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STONE COLUMN GROUND IMPROVEMENT AGAINST LIQUEFACTION FOR THE PREVEZA-AKTIO IMMERSSED TUNNEL

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ABSTRACT

The construction of the road fixed-link crossing between Preveza and Aktio on the north-western coast of Greece has recently been completed. The project consists of an immersed single-tube tunnel under the strait and two cut-and-cover tunnels at the ends of the immersed part. The immersed section is constructed of eight precast rectangular concrete elements placed on the sea bottom at a depth of 25 m. The site is characterized by high seismicity. Foundation soil consists of Holocene marine sediments comprising irregular layers of sands, silts and clays that extend to great depth in the central part of the strait. Along the entire tunnel alignment the silty sands and sandy silts of the top layers are classified as potentially liquefiable due to their low density. Soil improvement in the form of stone columns has been carried out below the tunnel foundation for reducing risk from liquefaction. The soil conditions and the seismic input parameters are presented and an overview of the extensive dynamic laboratory testing program conducted is given. The seismic site response analyses performed for determining the depth of ground improvement are summarized. The design of the stone columns as well as a practical method for estimating the stiffness of the composite soil are described.

INTRODUCTION

As part of a major highway system the road fixed-link crossing between Preveza and Aktio on the north-western coast of Greece has recently been constructed and opened to traffic in July 2002.

After an international tender the contract was awarded in 1995 by the Hellenic Ministry for the Environment, Physical Planning and Public Works to a joint venture company between Christiani & Nielsen Ltd and T.E.G.C. (now Empedros S.A.). The tunnel project -a design and built contract- has been partially funded by the European Union, the total costs reaching 74 million Euro. The design of the tunnel itself has been carried out by Comar Engineers A/S, Copenhagen. GuD Geotechnik und Dynamik Consult GmbH, Berlin worked as sub-consultant to Comar Engineers A/S in geotechnical earthquake design.

The total length of the fixed link is approx. 4.7 km, of which 909 m is an immersed single tube two-lane tunnel, 509 m is a diaphragm wall type cut-and-cover tunnel on the Preveza side and 152 m is cut-and-cover tunnel on the Aktio side. The remaining length is approach roads on both sides. The immersed section is constructed from eight precast rectangular concrete elements approx. 60 m to 135 m long casted in a dry dock, towed and immersed in a trench excavated on the sea bottom at a depth of 27 m. The overall section of the elements is 12.6 m wide and 8.75 m high.

The site is located in an area of severe seismic activity. The geotechnical exploration program during the tender phase has revealed an irregular stratigraphy along the tunnel axis and also the existence of loose silty/sandy layers that would liquefy in the event of a major earthquake. The installation of stone columns below the entire length of the tunnel has been proposed as an effective countermeasure.

The next sections present in concise form the prevailing soil conditions, the seismic input parameters selected, an overview of the geotechnical investigation program, the seismic site response analyses conducted for the liquefaction study and the methodology followed in the design of the stone column foundation.

SITE GEOLOGY

The bedrock mainly consists of Triassic-Jurassic limestone of the Ionian zone but was not identified during the ground investigation. The oldest layers penetrated in the boreholes are Pliocene/Pleistocene deposits comprising mostly hard clays, silts and marls with inclusions of layers of sands. They are overlain along the entire length of the tunnel by nonhomogeneous Holocene marine sediments that comprise layers of sands, silts and clays, partially with a high content of organic matter. From the longitudinal section shown in Fig. 1 it can be seen that the immersed tunnel is founded wholly within these soft sediments.

