PILE FOUNDATION FOR THE SEISMIC ISOLATION OF GIRDER BRIDGES

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SUMMARY

This paper examines the possible uses and the advantages of seismic isolation at the base of girder bridges. The attention is focused in particular on the feasibility of adopting a system based on the abatement of the horizontal stiffness of foundation piles, as is obtained by using piles partly non-confined by the surrounding soil. Eight bridges with standard characteristics, representing a sufficiently wide class of girder bridges, were selected. On these structures, subjected to real seismic excitation, a parametric analysis was conducted with the aim of evaluating the optimal characteristics of the isolation and the applicability of the technique.

INTRODUCTION

Base isolation systems have proved very effective in the seismic isolation of buildings (Soong & Costantinou 1994). In frame structures, in fact, the severity of seismic damage is correlated not only to maximum storey acceleration, but also to maximum inter-storey slip. The function of base isolation therefore has the twofold effect of serving as an effective filter for earthquake harmonics and at the same time of dislocating shear strains in the isolation devices. When dealing with bridge structures, the latter requirement loses importance whilst, on the other hand, the success of isolation at the pier top for structures of this type (1,2) can be associated with the construction process and lower additional load-effects to which the isolation device is subjected (absence of the weight of the pier and second order effects). From a purely dynamic standpoint, the closer is the structural model to a single DOF system (with increasing span and decreasing pier height), the greater is the tendency of the two isolation systems to be equally effective.

Despite the foregoing considerations, base isolation may sometimes prove more effective even in bridge design. For instance, the use of foundation piles as an isolation system offers a number of advantages: conceptual simplicity, mature technology, considerable mechanical strength, the possibility of exploiting the hysteretic damping of the soil or filler materials.

In the system examined the upper part of the pile is released from the surrounding soil and is connected to the structure above it by means of a hinged restraint. The structure restrained to the set of piles therefore acquires a predominantly translational degree of freedom. This leads to an increment in the fundamental period of the structure, meeting the objective of providing a sufficient degree of isolation and at the same time filtering the excitation components of the higher modes.

The advisability of adopting the proposed system depends of a multiplicity of parameters which can be grouped into four classes: parameters of the structure on a fixed base, spectral characteristics of the excitation, parameters of the isolation system, soil parameters. A preliminary spectral analysis, based on the response spectrum of the Eurocode 8 (1994), leads to the following qualitative conclusions

This technology, especially in view of the large pile diameters generally employed in bridge construction, will not supply a high degree of isolation. This entails the adoption of great free lengths, whose feasibility should be evaluated for the different types of structure.

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2) The use of concrete piles might find a limitation in the strength of the material. It should be noted however that the problem is attenuated with increasing pile diameter, this being an aspect of special interest in bridge isolation.

3) The influence of soil parameters is quite modest. The assumption of a homogenous soil is on the safe side.

In the following, the response to seismic excitation of eight bridge structures, selected to embody standard bridge design characteristics, is analysed by means of 2-DOF models. This makes it possible to outline the realm of applicability of this technology.

1. STIFFNESS MATRIX OF LATERALLY LOADED PILES

In general, two types of models are taken into account for the soil around the pile: Winkler's model, and, as an alternative, the continuous elastic medium model. Either approach requires appropriate numerical procedures and calibration processes. A finite elements numerical analysis of a stiff body in a heterogeneous linear elastic semi-space, having a shear modulus which varies linearly as a function of depth, leads to the following formulae for the determination of displacement and rotation values at the top of the pile (Randolph 1981):

$$u = F_G \cdot \left[0.27 \cdot H \cdot (l_c/2)^{-1} + 0.3 \cdot M \cdot (l_c/2)^{-2}\right]$$

$$\vartheta = F_G \cdot \left[0.30 \cdot H \cdot (l_c/2)^{-2} + 0.8 \cdot M \cdot \rho^{1/2} (l_c/2)^{-3}\right] \tag{1}$$

having defined:

$$F_G = \frac{\left\{E_p / \left[G_c \cdot (1 + 0.75 \cdot \nu)\right]\right\}^{1/7}}{\rho \cdot G_c \cdot (1 + 0.75 \cdot \nu)}$$

$l_c = D \cdot \left\{E_p / \left[G_c \cdot (1 + 0.75 \cdot \nu)\right]\right\}^{2/7}$, *critical depth*

$G_c = G(l_c/2)$, *shear modulus at a depth corresponding to half the critical value*

$\rho = G(l_c/2)/G(l_c/4)$, *degree of homogeneity*

where: u, ϑ: displacement and rotation at the top of the pile; H, M: shear and moment at the top of the pile (in the section of transition between the non confined and the confined portions of the pile); D: pile diameter; E_p: effective Young's modulus.

