EVALUATION OF SAND LIQUEFACTION ALONG THE ADRIATIC COAST IN ALBANIA: A CASE STUDY FROM DURRES AREA

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EVALUATION OF SAND LIQUEFACTION ALONG THE ADRIATIC COAST IN ALBANIA: A CASE STUDY FROM DURRES AREA

By

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Abstract

The phenomenon of liquefaction is considered one of the most difficult problems of Geotechnical seismic, but has over 45 years that different scientists from around the world are dealing with this problems and issues are very revealing about in terms of terminology and the problems associated with liquefaction phenomenon that occurs during vibrations mainly land

without cohesion and saturated with fast water. Their quick loading makes an immediate rise in pressure in the pore water, reduce pressure on the soil skeleton doing not resist more cuts.

Albania as a country with high seismic activity there is possibility of infection from this phenomenon. Potentially this phenomenon is vulnerable coastal part (about two thirds of the line Coast). Given that in all the coastal settlements are taking a very big jerk especially during these last 20 years, comes the necessary study this phenomenon with a special importance because otherwise it would be disastrous if ignored or overlooked. What is of concern is that this phenomenon is not given due importance during these years of transition it comes to various reasons (such as for example, properly acknowledge the phenomenon and its evaluation methods, etc...).

Therefore in the future comes with a crucial need for Geotechnical Engineer assessment of this phenomenon and the problems associated with it, to have a quality construction and safe. A picture about this phenomenon, its assessment methods, measures for its elimination are given in this thesis.

Abstrakti i tezës i prezantuar në Senatin e Universitetit Epoka në

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Kryetar : Asistent Profesor Doktor Ylber MUCEKU

Fakulteti : Fakulteti i Arkitekturës dhe Inxhinierisë

Abstrakti

Fenomeni i lëngëzimit konsiderohet një nga problemet më të vështira të gjeoteknikës sizmike,por ka mbi 45 vjet që shkencëtarë të ndryshëm nga e gjithë bota janë marrë me këtë problëm dhe shumë ceshtje janë zbardhur në lidhje me të si në drejtim të terminologjisë ashtu dhe të problemeve që lidhen me të.Lëngëzimi është një fenomen që shfaqet gjatë vibrimeve të tokave kryesisht pa kohezion dhe të ngopura me ujë.Ngarkimi i shpejtë i tyre bën që të rritet në mënurë të menjëhërshmë prësioni në ujin e poreve të zvogëlohet presioni në skelet duke bërë që dheu të mos rezistojë më në prerje.

Shqipëria duke qenë një vend me aktivitet të lartë sizmik ekziston mundësie e prekjes nga ky fenomen .Potencialisht nga ky fenomen është e prekshme pjesa bregdetare (rreth 2/3 e vijës bregdetare).Duke qenë që në të gjithë pjesën bregdetare ndërtimet janë duke marrë një hov shumë të madh sidomos gjatë këtyrë 20 viteve të fundit, del e deomosdoshme studimi i këtij fenomeni me një rëndësi të vecantë pasi ndryshe do të ishte katastrofik nëse nuk merret parasysh ose nëse neglizhohet.Ajo që është për tu shqetësuar është se këtij fenomeni nuk i është kushtuar rëndësia e duhur gjatë këtyre viteve të tranzicionit kjo vjen për arsye të ndryshme(si psh.mosnjohja sic duhet e fenomenit dhe metodat për vlerësimin e tij etj.).

Prandaj në të ardhmen del me një rëndësi vendimtare nevoja e Inxhinierit Gjeoteknik për vlerësimin e këtij fenomeni dhe problemet që lidhen me të ,për të pasur një ndërtim cilësor dhe të sigurt. Një tablo në lidhje me këtë fenomen ,metodat për vleresimin e tij ,masat për evitimin e tij jepen në këtë punim.

Acknowledgements

I take this opportunity to express my gratitude to the people who have contributed in the successful completion of this project. I would like to show my greatest appreciation to Assistant Professor Doctor Ylber MUCEKU. My thanks to him, seem not be enough regarding to his tremendous support and help. Without his encouragement and guidance this project would not have been materialized.

The guidance and support received from all the members who contributed and who are contributing to this project, was vital for the success of the project. I am grateful for their constant support and help. Also I want to thank my friends Ditmir Sufaj and Eni Hajdini for their valuable support.

Edison HOTI

Declaration

I hereby declare that the thesis is based on my original work except for quotations and citations which have been duly acknowledged. I also declare that it has not been previously or concurrently submitted for any other degree at EPOKA University or other institutions.

EDISON HOTI

Date:

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LIST OF ABBREVIATIONS

W_L	Upper limit of plasticity
W_p	Lower limit of plasticity
I_p	Plasticity index
W_n	Natural water content (ASTM D 2216)
C	Cohesion (ASTM D 2166)
φ	Internal friction angle (ASTM D 2166)
E	Deformations module of compressibility
E_{1-3}	Deformation module
W_n	Natural humidity
I_k	Consistency index
I_c	Consistency
γ_o	Specific weight
γ	Natural volume weight
γ_{sk}	Volume weight of frame

n	Porosity
e	Porosity coefficient
σ_c	Uniaxial compressive strength
Qh	The quaternary deposits
N	Neogene deposits
N_1^3	Messinian deposits
N_1^2t	Tortonian deposits
$N_2^1 h$	Helmas – Pliocen formation
N_2^2rr	Molasses deposits
Ms	Magnitude of earthquake
Io	Intensity of earthquake
Q_4	Deposits of eluvial-deluvial slopes
N^1_2h	Molasses deposits of lower Pliocene
N^3_1m	Messinian molasses deposits

CHAPTER I

1 INTRODUCTION

1.1 History of Liquefaction Phenomenon

The sand liquefaction is a phenomenon where the earth resistance falls rapidly or is eliminated from seismic. Liquefaction is one of the most dangerous phenomena on the mood dynamic. As the result of this phenomena many buildings, bridges, road, walls etc… receive a catastrophic damage which may cause even their total destruction. Liquefaction and other phenomena’s are responsible for the structure's fall during the earthquakes in all around the world.

