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INFLUENCE OF SOIL-STRUCTURE INTERACTION ON THE SEISMIC RESPONSE OF BRIDGE PIERS

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SUMMARY

The effect of soil-structure interaction on the non-linear seismic response of bridge piers is studied through an extensive parametric analysis. Piers and soil-foundation are modelled separately and assembled through the substructure method in the time domain. Shallow and stiff foundations embedded in a homogeneous soil are considered.

The soil is idealized as a linearly elastic, semi-infinite medium. It is modelled by Lumped Parameter Models (LPM). LPM's are a suitable combination of simple truss elements, viscous dashpots and concentrated masses, which can be analysed in the time domain together with a non-linear structure. The constant coefficients of the LPM's are calibrated on the results obtained through Cone Models (CM), which provide a simplified representation of the interaction between a massless rigid circular basemat and a halfspace in the frequency domain. They permit to calculate the coefficients of the stiffness matrix at the soil-structure interface as a function of the natural frequency.

The piers are modelled through linear beam elements, distributed masses and viscous mass-proportional dashpots. The material non-linearities are concentrated in a rigid-plastic hinge with hardening at the pier base. Several piers with different dynamic properties are considered in the study, to represent real cases.

The seismic analysis of the assembled model is carried out in the time domain, considering several artificial accelerograms generated according to EC8, which simulate the propagation of horizontal shear waves through the soil. Soil properties are varied to evaluate their influence on the seismic response of the piers.

The results show that the stiffer the superstructure the stronger the influence of soil-structure interaction. In particular, neglecting interaction can lead to significant underestimation of displacements and curvature ductility demand. Moreover, the behaviour of flexible and slender structures can be significantly affected by the rotational component at the base due to inertial interaction.

INTRODUCTION

The seismic response of a structure is usually evaluated assuming the free-field motion at its base. This hypothesis is in principle acceptable when the soil is very stiff and soil-structure interaction effects are negligible. Actually, the base motion can differ significantly from the free-field motion, not only for what concerns the traslational components, but also for the rotational component, which can turn out to be very important for slender structures, such as bridge piers. The presence of soil as a non-rigid medium at the base of a bridge pier can exert two effects on its dynamic response. First, the propagation of the seismic waves through the superficial layers – often made by alluvial deposits – can vary the frequency content of acceleration at the top,

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compared to deeper rock layers (kinematic interaction). Second, the flexibility of soil reduces the system stiffness (inertial interaction) and allows for energy re-transmission from structure to soil (radiation damping). These phenomena can produce opposite effects on the seismic behaviour of a bridge, which are not always favourable.

Soil-structure interaction has been largely studied in order to set up simplified design procedures with a good degree of accuracy [Scott 1981], [Gazetas 1991], [Wolf 1994]. Recently, attractive theories have been developed, which are based on the finite elements method [Wolf 1997]. In spite of that, it seems difficult to develop a method which is able to model both soil and structure together with adequate accuracy, taking into account the non-linear behaviour of the structure under strong earthquakes. Consequently, the structural engineer has to choose either a *structural* or a *geotechnical* approach, depending on whether he is more interested in the response of the structure – considering a detailed structural model together with a simplified soil model – or vice-versa. In order to investigate the actual behaviour of important structures subjected to significant soil-structure interaction effects, and marked anelastic phenomena in the structure as well, it is important to set up a method which takes into account both effects with adequate and comparable accuracy.

Summarising the results of the studies in this field carried out by several authors in the last 30 years, the present work investigates the dynamic behaviour of bridge piers under strong earthquakes, using a general approach which could be extended to more complex structures. The ductility demand and the relative displacements between substructure and superstructure are checked to evaluate the performances required to bearings and joints [CEN 1994]. Using the substructure method, it is possible to analyse kinematic and inertial interactions separately. In the current practice, however, the properties of the earthquake at the surface are defined by the design codes as a function of soil characteristics. Therefore, attention has been focused on the modifications produced by stiffness reduction and radiation damping on the structural response, while the filtering effect on the propagation of seismic wave through soil layers has been neglected. The most common arrangements of bridges have been examined [Calvi, Priestley 1991]. In particular, reference is made to simply supported deck bridges, with span length ranging from 20 to 40 m, and the substructure made of single column hollow circular piers. Basically, piers behave like cantilevers and can be analysed separately, both in the longitudinal and in the transverse direction.

