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Soil liquefaction potential assessment of a coastal foundation ground and its suitability for a CCGT Power Plant construction in Greece

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ABSTRACT

Clayey silty up to silty sandy and sandy soils are generally recognized to have a significant liquefaction potential when extended submerged below water table. This phenomenon raises a major concern to the foundation and structural engineer. Low plasticity silts, silty clays and silty sands occur extensively as recent alluvial deposits in the southern coastal region of Elefsina Municipality in Attica Prefecture, Greece.

In this area, a Combined Cycle Gas Turbine (CCGT) Power Plant is planned to be constructed and its foundation stability and durability reassurance is of utmost importance to structural engineers. In the study of the geotechnical ground investigation for the foundation design of the CCGT project, a number of field and laboratory tests were carried out.

For evaluating its foundation soil liquefaction potential and risk during an earthquake, some internationally accepted guidelines are available based on soil density, void ratio, plasticity index, standard penetration test values, and other simple soil properties.

The liquefaction behavior and potential of this kind of foundation soils stratified in the alluvial deposits has been studied thoroughly based on both Seed’s and Idriss’s procedure / relationships as well as Prakash’s limit state methodology, using S.P.T. results and an algorithm program / software code, that was developed and published by the author. The S.P.T. tests were executed inside the twenty investigation - sampling boreholes of a depth range from 10 up to 50 meters each one, in an 100.000 s.m. plot, where a Combined Cycle Gas Turbine (CCGT) Power Plant is planned to be constructed.

According to the results of these analyses and assessments the well documented and argued necessity is deduced either for transferring the project foundation loads to underlying deeper and more competent bearing strata and layers, or for strengthening, geotechnically upgrading (ground improvement), stabilizing and cement grouting the foundation ground of the CCGT Power Plant using jet grouting piles techniques.

Finally, the exact depth range under the CCGT Power Plant foundation site that is prone and dangerous to be liquefied in the event of a strong seismic shock and vibration was determined and diagrammatically presented and the remedial measures to be taken were suggested. Hence, in this way the liquefaction risk can be mitigated or even deterred from the incompetent upper natural soil layers of the project foundation ground.

KEYWORDS: Soil Liquefaction Potential; Standard Penetration Tests & Values (S.P.T.); Grain Size Distribution; Sand, Silt; Clay; Ground Improvement; Geotechnical Properties; Foundation Ground; Alluvial Deposits; CCGT Power Plant

INTRODUCTION

It is well known that soil liquefaction describes a phenomenon whereby a soil substantially loses strength and stiffness in response to an applied stress, usually earthquake vibration or other rapid loading, causing it to behave like a liquid.

Earthquake liquefaction is a major contributor to urban seismic risk (Poulos H.G. 2004). The shaking causes increased pore water pressure which reduces the effective stress, and therefore reduces the shear strength of the sand. If there is a dry soil crust or impermeable cap, the excess water comes sometimes to the surface through cracks in the confining layer, bringing liquefied soil with it, creating soil boils, colloquially called "soil or sand volcanoes" (Crone, Anthony J; Wheeler, Russell L (2000)).

Cyclic shear stresses develop in the soil deposits as a result of passage of seismic waves through them. These stresses may result in progressive build up of pore water pressure in certain types of soils in a saturated state. Cohesionless soils of loose and medium density have a tendency to compact under vibrations leading to decrease in the inter-granular space. This tendency for volume decrease gives rise to increase in pore water pressure. The progressive build up of pore water pressure may eventually become large enough resulting in complete loss of shear strength accompanied by large deformations and failure. (Prakash S. and Puri V. K., (2003)).

Liquefaction is more likely to occur in loose to moderately saturated granular soils with poor drainage, such as silty sands, sandy silts, clayey sands (Jefferies, M. and Been, K. (2006)) or sands and gravels capped or containing seams of impermeable sediments (Youd T.L. and Idriss, I.M. (2001)). During wave loading, usually cyclic undrained loading, e.g. seismic loading, loose sands tend to decrease in volume, which produces an increase in their pore water pressures and consequently a decrease in shear strength, i.e. reduction in effective stress.

Deposits most susceptible to liquefaction are young (Holocene-age, deposited within the last 10,000 years) sands and silts of similar grain size (well-sorted), in beds at least meters thick, and saturated with water. Such deposits are often found along riverbeds, beaches, dunes, and areas where windblown silt (loess) and sand have accumulated. Some examples of liquefaction include quicksand, quick clay, turbidity currents, and earthquake liquefaction.

Depending on the initial void ratio, the soil material can respond to loading either strain-softening or strain-hardening. Strain-softened soils, e.g. loose sands, can be triggered to collapse, either monotonically or cyclically, if the static shear stress is greater than the ultimate or steady-state shear strength of the soil. In this case flow liquefaction occurs, where the soil deforms at a low constant residual shear stress. If the soil strain-hardens, e.g. moderately dense to dense sand, flow liquefaction will generally not occur. However, cyclic softening can occur due to cyclic undrained loading, e.g. earthquake loading. Deformation during cyclic loading will depend on the density of the soil, the magnitude and duration of the cyclic loading, and amount of shear stress reversal. If stress reversal occurs, the effective shear stress could reach zero, then cyclic liquefaction can take place. If stress reversal does not occur, zero effective stress is not possible to occur, and then cyclic mobility takes place (Robertson, P.K., and Fear, C.E. (1995)).

The resistance of the cohesionless soil to liquefaction will depend on the density of the soil, confining stresses, soil structure (fabric, age and cementation), the magnitude and duration of the cyclic loading, and the extent to which shear stress reversal occurs (Robertson, P.K., and Wride, C.E. (1998)).

The phenomenon of liquefaction has been extensively studied for the case of cohesionless soils under seismic loading conditions. The international research on liquefaction behavior of cohesionless soils has shown that reasonable estimates of liquefaction potential and prediction can be made based on simple in-situ test data such as standard penetration values (S.P.T. tests), some lab tests and the experience during the past earthquakes, (Prakash,1981; Youd and Idriss ,2001; Youd et al, 2001; Bouckovalas G. et al, 2001, 2006 ; Seed,1976,1979; Seed and Idriss, 1967,1971, 1981; Seed and DeAlba, 1986; Seed, Idriss and Arango, 1983; Lee and Seed, 1967; Seed and Harder, 1990; Seed et al , 1984,1985).

The cyclic stress approach (Seed and Idriss ,1981) and the cyclic strain approach (Dobry et al, 1982), as well as the Standard Penetration Tests empirical method (Prakash S., (1981, 2003)), are commonly used for evaluation of liquefaction potential of silty soils up to silty sands.

All the above mentioned studies have proven that the effects of soil liquefaction on the built environment can be extremely damaging and that liquefaction can cause damage to structures in several ways. Buildings whose foundations bear directly on clayey silty up to silty sandy and sandy soil which liquefies will experience a sudden loss of support, which will result in drastic and differential settlement of the building. Buildings constructed on pile foundations may lose support from the adjacent soil and buckle. Liquefaction causes differential and irregular settlements in the area liquefied, which can damage buildings and break underground utility lines where the differential settlements are large. Pipelines and ducts may float up through the liquefied clayey silty up to silty sandy and sandy soil. Sand boils can erupt into buildings through utility openings, and may allow water to damage the structure or electrical systems. Areas of land reclamation are often prone to liquefaction because many are reclaimed with hydraulic fill, and are often underlain by soft soils which can amplify earthquake shaking.

Object and scope of the present research paper is the analysis, calculations and control of the liquefaction risk potential and probability of the upper natural soil layers of foundation ground, that consist mainly of Silty and Sandy Quaternary Holocene Alluvial deposits with clay intercalations, of a consistency of very dark grey to grey-black colored, very fined grained silty sand up to at certain places sandy silt, of a very low relative density, with a little at certain places gravels and clay, and with a lot of organic admixtures, that could be triggered and caused from a potentially high seismic activity in the sea side region where the CCGT Power Plant is planned to be founded.

Finally, evaluation, prediction and mitigation proposals of the potential damage from liquefaction of a Combined Cycle Gas Turbine (CCGT) Power Plant Project construction in Greece are presented in this research work.

STUDY AREA DESCRIPTION AND INVESTIGATION POINTS

The wider foundation area of the CCGT Power Plant is located in the south-eastern coastal region of Elefsina Municipality of Attica District, Greece, next to the sea front of Elefsina Golf, as shown in the map of fig. (1).

The plot section under study has an extent of roughly 25 acres, on which the CCGT Power Plant will be constructed, and it is placed south-eastern of the central urban area of Elefsina City. Furthermore, the plot section under study extends exactly eastwards and southward of the zone where the river Sarantapotamos discharges into the sea, and hence on estuary alluvial fan deposits where on top of them there exists man made backfilling materials as described later on.

The absolute elevation of ground surface in the examined area ranges from 0,55 m up to 4,16 m above sea level (a.s.l.), with the mean absolute elevation to be at 2,38 m a.s.l., and its surface presents almost level inclinations, in the order of 0,1 % to the south.

The narrow project foundation area, before the dumping and the configuration of the artificial embankments (man made backfilling material), was located inside the marine section environment of the gulf of Elefsina and not in the coastal mainland. This fact with the exact delimitation of the old shoreline (red line), before dumping the artificial backfilling material, is depicted in the topographic diagram of fig. (2).

The old seabed in the area in question ranged in depth from 0,00 m up to 15,00 roughly meters below sea level. Obviously, a great deal of work of back filling in this zone was materialized for the extension of ground towards sea. Moreover, according to detailed data that resulted from geotechnical engineering site investigation and characterization study using twenty (20) investigation - sampling boreholes, it was revealed that the artificial embankments (man made backfilling material) that were placed in this coastal marine section in the gulf of Elefsina have a thickness that ranges from 0,00 m up to 17,00 meters. The exact borehole locations, inside which S.P.T. tests were executed, according to ASTM D1586 or British Standard BS EN ISO 22476-3, for the soil liquefaction potential and risk assessment, are shown in the topographic diagram of fig. (3).

The man made backfilling material is non-homogeneous and anisotropic. In certain places, materials like rubble that contains pieces of concrete, soils and timbers, metallurgic furnace slag and materials from cemented / conglomerated furnace slag are present, along with distributed exceptionally hard materials of dimensions up to 1,50 m consisting of metal objects such as steel items, iron blocks, metal alloys, etc.

Additionally, the man made backfilling material has a very high unit weight compared to the underlying natural soil formations and in combination with their increased at certain places thickness, imposed significant overburden pressure into the underlying geological formations and layers. This resulted in the overall strengthening of the natural layers in terms of their shearing resistance, since on one hand the fine grained layers (silts and clays) were consolidated due to pre-loading, and on the other hand the coarse grained layers (sands, gravels, pebbles) were compacted because of the surcharge and vibration that they underwent during the dumping process.

