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Effects of Foundation Soil Stiffness on the 3-D Modal Characteristics and Seismic Response of a Highway Bridge

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Abstract

This paper presents the main results of an investigation into the influence of foundation soil stiffness on the modal characteristics and seismic response of a highway bridge with spread foundations. A sensitivity study including six types of soil profiles is first performed to examine the effects of foundation soil stiffness on critical dynamic response parameters of the bridge. 3-D modal characteristics of lateral, vertical, longitudinal and torsional vibrations modes are identified and compared for various soil and rock conditions. The study is then extended to estimate the time history nonlinear seismic response of coupled foundation soil-bridge models utilizing an extension of the Fast Nonlinear Analysis (FNA) algorithm. Two different approaches for modeling soil foundation interaction are considered: a PGA consistent linear soil model and an advanced non linear soil model. Results show, among others, the major influence of soil flexibility effects on the modal characteristics and seismic response behavior of the bridge. Moreover, the results illustrate clearly the importance of soil nonlinearity on coupled foundation soil-bridge response. It follows, from a design perspective, that analytical models used for the seismic analysis of bridge structures should explicitly consider the effects of soil-structure interaction including nonlinear soil behavior.

Keywords: *highway bridge, 3-D modal characteristics, foundation soil stiffness, sensitivity analysis, nonlinear soil-structure interaction, earthquake bridge response, fna algorithm*

1. Introduction

Current structural design procedures used for the seismic analysis and design of bridges are based upon the common assumption that foundation soil of bridge supports is rigid (i.e., embedded in solid rock). Little attention, if any, is usually given to Soil Structure Interaction (SSI) effects on dynamic behavior of bridges despite the fact that failure of bridge substructure and foundations during earthquakes are the most common cause of damage or collapse of bridge structures. In part, this may be attributed to (i) complexity associated with SSI effects and lack of code specifications with respect to seismic analysis and design of bridge-foundation soil systems (ii) difficult estimation of stiffness and damping characteristics of the wide variety of soil profiles encountered in practice (iii) very limited number of investigations including comparative numerical and experimental studies on SSI effects. Past earthquake investigations have led to increased concerns regarding the importance of soil response and SSI effects on the response analysis and performance of several bridge structures. (e.g., Crouse *et al*., 1987; Levine and Scott, 1989; Spyrakos, 1990; Trifunac and Todorovska, 1996; Tangaonkar and Jangid, 2003). In general, SSI effects are considered for bridge structures founded on deformable soils. Foundation soil stiffness and soil conditions can have substantial effects on vibration control and seismic performance of structures (Cook *et al*., 1995; Elassaly *et al*., 1995; Tiliouine and Moussaoui, 1996; Lihua, 2012). From a design perspective, bridge types that are particularly sensitive to SSI effects under seismic strong ground motions include in particular, bridge with integral (Spyrakos and Loannidis, 2003) and full height abutments (Tsang *et al*., 2002), cable supported bridge systems (Elassaly *et al*., 1995; Shehata and Toshi, 2013) as well as long span R.C. box girder bridges. Bridge dynamics characteristics are of critical importance in seismic analysis of foundation soil-bridge systems since frequency characteristics provide extremely useful information on possible resonant conditions with maximum dynamic amplification in various modes and on selection of an appropriate time step for transient response analysis. Modal shapes are also needed to identify most flexible regions of foundation soil-bridge systems and to compute the effective modal mass in order to determine the contribution of the most significant modes to the dynamic response of these systems. However, the dynamic characteristics of such systems can be altered during severe ground motions (Hardin and Drnevich, 1972a; 1972b; FEMA356/ASCE, 2000).

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In particular soil deformations and stress intensity can have significant influence on mechanical properties of soils (Hardin and Drnevich, 1972a; 1972b) and response of soil structure systems during strong earthquakes (e.g., Kobayashi, 2002). Effects of nonlinear SSI on bridge response can result in large seismic ductility demands (Pecker, 2011).

Research on seismic SSI over several decades have been essentially based on the assumption of linear (or at most equivalent-linear) elastic soil behaviour (e.g., Idriss, 1968; Vetetsos and Wei, 1971; Wolf, 1988). This assumption is to a large extent, consistent with the prevailing norms of seismic design philosophy of foundation soil-structure systems: i.e., to avoid full mobilization of strength (or substantial plastic deformation) in foundation elements by guiding failure to the aboveground structure elements.

However, seismic motions recorded in the last twenty five years (e.g., Loma Preita, (1989); Northridge; (1994) and Kobe (1995) earthquakes) have led to an increased awareness concerning effects nonlinearity on the dynamic response of highway bridges subjected to severe earthquake ground motions. The need for consideration of soil nonlinearity in the design of foundation soil-structure systems is well recognized in the specialized literature (e.g. FEMA356/ASCE, 2000; EC8, 2005; AASHTO, 2011), despite a significant lack of reliable nonlinear SSI models capable of capturing the permanent and cyclic soil deformations under seismic strong ground motions. Much effort (experimental as well as analytical) has been put in this direction (FEMA356, 2000 and FEMA440, 2005) in the last fifteen years to develop efficient analysis procedures which can lead to a better understanding and a more realistic prediction of the nonlinear dynamic response of the coupled foundation soil-bridge systems including the influence of both supporting soil stiffness and inelasticity (Paolucci, 1997; Faccioli *et al*., 1998; Gajan *et al*., 2005; Paolucci *et al*., 2008; Anastasopoulos *et al*., 2011; Pecker, 2011).

In the first part of the paper, the significance of accounting for the SSI effects on the dynamic characteristics of a typical highway bridge is examined. In order to quantify the effects of SSI, various modal response parameters of a typical bridge-foundation soil system are compared to those of the bridge resting on rigid foundations.

Sensitivity studies have been performed to investigate the effects of foundation soil stiffness on the modal response of the bridge. A total of six types of foundation soil profiles, ranging, from S_E high soft soil to S_A low rock conditions have been considered. Moreover, 3-D modal characteristics of lateral, vertical, longitudinal and torsional vibrations of higher modes of the bridge are identified and compared for stiff soil and rock conditions.

In the second part of the paper and in order to further assess the effects of nonlinear foundation soil stiffness on the overall dynamic response of the highway bridge under earthquake strong ground motions, two different soils models are employed for a comparative assessment of the bridge responses: the commonly used linear lumped-parameter soil model and an advanced plasticity soil model based on the hysteretic Bouc-Wen model.

In the nonlinear hysteretic Bouc-Wen soil model, the SSI is simulated using translational and rotational Wen link elements lumped at the centroid of spread foundations. In order to quantify the effects of SSI, the nonlinear time history responses of critical bridge seismic design parameters in terms of pier base shear, displacement at mid-central span and Normal Rubber Bearing (NRB) shear strain at bridge abutments have been evaluated and compared to those of bridge with rigid foundations.

2. Description of Bridge Structure

The Beni-Chograne highway bridge shown in Fig. 1, is a typical long span R.C. box girder bridge with bearing devices at abutments. It is located in earthquake zone IIa of North Western Algeria characterized by an expected Peak Ground Acceleration (PGA) equal to 0.165 g. In order to clarify the effects of foundation soil stiffness on the 3D modal characteristics and overall seismic response of the bridge, a sensitivity analysis based on six types of foundation soil profiles is considered (cf. Section 3).

The bridge has an overall length of 216 m and consists of three continuous spans in prestressed concrete with a mid-span length of 100 m and two end spans of 58 m length each, as indicated in Fig. 2(a).

The superstructure consists of a longitudinally R. C. deck, 9.50 m wide with variable height, (see Fig. 2(b)) and moment of inertia in accordance with the following expression:

$$I = I_0\left(1 + K\left(\frac{x-\alpha L}{L-\alpha L}\right)^2\right)^{\frac{5}{2}} \tag{1}$$

In this expression, K is a constant determined as follows:

Fig. 1. Beni-Chograne Bridge Configuration

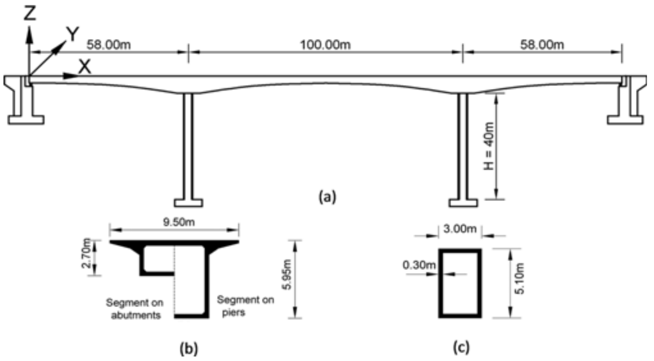

Fig. 2. Description of Bridge: (a) Elevation of Bridge, (b) Cross-section of Segments, (c) Cross-section of Pier

$$K = \left(\frac{I_1}{I_0}\right)^{0.4} - 1 \tag{2}$$

where I_1 represents the moment of inertia of segment on pier and I_0 the moment of inertia of segment cross section on abutments or piers. The parameter α is equal to the ratio of end span length with constant cross section, (αL), to the total length (L) of the end span.