These formulae may be deemed to apply if the depth of the pile in the soil is at least equal to the critical length. If this condition is met, the pile can be regarded as a flexible element embedded in an elastic medium. The critical length can be taken to be the portion of pile effectively interacting with the soil. The degree of homogeneity expresses the variation in the elastic characteristics of the soil as a function of depth. It takes on a unit value in homogeneous soils. The presence of a set of piles causes of loss of efficiency due to the interaction between the individual piles. In several tests conducted by Williams (1979) on sets of two and three piles in dense sand this reduction was seen to be as high as 40% of the efficiency coefficient. This can determine an increase of the isolation level, which in this study was not taken into account.

The seismic isolation system aims to reduce the translation stiffness of the piles by creating a length along which the pile is not confined in the soil. The connection with the structure above produces a hinged restraint, so as not to load the piles with a moment at the top. The relationships above show that, in homogeneous soil, releasing a concrete pile, 400 mm in diameter, driven in clay soil (G_C=12 MPa and ν=0.15), over a length corresponding to one third of the critical value (about 4 m) leads to a reduction in stiffness of about five times. As for the load-effects on the pile, numerical tests on piles loaded with horizontal forces have shown that, with a good approximation, the maximum bending force on the pile corresponds to the moment in the section of transition between the confined and the non confined portions of a pile. Furthermore, the applicability of this principle increases with increasing free length-to-critical length ratio and with increasing soil homogeneity.

2. 2-DOF MODEL OF THE ISOLATED BRIDGE

The numerical experiments described in the following paragraphs were conducted with reference to the discrete simplified model shown in Fig. 1. The dynamic equations associated with this model are (Kelly 1990):

$$m_{tot}\ddot{x}_b + \mathbf{a}^{\mathbf{T}}\mathbf{M}\ddot{\mathbf{x}} + c_b\dot{x}_b + k_b x_b = -m_{tot}\ddot{x}_g\,;$$
$$\mathbf{M}^{\mathbf{T}}\mathbf{a}\ddot{x}_b + \mathbf{M}\ddot{\mathbf{x}} + \mathbf{C}\dot{\mathbf{x}} + \mathbf{K}\mathbf{x} = -\mathbf{M}^{\mathbf{T}}\mathbf{a}\ddot{x}_g\,. \qquad (2)$$

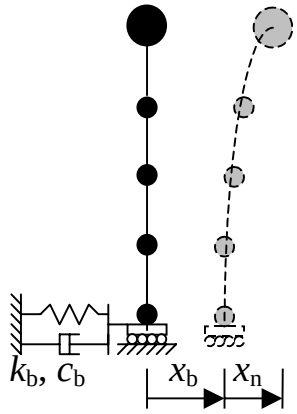

Fig. 1: Simplified 2-DOF

where m_{tot} : total system mass; c_b , k_b : damping and stiffness of the isolation; **M**, **C**, **K**: mass, damping and stiffness matrices of the superstructure on a fixed base; x_b : isolation displacement relative to the soil; **x** : superstructure displacement vector relative to the fixed base; **a** : entrainment vector; $\ddot{x}_g$: soil acceleration. Let us assume that the motion of the superstructure on a fixed base is closely approximated by its first vibration mode (Kelly 1990, De Nicolo et al. 1997). This amounts to setting $\mathbf{x} \cong {}_1\mathbf{w}x_n$, where x_n is the displacement at the top of the structure relative to the fixed base, and ${}_1\mathbf{w}$ is the first modal eigenvector, with ${}_1w_n$=1. By substituting into (2) we get:

$$m_{tot}\ddot{x}_b + \Gamma_1 M_1\ddot{x}_n + c_b\dot{x}_b + k_b x_b = -m_{tot}\ddot{x}_g\,;$$
$$\Gamma_1 M_1\ddot{x}_b + M_1\ddot{x}_n + 2\xi_1\omega_1\dot{x}_n + \omega_1^2 M_1 x_n = -\Gamma_1 M_1\ddot{x}_g\,. \qquad (3)$$

Where ω_1 and ζ_1 are the frequency and the damping factor of the first mode of the structure on a fixed base; $M_1 = {}_1\mathbf{w}^T M_1\mathbf{w}$ and $\Gamma_1 = \left({}_1\mathbf{w}^T M_1\mathbf{a}\right)/M_1$. are the modal mass and the first mode participation factor. In this form, the problem of base isolation is governed not only by the stiffness and damping characteristics of the system adopted, but also by: the frequency and damping of the first mode of the superstructure, the modal mass, the total mass and the first modal participation factor (in bridges the latter is very close to 1).

