GEOTECHNICAL ISOLATION OF PILE-SUPPORTED BRIDGE PIERS USING EPS GEOFOAM

X. Karatzia(1) and G. Mylonakis(2)

(1) *Ph.D Candidate, Department of Civil Engineering, Univ. of Patras, Rio 26500, Greece, xkar@upatras.gr*
(2) *Professor, Dept. of Civil Engineering, Univ. of Bristol, Queens Building, Bristol BS8 1TR, U.K.; Professor, Dept. of Civil Engineering, Univ. of Patras, Rio 26500, Greece; Adjunct Professor, Univ. of California at Los Angeles, CA 90095, g.mylonakis@bristol.ac.uk*

Abstract

A novel method for reducing the seismic forces imposed on pile-supported bridge piers, is presented. To this end, a flexible bridge pier founded on a single pile is enhanced by adding an annular zone of soft viscoelastic material, such as EPS geofoam, over the upper portion of the pile. It is shown that the annular zone operates as an isolation mechanism, increasing the fundamental natural period of the system and altering its damping. The problem is treated analytically and a simple, design-oriented analysis procedure is developed. Analytical closed-form solutions are derived for: (a) the overall compliance of the pier-pile-geofoam-soil system by means of a dynamic Winkler model of pile-soil interaction, (b) the fundamental natural period, and (c) the overall damping of the system. Based on a response spectrum method, the inertial forces acting at the pile head are determined. The increase in displacements due to the presence of the geofoam is acknowledged, but not investigated here. The influence of problem parameters such as stiffness, thickness and depth of inclusion are examined.

Keywords: geotechnical isolation; pile foundation; bridge pier; geofoam; period elongation

1. Introduction

The seismic response of pile-supported bridge piers encompassing soil-structure interaction has been thoroughly studied by researchers [1 – 7]. Findings of the above research efforts have been implemented in design methods or code-type provisions [8, 9]. Moreover, seismic isolation of bridges is a critical subject in earthquake engineering and there is a sustained interest in developing new methods with emphasis on reliability and reduced cost. An efficient isolation system protects a bridge structure by reducing the seismically-induced forces and/or increasing the damping of the structure. The commonly used elastomeric bearings placed between the bridge deck and the supporting pier, shift the fundamental period of the bridge structure to longer periods to reduce acceleration, while dampers provide energy absorption, thus limiting relative displacements.

Although such isolation devices have been proven to perform well, there exist cases during strong earthquake events where they have failed [10, 11]; the most frequent failure modes being span unseating due to large induced displacements, ruptured bearings and lift-off failure of the bearings. To minimize the likelihood of these failures, dampers, cable restrainers, span unseating devices, shear keys and other advanced structural techniques are employed. However, the efficiency of such systems remains questionable. On that basis, a number of alternative earthquake resisting systems (ERS) – facilitated by new innovative bridge construction methods – have been developed. An instructive literature review of hybrid seismic isolation methods is given in [12].

In recent years, the concept of “Geotechnical Seismic Isolation” [13, 14] has gained momentum among researchers as an alternative to conventional Structural Seismic Isolation applications. Researchers [15, 16] have suggested soil improvement around the foundation of building structures by means of smooth synthetic liners and rubber-soil mixtures (RSM) for dissipating seismic energy. Xiong et al. (2014) [17] investigated experimentally, via shaking-table tests, the dynamic performance of GSI during earthquakes. Notwithstanding advances in characterizing the mechanical and dynamic properties of RSM [18 – 20], still remains the fact that properties of rubber-soil mixtures cannot be completely predictable and controllable, particularly upon installation around a foundation.

This study presents a novel geotechnical isolation system for pile-supported bridge piers using elastic inclusions such as geofoam materials around piles. Geofoam [21] and especially expanded polystyrene (EPS) is a promising material for such purposes due to well-known mechanical behavior, energy dissipation capability, ease of installation and replacement, and low cost. The potential of the specific material has been demonstrated by a numerous geotechnical applications such as lightweight fill in road embankments, reduction of lateral pressures on retaining walls, slope stabilization, foundations fill and ground vibration isolation with in-filled geofoam barriers.

EPS foam is a good candidate material for the proposed geotechnical isolation of bridge piers because it possesses sufficient compressive and shear strength to undertake the lateral soil pressures, while at the same time renders the foundation flexible, since the foam-soil stiffness ratio is small. Additionally, with reference to such applications, the mechanical and dynamic properties of EPS have been extensively investigated in recent times [22, 23].

It is shown that the inclusion of geofoam around a pile provides seismic isolation in terms of an increase in the fundamental natural period and, hence, in a reduction in seismic forces. The stiffness, fundamental period and effective damping of the pier-pile-foam-soil system are obtained analytically, based on pertinent dynamic Winkler considerations. Using a response spectrum approach, a design-oriented analysis procedure is developed.