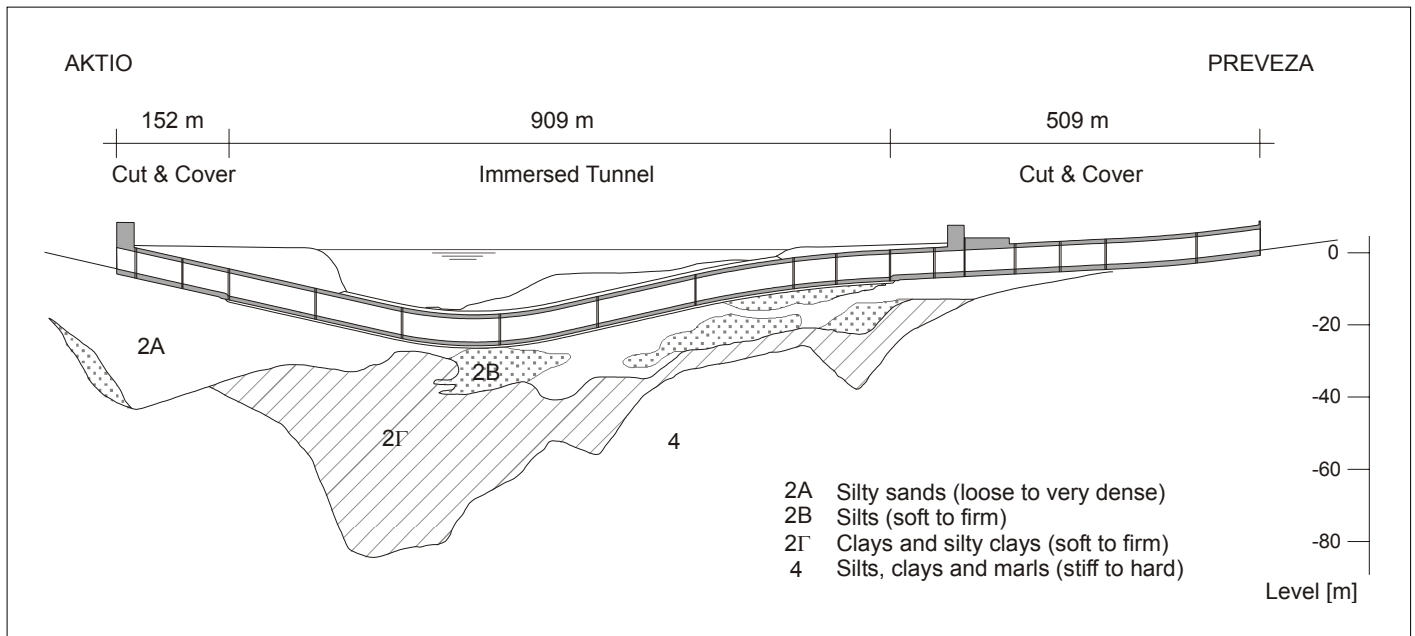


Fig. 1. Longitudinal section showing geology with soil units.

SEISMIC INPUT PARAMETERS

The seismological study carried out in 1995 concluded that the site is characterized by high seismicity and is also surrounded by zones of high seismicity. Earthquakes of magnitudes 6.2 to 6.3 have been observed in the past at close (10 km), moderate (30 km), and far (120 km) distances from the site.

From a statistical analysis it is concluded that earthquakes in the region with return periods 475 and 759 years are of magnitudes 7.3 and 7.5, resp. A detailed seismic hazard estimation by applying different methodologies yielded a peak ground acceleration at the free surface of 0.32g for a return period of 475 years and 0.40g for 949 years return period.

Synthesized strong ground motions have also been derived by considering three cases: i) near field, ii) moderate distance and iii) far field earthquake with different values of magnitude M_s , epicentral distance D , and focal depth H : i) $M_s = 6.5$, $D = 10$ km, $H = 10$ km, ii) $M_s = 7$, $D = 30$ km, $H = 15$ km, and iii) $M_s = 7.5$, $D = 70$ km, $H = 18$ km. The maximum acceleration a_{max} at base rock level ranged from 0.27g for the near field to 0.12g for the distant earthquake. For the bedrock a shear wave velocity $V_s = 600$ m/s was assumed.

Based on these findings the following values were finally suggested for use in the tunnel design: Synthesized strong motions were scaled so as to yield $a_{max} = 0.33g$ for the near field earthquake and were assigned to the rock outcrop. The accelerogram for the near field design earthquake and its response spectrum are depicted in Fig. 2.

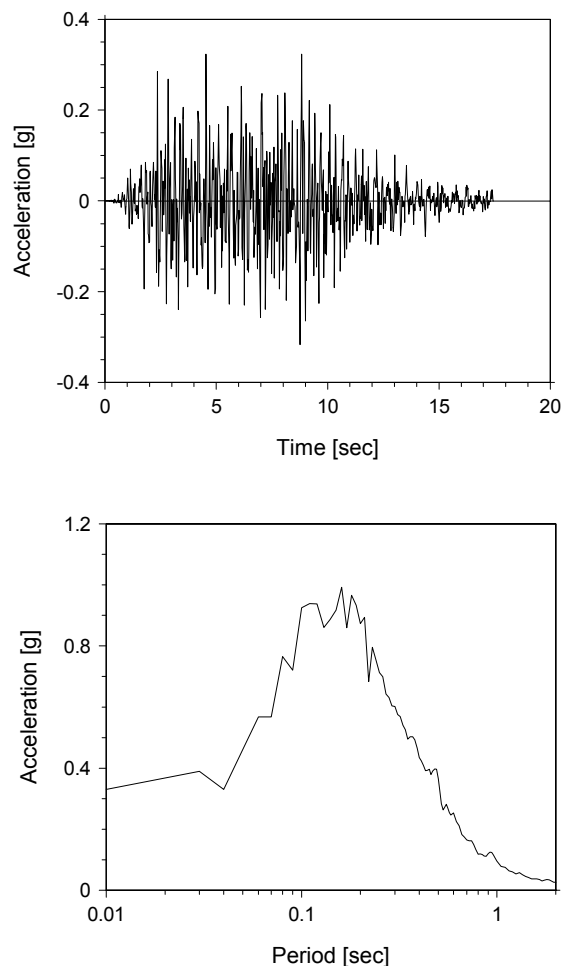


Fig. 2. Time history and 5% damping response spectrum of the near field design earthquake scaled at 0.33g.