The studies about this phenomenon have started in 1964 because of the earthquake in Japan, where buildings weren't destructed by the structural part but, they had just lost their stability because of that phenomenon. Below, will be shown some characteristically images of destructions regarding to this phenomenon. The increase of water pressure can cause a catastrophic destruction of dam; we saw this happen in San Fernando in 1971. The wall damage was caused because of a lateral spread, at Kobe in 1995. The damages on the Kobe's port in 1995 happened because of liquefaction phenomenon (Figure 1).

Figure 1.The apartment block destruction as the result of lateral spread, Niagata, 1964 [1]

Increased pressure in the pore water can cause catastrophic destruction of dams. It is proved during the 1971 earthquake in San Fernando (Figure 2, 3, 4, 5).

Figure 2.Huge damages on the dam's wall happen due to the lateral spread, Kobe 1995 [2]

Figure 3.Damages seen in Kobe port in Japan during the earthquake of 1995 due to the occurrence of the phenomenon of liquefaction [3]

Figure 4.A considerable amount of damages on the seaport in Kobe during the earthquake in 1995 [4]

Figure 5.The destruction of the bridge as the result of lateral spread, Niagata 1964 [5]

Even though the liquefaction phenomenon is a very complicated phenomenon, more than 45 years many scientists have studied it. They came to a couple conclusions, before and during the liquefaction phenomena, that can determine the risk of liquefaction potential.

1.2 Where Does The Liquefaction Phenomenon Occur?

It occurs on lands that are not consolidated and not cohesive, mainly deposits quaternary geological ones, to Holocene and rare in the Pliocene deposits. Usually susceptible to liquefaction are fluvial deposits, colluvial and eolitics (harrow) when saturated with water.

The phenomenon of liquefaction is observed in deposits near the sea, lakes and river terraces. Susceptible to liquefaction, are also warheads carried by man (artificial) especially when care is not taken in good compaction of the fill material. Also an increase in the groundwater level increases the likelihood of occurrence of the liquefaction phenomena. Liquefaction is closely associated with the fall of earthquakes, and therefore can happen in areas where, liquidation is in a designated area with seismic activity.

1.3 When Does Liquefaction Phenomenon Occur (Liquefaction Mechanism)?

Liquefaction occurs mainly in non-cohesive soils Saturated (saturated with water), where all sieving space between is filled with water. Before the earthquake occurs, the water pore's pressure is very small (Pu). In the moment that the seismic shaking occurs we have an immediate increase of pore water pressure (Pu) but, given that the earthquake's process water is too short is is not likely to emerge from the pores sending strains of land effectively to 0. Given that:

$$\tau = \sigma_{ef} \cdot tg\varphi + c,\ \ \sigma_{ef} = P - P_u \quad \text{but when} \quad \text{që}\ \ P{=}P_u \ \rightarrow \sigma_{ef} = 0 \quad \text{it means that} \tag{1}$$
$$\tau = \sigma_{ef} \cdot tg\varphi + c = 0 \rightarrow \quad \text{The land looses it's rezistence ability}$$

The effect of liquefaction in buildings, in these causes brings catastrophic breach of their land. In retaining walls, as a result of high pressure exerted it can cause dislocation or sliding wall holder. In effect, the foundations of dams can cause landslides and destruction dams, which are catastrophic for areas below the dam in the effects of liquefaction can include major land slide, which in banks scars associated with river beside the river.

Liquefaction causes extensive damage to bridges that cross the river. Most liquefaction phenomena are exposed to coastal buildings and dock port. Port quays, being working structure for lateral earth pressures (like structure holder) at the time of occurrence of the phenomenon of liquefaction in port quay exercised a much greater pressure than he calculated. As a result of this pressure is not taken into account aprons port will lose sustainability in the sliding or tumbling. Human and material damage.

1.4 Why Does The Liquefaction Phenomenon Occur?

To understand why this phenomenon of liquefaction occurs, we must understand the conditions that exist in a soil deposit before the occurrence of earthquake, destroys the land. The land deposits consists of an individual particle ensemble of the soil. If you will look closely at these particles we can see that the particle is in contact with a number of neighboring particles, just as a result of the contact resistance of the soil produced in denominations equation (2):

$$\tau = \sigma_{ef} \cdot tg\varphi + c \tag{2}$$

When the contact forces are large, the pore water pressure is low and vice versa when Contact forces are small pore water pressure is high. It is precisely for this reason that as a result of pore water pressure in the particles loses contact with each other without producing resistance to friction, causing liquefaction of the land. Deposit illustration is given by the following (Figure 6).

Figure 6.a) Low level of pressure b) High level of pressure

Particles have no contact with each other, particles are in contact with each other, and the level of pore water pressure is great the level of pore water pressure is low. Liquidation is more likely to happen [6] after dealing with a structure loose, the sand is saturated with water and apply a load faster. Since the earthquake acting for a short time, it is not enough time for the water to emerge from the pores of the soil rye and compressed (squeezed). We limit will not have any contact between particles completely lost resistance to cutting to her, thus it will have a mass of rigid but a mass of liquid. So” lock” water and prevents soil particles that have direct contact with each other, it is associated with an increase in pore water pressure reducing contact forces between particles (or minimize them until 0)

CHAPTER II

2 LITERATURE RIVIEW

2.1 Earthquakes. What are earthquakes?

Earthquakes are natural phenomena most frightening and dangerous that happen in nature. Earthquakes caused by a wide range of phenomena that can be natural or as a result of activities by different people [7]. After a period of time including nuclear explosions underground reservoirs, major water process etc... But the vast majority of strong and damaging earthquakes have natural origin. The position of the occurrence of the earthquake, it's the magnitude, geology and population density are Key factors that contribute to cause the earthquake .Catastrophic effects may be done in people and in economy . Although earthquakes can occur anywhere on earth, more than 90% of them occur in tectonic contacts which are always in a dynamic continued process.

They are against each other. In land movement of its measures to ensure that the rocky material plans to fill spaces breaches of continuous accumulations that occur relative deformation, accompanied by corresponding feedback. In given moment, deformities and reaches capacity in border to resistant rock. This marks precisely the moment the so-called "earthquake", [8] which appears as a fracture and Strong motion, sudden, the contact between two sliding blocks or micro plate neighbors (Figure 7).