Young's moduli of concrete are taken respectively as 33 GPa for the R.C. piers and 36 GPa for the bridge deck. Mass density is taken as 2500 Kg/m^3. A value of 5% damping in the first and second modes of vibration is considered for the seismic time history response of the bridge. The bridge is supported by two intermediate RC piers and two abutments, both resting on rigid spread footings. The intermediate R.C. piers of equal height of 40 m, with identical hollow rectangular cross sections, are fixed at their upper ends (Fig. 2(c)). Normal rubber bearings are located at abutments only.

3. Types of Foundation Soil Profiles

In order to assess the effect of foundation soil stiffness on the vibratory characteristics of the bridge, a sensitivity study is performed using six types of foundation soil profiles in accordance with prevailing site classifications (e.g., ATC, 1996; FEMA356, 2000) ranging from low to high soil materials properties: S_D: stiff soil; S_C: dense soil, soft rock and S_B: rock. Typical values of corresponding soil material properties such as weight density, shear wave velocity, initial shear modulus given in Table 1 in conventional units are derived using ATC data (ATC, 1996). Low soft soil conditions are not included in the parametric study as they require site specific geotechnical investigations and foundation systems other than spread footings. Also high hard rock conditions (S_A) are not considered as they yield practically similar results as rock conditions.

Table 1. Typical Soil Material Properties for Six Representative Soil Profiles

Type of soil	Description	Shear wave velocity Vs (m/s)	Weight density γ (KN/m^3)	Initial shear moduli G_0 (MPa)	
				Low	High
S_A	Hard Rock	>1524	-	5210	-
S_B	Rock	762 to1524	>22	1303	5210
S_C	Dense soil-Soft Rock	366 to 762	21 to 22	287	1303
S_D	Stiff Soil	183 to 366	19 to 21	65	287
S_E	Soft Soil	< 183	14 to 19	æ	< 65

SE and SA types of soil conditions are included in Table 1 and Table 2 for the purpose of completeness only.

The corresponding effective shear modulus G and effective shear wave velocity V_s' have then been estimated for a PGA value equal to 0.165 g and each type of soil profile using procedures given in (FEMA356, 2000). Final results are presented in Table 2 below. Clearly for small reduction factors, $G \approx G_0$ and $V_s \approx V_s'$.

For the sake of the clarity stiffness of equivalent springs and damping of equivalent dashpots representing the foundation soil properties are discussed in the next section.

4. Foundation Soil Stiffness and Damping Matrices

The values of soil stiffness and viscous damping corresponding to the degrees of freedom at the base of the supporting piers are considered herein frequency independent (i.e., the values of associated impedance functions at frequencies close to zero) and can be computed from the solution of a circular footing bonded to the surface of an elastic half space (e.g., Gazetas, 1991; Yohchia, 1996; Wolf, 1997):

$$K_x = \frac{8G}{2-\upsilon} R_x \qquad K_{\theta_x} = \frac{8G}{3(1-\upsilon)} R_{\theta_x}^3 \tag{3}$$

$$C_x = \frac{4.6G}{(2-\upsilon)V_s'} R_x^2 \qquad C_{\theta_x} = \frac{0.4G}{(1-\upsilon)V_s'} R_{\theta_x}^4 \tag{4}$$

Table 2. Effective Shear Moduli and Shear Wave Velocity for Six Representative Soil Profiles (PGA = 0.165 g)

Type of soil	Description	Poisson's ratio ν	Effective shear modulus G (MPa)		Effective shear wave velocity V_s' (m/s)	
			Low	High	Low	High
S_A	Hard Rock	0.25	5210	-	1524	-
S_B	Rock	0.25	1290	5158	758	1516
S_C	Dense Soil-Soft Rock	0.30	261	1185	349	727
S_D	Stiff Soil	0.40	53	235	166	331
S_E	Soft Soil	0.45	-	< 32	-	< 128

$$K_y = \frac{8G}{2-\upsilon}R_y \qquad K_{\theta_y} = \frac{8G}{3(1-\upsilon)}R_{\theta_y}^3 \tag{5}$$

$$C_y = \frac{4.6G}{(2-\upsilon)V_s'}R_y^2 \qquad C_{\theta_y} = \frac{0.4G}{(1-\upsilon)V_s'}R_{\theta_y}^4 \tag{6}$$

$$K_z = \frac{4G}{1-\upsilon}R_z \qquad K_{\theta_z} = \frac{16G}{3}R_{\theta_z}^3 \tag{7}$$

$$C_z = \frac{3G}{(1-\upsilon)V_s'}R_z^2 \qquad C_{\theta_z} = \frac{0.8G}{V_s'}R_{\theta_z}^4 \tag{8}$$

The coefficients (K_x, C_x), (K_y, C_y) and (K_z, C_z) denote translational stiffness and associated dashpot coefficients along the x, y and z axis respectively, while $(K_{\theta_x}, C_{\theta_x})$. $(K_{\theta_y}, C_{\theta_y})$ and $(K_{\theta_z}, C_{\theta_z})$ denote rotational stiffness and corresponding dashpot coefficients about the same axes. The extra-diagonal terms of the 6×6 foundation soil stiffness and damping matrices can be computed from:

$$K_{x\theta_y} = \frac{0.56G}{(2-\upsilon)}R_{\theta_y}^2 \qquad K_{y\theta_x} = \frac{0.56G}{(2-\upsilon)}R_{\theta_x}^2 \tag{9}$$

$$C_{x\theta_y} = \frac{0.4G}{(2-\upsilon)V_s'}R_{\theta_y}^3 \qquad C_{y\theta_x} = \frac{0.4G}{(2-\upsilon)V_s'}R_{\theta_x}^3 \tag{10}$$

For a rectangular foundation with dimensions L and B (where, L: long side dimension of contact area; B: short side dimension of contact area). The radius for the equivalent circular foundation is given by:

$$R_x = R_y = R_z = \sqrt{\frac{BL}{\pi}} \tag{11}$$

$$R_{\theta_x} = \left(\frac{BL^3}{3\pi}\right)^{\frac{1}{4}} \tag{12}$$

$$R_{\theta_y} = \left(\frac{LB^3}{3\pi}\right)^{\frac{1}{4}} \tag{13}$$

$$R_{\theta_z} = \left(\frac{BL(B^2+L^2)}{6\pi}\right)^{\frac{1}{4}} \tag{14}$$

The spring coefficients K (associated to a given degree of freedom) for shallow rectangular footings, used in the present study, have been determined by modifying the corresponding solution for circular footings bonded to the surface of an elastic half-space, as follows:

$$K = \alpha\beta K_0 \tag{15}$$

where, K_0 = Stiffness coefficient for the equivalent circular footing. The factors α and β are foundation shape and embedment correction factors corresponding to the given degree of freedom. These factors can be evaluated using procedures presented in (e.g., Yohchia, 1996; FEMA273; 1997).

The values of attached soil mass corresponding to the degrees of freedom at the base of the supporting of piers can be expressed as:

$$M_x = 0.28\rho R_x^3 \qquad M_{\theta_x} = 0.49\rho R_x^5 \tag{16}$$

$$M_y = 0.28\rho R_y^3 \qquad M_{\theta_q} = 0.49\rho Ry^5 \tag{17}$$

$$M_z = 1.50\rho R_z^3 \qquad M_{\theta_z} = 0.70\rho R_{\theta_z}^5 \tag{18}$$

The coefficients M_x, M_y and M_z designate translational masses along the x, y and z axes respectively, while $M_{\theta_x}, M_{\theta_y}$ and M_{θ_z} denote rotational masses about the same axes.

The values of foundation stiffness and viscous damping coefficients for the six above mentioned soil profiles are summarized respectively in Table 3 and Table 4, below.

It is seen from Table 3 and Table 4 that shear modulus reduction effects on soil stiffness and soil damping coefficients are significantly more pronounced for the softer soils. Moreover, the coupling terms of foundation soil stiffness and damping matrices are negligible in comparison to the other terms of the corresponding matrices.

Table 3. Coefficients of Foundation Soil Stiffness Matrix for Six Representatives Soil Profiles

Type of soil	S_D (Stiff soil)		S_C (Dense soil-Soft Rock)		S_B (Rock)	
Soil Stiffness	Shear modulus G, (MPa)					
	Low	High	Low	High	Low	High
	53	235	261	1185	1290	5158
K_x (MN/m)	2881	12738	13305	60417	63850	255401
K_y (MN/m)	2812	12433	12986	58971	62323	249290
K_z (MN/m)	3018	13342	12691	57631	58518	234070
K_{θ_x} (MN.m/rd)	83168	367691	349754	1588234	1612668	6450672
K_{θ_y} (MN.m/rd)	45271	200145	190382	864525	877825	3511301
K_{θ_z} (MN.m/rd)	80981	358020	397314	1804203	1962815	7851259
$K_{x\theta_y}$ (MN.m/rd)	475	2102	2195	9969	10536	42144
K_{θ_x} (MN.m/rd)	713	3153	3293	14954	15804	63215

Table 4. Coefficients of Foundation Soil Damping Matrix for Six Representative Soil Profiles

Type of soil	S_D (Stiff soil)		S_C (Dense soil-Soft Rock)		S_B (Rock)	
Soil Damping	Shear modulus G, (MPa)					
	Low	High	Low	High	Low	High
	53	235	261	1185	1290	5158
C_x (MN.s/m)	33	72	76	165	174	349
C_y (MN.s/m)	33	72	76	165	174	349
C_z (MN.s/m)	57	126	120	261	265	530
C_{θ_x} (MN.m.s/rd)	364	806	766	1672	1697	3394
C_{θ_y} (MN.m.s/rd)	162	358	341	743	754	1509
C_{θ_z} (MN.m.s/rd)	316	698	775	1690	1839	3677
$C_{x\theta_y}$ (MN.m.s/rd)	12	27	28	61	64	128
$C_{y\theta_x}$ (MN.m.s/rd)	22	49	51	111	118	235

5. Bridge Modelling

The superstructure and substructure of the highway bridge modelled as a lumped mass system divided into a number of small discrete 3-D frame elements. Each adjacent element is connected by a node and at each node six degrees of freedom are considered: three translational in X, Y and Z directions and three rotational about the same directions (see Fig. 3).

