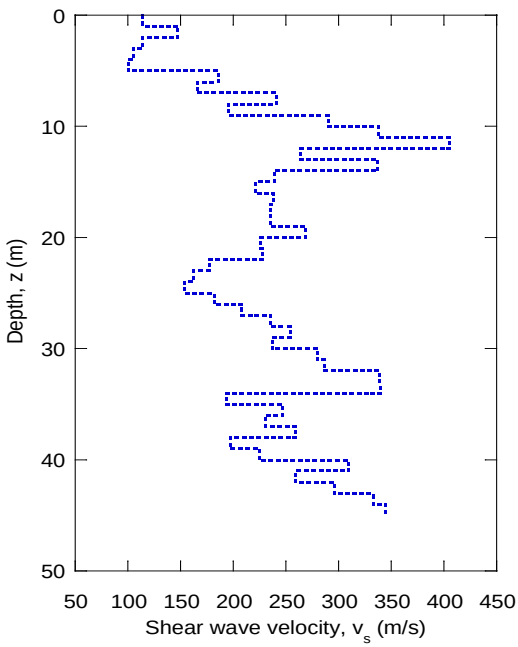


Fig. 6. Shear wave velocity distribution measured with cross hole.

Normalized modulus degradation and damping curves. For clays. Due to the lack of experimental information regarding the soil dynamic properties of the materials found at the site, these were estimated based on the normalized modulus degradation and damping curves proposed by Vucetic and Dobry [1991], as a function of plasticity index, (PI), considering the information gathered from index properties (Fig. 7). For completeness, these curves were compared with those obtained using the model proposed by Romo [1995], which is described by the following equations:

$$G = G_{\max}\left(1 - H(\gamma)\right) \quad (1)$$

$$H(\gamma) = \left[\frac{(\gamma/\gamma_r)^{2B}}{1+(\gamma/\gamma_r)^{2B}}\right]^{A'} \quad (2)$$

$$\lambda = \lambda_{\max}\left(1 - \frac{G}{G_{\max}}\right) \quad (3)$$

$$A' = A + I_r \quad (4)$$

Where:

G is the dynamic shear stiffness,

G_{max} is the small strain shear stiffness,

λ is the damping,

H(γ) is function of the shear strain,

γ is the shear strain,

λ_{max} is the maximum soil damping (i.e., near dynamic failure), considered as 14% for México City clays,

A and B are soils parameters obtained as proposed by Romo [1995], which define the geometry of the curve G-γ and are a function of the plasticity index of the soil,

γ_r is a fix reference value of the shear strain corresponding to 50% of modulus degradation,

I_r is the relative consistency, which can be expressed as

$$I_r = \frac{\omega_L - \omega_n}{PI},$$

and ω_L, ω_n and PI are the liquid limit, natural water content and plasticity index of the soil respectively.

This model has shown to provide reliable estimations of the dynamic shear stiffness and damping variation with shear strains for clays (e.g., Flores and Romo [2001], Gonzalez [2005], Mayoral *et al.* [2008], Mayoral *et al.* [2009]). Fig. 7 shows the normalized shear stiffness and damping curves obtained with the model and those obtained using Vucetic and Dobry [1991], which basically enveloped the model curves. It is warrant to mention that the expression proposed by Romo [2005] will predict the same curve for the three values of plasticity index considered (i.e. 15, 30 and 50%), since this model was developed for soft clays found at the lake zone, which exhibit large plasticity indices (higher than 100%).

For sands. Due to the practical difficulty associated with sampling the sand layers, the curves proposed by Seed and Idriss [1970] for normalized modulus degradation and damping curves, were used for the analyses (Fig. 8). These curves have been successfully used in 1-D wave propagation analysis to predict the measured response during the 1985 Michoacán earthquake (e.g., Mayoral *et al.* [2008], Seed *et al.* [1988]).

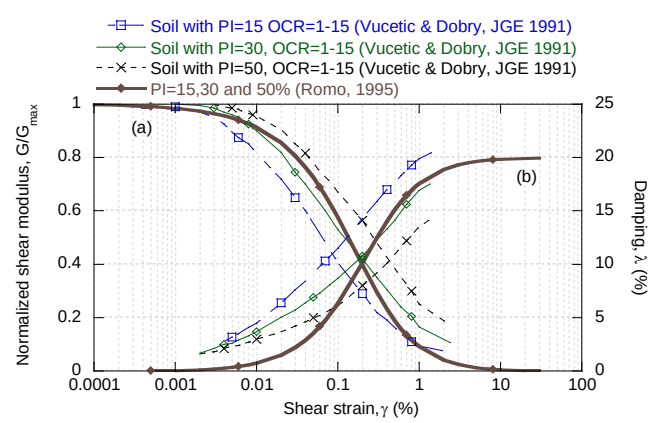


Fig. 7. Normalized shear stiffness (a) and damping (b) for clays used in the analysis.

