Preliminary SFSI Studies for the Messina Bridge Foundations

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ABSTRACT: The Messina Bridge in Italy, when completed will be the longest suspension bridge ever built, having a central span of 3300 m. In this paper, a generic study of the soil–pier–foundation interaction of the Calabrian-side pier of the preliminary Bridge design is examined. The massive concrete foundation of this pier is to be founded in a gravelly deposit, after the latter has been subjected to improvement by jet grouting. A parametric elastic study is first conducted to investigate the influence of the width and depth of soil improvement beneath the foundation. Then, the response of an alternative less conservative" foundation is investigated and compared to the "original" design. Non–linear features of material (soil) and geometry (uplifting, sliding, and second order effects), , as well as the flexibility of the tower, are taken into account.

1 INTRODUCTION

The Messina Bridge, would have made the span 60% longer than the Akashi Kaikyo Bridge in Japan (the currently largest suspension bridge, with a huge 1991 m central span). The bridge is planned to connect Reggio Calabria to Messina, Sicily. The big depth of water in the straits could only support the solution of a suspension bridge with a large central span. In the proposed project plan, the main span of 3300 m and the 2 x 183 m side spans are supported with suspenders from overhead main cables. Figure 1 depicts the longitudinal cross–section of the Bridge, along with the geological–geotechnical profile under the Bridge.

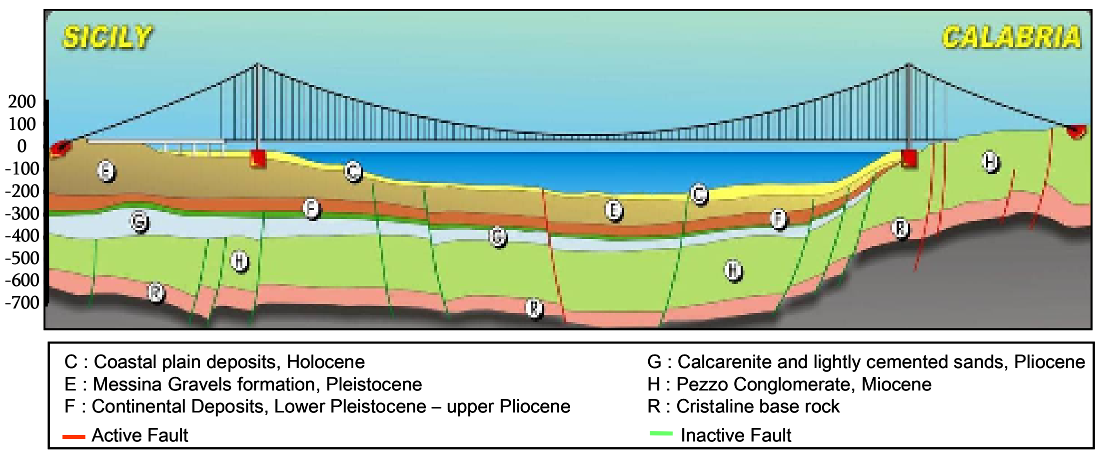


Figure 1. Longitudinal cross–section of the bridge, along with the geological–geotechnical profile (cour-

tesy of M. Jamiolkowski).

The height of the two offshore towers will be 383 m, in order to provide a minimum vertical clearance for navigation of 65 m. The Calabrian tower (Figure 2), examined in this study, will be founded on coastal sandy-gravelly deposits, which overlay Messina Gravel and Pezzo Conglomerate [Jamiolkowski & Lo Presti, 2003; Fiammenghi et al., 2009; Faccioli & Vanini, 2004]. (Coastal plane deposits had probably undergone liquefaction during the disasterous 1908 earthquake M_w = 7+ which occurred in Messina).

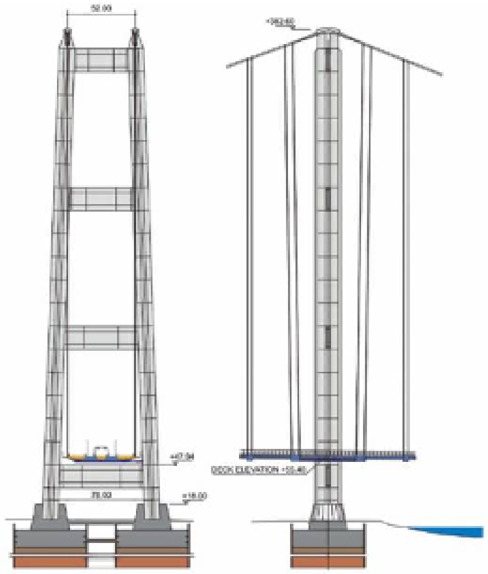


Figure 2. Front and side elevations of the main tower of the Messina Bridge (courtesy of M. Jamiolkowski).