The entire bridge system is approximated analytically by the 3-D FEM model presented in Fig. 4.

The bridge superstructure bridge consists of rigid abutments in longitudinal direction and is connected at lower columns by rigid elements. The support provided by the abutment is assumed to be fixed against lateral and vertical directions. It also is fixed against rotation about the longitudinal axis of the superstructure and has two rectangular normal rubber bearings. The rubber bearings used in the present study consist of alternate layers of

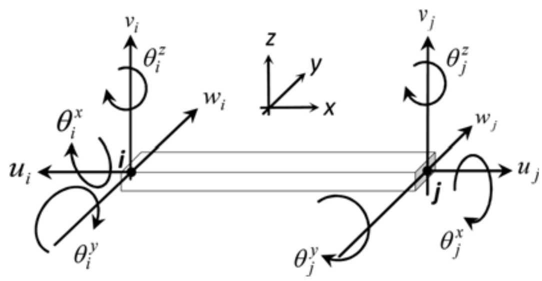

Fig. 3. Nodal Displacement of a 3-D Frame Element

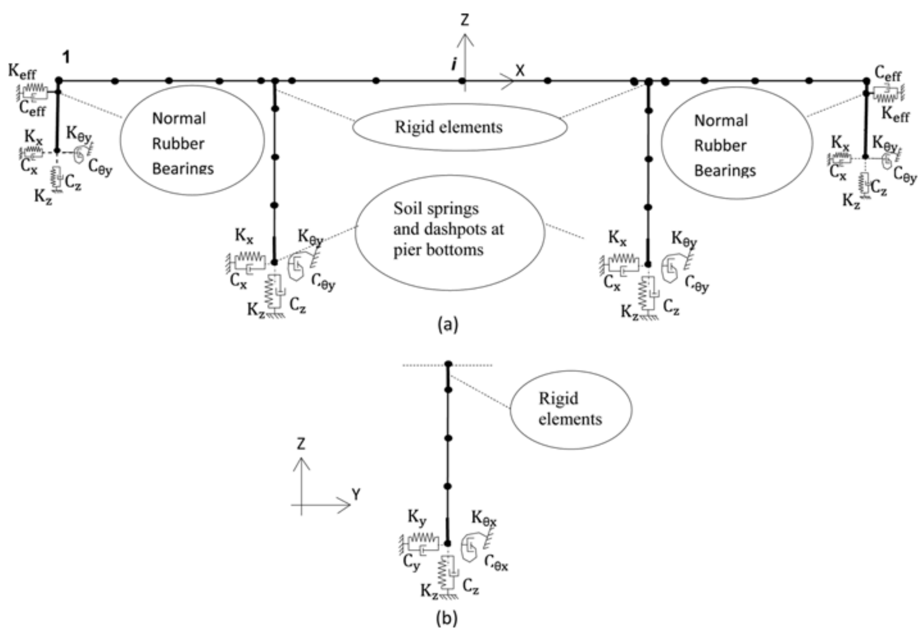

Fig. 4. 3-D Analytical Model of Study Highway Bridge: (a) Finite Element Model of Bridge; (b) Finite Element Model of Bridge Pier in Lateral Direction

rubber and steel plates. Due to the presence of alternate steel plates, these bearings are very stiff in the vertical direction but flexible in the longitudinal direction.

The stiffness and damping parameters of the rubber bearings are characterized by the effective stiffness (K_{eff}) and viscous damping (C_{eff}) in the longitudinal direction, as shown in Fig. 4, and can be expressed as:

$$K_{eff} = \frac{4\pi^2 W}{gT_b^2} \tag{19}$$

$$C_{eff} = 2\xi\sqrt{\left(\frac{W}{g}\right)K_{eff}} \tag{20}$$

where, W is the weight acting on an individual bearing; g is the gravitational acceleration constant and T_b is the time period of the bearing. The rubber bearings are modelled using a bilinear hysteretic model with effective stiffness in the longitudinal direction (e.g., Tiliouine and Ouanani, 2012).

6. Modal Analysis of Foundation Soil-Bridge Systems and Sensitivity Studies

Dynamic characteristics can be investigated using numerical or experimental techniques. Traditional modal testing has been extensively used in the past (e.g., Ewins, 2000). More recently, however, operational modal analysis has been utilized with advantage to extract structural dynamic characteristics from ambient vibrations (e.g., Ribeiro *et al*., 2012) and forced vibrations (e.g., Zwolski and Bien, 2011). Both stochastic subspace identification and enhanced frequency domain decomposition techniques in the time and frequency domains respectively, have been used successfully to identify dynamic properties of bridge prototypes (e.g., Altunisik *et al*., 2012).

Alternatively, various numerical procedures (e.g., Wilson, 2002; Chopra, 2011) have been devised to solve the eingenvalue problem resulting from the free vibration of response of structures. Both subspace iteration and Load Dependent Ritz (LDR) vectors have been used with advantage (e.g., Clough and Penzien, 1995; Tiliouine and Moussaoui, 1996; Wilson, 2002).

This being the case, the solution of the free vibration eigenvalue problem of the soil spring-bridge model, (when soil damping effects are not important) can be determined by solving the N × N system of matrix equations:

$$([K]-\omega_i^2[M])X_i = 0 \qquad i = 1,2,\ldots N \tag{21}$$

$$X_i = \{X_1, X_2, \ldots, X_n\}^T \tag{22}$$

where,

$$X_1 = \{u_1, w_1, v_1, \theta_1^x, \theta_1^y, \theta_1^z\}^T;\ X_2 = \{u_2, w_2, v_2, \theta_2^x, \theta_2^y, \theta_2^z\}^T$$
$$X_n = \{u_n, w_n, v_n, \theta_n^x, \theta_n^y, \theta_n^z\}^T$$

where, X_i denotes the ith modal displacement vector and ω the corresponding circular frequency. In the above expression $[K]$ and $[M]$ represent respectively the assembled stiffness and mass matrices of foundation soil-bridge system obtained by assembling the soil stiffness and mass matrices respectively with the structural elementary matrices.

$$[K_e] = \iiint [B]^T[D][B]dV \tag{23}$$

$$[M_e] = \iiint \rho[N]^T[N]dV \tag{24}$$

where, $[B]$ represents the derivative matrix of shape functions and N the shape functions matrix (e.g., Zienkiewicz and Taylor. 2005). ρ and D denote mass density and elasticity matrix respectively. The number of vibration modes to be retained in modal analysis is generally determined by using an effective modal mass equal at least to 90% or 95% of the total mass corresponding to a given direction.

The percentage of the total mass represented in the direction j by a truncated set of n $[M]$ orthonormal eigen-vectors can be determined from (e.g., Tiliouine and Moussaoui, 1996; Chopra, 2011).

$$e_{n,j} = \frac{1}{\{r_j\}^T[M]\{r_j\}}\left(\sum_{i=1}^{n} p_i^2 \cdot j\right) \times 100 \tag{25}$$

where, $p_{i,j}$ is the participation factor for mode $\{X_i\}$ computed as:

$$p_{i,j} = \{x_i\}[M]\{r_j\} \tag{26}$$

and $\{r_j\}$ is the influence coefficient vector expressing the n nodal displacements resulting from unit values of base displacements in direction j.

6.1 Effects of Foundation Soil Flexibility on Fundamental Vibration Modes of Foundation Soil-Bridge Systems

For free vibration response analysis of bridge-foundation soil systems, the interaction effect is essentially controlled by foundation soil stiffness which in turn is strongly dependent on

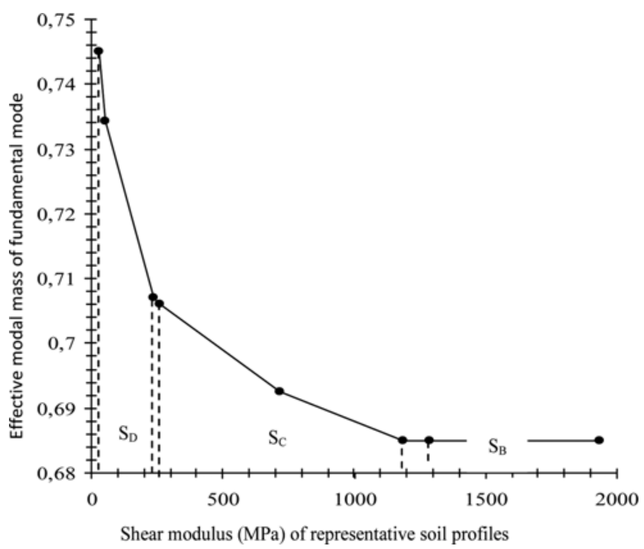

Fig. 5. Variation of Effective Modal Mass of Fundamental Mode (L1) as Function of Foundation Soil Flexibility