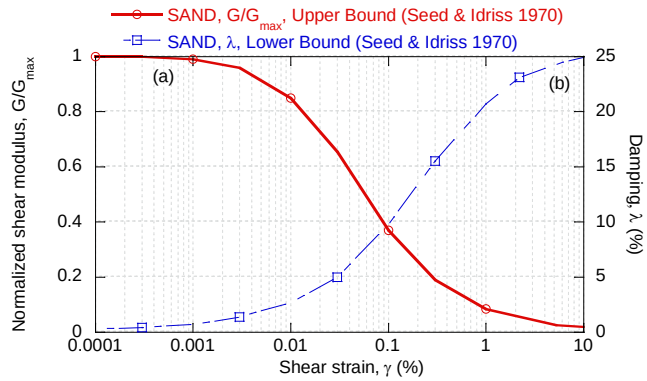


Fig. 8. Normalized shear stiffness (a) and damping curves (b) for sands used in the analysis.

Seismic Environment

The input motion used in the simulation (Fig. 9) was obtained from a time domain spectral matching of the response spectrum proposed in the Mexico city building code, RCDF, for the hill zone and type A structures (Fig. 9a), using the methodology proposed by Lilhanand [1988] as modified by Abrahamson [1993]. It is considered that this response spectrum represents conservatively the seismic environment likely to occur in the region.

CALIBRATIONS OF NUMERICAL MODELS

The calibrations of numerical models for seismic-soil-structure interaction, SSSI, studies are essential to reduce the inherent uncertainties associated with the analysis, which from the input parameter stand point, includes proper identification of subsoil conditions, soil and structural properties, and seismic environment, and to assess the validity of the final answer, considering that very often there is a lack of enough data to apply sophisticated numerical tools. This section presents the calibrations of the models proposed. The numerical simulation of the seismic response observed in one of the central supports of a vehicular bridge, hereafter referred to as Impulsora bridge was obtained. The numerical models were developed with the computer program SASSI 2000 (Lysmer *et al.* [2000]), using the flexible volume method. The flexible volume method is formulated in the frequency domain, through the complex response method and finite element technique as described by Lysmer [1978]. The whole soil-structure system is divided into two substructures: the foundation and the structure. In this partition, the structure consists of the superstructure plus the base minus the excavated soil. The foundation-structure interaction occurs at all basement nodes. Equivalent linear properties for the soil were estimated from 1-D wave propagation analysis using the program SHAKE (Schnabel *et al.* [1972]).

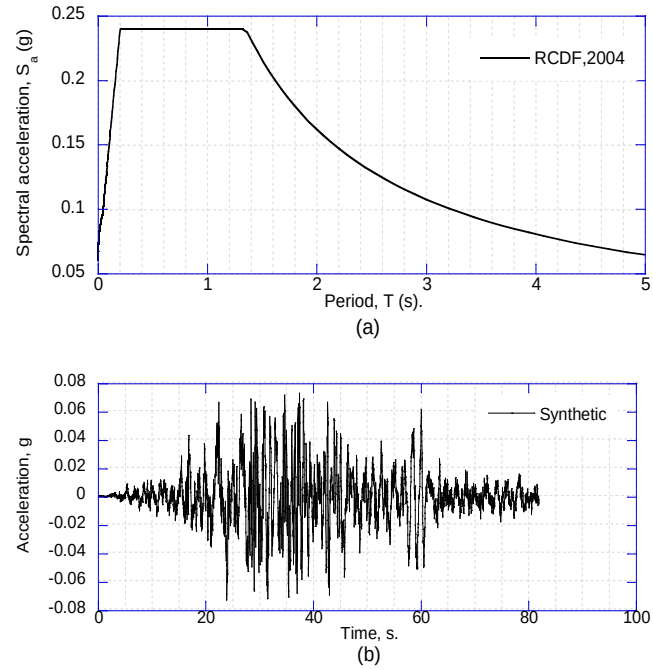
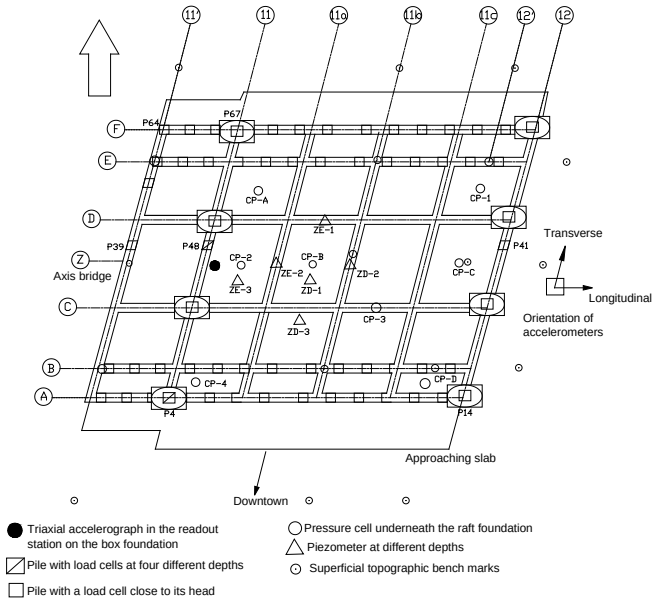