The proposed tower foundation (Figure 3) will consist of two circular massive concrete foundations (connected with a cross–beam), surrounded by diaphragm walls with tips at levels–60 m and –50 m. The diameter of each circular foundation is 48 m. Jet–grouted sand is used below and outside the diaphragm walls to improve the underlying soil. The immense dimensions and the unique shape of the proposed foundation system were chosen so that the Bridge can withstand severe earthquake motion (Jamiolkowski) . In the framework of this study, the influence of the width and depth of soil improvement beneath the foundation is investigated. Moreover, the response of a smaller "less conservative" foundation is examined and compared to the proposed design.

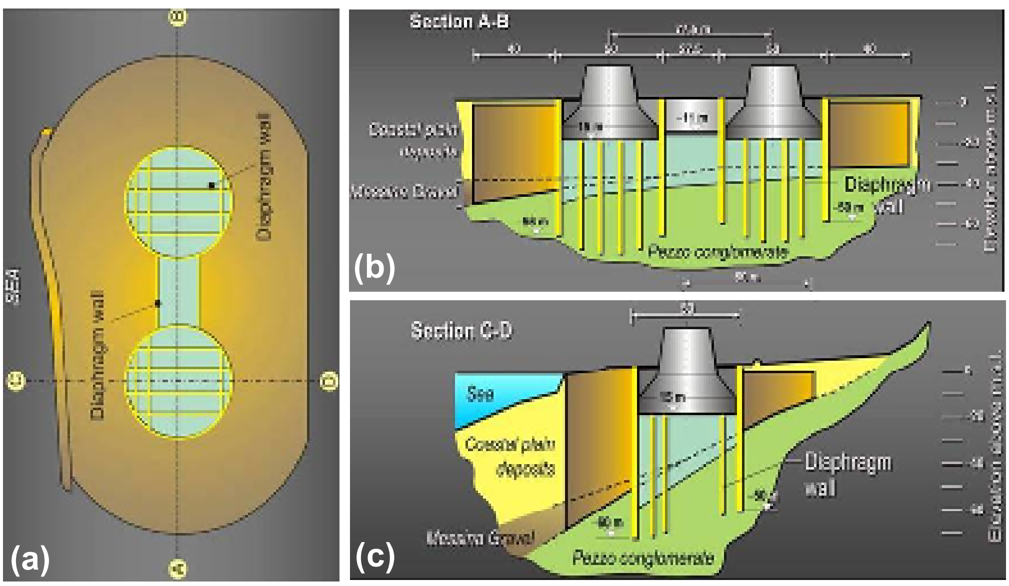


Figure 3. The Calabrian Tower foundation : (a) plan view, (b) transverse cross-section, and (c) longitudinal cross-section (courtesy of M. Jamiolkowski).

2. HOW BENEFICIAL IS THE PROPOSED GROUND IMPROVEMENT, AND TO WHAT EXTENT ?

The effect of the grouted area is certainly positive for static loading. However, under dynamic excitation, the presence of improved, and hence stiffer, soil may perhaps lead to amplification of the input motion, thus cancelling some of the benefits of increasing stiffness.

A parametric study is conducted to investigate the influence of the width and depth of the grouted area under the tower foundation. An equivalent plane strain B = 50 m footing, embedded at a depth of 17.5 m is considered to represent the Calabrian Tower foundation. The configuration of the simplified layout studied herein is depicted in Figure 4a. The foundation is supported on jet–grouted soil, which terminates at a depth of 50 m (bedrock). The shear wave velocity, Vs, is assumed equal to 200 m/s and 500 m/s for the non–improved and the jet–grouted sand, respectively. The lateral boundaries of the model are at distance equal to fourty times the depth of the embedded foundation ($40H_{emb}$), which constitutes the best trade off between simulation realism (i.e. avoidance of parasitic boundary effects) and computing time.

To evaluate the effect of the extent of ground improvement, seven scenarios of variable grouting width are first examined : B'/B = 0, 1, 2, 3, 4, 8, ∞ , where B = 50 m the width of the foundation.

Then, five scenarios of variable grouting depth are examined: with respect to the depth H' of the improved-soil area, we examine models with H'/H = 0, 0.5 (i.e. improvement at an area almost equal to the H_{emb}), 0.75 (improvement ending 8 m above the bedrock level), 0.88 (improvement ending 4 m above the bedrock level), and 1 (improvement ending at the bedrock level). Figures 4b and 4c depict the schematic views of the scenarios mentioned above.