GEOTECHNICAL INVESTIGATION PROGRAM AND SOIL PARAMETERS

An extensive in-situ and laboratory testing program has been carried out both off-shore and on-shore. Standard tests included all common laboratory tests for soil classification, soil shear resistance, and compressibility as well as Standard Penetration Tests (SPT) at the 14 borehole locations and a large number of Dynamic Probing Super Heavy (DPSH) along the tunnel alignment. Two pumping tests have been carried out to evaluate soil permeability.

In order to obtain a continuous profile of the soil conditions a total of 72 Cone Penetration Tests with pore pressure measurement (CPTU) have been performed on land and at sea. In addition 10 Seismic Cone Penetration Tests (SCPTU) have been conducted on land to gain information on the in-situ shear wave velocity.

Information on the dynamic soil properties (shear modulus and damping ratio) over a wide strain range was obtained from the results of 30 Resonant Column Tests. 18 Cyclic Direct Simple Shear Tests were carried out on sandy/silty samples to determine liquefaction potential in terms of pore water pressure build-up. Tests were suitably adjusted to allow for in-situ field conditions.

Idealized soil profiles for the seismic site response analysis were established by combining empirical correlations and results from the geotechnical investigation program. Particular attention has been paid to the nonlinearity of the soil properties. The degree of modelling accuracy is demonstrated next.

Soil Stratigraphy and Density

The layering sequence was taken from the borehole logs and verified from the results of the CPTU tests by using empirical charts from the literature. In-situ density of the cohesionless soils was estimated both from the SPT N -values and CPTU cone point resistance q_c using empirical relations that consider also the effects of effective confining pressure. Soil density inferred from CPTU tests was in general higher compared to that from SPT tests. The latter were used in the design, the former being considered as an addition safety factor.

Cyclic Soil Resistance

Stress-controlled cyclic simple shear tests were conducted both on sandy-silty and sandy soil samples. Due to the cohesion the former could be tested on undisturbed status while the latter have been tested at different values of relative density. Various combinations of cyclic shear stress τ_{cy} and effective consolidation stress σ_{vc}' have been selected. The onset of liquefaction was defined at 100% pore pressure ratio or/and when cyclic shear strain amplitude reached a value of 10% in double amplitude, Seed and Idriss [1982].

Seven tests have been conducted on recompacted sand samples ($d_{10} = 0.14$ mm, $d_{60} = 0.3$ mm) at two levels of cyclic stress ratio τ_{cy} / σ_{vc}' equal to 0.2 and 0.4, respectively. Consolidation stress σ_{vc}' and relative density D_r varied in narrow ranges from 120 kPa to 180 kPa and from 0.27 to 0.42, respectively, and no clear trend regarding their influence could be established from the tests. Number of cycles to liquefaction observed in the tests was equal to 2 at the higher level of cyclic stress ratio of 0.4, while at the lower level of 0.2 the respective cycles number varied between 6 and 10.

A silty sand sample with $d_{10} = 0.02$ mm and $d_{60} = 0.25$ mm tested at $\tau_{cy} / \sigma_{vc}' = 0.4$ with σ_{vc}' from 75 kPa to 150 kPa also showed initial liquefaction after 2 to 10 cycles. Typical sandy silt samples with 50% fines contents tested under $\sigma_{vc}' = 100$ kPa did not exhibit initial liquefaction but developed high strains reaching 10% double amplitude after 5 to 7 loading cycles at cyclic stress ratios between 0.3 and 0.4.

Dynamic Soil Stiffness and Soil Damping

Due to the strong variability of the soil strata and the large size of the site the project-specific resonant column tests were not sufficient to determine values for the dynamic soil properties at all desired locations. Additional data have been taken from tests on marine samples from a project in the Corinthian Gulf with similar soil characteristics. The test results referring to medium plasticity clays are presented in another paper by the authors, Vrettos and Savidis [1999]. Resonant column tests for each sample have been conducted at two levels of confining pressure. In the first stage strain levels were kept within the range of elastic soil behaviour while in the second stage the excitation force was increased reaching shear strain levels of up to 0.5%, depending on the soil material and the apparatus capacity.