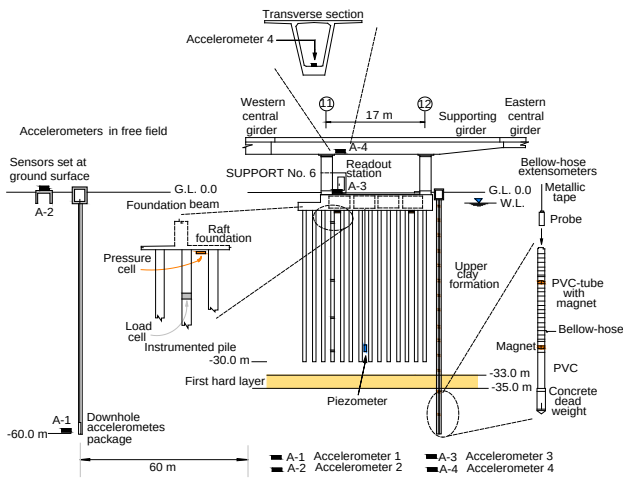
Fig. 9. (a) Proposed response spectra for the RCDF, 2004 for the Hill Zone and (b) synthetic earthquake.

The bridge has a mixed foundation consisting of a pile friction-box that was instrumented, along with the upper deck, to monitor the main geotechnical variables that control the structural behavior of this type of foundation system from the beginning of its construction up to now. In particular, the calibration of the model presented herein focuses only on seismic related aspects observed during one of the better documented cases, the June 15, 1999, Tehuacan, 7.0 M_w, earthquake.

Instrumented bridge. Impulsora bridge is located in the North-Eastern area of Mexico City as depicted Fig. 1. According to the Mexico City Building Code, the bridge is within the Lake Zone, which is characterized by the presence of very soft clay deposits interbedded with thin sand lenses. The bridge was instrumented at support 6, which corresponds to the central portion. As presented in Fig. 10, the foundation of this support consists of a partially compensated box foundation with friction piles. A plan view of the foundation is shown in Fig. 10a. The box foundation has a rhomboidal shape and 77 reinforced concrete piles which have a square section of 0.5 by 0.5 m, and 30 m long. The instruments are also presented in Figure 10a. The soil-structure system instrumentation is integrated by four accelerometers: one at 60 m of depth (A1), one at the surface (A-2) also in the free field, one at the box foundation center (A-3) and the last one in the upper support beam (A-4), 13 load cells, 6 piezometers and 8 pressure cells to measure soil-slab contact (Fig. 10b).



(a)



(b)

Fig. 10. (a) Plan view and (b) lateral view of Impulsora Bridge.

Subsoil conditions. The soil profile (Fig. 11) at the studied site presents a desiccated crust of clay at the top extending up to a depth of 1.0 m approximately, which is underlain by a 1.0 m layer of fill that rest on top of a soft clay layer with organic matter about 30.5 m thick. The water content of these materials ranged from 208 to 331 %, and the plasticity index from 224 to 312%. The undrained shear strength, s_u, varied from 10 to 15 kPa. Underlying the clay there is a 2.5 m average thick layer of very dense sandy silt ($(N_1)_{60}$ larger than 65), which sits on top of a stiff clay (s_u between 21 to 26 kPa) interbedded by sand lenses. The water content of this layer goes from 253 to 280% and the plasticity index from 188 to 243 % approximately. Underneath this elevation a competent layer of very dense sandy silts ($(N_1)_{60}$ lager than 100) is found. The corresponding in-situ V_s measurements are presented in Mendoza *et al.* [2009]. Average representative values of shear wave velocity of clayey materials, V_s, were found to be about 30 m/sec, at the upper soft clay layers (from 0 to 28 m) and from 30 to 100 m/s in the stiff clay layers. Sand lenses present values of V_s ranging from 340 up to 490 m/sec, in the deep hard deposits.

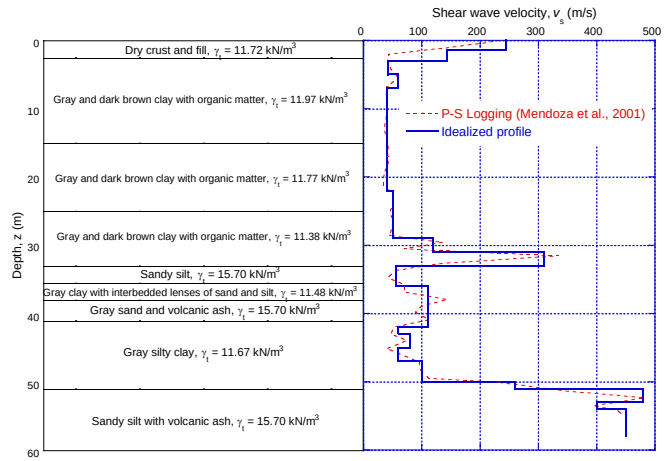


Fig. 11. Idealized soil profile and shear wave velocity distribution with depth of Impulsora Bridge.

Seismic soil-structure interaction analysis. The computation of the dynamic response of the soil deposit and of the soil-structure system was conducted for the June 15, 1999, Tehuacan Earthquake. The epicenter of this earthquake was located in the frontier between Puebla and Oaxaca State. It had a moment magnitude, M_w, of 7.0. Thus, it can be considered as a moderated-intensity earthquake. This event was recorded at the vertical array located in the free field. The locations and orientations of the accelerometers are presented in Fig. 10. Their orientations correspond to the longitudinal and transverse direction of the bridge. The dynamic response of the foundation-structure system was monitored by other two accelerometers, one located at the central portion of the box foundation (A-3), and one in the upper support beam (A-4).