Given the long time interval (in the order of 10 years) that has elapsed from the dumping of these artificial embankments it is obvious that the process of compaction and consolidation of the underlying natural geological formations and layers have progressed to a considerable extent rendering to these underlying natural geological formations and layers more improved geotechnical properties, characteristics and parameters and finally these natural geological formations and layers have been upgraded in terms of their geotechnical and soil mechanics behavior and properties. According to Soil Mechanics theory (Terzaghi K, Peck R.B., Mesri G. (1996)) and world widely observed practice in numerous case studies, the existence of surcharging on an underlying fine grained soil layer acts beneficially to that layer due to the pore water pressure dissipation which in turns results in an overall over-consolidation condition of the layer in question and hence strengthens it improving its shearing strength parameters and resistance.

However, we consider that although this surcharging due to the dumping of the man-made scrap fill materials has acted beneficially and positively to the foundation conditions of the CCGT Power Plant, a more detailed and profound geotechnical site investigation program with advanced laboratory testing (i.e. UU, CU, CUPP and CD Tiaxial tests) is absolutely required.

Figure 1: The study area where the Combined Cycle Gas Turbine (CCGT) Power Plant project is planned to be constructed in the coastal region of Elefsina Municipality, Greece.

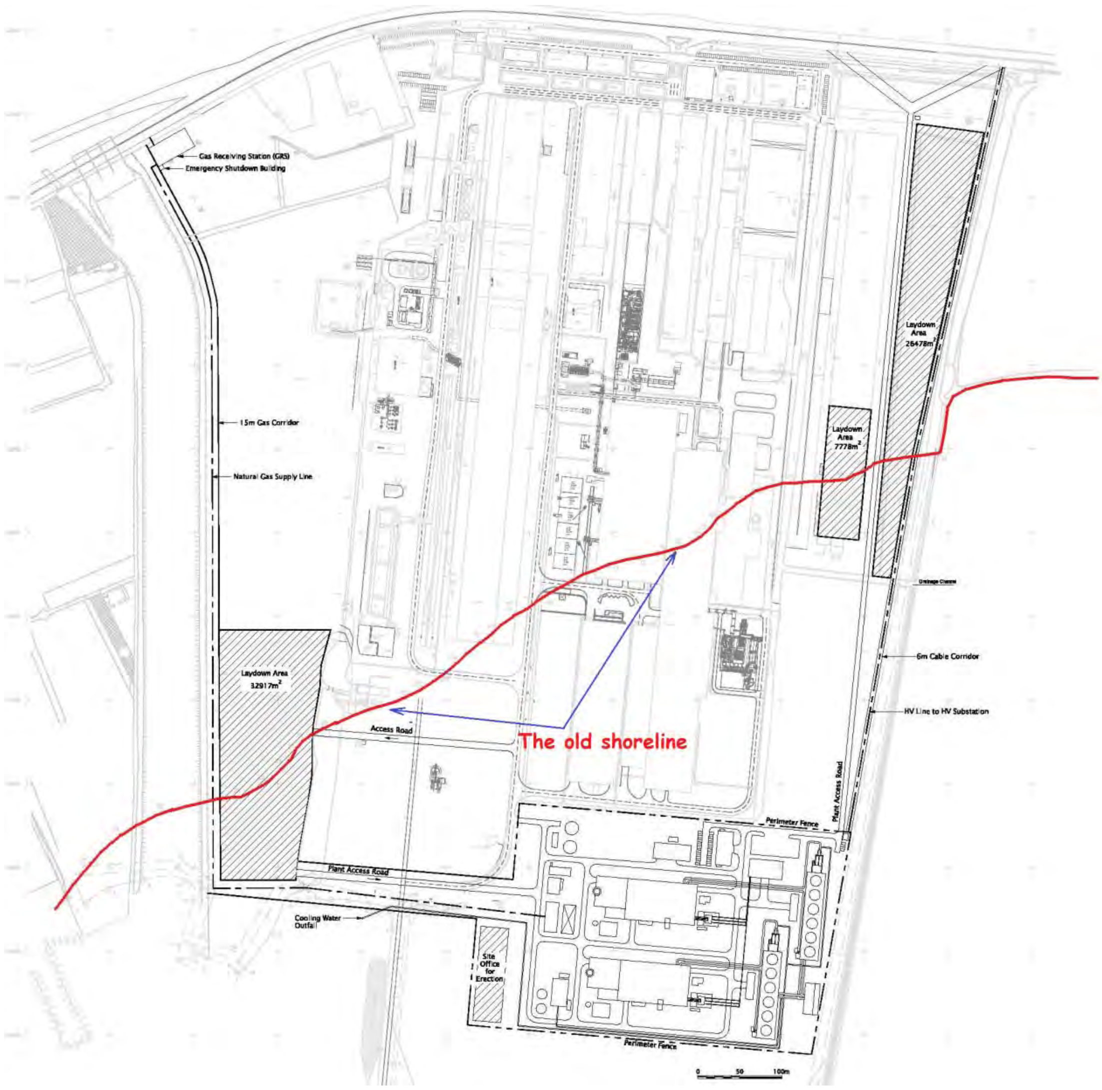


Figure 2: Topographic diagram showing the project foundation area in relation to the old shoreline (red line) before dumping the man made backfilling material consisting of rubble and metal objects that emanated from metallurgic furnace slag and steel block material.

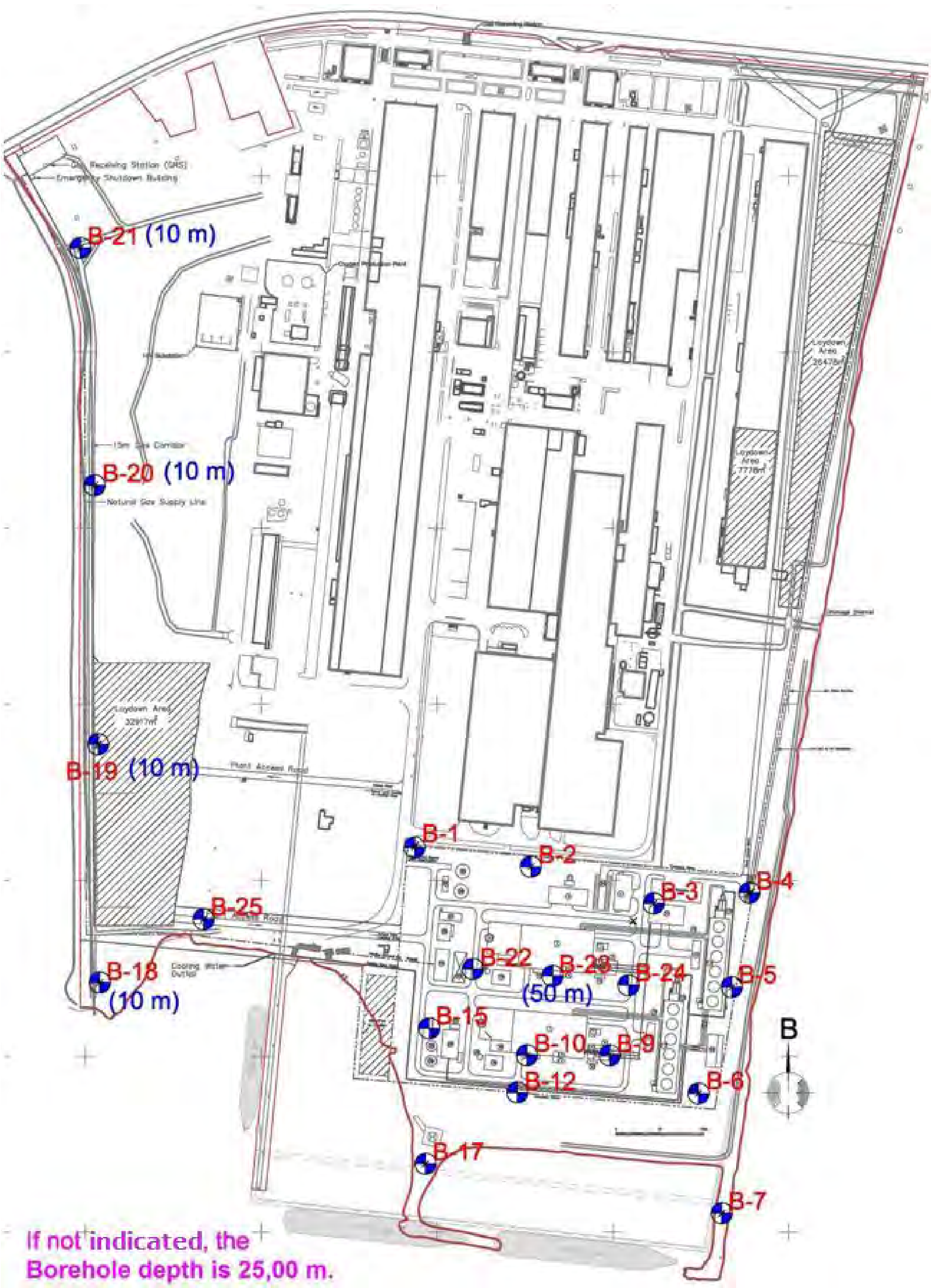


Figure 3: Topographic diagram showing the exact locations of the twenty investigation - sampling boreholes where S.P.T. tests were executed for the soil liquefaction potential and risk assessment, in the foundation area of the CCGT Power Plant and its pipeline alignments.

THE FOUNDATION AREA GEOMECHANICAL SOIL MODEL

Based on regrouping and unification techniques, the initial various «layers», as these were encountered and drilled in twenty (20) investigation - sampling boreholes, were classified in the correspondent ground category they belong. Subsequently, the various geotechnical properties, parameters and coefficients were regrouped and unified giving in each particular soil category (group), the minimum, the maximum and the average value of each correspondent property. The purpose of this regrouping / unification and homogenization of all layers and of their properties is for reasons of processing, evaluation and analysis of soil mechanics / foundation parameters of the CCGT Power Plant.

In Table 1 the unified geomechanical / stratigraphic foundation ground model is presented. This will be used in soil liquefaction potential assessment analysis and calculations, along with the average values of summarized soil mechanics properties (soil index coefficients, consistency, shearing strength parameters, compressibility, etc.) of each layer.

Table 1: Unified geomechanical / stratigraphic foundation ground model and soil / rock mechanics properties and parameters / coefficients, applied to the foundation zone of the CCGT Power Plant.

