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LIQUEFACTION RISK ASSESSMENT AND DESIGN OF PILE FOUNDATIONS FOR HIGHWAY BRIDGE

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SUMMARY

A major 1 km long bridge over Nestos river, currently under construction, is founded on fluvial deposits containing potentially liquefiable clean sand and silty sand in a medium-dense to loose state, for a total thickness of about 12 m. Geotechnical reconnaissance involved a considerable number of geotechnical boreholes, CPT, SPT, Crosshole, and laboratory tests. The total thickness of soil deposits is about 50 m and is underlain by good quality gneiss. Three different 'design' profiles are assigned but only one of them is used in this paper for 1–D equivalent–linear and nonlinear seismic response analyses. Nine actual accelerograms, significantly varying in frequency content define the base-rock input motion. Then, current state of practice of semi-empirical methodologies based on SPT, CPT and V_S profiles, are applied to access the safety factor against liquefaction, and are compared with results of theoretical 1–D nonlinear effective stress analyses. The consequences of liquefaction on foundation piles are determined by adopting a methodology inspired by the recent Japanese experience, which calls for three distinct and successive stages: "before", "during", and "after" liquefaction, including lateral spreading, in a simplified way. Kinematic soil-pile interaction and inertia soil-pile-structure interaction are considered. Specific countermeasures are proposed, such as construction of jet-grouted piles suitably arranged between the piles of some of the bridge piers.

INTRODUCTION

The examined case study refers to a major highway bridge of almost 1 km long, located at the Northeastern part of Greece. The region where the examined bridge is located is almost flat with a light bilateral inclination ranging from 0.5% to 1.5% towards Nestos riverbed. The above highway bridge consists of two independent branches. The total length of each branch is of 952 m and it consists of

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twenty spans, with the central spans being 65 m, 120 m, and 65 m long. A section of the central part of the bridge over the riverbed is sketched in Figure 1.

The subsoil consists of clean and silty sands, of loose to medium density, with thin layers or interlayers of sand-gravel mixture or silty–sandy clays, overlaying a good quality gneissic bedrock. It is prone to liquefaction under strong seismic motion. An integrated liquefaction risk analysis based on semi-empirical methodologies (CPT, SPT, and V_S), and also on theoretical results from 1–D equivalent linear and non linear effective stress analyses in an attempt to account for seismic site response. A cautious combination of theoretical and semi-empirical results allows the assignment of a well determined liquefiable zone with a high degree of confidence.

As for the consequences of the liquefied zone on foundation piles, according to the current state of art and practice, it is widely accepted that a 'universal' accurate solution, referring to seismic motion of a complex system, such as: soil – pile group – pier – bridge under conditions of liquefaction and lateral spreading, is rather unlikely to exist. However, due to Kobe earthquake (1995) experience, simplified methods of analyses have been proposed in the framework of recent Japanese Earthquake Resistant Design Codes [1]. A characteristic feature of the above simplified analyses is dissociation of pre- and post – liquefaction stages.

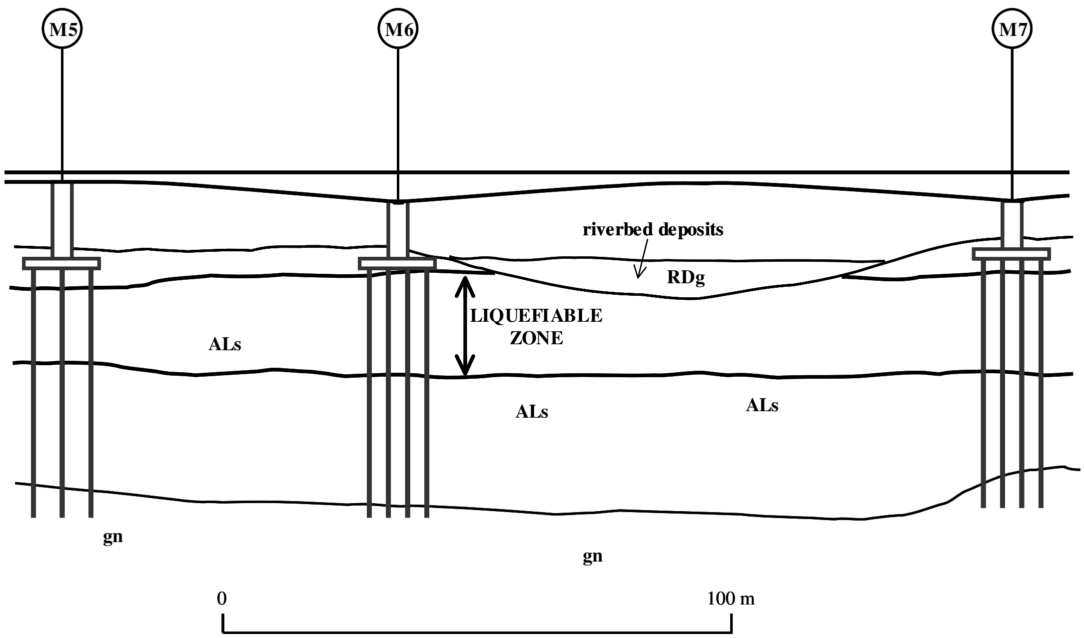


Figure 1. A section of the bridge including the 120 m long, central span over the riverbed.

SITE CHARACTERIZATION AND LIQUEFACTION SUSCEPTIBILITY

The general geology of the broad area consists of Pleistocenic sediments (gravel, coarse sand and moraine) and semi-consolidated pleistocenic sediments (cemented gravel and sands).