Free field response. The temporal and spatial variations of the free field motions at the site were obtained to compute the equivalent linear properties to be used in the SSSI analysis, and later to evaluate the motions of the foundation-structure system placed in the free field seismic environment. Although recent developments in numerical methods and computational capabilities allow including 2-D and 3-D effects in ground motion assessment, for the problem at hand, a 1-D approximation was deemed appropriate, considering that for wide valleys with relatively shallow deposits, where material stiffness increases with depth, the approximation of vertically propagating seismic longitudinal shear (SH) waves through horizontally layered deposits has proven to reproduce, with reasonable degree of accuracy, the recorded ground motions on a wide variety of soil materials (i.e., Rosenblueth [1952]; Idriss and Seed [1986]; Romo and Jaime [1986]; Seed *et al.* [1994]). Furthermore, equivalent linear properties can approximately account for the small degree of nonlinearities expected in the dynamic response of Mexico City clays for this moderated seismic event. Thus, the solution of the 1-D SH wave propagation problem was obtained for the seismological,

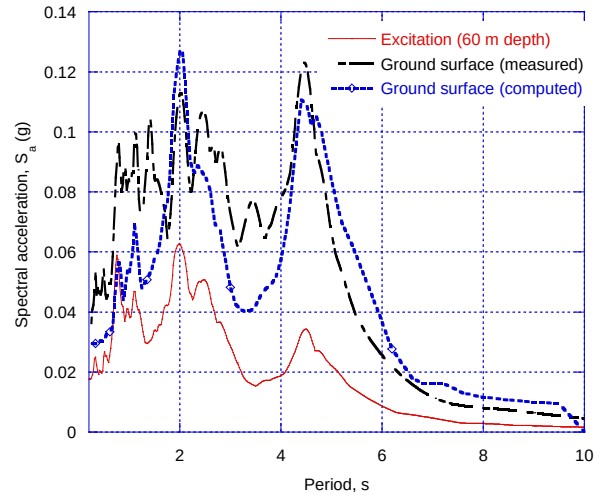
geologic, geotechnical, and geometrical characteristics using the computer program SHAKE as previously has been pointed out.

Input ground motion. The acceleration measured at the instrument located at 60 m of depth (A-1) during the seismic event was used as input ground motion for the analysis, and the waves were propagated to the surface. Thus, the effect of the source parameters and regional geology was, at least in principle, accounted for. The response spectra of the measured motions at the surface and at 60 m of depth are shown in Fig. 12 for the longitudinal and transverse components. The corresponding comparison between the measured and computed responses is also shown in Fig. 12. As it can be noticed in these figures, the computed response captures both the frequency content as well as the maximum spectral amplitudes for the longitudinal component.

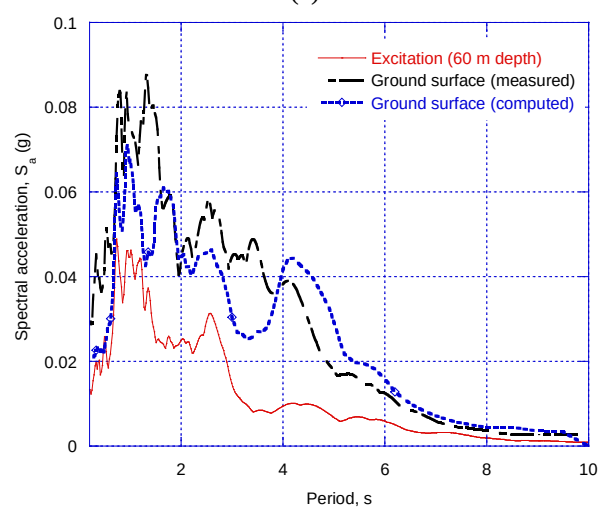
Soil-structure interaction. Two bidimensional models of the structure were developed for the analysis with the program SASSI2000, coupled to an axisymmetric tridimensional for the soil, one for the longitudinal direction and other for the transverse one (Fig. 13). The first model has a total of 76 quadrilateral elements and 300 beam elements. The second model has 28 quadrilateral elements and 420 beam elements. Each model was analyzed independently using the corresponding ground motion component as input motion. A halfspace transmitting boundary was used at the edges of the model to simulate free field conditions.

The soil profile was modeled as series of semi-infinite viscoelastic horizontal layers with equivalent linear properties (i.e., shear stiffness and damping) to account for soil non-linearities, resting on top of a viscoelastic halfspace (SASSI2000). The equivalent linear properties were computed at each depth through a 1D wave propagation iterative analysis (Mayoral *et al.* [2008]). Equivalent linear properties have been shown to yield good results based on the wide quasi-linear range behavior observed in Mexico City clays even for shear strains as large as 0.3% (Romo [1985], Romo *et al.* [1988]), due to their high plasticity.

As previously mentioned, the box foundation was represented with two-dimensional (2D) four nodded quadrilateral elements with equivalent volumetric weight and stiffness, representative of all the structural cells that comprise the box (Fig. 13). This approach allows for a better representation of both the geometrical mass and stiffness distribution within the foundation. Fig. 10 shows the bridge foundation axes selected for analysis. Axis A was considered for the longitudinal direction and axe 11 was chosen for the transverse direction.