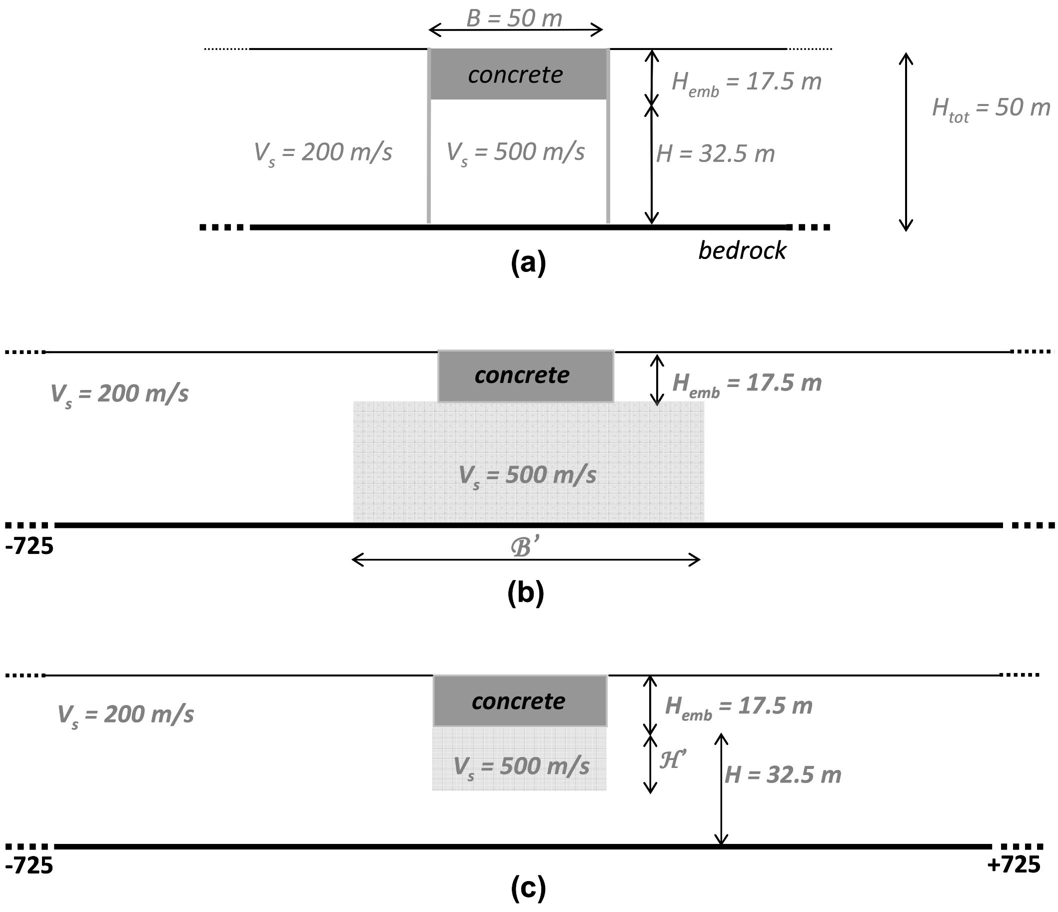
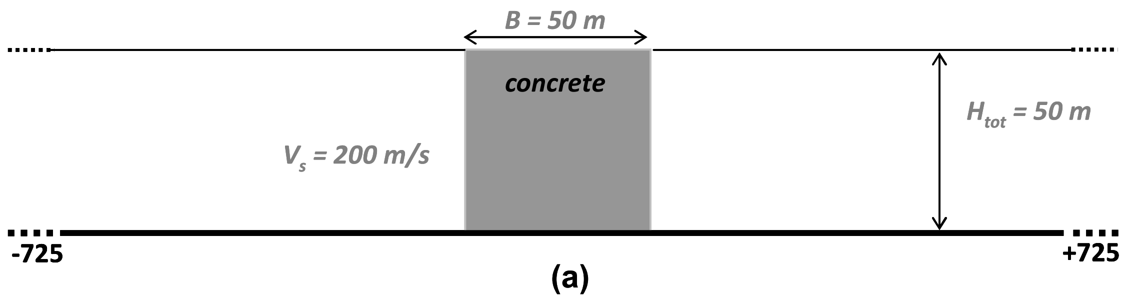


Figure 4. (a) Simplified layout used to simulate soil–foundation interaction concerning the Calabrian Tower foundation, (b) evaluation of the influence of the width of the grouted area, and (c) evaluation of

the influence of the depth of the grouted area.

To derive deeper insights, three other idealized models are also considered: (i) a model of total width equal to the foundation, to ignore the kinematic response of the neighboring soil layer (the boundaries coincide with the foundation edges); (ii) the same model, but with the bottom soil layer improved; (iii) a "concrete column" resting on the bedrock, surrounded by non–jet–grouted soil (Figures 5a and 5b, respectively).

The aforementioned soil–foundation systems were subjected to dynamic excitation. Idealized pulses (Ricker wavelets and Tsang–type pulses) and a real earthquake record (the JMA record from Kobe 1995) were applied at bedrock level. Figure 6 illustrates the elastic response spectra of the excitations used in the analyses.