From the various design equations available in the literature for the value of the shear modulus at low strain G_{max} the one suggested by Hardin [1978] for normally consolidated sands and clays was selected as the general equation

$$G_{max} = S \cdot [1 / (0.3 + 0.7 \cdot e^2)] \cdot (p_a \cdot \sigma_m')^{0.5} \quad (1)$$

where S is a stiffness factor, e is the void ratio, p_a is the atmospheric pressure, and σ_m' is the mean effective confining pressure. Hardin recommended $S = 625$.

Another estimate of G_{max} was made by combining the results of pocket penetrometer tests (unconfined compression strength q_u) and an empirical relation between the undrained shear strength c_u and G_{max} , whereby c_u is obtained from $c_u = q_u / 2$ and the ratio G_{max} / c_u is determined in dependence of the soil plasticity index following the summary by Weiler [1988].

Shear wave velocities at small strains $V_{S,max}$ (and accordingly G_{max}) were also estimated from the SPT N -values using the

empirical correlations given by Iwasaki [1988] for marine soils:

$$V_{s, \max} = 38.8 \cdot F_i \cdot (\sigma_0')^{0.175} \cdot (N+1)^{0.244} \quad (2)$$

where F_i is a soil factor with the values 1.0 for clay, 0.922 for sand, and 0.851 for gravel, σ_0' is the vertical effective overburden stress in kPa, and $V_{s, \max}$ is given in m/sec.

For the sandy soil the following relation based on the corrected and normalized SPT blow count number $(N_1)_{60}$ proposed by Seed et al. [1986] was also used to estimate in-situ shear modulus:

$$G_{\max} = 4400 \cdot [(N_1)_{60}]^{1/3} \cdot (\sigma_m')^{0.5} \quad (3)$$

with G_{\max} and σ_m' given in kPa.

SCPTU results showed considerably higher velocities than expected and were used mainly as indicator for the depth-variation of shear wave velocity.

Values for the shear modulus reduction and the damping ratio increase with increasing shear strain amplitude, G/G_{\max} vs. γ and D vs. γ , resp. for equivalent-linear dynamic analysis were taken from the site-specific resonant column tests and were checked against the set of curves proposed by Vucetic and Dobry [1991]. For the numerical analysis the foundation soils along the tunnel alignment have been divided into 17 types by introducing the following two general design equations:

$$G / G_{\max} = 1 / (1 + \alpha \cdot \gamma^c) \quad (4)$$

$$D = D_{\min} + D_{\max} / [1 + 1/(\beta \cdot \gamma^d)] \quad (5)$$

where γ is given in %, α , β , c , and d , are constants for the particular soil type and D_{\min} and D_{\max} are the values of damping ratio at the two ends of the $D(\gamma)$ curve, i.e. at very low and very high shear strain amplitudes, respectively. The large number of sub-groups reflects the complexity of the encountered soils conditions. Numerical values of the parameters for the soil profiles presented herein are given in Table 1.

In addition to these site-specific curves a set of curves, one for sands and one for clays, was also used to establish an alternative design soil profile. The respective G/G_{\max} vs. γ and D vs. γ relationships have been derived by linear regression for the data of the sample problem in the manual of SHAKE91, Idriss and Sun [1992]. The parameters in terms of equations (4) and (5) are given in Table 2. These curves describe a more linear soil behaviour and lead in general to higher amplification values.

The inherent uncertainty in modelling dynamic soil properties in site response analyses was tackled by first defining a best-estimate basic soil profile and then varying the dynamic soil properties within a prescribed range. The idealized basic

profiles at three representative borehole locations including CPTU-logs, SPT N -values, shear wave velocities, and material numbers are depicted in Fig. 3. The depth of the first layer, that has no entry, defines the depth of excavation for the tunnel.

Table 1. Soil parameters for the profiles in Fig. 3.

Material No.	α [-]	c [-]	D_{\min} [%]	D_{\max} [%]	β [-]	d [-]
1	25.0	1.0	1.5	24.0	7.4	1.0
2	22.0	1.0	2.0	24.0	7.4	1.0
3	7.7	0.8	2.0	20.0	5.4	0.8
4	28.0	1.2	6.0	22.0	4.8	1.0
5	23.0	1.0	1.8	20.0	5.4	0.8
6	7.7	1.1	2.0	20.0	3.7	1.0
7	9.5	0.8	2.0	21.0	6.0	0.8
8	7.7	0.8	2.0	20.0	5.4	0.8
9	20.0	1.0	2.0	22.0	9.0	1.0
10	6.5	1.0	3.5	18.0	3.9	0.8
11	5.0	1.0	4.0	24.0	1.7	1.0
12	25.0	1.0	1.5	22.0	7.7	1.0
13	23.0	1.3	2.5	22.0	7.3	1.3

Table 2. Soil parameters for alternative soil profile.