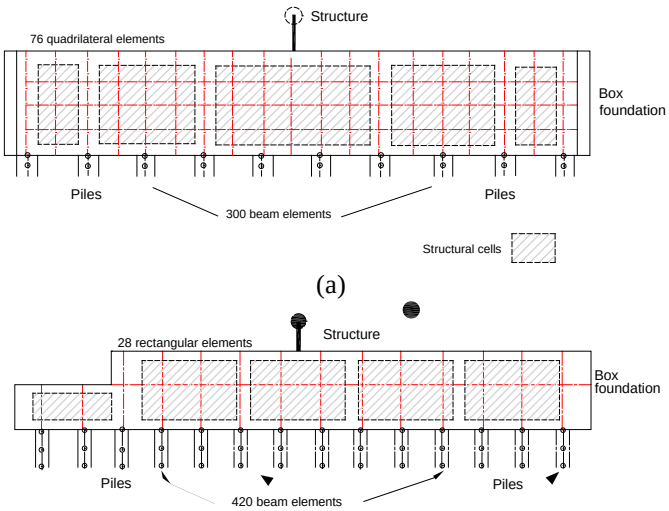
(a)



(b)

Fig. 12. Measured and computed response for longitudinal (a) and transverse (b) directions.

Regarding the piles, an equivalent stiffness was used considering tributary areas to account for collapsing the pile group into a row of piles, which were modeled as beam elements. The substructure damping was characterized by a Raleigh-type formulation. Considering that no gapping was expected to occur during the earthquake between the friction piles and the soil due to the stress increment caused by the presence of the building over the soil–foundation and the low intensity of the ground shaking, the beam elements are connected directly to the soil elements at the nodes, transmitting both strains and stresses.



(b)

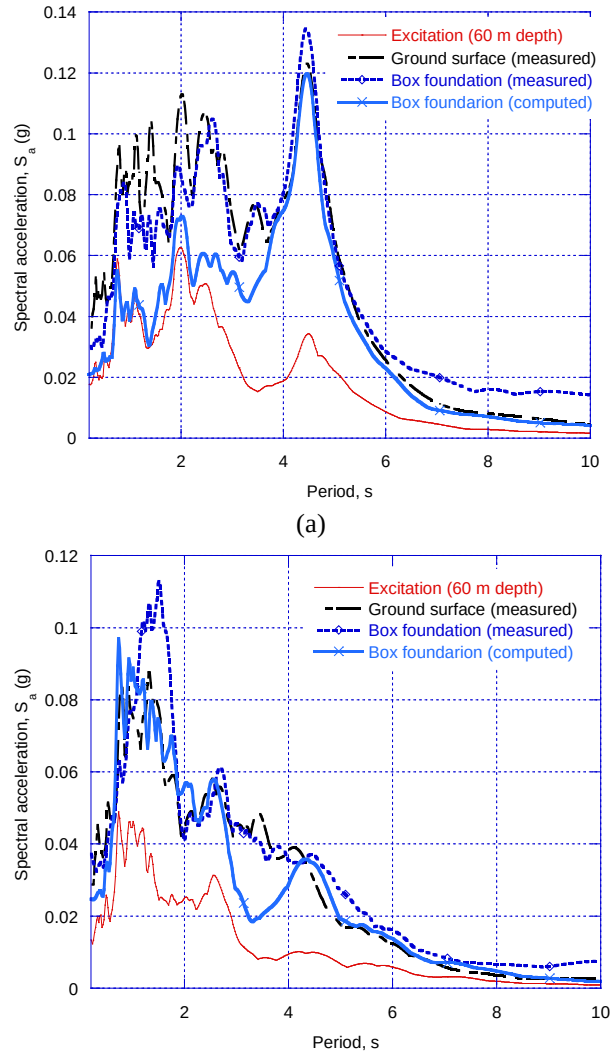
Fig. 13. Finite element model used for the longitudinal (a) and transverse (b) directions.

The total mass of the upper deck and columns was concentrated in a lumped mass and the superstructure was idealized with a stick-lumped model. This assumption was based on the observation that, for the cases studied herein, the transfer functions support beam/box foundation and central beam/box foundation, obtained from the measured response was close to one for the range of frequencies of interest (Mayoral *et al.* 2009). From this fact, it can be deduced that the bridge superstructure behaves almost as a rigid body with the foundation for this level of shaking. This may not be the case for higher intensity shaking, considering that the central beams are simply supported by the support beams. For the concrete structure, it was considered a Young modulus of 15,500 MPa, a Poisson ratio of 0.3, a unit weight of 23.5 kN/m^3 and a damping ratio of 3%. The structure damping was modeled with a Raleigh-type formulation.

Computed response. To asses the performance of the bridge–foundation system under the seismic environment considered, their seismic response was obtained at the central box node. The motions exhibited by this node are deemed representative of the whole box foundation behavior considering its large stiffness. Fig. 14 shows a comparison between the response spectra computed at the box foundation and the corresponding measured response, in the longitudinal and transverse directions. Overall, the models capture well the measured response for both the transverse and longitudinal directions.