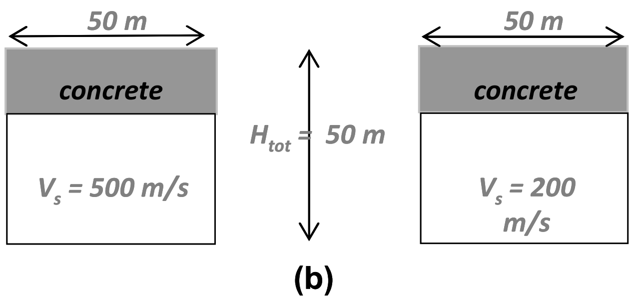


Figure 5. Evaluation of the influence of the grouted area using idealized models : (a) a "concrete column" resting on the bedrock, surrounded by non–jet–grouted soil ; (b) a model of total width equal to the foundation, containing jet–grouted soil, and a model of total width equal to the foundation, containing non–improved soil.

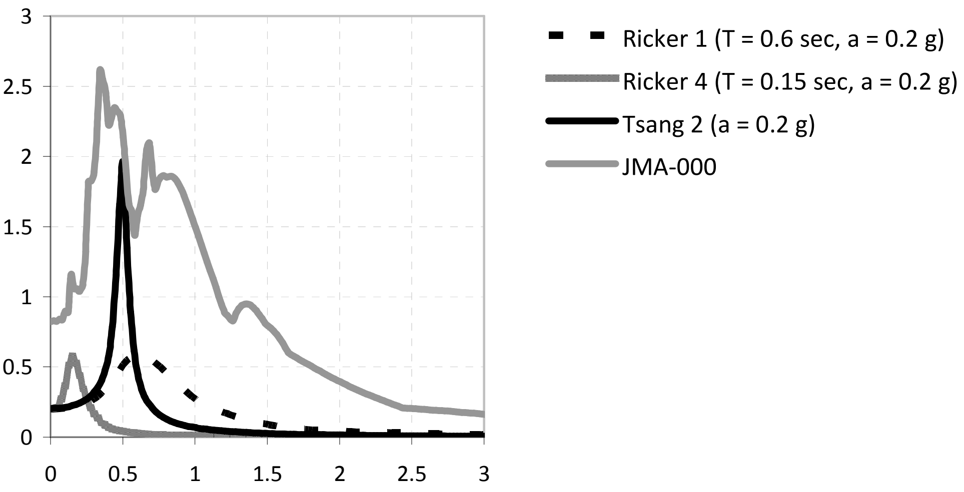


Figure 6. Elastic response spectra of the excitations utilized in the analyses.

From the investigation of the idealized models it is derived that, interestingly, soil improvement leads to a mechanism change in the response of the soil–foundation–pier system. Without improvement, the response is dominated by horizontal translation, resembling the behavior of a shear beam. In contrast, with soil improvement the response of the system is dominated by rocking.

The effect of the width of soil improvement is depicted in Figure 7, where the response of all seven alternatives when subjected to JMA record is illustrated, in terms of horizontal acceleration and displacement (both computed at the pier base), and rotation. It is revealed that rotation is larger for the $B'/B = 1$ model and decreases dramatically when $2 \leq B' \leq \infty$. Moreover, when soil improvement is applied in an area twice the foundation width or wider, horizontal acceleration and displacement reach relatively low values.