#	Depth of Contact of the Geologic Formations from (m) to (m)	Ground Category	Sym-bol
1	Minimum (*): 0,00 0,00 Maximum (*): 0,00 17,00	**(Ground Category 1): Shallow Artificial Embankments - (Man made backfilling material - Metallurgic furnace slag - Metals - Rubble).** («Dark grey to grey-brown colored, slightly silty and clayey sands-gravels and pebbles, of a very high relative density (well compacted), containing metallurgic furnace slag as well as materials from cemented / conglomerated furnace slag, with distributed exceptional hard materials of dimensions up to 1,50 m of metal objects such as steel items, iron blocks, metal alloys, etc, containing also at certain places demolition materials, such as tiles, timber, metals, and intercalations from non-reinforced concrete blocks». This layer, as a whole, constitutes an erratic, non-homogeneous and anisotropic foundation material. Non cohesive, coarse grained soil: «GP»).	
2	Minimum: 3,00 8,70 Maximum: 13,50 18,40	**(Ground Category 2): Silty and Sandy Quaternary Holocene Alluvial deposits with clay intercalations.** («Very dark grey to grey-black colored, very fined grained silty sand up to at certain places sandy silt, of a very low relative density, with a little at certain places gravels and clay, and with a lot of organic admixtures». According to the unified soil classification system «U.S.C.S.», as well as the British Standards: «British Soil Classification System for Engineering Purposes. B.S. 5930:1981», this soil type is characterized and classified as non cohesive up to slightly cohesive soil, due to the presence at certain places of silty and clayey binding material: «SM-SC up to at certain places CL-OL»). (N_{SPT} = 3 up to 13 with average value N = 6,2, w = 30,13 %, γ_{sat} = 19,07 kN/m^3, γ_d = 14,81 kN/m^3, γ_s = 25,70 kN/m^3, q_u = 41,14 kN/m^2, c_u = 9,27 kN/m^2, φ_u = 29,47 degrees, E_s = 21.917 kN/m^2, ν = 0,40, C_c = 0,27, e_o = 0,737, C_v = 0,955 x 10^{-4} cm^2/sec).	
3	Minimum: 0,00 11,00 Maximum:	**(Ground Category 3): Very soft to soft Clay of the Quaternary Holocene Alluvial deposits.** («Light chocolate brown colored, silty and sandy clay, of a medium plasticity, very soft to soft, with a few	

	13,20 20,50	gravels and at certain places thin intercalated pockets and layers, of a thickness up to 5 - 15 cm, of sand-gravels and pebbles». According to the unified soil classification system «U.S.C.S.», as well as the British Standards: «British Soil Classification System for Engineering Purposes. B.S. 5930:1981», this soil type is characterized and classified as cohesive soil: «CL-CH»). (N_{SPT} = 3 up to 40 with average value N = 15,0, w = 16,18 %, γ_{sat} = 20,34 kN/m^3, γ_d = 17,40 kN/m^3, γ_s = 26,12 kN/m^3, q_u = 175,11 KN/m^2, c_u = 92,50 KN/m^2, φ_u = 27,82 degrees, E_s = 19.732 kN/m^2, ν = 0,40, C_c = 0,13, e_o = 0,505, C_v = 1,463 x 10^{-4} cm^2/sec).	
4	Minimum: 11,00 50,00 Maximum: 20,50 50,00 (End of Investigation - sampling boreholes)	**(Ground Category 4): Coarse grained Quaternary Holocene Alluvial deposits.** («Grey-brown colored, silty and clayey sub-angular to rounded sand-gravels, of a moderate relative density up to high relative density, with a lot of cobbles and at certain places intercalated pockets, of a thickness of up to 15 - 40 cm clay layers». According to the unified soil classification system «U.S.C.S.», as well as the British Standards: «British Soil Classification System for Engineering Purposes. B.S. 5930:1981», this soil type is characterized and classified as non cohesive soil: «GC-GM»). (N_{SPT} = 19 up to REFUSAL, with average value N=46,4, w = 7,28 %, γ_{sat} = 20,00 kN/m^3, γ_d = 18,00 kN/m^3, c = 0,00 KN/m^2, φ = 40 degrees, E_s = 53.366,4 kN/m^2, ν = 0,25).	

(*) Depends on borehole / sampling location.

According to that soil model, the foundation soil layers up to a depth that ranges from 0,00 up to 17,00 m, consist of Shallow Artificial Embankments - (Man made backfilling material - Metallurgic furnace slag - Metals - Rubble), that are constituted of dark grey to grey-brown colored, slightly silty and clayey sands-gravels and pebbles, of a very high relative density (well compacted), containing metallurgic furnace slag as well as materials from cemented / conglomerated furnace slag, enclosing distributed metal objects of exceptionally hard materials of dimensions up to 1,50 m, such as steel items, iron blocks, metal alloys, etc, containing also at certain places demolition materials, such as tiles, timber, metals, and intercalations from non-reinforced concrete blocks. This layer, as a whole, constitutes an erratic, non-homogeneous and anisotropic foundation material.

Under Shallow Artificial Embankments - (Man made backfilling material - Metallurgic furnace slag - Metals - Rubble) from a depth that ranges from 3,00 to 13,50 m (or even directly on the surface of ground in certain locations), the upper layers of the Silty and Sandy Quaternary Holocene Alluvial deposits with clay intercalations are extended up to a depth that ranges from 8,70 m to 18,40 m, of a consistency of very dark grey to grey-black colored, very fined grained silty sand up to at certain places sandy silt, of a very low relative density, with a little at certain places gravels and clay, with a lot of organic admixtures.

Deeper, the intercalary layers of the very soft to soft Clay of the Quaternary Holocene Alluvial deposits are underlying, up to a depth that ranges from 11,00 to 20,50 m, consisting of light chocolate brown colored, silty and sandy clay, of a medium plasticity, very soft to soft, with a few gravels and at certain places thin intercalated pockets and layers, of a thickness up to 5 - 15 cm, of sand-gravels and pebbles.

Finally, under the very soft to soft Clay layers, the coarse grained Quaternary Holocene Alluvial deposits are underlying, up to a depth of 50,00 m, consisting of grey-brown colored, silty and clayey sub-angular to rounded sand-gravels, of a moderate relative density up to high relative density, with a lot of cobbles and at certain places intercalated pockets of a thickness of up to 15 - 40 cm clay layers.

From the above mentioned soils, the formations of the upper layers of the Silty and Sandy Quaternary Holocene Alluvial deposits with clay intercalations, are being subjected in a particularly increased danger of appearance to the liquefaction risk phenomenon (Liquefaction of foundation formations) that could be triggered of and caused by a potentially high seismic activity in the region, according to the findings of this research work as presented in a following section.

The Silty and Sandy Quaternary Holocene Alluvial deposits consist of a very low relative density, non cohesive up to slightly cohesive soil at certain places (due to the presence of silty and clayey binding material), constituted from very dark grey to grey-black colored, very fined grained silty sand up to at certain places sandy silt, with a little at certain places gravels and clay, and with a lot of organic admixtures. This soil type, according to the unified soil classification system «U.S.C.S.», as well as the British Standards: «British Soil Classification System for Engineering Purposes. B.S. 5930:1981», is characterized and classified as: «SM-SC up to at certain places CL-OL». The Standard Penetration Tests (S.P.T.) gave N-values, ranging from 3 to 13, with an average value equal to N = 6,2. According to the laboratory tests, its soil mechanics properties were determined and presented in table (2). Additionally, in fig. (4), an average and typical grain size distribution curve of the layer of Silty and Sandy Quaternary Holocene Alluvial deposits is depicted.

Table 2: Soil mechanics parameters and properties, showing the minimum, the maximum and the average value of each property of the layer of Silty and Sandy Quaternary Holocene Alluvial deposits.

Properties - Parameters - Coefficients	Sym bol	Min	Max	AVG
1. Grain size distribution.				
Content of Cobbles-Gravels (%)	P-G	0,00	29,82	6,12
Content of Sand (%)	S	7,11	61,85	34,87
Content of Silt - Clay (%)	M-C	28,20	92,89	59,01

2. Cohesiveness - Physical Properties	Sym bol	Min	Max	AVG
Liquid limit (%)	L.L.	24,57	53,86	37,34
Plastic limit (%)	P.L.	15,79	39,35	22,41
Plasticity Index (%)	P.I.	3,20	32,17	14,93
Moisture content (%)	W	12,77	71,91	30,13
Wet bulk density (gr/cm³)	γ_{sat}	1,895	1,994	1,944
Dry bulk density (gr/cm³)	γ_d	1,450	1,562	1,510
Soil particles specific gravity (gr/cm³)	γ_s	2,570	2,670	2,620
Voids Ratio	e	0,645	0,806	0,737

3. Mechanical Properties.	Sym bol	Min	Max	AVG
Unconfined (Uniaxial) Compression Strength				
Unconfined (Uniaxial) Compression Strength (kN/m^2)	q_u	13,25	94,62	41,14
Deformation (%)	ε	9,78	20,46	18,86
Direct Shearing Resistance				
U.U. Rapid unconsolidated shearing test.				

Cohesion (Total) (kN/m^2)	C_u	1,80	19,00	9,27
Angle of internal friction (Total) (^{o})	φ_u	27,50	31,30	29,47
Consolidation Test				
Compression Index	C_c	0,19	0,40	0,27
Coefficient of Consolidation (x 10^{-4} cm^2/sec)	C_v	0,648	1,159	0,955
Modulus of compressibility (kN/m^2) Loading range of 25 - 50 kN/m^2	E_s	3.920	6.380	5.463
Modulus of compressibility (kN/m^2) Loading range of 50 - 100 kN/m^2	E_s	7.460	11.950	9.360
Modulus of compressibility (kN/m^2) Loading range of 100 - 200 kN/m^2	E_s	12.530	39.450	21.917
Modulus of compressibility (kN/m^2) Loading range of 200 - 400 kN/m^2	E_s	20.520	31.930	26.653
Modulus of compressibility (kN/m^2) Loading range of 400 - 800 kN/m^2	E_s	30.060	80.230	57.233