THE URBAN OVERPASS CRITICAL SUPPORT SEISMIC RESPONSE

A bidimensional model of one of the more critical supports, coupled to an axi-symmetric 3D finite element model of the soil was developed to study both the transverse and longitudinal overpass seismic response (Fig. 15), using the computer program SASSI 2000.



(b)

Fig. 14. Response spectra comparison in the longitudinal (a) and transverse (b) directions.

A 2-D model is preferred over 3-D model because in a previous research Mayoral *et al.* [2009] demonstrated that using 2-D models can lead to a good approximation of the measured responses of an instrumented bridge located in the highly compressible soft clays found in Mexico City. The model has in the transverse and longitudinal directions 76 quadrilateral elements and 73 beam elements, respectively. Both transverse and longitudinal sets of models were analyzed using the same input motion, considering that these were obtained directly from time domain spectral matching of the recommended by the Mexico City building code, which is independent of the direction of the excitation. A half space transmitting boundary was used at both edges of the model to simulate the free field conditions.

Soil Model

The equivalent linear properties were computed at each depth through 1-D wave propagation iterative analyses (Mayoral *et*

al. [2008]) using the program SHAKE (Schnabel *et al.* [1972]). Studies carried out by several researches (e.g. Romo *et al.* [1988], Seed *et al.* [1988], Mayoral *et al.* [2008] and Romo *et al.* [2007]) have proven that using equivalent linear properties is enough to represent the soil-nonlinearities both in clayey materials and the sandy silts, at least for moderate to high level of shaking (M_w=6 to 8.2).

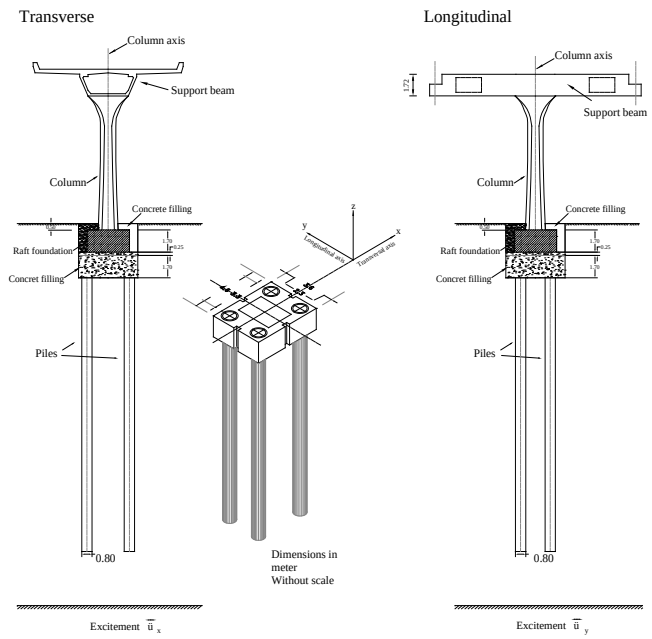


Fig. 15. Schematic representation of support analyzed

Substructure Model

The piles were modeled with tridimensional beam elements and the raft foundation and the concrete fillings with two-dimensional (2D) quadrilateral elements with equivalent volumetric weight and stiffness, representative of all the raft foundation. This approach allows for a better representation of both the geometrical mass and stiffness distribution within the foundation and upper deck. Both the transverse and longitudinal components were analyzed. As it was assumed with the Impulsora bridge, the piles were modeled as beam elements with equivalent stiffness considering tributary areas to account for collapsing the pile group into a row of piles. The substructure damping was characterized by a Raleigh type formulation. Due to the presence of the raft foundation and improved ground area, no gapping was expected to occur between the piles and the upper portion of the soil profile. The beam elements are connected directly to the soil elements at the nodes, transmitting both strains and stresses. Although beam elements available in SASSI2000 cannot capture the influence of the pile diameter during pile to pile interaction, in this particular case, due to the ground conditions the shaking intensity and the structural response is relatively low. Therefore, pile to pile interaction is expected to be small. The potential cracking of the piles was not considered, because as design requirement the piles must work in their elastic range.