By referring to the 2-DOF model described above and by integrating equation (3) step by step, it proved possible to study the feasibility of the isolation system being considered.

3. PARAMETRIC STUDY

The parametric study was conducted on a limited number of girder bridges (eight in all) suitably selected to be able to draw conclusions of a general nature. The main target of the parametric analysis was the stiffness of the piles and hence their free length. The study was performed for three types of reinforced concrete piles (diameters: 800, 1000, 1200).The following variables were checked to evaluate the quality of the isolation:

1) Percentage reductions in the load-effects on the pier;
2) Pier top displacement;
3) Pile top displacement;
4) Load effects on the piles.

Tables 1-2 give the data of the bridges examined and the number of piles to be fitted for three different pile diameters. Table 3 summarises the parameters characterising the dynamic model of the individual bridge.

Bridge	Type **1**	Type **2**	Type **3**	Type **4**	Type **5**	Type **6**	Type **7**	Type **8**
Pier height, H [m]	15	15	15	40	40	40	80	80
Deck length, L [m]	30	60	100	30	60	100	60	100
Deck mass [t/m]	20	26	30	20	26	30	26	30
Pier mass [t]	240	240	240	1500	1500	1500	4000	4000
Pulvino mass [t]	50	50	50	300	300	300	800	800
Pier section flexural stiffness (EJ) [Nm^2]e09	286	286	286	1228	1228	1228	1870	1870

Table 1: General data concerning the bridges examined.

Bridge	Type **1**	Type **2**	Type **3**	Type **4**	Type **5**	Type **6**	Type **7**	Type **8**
Pile Ø 800	6	12	-	14	20	-	-	-
Pile Ø 1000	4	8	14	10	14	20	-	-
Pile Ø 1200	3	6	10	8	10	14	18	22

Table 2: Number of piles adopted for the different bridge types.

Bridge	Type **1**	Type **2**	Type **3**	Type **4**	Type **5**	Type **6**	Type **7**	Type **8**
ω_1 [rad/s]	19.08	12.38	9.06	6.92	5.16	3.99	1.86	1.54
ζ_1 [%]	5	5	5	5	5	5	5	5
m_{tot} [t]	840	1800	3240	2100	3060	4500	5560	7000
M_1 [t]	698	1658	3098	1205	2163	3603	3173	4610
Γ_1	1.0358	1.0151	1.0081	1.1316	1.0731	1.0438	1.1332	1.0916

Table 3: Parameters of the dynamic model of each bridge.

In the following, reference shall be made to the "degree of isolation", α, i.e., the ratio between a structure assumed to be stiff on the isolation system and the period of the structure on a fixed base. The "degree of isolation" provides a quantitative measure of the shift towards lower isolation frequencies, of lesser significance in seismic spectra. Despite its qualitative nature, this parameter remains independent of the response spectrum employed. A number of standards and recommendations set minimum values for this degree of isolation, for instance by requiring that $\alpha \geq 2$. However, whilst values of α approaching 1 entail risks of coupling, exceedingly high values are often associated with an excessive global system flexibility, which is unsatisfactory. Another isolation parameter adopted is the percentage response difference which, though it is strictly correlated with α, supplies more conclusive quantitative evidence.

Fig. 2 shows the diagrams of the degree of isolation, α, plotted as a function of the free length of the pile, for two of the pile diameters considered. The diagrams are grouped for bridges of the same height. In view of the relatively modest influence of the elastic properties of the soil - as demonstrated by the preliminary sensitivity analysis - from now on we shall always refer to concrete piles driven in clayey soil (G_C=12 MPa and ν=0.15) under the assumption (to be on the safe side with respect to the degree of isolation) that ρ=1 (homogeneous soil). Apart from the obvious observation that, pile stiffness being the same, α increases markedly with natural frequency and, to a lesser extent, the frequency being the same, with total mass, the diagrams provide the following indications on the practical feasibility of the isolation system:

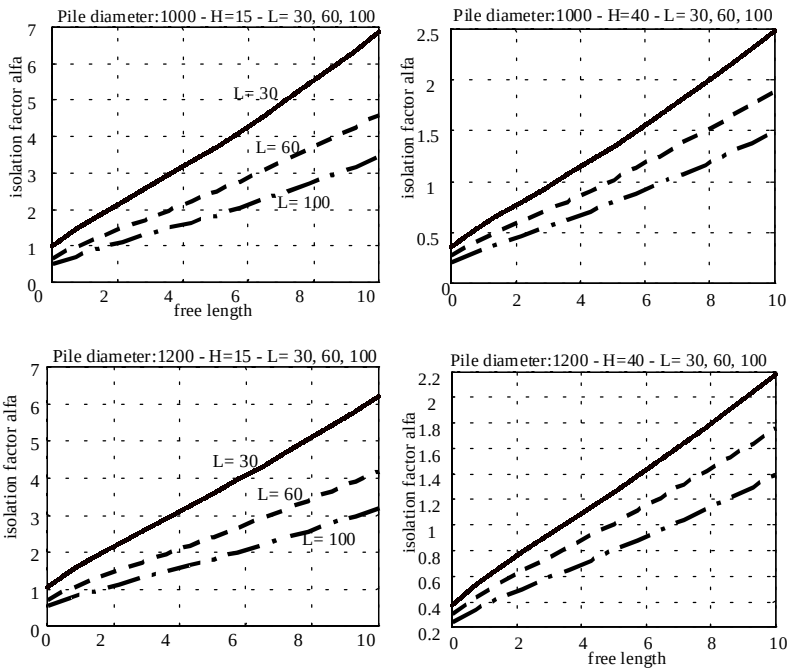

Figure 2: Degree of isolation α as a function

a) In bridges with very tall piers, high periods make this type of isolation unfit for practical design purposes; lower pile diameter does not make an appreciable contribution to the degree of isolation.

b) Conversely, in the design of bridges with relatively high natural frequencies, the system can be rated as satisfactory: system performance improves with decreasing pier height and span length. In this case too, the effects of fitting stiffer piles do not seem appreciable and therefore it might be advisable to adopt larger diameters to cope with strength problems.

It should be pointed out, however, that the degree of isolation is a qualitative parameter and the results may vary a great deal for different seismic spectra.

4. RESULTS OF THE NUMERICAL EXPERIMENTS

The acceleration signal used in the analysis refers to the Loma Prieta earthquake (1989, Natural Science Building at UC Santa Cruz, E/W direction, PGA=0.4248 g). Its elastic response spectrum (fig.3), reveals a rather typical frequency content, with a main peak at the low periods and a secondary peak on periods slightly higher than 1 s. Fig. 4 illustrates the response of different types of bridge to increasing isolation stiffness, more conveniently expressed in terms of "degree of coupling" Ω, being $\Omega^2 = 1/\alpha^2 = k_b / \left(\omega_1^2 \cdot m_{tot}\right)$.

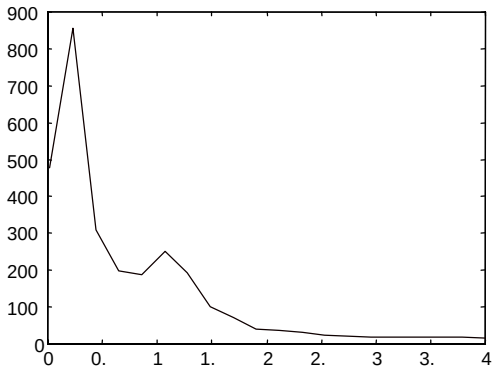

Figure 3: Response spectrum (Loma Prieta 1989, Natural Science Building, UC Santa Cruz, E/W, PGA=0.4248 g).

It can be seen, first of all, that the diagrams are clearly comparable, confirming that Ω, or α, are highly significant parameters.

Ω being the same, the displacements required of the isolation device to achieve a certain level of reduction of the response increase from type 1 to type 8 bridges, this increase being acceptable, however, with respect to the height of the pier. At the same time, deck displacements increase by one order of magnitude but, within the range of isolation degrees of interest, they remain at sufficiently low levels. It may be of some interest to note, in type 1 bridges, for Ω ≅0.3, the amplification effect produced on the piles by the secondary peak which the spectrum has on periods slightly higher than 1 s. In this range of periods, the system is governed by pile deformability, and this, in the presence of high period harmonics, might pose severe problems of displacements and load-effects on the piles.