At the study area where the bridge is scheduled to be founded, soil materials are distinguished as follows:

- River deposits (RD) of river Nestos consisting of sands, silty and clayey –silty sands, gravels and occasionally pebbles of gneiss or marble.
- Alluvial deposits (AD) essentially of sandy nature, with fluctuating percentage of clays, silts and gravels. Occasionally, thin layers of gravels and clays / silts are encountered. According to geotechnical investigation programs carried out for this area, alluvial deposits are of considerable thickness ranging from 12 to almost 55m. Those layers are relatively loose and considerably uneven, and in some cases an increased percentage of organics has been detected.

The bedrock consists mainly of biotitic gneiss (gn) locally interpolated by amphibolites and marbles of gray – green colour. Sedimentary layers become thicker and more cohesive or dense with depth. Free ground water level is normally located at a depth ranging from 1.5 to 6.0m below ground surface and the pore water pressure is hydrostatic from this level.

Based on geotechnical investigation the under study area, of almost 1 Km length, can be divided into three major regions, if physical, mechanical and dynamic characteristics of different soil layers are carefully combined. In the present work the most representative profile I, in terms of liquefaction ‘susceptibility’, corresponding to the central spans of the bridge is used for seismic analysis purposes and is presented in Figure 2.

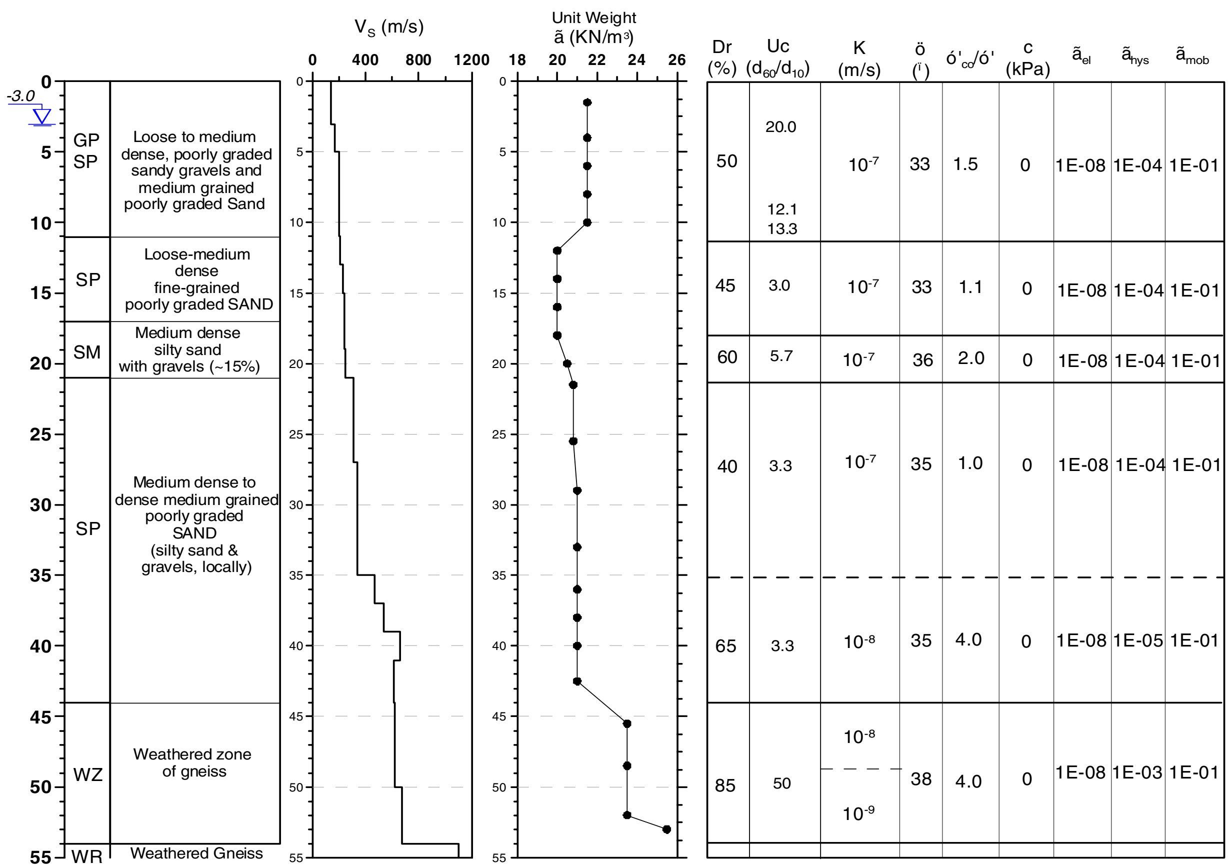


Figure 2. Representative soil profile of the central area (soil profile I) and dynamic properties used in effective stress seismic analyses.

A critical step to start with, is the assignment of soil layers liquefaction ‘susceptibility’, by implementation of compositional criteria relevant to soil texture, sieve analysis, grain size and physical characteristics.

Criteria described as above result from laboratory tests and observations on liquefied soils. Limits of sandy soils prone to liquefy according to uniformity coefficient (U_C), are proposed in Technical Standards for Port and Harbour facilities in Japan [2]. In case of increased percentage of gravels or fines, implementation of additional criteria referring to plasticity and water content are applied in order to quantify liquefaction 'susceptibility' according to Wang [3], Finn [4] and Andrews [5].