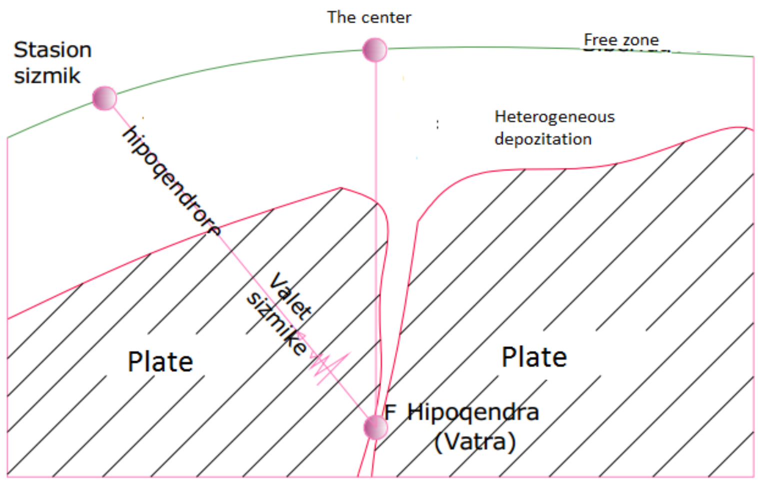

Figure 7.Micro plate neighbors

Area where starts process of fracture of measures rocky is called hypocenter or hearth of earthquake. Projection in surface or areas in surface of land directly over hypocenter is called epicenter [9]. At the time of the earthquake by the earthquake foci relieved instantly accumulated deformation energy which mainly transformed into kinetic energy. This transformation is performed in the form of seismic waves striking. These are waves that spread in all directions with great speed.

2.2 The Interaction of Tectonic Plates

The interaction of tectonic plates to each other in three ways:

1. Convergent, when plates "collide" with each other

As of this Interaction result can be caused both phenomena

1a-collision and consequently the formation of mountains.

1b-Immersion Flexural and dipping means of a plate under another

2. Divergent occurs when one plate is removed from another .As the result in the oceans appear Rift or oceanic ridges, divergent boundaries occur in the expansion of the oceans.

3. The limits in the form of large transformational violations, such is in the case of rupture of the San Andreas which have horizontal displacement.

Movements of rocky blocks similar to those of San Andreas classified as disconnect tectonic type "elastic push" (Figure 8).

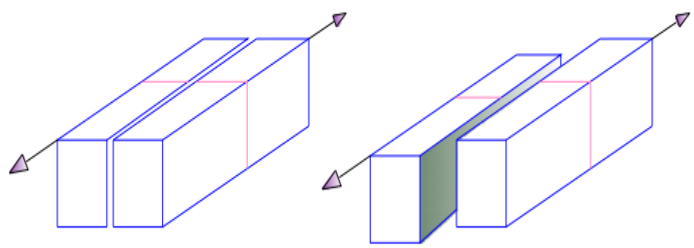

Figure 8.Before and after the earthquake

2.3 Types of tectonic

Earthquakes are the result of tectonic movements, their movements are Catastrophic sometimes

There are three types of detachment (Figure 9, 10, 11). :

1- Normal detachment

2- Normal detachment followed by an extension

3- Normal detachment which is associated with shrinking

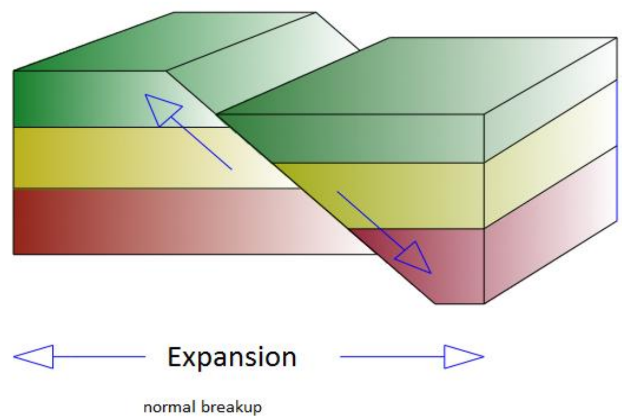

Figure 9.Normal detachment

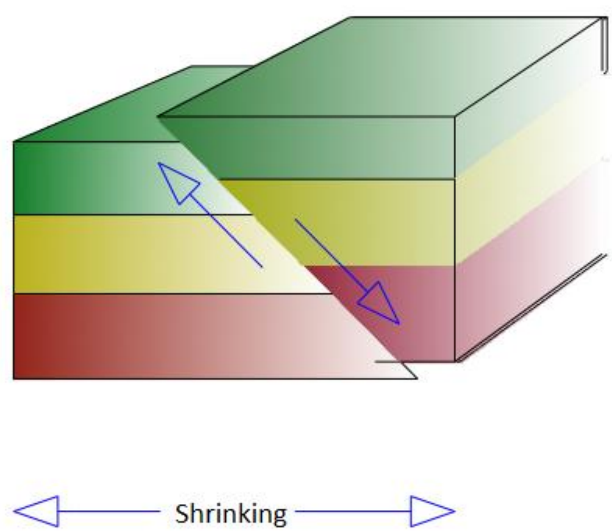

Figure 10.Normal detachment followed by an extension

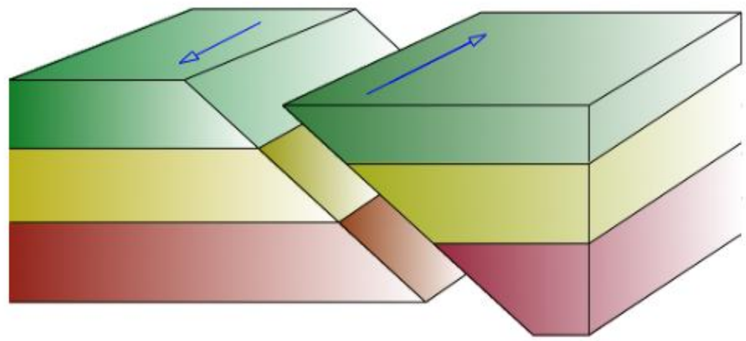

Figure 11.Normal detachment which is associated with shrinking

2.4 The process of earthquake

The earthquake process releases a huge amount of energy, where this energy comes from the center of the quake [10]. This energy is released from the center spread in all directions in form of seismic waves.