In the portion of the diagram of greatest significance for design purposes (values of Ω lower than 0.5) we observe a similar law in the attenuation of the response, but clearly, in tall bridges, with huge total mass, a certain degree of isolation can only be attained at the price of much higher deformability levels, whose feasibility is discussed later on.

The tests being considered were extended to the eight bridges described in Table 1, each of which was associated, compatibly with individual bridge design, with three different pile diameters, as listed in Table 2. For each of the corresponding dynamic models the integration of the equations of motion led to three types of charts:

1) Pile top displacement and relative displacement of the bridge top relative to the fixed base, as a function of the free length of the piles.

2) Percentage response of the bridge vs. the response of the non isolated one, as a function of the free length of the piles.

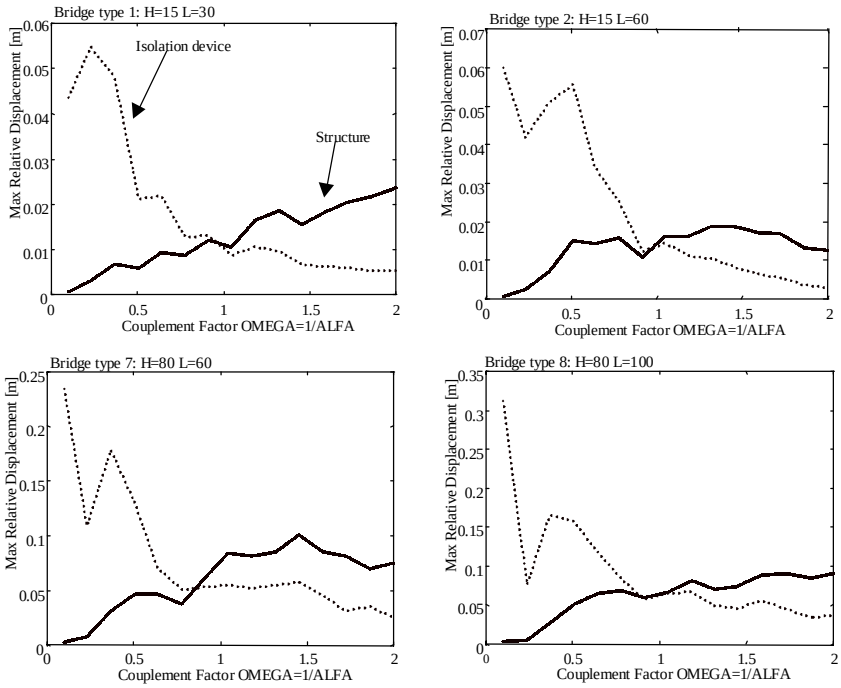

Figure 4: Relative displacements of the isolation system and the pier

3) Maximum non-dimensional moment on the foundation piles, as a function of the free length of the piles.

The diagrams of the first type (fig. 5) provide a general understanding of dynamic behaviour and make it possible to check for incompatible displacements. Admissible deck displacement relative to the ground has been assumed to be, indicatively, 1/300 the height of the pier (H/300). The diagrams of the second type (fig. 6) express the efficiency of the isolation system. Finally, the moment diagrams (fig. 7) illustrate the actual feasibility of the system, in terms of strength.