SITE RESPONSE AND LIQUEFACTION RISK ASSESSMENT

Nine accelerograms have been considered in this project to describe the excitation at rock outcrop. Based on seismic hazard analysis for the area under consideration, the peak ground horizontal acceleration at outcropping bedrock conditions was at 0.26g.

Site response analyses were performed using *total* and *effective* stress analyses. Total stress analyses were performed with SHAKE [6]. In this paper site response analyses are restricted only to the central area of the project, as being the most representative and appropriate for liquefaction risk assessment. Shear modulus ratio, G/G_{max}, and damping ratio, D_S, curves as function of shear strain, were obtain from the literature [7], [8], [9,10], [11], [12]), as shown in Figure 3. Non-Linear analyses [13], and effective stress analyses using elastoplastic multimechanism model [14], [15], [16,17] and the reduced-order bounding hypoplasticity model [18, 19] were also performed.

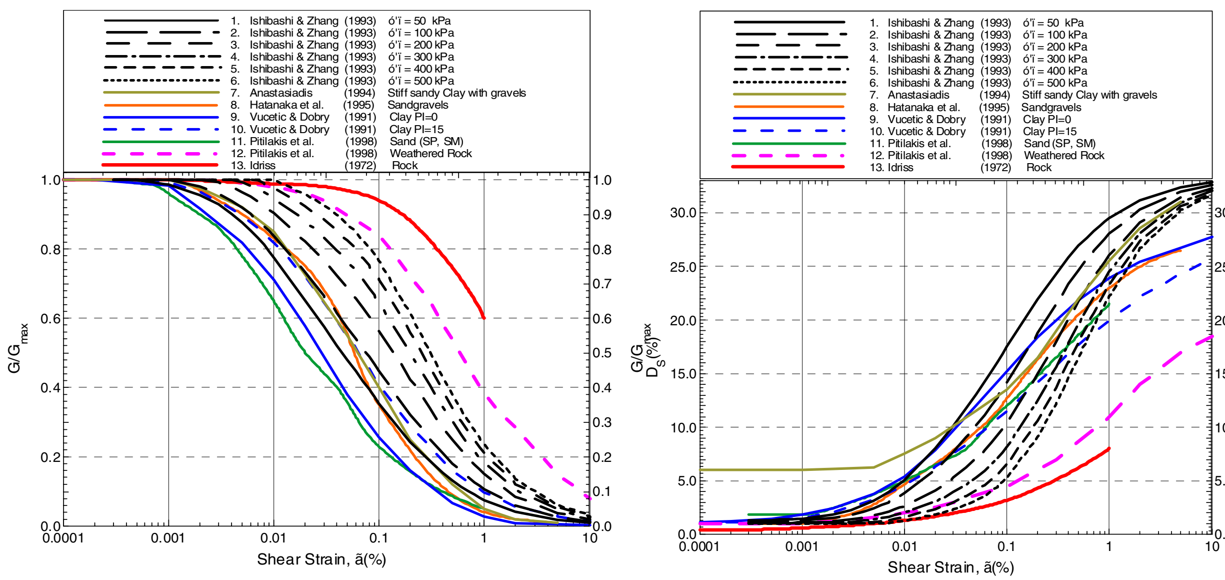


Figure 3. Variation of shear modulus (G/G_{max}) and damping ratio (D_S) with shear strain (γ%): curves from literature used in seismic response analyses (equivalent linear).

Comparison of peak acceleration, shear strain and stress values obtained from total and effective stress analyses is portrayed in Figure 4. For the majority of input motion accelerograms, shear strains calculated with equivalent–linear methodology appear to be rather unrealistically high. On the other hand, some of the inelastic methods of analysis may be somewhat "unsafe" in leading to reduced amplitudes of acceleration. It is believed that reality lies somewhere in between.