2. Problem description

The problem at hand refers to a cantilever structure, which can be considered an idealization of an actual bridge segment. Fig. 1a illustrates a single-column bent of height H supported on a flexible pile, provided with a compressible annular EPS coat up to a depth D_e, embedded in a soil layer of thickness H_s. The system is excited by vertically propagating *S*-waves. The pile is considered a linearly viscoelastic solid cylindrical beam of diameter d, Young's modulus E_p and linear hysteretic damping β_p. The upper part of the pile is surrounded by a soft inclusion of thickness t, Young's modulus E_{inc} and Poisson's ratio v_{inc}. The single-column bent is essentially a single-degree of freedom oscillator of height H and same diameter as that of the pile, and mass m. The soil is modeled as a linearly viscoelastic medium of Young's modulus E_s, Poisson's ratio v_s, mass density ρ_s and linear hysteretic damping β_s. Detailed information on the soil, pile, inclusion and structural properties used in the analysis are provided in Table 1.

A parametric study is conducted to investigate the influence of salient problem parameters, such as the stiffness ratios E_{inc}/ E_s and E_p / E_s, the thickness ratio t/d, the slenderness ratio H/d and the embedment depth of foam D_e / L_a on the dynamic response of the soil-pile-bridge system.

Table 1 – Soil, pile, inclusion and structural properties employed

Soil	Pile	Inclusion (geofoam)	Superstructure
$E_s = 2.5\times10^4\ kPa$ $\rho_s = 2\ Mg / m^3$ $\beta_s = 10\ \%$ $v_s = 0.4$	$E_p = 2.5\times10^7\ kPa$ $d = 1\ m$ $L = 20\ m$ $\rho_p = 2.5\ Mg / m^3$ $\beta_p = 5\ \%$	$E_{inc} / E_s = 0.01 – 1$ $\beta_{inc} = 10\ \%$ $v_{inc} = 0.15$ $t / d = 0.25, 0.5, 1$ $D_e / L_a = 0.25, 0.5, 1$	$E_{str} = 2.5\times10^7\ kPa$ $I_{str} = 0.049\ m^4$ $d_{str} = 1\ m$ $\beta_{str} = 5\ \%$ $H / d = 5, 10, 20$

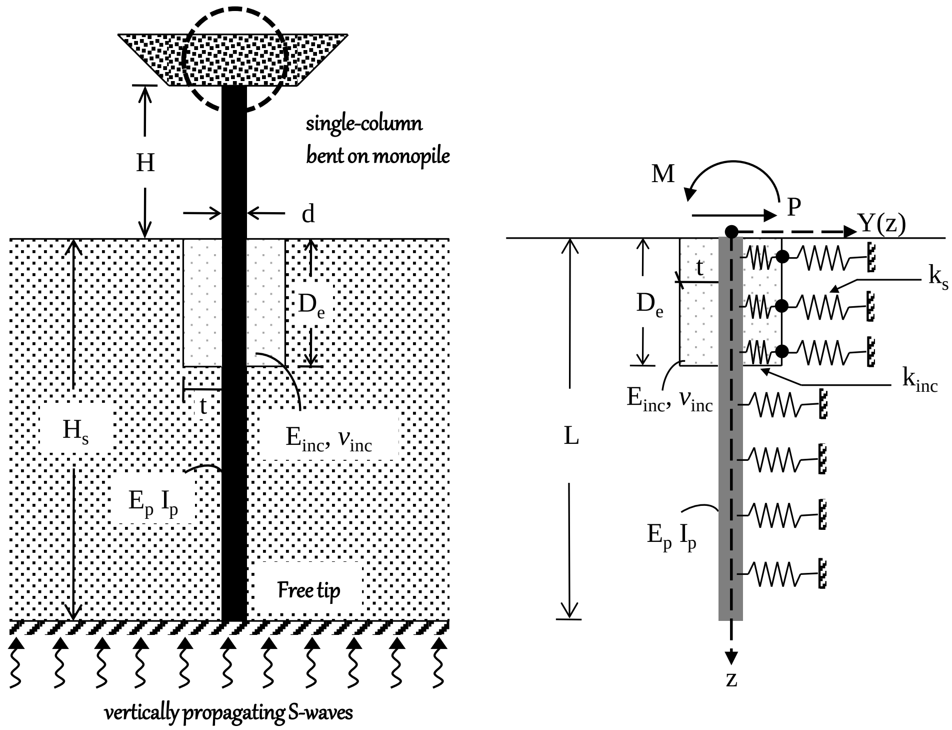

Fig. 1 – a) The bridge-pier system founded on a single pile provided with inclusion; b) The associated beam-on-Winkler-foundation model.