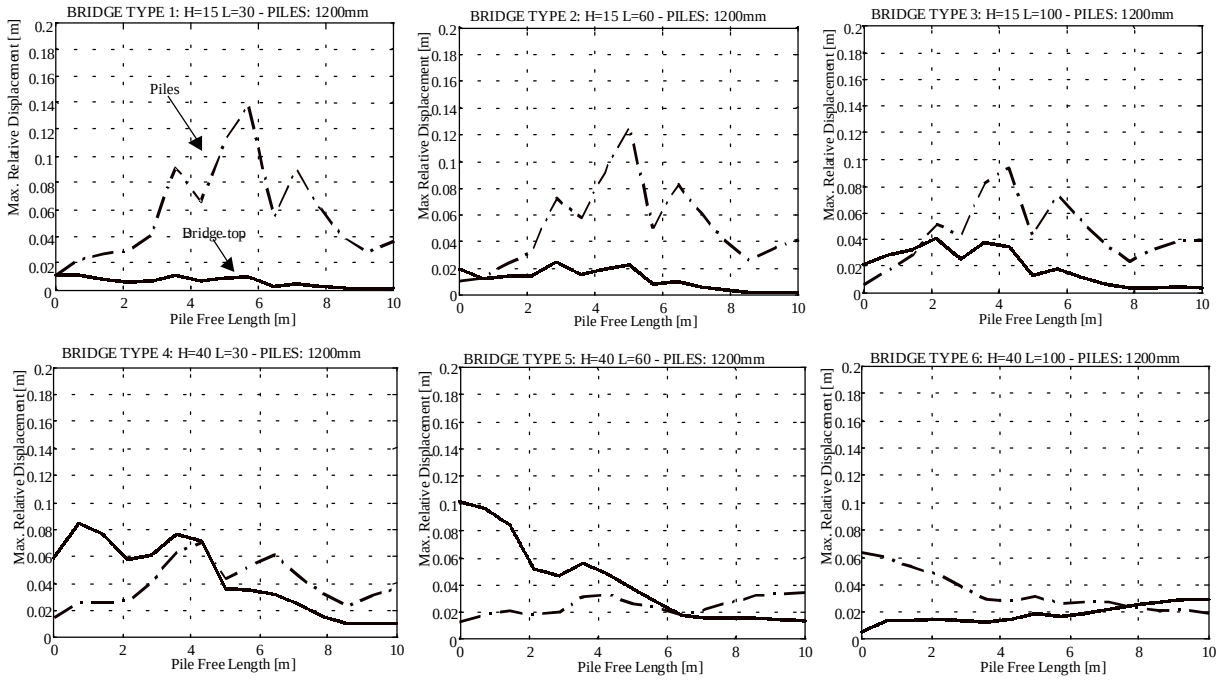

Figure 5: Relative displacements of the isolation system and the pier as a function of the free length of the (1200 mm) piles for type 1 - 6 bridges.

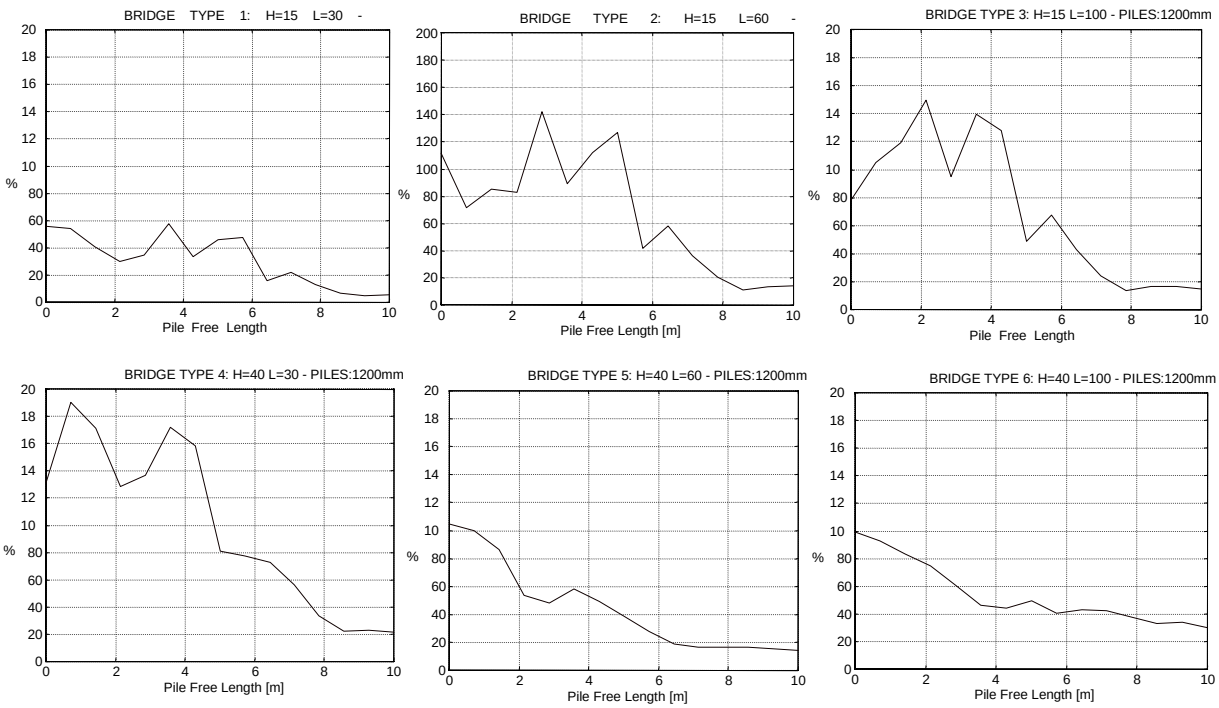

Figure 6: Percent response of the pier (vs. the response with no isolation system) as a function of the free length of the (1200 mm) piles for type 1 - 6 bridges.