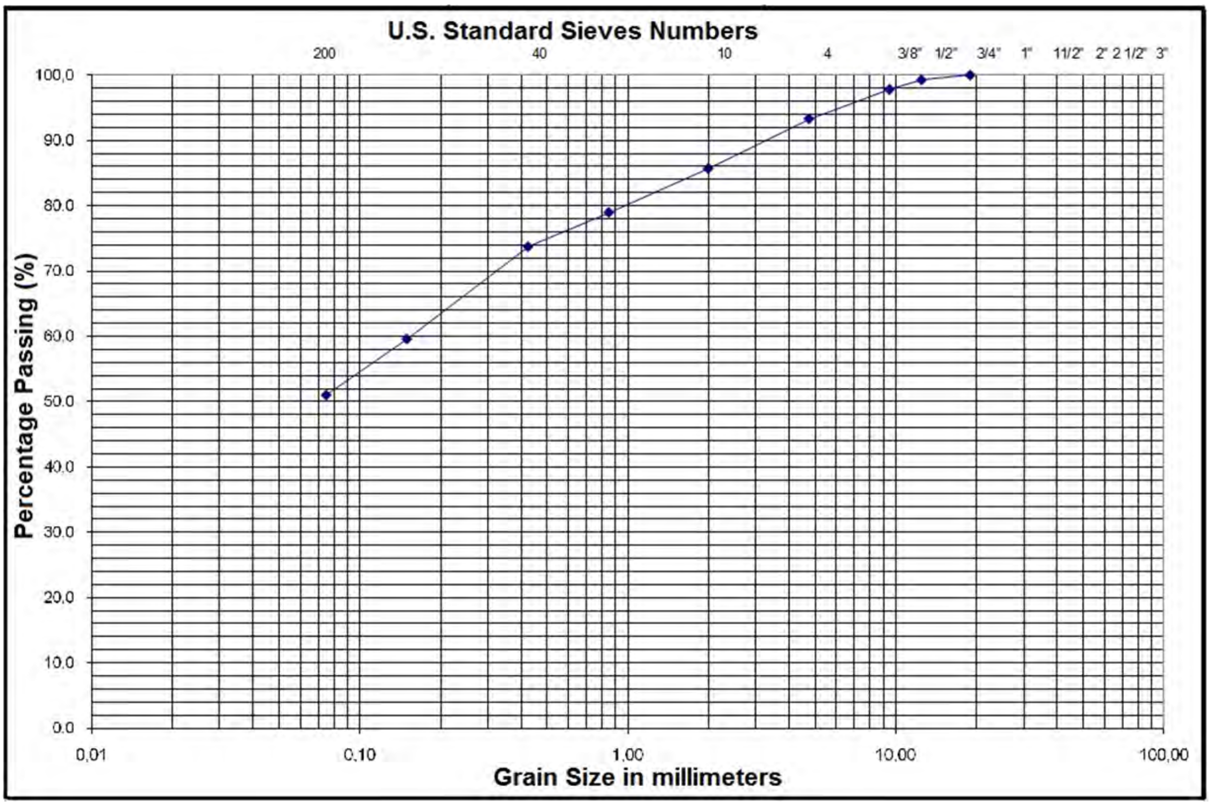


Figure 4: Average and typical Grain Size Distribution Curve of the layer of Silty and Sandy Quaternary Holocene Alluvial deposits.

Based upon the results of the Standard Penetration Tests (S.P.T.) that were executed gradually up to a depth range of 10 to 50 meters inside all investigation boreholes, an analysis and assessment of the liquefaction risk potential of the upper natural soil layers of foundation ground (Liquefaction of foundation formations) that could be triggered of and caused by a probable seismic activity with seismic magnitudes of 6,0 and 7,5 degrees on Richter scale was carried out. This analysis was based upon the procedure / relationships that was derived by Seed and Idriss (1971, 1981), Seed et al. (1979, 1984, 1986, 1990), and on the limit state methodology developed by Prakash (1981, 2003), using for this purpose S.P.T. results and an algorithm program / software code, that was developed and published by the author (Sachpazis C. I. (1992)).

From the results of this analysis, an assessment of the liquefaction risk potential and probability was determined concerning the CCGT Power Plant foundation ground, as well as the required special and suitable geotechnical / soil mechanics measures and interventions either for strengthening, geotechnically upgrading (ground improvement), stabilizing and cement grouting the foundation ground of the CCGT Power Plant using jet grouting piles techniques, or for transferring the project foundation loads to underlying deeper and more competent bearing strata and layers, with the construction of a pile group of various diameters and lengths, connected by concrete pile caps-beams-slabs. In this way, the dangerous phenomenon of potential foundation soil liquefaction failure will be mitigated or even deterred from the incompetent upper natural soil layers of the project foundation ground, in the event of a probable strong seismic shock and vibration.

CORRELATING S.P.T. N-VALUES TO GROUND LIQUEFACTION POTENTIAL BEHAVIOR

Research performed by Seed and Lee (1966), Finn et al (1970, 1976), Seed (1976, 1984, 1986, 1990), Casagrande (1976), Prakash and Gupta (1970), Prakash (1981, 2003), Seed and Idriss (1971, 1981), Seed et al (1979), proved that the factors influencing the liquefaction characteristics of a soil formation are:

1. Dimensions of depositions and their places of drainage system,
2. Structure and texture of the ground,
3. Loading history of the ground,
4. Grain size distribution of the ground,
5. Unit weight of the ground (density and gravity acceleration),
6. Size and type of imposing loads,
7. Moisture content, and
8. Content of trapped air.

According to Woods (1978), Seed (1979), and Prakash (1981) the Standard Penetration Test (S.P.T.) is a reliable method that could be used in an empirical way for the correlation with liquefaction risk probability of the ground.

Seed (1979) defined the relationship in between corrected with depth N-value of S.P.T. and cyclic stress ratio ($T_o/T_{av'}$) in relation to various seismic magnitudes (6,0 - 7,5 - 8,25 degrees on Richter scale) (see fig. (5)).

Based on this relationship, Prakash (1981) proposed the application of the following methodology for estimating the liquefaction risk probability of the ground that is subjected to natural or artificial vibrations. For the sake of analysis the following input data are required:

A. Execution of Standard Penetration Test (S.P.T.), according to ASTM D1586 or British Standard BS EN ISO 22476-3, and acquisition of test results for various depths. If necessary, the test results should be corrected according to Peck, Hanson and Thornburn (1974) proposals.
B. Depth of piezometric surface (water table) of the underground water bearing horizon in the area in question.
C. Unit weights of ground layers.
D. Maximum expected seismic magnitude.

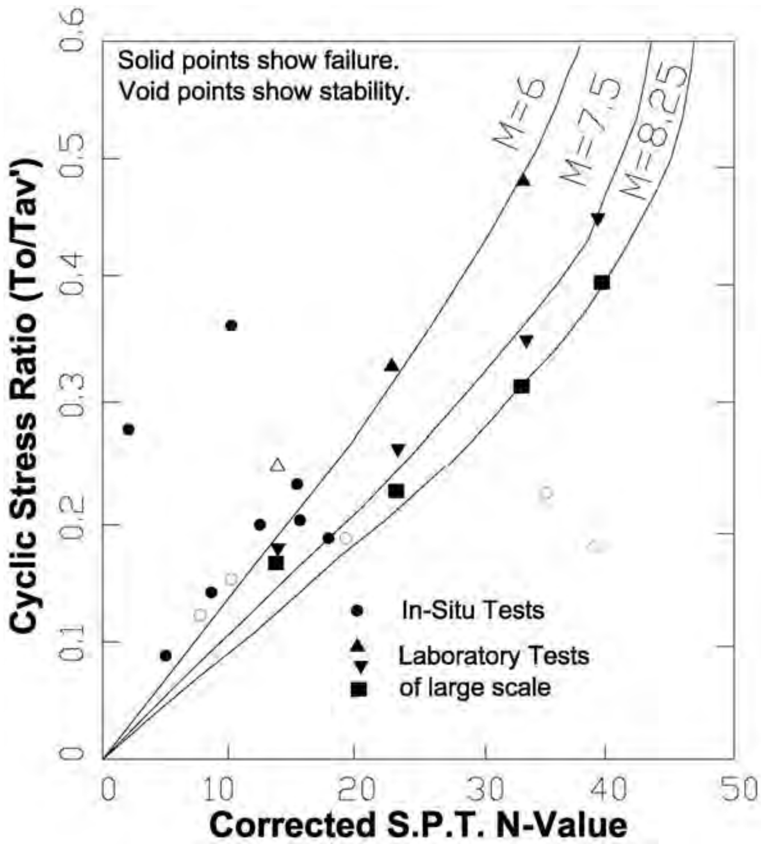


Figure 5: Relationship diagram between liquefaction behavior of the ground and Standard Penetration Test (S.T.P.). (after H.B. Seed, 1979).

The stages of the proposed methodology are:

1. Determination of the maximum shearing stress T_{av} that develops in foundation soil from expected earthquakes and that could likely cause liquefaction of the foundation ground, using the following relationship:

$$T_{av} = 0,65 \text{ x } (a_{max}) \text{ x } \gamma \text{ x } h \text{ x } R_d$$

where:

h	=	depth of overburden ground,
γ	=	unit weight of ground,
g	=	acceleration of gravity,
R_d	=	Factor dependant on depth, (see fig. (6)),
a_{max}	=	Maximum expected earthquake acceleration.

2. Determination of the cyclic stress ratio ($T_o/T_{av'}$), from diagram (A), and calculation of the shearing resistance (T_o) that develops in the foundation soil during the earthquake and resists the possibility of ground liquefaction, in relation to the maximum expected seismic intensities,

3. Comparison in between T_{av} and T_o shows whether liquefaction in the ground will or not occur for the granted characteristics of the location and the elevation that the analysis is

materialized. If $T_{av} > T_o$, then there is a danger for ground liquefaction, whilst if $T_{av} < T_o$, there is not.

Based on the above mentioned procedure and methodology an algorithm program / software code «Liquefac.bas» was developed and published by the author (Sachpazis C. I. (1992) and used in the present research work.

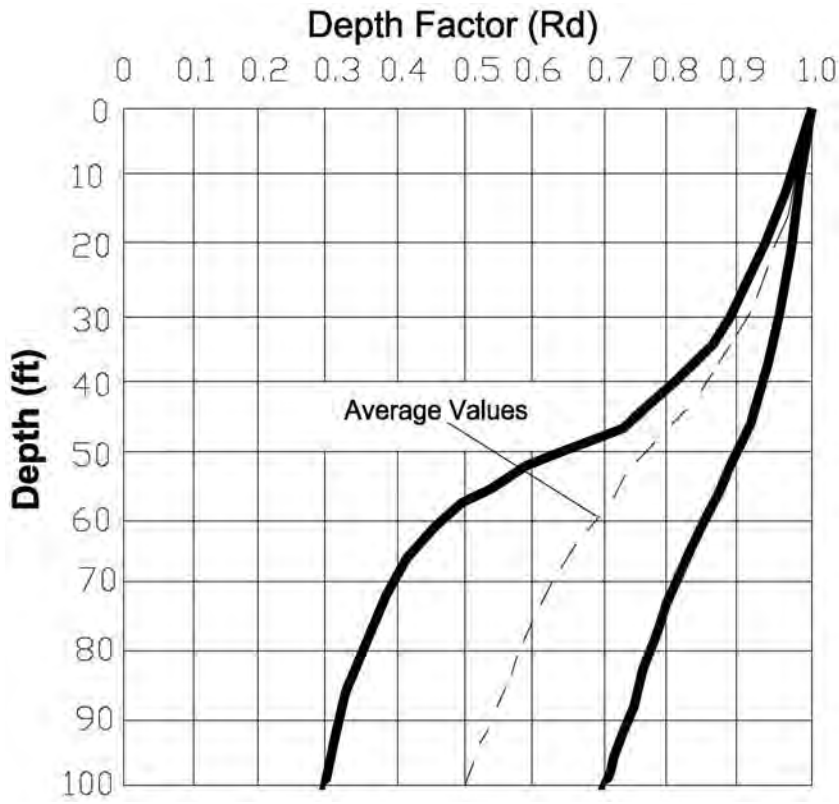


Figure 6: Diagram showing values of factor R_d for various types of soils in Liquefaction Risk Analysis (after S. Prakash, 1981).

