Bridge foundation on very soft alluvia with stone column ground improvement

Fondation de pont sur alluvions très mous et amélioration du sol avec des colonnes ballastées

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ABSTRACT: The present paper proposes technical solutions for a road design project comprising both bridges and high embankments in the Region of Western Greece, where major geotechnical issues had to be dealt with. The very low P-y reaction of the soft silty clays and the eventual liquefaction of the silty sand layers embedded within the foundation soil, the high ground water table and the high seismicity of the area, led to the decision to improve the top part of the natural soil given the necessity for an acceptable solution in terms of both dimensions and cost. Among other possible methods of soil improvement, the application of stone columns followed by preloading was selected. A comparative parametric stability analysis of the bridge embankments and the pile foundations for bridge piers, with or without the presence of stone columns, quantified the benefits from the proposed ground improvement method and verified that the completion of this project is feasible within acceptable performance, safety and cost limits.

RÉSUMÉ : La communication propose des solutions techniques pour l'élaboration d'un projet de route qui comprend des ponts et des remblais de grande hauteur dans la région de la Grèce occidentale où il a fallu faire face à des problèmes géotechniques importants. La très faible résistance des argiles limoneuses molles aux sollicitations horizontales et la liquéfaction éventuelle des couches du terrain formées de sables limoneux qui sont contenues dans le sol de la fondation, la nappe phréatique élevée et la haute séismicité de la région, ont conduit à la décision d'améliorer la partie supérieure du sol naturel en prenant en considération la nécessité de trouver une solution acceptable en ce qui concerne les dimensions et le coût. Parmi d'autres méthodes d'amélioration du sol, il a été choisie l'utilisation des colonnes ballastées suivie d'un préchargement du sol. Une analyse paramétrique comparative de stabilité des remblais des ponts et des fondations des piliers des ponts avec ou sans la présence des colonnes ballastées, quantifient les bénéfices obtenus par l'utilisation de la méthode d'amélioration du sol proposée et vérifie que l'achèvement de ce projet est réalisable avec une performance acceptable en termes de sécurité et de coûts

KEYWORDS: road project, bridge foundation, soft alluvia, liquefaction, ground improvement, stone column, preloading.

1 INTRODUCTION

A significant project for road infrastructure is currently under way in western Greece, perfecture of Aitoloakarnania, concerning the construction of a 13,1km part of a public provincial road connecting the municipality of Astakos to the bridge of Gouria.

Owner of the project is the Greek State and the Supervising Authority is the Directorate of Studies for Road Works, General Directorate of Road Works, Ministry of Development, Competitiveness, Infrastructure, Transport and Networks. Following the necessary competitive procedure, the design of the project was assigned to a joint scheme of specialized design offices, covering the involved scientific areas.

This paper focuses on the technical solutions proposed for the geotechnical issues that arose with reference to the stability of embankments and bridge foundation.

2 PROJECT OVERVIEW

The importance of this project lies in its expected contribution to the improvement of road access towards western Aitoloakarnania and mainly the touristic zone of Astakos-Mytikas-Palairos. It is also anticipated to take over some of the traffic load of other local axis and to support the increase in use of an existing tunnel nearby. What is more important though, is the expected traffic load assumption for the shipbuilding and industrial zone of Astakos, which in the future will be the base for development in the whole area.

The realization of the project will improve the accessibility of the area and will facilitate road connection between cities and existing or planned infrastructure, decreasing time demands and improving safety and comfort requirements

This road axis under study forms a part of the connection of Astakos and the port of Platygiali with the major motorway of "Ionia Odos", passing through the bridge of Gouria and the existing tunnel of Saint Elias. The road section is 11,0m wide (1 lane per direction). From geotechnical point of view, it is to be mentioned that the whole project comprises 6 bridges (15-105m long) and a significant length of embankments between 2 and 7 meters high.

Major geological and geotechnical issues that arise for the last 10km of the road are related to the very low altitude of the ground and the lack of inclination, the high ground water table, the insufficient drainage system and the presence of silty clays and sands, often with high content of organics. The whole situation is aggravated by the liquefaction potential of the silty sand layers embedded within the foundation soil, in connection with the high seismicity of the area.

During the preliminary design stages, it became obvious that the most significant geotechnical problems for the realization of the project would be related to the load bearing capacity of the soil, the expected subsidence under static loading and the eventual liquefaction phenomena.