3. Proposed solution

3.1 Stiffness and damping coefficients of a pile enhanced with inclusion

Fig. 1b depicts the modeling approach of the pile – soft inclusion – soil system. The stiffness of the inclusion and the stiffness in the outer region of soil are denoted with k_{inc} and k_s, respectively. The behavior of a flexible cylindrical pile surrounded by a compressible annular zone of finite thickness has been investigated in the past and analytical solutions for the stiffness of pile – inclusion – soil system have been derived [23 – 25]. Therefore, the stiffness coefficient in case of a pile enhanced with an elastic or viscoelastic annular soft zone is given by the relationship

$$k_{inc}=\left(\frac{4\pi E_{inc}}{1+v_{inc}}\right)\frac{\left(1-v_{inc}\right)\left[\left(2t/d+1\right)^2+1\right]}{\left(1-2v_{inc}\right)^2\left[\left(2t/d+1\right)^2-1\right]+\left(3-4v_{inc}\right)\left[\left(2t/d+1\right)^2+1\right]\ln\left(2t/d+1\right)} \tag{1}$$

where E_{inc} and v_{inc} are the Young's modulus and the Poisson's ratio of the inclusion, respectively; t is the thickness of the inclusion and d is the pile diameter. The stiffness coefficients are based on the assumptions of perfectly smooth interfaces between pile and inclusion, and between inclusion and soil. k_s denotes the value of the soil spring constant, i.e. $k_s = \delta\, E_s$, with δ being a dimensionless coefficient typically varying between 1 and 2.5 depending on soil inhomogeneity, pile-soil stiffness contrast and boundary conditions at the pile head [5, 26]. A value of 2 was selected in the analyses.

In the realm of the Winkler model, the stiffness of the overall soil system up to depth D_e is determined by combining the compliance of the inclusion with that of the surrounding soil medium in the form of a pair of springs attached in a series (Fig. 1b). Accordingly, the stiffness of the overall soil system up to depth D_e is determined as

$$\hat{k}_s = k_{inc} \frac{1}{1+\left(k_{inc}/k_s\right)} \tag{2}$$

This treatment is approximate because the distortion of the inclusion-soil interface is not considered, thus leading to a somewhat stiffer system. However, because the soil material is typically more than an order of magnitude stiffer than that of the inclusion i.e $k_{inc}/k_s \leq 10^{-1}$, the overall stiffness practically coincides with that of inclusion [24, 25].

Given the material around the pile up to a depth D_e is different from the material at deeper elevations, one may assume an equivalent two-layer profile. Under this assumption, an upper soil layer of thickness D_e, stiffness $\hat{k}_s$ and Winkler parameter λ_1 followed by a thick second layer of stiffness k_s and Winkler parameter λ_2 is considered. The associated Winkler parameters are obtained from the familiar expressions

$$\lambda_1 = \left(\frac{\hat{k}_s}{4E_pI_p}\right)^{1/4} \quad , \quad \lambda_2 = \left(\frac{k_s}{4E_pI_p}\right)^{1/4} \tag{3}$$

The shape parameter μ is the average value of λ_1 and λ_2 over the active pile length L_a [1]

$$\mu = \frac{D_e}{L_a}\lambda_1 + \frac{1}{L_a}\left(1-\frac{D_e}{L_a}\right)\lambda_2 \tag{4}$$

Keeping in mind that for inhomogeneous soil, the shape parameter μ can be realistically approximated as $\mu = 2.5 / L_a$ [27], one may compute the active pile length L_a as a function of the inclusion length D_e through the formula

$$L_a/D_e = \frac{2.5\,D_e^{-1} + \left(\lambda_2 - \lambda_1\right)}{\lambda_2} \tag{5}$$

Using the virtual work method and employing a set of pertinent shape functions for pile deflection [1, 28, 29], the static stiffness coefficients K_{hh}, K_{rr} and K_{hr} corresponding to swaying, rocking and cross-swaying-rocking, respectively, at the pile head can be readily determined. For a two-layer soil profile these stiffness terms are

$$K_{hh} = E_pI_p\mu^3\left(1+s_{hh}\right), \quad K_{rr} = \frac{3}{2}E_pI_p\mu\left(1+s_{rr}\right), \quad K_{hr} = E_pI_p\mu^2\left(1+s_{hr}\right) \tag{6}$$

where

$$s_{hh} = \frac{16\delta}{\pi}\frac{\hat{k}_s}{k_s}\left(\frac{E_p}{E_s}\right)^{-1}\left(\frac{1}{\mu d}\right)^4\left\{3-\left(1-\frac{k_s}{\hat{k}_s}\right)e^{-2\mu D_e}\left[2+\cos\left(2\mu D_e\right)+\sin\left(2\mu D_e\right)\right]\right\} \tag{7}$$

$$s_{rr} = \frac{16\delta}{3\pi}\frac{\hat{k}_s}{k_s}\left(\frac{E_p}{E_s}\right)^{-1}\left(\frac{1}{\mu d}\right)^4\left\{1-\left(1-\frac{k_s}{\hat{k}_s}\right)e^{-2\mu D_e}\left[2-\cos\left(2\mu D_e\right)+\sin\left(2\mu D_e\right)\right]\right\} \tag{8}$$

$$s_{hr} = \frac{16\delta}{\pi}\frac{\hat{k}_s}{k_s}\left(\frac{E_p}{E_s}\right)^{-1}\left(\frac{1}{\mu d}\right)^4\left\{1-\left(1-\frac{k_s}{\hat{k}_s}\right)e^{-2\mu D_e}\left[1+\sin\left(2\mu D_e\right)\right]\right\} \tag{9}$$