Material No.	α [-]	c [-]	D_{\min} [%]	D_{\max} [%]	β [-]	d [-]
21 - Clay	3.0	0.77	0.0	30.0	2.2	0.67
22 - Sand	17.1	1.0	0.0	30.0	2.2	0.67

ASSESSMENT OF LIQUEFACTION SUSCEPTIBILITY

The classical method based on a total stress approach was adopted for the liquefaction study.

The cyclic stress developed in the soil due to the earthquake shaking was calculated by the code SHAKE91 at five representative locations along the tunnel alignment. Between these five locations the cyclic stress ratio was determined by the simplified method suggested by Seed and Idriss [1982] considering an appropriate depth reduction factor.

The cyclic resistance of the soil was determined on the basis of field tests, the results of cyclic tests being used mainly as supporting evidence. The simplified procedure based on SPT blow count numbers was adopted, Seed et al. [1985]. Appropriate scaling factors were applied to account for earthquakes with magnitude less than 7.5, Seed and Idriss [1982], and also for the effects of overburden stress, Seed and Harder [1990].

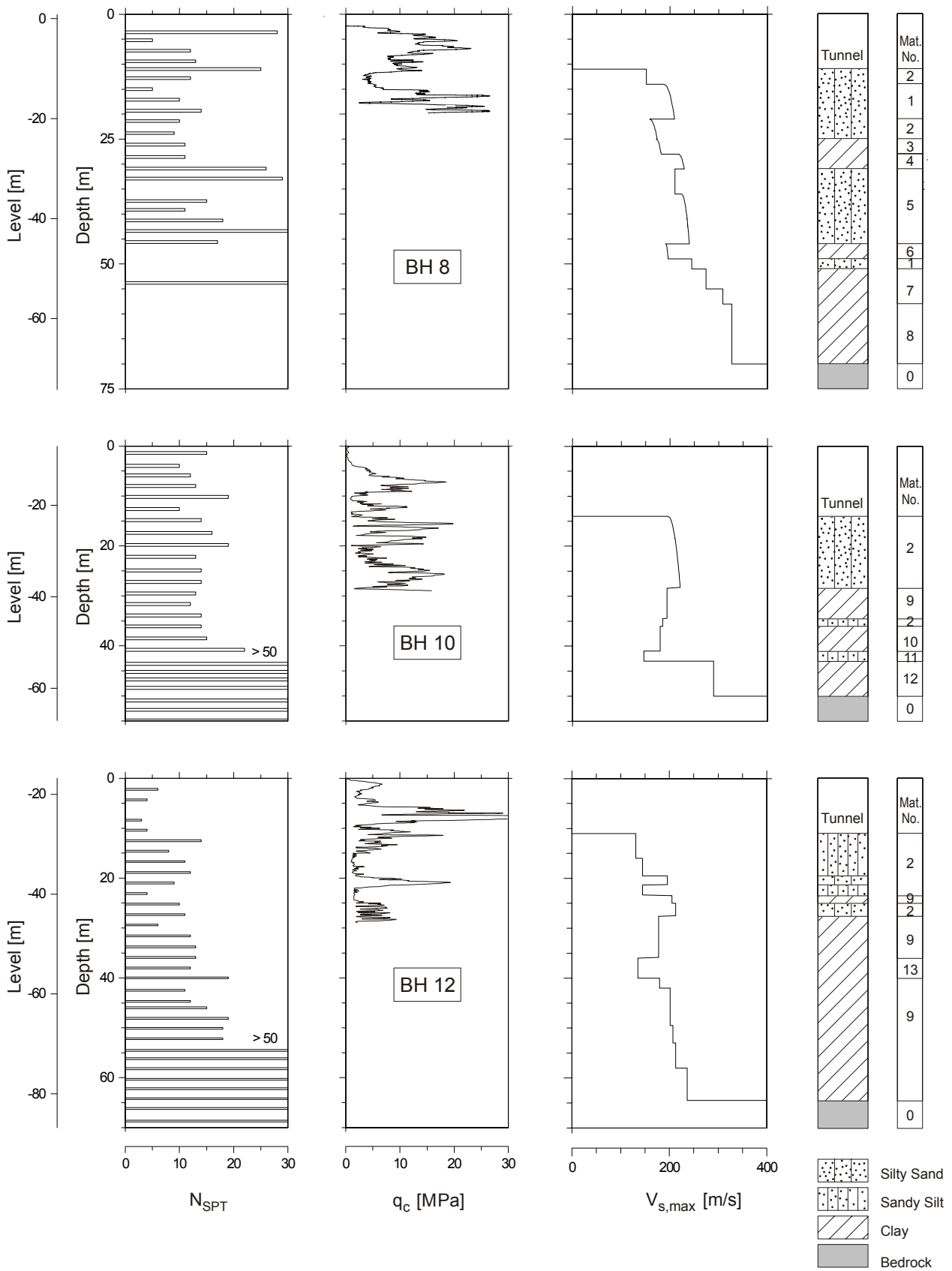


Fig. 3. Representative soil profiles at three locations. BH 8 is near the portal at the Aktio shore, BH 12 is at the center of the strait, and BH 10 is midway between the center of the strait and the Preveza shore.