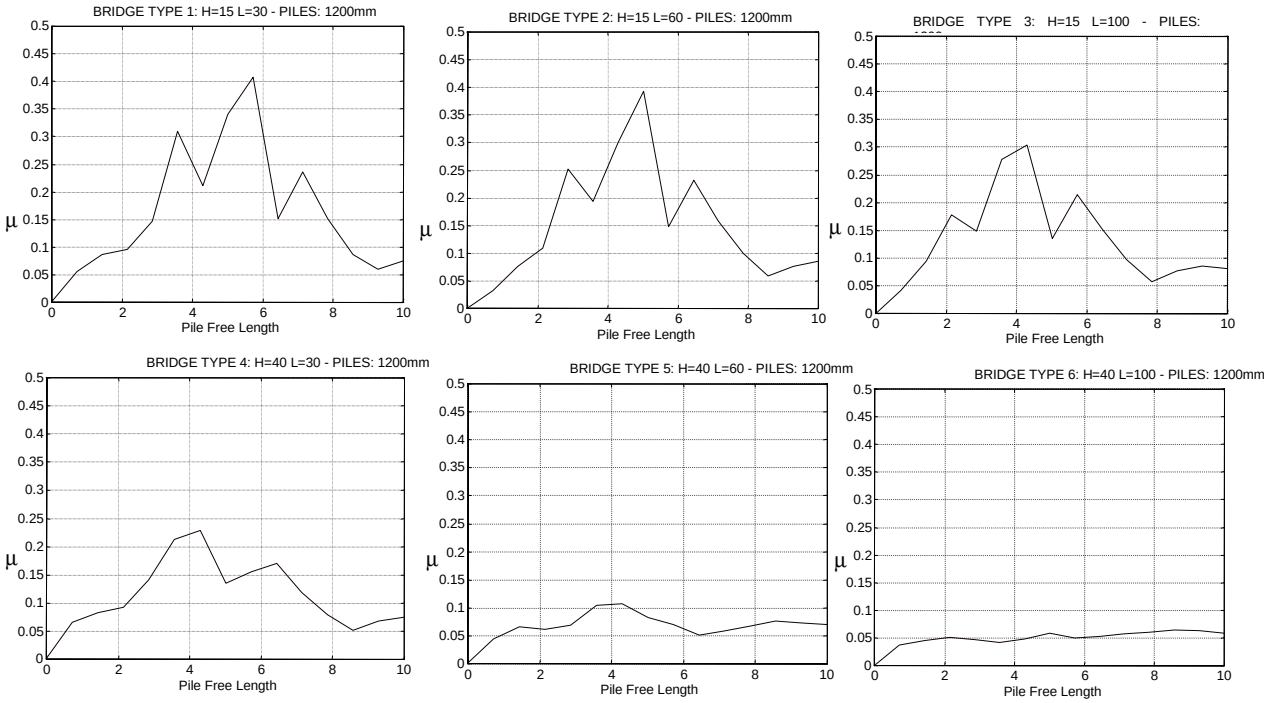

Figure 7: Maximum non-dimensional moment on the piles (1200 mm) for type 1 - 6 bridges.

Fig. 5 shows the relative displacements of the piles and structure as a function of the free length of the piles. These charts refer to type 1 - 6 bridges, with 1200 mm piles. We can discern two marked displacement peaks, caused by the passage at the second peak of the spectrum (after ca 1.05 s). These peaks move towards the origin with increasing initial period, i.e. with decreasing degree of isolation required.

Type 1-3 bridges (H=15m)

In the three bridges of lesser height, at the peaks, pile displacements can be as high as H/100. Hence, in such bridges, free lengths of over 4 m give rise to excessive displacements, whilst in order to achieve a good degree of isolation (α=2.5 for average characteristic spectra), pile free length must be comprised between 2.5 and 8 m (see fig. 2).

This last consideration is confirmed by the diagrams given in Fig 6, which illustrates the percentage response, with reference to that of a non isolated system, for pile diameter of 1200 mm. In type 1 bridge, hinged restraint alone makes it possible to obtain considerable response reductions, so that free lengths lower than 4 m provide a satisfactory degree of isolation without posing excessive displacement problems. In type 2 - 4 bridges, instead, free length values must be higher than 6 m, to reduce the response sufficiently and at the same time cross the displacement peaks. In type 3, and especially type 4, bridges, the negative results are compounded by the fact that their natural period falls in a trough of the spectrum, where the isolation device causes a raise in the response.