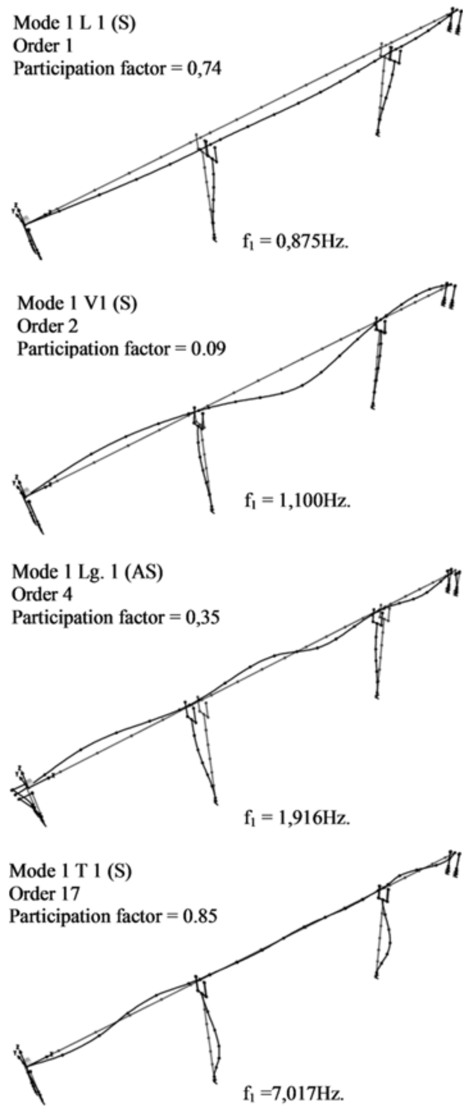

Fig. 6. Mode Shapes of the Bridge and Corresponding Participation Factors for Stiff Soil Conditions

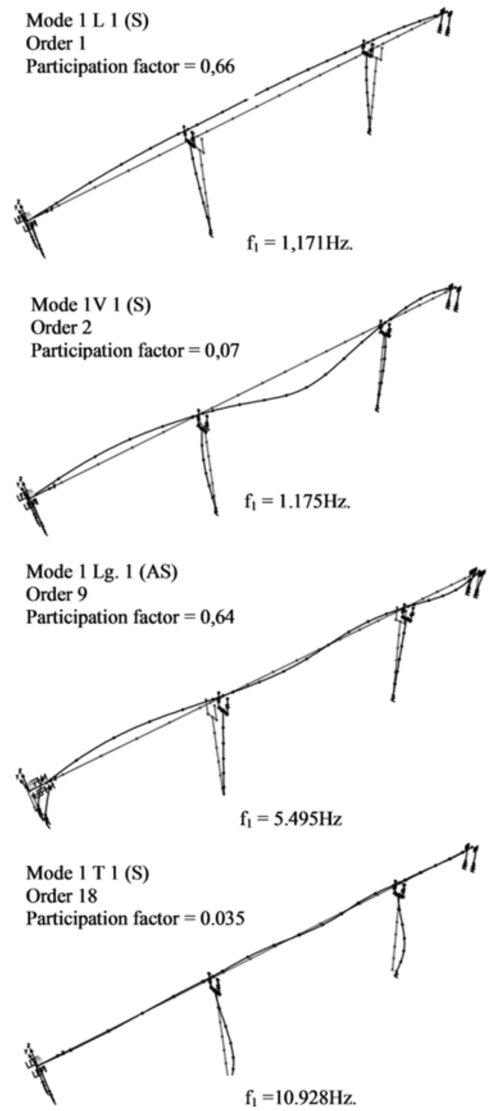

Fig. 7. Mode Shapes of the Bridge and Corresponding Participation Factors for Rock Sites Conditions

the type of soil profile considered.

Figure 5 shows the variation of the effective modal mass corresponding to the part of the total mass responding in the fundamental mode of vibration L1 of the bridge-foundation system as a function of the foundation soil flexibility. In earthquake engineering, the concept of effective mass is often used to indicate the relative contribution of a given mode of vibration to the structural response of the system.

It is noted that the effective mass in the fundamental mode of vibration decreases with the increase of foundation stiffness. For a bridge with a flexible foundation, a relatively small number of low-frequency modes is thus able to represent adequately the dynamic response of the system. This will also represent a significant computational advantage if time history analysis is to be carried out from a reduced system of dynamic equilibrium equations expressed in generalized coordinates.

In addition, for illustration purposes, first 3-D modal characteristics of lateral, vertical, longitudinal and torsional vibrations of both symmetrical (S) and assymetrical (AS) higher modes of the bridge have been identified and compared for stiff soil and rock conditions. A 3-D graphical representation of the corresponding mode shapes are presented in Figs. 6 and 7 for stiff soil and rock conditions respectively.

Major differences in the dynamic characteristics and modal response parameters of the bridge-foundation systems are observed. Consequently, it is important from a design point of view that analytical models used in seismic evaluation of bridge structures explicitly consider SSI effects.

6.2 Effects of Foundation Soil Flexibility on Higher Vibration Modes of Foundation Soil-Bridge Systems

Figure 8, illustrates the variation of the first twentieth modal frequencies of the bridge-foundation system for stiff soil and rock condition as a function of modal order, along with the

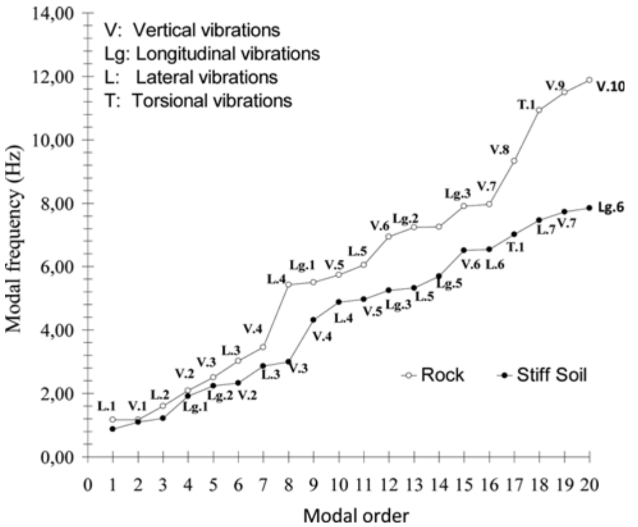

Fig. 8. Variation of the First Twenty Modal Frequencies of the Bridge-Foundation System for Stiff Soil and Rock Conditions

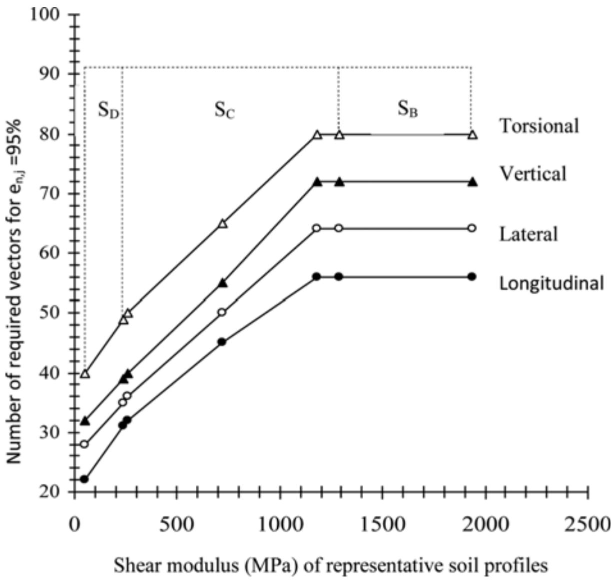

Fig. 9. Variation of the Number of Modes as Function of Foundation Soil Flexibility

corresponding dominant mode shapes denoted herein by L for lateral, Lg for longitudinal, V for vertical and T for torsional vibrations.

It is clearly seen that the decrease in modal frequencies due to SSI effects is more significant for the higher modes than the lower frequency modes of vibration. It is also observed that SSI effects may affect the nature of the dominant mode shape of vibration especially for the higher vibration modes (see e.g., changes of dominant shape Lg.1 to V.4; and T.1 to L.7 for the 9th and the 18th modes of vibration respectively). Similar observations can be made for the second modes (see V.2 to Lg.1; and Lg.2 to L5 for the 4th mode and the 13th modes of vibration respectively) and the third modes (L.3 to V.2; and V.3 to L.g.2; and Lg.3 to V.6 for the 6th mode and 5th mode and the 15th modes of vibration). This change in behavior of modal shapes is even more pronounced for the other higher modes.

The frequencies of the combined system as a function of the foundation flexibility are presented in Table 5 for the first three modes of vibration (i.e for the 1st and 3rd mode in the lateral direction and for the 2nd mode in vertical direction).

It can be noticed from this table that the modal frequencies of vibration increase (i.e., periods of vibration shorten) with increasing foundation stiffness regardless of vibration mode considered. An approximately 25% decrease in the frequency value of the fundamental lateral L1 mode is observed for the case of low stiff soil profile S_D, as compared to the case of high rock site condition S_B or equivalently to the rigid foundation assumption (i.e., when SSI effects are not included).

It is also observed that the effect of coupling terms of the foundation soil matrix on the fundamental frequency characteristics can be ignored. The same conclusion holds for the higher modes.