Due to the vast amount of CPT results available and in order to attain a consistent evaluation of liquefaction resistance, criteria based on CPT tests were also applied at each site location investigated using the charts by Robertson and Fear [1995].

For the assessment of liquefaction susceptibility using results of cyclic tests the number of significant cycles for the earthquake under consideration is needed. This number was determined by interpolation from the recommendations by Seed and Idriss [1982] yielding a value of $N_{eq} = 8$ for $M_s = 6.5$.

The seismic site response analysis was first conducted at the location BH 8. All three synthetic earthquakes (near field, moderate distant, and far field) were considered at this location. Comparison of results showed that the near field earthquake clearly yields the highest seismic response values. Thus, for the other profiles and for the subsequent analysis of the tunnel foundation only this earthquake was used as seismic input. Figure 4 shows typical results for the distribution of cyclic stress ratio at the locations BH 8 and BH 12, whereby τ_{av} denotes the average shear stress due to the earthquake and σ_0' is the vertical effective overburden stress.

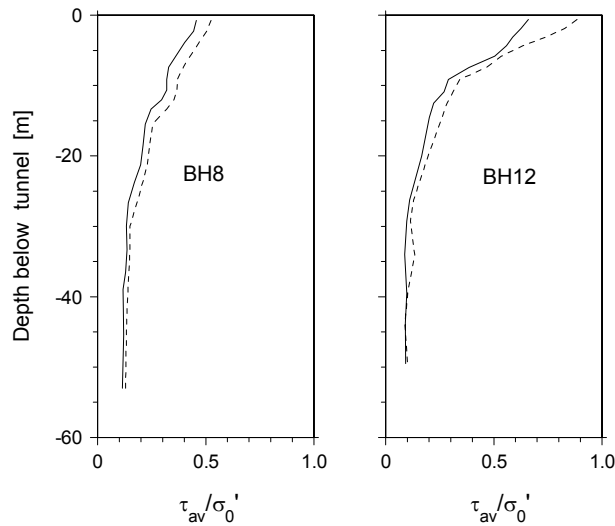


Fig. 4. Cyclic stress ratio due to near field earthquake. The solid lines are for the basic soil profiles, the dotted lines for profiles with 20% higher shear modulus for all soil layers.

The screening procedure described above yielded a liquefiable zone of approx. 15 m thickness along the entire length of the immersed and most of the cut-and-cover parts of the tunnel. The installation of stone columns was proposed as an effective countermeasure primarily due to the increased drainage capacity of the improved composite soil. The associated densification and reinforcement of the loose soil layers was considered as a secondary benefit. Although at some locations no improvement was necessary for protection against liquefaction, stone columns have been placed along the entire

length of the tunnel to provide a continuous distribution of foundation stiffness and dynamic forces in the structure.

DESIGN OF STONE COLUMNS

The method suggested by Seed and Booker [1977] was used for the analysis. The liquefaction potential of the specific site is first evaluated without drains, obtaining the earthquake cycles ratio N_{eq}/N_{liq} , where N_{eq} is the equivalent number of cycles induced by the design earthquake, and N_{liq} is the number of cycles needed to initiate liquefaction. Then, for a given stone column radius a the factor T_{ad} relating the earthquake duration to the consolidation properties of the natural soil, is computed

$$T_{ad} = (k / \gamma_w) \cdot [t_d / (m_v \cdot a^2)] \quad (6)$$

where k is the permeability of the soil in the horizontal direction, γ_w is the unit weight of water, t_d is the duration of shaking, and m_v is the coefficient of compressibility of the natural soil. Other parameters that need to be defined are the pore pressure ratio,

$$r_g = u / \sigma_0' \quad (7)$$

which relates the excess pore pressure u to the effective stress σ_0' , and $2 \cdot b$ which is the effective spacing between stone columns.

The curves relating the greatest pore pressure ratio generated anywhere in the soil/drain system and the properties of the soil and the earthquake are taken from Seed and Booker [1977]. After specifying a maximum allowable value for the pore pressure ratio (here $r_g = 0.6$), a/b is determined from the charts for given T_{ad} and N_{eq}/N_{liq} .

The permeability of the stratified sandy or/and silty sandy soil was determined by combining results from in-situ pumping tests and empirical relations on the basis of grain size distribution curves yielding an average value of the horizontal permeability $k = 2 \cdot 10^{-5}$ m/sec. The coefficient of volume compressibility was determined from the results of consolidation tests at the relevant level of overburden stress yielding an average value of $m_v = 4 \cdot 10^{-5}$ m²/kN.