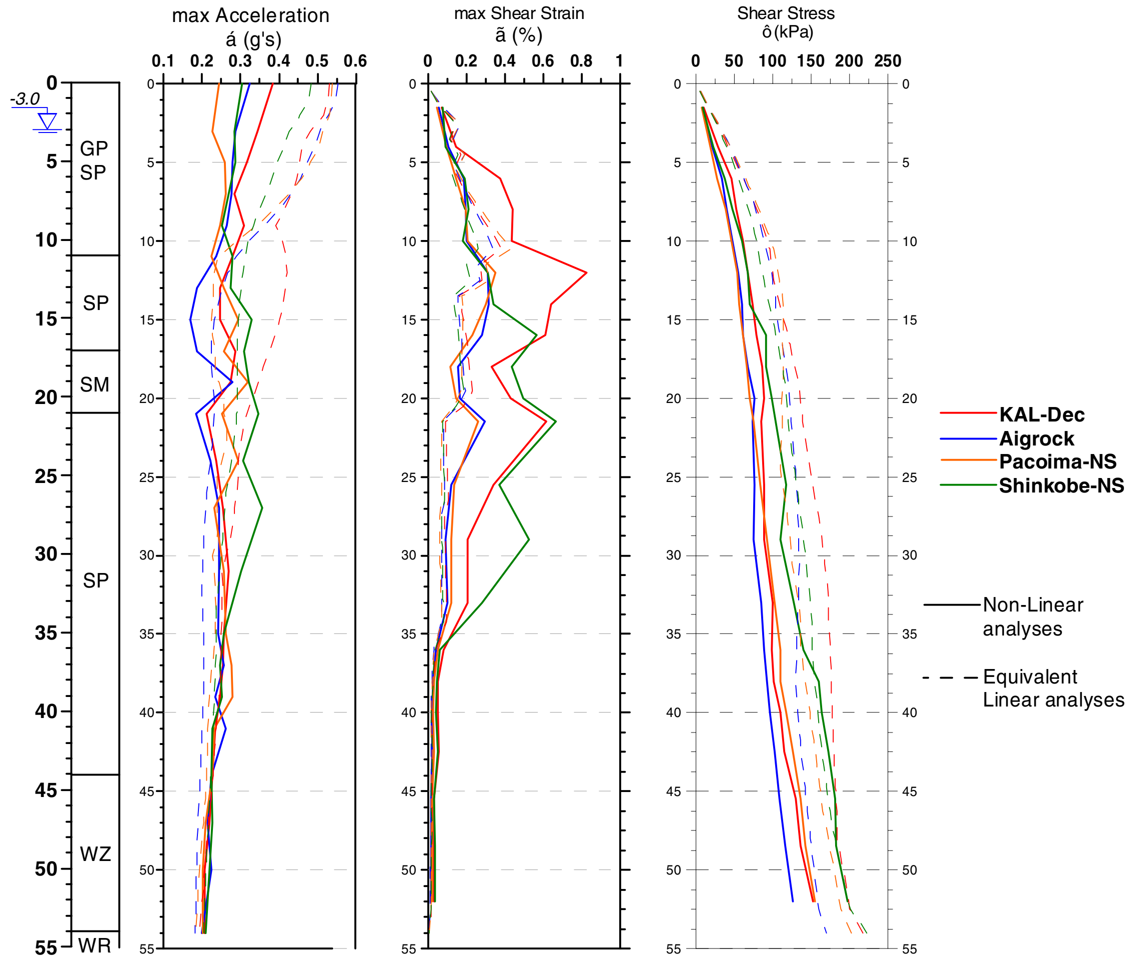


Figure 4. Soil Profile I - Peak accelerations, shear strains and stresses : confrontation of results from non-linear (solid lines) and equivalent linear (dashed lines) analyses with 4 input motions.

Highly non-linear behavior of loose clean-sandy layers is depicted in calculated excess pore water pressure ratio time histories (Figure 5a) indicating high liquefaction risk. Maximum pore water pressure variation with depth is shown in Figure 5b. It is easily shown that the zone of high pore water pressure in excess of 80% extends from 5 to 16m below free ground surface and also another zone is delimited between 22 and 32m. It is then theoretically concluded that liquefaction risk is very high at the aforementioned depths. Despite theoretical results for the deeper zone, we cannot convincingly argue that liquefaction will occur for depths exceeding 25 m to 30 m since almost no evidence of liquefaction at such depths exists.
Response spectra resulting from total and effective stress analyses are presented in Figure 6 and exhibit relatively high values at periods between 0.2 and 0.8 sec and to some extent up to 1.2 sec.

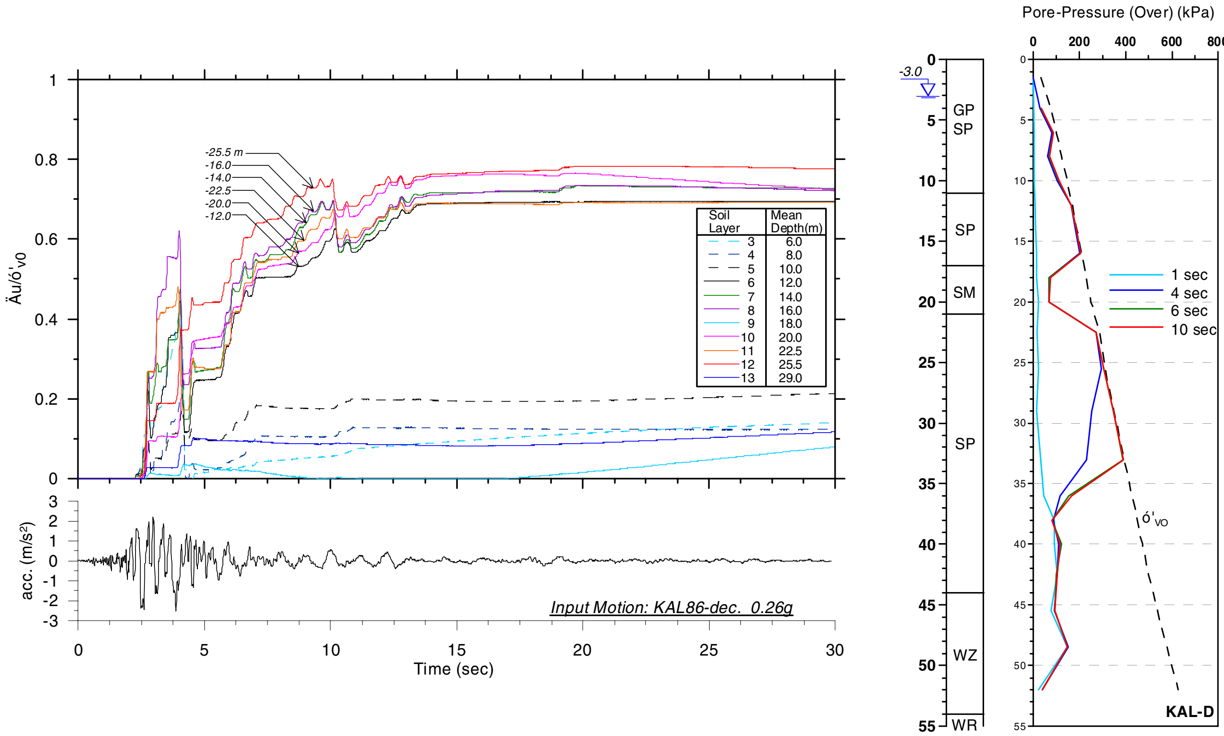


Figure 5. Soil Profile I - Effective stress analysis: (a) Calculated excess pore water pressure ratio time histories at various depths and (b) maximum pore water pressure variation with depth.