2.5 Effects of Earthquakes

Effects of earthquakes can be classified into:

1- Primary effects
2- Secondary effects

In primary effects are the primary characteristics resulting from the earthquake. In those effects are included fractures in slope, soils detachment, and natural swerves.

While the secondary effects of earthquakes that cause different types of damage are:

1- Landslides

Seismic events can trigger landslides or landslides which have catastrophic effects on people and buildings.

2- Tsunamis

Tsunamis are caused when the earthquake epicenter is located in the ocean or in open sea .Tsunamis have a wavelength very huge .Tsunamis have a very low altitude in ocean open, and their speed is very high [11]. When tsunami approaches coastline shallow, its speed decreases as elevation increases drastically, causing catastrophic disasters on earth.

3- Liquidation

This phenomenon is more likely to occur in deposits like sand, etc. Artificial structure padding supported on these deposits of sand saturated with water (saturated), they have great potential to complete the destruction as a result of occurrence of the phenomenon of liquefaction.

4- Fires

Earthquakes also cause fires .As the result of earthquakes it can explode gas deposits, may fall high voltage lines, which lead to amputations and occurs fires .And water pipes makes it impossible to extinguish fires.

2.6 Properties of seismicity in Albania

2.6.1 General

Albania is part Mediterranean seismology. Of the generation studies at European level, Albania is considered one of the countries with the highest seismic activity and ranks right behind Turkey and Greece.

2.6.2 Seismicity of Albania

The main cause of the seismicity of Albania's Adriatic plate collision which considered as part of the African slab, with Albanian orogjenin (part of the slab Eurasian) [12]. This clash is the main cause of tectonic fractures in the system Albania - as they stretch the northwest-southeast direction and the direction of those transverse northeast-southwest. This system is a generator of earthquakes in Albania and around her.

As can be seen from the epicenter maps seems clear that earthquakes are homes concentrated in the area near the Adriatic and Ionian coast, which represents the area seismically Ionian - Adriatic, it is in contact with orogjenin plaque Albanian Adriatic (Continental) and in the east that includes Korce - Ohrid Lake - Peshkopi. These two seismically areas suspended from the transverse seismically a generation, passing from Vlorë [13], Debar and Elbasan east goes up in southwestern Bulgaria and the Channel Otranto. This generation is very active transverse seismically.

Besides the three seismically areas mentioned above, there are some seismic areas like Shkoder - Peja, Vlora – Tepelenë etc. The study of earthquakes energy spread on the soil surface is drawn Map of Maximum intensity observed for the period 1800-1998 [14]. In the present maximal effects observed during earthquakes fallen in our country. In the present areas where maximum intensity was observed IX forehead, face VIII, VII and VI face . From This map appears clear that the entire territory of Albania has proved damaging earthquake.

From the statistical study of earthquakes Albania show that 80-97% probability 7 withstood earthquakes (magnitude M.5.5) can be expected every five years, they face 8 (MS> 6.0) each 10 years, while 9 face (MS> 6.6) every 30 years [15]. While the maximum possible earthquake could this magnitude MS = 7.5, and it happens once in 300-400 years. Based on the data from seismological and geological studies - conducted in geophysical our country is also compiled a map of areas with Albania seismically the expected seismic potential. In this map are outlined areas that can generate seismically earthquake with magnitude to 7.5, such as areas in the north Adriatic coast of Clarendon in Ulcinj.

All coastal areas of the Adriatic and Ionian region-Ohrid-Debar Korca, Vlora crosses generations - Elbasan - Debar and Vlora - Tepelenë can generate maximum earthquakes of magnitude up to MS = 7.0; area along them inside the continent can generate earthquakes with maximum magnitude MS = 6.0. So as noticed, our country could be affected by a powerful earthquake. In 1980 it is Seismic area Map published in Albania, with scale 1 500 000, which represents the risk

The expected seismic with 70% probability for the next 100 years, according to seismic intensity, Averaged for the site conditions. Seismic area Map shows areas where the intensity of Expected can be 8, 7 and 6 face the scale MSK – 1964 [16]. In some regions, for conditions weak plot, as in Shkoder, Durres, Vlora, Korca, Pogradec and near Lushnja, Seismic intensity can reach 9 face. This map is an integral part of anti-seismic design Technical Condition KTPN - 89, and serves all the usual buildings that made in our country. As seen from this map and statistical study of earthquakes mentioned above, Albania has been facing and may face in the future with earthquakes powerful, which is not to take the necessary protective measures, can cause

Casualties and considerable material damage. Therefore, attention should be drawn to all buildings in our country must necessarily respect the criteria anti-seismic construction [17]. For this we need careful drafting of the site selection project by specialists who know and apply anti-seismic design requirements and strict implementation of the project using recommended construction materials.

2.6.3 Some earthquakes for which the liquefaction phenomenon is observed in Albania.

Durres December 17, 1926 (M = 6.0-6.3; Io = IX). Liquefaction of sands between Durres and Shijakut. April 15, 1979 earthquake with epicenter 120 km north of Durres. As a result of liquefaction of sands Cession was irregular throughout the Montenegrin coast. Fier in March 18, 1962 (M = 6.0, Io = VIII). Soil liquefaction in the city and around it, like cracks shatervane ground water associated with the mixed with sand and plow the Seman river shores.

Shkodra April 15, 1979 (M = 6.9, Io = IX) with epicenter in the Adriatic Sea only 30 km from the city Shkodra. As a result of liquefaction of sandy land on both sides of the river Buna are observed very unstable dynamic phenomenon, as Cession irregular, ground cracks with fountain water mixed with sand, plow Buna River.

CHAPTER III

3 EVALUATION OF LIQUEFACTION USING METHODS AND FIELD INVESTIGATION LABORATOR.

3.1 Methodology

Field investigations (or geotechnical) are routinely performed to test all the project processes .Geological studies and groundwater discoveries are part of program Field investigation to be made even when we do not need data to assess liquefaction.