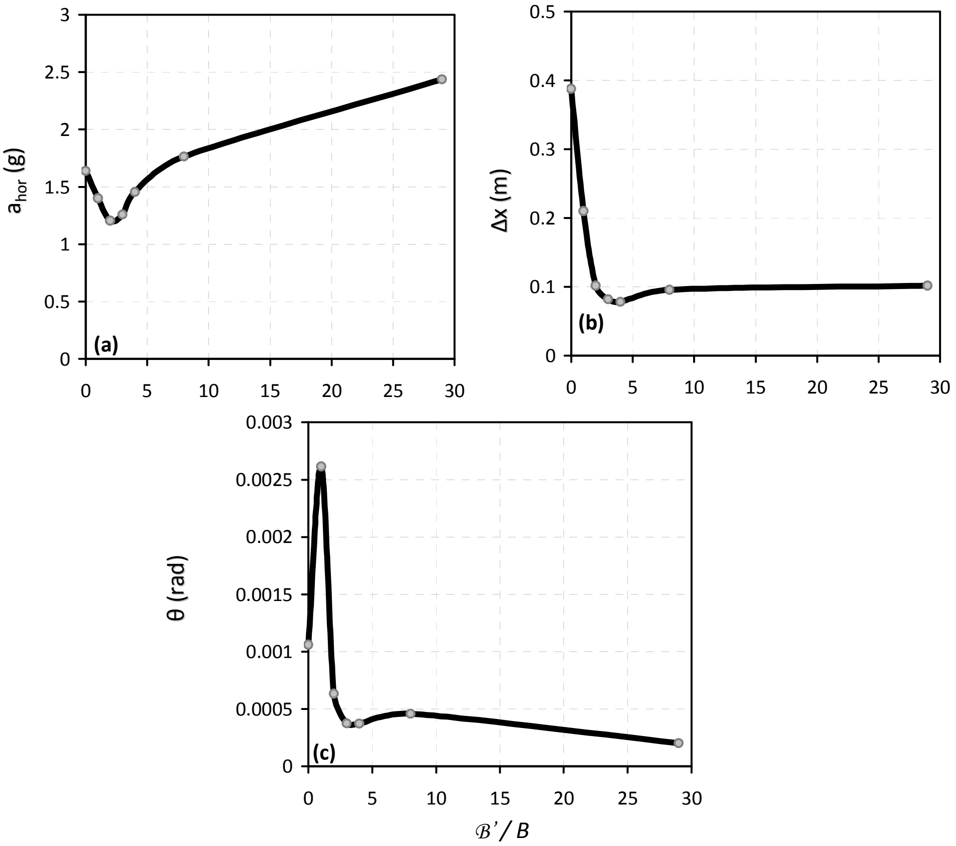


Figure 7. Synopsis of the effect of the width of the grouted area — results for all seven alternatives subjected to the JMA record : (a) peak acceleration at the pier base; (b) peak lateral displacement at the pier base; and (c) peak rotation at the base of the pier, all as a function of B′/ B

The response to the JMA record of the models in which depth of soil improvement is parametrically varied, is illustrated in Figure 8. Apparently, rocking phenomena are more intense when soil improvement ends slightly above the bedrock, i.e. when the flexible caisson–like improved soil column is floating above the bedrock. However, rotational response is not dominant when the grouted area is extended at a depth almost equal to the depth of the embedded foundation.

Based on the aforementioned, a “combination, with jet–grouting to a depth below the foundation base almost equal to the depth of the embedded foundation and a width twice that of the foundation is considered optimum in terms of seismic performance, at least for the cases examined herein. The existence of the improved soil layer at an area of that size, leads to attenuation of horizontal acceleration and displacement, without causing any substantial rotation of the pier.

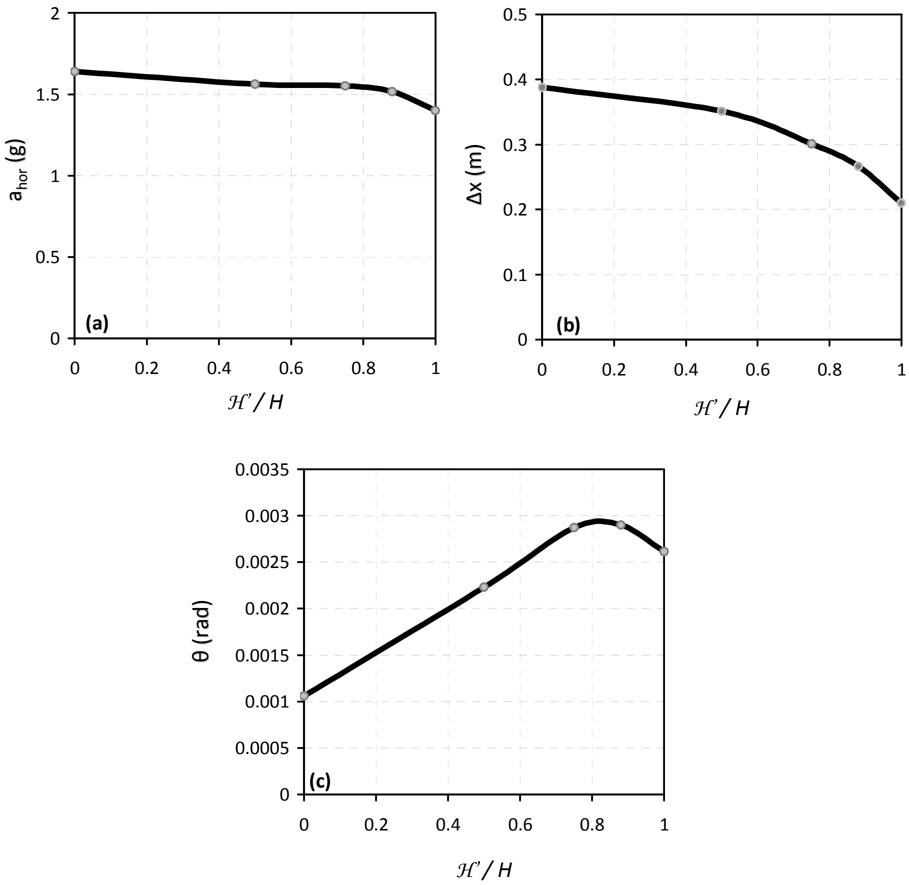


Figure 7. Synopsis of the effect of the depth of the grouted area – results for all five alternatives subjected to the JMA–000 record : (a) peak acceleration at the pier base; (b) peak lateral displacement at the pier base; and (c) peak rotation at the base of the pier.