Fig 7 shows the non-dimensional maximum peaks on the individual pile $\mu = M/(\pi \cdot R^2 \cdot f_{cd})$, where M is the moment on the pile, R is the radius, and f_{cd}=18.75 N/mm^2 is the design strength of concrete, with R_{ck}=37 N/mm^2. The results seem encouraging, if one considers that, by designing the piles so as to confer sufficient ductility to the section, we obtain resisting moments of μ= 0.4÷0.5. It should be noted that the risk of pile failure is limited to particular free length values and to the bridges with the lowest period. Thus, the problem of excessive pile displacement, due to the low frequency energy contents, go hand in hand with excessive load-effect problems. It should be noted, however, that the use of larger diameter piles has a beneficial effect, both in terms of displacements and in terms of moments, without giving rise to an appreciable increase in response in the range of free length of interest.

Apart from the result relating to the bridge of type 1, which is markedly influenced by the shape of the spectrum, it can be seen that pile displacement and load-effect peaks correspond to the optimal isolation levels. This suggests that, in bridges of this type, free lengths lower than 6 m should be used with great caution, after a careful study of displacements and load-effects in the foundation piles. In this range of initial periods, the degree of isolation, α, is well correlated with the attenuation of the response.

Results obtained on 1000 mm piles showed that the increase in diameter causes a very modest shift of the response to greater free length. It is of some interest to report the 20% increase in the maximum μ on the piles.

Type 4-6 bridges (H=40m)

Where H=40m bridges are concerned (types 4-6), the displacements are seen to be modest with respect to the height of the bridge (ca H/400). The presence of a peak in the spectrum on high periods (ca 1.05 s) means that in bridges with periods just beyond the peak (bridges 4 and 5), the response is reduced with free lengths of only 4 to 6 m, which, surprisingly, correspond to low degrees of isolation (of about 1.5). In these bridges, in the presence of highly deformable soils, which widen the earthquake spectrum, no matter how the degree of isolation, even relatively small period increments might be decisive for the survival of the structure. Type 4-8 bridges display moment values on the safe side, decreasing with the initial period, on account of their being situated in low amplification zones of the spectrum (Fig. 3).

5. CONCLUSIONS

The main problem of using foundation piles for seismic isolation is the limited degree of isolation that can be achieved, first of all because the free lengths that can be realised are modest and secondly on account of pile displacement and load-effects. On girder bridges, characterised by highly variable and often high periods, the degree of isolation parameter (α) is not sufficient to predict the effectiveness of the isolation system. In such conditions, in fact, a decisive role is taken on by the shape of the seismic spectrum. A parametric study of the seismic response of eight types of isolated bridges has led to the following observations:

1) When the seismic spectrum is concentrated on the low periods, the degree of isolation required is low enough to be able to apply the proposed isolation system to bridges characterised by low natural periods. In 15 m tall bridges (types 1-3), the optimal degree of isolation is achieved with free lengths of 4 to 8 m, not far, that is, from the values for which displacements and load-effects reach critical levels. This problem, however, can be partially overcome by fitting larger diameter piles. In the field of interest, in fact, this intervention has minor effects on the response and it proves advantageous in terms of displacements and load-effects. It should be noted, among other things, that excessive pile displacement values (0.05-0.08 m) are found to occur precisely for those values of free length which are most detrimental to pile strength (μ=0.45-0.5). We conclude that in bridges with short piers (~15 m), the advisability of pile isolation should be evaluated case by case and its use should be rated as particularly critical in the presence of seismic components with high periods and in highly deformable soils. Furthermore, the isolation system is especially vulnerable to irregular spectra, with peaks at the periods higher than 1 s.

2) 40 m tall bridges (types 4-6), with periods of ca 1 s, fall in a zone of the spectrum where, particularly in deformable soil, even a limited degree of isolation achieves an substantial attenuation of the response. In this case, displacement values are generally acceptable with respect to the height of the pier and moments are reduced, since we are working in lower amplification period zones. On such bridges, the efficiency of the isolation depends greatly on the seismic spectra adopted and the use of the proposed system may prove advantageous. Taller bridges (types 7-8) are in zones of the spectrum where the isolation becomes ineffective.

6. REFERENCES

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