The parameters associated with the design earthquake of magnitude $M_s = 6.5$ are: the number of equivalent cycles $N_{eq} = 8$, and the duration of ground shaking $t_d = 15$ sec. The number of cycles to cause initial liquefaction was determined from the results of the cyclic simple shear tests described in a previous section. An average value $N_{liq} = 4$ was selected.

By choosing a stone column diameter of $2 \cdot a = 0.6$ m all other parameters can be calculated yielding $b = 1$ m. For an

equivalent square grid the spacing between stone columns (center to center) is finally obtained by

$$s = \sqrt{\pi} \cdot b \approx 1.8 \text{ m} \quad (8)$$

The grading of the stone column material is selected so as to satisfy two requirements: i) difference in permeability of the drain material and the surrounding soil should be large enough to permit a hydraulic gradient, and ii) particle sizes of the drain material should be small enough to prevent clogging.

Usually, in geotechnical practice the criteria suggested by Terzaghi are applied, i.e.

$$D_{15} / d_{15} > 4 \quad \text{and} \quad D_{15} / d_{85} < 5 \quad (9)$$

where D and d refer to grain diameter of the filter material and the natural soil, respectively. However, the application of these criteria would lead to a non-practicable solution. Laboratory tests in Japan indicate that for dynamic loading a coarser drainage material is needed to guarantee effective permeability and a less restrictive lower limit for clogging may be applied. The corresponding design equation suggested by Saito et al. [1987] is

$$20 \cdot d_{15} < D_{15} < 9 \cdot d_{85} \quad (10)$$

and was adopted for the design.

The resulting grain size distribution of the stone column material is given in Fig. 5 together with a typical curve for the liquefiable loose silty sand.

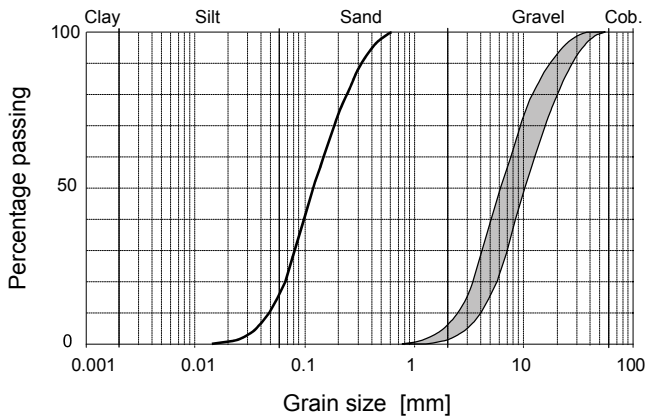


Fig. 5. Grain-size distribution of the stone column material and of the liquefiable sandy soil.

SITE RESPONSE ANALYSIS FOR THE IMPROVED SOIL

For the seismic site response analysis a composite soil consisting of stone columns and natural soil is used. The stiffness of this composite system is determined by applying the concept of stress concentration. The simplified analysis suggested by Priebe [1990], [1995] is adopted. Based on static

considerations the load concentration is expressed in terms of an area replacement ratio

$$AR = A_c / A \quad (11)$$

where A_c is the cross-sectional area of each stone column, and A is the total area of influence of each column. Here $A_c = 0.282 \text{ m}^2$ and $A = 3.24 \text{ m}^2$ which gives $AR = 0.087$.

With regard to settlements of the treated ground and accordingly to the mean stiffness of the composite system the effect of stone column placement is generally described by an improvement factor n . This improvement factor depends on the friction angle of the stone column material φ_c and on the area replacement ratio. Assuming $\varphi_c = 40^\circ$ we obtain from Priebe [1995] for the improvement factor a value of $n = 1.45$.

To account for the effects of stone column compressibility this factor is modified in dependence of the stiffness ratio E_c / E_{soil} , where E_c and E_{soil} are the elastic moduli of the stone column material and the natural soil, respectively. Values for E_c / E_{soil} at small strains have been derived from the data for gravels summarized by Ishihara [1992]. The different shear modulus reduction relationships of the two soil types of the composite soil system have been considered by means of an approximate averaging procedure. At the location of BH 8 for example a value of $E_c / E_{\text{soil}} = 3$ was chosen.

The modified improvement factor is determined next from Priebe [1995] to $n_1 = 1.39$ at BH 8. The average stiffness of the composite soil E_m is finally calculated from the modified improvement factor n_1 and the stiffness ratio E_c / E_{soil} . At the location BH 8 we obtain $E_m = 1.27 \cdot E_{\text{soil}}$, and the shear modulus of the top silty sand layer that has been improved by stone columns is calculated by multiplying the respective value of the unimproved soil as given in Fig. 3 by the factor 1.27. For the other locations the factor E_m / E_{soil} varied slightly ranging between 1.54 for the organic soft top layers at Preveza side to 1.2 for the top foundation layers at the locations in the center of the strait.