3 GEOTECHNICAL CONDITIONS

The area where the bridge foundation will be constructed consists of soft and compressible saturated alluvial soils, while the water table is located at ground level. The prevailing geotechnical conditions at these areas can be simplified in two main profiles.

Soil profile I (Fig. 1) is encountered in the majority of the bridge sites. Its main characteristic is the surficial layer of fine-grained medium plasticity soil. According to the geotechnical exploration results, this soil layer consists mainly of low to medium plasticity silts (ML) and clays (CL), with thin layers of high plasticity silts (MH), fat clays (CH) and organic clays (OL). The thickness of this layer varies between about 22.5 to 35m. Below this layer, to the depth of 40m, either a medium to dense non-cohesive soil unit (SC, SM) or a dense cohesive soil unit (CL) are present. Rock or any other rock-like geological formation was not encountered at any of the locations explored.

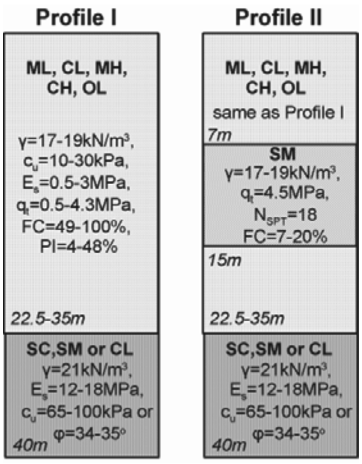

Figure 1: Representative geotechnical profiles

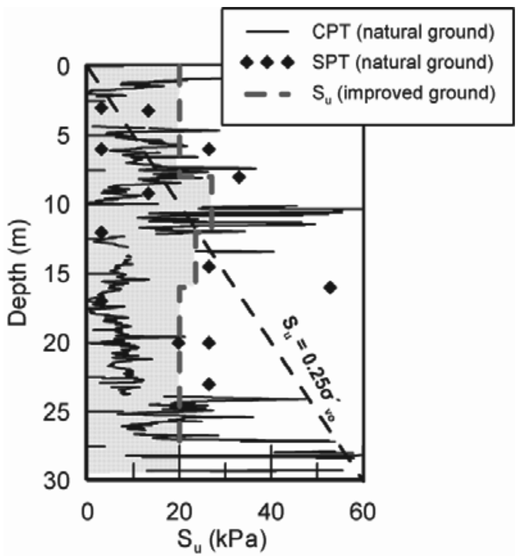

Figure 2: Distribution of undrained shear strength with depth for profile I conditions, before and after the improvement

Figure 2 presents an estimation of the undrained shear strength of the surficial fine-grained soil unit of Profile I, based on the results of typical CPT & SPT recordings. An estimation of undrained shear strength for normally consolidated clays is also presented, based on Jamiolkowski et al. (1985) (see Eq.1):

$$S_u = 0.25\ \sigma'_{vo} \quad (1)$$

where σ'_{vo} is the geostatic effective vertical stress. Comparing these two estimations, it is concluded that the surficial fine-grained layer is normally or even at some depths under-consolidated, with low values of undrained shear strength. Thus, the bearing capacity of this formation is considered low and significant settlements are expected during loading, with the necessary consolidation time to exceed the acceptable time limits (horizontal coefficient of consolidation ranging between c_h=7x10^{-7} – 9x10^{-6} m^2/sec based on CPTu dissipation tests). The lateral resistance of this layer is also considered very low, leading to large horizontal displacements and structural forces, especially during seismic loading.

With regard to the seismic response, profile I belongs to group type S1 according to EC8. The average shear wave velocity $V_{s,30}$ generally ranges between 85 and 140m/sec, as computed from the CPT recordings:

$$G_{max,o} = (q_t - \sigma_v)x0.0188x10^{0.55Ic+1.68} \quad (2)$$

where I_c is a soil behavior type index (Robertson, 2009). Thus, special study is required for the definition of the seismic action, which will take into account the non-linear response of the soil layers and the dependence of soil moduli and internal damping on cyclic strain amplitude.

Profile II (Fig. 1) represents the soil conditions prevailing at one bridge site. The soil conditions resemble those of Profile I, with the exception of an 8m thick layer of loose silty sand that interrupts the surficial fine-grained formation. This non-cohesive formation (SM according to USCS) is relatively close to ground surface (at the depth of 7m), while it is classified as non-plastic, with fines content between 7 and 20% and potentially liquefiable under seismic conditions.