Among the main requirements regarding field investigations in relation to the valuation of Liquefaction is the evidence:

1. Commercial standard penetration (SPT)

2. Cone penetration (CPT)

3. Speed transversal waves spread of VS

3.2 Soil Properties

Prospecting and geological discoveries are very important to provide information on Connection with the extension of unconsolidated deposits which tend to liquid. Information such can be given and geological maps that describe nature of the deposits, their geological age and origin of Quaternary deposits liquefaction potential. It should also be an analysis of groundwater level which includes the Met high water and achieve the highest possible groundwater that may be achieved in the future.

During field investigations should be processed maps with boundaries deposits unconsolidated liquefaction potential. Typical liquefaction occurs in floodplain soils with little cohesion, sands, deposits belonging to Holocene and Pleistocene geological age of late in areas where the groundwater level is smaller than about 15 m (50 feet). According to geological studies that are owned deposits liquefaction are: coastal areas, lakes surrounding areas, areas (lands that are formed as a result of eolitik process).

3.2.1 Determination of soil characteristics

Two tests are used more for Determination of geotechnical characteristics of the soil using field investigation methods:

1. SPT Tests
2. CPT Tests

Both of these tests can give a judgment on whether there is or is not the earthquake in liquidation .In case of floodplain soils or sandy soils-lemur to evaluate liquefaction Samples must be taken during field investigations and tested in cyclic shearing. Exploration programs should be designed to determine the stratigraphy of the soil profile, groundwater level, as well as other indicator used to evaluate the liquefaction or liquefaction potential from field trials and other correlates. Also associated field trials and laboratory tests to determine necessary parameters needed to evaluate the potential for liquefaction.

3.2.2 The depth of liquefaction potential

Traditionally depth of 15m (50 feet) has been used as depth assessment liquefaction potential. Seed and Idris did not suggest a minimum depth evaluation of the potential for liquefaction but indicate a value of 12m (40 feet) where liquefaction can be estimated reasonable. Experience showed a depth of 15m may be appropriate for evaluation of liquefaction potential in most cases. In the case of construction of deep foundations as pilots kesona or depth of investigation for evaluating liquefaction potential must be below a minimum level lower Assuming foundation (KESON or pilots). In the case of liquefaction potential lies below a certain depth, then the investigation process should continue until the soil encountered that are not able to be fluid and at least 3m (10 feet).

3.3 The liquefaction evaluation method using standard penetration (SPT)

3.3.1 Test procedure

This method treats the determination of the resistance of soil at the base of a drill through dynamic penetration of the central tube which ends with a crane and sample extraction to identify broken earth. After deducting crane and busting axles (that hammer above) at the end of the pit drill, the specified depth .crane should sink into the ground in order to drill. Hammer 150mm with the help of the mass falls from height 760mm 63.5kg and recorded number of shocks that plunge crane150mm.Pastaj crane sink to the ground with two degrees each. There can be registered 150mm successive shocks to the number of degrees every dive .Total number shock that causes the penetration of 300mm crane called resistance to penetration (N).

If crane last dives down even as the effect of static weight personal axes shock and hammer, corresponding penetration should not be taken for immersion and this should be recorded (Figure 12).

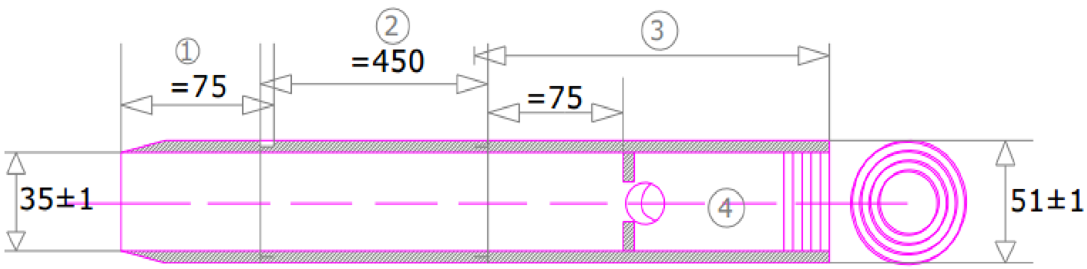

Figure 12.longitudinal cutting of the test SPT crane

Legend:

1. Hooves of drill

2 The rift

3. Sistem connector

4-Head

Below we show a longitudinal incision crane SPT test. From the test results issued a number SPT N-shock for 300mm penetration, this number corrected to give an (N1) 60 referring hammer report energy ER (which expresses the ratio between the real power transmitted to the collision axis with energy indirect free kick to hammer, expressed in %) and a vertical effective stress

Correction is made according to the following formula:

$$N_{160} = N_m \cdot C_N \cdot C_E \cdot C_B \cdot C_R \cdot C_S \quad (3)$$

Where:

N_M is uniform penetration resistance standard.

C_N is the correction factor for depth.

C_E is the correction factor for the ratio of power hammer.

C_B is the correction factor for the diameter of the probe.

C_R is the correction factor for the length of the tube.

C_S is the correction factor for the model.

Pressure correction factor to over consolidation, C_N, can be calculated through the formula:

$$C_N = \left(\frac{P_a}{\sigma_0^{'}} \right)^{0,5} \qquad (2) : (4)$$

Where: P_a

Free is approximately 100 kPa or atmospheric pressure,

$\sigma_{v0}^{'}$ Is the vertical effective pressure at a depth of penetration over consolidation standard?

Sample. In the table below are the suggested correction factors for other adjustments:

From the theoretical analysis and experimental data is extracted SPT and CPT link following:

$$C_N = \frac{P_a}{\sigma'_{V0}}^{m} \quad (5)$$

Where: exponent m has a linear dependence on density reactive Dr.

as follows: $m = 0.784 - 0.521 \cdot D_r$ (6)

Seed has developed a graphic that links or

Seed did a graph which correlates values (N1) 60 with resistance during liquefaction (Figure 13).