Figure 9, illustrates the variation of the number of modes required to reach an effective modal mass of 95% in the longitudinal, lateral, vertical directions corresponding to the X, Y and Z directions respectively, as a function of foundation soil flexibility for the six representative soil profiles. Also shown in the Fig. 9, is the variation of number of required modal shape

Table 5. Natural Frequencies (Hz) for Six Representative Soil Types

Type of soil		SD (Stiff soil)		SC (Dense soil-Soft Rock)		SB (Rock)	
		Shear modulus G, (MPa)					
		53	235	261	1185	1290	5158
1st mode (L.1)	With soil matrix coupling terms	0.877	1.052	1.056	1.135	1.139	1.171
	Without soil matrix coupling terms	0.875	1.051	1.055	1.135	1.138	1.171
	Error in %	0.23	0.10	0.09	0.00	0.09	0.00
2nd Mode (V.1)	With soil matrix coupling terms	1.116	1.152	1.153	1.170	1.171	1.175
	Without soil matrix coupling terms	1.100	1.151	1.152	1.170	1.171	1.175
	Error in %	1.45	0.09	0.09	0.00	0.00	0.00
3st mode (L.2)	With soil matrix coupling terms	1.280	1.431	1.436	1.545	1.549	1.597
	Without soil matrix coupling terms	1.219	1.429	1.434	1.543	1.549	1.596
	Error in %	0.23	0.14	0.14	0.13	0.00	0.06

Table 6. Peak Deck Displacement (m) for Seismic Load Cases

Type of soil		S_D (Stiff soil)		S_C (Dense soil-Soft Rock)		S_B (Rock)	
		Shear modulus G, (MPa)					
		Low	High	Low	High	Low	High
Load case		53	235	261	1185	1290	5158
I	U	0.0138	0.0041	0.0040	0.0031	0.0030	0.0027
	V	0.0170	0.0157	0.0156	0.0149	0.0149	0.0147
	W	0.0073	0.0068	0.0068	0.0067	0.0067	0.0067
II	U	0.0041	0.0012	0.0011	0.0009	0.0009	0.0008
	V	0.0567	0.0522	0.0521	0.0498	0.0497	0.0490
	W	0.0073	0.0068	0.0068	0.0067	0.0067	0.0067
III	U	0.0041	0.0012	0.0011	0.0009	0.0009	0.0008
	V	0.0170	0.0157	0.0156	0.0149	0.0149	0.0147
	W	0.0242	0.0228	0.0228	0.0223	0.0222	0.0223

vectors to reach 95% of the effective modal mass associated with the torsional modes of vibration (around the X axis).

The results indicate that, the number of modes required to reach the specified value of the effective modal mass increases almost linearly with the increase of the foundation soil stiffness from low stiff soil profile type (S_D) up to dense soil-soft rock profile type S_C.

For rock profile types (S_B and S_A) the number of required modes to reach 95% of the effective modal mass is constant for all motions and equal to that of corresponding to the case of rigid foundation soil (i.e., when SSI effects are ignored). For a bridge with a very stiff foundation, a relatively large number of high frequency modes is thus necessary for an adequate representation of the dynamic response of the system. It also should be observed that for any specified value of foundation soil stiffness (i.e., a given soil-profile), the number of modes required to reach an effective modal mass of 95% is significantly less in the longitudinal direction than for the other directions, especially in the vertical and the torsional vibrations. This is due to the fact that the bridge foundation system is stiff in the latter directions and rather flexible in longitudinal direction.

6.3 Multimodal Seismic Response Spectrum Analysis of Coupled Bridge-Foundation System

In order to further assess the effects of foundation soil stiffness on the overall seismic response of the bridge, the six previous types of soil profiles are used again. As shown in Table 6, the soil profiles range from low stiff soil to high rock site conditions.

The RPOA 2008 design spectrum at 5% damping, scaled to a peak acceleration of 0.165 g was used for the earthquake loading. Four types of response spectra have been recommended in RPOA 2008 guidelines for soil types S_D (High soft soil) through S_B (high stiff soil/low rock condition). The corresponding values of effective shear modulus and effective shear wave velocity have been evaluated and reported in Table 2 for the six profiles. To account for 3-D multidirectional shaking, calculations are conducted for earthquake loading in three orthogonal directions (longitudinal, transversal and vertical). The three load cases for the seismic analyses are: (a) Load case I: 1.0 Longitudinal + 0.3 Transverse + 0.3 Vertical loadings; (b) Load case II: 0.3Longitudinal + 1.0 Transverse + 0.3 Vertical loadings and (c) Load case III: 0.3 longitudinal + 0.3 Transverse + 1.0 Vertical loadings.

Table 6 shows the effect of varying foundation soil stiffness on peak deck displacements at mid-central span, where U, V and W denote the peak deck displacements along the longitudinal x, lateral y and vertical z axes respectively.

The results clearly indicate that the maximum system response is exhibited for low stiff soil profile S_D in the lateral direction (as expected) for all load cases.

From Table 6, it is also seen that SSI effects are more pronounced in the bridge longitudinal direction. As a matter of fact, it can be observed that in case of bridge with rigid base, the maximum longitudinal displacement at mid-central span is 0.0027 m, while in case of bridge on high stiff foundation soil (actual condition), it is 0.0041 m. This indicates that there is about 52% rise in the magnitude of the longitudinal displacement in case of flexible foundation soil.

Furthermore, in order to check the results obtained by using the 3-D multimodal seismic response spectrum method, Newmark direct time integration (with parameters, $\alpha = 0.5$ and $\beta = 0.25$) has been applied to the N×N system of dynamic equilibrium equations of the coupled bridge-foundation soil system under actual site condition (high stiff soil) for load case II. The results are presented in Table 7.

It is clearly seen that for the case at hand, practically the same results are obtained when using the multimodal response spectrum method with Complete Quadratic Combinaison (CQC) of the

Table 7. Peak Deck Displacement (m) using CQC and Direct Time Integration Methods

Type of soil		S_D (Stiff soil)	
Load case	Displacement (m)	CQC method	Direct time integration
II	U	0,0017	0.0012
	V	0,0449	0.0522
	W	0,0062	0.0068

modal responses and Newmark direct time integration methods (Tiliouine and Ouanani, 2011). This also shows that for the moderate PGA used in the modal analysis, the effects of soil nonlinearity on the soil damping matrix can be neglected for all practical purposes.

7. Nonlinear Time History Analysis of Seismic Response of Foundation Soil-Bridge System

In the second phase of this work, the previous study is now extended to estimate the dynamic response characteristics and to predict the earthquake response of the study highway bridge. Two approaches to model soil foundation interaction are considered. These are (i) the PGA consistent linear soil model and (ii) an advanced plasticity soil model based on the Bouc-Wen hysteretic model. Results from both linear and nonlinear analysis are compared.

7.1 Foundation Constitutive Model Soil for Foundation Domain

The plasticity model is based on the hysteretic behavior proposed originally by Bouc (1971) and subsequently extended by Wen (1976) and (Baber and Wen, 1981). Relevant applications to the present work include among others (Pires, 1996) and (Gerolymos and Gazetas (2005; 2006)).

Fig. 10, below, illustration typical parameters of load (f) – displacement (d) characteristics used for Wen plasticity property.

The nonlinear force-deformation relationship is given by:

$$f = rkd + (1-r)f_y z \tag{27}$$

where, k is the elastic spring constant, f_y the yield force, r the specified ratio of post-elastic stiffness to elastic stiffness (k) and z is an internal hysteretic variable. This variable has a range of $|z| < 1$, with the yield surface represented by $|z| = 1$.

The initial value of z is zero and it evolves according to the differential equation:

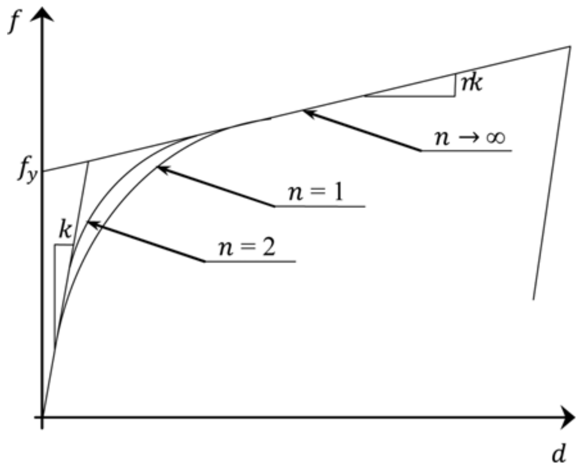

Fig. 10. Definition of Parameters for the Wen Plasticity Property

$$\dot{z} = \begin{cases} \frac{k}{y}\dot{d}(1-|z|^n) & \text{if} \quad \dot{d}z \geq 0 \\ \frac{k}{y}\dot{d} & otherwise \ \geq 0 \end{cases} \tag{28}$$

where, n is an exponent greater than or equal to unity. Large values of this exponenent increase the sharpness of yielding as shown in Fig. 10. The pratical limit for (n) is about 20. Nonlinear step by step time integration method is used to perform the dynamic analysis of the coupled foundation soil-bridge model corresponding to actual soil conditions at the site of construction.