After presenting the model and the method used, the results of an extensive parametric investigation are examined. Three main parameters are varied: the height of the pier, the span length and the shear wave velocity of the soil, which is assumed to be homogeneous.

MODELS OF SOIL, STRUCTURE AND SEISMIC ACTION

Soil and foundation

Simplified but reliable approaches to soil-structure interaction, also capable to consider different types of foundation, can make use of two models: Cone Models (CM) and Lumped Parameter Models (LPM).

CM's describe the dynamic interaction between an elastic halfspace and a massless rigid disk in the frequency domain [Wolf 1994]. Several shapes and soil configuration can be considered through CM, as shallow or deep foundations, embedded in a homogeneous or layered soil, the latter idealized as a set of horizontal layers resting on a rigid or flexible halfspace. Rectangular or more complex shape foundations can also be considered through an equivalent disk radius. However, as circular hollow piers are considered in this study, their foundations are also assumed as circular, rigid and perfectly sticked to the soil.

The dynamic stiffness coefficient for the soil-foundation system, referred to the *j*-th degree of freedom, is expressed in complex form as:

$$S_j(a_0) = K_{s,j}(k_j + i a_0 c_j) \tag{1}$$

where: $K_{s,j}$ is the static stiffness;
k_j is the real part of the dynamic term (dimensionless stiffness);
c_j is the imaginary part of the dynamic term (dimensionless damping);
$a_0 = \dfrac{\omega r}{v_s}$ is the dimensionless frequency, with r radius of the disk and v_s shear wave velocity.

As CM's provide the dynamic stiffness matrix of the soil-foundation system as a function of frequency ω, they cannot be employed for the analysis in the time domain.

LPM's consist of elastic truss elements, viscous dashpots and concentrated masses assembled in a simple model with internal and external degrees of freedom [Wolf 1994]. The dynamic stiffness of the model is a function of

the natural frequency, but the individual stiffness, damping and mass coefficients are constant. Therefore, LPM's are suitable to analyse the soil-foundation system in the time domain.
The LPM dynamic stiffness is expressed as fraction of two polynomials with real coefficients:

$$\frac{S(a_0)}{K_s} \cong S_s + \frac{1+p_1 ia_0 + p_2 (ia_0)^2 + ... + p_{m-1}(ia_0)^{m-1}}{1+q_1 ia_0 + q_2 (ia_0)^2 + ... + q_m (ia_0)^m} \tag{2}$$

where:

S_s is the singular stiffness [$S(a_0 \to \infty)$];

m is an integer depending on the approximation needed;

p_i, q_i are the polynomial coefficients.

The polynomial coefficients are evaluated by fitting the LPM stiffness on that of the CM through the least squares method. The ratio of the polynomials in (2) can be expressed as a partial-fraction of the form

$$\sum_{k=1}^{m} \frac{A_k}{ia_o - x_k} \tag{3}$$

where:

x_k are the roots of the denominator;

A_k are the residues at the poles.

Each term of the partial-fraction expansion of the total stiffness coefficient is represented by a discrete element model, arranged in parallel in the LPM. Springs, dashpots and masses constant coefficients of the LPM are determined accordingly as function of the polynomials. After having assembled the LPM with the superstructure model, a general purpose program can be used to analyse the soil-structure system.

Structure

An individual bridge pier is analyzed with reference to an indefinitely long viaduct with constant weight per unit length. The decks are supposed to be rigid and fastened to the piers both in the longitudinal and the transverse directions. The piers have constant hollow circular cross section with 3 and 3.6 m internal (*d*) and external (*D*) diameters, respectively. The members have been proportioned enforcing that the maximum compressive stress remains below 20% of the compressive strength under dead and live loads. The piers stiffness depends on concrete strength f_{ck}, taken equal to 35 MPa. Moreover, to account for cracking, the moment of inertia of the gross section has been reduced by 25%.

A shallow rigid foundation is considered in the study. It is cylindrical with 10 m diameter and e = 5 m embeddement in an elastic homogeneous halfspace. The characteristics of the halfspace are:

- mass density ρ = 2000 kg/m^3;
- Poisson's ratio ν = 0.25;
- hysteretic damping ζ_g = 5%;
- S-wave velocity v_s, ranging from 100 to 500 m/s.