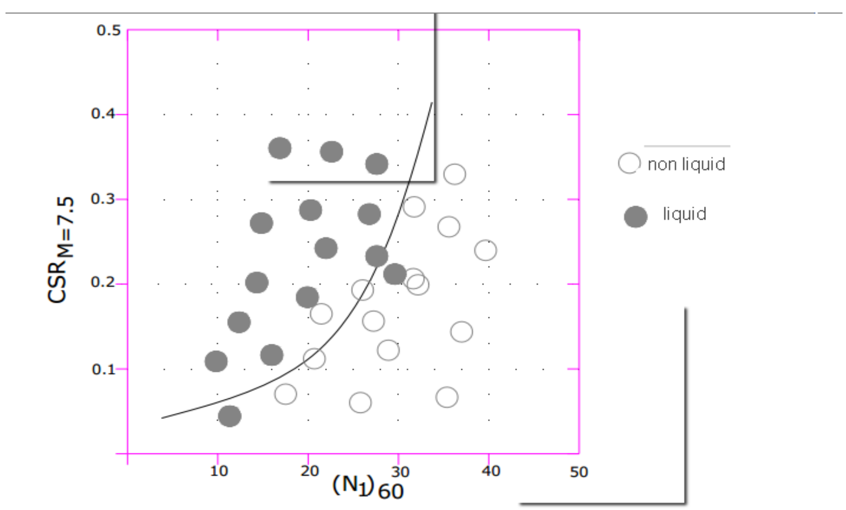

Figure 13.The relationship between CSR causing liquefaction and

for M = 7.5 magnitude

For soils containing fine fractions ($I_P \neq 0$) CSR multiplied by a factor F and the boundary separator depends on the fraction of finely %.

CSR x F where:

$$F = \begin{cases} 1 \; for \;\; l_p \leq 0 \\ 1 + 0.022(l_p - 10) \;\; for \;\; l_p \geq 10 \end{cases} \tag{7}$$

Below is how it changes the relationship of CSR in connection with $(N_1)_{60}$ in order to content (%) of dust (Figure 14).

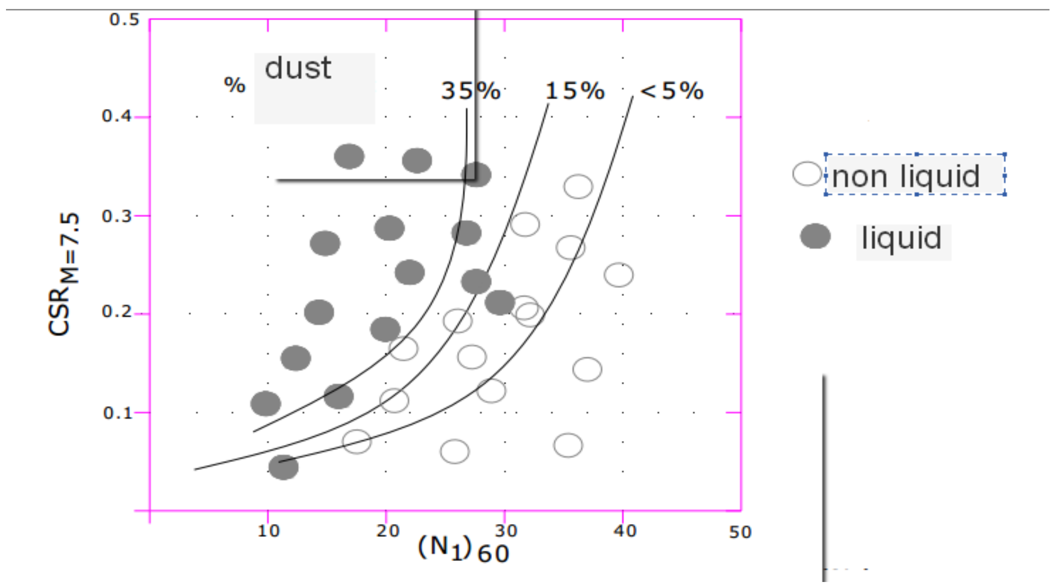

Figure 14. The relationship between CSR causing liquefaction and for M = 7.5 magnitude.

3.3.2 Semi-empirical method for the evaluation of liquefaction using the penetration test

Standard (SPT). From the numerous tests that are done is found a correlation between CRR and $(N_1)_{60}$. The value of $(N_1)_{60}$ affects the material and content of finely FC (roughly content). This impact is taken into account through the following relationship: $(N_1)_{60\ CS} = (N_1)_{60} + \Delta (N_1)_{60}$ (6) where: $\Delta (N_1)_{60}$-matter content of the material gets cramped and given the formula:

$$\Delta(N_1)_{60} = exp\left(1.63 + \frac{9.7}{FC} - \frac{15.7^2}{FC^2}\right) \quad (8)$$

Where: FC-% of fine material.

CRR value for M = 7.5 magnitude earthquake and effective vertical constraints $\sigma'_{vo} = 1atm$ 1: may be calculated based on the values of (N1) 60 CS issued from SPT tests by

the following relation:

$$CRR = exp\left\{\frac{(N1)60\ CS}{14.1} + \left(\frac{(N1)60\ CS}{126}\right)^2 - \left(\frac{(N1)60\ CS}{23.6}\right)^2 + \left(\frac{(N1)60\ CS}{25.4}\right)^4 - 2.8\right\} \quad (9)$$

Using the above equation provides a favorable assessment report the cyclic strain needed to cause liquefaction of soils without cohesion with fine material content in low percentage. It is suggested that the behavior of soils without cohesion should include land (soil) fractions Petty (dust) of which have a plasticity indicator (PI) of less than 5, then PI < 5.

Special attention should be paid to the evidence and procedure JTS in order to draw a more representative value $(N_1)_{60}$, to provide a more reliable analysis liquefaction assessment.

CHAPTER IV

4 EVALUATION OF LIQUIFACTION BY USING SEMI-EMPIRICAL METHOD

Semi-empirical method is used for the evaluation of liquefaction potential in soils without cohesion during earthquakes. To assess the onset of liquefaction we should set a parameter caused by regularities earthquakes and compare this with the resistance that shows ground during liquefaction.