A preliminary liquefaction analysis with NCEER methodology (Youd et al. 2001) for CPT recordings revealed that this non-cohesive formation is liquefiable. As shown in Fig. 4, the factor of safety against liquefaction is well below unity for the silty sand layer, revealing its high liquefaction potential. Hence, although this soil layer presents higher stiffness ($V_{s,30}$=140m/s) and bearing capacity for static loading, as compared to the clay layer, its liquefaction potential deteriorates its mechanical properties. Thus, during earthquake loading, loss of bearing capacity, lateral stiffness degradation and settlements are expected to occur, increasing this way superstructure displacements and structural forces. Furthermore, Profile II is now characterized as Group type S2 according to EC8 and special study is needed to define the seismic action and the exact liquefaction potential.

4 DESIGN CONCEPT

As a result of the existing poor soil conditions, the foundation of the foreseen bridge piers on surface foundations was excluded and was replaced by a group of piles with a rigid pile cap. However, due to the high seismicity of the area, the very low P-y reaction of the soft silty clays and the eventual liquefaction of the silty sand layer led to extreme internal forces of the piles and increased dis-proportionally the cost of the project. Hence, the necessity of an acceptable solution in terms of both dimensions and cost, led to the decision to improve the top part of the natural soil.

Among a number of possible methods of soil improvement that were examined, it was decided to proceed with the application of gravel piles followed by preloading. Plastic drains are also prescribed to act as secondary drainage system for greater soil depths.

The main aim of pre-loading was to increase the undrained shear strength of the surficial fine-grained soil unit. The improved undrained shear strength (when the increase of effective stress due to surcharge exceeded 10% of its initial value), was estimated according to Eq. 4:

$$S_{u,f} = S_{u,o}\ OCR^{0.8} \quad (3)$$

with $S_{u,o}$ reflecting the anticipated undrained shear strength for normally consolidated clays (see Eq. 2). The increase of effective vertical stress at each depth was computed according to the well known Westergaard solutions, taking into account the increase of soil stiffness at upper layers, where gravel pile installation accompanies pre-loading. The effect of pre-loading reduces with depth, while a percentage of the surcharge load is used for the increase of OCR, due to the distribution of the external load between gravel piles and original soil. Despite that, the anticipated increase of undrained shear strength at upper layers (i.e. at layers that are crucial for the overall safety of the bridge embankments) is considered substantial, while its secondary effects such as the acceleration of consolidation at layers that were found under-consolidated and the reduction of downdrag forces at piles (i.e. by allowing the consolidating soil to settle before construction) increase its efficiency. The prescribed pre-loading embankment were wider from the bridge embankment / pile cap by 2.5-3.0m at each side, in order to apply uniform stress at the area of interest, while its height generally varies between 3 and 7m.

Stage construction of pre-loading embankment was decided (with height increments between 1.5-2.0m), due to the poor soil conditions, followed by continuous settlement and pore-pressure dissipation recordings. Figure 2 presents the anticipated final (after improvement) distribution of S_u with depth for the CPT recording presented in Section 3.

Gravel pile installment is prescribed ahead of pre-loading, consisting of 0.80m diameter piles in a 1.80 x 1.80m square arrangement (denoting replacement percentage equal to a_s= 0.78x $(0.8/1.8)^2$ =15.4%). Gravel pile length varies between 8 and 13m, depending on soil conditions.

The installation of gravel piles increased the mechanical properties of the upper cohesive fine-grained layers and subsequently increased the general stability of bridge & pre-loading embankments. The following equivalent strength parameters were used (Van Impe & De Beer, 1983):

$$c_{eq} = (1-a_s)\, S_{u,f} \tag{4a}$$

$$\tan\varphi_{eq} = [n a_s /(n a_s +1 - a_s)]\tan\varphi_1 \tag{4b}$$

where c_{eq} & φ_{eq} denote the equivalent cohesion & friction angle of the composite system respectively, φ_1 denotes the friction angle of gravels (assumed equal to 42°), a_s denotes the replacement ratio (equal to 0.154) and *n* denotes the ratio of the load taken by the gravel pile versus the surcharge load. The contribution of geostatic stresses is omitted; while outside the embankment limits (where no surcharge is applied) *n* equals 1.0. The improved shear strength of the composite system, combined with the increase of the undrained shear strength due to pre-loading proved adequate for the construction of the bridge embankments with acceptable factor of safety under both static and seismic conditions (e.g. the static F.S. increased from 0.64 to 1.51 for a representative height of 4m).