PROBABILITY ANALYSIS RESULTS OF SOIL LIQUEFACTION RISK

In the following table: 3, the respective results of the Standard Penetration Tests (S.P.T.) values are presented as executed in the twenty (20) investigation - sampling boreholes (B1, B2, B3, B4, B5, B6, B7, B9, B10, B12, B15, B17, B18, B19, B20, B21, B22, B23, B24, and B25) in the area that the foundation of the CCGT Power Plant is planned to be constructed.

Table 3: Standard Penetration Tests (S.P.T.) results in the investigation - sampling boreholes B-1 to B-25.

BH # / TEST #	Depth of executed Test S.P.T. (m)		Average Depth of executed S.P.T. (m)	S.P.T. result (N)
	From	To		
B-1/1	11,50	11,95	11,73	42
B-1/2	17,60	18,05	17,83	19
B-1/3	20,30	20,75	20,53	80
B-1/4	23,00	23,25	23,13	Refusal
B-2/1	13,00	13,45	13,23	23
B-2/2	16,00	16,45	16,23	60
B-2/3	19,30	19,75	19,53	60
B-2/4	22,60	23,05	22,83	67
B-3/1	14,10	14,55	14,33	13
B-3/2	17,60	18,05	17,83	29
B-3/3	20,40	20,85	20,63	38
B-3/4	23,20	23,65	23,43	68
B-4/1	12,30	12,75	12,53	61
B-4/2	15,50	15,95	15,73	13
B-4/3	18,50	18,95	18,73	39
B-4/4	21,50	21,95	21,73	50
B-4/5	24,60	25,05	24,83	11
B-5/1	14,60	15,05	14,83	3
B-5/2	17,50	17,75	17,63	Refusal
B-5/3	20,20	20,65	20,43	41
B-5/4	23,00	23,30	23,15	Refusal
B-6/1	14,00	14,45	14,23	56
B-6/2	17,60	18,05	17,83	8
B-6/3	20,60	21,05	20,83	58
B-6/4	24,00	24,45	24,23	5
B-7/1	18,50	18,95	18,73	70
B-7/2	21,70	22,15	21,93	57
B-7/3	24,60	25,05	24,83	29
B-9/1	12,80	13,25	13,03	7
B-9/2	16,00	16,45	16,23	30
B-9/3	19,00	19,45	19,23	25
B-9/4	22,00	22,45	22,23	55
B-10/1	19,50	19,95	19,73	39
B-10/2	22,50	22,95	22,73	39
B-12/1	16,00	16,45	16,23	18
B-12/2	19,00	19,45	19,23	57
B-12/3	22,00	22,45	22,23	60
B-12/4	24,60	25,05	24,83	47

B-15/1	12,30	12,71	12,51	Refusal
B-15/2	15,30	15,37	15,34	Refusal
B-15/3	18,40	18,85	18,63	17
B-15/4	21,40	21,84	21,62	Refusal
B-15/5	23,40	23,85	23,63	18
B-17/1	14,60	15,05	14,83	13
B-17/2	17,50	17,95	17,73	10
B-17/3	20,50	20,95	20,73	40
B-17/4	23,50	23,95	23,73	43
B-18/1	8,10	8,55	8,33	3

Table (3) Cont'd.

BH # / TEST #	Depth of executed Test S.P.T. (m)		Average Depth of executed S.P.T. (m)	S.P.T. result (N)
	From	To		
B-19/1	4,50	4,95	4,73	9
B-19/2	7,20	7,65	7,43	4
B-19/3	9,60	10,05	9,83	19
B-20/1	6,50	6,95	6,73	3
B-20/2	8,00	8,45	8,23	8
B-21/1	3,60	4,05	3,83	5
B-21/2	6,70	7,15	6,93	4
B-21/3	9,60	10,05	9,83	52
B-22/1	14,60	15,05	14,83	20
B-22/2	17,50	17,95	17,73	6
B-22/3	20,50	20,95	20,73	57
B-22/4	23,00	23,45	23,23	68
B-23/1	17,10	17,55	17,33	16
B-23/2	20,50	20,95	20,73	43
B-23/3	24,00	24,45	24,23	66
B-23/4	27,00	27,09	27,05	Refusal
B-23/5	30,00	30,22	30,11	Refusal
B-23/6	33,00	33,14	33,07	Refusal
B-23/7	35,80	35,91	35,86	Refusal
B-23/8	38,90	39,30	39,10	Refusal
B-23/9	42,10	42,37	42,24	Refusal
B-23/10	45,60	45,67	45,64	Refusal
B-23/11	48,30	48,43	48,37	Refusal
B-24/1	13,10	13,55	13,33	3
B-24/2	16,00	16,45	16,23	24
B-24/3	19,00	19,45	19,23	19
B-24/4	22,50	22,95	22,73	48
B-25/1	5,60	6,05	5,83	7
B-25/2	9,10	9,55	9,33	5
B-25/3	12,00	12,45	12,23	45
B-25/4	15,00	15,45	15,23	25
B-25/5	18,00	18,45	18,23	42
B-25/6	21,00	21,45	21,23	12
B-25/7	23,50	23,95	23,73	19

The depth of piezometric surface (water table) of the underground water bearing horizon was determined nearby the ground surface, equal to 1,00 meter below ground surface level. The average unit weight of the ground is taken equal to 18,00 KN/m^3. For the analysis and check of the liquefaction risk probability of the Quaternary Holocene Alluvial deposits in the foundation area of the CCGT Power Plant, two seismic intensity magnitudes, equal to (6,0) and (7,5) degrees on the Richter scale respectively, were used. The ratio (a_{max}/g) is taken for the study region, equal to 0,084 gs (average value between Drakopoulos, J, and Markopoulos C. (1982) proposed value and the value derived from empirical formula by Galanopoulos A. G. (1988)).

In the following Table 4, the concentrative-summary results of all analyses for the liquefaction risk probability of the shallow ground formations of the Quaternary Holocene Alluvial deposits are presented tabulated as executed in the twenty investigation - sampling boreholes (B1, B2, B3, B4, B5, B6, B7, B9, B10, B12, B15, B17, B18, B19, B20, B21, B22, B23, B24, and B25) in all examined depths (elevations).

Table 4: Concentrative results of liquefaction analysis and assessment in the locations of boreholes B-1 to B-25.

BH # / Elevation #	Mean S.P.T. Depth (m)	S.P.T. N-Value	Tav (KPa)	To (KPa) in 6 Richter	To (KPa) in 7,5 Richter	Liquefaction Risk?
B-1/1st	11,73	42	9,942276	68,69435	50,37429	NO / NO
B-1/2nd	17,83	19	12,24333	38,55131	28,77315	NO / NO
B-1/3rd	20,53	80	12,96431	289,5059	260,6366	NO / NO
B-1/4th	23,13	Refusal	13,60326	426,5458	395,9147	NO / NO
B-2/1st	13,23	23	10,71166	35,26805	26,32626	NO / NO
B-2/2nd	16,23	60	11,81451	158,4367	134,2848	NO / NO
B-2/3rd	19,53	60	12,71066	190,6512	161,5886	NO / NO
B-2/4th	22,83	67	13,53283	257,5416	224,2226	NO / NO
B-3/1st	14,33	13	11,16178	21,33654	15,82202	NO / NO
B-3/2nd	17,83	29	12,24333	61,48789	45,86409	NO / NO
B-3/3rd	20,63	38	12,98755	102,9101	73,71677	NO / NO
B-3/4th	23,43	68	13,67091	269,394	235,2953	NO / NO
B-4/1st	12,53	61	10,39001	125,0362	106,4416	NO / NO
B-4/2nd	15,73	13	11,65342	23,42106	17,36778	NO / NO
B-4/3rd	18,73	39	12,47989	97,49622	69,54083	NO / NO
B-4/4th	21,73	50	13,25073	164,9774	131,7511	NO / NO
B-4/5th	24,83	11	13,98309	31,39843	23,19722	NO / NO
B-5/1st	14,83	3	11,34402	5,126729	3,778549	YES / YES
B-5/2nd	17,63	Refusal	12,19695	325,119	301,7715	NO / NO
B-5/3rd	20,43	41	12,94069	115,2112	83,21971	NO / NO
B-5/4th	23,15	Refusal	13,60785	426,9146	396,257	NO / NO
B-6/1st	14,23	56	11,12365	126,5622	105,1534	NO / NO
B-6/2nd	17,83	8	12,24333	16,43688	12,11446	NO / YES
B-6/3rd	20,83	58	13,03286	194,3023	163,1346	NO / NO
B-6/4th	24,23	5	13,83768	13,96051	10,2893	NO / YES
B-7/1st	18,73	70	12,47989	223,4823	196,3772	NO / NO

B-7/2nd	21,93	57	13,30481	199,8047	166,9012	NO / NO
B-7/3rd	24,83	29	13,98309	85,62783	63,87019	NO / NO
B-9/1st	13,03	7	10,62255	10,51043	7,746491	YES / YES
B-9/2nd	16,23	30	11,81451	58,08765	43,32382	NO / NO
B-9/3rd	19,23	25	12,62702	56,28037	41,99884	NO / NO
B-9/4th	22,23	55	13,3836	192,891	159,3553	NO / NO
B-10/1st	19,73	39	12,76449	102,7016	73,25364	NO / NO
B-10/2nd	22,73	39	13,50873	118,3176	84,39206	NO / NO
B-12/1st	16,23	18	11,81451	33,2708	24,8126	NO / NO
B-12/2nd	19,23	57	12,62702	175,205	146,3525	NO / NO
B-12/3rd	22,23	60	13,3836	217,0085	183,928	NO / NO
B-12/4th	24,83	47	13,98309	172,3501	134,0787	NO / NO
B-15/1st	12,51	Refusal	10,38042	267,3698	214,1328	NO / NO
B-15/2nd	15,34	Refusal	11,51881	282,8886	262,5738	NO / NO
B-15/3rd	18,63	17	12,4493	36,10035	26,89934	NO / NO
B-15/4th	21,62	Refusal	13,22047	398,6996	370,0681	NO / NO
B-15/5th	23,63	18	13,71446	48,44048	36,1258	NO / NO
B-17/1st	14,83	13	11,34402	22,08101	16,37407	NO / NO
B-17/2nd	17,73	10	12,2204	20,43086	15,05814	NO / NO
B-17/3rd	20,73	40	13,01039	112,405	79,8588	NO / NO
B-17/4th	23,73	43	13,73577	144,1189	107,1542	NO / NO
B-18/1st	8,33	3	7,564661	2,87968	2,122408	YES / YES

Table (4). Cont'd.