This method uses several parameters that are: r_d, MSF, K_σ and C_N. This kind of parameter determined by the ratio method is cutting cyclic strain (CSR) to ground movements caused by the earthquake, and this parameter (CSR) compared with during liquefaction resistance (CRR).

To determine the ratio of cyclic strain cutter use the method proposed by Seed-Idris authors by whom: Obtained a soil column with a certain height which is subject to an Acceleration assigned based on earthquake magnitude and maximum shearing stress is calculated based on column (Figure 15).

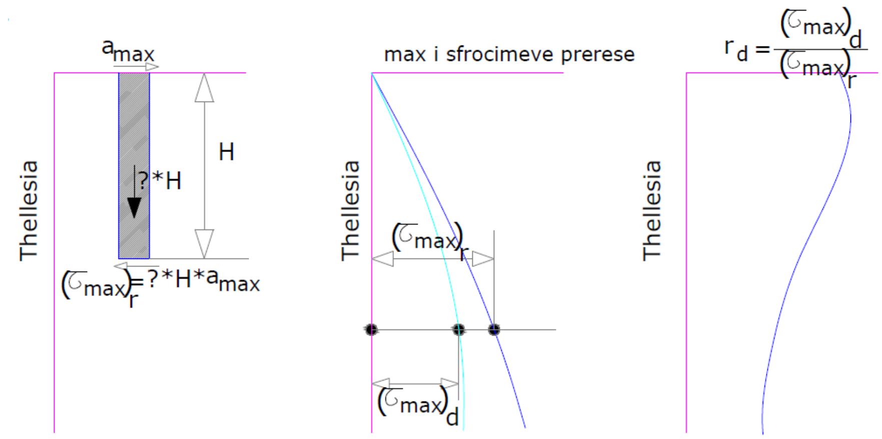

Figure 15. Acceleration assigned based on earthquake magnitude and maximum shearing stress

The proposed formula for calculating cutting cyclic strain caused by the earthquake is given as follows:

$$CSR = 0.65 \cdot \left(\frac{\sigma_{v0} \cdot a_{max}}{\sigma_{v0}^{'}}\right) r_d \qquad (10)$$

Amax.-maximum acceleration caused by earthquake according [19] CSR magnitude calculated for M = 7.5 is given by the formula:

$$CSR_{M=7.5} = 0.65 \cdot \left(\frac{\sigma_{v0} \cdot a_{max}}{\sigma_{v0}^{'}}\right) \frac{r_d}{MSF} \qquad (11)$$

It is a graphic design that connects equivalent number of cycles (Nek) for measuring 0.65 τ_{max}

Earthquake M as follows (Figure 15):

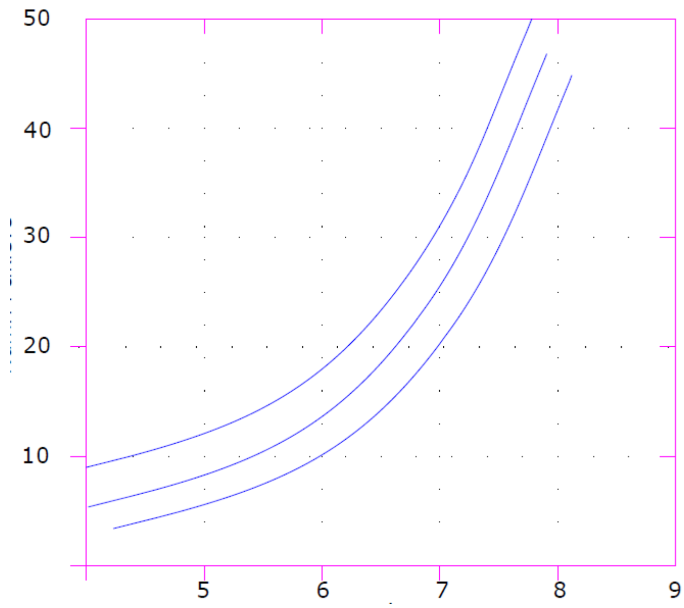

Figure 16.The relationship between the magnitude and number of cycles MSF

MSF called´ Magnitude scale factor used to calculate and CSR during an earthquake the magnitude M in a CSR equivalent to an earthquake of magnitude M = 7.5.MSF determined as follows:

$$MSF = \frac{CSR_M}{CSR_{M=7.5}} \qquad (12)$$

MSF provides a representation of the effect of vibration duration or cycle number. Equivalent number of MSF strains obtained from: 1. Equivalent number of cycles magnitude earthquake with M 2.Relations based on laboratory work between the cyclic strain report cause liquefaction and cyclic strain uniform number. The link between CSR required to cause liquefaction and uniform number of strains Cyclic provides a way to convert a series of constraints in a timely equivalent number of uniform cyclic strain. For example, the recorded data to show that for Nigata 10 cycles uniform CRR = 0:45 to cause liquefaction, and 40 cycles should be uniform with CRR = 0.3 to cause liquefaction .This way is caused the series if the time constraints consists in two cycles, a cycle with a maximum of 0:45 and another cycle With a maximum 0.3.this can be converted into an equivalent set of uniform strain in time:

On occasion with 0:45 we have (1 +10 / 40) = 1.25

On occasion with 0.3 we have (1 +40 / 10) = 5

Idris MSF provides a definition of the function of magnitude below (Figure 17):

$$MSF = 6.9\exp\left(\frac{-M}{4}\right) - 0.058$$

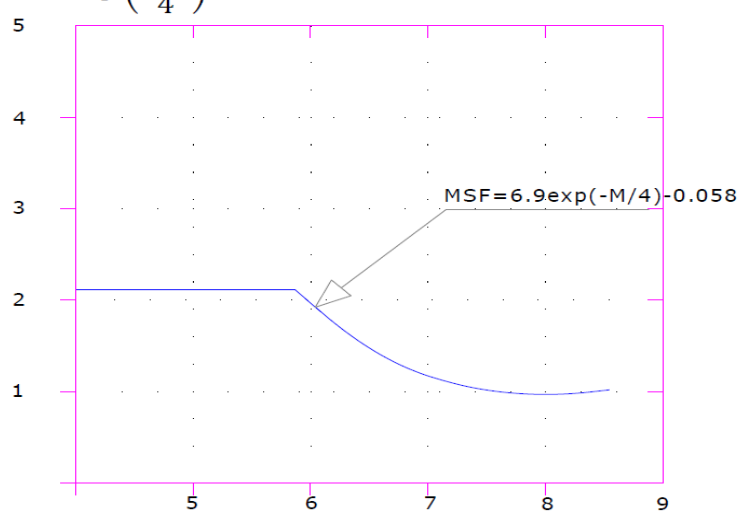

Figure 17. Relationship between magnitude and MSF

MSF has a close connection with the reduction coefficient constraints. For an earthquake magnitude small coefficient results of a small reduction constraints and values of MSF.