Note that, besides the improvement of shear strength characteristics, the inclusion of gravel columns combined with pre-loading has altered the seismic ground response relative to free-field. In order to take into account this effect, the shear wave velocity and the spring stiffness in P-y curves of the relevant soil layers were appropriately increased. Namely, the formula presented by Baez & Martin (1993) was used for the estimation of the maximum shear modulus of the composite system:

$$G_{max,eq} = G_{max,i}\, a_s + G_{max,p}\, (1-a_s) \tag{5}$$

where $G_{max,eq}$ is the maximum equivalent shear modulus, $G_{max,i}$ is the maximum shear modulus of the fine-grained layer after pre-loading, $G_{max,p}$ is the maximum shear modulus of the gravel pile and a_s is the replacement ratio (here 0.154). The maximum shear modulus of the fine-grained layer after pre-loading was computed as follows (Weiler, 1988):

$$G_{max,i} = G_{max,o}\, OCR^{0.5} \tag{6}$$

where $G_{max,o}$ is the maximum shear modulus of unimproved soil, as computed by Eq. 2. The maximum shear modulus of the gravel pile was computed assuming a dense configuration (e=0.55). Figure 3 presents the shear wave velocity profile of the composite system for the CPT recording of Fig. 1. The average shear wave velocity $V_{s,30}$ for this profile increased from 86 to 140m/s, reflecting stiffer ground conditions. This increase was also implemented to the P-y curves, by increasing the horizontal subgrade reaction coefficient k. The increase was assumed proportional to the ratio $G_{max,eq}/G_{max,o}$, while for the unimproved soil coefficient k was computed according to DIN4014 for bored cast-in-place piles.

For the case of Profile II, where a non-cohesive liquefiable layer is present, the gravel piles are expected to act as a countermeasure against liquefaction. The gravel piles will be constructed via bottom-feed vibro-replacement, while a proper gradation curve range is prescribed in order to ensure the effective drainage of earthquake-induced excess pore-pressures. During vibro-replacement, the non-cohesive layer is expected to be densified and increase its resistance to liquefaction. Based on Mizuno et al. (1987), the average measured tip resistance is expected to increase between gravel piles from 4.5MPa to 9.5MPa, providing an adequate liquefaction resistance. Figure 4 compares results from the preliminary (before improvement) and the detailed (after ground improvement) liquefaction study, which show the minimization of liquefaction potential. The densification of the non-cohesive layer due to pre-loading and the potential dissipation of excess pore pressures were conservatively ignored. It is noted that even if densification was ignored, drainage through gravel piles would retain excess pore pressure ratio r_u well below 0.5, as computed according to Seed & Booker (1977) and Bouckovalas et al. (2011) for the given characteristics and gravel pile geometry.

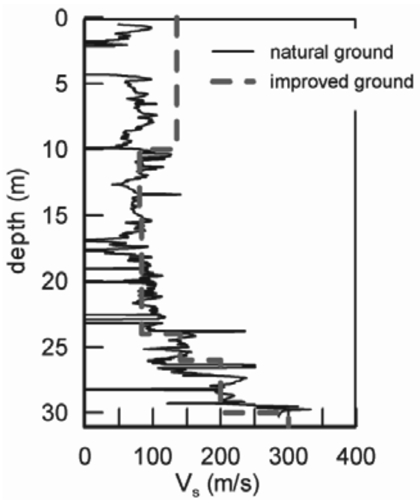

Figure 3: Distribution of shear wave velocity with depth for profile I, before and after the improvement

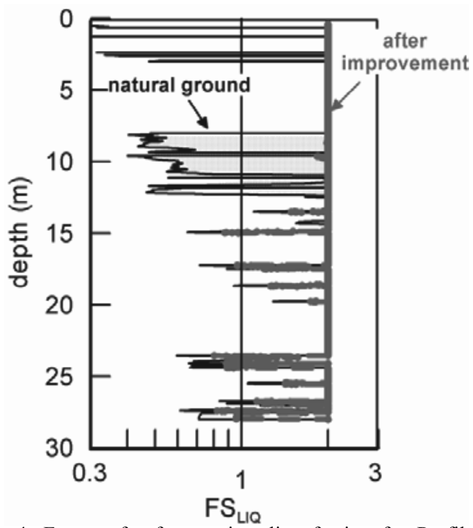

Figure 4: Factor of safety against liquefaction for Profile II, before (preliminary results) and after improvement (detailed study).