BH # / Elevation #	Mean S.P.T. Depth (m)	S.P.T. N-Value	Tav (KPa)	To (KPa) in 6 Richter	To (KPa) in 7,5 Richter	Liquefaction Risk?
B-19/1st	4,73	9	4,501013	4,905481	3,615482	NO / YES
B-19/2nd	7,43	4	6,833596	3,424733	2,524128	YES / YES
B-19/3rd	9,83	19	8,693183	21,25403	15,86316	NO / NO
B-20/1st	6,73	3	6,250546	2,32656	1,714743	YES / YES
B-20/2nd	8,23	8	7,484462	7,586959	5,59181	NO / YES
B-21/1st	3,83	5	3,681559	2,206718	2,206718	YES / YES
B-21/2nd	6,93	4	6,418421	3,194267	3,194267	YES / YES
B-21/3rd	9,83	52	8,693183	78,8967	78,8967	NO / NO
B-22/1st	14,83	20	11,34402	33,7288	25,19154	NO / NO
B-22/2nd	17,73	6	12,2204	12,25852	9,034885	NO / YES
B-22/3rd	20,73	57	13,01039	188,8715	157,7684	NO / NO
B-22/4th	23,23	68	13,62612	267,0944	233,2868	NO / NO
B-23/1st	17,33	16	12,12351	31,6368	23,55032	NO / NO
B-23/2nd	20,73	43	13,01039	125,899	93,60756	NO / NO
B-23/3rd	24,23	66	13,83768	268,0773	232,6159	NO / NO
B-23/4th	27,05	Refusal	14,48459	498,8355	463,0131	NO / NO
B-23/5th	30,11	Refusal	14,93866	555,0813	515,3909	NO / NO
B-23/6th	33,07	Refusal	15,59068	609,8518	566,057	NO / NO
B-23/7th	35,86	Refusal	16,20526	661,3028	613,8133	NO / NO
B-23/8th	39,1	Refusal	16,91896	721,0524	669,2721	NO / NO
B-23/9th	42,24	Refusal	17,61063	778,9579	723,0194	NO / NO
B-23/10th	45,64	Refusal	18,35957	841,6581	781,2169	NO / NO
B-23/11th	48,37	Refusal	18,96093	892,0026	827,9461	NO / NO
B-24/1st	13,33	3	10,75537	4,608179	3,396362	YES / YES
B-24/2nd	16,23	24	11,81451	45,38281	33,87135	NO / NO
B-24/3rd	19,23	19	12,62702	41,57833	31,0324	NO / NO
B-24/4th	22,73	48	13,50873	162,7056	127,7641	NO / NO
B-25/1st	5,83	7	5,478972	4,702671	3,466005	YES / YES
B-25/2nd	9,33	5	8,341258	5,375633	3,961998	YES / YES
B-25/3rd	12,23	45	10,24377	79,58356	60,63284	NO / NO
B-25/4th	15,23	25	11,47975	44,57359	33,26273	NO / NO
B-25/5th	18,23	42	12,3299	106,7603	78,28845	NO / NO
B-25/6th	21,23	12	13,11884	29,22817	21,63719	NO / NO
B-25/7th	23,73	19	13,73577	51,30806	38,29427	NO / NO

DISCUSSION AND CONCLUSIONS

It is well known that the vibrations triggered by the seismic waves, cause to non cohesive, and especially to loose sandy soils, a closer grain packing arrangement, resulting in most cases in the development of very high and not accepted total or differential settlements. In particular, when this soil type is saturated the liquefaction risk dramatically increases.

Out of three distinct types of seismic elastic waves (P, S and R), R waves are shallow - surficial and transmit 67% of the released seismic energy (after Miller and Pursey, 1954, 1955) and hence they are most dangerous for the foundations upon or close to the ground surface, causing a high liquefaction risk especially in non cohesive loose and saturated soils.

Seed and Idriss (1971, 1981) and Seed et al. (1979, 1984, 1986, 1990) followed by additions and modifications by Prakash (1981, 2003), proposed a methodology by which the risk of development of soil liquefaction failure in non cohesive loose and saturated soils can be estimated, using in-situ dynamic tests, such as Standard Penetration Test (S.P.T.) performed according to ASTM D1586 or British Standard BS EN ISO 22476-3.

As resulted from the in-situ investigations and tests, the area where the anticipated CCGT Power Plant is planned to be founded, is structured in its upper surface layers from the formations of artificial embankments (Man made backfilling material - Metallurgic furnace slag - Metals - Rubble), as well as from Quaternary Holocene Alluvial deposits that are mainly being in a loose state and in a very low relative density. The underlying layers of this depositions are constituted from the lower layers of the coarse grained Quaternary Holocene Alluvial deposits, of a consistency of grey-brown to flesh colored at certain places, silty and clayey sub-angular up to rounded sand-gravels, of a moderate relative density up to a high relative density, with a lot of cobbles and with at certain places intercalated pockets of clay layers, of an up to 15 - 40 cm thickness, that were encountered in the investigation - sampling boreholes in a depth that ranges from 11,00 m up to 20,50 m, as resulted from the elements and data of all twenty (20) investigation - sampling boreholes: B1, B2, B3, B4, B5, B6, B7, B9, B10, B12, B15, B17, B18, B19, B20, B21, B22, B23, B24, and B25.

In the study area, there were executed in-situ dynamic tests, using the Standard Penetration Test (S.P.T.), according to ASTM D1586 or British Standard BS EN ISO 22476-3, and tables of the variation of N number value of S.P.T. test versus depth in each borehole were compiled.

According to the N number values and the methodology developed by Seed and Idriss (1971), Seed (1979), and Prakash (1981), it became eventually possible to be calculated, in various control depths of each borehole, both the shearing resistance (T_o) that is developed in the foundation soil during the earthquake and resists the possibility of ground liquefaction in relation to the maximum expected seismic intensities, and the shearing stresses (T_{av}) that is developed in the ground in the same depths during the seismic shock and potentially cause liquefaction of the foundation ground.

The results of this liquefaction analysis are diagrammatically presented in the following twenty figures, fig. (C1), fig. (C2), fig. (C3), fig. (C4), fig. (C5), fig. (C6), fig. (C7), fig. (C9), fig. (C10), fig. (C12), fig. (C15), fig. (C17), fig. (C18), fig. (C19), fig. (C20), fig. (C21), fig. (C22), fig. (C23), fig. (C24), and fig. (C25), for each borehole respectively. However, for the sake of limiting and decreasing the presentation space in this paper, it was decided to be presented only two characteristic and representative liquefaction analysis diagrams out of the twenty diagrams as mentioned above. These chosen diagrams are fig. (C4) and fig. (C25). The rest of them are only presented in fig. (C26), as thumbnails in a low resolution and size.

In these figures, both the shearing resistances (T_o) that are developed in the foundation soil in various control elevations (depths) during the earthquake and resist the possibility of ground liquefaction, in two (2) maximum examined seismic intensities 6,0 and 7,5 Richter, and the shearing resistances (T_{av}) that are developed in the ground in the same elevations (depths) during the seismic shock and potentially cause liquefaction of the foundation ground, are presented. From these figures, it is resulted and concluded that the territorial area where the CCGT Power Plant is planned to be founded, up to a maximum depth equal to 18,40 meters, it is not safe in terms of the probability and risk for development of the liquefaction phenomenon in the upper natural soil layers of the foundation ground (Liquefaction risk of foundation formations) in possible seismic intensities equal to M = 6,0 and 7,5 degrees on Richter scale respectively. Specifically speaking, it is proved that there exists an upper soil zone that is prone and dangerous to be liquefied in the event of a strong seismic shock and vibration, and that is developed in depth that ranges from place to place between 4,00 m and 18,40 m approximately, under the surface of the artificial (man made) ground as it has been shaped out recently after the embankments and backfilling works.

Using the soil liquefaction assessment methodology as described above, the following boreholes have been identified to present high potential and risk for soil liquefaction hazards: B5 B6, B9, B18, B19, B20, B21, B22, B24 and B25,. i.e. nearly half of all the boreholes executed. The majority of these locations are directly beneath the proposed CCGT Power Plant foundations.

According to the above mentioned analysis and assessments it is conclusively deducted that the area on which the CCGT Power Plant is going to be founded, is not safe in terms of the probability and risk for development of the liquefaction phenomenon of the foundation ground, i.e. presents high liquefaction risk. Therefore, there should be executed special and suitable geotechnical / soil mechanics measures and interventions either for strengthening, geotechnically upgrading (ground improvement), stabilizing and cement grouting the foundation ground of the CCGT Power Plant, using jet grouting piles techniques, or for completely avoiding the placement of the foundations of the project structures on the upper layers of the ground formations by transferring the project foundation loads to underlying deeper and more competent bearing strata and layers, with the construction of a pile group of various diameters and lengths, connected by concrete pile caps-beams-slabs system. By this way, the dangerous phenomenon of potential foundation soil liquefaction failure can be mitigated or even deterred from the incompetent upper natural soil layers of the project foundation ground, in the event of a probable strong seismic shock and vibrations.

Finally, it should be pointed out that should the transferring of the project foundation loads to underlying layers with the construction of a pile group be chosen, the effects of pile negative skin friction and/or loss of lateral support (buckling) in deep foundations should be considered and taken seriously into account by the responsible foundation design engineer, due to the potential soil liquefaction risk.

Fig. (C4). Diagram of Foundation Ground Liquefaction Risk Analysis in the location of Borehole B4.