2. FEASIBILITY OF A LESS CONSERVATIVE FOUNDATION SCHEME

Realistic modeling of soil nonlinearity is mandatory, especially when considering heavily–loaded foundation systems lying on relatively soft soils. Besides, the idea of taking advantage of soil inelasticity in order to limit the structural distress during strong seismic shaking has proven beneficial in a variety of cases. In the framework of this study, we compare the response of the proposed foundation to a less conservative alternative.

A series of two dimensional finite element analyses is performed for the two foundation systems : B = 40 m and H_{emb} = 10 m, B = 50 m and H_{emb} = 17.5 m. A lumped mass structure with an equivalent plane–strain square footing is considered to represent the bridge–pier–foundation system. The action of the cables and the deck upon the bridge tower are replaced by an equivalent vertical force *of* 1 GN acting on the top of the tower. The tower is simulated with linear beam elements, with a distributed mass. The soil and the footing are simulated with quadrilateral 2–D plane strain elements, and the mass of the footing is also considered. The horizontal spring, representing the horizontal constraint at the top of the tower due to the presence of the suspension cables, ke is considered as follows:

$$k_e = \frac{E_c A_c}{L_e} + \frac{E_c A_c}{L_{el}} \tag{1}$$

where A_c , E_c are the area and the Young's modulus of the main cable, respectively, and L_e , L_{el} are the equivalent lengths of the cables (Younis and Gazetas, 1996). Figure 8 illustrates the simplified layout utilized in the analyses, along with the quantities studied herein.

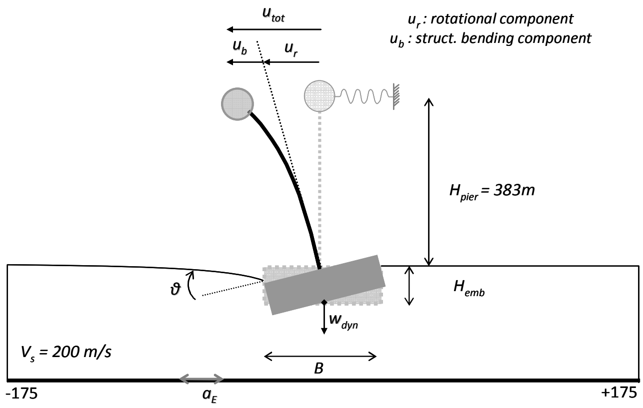


Figure 8. The configuration of the soil–foundation–bridge pier system considered for the non–linear analyses.

1.1 *Constitutive Modelling*

Gap elements simulate the soil–footing interface, allowing for *due consideration of* sliding and separation. The rigid bottom boundary, is placed at a depth of 50 m.

The response of the two systems to static and dynamic loading is investigated in terms of nonlinear behavior of the soil and the superstructure. For all stages of analysis, P–δ (second order) effects are taken into consideration. Nonlinear soil behaviour is modeled with an elasto-plastic constitutive relation with the Von Mises failure criterion, nonnlinear kinematic hardening, and associative plastic flow rule. Specifically, a pseudo–static pushover analysis is first conducted with a Mohr–Coulomb model, and then an equivalent S_u is computed for the Von Mises model (through trial and error method), to match the results. After multiple attempts, two alternatives were calibrated: (i) an equivalent homogeneous–kinematic hardening model, and, (ii) an equivalent layered kinematic hardening model, with S_u linearly increasing with depth. In both cases, the "matching" was performed conservatively (i.e. the ultimate load of the calibrated Von Mises model was lower than the one predicted using the Mohr–Coulomb model).

The aforementioned soil–foundation–bridge pier systems were also subjected to dynamic excitation. Apart from idealized pulses (Ricker pulses and modified Tsang wavelets), and real earthquake records, modified records aiming to match with the design spectrum of the Messina Bridge were also utilized as bedrock seismic excitation to the nonlinear time history analyses. The design spectrum of the Messina Bridge, is the result envelope of several spectra corresponding to real or fitted time histories. The design earthquake of return period 2000 years is considered, with peak ground acceleration in the free field equal to 0.64 g.

Two original records were modified: (i) the Aegion record, from an earthquake of moderate intensity experienced in Greece in 1995, and (ii) the TCU 068–ew record, from an earthquake of high intensity occurred in Taiwan, 1999. The modification was done using the SPEC–PRO–2008 [Gerolymos, 2008]. Specifically, an iterative procedure is adopted in which an input motion is modified, as far as its frequency content is concerned.