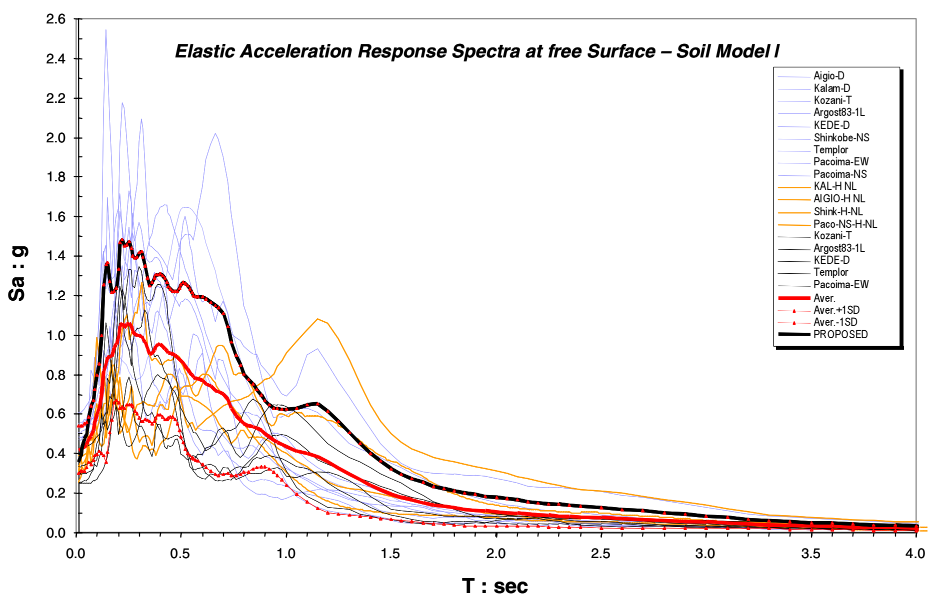


Figure 6. Elastic acceleration response spectra at free ground surface (Soil Profile I): synthesis of total and effective stress analyses results.

Mean elastic acceleration response spectrum one standard deviation (m $\pm\sigma$) is also included in Figure 6. The design peak ground horizontal acceleration adopted for this case study is 0.36g, mainly based on theoretical results of non-linear and effective stress analyses.

Evaluation of liquefaction risk was based on results of theoretical analyses and selected semi-empirical methodologies, with emphasis on "stress–based" procedures. More specifically, the amplitude of earthquake-induced demand (quantified by cyclic stress ratio parameter – CSR) was determined using simplified relation-ships and peak ground horizontal "design" acceleration values, and also from results of theoretical analyses. The soil strength against liquefaction (quantified as a function of the cyclic resistance ratio – CRR) was determined by empirical relationships provided in EC8 [12], NCEER 97 13] based on field data such as: SPT, CPT and V_s. Figures 7 and 8, based on a characteristic borehole and static penetrometer borehole, show synthetical results of semi-empirical methodologies applied to assess factors of safety against liquefaction (FS_L) and an accurate, as possible, description and evaluation of liquefaction risk.