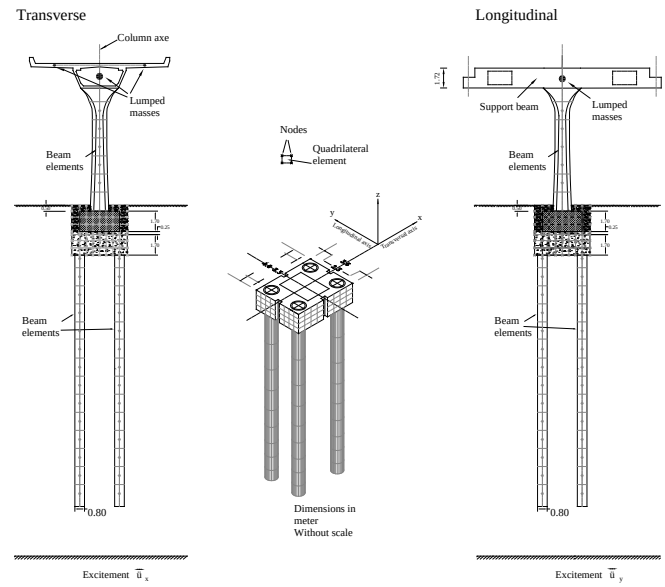


Fig. 16. Finite element models for support analyzed

Superstructure Model

For the transverse direction, the total mass of the upper deck and support beam was concentrated in three lumped masses to distribute the upper deck inertia (Fig. 16), connected by rigid members to the beams elements that represent the columns, simulated with eleven beam elements. This allows modeling potential rocking of the overpass deck. This assumption was based on the fact that, for the cases studied herein, the support beam, and the deck are structurally tied to the column. The structure, including the column and upper deck were pre-stressed and made of high strength concrete, as presented in Table 1. Thus, for the column and raft foundation it was considered a Young modulus of 30,000 MPa, a Poisson ratio of 0.3, a unit weight of 23.5 kN/m^3 and a damping ratio of 3 %. The structure damping was modeled with a Raleigh type formulation. For the concrete filling it was assumed a young modulus of 19,400 MPa.

Table 1. Strength concrete of structural members

Structural Element	f'_c (kPa)
Reinforced concrete slab	24500
Piles	24500
Fluid filling	24500
Prefabricated columns and raft foundations	59000
Fluid filling for the excavation	690

Computed response

Figure 17 shows the transfers functions between foundation and free field, and the support beam and foundation. It can be noticed that the support beam behaves almost as a rigid body with the foundation within the range of frequencies of 0.3 to 1.5 Hz approximately. Similar conclusions were obtained by Mayoral *et al.* 2009 from the measures taken at the Impulsora Bridge for several earthquakes. Furthermore, the foundation also follows the free field within this frequency range. Thus, earthquakes generated in the subduction zone of the Pacific Ocean (e.g. 1985 Michoacan Earthquake), which have caused the largest damage observed in the City up to now, will not tend to amplify the response of the support beam importantly because the energy of these earthquakes are mostly concentrated around 0.5 Hz. The response of the structure is in the high frequency range, which is consistent with the high stiffness of both the foundation system, and the column supporting the upper beam. On the other hand, in the high frequency range (i.e. larger than 2 Hz), an important amplification of the foundation and structure responses of about 3 and 2.2, with respect to the free field and foundation respectively, can be seen in figure 18, which presents the relative amplitude of spectral accelerations computed between the structure and the foundation, and the foundation and the free field. This effect of amplification of the foundation motion with respect to the free field in the high frequency range and in the transverse component, has been also measured in narrow structures located within the Mexico City Valley (figure 19), and can be explained in terms of the energy feeding back from the bridge swinging motion to the incoming waves from the soil. Although important, these spikes observed in the high frequency range of the computed spectral ordinates will not modify importantly the computed displacement time histories (see figure 20), having maximum relative movements on the order or 3 cm between the support beam and the foundation. The displacements shown herein correspond to total displacements; the distortion generated between the raft foundation and central column was obtained from the subtraction of the maximum displacements among both points divided by the length columns. The distortion obtained at the support analyzed, in the transverse direction, was 0.0011 (Table 2). Overall, the simulation is consistent with observations gathered in the Impulsora bridge.

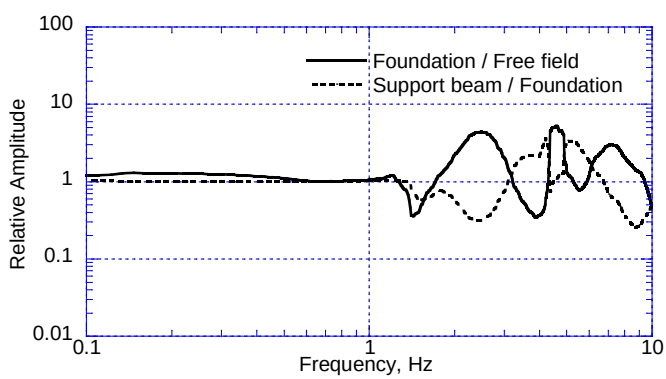


Fig. 17. Transfer functions of the critical support at the transverse direction.

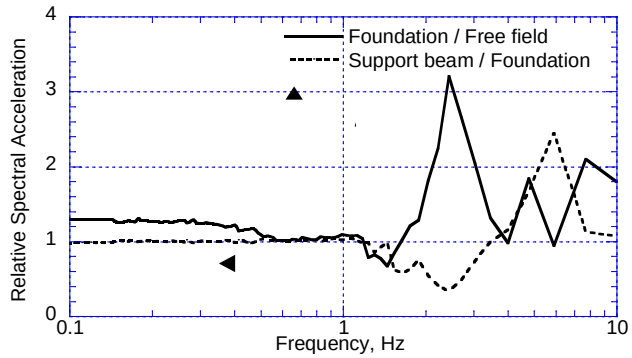


Fig. 18. Interaction effect in the critical support at the transverse direction.