7.2 Numerical Results and Discussion

The method of nonlinear time history analysis used in code SAP2000(2012) is an extension of the Fast Nonlinear Analysis (FNA) method developed by Wilson (Ibrahimbegovic and Wilson, 1990) which uses link elements to simulate nonlinearity. For further details on the analysis procedure, the reader can refer to the SAP2000(2012) user manual. The geometrical and material properties of the highway bridge with spread foundations have been described in Section 2. The material properties of foundation soil corresponding to the actual soil condition (types S_D: high stiff soil profile) are: the weight density γ = 21 KN/m^3; Poisson's ratio ν = 0.40 and initial shear wave velocity V_s = 366 m/s. In order to clarify the effect of material nonlinearity of foundation soil, the 3-D finite element model of the bridge foundation system (see Fig. 4) is now subjected to a simulated earthquake accelerogram compatible with RPOA(2008) spectrum scaled by factor of 2 (i.e., PGA = 0.33 g) as shown in Fig. 11. More details on simulation techniques for the generation of spectrum compatible earthquake motions can be found in reference (Tiliouine *et al.*, 2000). The PGA consistent reduction factor in accordance with FEMA guidelines is found to be equal to G/G$_o$ = 0.59.

The associated values of foundation stiffness and viscous damping coefficients to be used for Wen link elements have been evaluated and are reported in Table 8.

The time history response of pier base shear and deck displacement at mid-central span of study bridge under simulated earthquake ground motions corresponding to load case II and

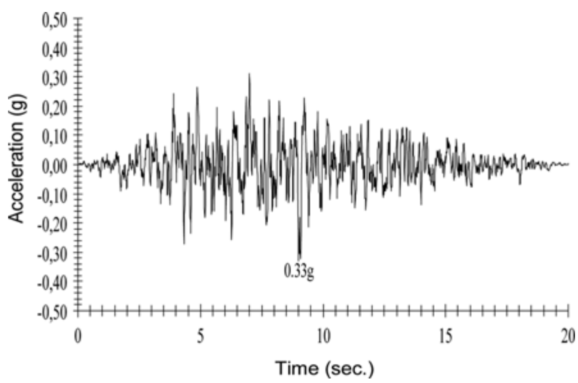

Fig. 11. Simulated Earthquake Accelerogram Compatible with RPOA Spectrum

Table 8. Effective Stiffness and Effective Damping Coefficients of Foundation Soil Matrix

S_D (High stiff soil): $G/G_o = 0.59$, $V_s'/V_s = 0.78$			
K_x (MN/m)	7856	C_x (MN.s/m)	54
K_y (MN/m)	7668	C_y (MN.s/m)	54
K_z (MN/m)	8228	C_z (MN.s/m)	93
K_{θ_x} (MN.m/rd)	226764	C_{θ_x} (MN.m.s/rd)	596
K_{θ_y} (MN.m/rd)	123435	C_{θ_y} (MN.m.s/rd)	265
K_{θ_z} (MN.m/rd)	220800	C_{θ_z} (MN.m.s/rd)	517
$K_{x\theta_y}$ (MN.m/rd)	1296	$C_{x\theta_y}$ (MN.m.s/rd)	20
$K_{y\theta_x}$ (MN.m/rd)	1944	$C_{y\theta_x}$ (MN.m.s/rd)	36

Table 9. Comparison of CPU Time between FNA and Direct Time Integration Methods

Analysis method	Time (sec.)	Peak pier base shear (KN)	Peak deck displacement (m)	Peak isolator shear strain (%)
Fast Nonlinear	11	3739.63	0.100	13.25
Direct time Integration	725	3750.68	0.114	14.04

isolator displacement at abutments associated to load case I (i.e., in the most critical directions) for the study bridge have been determined using Newmark direct time integration method with parameters $\alpha = 0.5$ and $\beta = 0.25$ (i.e., using the unconditionally stable average acceleration method) and the FNA algorithm. It is seen from Table 9 that for practically the same degree accuracy, the FNA algorithm is much more efficient than direct time integration method.

Time history results are reported in Fig. 12

From Fig. 12(a) it is seen that, there is a substantial 41% reduction in the pier base shear due to the soil flexibility effect. It is also observed from Figs. 12(b) and 12(c) that soil flexibility influences significantly, especially during the input strong ground motion phase, the values of deck displacement at mid-central span and bearings shear strain at abutments. The maximum deck displacement at mid-central span and peak bearing shear strain at abutment locations are found to be 0.064 m and 5.60% respectively for bridge with rigid foundations, while they are respectively equal to 0.10 m and 13.25% for flexible foundation soil which corresponds to significant increases of more than 55% and 130% respectively when the effects soil flexibility are considered. Thus, flexibility of the surrounding soil tends to reduce the earthquake forces induced in the bridge and substantially increase bridge deck displacement and shear strain at abutments. Maximum bearing displacement is a quantity of prime interest in the design of bridge structures because if it exceeds certain limits, the bearings may fail resulting into the bridge collapse.

In order to further assess the effects of soil nonlinearity, time variation of base shear acting at base of footing, displacement at mid-central span of the deck and shear strain bearings at

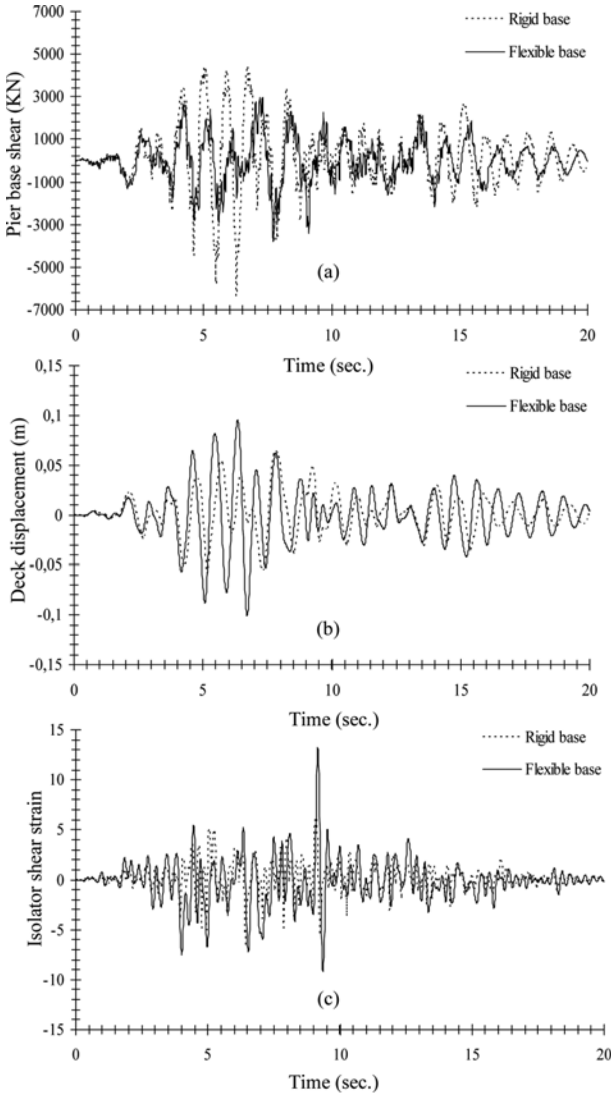

Fig. 12. Time Variation of: (a) Pier Base Shear, (b) Deck displacement, (c) Isolator Shear Strain for Study Bridge

abutments are investigated using both the PGA consistent linear lumped parameter soil model and the advanced plasticity soil model based on the Bouc-Wen hysteretic model. Time history bridge responses in terms of pier base shear, deck displacement at mid-central span and bearings shear strain at abutments are displayed in Figs. 13(a), 13(b) and 13(c), respectively.

From Fig. 13(a) it is seen that, there is a substantial 36% reduction in the pier base shear due to the soil nonlinearity effect. It is also observed from Figs. 13(b) and 13(c) that the nonlinearity of foundation soil influences significantly, especially during the input strong ground motion phase, the values of deck displacement at mid-central span and shear strain at abutment bearings. The maximum deck displacement and peak bearing shear strain at abutment locations are found to be 0.088m and 8.85% respectively for bridge with linear soil model, while they are equal to 0.10 m and 13.25% for nonlinear soil model which corresponds to increases of more than 12% and 45% respectively

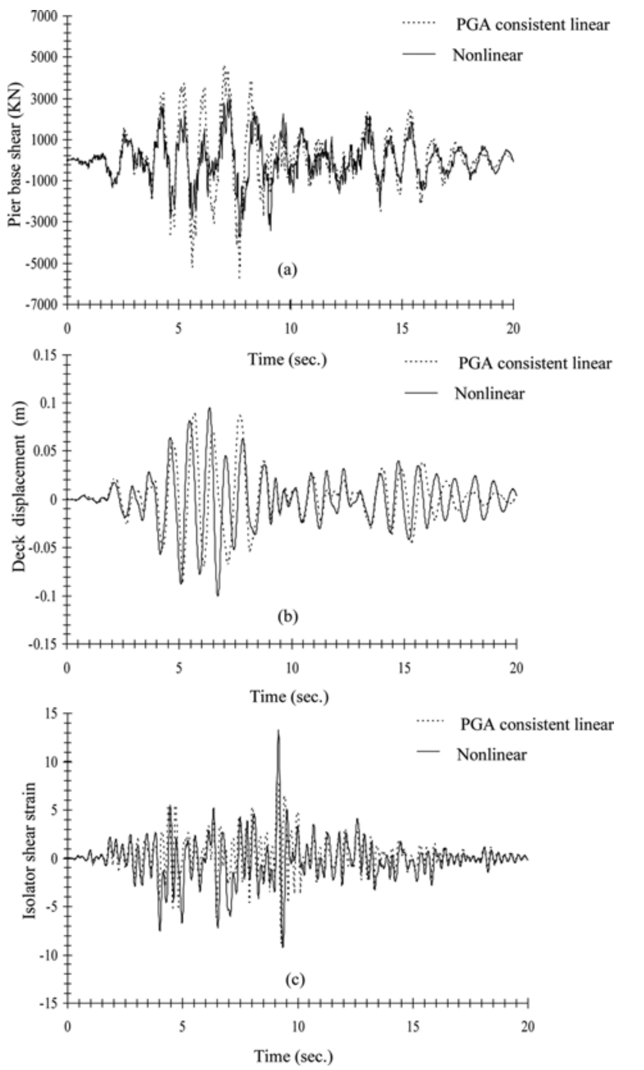

Fig. 13. Time Variation of (a) Pier Base Shear, (b) Deck displacement and (c) Isolator Shear Strain for Study Bridge