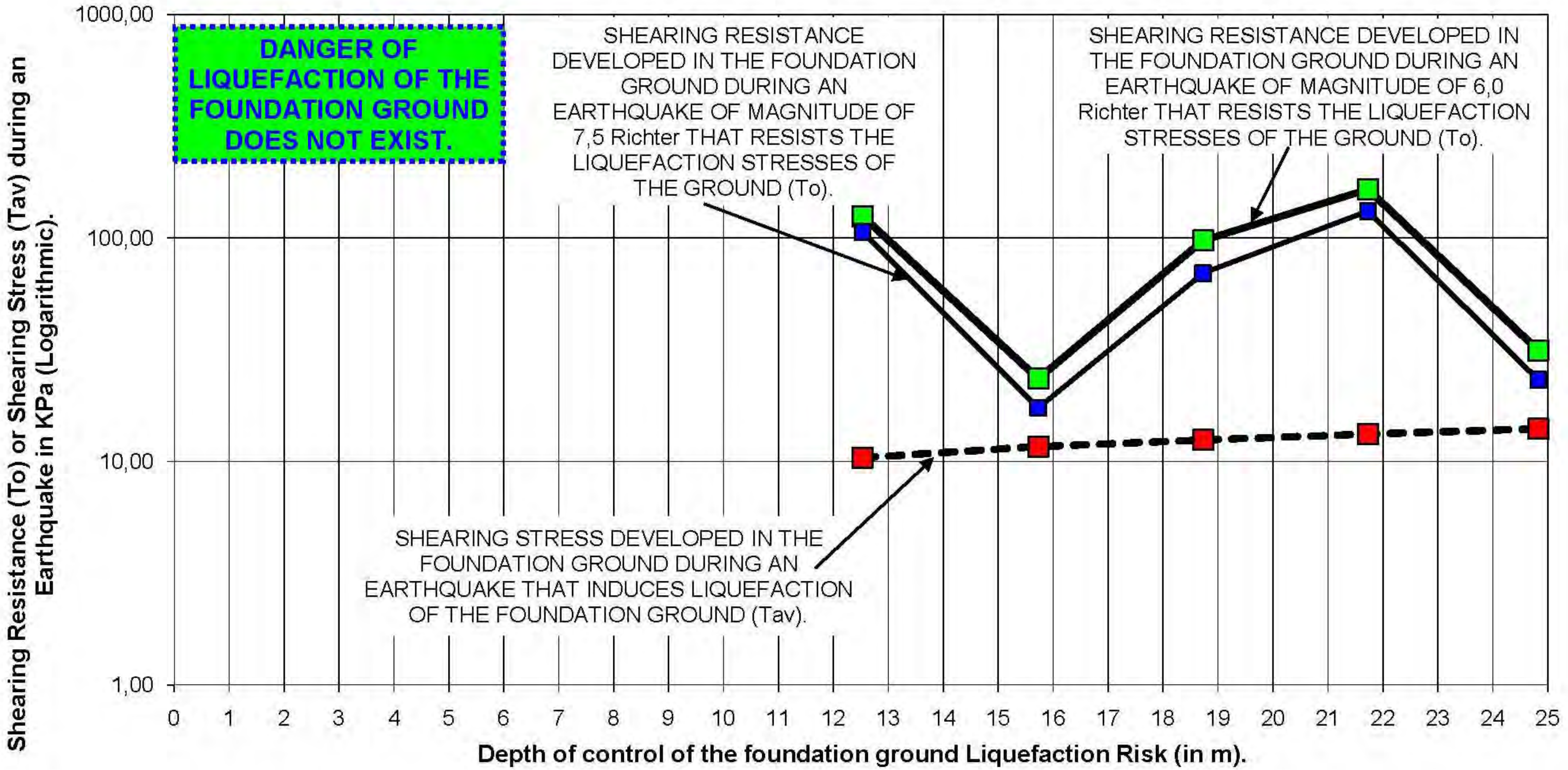


Fig. (C25). Diagram of Foundation Ground Liquefaction Risk Analysis in the location of Borehole B25.

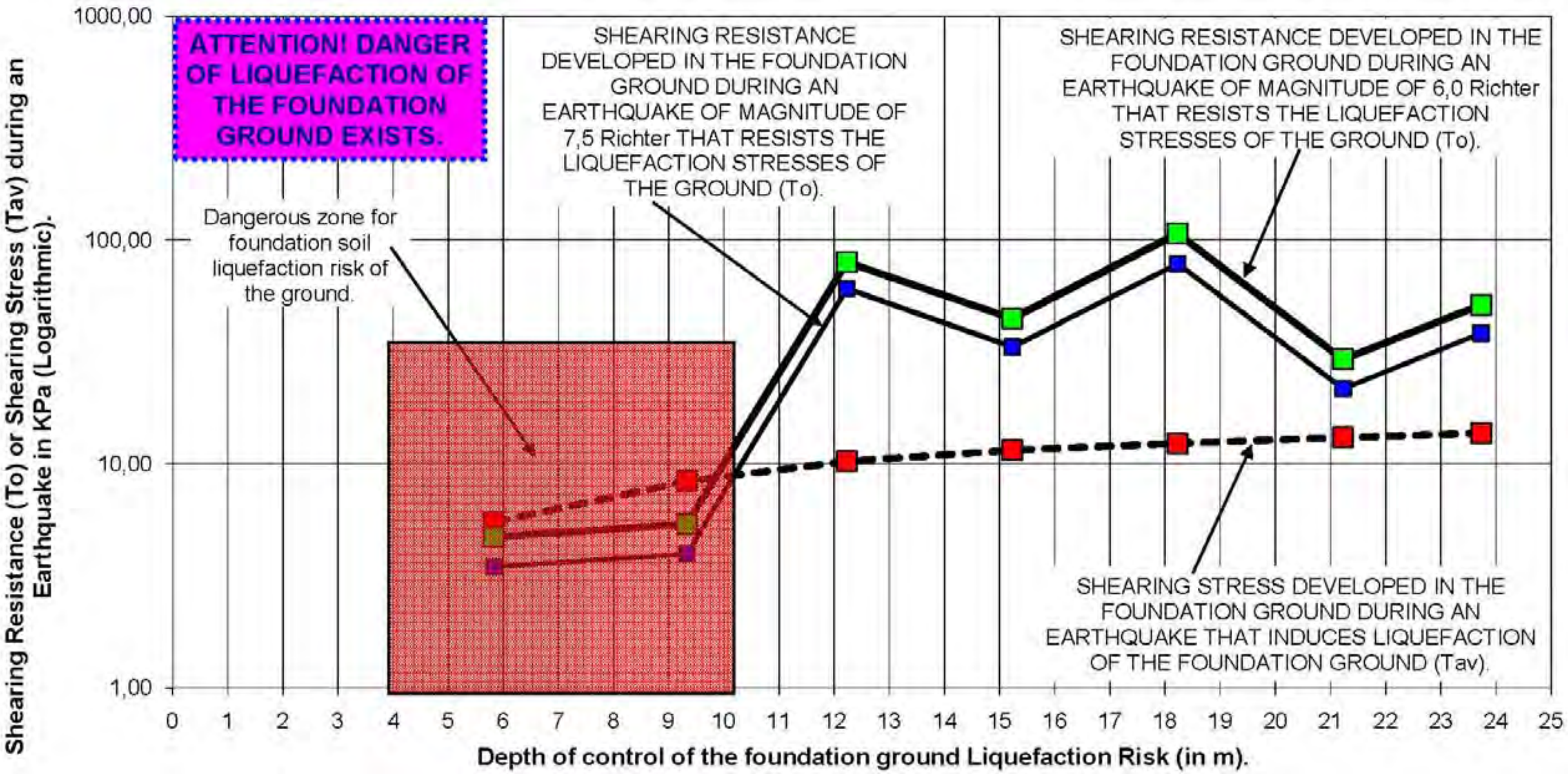


Fig. (C26). Liquefaction analysis diagrams presented as thumbnails in a low resolution and size for decreasing the presentation space reasons.

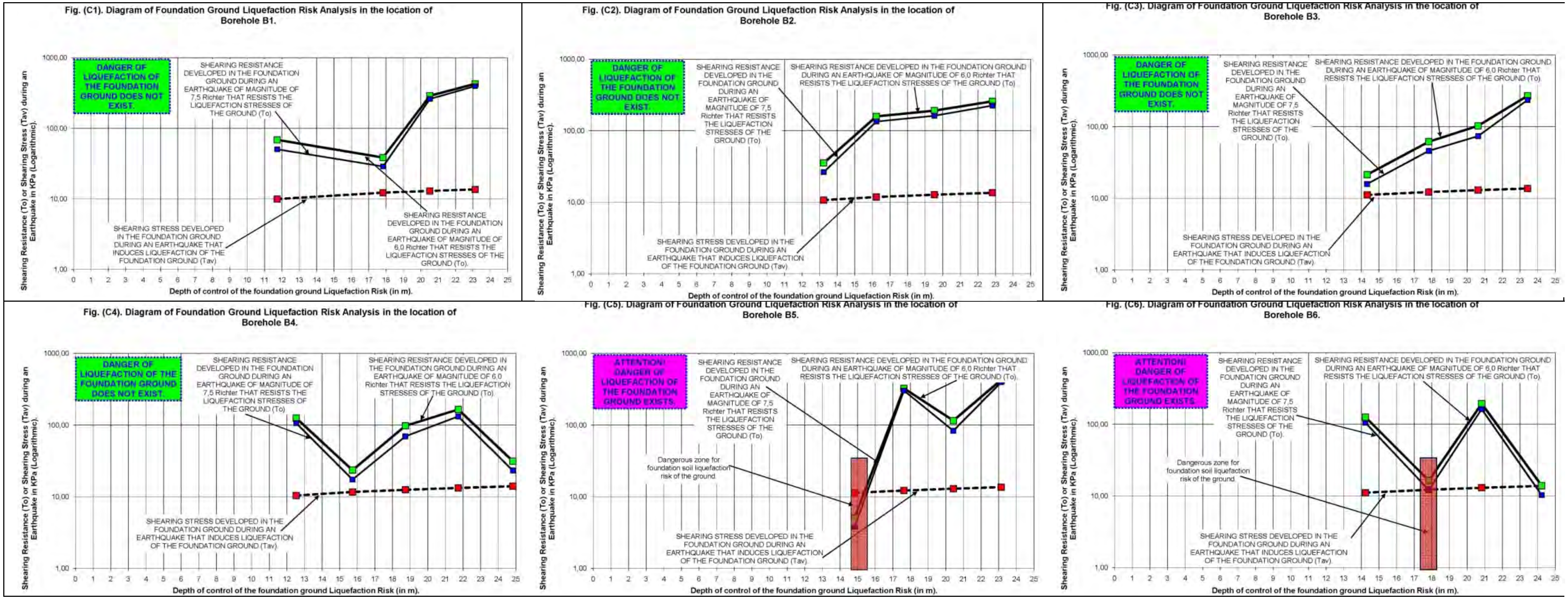


Fig. (C7). Diagram of Foundation Ground Liquefaction Risk Analysis in the location of Borehole B7.

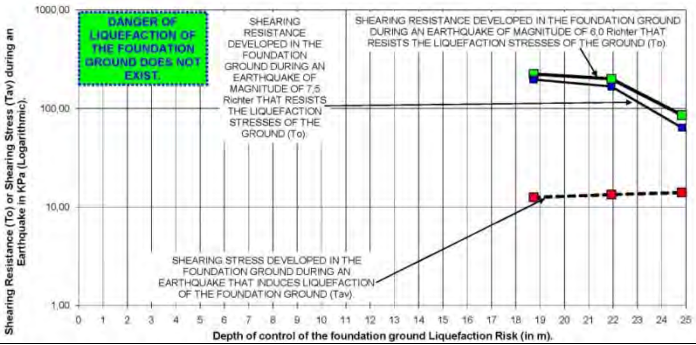


Fig. (C9). Diagram of Foundation Ground Liquefaction Risk Analysis in the location of Borehole B9.

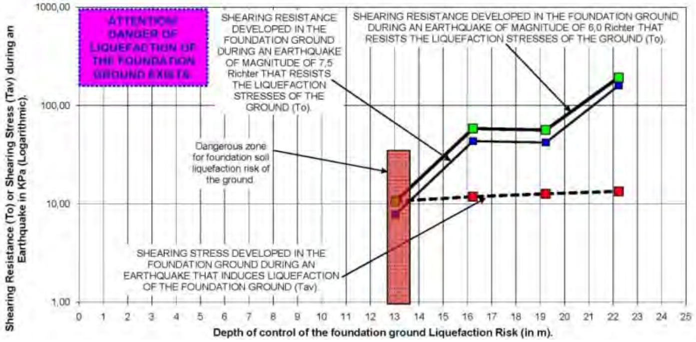


Fig. (C10). Diagram of Foundation Ground Liquefaction Risk Analysis in the location of Borehole B10.