The dynamic analyses of the soil-structure system (pier and foundation) are performed through the model shown in fig. 1. The pier is modelled through linear elastic beam elements, distributed masses and viscous mass-proportional dashpots. The material non-linearities are concentrated in a plastic hinge at the pier base, whose length is about 0.5 *D* (as used for the calculation of curvature ductility demand). The moment-rotation relation of the plastic hinge is rigid-plastic with about 1% hardening. The yielding moment is derived according to a response spectrum analysis based on EC8, assuming the following parameters:

- behaviour factor q = 4;
- importance factor I = 1;
- viscous damping ratio ζ_s = 2.5%;
- response spectrum with maximum amplification 2.5 between 0.15 and 0.6 s;

- 0.35 g PGA.

A lumped mass matrix and a viscous damping matrix are considered. The viscous damping is mass-proportional and equal to 2.5% of the critical damping.

2.3 Seismic action

The seismic action is represented through 5 artificial accelerograms matching the elastic response spectrum of EC8 for soil type B [CEN 1994]. The duration is 30 s. The time modulating function has the constant branch equal to 1 in the interval from 2 to 25 s, with a linear ascending branch between 0 and 2 s and a linear descending branch between 25 and 30 s. The accelerograms have been scaled to 0.35 g PGA.

In order to calculate the dynamic response under seismic excitation, the driving loads are to be calculated. To this end, three steps are necessary:

1) the dynamic stiffness matrix of the free-field $\mathbf{S}^f(\omega)$ (superscript f denotes free-field) is determined at the nodes of the soil-foundation interface discretized through CM;
2) the free-field displacement at the same nodes $\mathbf{u}^f(\omega)$ due to the propagation of the seismic wave are calculated;
3) the driving loads are calculated as $\mathbf{S}^f(\omega)\ \mathbf{u}^f(\omega)$.

This procedure is suitable for the analysis in the frequency domain. In the time domain, the above steps need a primary Fourier transform of the motion in the frequency domain and an inverse Fourier transform of the driving loads in the time domain. S-waves, with a horizontal particle motion propagating vertically in a homogeneous halfspace with the shear wave velocity v_s, are addressed. In this case, the free-field amplification effects are negligible and no site-response analysis is necessary. Anyway, to calculate the driving loads, the free-field motion is to be determined in the nodes which subsequently will lie on the structure-soil interface. As the control motion is specified at the free surface, the calculation proceeds from this point downwards.

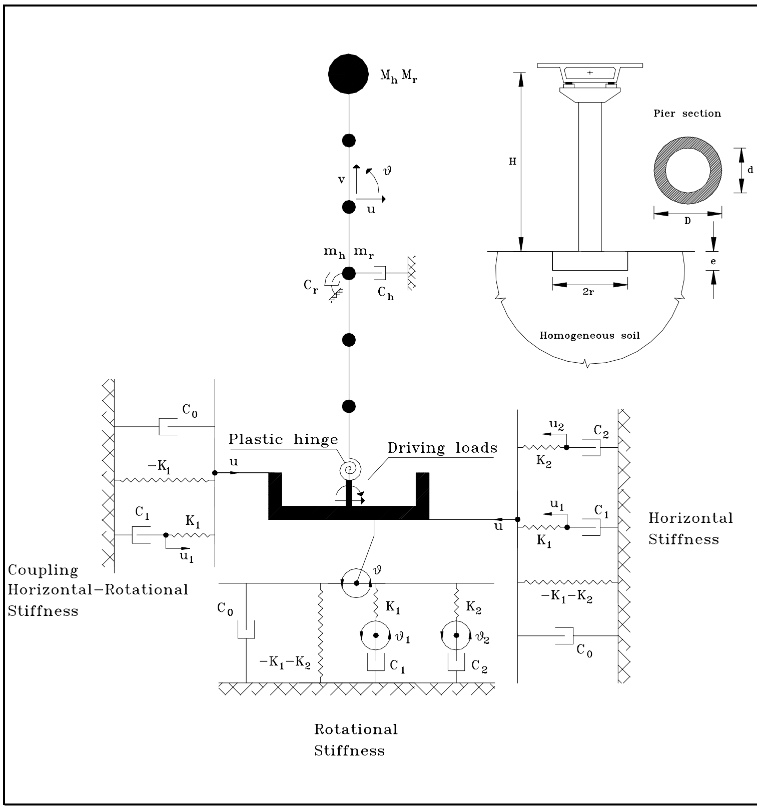


Figure 1 – Model of pier on a cylindrical foundation embedded in soil

PARAMETRIC INVESTIGATION

Parametric analysis

The following parameters have been selected, as they are deemed the most important ones in practical design:

- pier height (H = 10, 20, 30, 40, 50 m);
- deck length, taken equal to 20, 30, 40 m; this parameter affects the mass and the inertia forces at the top of the pier and the axial force at the pier base; the dead weight for unit length of the deck is taken equal to 200 kN/m, therefore the deck weight results: P = 4000, 6000, 8000 kN;
- shear wave velocity of soil (v_s = 100, 200, 300, 400, 500 m/s); the case v_s = 100 m/s is an extreme case to emphasize the interaction effect. Another extreme case is $v_s = \infty$ (no interaction), which represents the reference case for the analyses with interaction.

All the cases resulting from the possible combinations of the above parameter values have been analyzed, subjected to five generated accelerograms, as explained above. All the other characteristics are kept unchanged, the size of the foundation cap included, in order to make the comparison among the different cases and the evaluation of the influence of the selected parameters easier.

Results

The structural response quantities considered in the analyses are:

- the elastic period T, calculated from the free vibrations of the pier;
- the equivalent viscous damping ζ_S, representing the dissipation of energy due to both structural damping and radiation damping, calculated from the decay of the free vibrations of the pier;
- the average of the maximum displacements of the pier top relative to the base u;
- the average of the hysteretic energies dissipated through the plastic hinge at the pier base W;
- the average of the displacement ductility demands μ;
- the average of the curvature ductility demands χ.

The average quantities are referred to the five analyses carried out on the same structural model with different accelerograms. The final results are expressed as a function of v_s. They are divided by the corresponding results obtained for the fixed base structure, to obtain dimensionless quantities.

The diagrams reported in the following refer to the case of P = 6000 kN. When P is varied to 4000 kN or to 8000 kN, variations up to 20% of the response quantities have been found.

Elastic period

In fig. 2 there is shown the elastic period divided by the elastic period of the pier with fixed base as a function of the shear wave velocity of soil. The elastic period of the pier with fixed base ranges between 0.25 and 3.5 s, covering a large number of real cases. The analyses with interaction show that the fixed base hypothesis can lead to underestimate the elastic period of the soil-structure system up to 40% in many real cases. Differences up to 120% are found for v_s = 100 m/s, but they are not significant for practical applications, though they prove that the overall pier flexibility exhibits an exponential increase as soil stiffness decreases. These differences are more pronounced for squat piers (the prominent case occurs for H = 10 m) and soils of medium stiffness (v_s = 200÷300 m/s). As soil stiffness increases beyond 400 m/s, the variations are always below 10%.

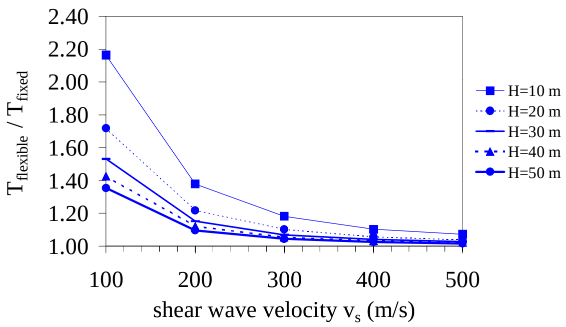
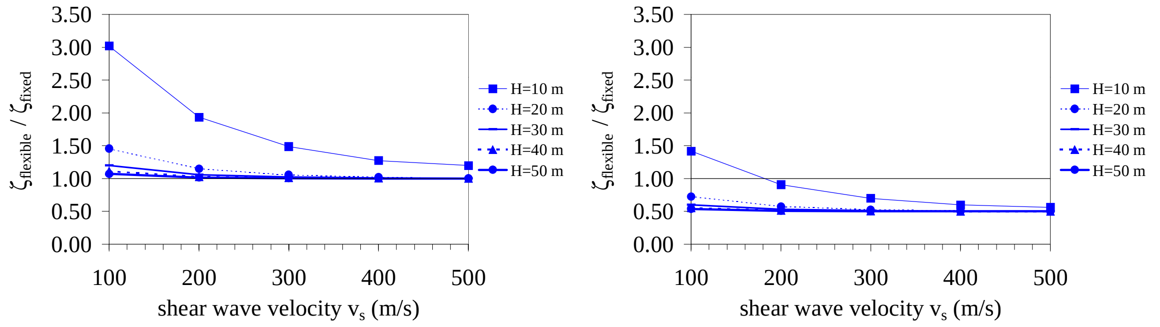