In addition to lowering stiffness, the inclusion also modifies the effective damping of the system. The effective damping is controlled by the inclusion-to-soil stiffness contrast (k_{inc}/k_s) through the expression

$$\hat{\beta}_s = \left[\frac{1}{1+\left(k_{inc}/k_s\right)}\right]\beta_{inc} + \left[\frac{\left(k_{inc}/k_s\right)}{1+\left(k_{inc}/k_s\right)}\right]\left(\beta_s+\beta_r\right) \tag{10}$$

with β_{inc}, β_s and β_r being the inclusion damping, soil material damping and radiation damping, respectively; the terms in brackets can be interpreted as weight factors. The damping coefficients pertaining to a pile foundation are obtained according to the following mixing rules [9]

$$\beta_{hh} = \frac{3}{4}\hat{\beta}_s + \frac{1}{4}\beta_p, \quad \beta_{rr} = \frac{1}{4}\hat{\beta}_s + \frac{3}{4}\beta_p, \quad \beta_{hr} = \frac{1}{2}\hat{\beta}_s + \frac{1}{2}\beta_p \tag{11}$$

where β_{hh}, β_{rr} and β_{hr} denote the damping coefficients at the pile head corresponding to swaying, rocking and cross-swaying-rocking, respectively. Note that the effect of radiation damping coefficient is small in the presence of an EPS coat and is ignored in the present analyses.

3.2 Vibrational properties of the pier-pile-inclusion-soil system

The total displacement of the flexible bridge pier of Fig. 1, modeled as a single-degree-of-freedom oscillator, consists of two modular components: (1) a horizontal displacement and a rotation due to coupling between foundation swaying and rocking, (2) an additional displacement reflecting the compliance of the superstructure. This implies that the compliances of the foundation and the superstructure can be regarded as a pair of complex-valued springs assembled in parallel, under a common imposed load. Accordingly, the total stiffness $\tilde{K}$ of the system is given by the combination formula

$$\tilde{K} \cong \frac{K_f K}{K_f + K} \tag{12}$$

where K corresponds to the stiffness of a fixed-base bridge pier while K_f to the corresponding stiffness of a rigid bridge pier on a pile foundation, computed as [5]

$$K_f \cong \frac{K_{hh}\,K_{rr} - K_{hr}^2}{K_{rr} + \left(w+1\right)H\,K_{hr} + w\,H^2 K_{hh}} \tag{13}$$

K_{hh}, K_{rr} and K_{hr} referring to the swaying, rocking and cross-swaying-rocking impedances of the pile head and H being the pier height. The parameter w accounts for the effect of fixity conditions at the top of the column (w = 1, 0.5 for free and fixed condition, respectively). For a pile enhanced by an EPS coat, Eqs. (6) – (9) should be employed in Eq. (13) for determining the stiffness of the system.

Replacing the above stiffness terms with the complex impedances, $K_{hh}^* = K_{hh}\,(1 + 2i\beta_{hh})$, $K_{rr}^* = K_{rr}\,(1 + 2i\beta_{rr})$ and $K_{hr}^* = K_{hr}\,(1 + 2i\beta_{hr})$ and after some tedious algebra, the stiffness and damping coefficients of a rigid bridge pier on a flexible pile foundation may be obtained from the exact expressions [30, 31]

$$K_f = HK_{hr}\frac{K_{hh}K_{rr}\left[\chi_1+\chi_2+2\left(-\chi_3+4\chi_4\beta_{hr}\right)\right] - K_{hr}^2\left[\chi_5+8\chi_6\beta_{hr}+\chi_7\left(1-4\beta_{hr}^2\right)\right]}{\left(H^2K_{hh}+2HK_{hr}+K_{rr}\right)^2 + 4\left(H^2K_{hh}\beta_{hh}+2HK_{hr}\beta_{hr}+K_{rr}\beta_{rr}\right)^2} \tag{14}$$

$$\beta_f = \frac{\chi_1\beta_{rr}+\chi_2\beta_{hh}+2\left(\chi_3\beta_{hr}+\chi_4\right) - \dfrac{K_{hr}^2}{K_{hh}K_{rr}}\left[\chi_5\beta_{hr}+2\chi_7\beta_{hr}-\chi_6\left(1-4\beta_{hr}^2\right)\right]}{\chi_1+\chi_2+2\left(-\chi_3+4\chi_4\beta_{hr}\right) - \dfrac{K_{hr}^2}{K_{hh}K_{rr}}\left[\chi_5+8\chi_6\beta_{hr}+\chi_7\left(1-4\beta_{hr}^2\right)\right]} \tag{15}$$

χ_1, χ_2, χ_3, χ_4, χ_5, χ_6, χ_7 being dimensionless quantities given in the Appendix.