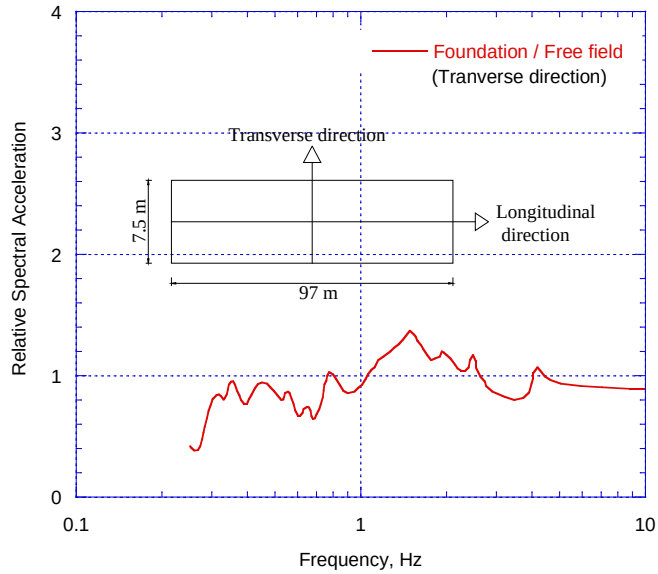


Fig. 19. Interaction effect in the instrumented building Kennedy at the transverse direction (After Romo 1990).

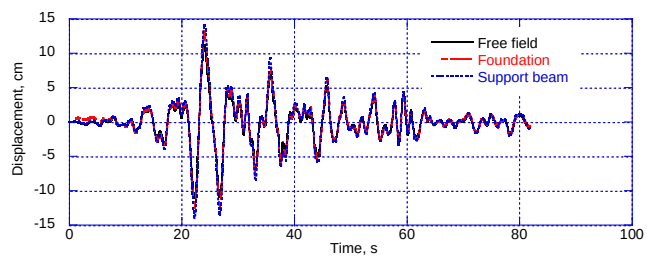


Fig. 20. Displacement time histories computed at the transverse direction.

A similar trend is observed in the longitudinal component. Again the response of the structure is high frequency, and the model shows amplification in the high frequency range (figure 21). The spike observed in the high frequency range of the computed response can also be spurious amplifications generated when modeling the upper structure as lumped mass, considering that in reality a frame will develop by the restriction caused by friction between the support and central beams, which in turn is connected to the support beam of the

nearby supports. These spikes in the computed response however, will not affect the displacements time histories computed, as depicted in figure 22. The maximum relative displacement between the ground and the upper beam in the longitudinal direction is about 3.5 cm. The distortion computed in the longitudinal direction was 0.0017.

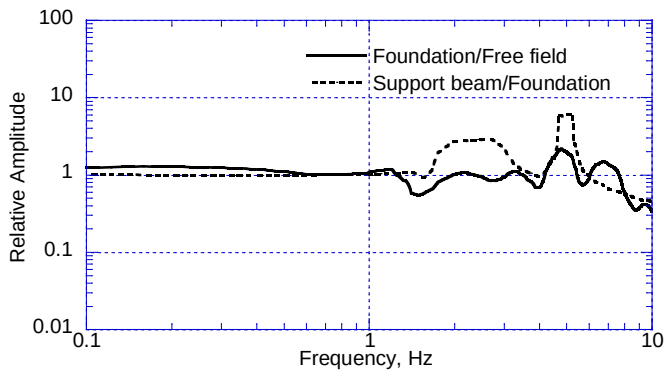


Fig. 21. Transfer functions of the critical support at the longitudinal direction.

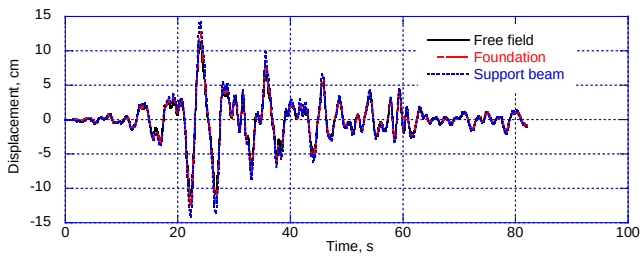


Fig. 22. Displacement time histories computed at the longitudinal direction.

Table 2. Computed maximum displacements and distortions

Free field	Transverse direction			Longitudinal direction		
	Found.	Support beam	Column distortion	Found.	Support beam	Column distortion
(cm)	(cm)	(cm)		(cm)	(cm)	
11.33	13.25	14.21	0.0011	12.91	14.39	0.0017

CONCLUSIONS

This paper presents the seismic analysis of one of the most critical supports of an urban overpass to be built in the hill and transition zones in Mexico City. Initially, the analysis methodology and models were validated analyzing the seismic response that other instrumented bridge exhibited during the June 15th, 1999 Tehuacan (M_w=7) Earthquake. This second bridge is located also within the surroundings of Mexico City, but in the lake zone, where highly compressible clays are found. Good agreement was observed between the computed and measured response. Based on the data gathered, it was concluded that the pile-box foundation system follows the ground in the longitudinal direction, having a minimum dynamic interaction. However, in the transverse direction, the interaction effects are significant, increasing the peak spectral response of the upper deck in almost 30 % with respect to that measured at the foundation for the fundamental period of the soil.

Regarding the urban overpass, an important amplification of the foundation and structure responses of about 3 and 2.2, with respect to the free field and foundation respectively was computed. These amplifications, however will not lead to important structure or foundation displacements.

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