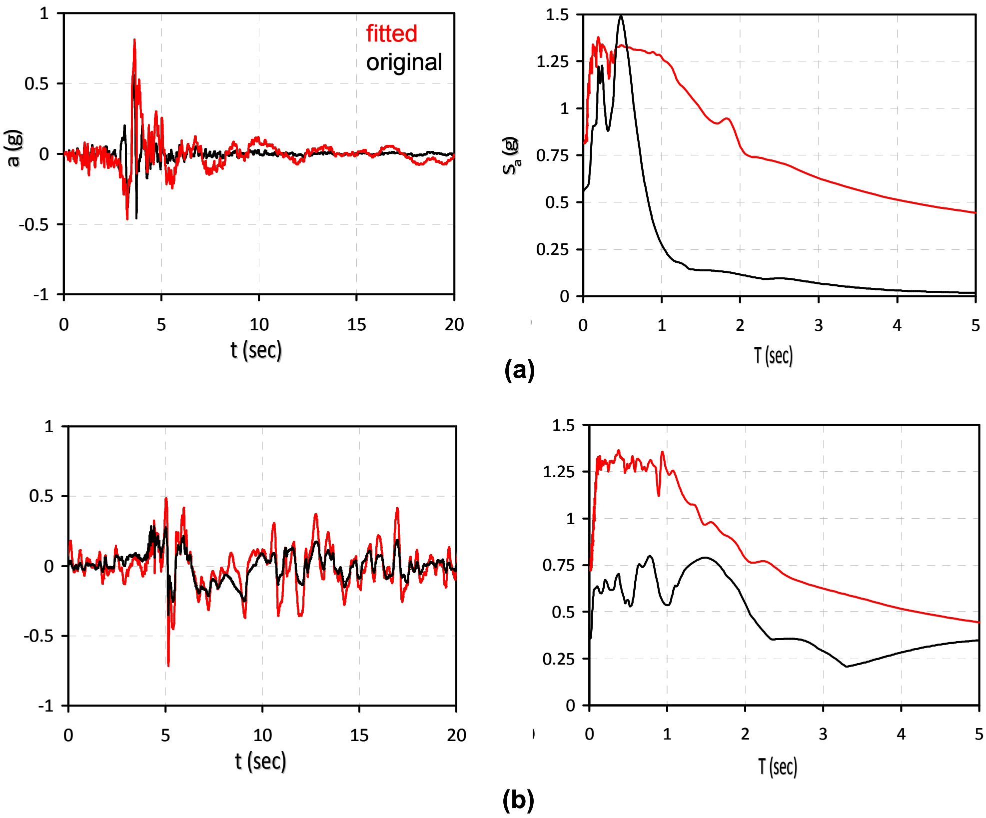


Figure 9. Original accelerogram and modified motion fitted to the target design spectrum, along with their elastic response spectra : (a) Aegion 1995, and (b) TCU 068–ew record from Chi-Chi 1999.

The response of the two alternatives to the two spectrum-compatible motions is depicted in Figure 10 in terms of horizontal acceleration (at the pier base), settlement, rotation (at the pier base) and horizontal displacement at the deck level (horizontal drift). The amplitude of horizontal displacement at the deck level is a matter of great importance for the response of bridges. The total displacement is composed of a rotational component (u_r), which is the result of the rotation of the footing, and a structural bending component (u_b), which is the result of pier bending.

It is revealed that that the dynamic behaviour of the less conservative foundation is congruent with the concept studied herein: taking advantage of soil inelasticity in order to limit the structural distress during strong seismic events. Indeed, the acceleration transmitted to the superstructure is effectively limited, for both excitation cases. However, the increased settlement of the tower is the main (and severe) penalty to pay: settlement roughly reaches 55 cm subjected to the Aegion motion. But notice that the conservatively chosen alternative foundation system chosen, also leads to appreciable rotation, which causes increased horizontal drift at the deck level..

3. TENTATIVE CONCLUSIONS

The main conclusions of this exploratory study can be summarized as follows :

- The effect of grouting area is certainly positive for static problems. However, under dynamic excitation, the presence of improved, and hence stiffer, soil may lead to amplification of the input motion, worsening the seismic response of the soil-foundation system. Interestingly, soil improvement also leads to a mechanism change in the response of the soil–foundation–pier system. Without improvement, the response is dominated by horizontal translation, resembling the behavior of a shear beam. In contrast, with soil improvement the response of the system is dominated by rocking. This phenomenon is more intense when the improvement ends slightly above the bedrock, i.e. when the flexible caisson–like improved soil column is floating above the bedrock.