Table 10. Effects of Linear and Nonlinear Foundation Soil Models on Bridge Peak Response for Actual Site Conditions

Bridge with	Pier Base Shear (KN)	Deck displacement (m)	Isolator shear strain (%)
Rigid foundation	6347.48	0.064	5.60
Foundation on linear soil model	5820.45	0.088	8.85
Foundation on nonlinear soil model	3739.63	0.100	13.25

when the effects of soil nonlinearity are considered. Thus, similarly to foundation soil flexibility effects, the nonlinearity of the surrounding soil tends to reduce the earthquake forces induced in the bridge structure and increase bridge displacement and shear strain at abutments.

In addition, the effects of linear and nonlinear foundation soil models on bridge peak responses for the actual soil conditions at the site construction are summarized in Table 10.

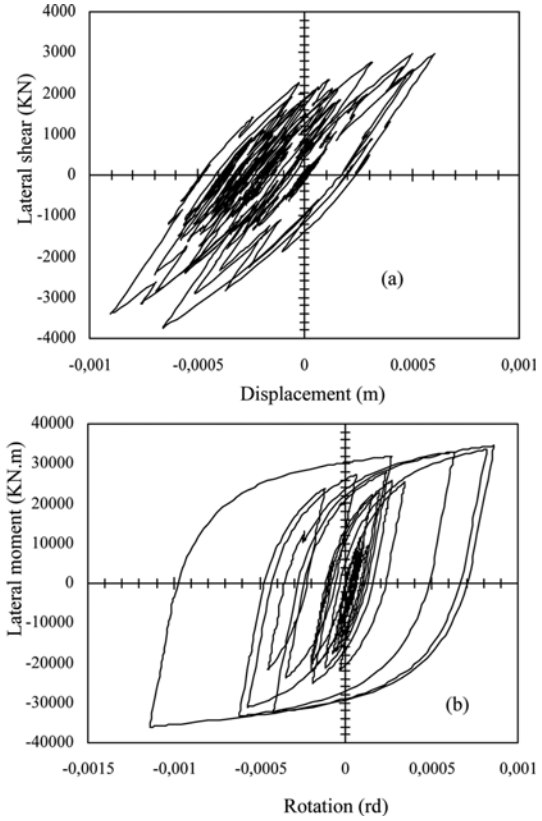

Fig. 14. Load Deformation Characteristics: (a) Lateral Shear, (b) Lateral Moment

Again, the results show clearly the importance of soil nonlinearity on bridge response under severe seismic ground motion.

Moreover and for the purpose of illustration, typical variations of load deformation characteristics of soil response in terms of lateral shear-displacement (in direction y-y) moment-rotation (about x-x axis) are depicted in Figs. 14(a) and 14(b) respectively.

It is seen that the Bouc-Wen hysteretic model is capable of representing adequately strong nonlinear hysteretic behaviour as well as strength and stiffness variation under earthquake ground motion, which makes it particularity well suited for the present application.

8. Conclusions

The significance of SSI effects on both 3-D modal characteristics of a highway bridge with spread foundations and the seismic response under moderate and severe ground motions using finite element method have been assessed.

Two different foundation soil models are investigated: A PGA consistent linear lumped-parameter soil model and an advanced plasticity soil model utilizing hysteretic Bouc-Wen link elements.

In the PGA consistent linear lumped parameter soil model, the soil surrounding the foundation of piers is modeled by frequency independent coupled soil spring model. In order to quantify the effects of SSI, various modal response parameters of the bridge-foundation system are compared to those of the bridge with rigid foundations. Emphasis has been placed on modal response behavior since frequency characteristics together with vibration mode shapes and effective modal mass are the most critical parameters governing the dynamic response of a soil-structure system. In addition, sensitivity studies have been conducted to investigate the effects of foundation soil stiffness on the overall modal and seismic behavior of the bridge. In addition, 3D multimodal response spectrum method is used for seismic analysis of coupled bridge-foundation system under moderate ground motions. The results are then checked by Newmark direct time integration method.

In the nonlinear hysteretic Bouc-Wen soil model, the SSI is simulated using translational and rotational Wen link elements acting at the centroid of spread foundations. In order to quantify the effects of SSI, nonlinear seismic response of critical seismic design parameters in terms of pier base shear, displacement at mid-central span and bearings shear strain at bridge abutments with nonlinear flexible foundation soil have been evaluated and compared to those of bridge with rigid foundations. In addition and in order to illustrate the effects of soil yielding, both linear and nonlinear of dynamic analyses have been performed and compared.

From the results obtained in this investigation, the following main conclusions can be drawn:

As regards to 3D bridge modal characteristics and overall seismic response under moderate ground motions:

– Major differences in the 3-D dynamic characteristics and modal response parameters of the bridge-foundation system are observed when SSI effects are considered. Consequently, it is important from a design point of view that analytical models used in seismic evaluation of bridge structures explicitly consider SSI effects.
– The SSI effects are found to be more pronounced for higher modes of vibration in comparison to the lower modes. SSI effects affect the bridge response not only through a systematic decrease of all modal frequencies but also via a substantial change in nature of dominant shapes especially for the higher modes of vibrations.
– The significance of soil flexibility on effective mass in the fundamental mode of vibration of bridge foundation system, on the number of higher modes required to reach 95% of effective modal mass in a given direction and on coupling terms of foundation soil matrix have been assessed.
– Under moderate PGA value (0.10 up to 0.16 g), effects of soil nonlinearity for soil class S_A, S_B, S_C and high S_D on soil damping matrix will not significantly influence the bridge seismic response and can be neglected for all pratical purpose.

As regards to 3D coupled response of bridge foundation system using PGA consistent linear and non linear soil models under severe ground motions:

– Numerical results show clearly the importance of foundation soil flexibility and soil nonlinearity effects on seismic response of bridge structures.
– The base shear at pier footing was found to have substantial decrease (particularly during strong motion phase) while deck displacement at mid-central span and especially bearings shear strain distortion at abutments were found to have significantly increased when the flexibility of foundation soil was considered.
– When material nonlinearity of foundation soil was considered, the bridge response showed increased amount of peak deck displacement and bearings shear strain distortion, but significant decrease of base shear (especially during the strong motions phase) as compared to the PGA consistent linear soil model.
– The Bouc-Wen hysteretic model is capable to represent soil nonlinearity and hysteretic behaviour as well as strength and stiffness degradation under seismic loading. It is numerical implementation for SSI problems relatively simple and requires, when using (FNA algorithm) little computation effort. It also provides realistic physical insight into the behavior of coupled foundation-soil bridge systems under severe seismic ground motions.

As a possible extension of the present work and for more versatile conclusions, different earthquake input excitations (including eventually spatially variable ground motions) and different bridge-foundation configurations, such as bridges with integral or full height abutment types founded on relatively soft sites could be investigated.

References

AASHTO LRFD (2011). *Bridge design specifications,* American Association of State Highway and Transportation Officials, Washington, D.C., USA.

Altunisik, A. C., Bayraktar, A., and Sevim, B. (2012). “Operational modal analysis of a scaled bridge model using EFDD and SSI methods.” *Indian Journal of Engineering and Materials Sciences,* Vol. 19, No. 5, pp. 320-330.

Anastasopoulos, I., Gelagoti, F., Kourkoulis, R., and Gazetas, G. (2011). “Simplified constitutive model for simulation of cyclic response of shallow foundations: Validation against laboratory tests.” *Journal of Geotechnical and Geoenvironmental Engineering*, Vol. 137, No. 12, pp. 1154-1168.

ATC-40 (1996). *Seismic evaluation and retrofit of concrete buildings*, Applied Technology Council, California, USA.

Baber, T. T. and Wen, Y. K. (1981). “Random vibration of hysteretic degrading systems.” *J. Eng. Mech.,* ASCE, Vol. 107, pp.1069-1087.

Bouc, R. (1971). “Modele mathematique d’hysteresis.” *Acustica*, Vol. 24, No. 1, pp. 16-25 (in French).

Chopra, A. K. (2011). *Dynamics of structures*: *Theory and applications to earthquake engineering*, Prentice-Hall, New Jersey, USA.

Clough, R. and Penzien, J. (1995). *Dynamics of structures*, Computers and Structures, Inc., Berkeley, USA.