The natural period of the interacting system, considering both soil-structure interaction and the EPS coat, is given by

$$\frac{\hat{T}}{T} = \sqrt{1 + \frac{K}{K_f}} \tag{16}$$

It is also recalled that the fixed-base fundamental period of the pier is

$$T = 2\pi\sqrt{m/K} \tag{17}$$

m being the oscillator mass.

In the same vein, the effective damping of the system is determined as

$$\hat{\beta} = \frac{1}{1 + \frac{1+(2\beta)^2}{1+(2\beta_f)^2}\left(\frac{K}{K_f}\right)}\beta + \frac{1}{\frac{1+(2\beta_f)^2}{1+(2\beta)^2}\left(\frac{K_f}{K}\right)+1}\beta_f \tag{18}$$

with β being the damping ratio of the structure.

To assess the effect of the inclusion on the fundamental period of the flexible soil-pile-bridge system, the following closed-form equation is obtained

$$\frac{\hat{T}}{\tilde{T}} = \sqrt{\frac{1 + (K/K_f)}{1 + (K/\tilde{K}_f)}} \tag{19}$$

where $\tilde{K}_f$ is the stiffness of the flexible pier-pile-soil system considering only the effect of soil-structure interaction without the EPS coat. In this case Eqs. (12) and (13) still hold with the stiffness coefficients atop the pile head being $K_{hh} = 4E_pI_p\lambda^3$, $K_{rr} = 2E_pI_p\lambda$ and $K_{hr} = 2E_pI_p\lambda^2$, corresponding to homogeneous soil conditions, with λ being equal to λ_2.

Eqs. (16) and (19) demonstrate that the inclusion acts as an elementary base-isolation mechanism increasing the fundamental period of the structure. Considering SSI indicates that foundation is compliant and, therefore, the fundamental period of the system is longer than the fixed-base fundamental period. In the realm of a spectral analysis, the increase in period results to a change in spectral acceleration and, hence, a change in seismically-induced forces in the structure, as illustrated in Fig. 2. Generally speaking, the effect of SSI on the design forces is related to the slope of the response spectrum: a positive slope results in an increased base shear while a negative slope results in a reduced base shear [9]. The use of EPS coat around the pile makes the system more flexible and its fundamental period longer than the ordinary (with SSI but no EPS coat) period of the pier. From the elastic design spectrum of Fig. 2, it is evident that this period shift can lead to an increase or decrease in seismic demand depending on the circumstances.

A common problem in isolation methods lies in the increased displacements resulting from the increase in period, since the latter is shifted in or near the displacement-sensitive region of the spectrum. This means that structural displacements should be checked to be within acceptable limits. Such checks lie beyond the scope of this work.

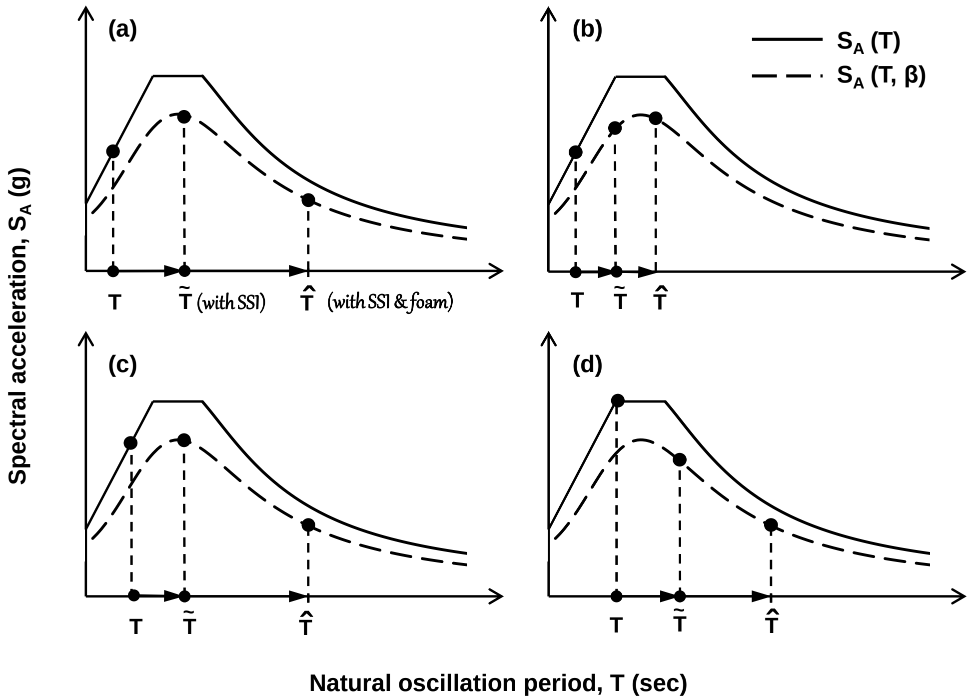

Fig. 2 – Period elongation of the system due to soil-structure-interaction and use of geofoam; T and β indicate the natural oscillation period and the effective damping of the system.