Following the same methodology one may obtain the ratio of the residual stress in the natural soil p_{soil} to the total stress p

$$p_{\text{soil}} / p = 1 / n_1 \quad (12)$$

which may be used in the design when estimating the remaining liquefaction potential of the improved soil. Following the suggestion by Priebe [1990] the cyclic stress ratio developed in the field by the earthquake is multiplied by this stress reduction factor to yield an approximate value of the expected cyclic stress ratio in the natural soil between the stone columns. For the present case $p_{\text{soil}} / p = 0.73$ at BH 8. However, this beneficial effect was not considered in the design of the stone columns being regarded as an additional safety factor.

The site response analysis was performed using SHAKE91 and followed the same lines as for the unimproved soil. Response spectra have been computed by considering four profiles at each location: i) basic profile, ii) basic profile with 20% higher G_{max} for all layers, iii) basic profile but with shear modulus reduction and damping relationships from Table 2, iv) like profile iii) but with 20% higher G_{max} . At each frequency the maximum value of the response from all profiles considered was taken to construct an envelope curve to be used in the subsequent 3D-structural analyses of the tunnel. Typical results at two locations are given in Fig. 6.

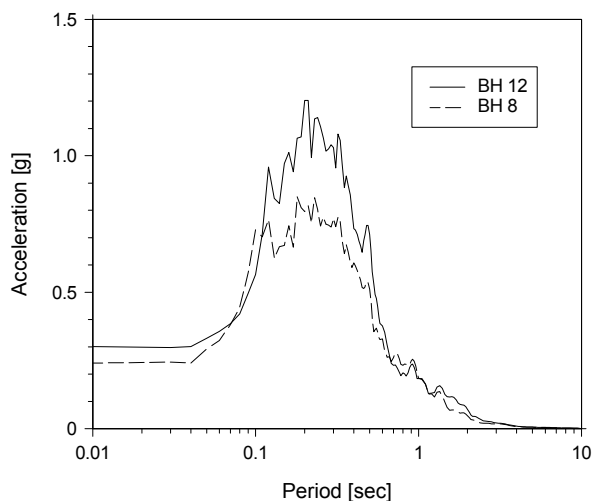


Fig. 6. Envelope curve of response spectra for 5% structural damping at locations BH 8 and BH 12.

INSTALLATION OF STONE COLUMNS

A total of 8782 columns of 60 cm diameter have been installed using the wet displacement method. The procedure has been performed for the marine section, as well as for most of the land based cut-and-cover tunnels.

Before start of regular column production a trial field test has been conducted on land to prove the effectiveness of the proposed construction method. The trial field was representative in terms of composition and thickness of the soil layer to be improved. 25 stone columns with 15 m length were constructed in a square grid at a center to center distance of 1.8 m. Before and after the installation dynamic probings were carried out. After the installation of trial stone columns the area was excavated, the columns were inspected with regard to integrity, and the cross section area of the columns was measured. Samples of the stone column material were taken for sieve analysis and density determination.

The marine stone columns have been installed in the dredged trench at sea from a barge equipped with a crane. Nominal length of stone columns was 15 m, i.e. the maximum installation depth was 42 m below sea level. A normal

310 mm vibroflot was modified to incorporate an upper concentric chamber that has been charged with sufficient graded stone for each column. A valved stone delivery down-pipe fitted to the side of the flot body allowed the stone to be bottom fed. The stone was fed down the delivery pipe as the flot was withdrawn and compacted by surging the tool in the normal fashion. The construction method applied allowed the vibroflot to completely immerse, thus limiting the required maximum length to 20 m. For each column a continuous record of data on pile length, compaction force (between 180-220 bar), time taken, inclination, and accurate position was obtained by telemetry and stored electronically.

Stone columns on land have been installed in the same manner as at sea by using a vibroflot suspended from a crawler crane.

MONITORING

A structural and seismic monitoring system has been installed after completion of works in 2002 to observe ambient static and dynamic loads including seismic activity. Three triaxial force-balanced accelerometers are placed on the immersed tunnel structure and one in the freefield at the Preveza shore. In addition forty-four linear variable displacement transducers (LVDT) are mounted at the elements tunnel joints.

No significant earthquake occurred in the broad region since completion of the seismological study. A moderate earthquake of magnitude $M_w = 5.3$ took place in 2002 on the Ionian Sea approx. 120 km south of the site. However, instrumentation was still not in operation, so that no recording is available.

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