Cook, T. L., Burdette, E. G., Graves, R. L., Goodpasture, D. W., and Deathrage, J. H. (1995). *Effect of varying foundation stiffness on seismically induced loads in bridge bents: A sensitivity study,* Washington, D.C., USA.

Crouse, C. B., Hushmand, B., and Martin, J. (1987). “Dynamic soil-structure interaction of a single-span bridge." *Earthquake Engng. Struct. Dyn.,* Vol. 15, No. 6, pp. 711-29.

CSI, csiBridge (2012). "Computer and Structures. Inc. SAP2000 (Nonlinear version 15)." *Nonlinear User's Manual Reference*, Berkeley, CA, USA.

EC8 (2005) (EN 1998-2 Eurocode 8) (2005). *Part 2: Bridges,* Design of Structures for Earthquake Resistance.

Elassaly, M., Ghali, A., and Elbadry, M. (1995). "Influence of soil conditions on the seismic behavior of two cable-stayed bridges." *Can. J. Civil Engng,* Vol. 22, No. 5, pp. 1021-1040.

Ewins, D. J. (2000). *Modal Testing: Theory, practice and application,* Research Studies Press Ltd., Baldock, Hertfordshire.

Faccioli, E., Paolucci, R., and Vanini, M. (1998). *3D site effects and soil-foundation interaction in earthquake and vibration risk evaluation,* Final Report of the European research project TRISEE, Politecnico di Milano, Italy.

FEMA 273 NEHRP (1997). *Guidelines for the seismic rehabilitation of buildings,* Federal Emergency Management Agency, Washington D.C, USA.

FEMA 356/ASCE (2000). *Seismic rehabilitation prestandard*, Federal Emergency Management Agency Washington, D.C., USA.

FEMA 440 (2005). *Improvement of nonlinear static seismic analysis procedures,* Federal Emergency Management Agency, D.C., USA.

Gajan, S., Phalen, J. D., Kutter, B. L., Hutchinson, T. C., and Martin, G. (2005). "Centrifuge modeling of load deformation behavior of rocking shallow foundations." *Soil Dynamics and Earthquake Engineering,* Vol. 25, No. 7, pp. 773-783.

Gazetas, G. (1991). *Foundation vibrations: Chapter 15, in foundation engineering handbook*, H-Y Fang H-Y, Van Nostrand Reinhold, New York.

Gerolymos, N. and Gazetas, G. (2005). "Constitutive model for 1D cyclic soil behaviour applied to seismic analysis of layered deposits." *Soils Found,* Vol. 45, No. 3, pp. 147-159.

Gerolymos, N. and Gazetas, G. (2006). "Development of Winkler model for static and dynamic response of caisson foundations with soil and interface nonlinearities." *Soil Dynamics and Earthquake Engineering* Vol. 26, No. 5, pp. 363-376.

Hardin, B. O. and Drnevich, V. P. (1972a). "Shear modulus and damping in soil: Measurement and parameter effects." *Journal of the Soil Mechanics and Foundation Engineering Division*, ASCE, Vol. 98, No. 6, pp. 603-624.

Hardin, B. O. and Drnevich, V. P. (1972b). "Shear modulus and damping in soils: Design equations and curves." *Journal of the Soil Mechanics and Foundation Division,* ASCE, Vol. 98, No. 7, pp. 667-692.

Ibrahimbegovic, A. and Wilson, E. L. (1990). "A methodology for dynamic analysis of linear structure-foundation systems with local nonlinearities." *Earthquake Engineering and Structural Dynamics*, Vol. 19, pp. 1197-1208.

Idriss, I. M. and Seed, H. B. (1968). "Seismic response of horizontal soil layers." *ASCE J. Soil Mechanics Foundation Division*, Vol. 94, pp. 1003-1031.

Kobayashi, T., Yoshikawa, K., Takaoka, E., Nakazawa, M. and Shikama, Y. (2002). "Time history nonlinear earthquake response analysis considering materials and geometrical nonlinearity." *Nuclear Engineering and Design*, Vol. 212, Nos. 1-3, pp. 145-154.

Levine, M. B. and Scott, R. F. (1989). "Dynamic response verification of simplified bridge foundation model." *International Journal Geotechnical Engineering,* Vol. 15, No. 2, pp. 1246-1261.

Lihua, Z., Leiqing, F., Kai, H., and Liyuan, W. (2012). "Vibration control of soil-structure systems and pile-soil-structure systems." *KSCE Journal of Civil Engineering*, KSCE, Vol. 16, No. 5, pp. 794-802.

Paolucci, R. (1997). "Simplified evaluation of earthquake induced permanent displacement of shallow foundations." *Journal of Earthquake Engineering,* Vol. 1, No. 3, pp. 563-579.

Paolucci, R., Shirato, M., and Yilmaz, M. T. (2008). "Seismic behavior of shallow foundations shaking table experiments vs numerical modeling." *Earthquake Engineering and Structural Dynamics,* Vol. 37, pp. 577-595.

Pecker, A. (2011). "Influence of nonlinear soil structure interaction on the seismic demand of bridges." *Proceedings of the International Conference on Innovations on Bridges and Soil Bridge Interaction*, Athens, Greece.

Pires, J. A. (1996). "Stochastic seismic response analysis of soft soil sites." *Nuclear Engineering and Design*, Vol. 160, No. 3, pp. 363-377.

Ribeiro, D., Calcada, R., Delgado, R., Brehm, M., and Zabel, V. (2012). "Finite element model updating of a bowstring-arch railway bridge based on experimental modal parameters." *Engineering Structures,* Vol. 40, No. 7, pp. 413-435.

RPOA (2008). *Partie I: Ponts*, Règlement Parasismiques Applicables au Domaine des Ouvrages d'Arts, Algeria.

Shehata, E. A. and Toshiro, H. (2013). "Soil-structure interaction modeling effects on seismic response of cable-stayed bridge tower." *International Journal of Advanced Structural Engineering*, Vol. 5, No. 8, pp. 1-17.

Spyrakos, C. (1990). "Assessment of SSI on the longitudinal seismic response of short span bridges." *Engng. Struct.,* Vol. 12, No. 1, pp. 61-66.

Spyrakos, C. and Loannidis, G. (2003). "Seismic behavior of a post-tensioned integral bridge including Soil-Structure Interaction (SSI)." *Soil Dynamics and Earthquake Engineering,* Vol. 23, No. 1, pp. 53-63.

Tiliouine, B. and Moussaoui, S. E. (1996). "Vibration des grands systèmes: Application à l'analyse de l'interaction dynamique sol-barrage." *Actes du 4éme Colloque National de L'Association Française de Génie Parasismique*, Paris, France.

Tiliouine, B. and Ouanani, M. (2011). "Réponse sismique 3-D d'un Pont-Caisson à inertie variable." *Proceedings of the 8th French National Conference on Earthquake Engineering: Dynamic and Vibratory Aspects in Civil Engineering,* AFPS Paris, France.

Tiliouine, B. and Ouanani, M. (2012). "3-D Nonlinear earthquake response of R.C. box girder bridges with expansion joints and bearing devices." *15th World Conference on Earthquake Engineering*, Lisbon, Portugal.

Tiliouine, B., Hammoutene, M., and Bard, P. Y. (2000). "Phase angle properties of earthquake strong motions: A critical look." *12th World Conference on Earthquake Engineering*, Auckland, New Zealand.

Tongaonkar, N. P. and Jangid, R. S. (2003). "Seismic response of isolated bridges with soil-structure interaction." *Soil Dynamics and Earthquake Engineering*, Vol. 23, No. 4, pp. 287-302.

Trifunac, M. D. and Todorovska, M. I. (1996). "Nonlinear soil response - 1994 Northridge, California, earthquake." *Journal of Geotechnical Engineering,* Vol. 122, No. 9, pp. 725-735.

Tsang, N., England, G. L., and Bush, D. I. (2002). *Soil structure interaction of integral bridge with full height abutments,* 15th ASCE Engineering Mechanics Conference, New York.

Vetetsos, A. S. and Wei, Y. T. (1971). "Lateral and rocking vibration of footings." *Journal of the Soil Mechanics and Foundation Division*, Vol. 97, No. 9, pp. 1227-1248.

Wen, Y. K. (1976). "Method of random vibration for hysteretic systems." *Journal of Engineering Mechanics Division,* ASCE, Vol. 102, No. 2, pp. 249-263.

Wilson, E. L. (2002). *Three-dimensional static and dynamic analysis of structures: A physical approach with emphasis on earthquake engineering,* Computer and Structure, Inc. Berkeley, California, USA.

Wolf, J. P. (1988). *Soil-structure interaction analysis in time-domain,* N.J. Prentice-Hall, Englewood Cliffs.

Wolf, J. P. (1997). "Springs-Dashpots-Mass models for foundation vibration." *Earthquake Engineering and structural dynamics*, Vol. 26, No. 9, pp. 931-949.

Yohchia, C. (1996). "Modeling and analysis methods of bridges and their effects on seismic responses: I theory." *Computers and Structures,* Vol. 59, No. 1, pp. 81-89.

Zienkiewicz, O. C. and Taylor, R. L. (2005). *The finite element method, solid mechanics,* Upper Saddle River, New Jersey, USA.

Zwolski, J. and Bien, J. (2011). "Modal analysis of bridge structures by means of forced vibration tests." *Journal of Civil Engineering and Management,* Vol. 17, No. 4, pp. 590-599.