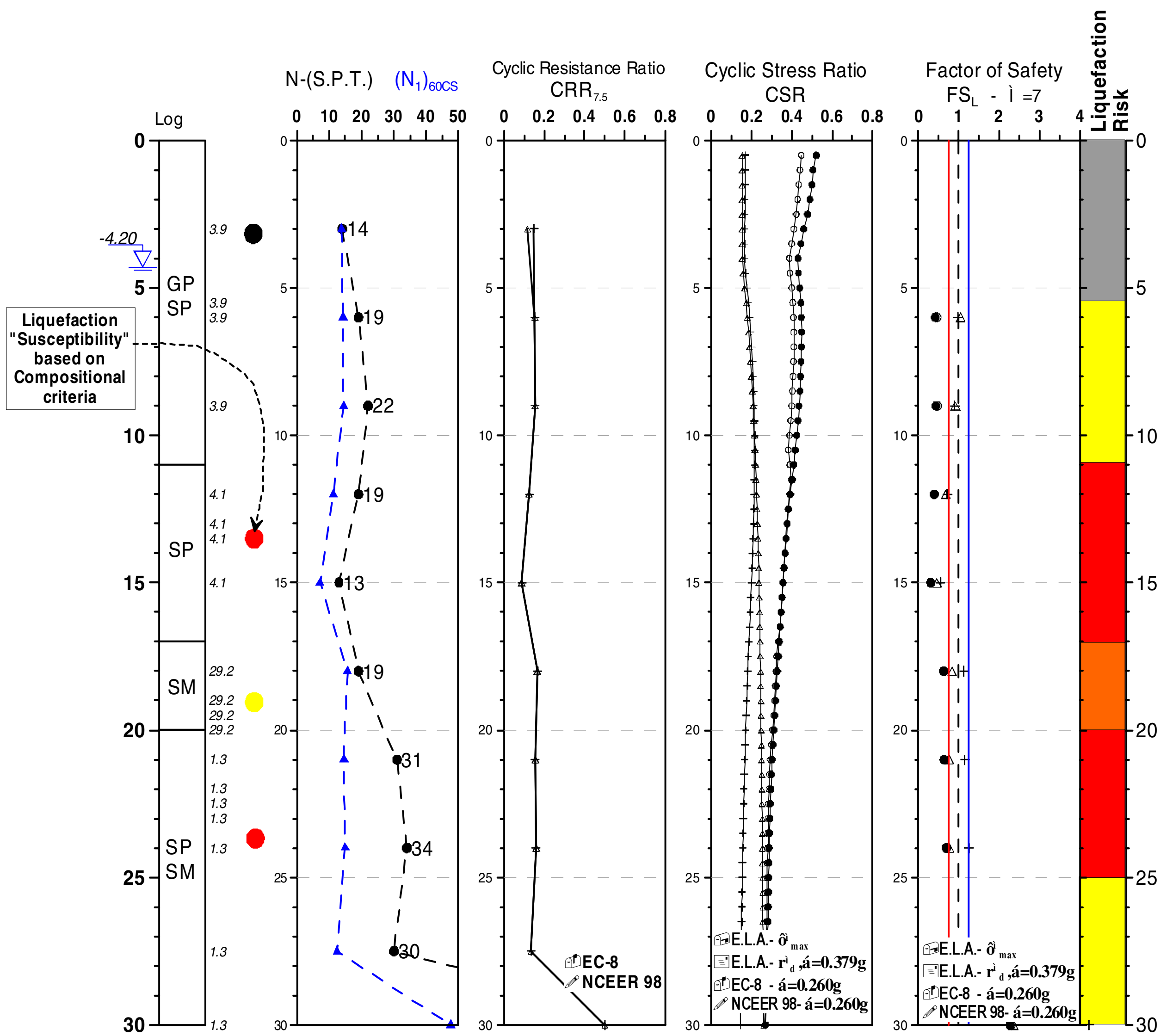


Figure 7. Implementation of various methodologies for estimating cyclic resistance ratio (CSR) and safety of factor against liquefaction (FS_L) from SPT tests.

The calculated factors of safety against liquefaction, based on SPT values, specify zones presenting a high risk to liquefy. For the above borehole the high risk zone for liquefaction to occur ranges from 11 to 25 m of depth. The estimated risk is slightly reduced between 17 and 20 m. The bullets of red, yellow and black color are indicative of liquefaction 'susceptibility' (Red color: high susceptibility, yellow color: medium 'susceptibility', green color: small 'susceptibility' and black color: refers to soil sample not prone to liquefy).

The CPT based factors of safety against liquefaction delimit a more extended zone of medium high to high risk of liquefaction from 7 to 25 m. They differ slightly from the SPT based factor. Those differences are deemed justified, since SPT measurements are relatively sparsed (every 2 to 2.5m), compared to CPT values registered every 0.2m, as shown in Figure 8.