Finally, consolidation process is expected to be accelerated with the presence of gravel piles. Excess pore pressures for each loading stage are expected to diminish within 19 days, assuming conservatively only radial flow towards the gravel piles and horizontal coefficient of consolidation equal to c_h=7x10^{-7}m/s^2. The actual consolidation time is expected to be even lower, considering the actual 2D water flow, the presence of horizontal layers of higher permeability and the additional discharge from the secondary pipe drains that are prescribed.

5 SEISMIC GROUND RESPONSE ANALYSES

Besides ground improvement, detailed ground response analyses were also crucial for the successful completion of the project. Since, both Profile I & II belong to group type S1 & S2 according to EC8, special study was necessary to define the proper seismic action and the exact liquefaction potential. Thus, 1D equivalent linear analyses were performed with the equivalent-linear frequency domain method (e.g. Schnabel et al. 1972). Modulus reduction and hysteretic damping curves were used as a function of cyclic strain amplitude (Vucetic & Dobry, 1991), and introduced the non-linear behavior of soil layers in ground response analyses, according to its layers' plasticity index. According to EC8 provisions, three different accelerograms were used, which cover a wide range of frequencies and are representative of the seismic region.

Shear wave velocities of the improved ground were computed according to Eq. 5, while the peak ground acceleration at bedrock outcrop was calibrated to 0.24g, according to the Greek Annex of EC8 for the area under investigation. Since no bedrock was found, artificial bedrock was used at the end of each borehole, while the bedrock shear wave velocity was assumed to range between 300 and 550m/s, providing a high impedance ratio compared with the soil column characteristics. Thus, radiation damping was conservatively minimized. Fig. 5 shows representative results from ground response analyses conducted in Profile II. Significant de-amplification of the seismic motion is observed, due to the flexibility of the soil column but also due to the non-linearity exhibited by the soil layers. The computed peak ground acceleration at ground surface ranges between 0.20 to 0.24g, significantly lower from the 0.32g required by EC8 for the flexible soil type D. Thus, the structural forces due to seismic loading were significantly reduced, while the factor of safety against liquefaction was substantially increased.

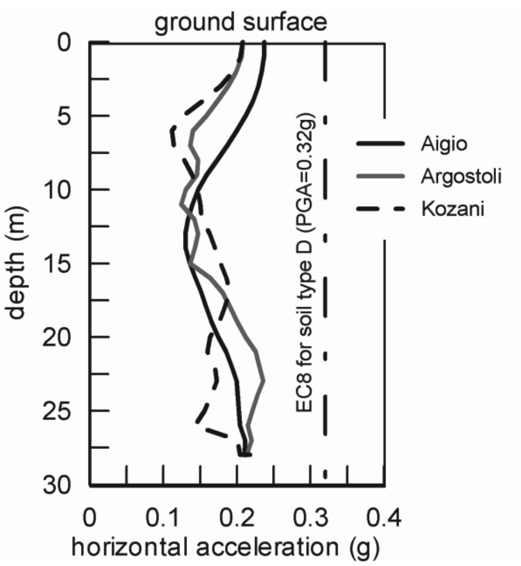

Figure 5: Distribution of peak ground acceleration with depth for Profile II using three different accelerograms.

6 CONCLUSION

The present paper presents details of the technical solution proposed for a road design project in Western Greece, where major geotechnical issues had to be dealt with for the foundation of bridges and high embankments. Geotechnical investigations revealed very poor soil conditions consisting of silty clays and sands, often with high content of organics, and high ground water table that locally appeared on the ground surface. As a result, the foundation of foreseen bridge piers on surface foundations was excluded and was replaced by a group of piles with a rigid pile cap. Among a number of possible methods of soil improvement that were examined, it was finally decided to proceed with the application of stone columns followed by preloading. This way, the following were accomplished:

- increase of the general stability of the bridge embankments
- increase of the bearing capacity of foundation soil layers
- reduction of internal forces of piles
- acceleration of the stage of primary consolidation of silty clay-sands and
- reduction of the liquefaction potential of sandy layers.

All of the above effects were verified by site-specific computations and implemented to the design of the relevant works

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