3.3 Determination of inertial forces based on the response spectrum

In the realm of the response spectrum method, peak earthquake response of a single-degree-of-freedom system can be obtained from the spectral acceleration $S_A(\hat{T},\hat{\beta})$ corresponding to the natural oscillation period $\hat{T}$ and a pertinent adjustment associated with the effective damping ratio $\hat{\beta}$. The correction for kinematic effects related to soil-pile interaction may be important, yet this lies beyond the scope of the present work.

From the above analysis, the base shear V_b in the pier and the corresponding overturning moment M_b are

$$V_b = m\, S_A\left(\hat{T},\hat{\beta}\right) \tag{20}$$

$$M_b = H\, V_b \tag{21}$$

where H denotes the pier height.

4. Parametric study for system period and damping

From the proposed solution, it is evident that the response of the system depends on the viscoelastic properties of soil, the EPS coat, and the properties of the foundation and the superstructure. The crucial parameters investigated here are the thickness ratio t / d, the inclusion-soil stiffness contrast E_{inc} / E_s, the pile-soil stiffness contrast E_p / E_s, the slenderness ratio H / d and the dimensionless inclusion length D_e / L_a.

The ratio of system period over the fixed-base period $\hat{T} / T$ (Eq. 16) and the damping ratio $\hat{\beta} / \beta$ (Eq. 18) are shown in Fig. 3, as functions of the inclusion-soil stiffness contrast E_{inc} / E_s. Evidently, the period elongation becomes more pronounced as the inclusion becomes softer (i.e. as E_{inc} / E_s ratio decreases). The variation in thickness ratio t / d has a moderate impact on the natural period, demonstrating that as the thickness of the soft zone around pile increases, the system tends to be more flexible. Accordingly, the length of the EPS coat D_e / L_a