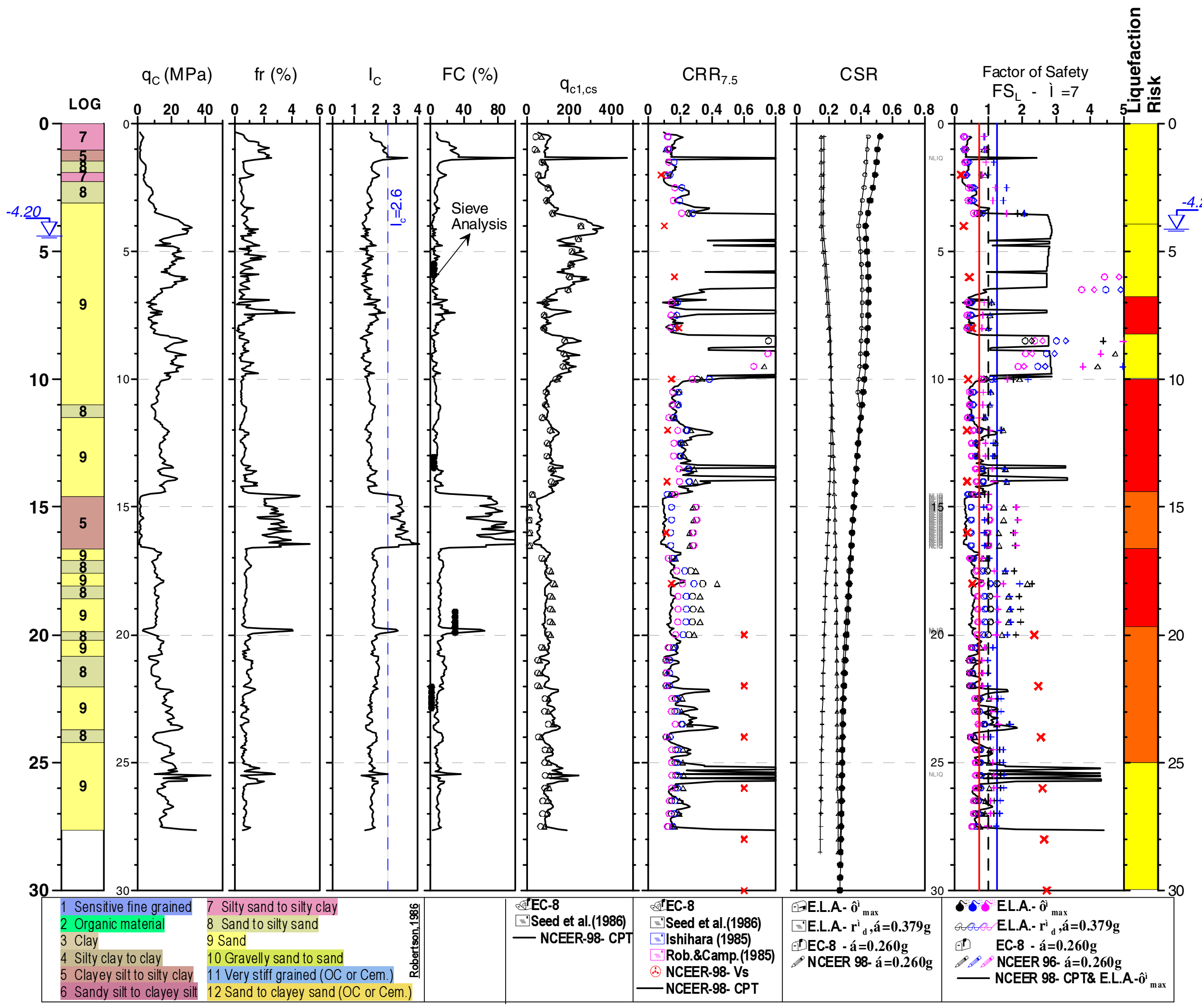
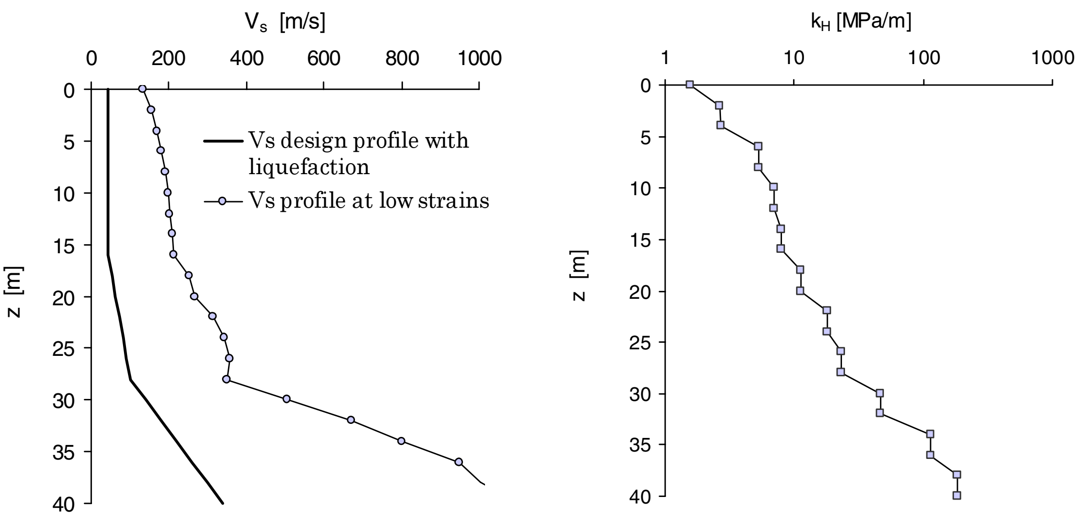


Figure 8. Implementation of various methodologies for estimating cyclic resistance ratio (CSR) and factor of safety against liquefaction (FS_L) from CPT and Cross-Hole tests.

The kinematic response of the soil-pile system after excess pore water pressures arise is subdivided into two distinct "stages" of response:

- Stage A, when considerable pore water pressures have developed ($\Delta u > 0.50\,\sigma'_{vo}$) but no widespread liquefaction has yet occurred. At this stage, the soil reaction to pile is drastically reduced, but it still preserves a non-zero shear stiffness so that seismic waves can propagate through and affect the pile. Thus, the nature of pile response is still predominantly dynamic but is also significantly different from a linear or moderately nonlinear behavior.
- Stage B, which takes place after the earthquake loading has ended. Now, "lateral spreading" of the liquefied soil may take place under certain conditions, such as an inclination of the soil layers or the surface resulting in a quasi-static, gravity-induced loading to the pile. Static analysis can be performed to estimate the pile response by imposing a displacement loading along the supports of springs connecting soil and pile; no inertial loading from the superstructure applies.

The analysis of the soil-pile system in the realm of a limited pore water pressure build-up (stage A) is based on the procedure adopted by the conventional soil-pile interaction analysis. However, in this case quite "softer" Winkler soil springs along the pile account for the soil nonlinearities arising in the vicinity of the pile. In the Nestos Bridge case, the calibrated distributed springs are portrayed in Figure 9.



(a) shear wave velocity **(b) Winkler soil springs**

Figure 9. Shear wave velocity and winkler springs distribution with depth for the soil-pile interaction analysis. The V_s profile at low strains is applied for the equivalent non-linear case where no liquefaction is accounted for.

The numerical analysis of the pile response under earthquake motion is performed through the semi-analytic algorithm SPIAB (Soil-Pile Interaction analysis for bridge piers, Mylonakis and Gazetas, Gerolymos et al 1999). At this stage, inertia loading to pile is negligible due to strongly reduced acceleration levels, thus a kinematic interaction analysis is adequate to predict the total system response.

The time history of the Pacoima dam recorded in the Northridge 1994 earthquake is used as excitation at outcropping bedrock, after scaling down to 0.26 g to be compatible to the Greek code provisions (see Figure 10).