Figure 2 – Period ratio (structure with interaction / fixed base structure)

Equivalent viscous damping ratio

The dissipation of energy through radiation damping, expressed in terms of equivalent viscous damping ratio, is significant only for H = 10 m, as shown in fig. 3a. In this case the differences of the overall damping (structural and radiation) range between 0 and 200%, the latter occurring when v_s = 100 m/s. When v_s = 200 m/s, the differences reduces to 100%. For taller piers the differences are always below 20%, except v_s = 100 m/s when they increase up to 50%.

Fig. 3a relates to a fixed base structure with 2.5% structural damping ratio, while in fig. 3b the structural damping ratio of the fixed base structure is 5%. In this latter case it appears that a fixed base structure analysis can lead to overvaluations around 50% in most cases. Assuming a structural damping ratio equal to 5% is appropriate only when H = 10 m and v_s = 200 m/s. For H = 10 m and v_s = 100 m/s, an undervaluation of damping around 50% occurs.



a) ζ_{fixed} = 2.5%   b) ζ_{fixed} = 5%

Figure 3 – Equivalent damping ratio (structure with interaction / fixed base structure)

Maximum relative displacements

As shown in fig. 4, the maximum displacements of the pier top relative to the base are generally underestimated by the fixed base model, especially for squat piers (H = 10 m) and low shear wave velocity. The values of the structure with interaction can be 60% greater than those of the fixed base structure. The maximum value occurs when v_s = 200 m/s. If v_s = 100 m/s the differences can increase exponentially up to 100%. For slender structures, they oscillate around 20%, when v_s ranges between 200 and 500 m/s, while for very soft soil they can increase up to 70%. A 5% structural damping ratio does not produce significant variations.

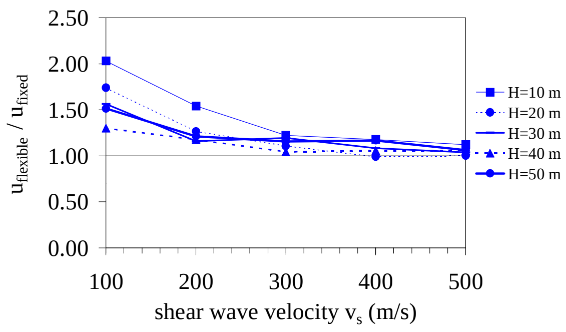
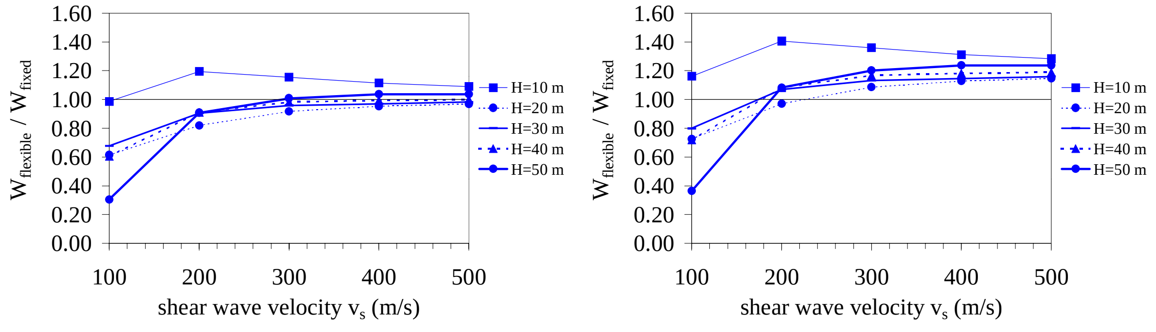


Figure 4 – Top displacement ratio (structure with interaction / fixed base structure)

Energy dissipation at the pier base

Energy dissipation through hysteresis in the plastic hinge at the pier base can exhibit opposite trends, depending on the pier height (fig. 5a). For squat structures (H = 10 m), the analysis with interaction leads to a more significant dissipation than the fixed base structure. It can increase up to 25% over, if a 2.5% structural damping ratio of the fixed base structure is used, but can also reach 40% when the structural damping ratio is 5% (fig. 5b). The highest values occur when v_s = 200 m/s. As showed in fig. 5a, when the pier height increases, energy dissipation is generally overestimated by a fixed base structure analysis up to 20%. When soil is softer (v_s=100 m/s) the differences can increase up to 70%. Assuming a 5% structural damping ratio of the fixed base structure, analyses of more slender piers without interaction underestimate energy dissipation up to 25% when soil is stiff, but the tendency is opposite when the shear wave velocity of soil is less than 200 m/s.