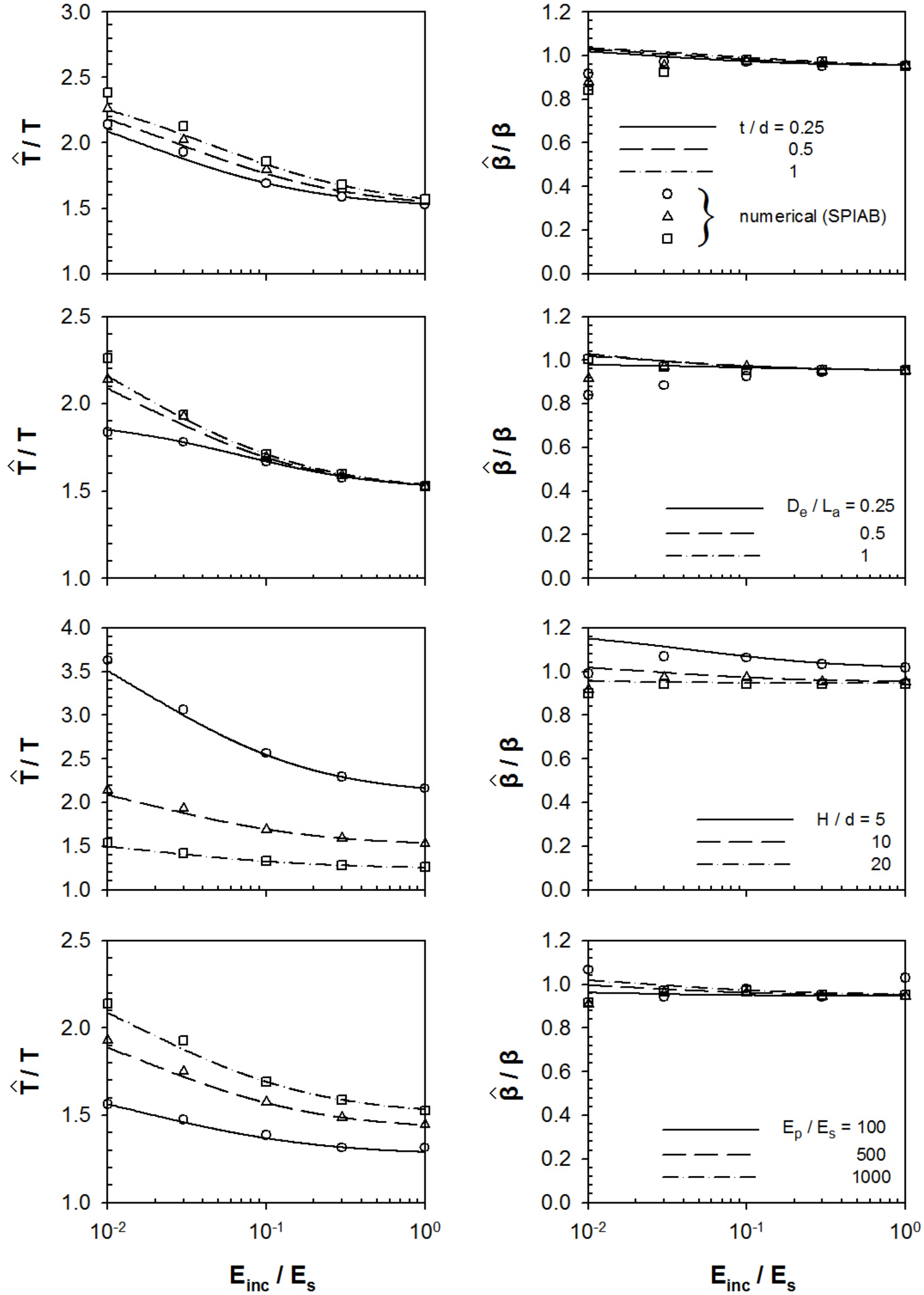

Fig. 3 – System period and damping with reference to the fixed-base system as a function of geofoam-soil stiffness contrast E_{inc} / E_s; $t / d = 0.25$, $D_e / L_a = 0.5$, $E_p / E_s = 10^3$, $H / d = 10$, $\beta_{inc} = 10\%$.

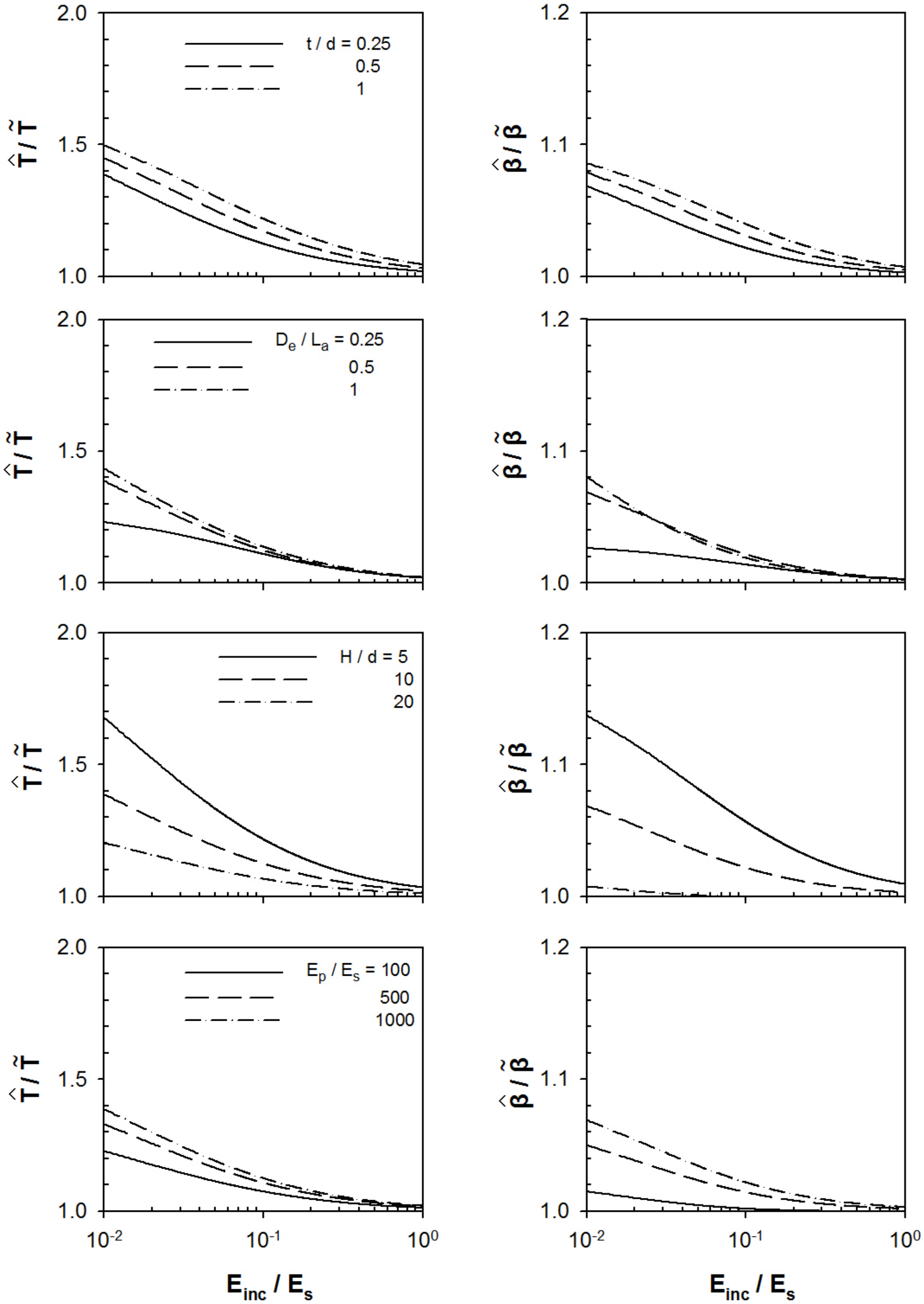

Fig. 4 – System period and damping with reference to the corresponding flexible-base system values as a function of geofoam-soil stiffness contrast E_{inc} / E_s; $t / d = 0.25$, $D_e / L_a = 0.5$, $E_p / E_s = 10^3$, $H / d = 10$, $\beta_{inc} = 10\%$.

seems to be of secondary importance. Remarkably, a short segment ($D_e / L_a = 0.25$) can affect substantially the period of the system and might provide a sufficient engineering solution.