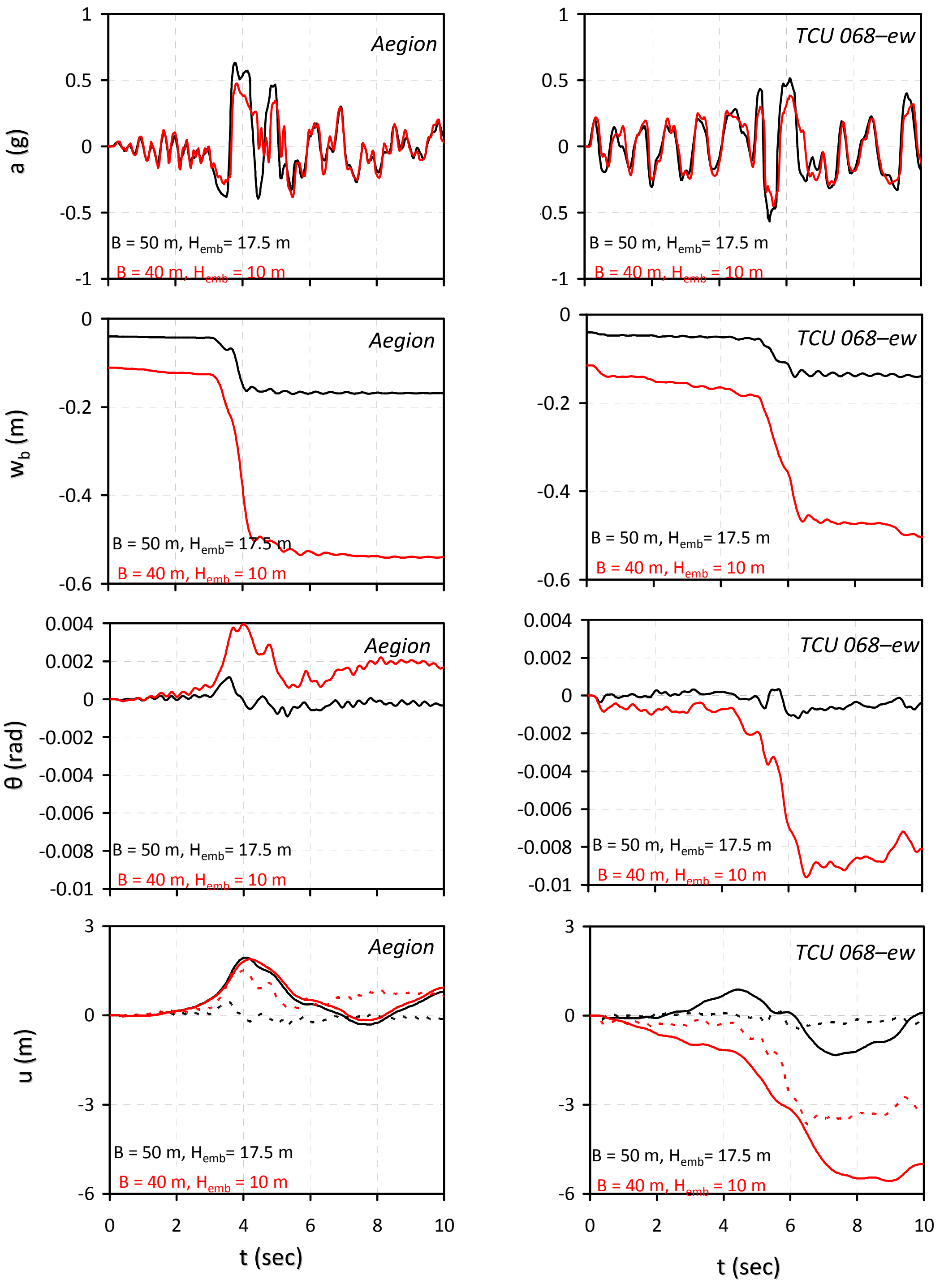


Figure 10. Dynamic analysis of the two alternative foundations subjected to the modified Aegion and TCU 068–ew motions : acceleration at the pier base a, foundation settlement w_b , rotation θ at the base of the tower, and drift u at the top of the tower.

- Rotational response is not dominant when the grouted area is extended at a depth almost equal to the depth of the embedded foundation and at a width twice as much. Besides, the existence of the improved–soil layer leads to attenuation of horizontal acceleration and displacement. Thus, this combination is considered optimum in terms of seismic performance, at least for the cases examined herein.

- Realistic modelling of soil inelasticity the structural distress during strong seismic shaking is reduced. This concept is investigated in this study, comparing the response of the conservative foundation to an under–designed alternative of smaller dimensions. It is shown that the acceleration transmitted onto the Tower is effectively limited due to mobilization of mechanisms. However, the increase of foundation settlement along with substantial increase of rotation and horizontal displacement is a serious disadvantage.

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