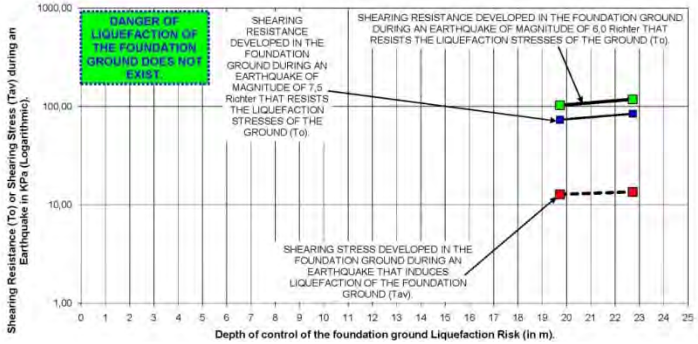


Fig. (C12). Diagram of Foundation Ground Liquefaction Risk Analysis in the location of Borehole B12.

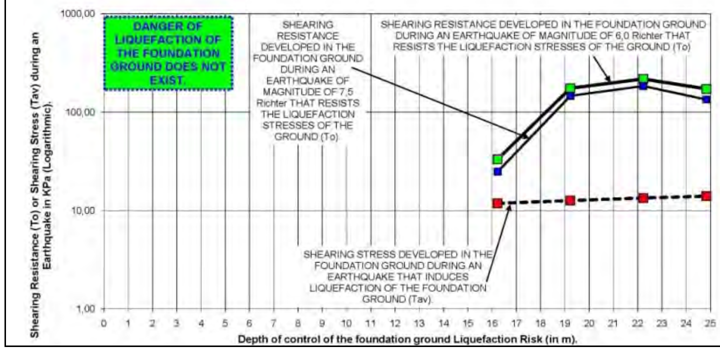


Fig. (C15). Diagram of Foundation Ground Liquefaction Risk Analysis in the location of Borehole B15.

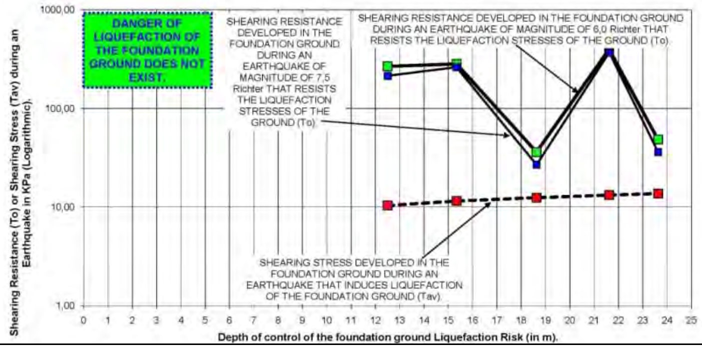


Fig. (C17). Diagram of Foundation Ground Liquefaction Risk Analysis in the location of Borehole B17.

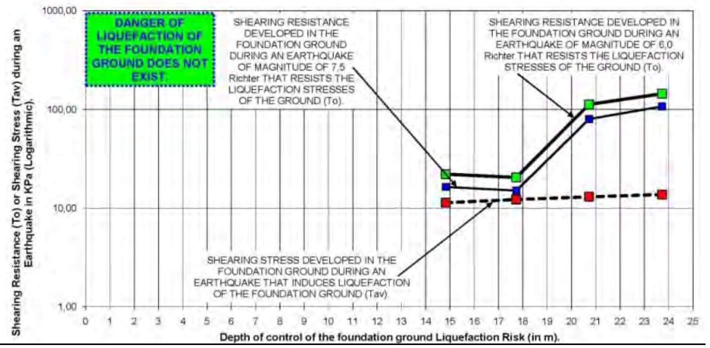


Fig. (C18). Diagram of Foundation Ground Liquefaction Risk Analysis in the location of Borehole B18.

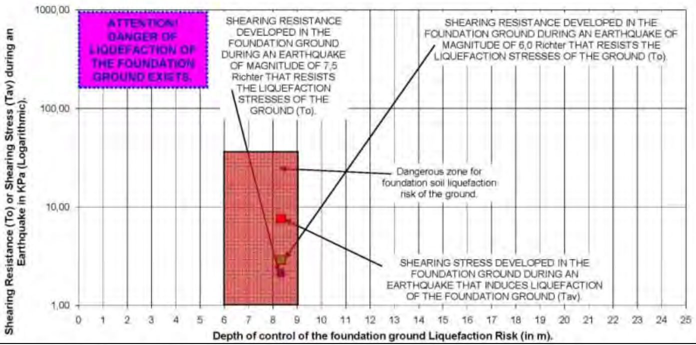


Fig. (C19). Diagram of Foundation Ground Liquefaction Risk Analysis in the location of Borehole B19.

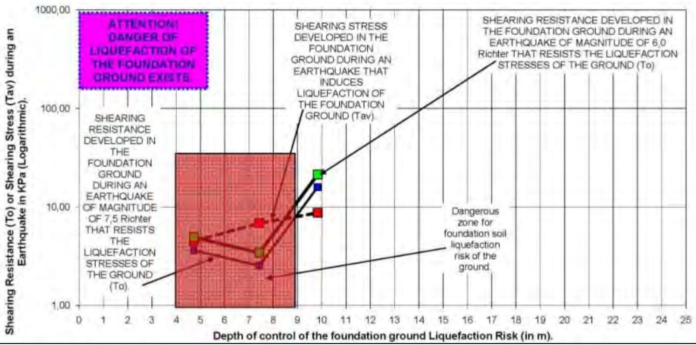


Fig. (C20). Diagram of Foundation Ground Liquefaction Risk Analysis in the location of Borehole B20.

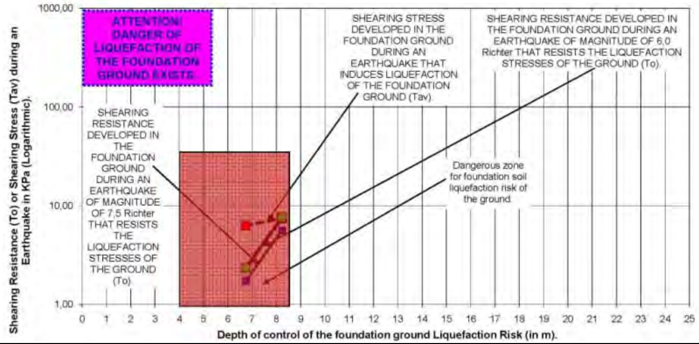


Fig. (C21). Diagram of Foundation Ground Liquefaction Risk Analysis in the location of Borehole B21.

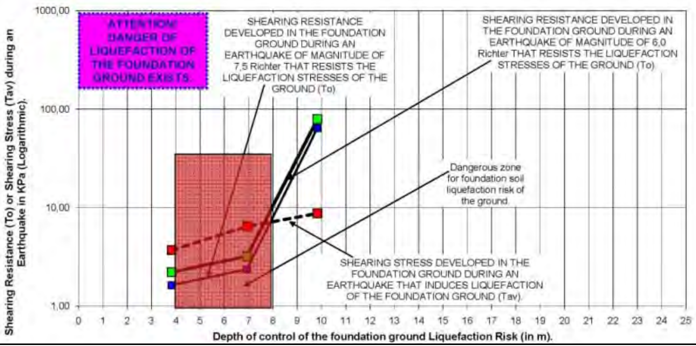


Fig. (C22). Diagram of Foundation Ground Liquefaction Risk Analysis in the location of Borehole B22.

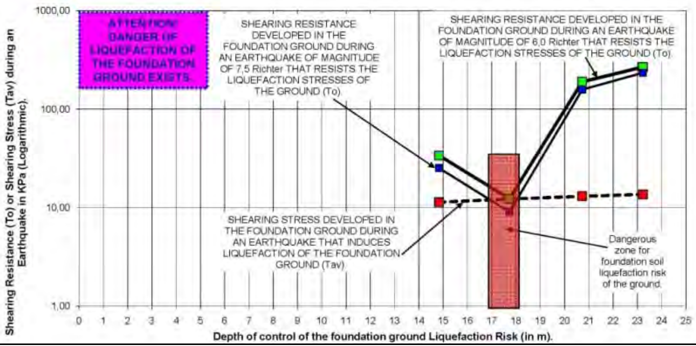


Fig. (C23). Diagram of Foundation Ground Liquefaction Risk Analysis in the location of Borehole B23.

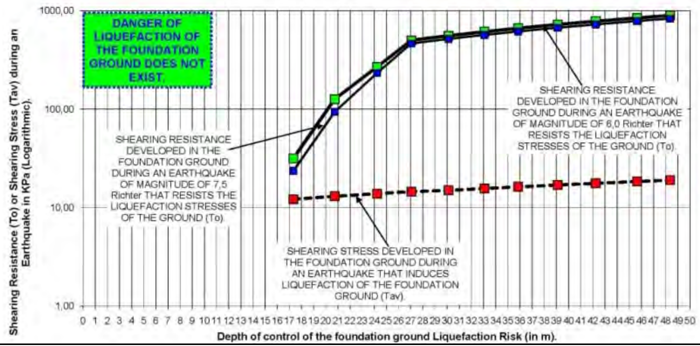


Fig. (C24). Diagram of Foundation Ground Liquefaction Risk Analysis in the location of Borehole B24.

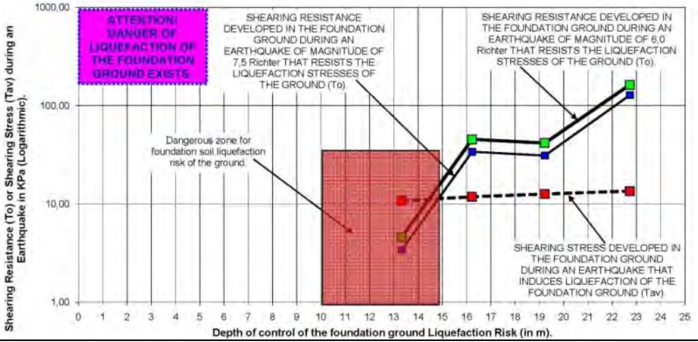
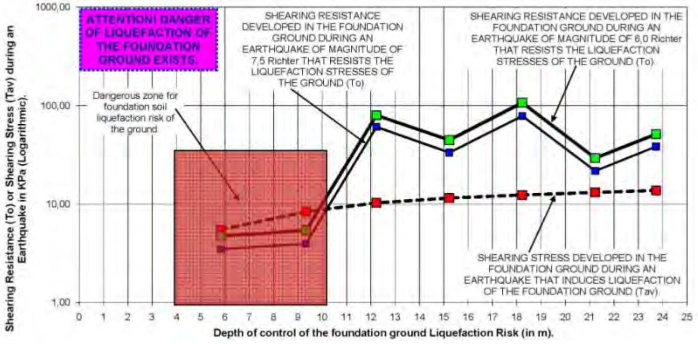


Fig. (C25). Diagram of Foundation Ground Liquefaction Risk Analysis in the location of Borehole B25.



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