a) ζ_{fixed} = 2.5% b) ζ_{fixed} = 5%

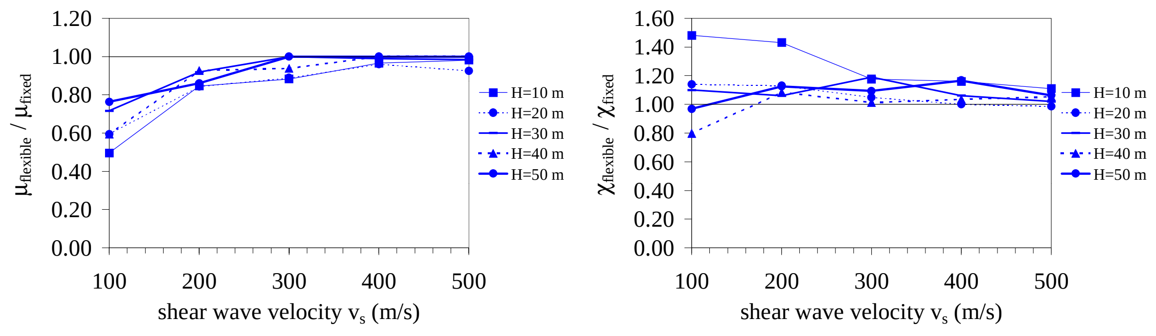
Figure 5 – Dissipated energy ratio (structure with interaction / fixed base structure)

Displacement ductility demand

The displacement ductility demand is the maximum top displacement divided by the top displacement when yielding is first reached at the pier base. It is generally overestimated when neglecting interaction. Excluding the case v_s = 100 m/s, the differences range from 0 to 20%, the latter for v_s = 200 m/s (fig. 6a). Between 100 and 200 m/s overestimation is between 20 and 50%.

Curvature ductility demand

The strongest increase of curvature ductility demand induced by soil-structure interaction is registered for H = 10 m (fig. 6b). In this case the increase is always over 10%. Excluding the case v_s = 100 m/s, when differences can be up to 50%, the maximum value is nearly 40% greater than that of the fixed base structure, and occurs at v_s=200 m/s. As the pier height increases, the differences decrease. Moreover, increasing the structural damping of the fixed base pier, the ratio of the curvature ductility demands does not vary significantly.



a) displacement b) curvature

Figure 6 – Ductility demand ratios (structure with interaction / fixed base structure)

CONCLUSIONS

The results of an extensive parametric investigation on the seismic response of bridges have been illustrated. The bridge piers are modelled in the non-linear range and a Lumped Parameter Model calibrated through the use of a Cone Model takes the effects of the soil-structure interaction into account. To represent real cases, usual pier characteristics have been considered, together with accelerograms consistent with the spectrum EC8, soil type B.

Some interesting remarks can be done. The flexibility of the soil-foundation system can significantly affect the period of vibration of the structure. This generally implies that an analysis neglecting interaction leads to larger stress values, like the total transverse shear at the pier base, and thus to an over-conservative design with respect to this quantities. On the other hand, the displacements at the top of the structure relative to the base are larger because of the foundation rocking. Therefore, connections and bearing are to be carefully designed, especially for squat pier bridges. The level of damage at the pier base can be strongly affected by the soil-foundation flexibility. In fact, two opposite trends can occur, depending on the pier height. For piers of limited height, the level of damage appears to increase when taking into account soil-structure interaction effects. This is confirmed by the curvature ductility demand. In this case, the analysis of the structure with fixed base can underestimate the amount of transverse reinforcement required to get a sufficient ductility in the plastic hinge zones. For taller piers, soil-structure interaction reduces damage. Therefore, at least in these specific situations, it appears appropriate to use a smaller behaviour factor, in order to reduce the level of damage. On the contrary, the required displacement ductility factor always decreases, particularly when the soil is very soft. It is important to stress that accounting for radiation damping through an equivalent structural damping equal to 5% is often not justified, because the amount of energy radiated in the soil exhibits large variations, depending on both structure and soil characteristics.

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