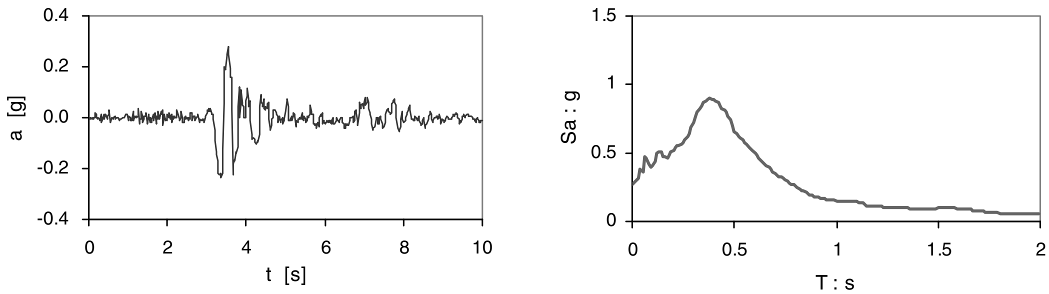


Figure 10. The acceleration time-history and response spectrum at 5% damping ratio of the scaled Pacoima dam record.

The results are plotted in Figure 11 for a 40 m long pile in terms of bending moment and shear force distributions. The results of the "moderately nonlinear case" which does not account for any pore-pressure built-up are also plotted in this figure. It is clear that the slightly liquefied soil has a strongly detrimental effect on the response. The magnitude of the bending moment below the pile cap (z = 0) increases to 2800 kNm, which is nearly double the "no-liquefaction" value. Furthermore, the shear force is even more amplified reaching about 480 kN at the depth of 10 m. The detrimental effect of pore-pressure development is mainly the outcome of strongly amplified free-field shear deformations due to soil softening; this overshadows the beneficial reduction of Winkler spring stiffness.

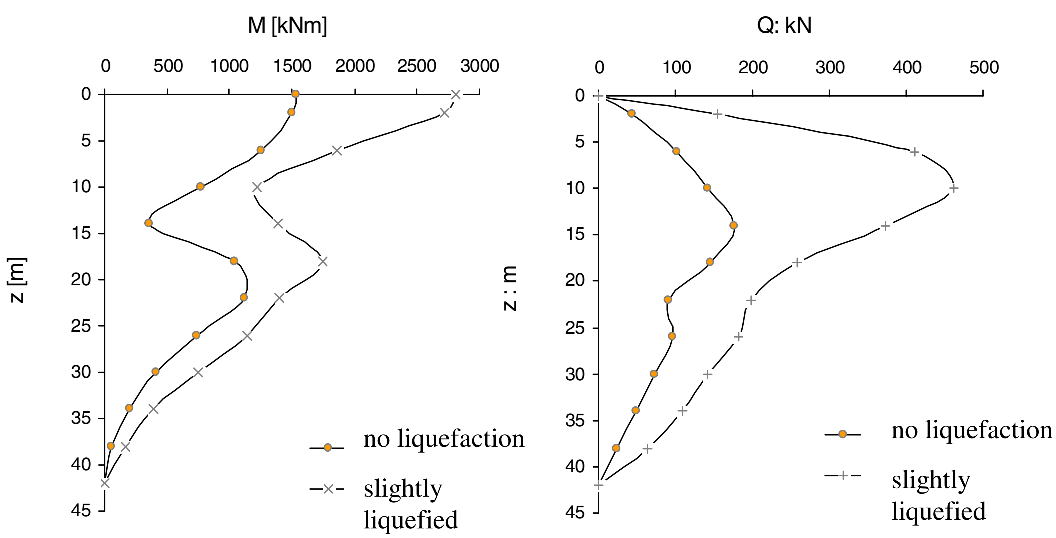


Figure 11. Bending moments and shear forces along the pile for the case of slightly liquefiable soil in comparison with the moderately nonlinear response without pore-pressures built up.

The lateral spreading potential of the liquefiable soil is of rather minor importance, regarding that the liquefiable layers and ground surface extend almost horizontally. A possible exception to the general case involves the two piers M6 and M7 of the central span (see Figure 1) where a slight increasing of ground surface inclination is possible due to a future erosion of the riverbed deposits. For these piers, specific countermeasures are proposed to prevent any additional loading to the piles after lateral spreading onset.

Such measures include the construction of 0.8 m diameter jet-grouted piles suitably arranged between the piles. The configuration of ground improvement for the pier M6 case is sketched in Figure 12.

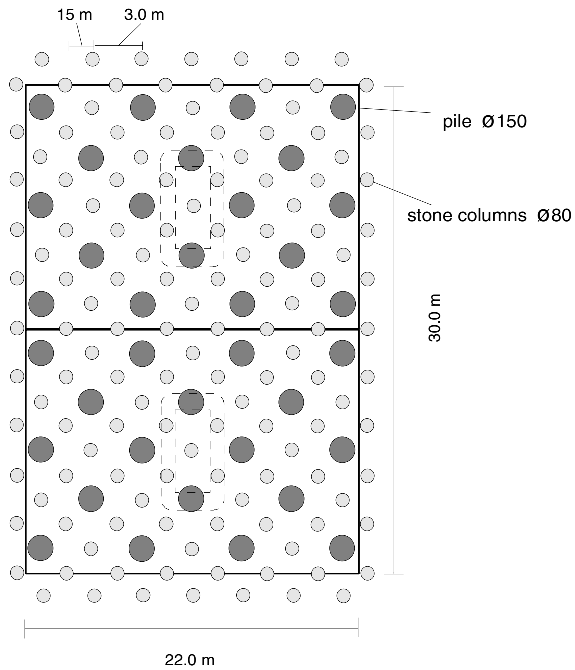


Figure 12. Configuration of the stone columns for the pier M6 case.

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