Of particular interest are the results associated with the influence of slenderness ratio H / d. Naturally, the period of squat structures is significantly affected by the presence of the EPS coat, with the natural period increasing drastically. On the other hand, the period of tall slender structures is less sensitive to the addition of EPS. Also, the period of the system increases with increasing pile-soil stiffness contrast E_p / E_s, which can be interpreted in the same context. These observations conform to those of classical SSI theory [32].

Regarding the effective damping of the system, for a given set of soil and pile parameters the damping ratio of the EPS coat seems to be important. Based on experimental results [22], the damping ratio of geofoam may reach 10% at strains on the order of 10^{-1}, which indicates that the geofoam may provide sufficient energy dissipation for isolation purposes. Accordingly, a value of 10% is utilized for β_{inc}. From Fig. 3 it is shown that the effective damping of the system seems to be unaffected by the presence of geofoam. This is anticipated if one considers that the soil material damping ratio employed in the current analysis is 10%. For β_{inc} = 5% and the same soil properties, the effective damping of the system naturally decreases (not shown). Note that no radiation damping has been considered in the analyses at hand, yet its influence is not expected to be dominant.

In the same graph, results obtained by means of the computer code SPIAB [1] are presented, in which the problem is solved in an exact manner in the realm of Winkler theory. Evidently, comparison between the proposed analytical solution and the numerically evaluated results is meaningful.

It is instructive to present results for period and damping normalized with the corresponding properties of the SSI system without the EPS coat. Fig. 4 depicts the variation of $\hat{T} / \tilde{T}$ (Eq. 19) and $\hat{\beta}/\tilde{\beta}$ ratios with the EPS-soil stiffness contrast, E_{inc} / E_s. The influence of the group of parameters described before is evident, with the results exhibiting similar trends. The ordinates of curves for natural period and damping naturally attain values larger than 1.

5. Conclusions

The geotechnical isolation method presented in this study provides a simple inexpensive way of seismic isolation of bridge piers. The use of EPS foam coat around the pile provides a convenient extension to the classical SSI concept, by increasing the overall system period and altering the effective damping. A simplified procedure, based on the Winkler model of soil reaction, for predicting the overall period and damping of a soil – EPS coat – pile – pier system was presented as proof of concept. Despite the idealized nature of the model, the method lays the basis for a new design procedure, which may also be applicable to pile groups. The simplified analytical procedure is outlined below:

1. Select the embedment depth of inclusion D_e and the inclusion stiffness k_{inc}, through E_{inc}, v_{inc}, t.
2. Determine the pile active length L_a and the shape parameter μ for the equivalent soil profile, according to Eqs. (4) – (5), based on geotechnical properties.
3. Compute the dynamic pile stiffness coefficients – K_{ij} (Eqs. 6 – 9) and the corresponding damping coefficients – β_{ij} (Eqs. 11a-c).
4. Determine the fundamental period $\hat{T}$ and the effective damping $\hat{\beta}$ of the system incorporating the effects of SSI and the inclusion according to Eqs. (16) and (18).
5. Estimate the damping modifier according to formulas of the type $D(\hat{\beta}) = [10/(5+\hat{\beta})]^{0.5}$, or more recent proposals by [33].
6. Determine the peak acceleration $S_A(\hat{T})$ of the interacting system using a pertinent response spectrum, for a given value of damping ratio (e.g. 5%).
7. Modify peak acceleration $S_A(\hat{T})$ to conform to the level of effective damping of the system $S_A(\hat{T}) \times D(\hat{\beta})$.
8. The seismic force demand on the system can then be evaluated by means of Eqs. (20) and (21).
9. The corresponding displacement demand can be established via standard procedures of structural dynamics.

6. Appendix

In Eqs. (14) and (15), parameters $\chi_1, \chi_2, \chi_3, \chi_4, \chi_5, \chi_6, \chi_7$ are

$$\chi_1 = \frac{HK_{hh}}{K_{hr}}\left(1+4\beta_{hh}^2\right) \quad , \quad \chi_2 = \frac{K_{rr}}{HK_{hr}}\left(1+4\beta_{rr}^2\right) \quad , \quad \chi_3 = -1+4\beta_{hh}\beta_{rr} \quad , \quad \chi_4 = \beta_{hh}+\beta_{rr} \tag{22}$$

$$\chi_5 = 2\left(1+4\beta_{hr}^2\right) \quad , \quad \chi_6 = \frac{HK_{hh}}{K_{hr}}\beta_{hh} + \frac{K_{rr}}{HK_{hr}}\beta_{rr} \quad , \quad \chi_7 = \frac{HK_{hh}}{K_{hr}} + \frac{K_{rr}}{HK_{hr}} \tag{23}$$

7. Acknowledgements

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8. Copyrights

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