Seed and Idris put strain reduction coefficient as a parameter that determines Cyclic strain ratio of land to a flexible column with a column cyclic strains rd rigit .Values of land given by a curve suggested by the authors in order to depth .This value is found for a depth of up to 15m (50feet).

Rd is given by the following formula:

$$ln r_d = \alpha(z) + \beta(z)M$$

Ku:

$$\alpha(z) = -1.012 - 1.126 \sin\left(\frac{z}{11.73} + 5.113\right)$$

$$\beta(z) = 0.106 + 0.118 \sin\left(\frac{z}{11.28} + 5.142\right) \qquad (13)$$

The above equations apply to depth z <34m, the depth z> 34m is:

$$r_d = 0.12 \exp(0.22M) \qquad (14)$$

Uncertainties about the increase of depth rd make the above equations used for less than 20m.For depth evaluation of liquefaction in greater depth they often include conditions and will be more detailed analysis in order which can be justified difficulty .The values of the magnitude M = 5.5, M = 6.5, M = 7.5 and M = 8 we use the above equations that are given in the chart below (Figure 18):

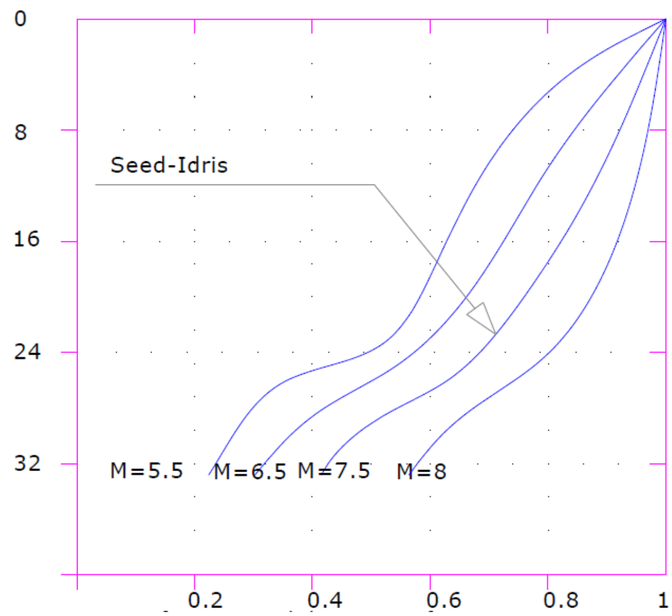

Figure 18.The relationship between the depth and the strain reducing coefficient

Seed has proposed values are not only a function of the depth and magnitude of earthquake but also:

The level of vibration, the vibration duration and speed of spread. These factors that we mentioned before add some complications in liquefaction determination and reduce his trust if they are not taken for granted. But in terms of the scope of engineering applications is suggested that the use of equations above would provide a certain degree of accuracy and reliability. Assessment resistance cyclic ratio (CRR) for the case of the slopes is given by the formula:

$$CRR = CRR_{\sigma=1,\alpha=0} \cdot k_{\sigma} \cdot k_{\alpha} \qquad (15)$$

Where:

$k_\sigma \rightarrow$ correction factor for overload.

$k_\alpha \rightarrow$ correction factor for static strain cutter

$$\alpha = \frac{\tau_{\text{horizontal statics}}}{\sigma'_V}$$

k_σ is determined as follow

$$k_\sigma = 1 - C_\sigma \ln\left(\frac{\sigma'_{V0}}{P_a}\right) \tag{16}$$

And:

$$C_\sigma = \frac{1}{18.9 - 2.55 \cdot (N_{160})^{0.5}}$$

Or:

$$C_\sigma = \frac{1}{18.9 - 17.3 \cdot D_r} \qquad D_r = \frac{N_{160}}{46}^{0.5}$$

$$D_r = 0.478 q_{C1N}^{0.264} - 1.063$$

$$C_\sigma = \frac{1}{18.9 - 2.55 q_{C1N}^{0.264}} \tag{17}$$

4.1 Analytical Evaluation of Liquefaction

The procedure of liquefaction analysis can be summarized as follow:

Safety factor given from report:

$$FS = \frac{CRR}{CSR} = \frac{\text{Rezistence against the liquifaction}}{\text{Strains in cycle cut caused from earthquake}} \tag{18}$$

CRR is determined by the formula:

$$CRR = CRR_{7.5} \cdot K_\sigma \cdot K_\alpha \cdot MSF$$

And

$$CSR = 0.65 \cdot \left(\frac{\sigma_{vo} \cdot A_s}{\sigma'_{vo}}\right) r_d \tag{19}$$

Where: $CRR_{7.5}$ →The raport of cyclic rezistence for earthquakes with magnitude M=7.5 dhe is determined as follow:

$$CRR_{7.5} = \frac{1}{34 - (N_1)_{60CS}} + \frac{(N_1)_{60CS}}{135} + \frac{50}{[10 \cdot (N_1)_{60CS} + 45]^2} - \frac{1}{200} \tag{20}$$

K_σ →overload factor correction and is defined as follows:

$$K_\sigma = \left(\frac{\sigma_{vo}}{2.12}\right)^{f-1} \text{dhe } 1.5 \leq K_\sigma \leq 9^{f-1} \tag{21}$$

f→relative density factor of soil and is determined as follow:

$$f = 0.831 - \frac{(N_1)_{60CS}}{160} \quad dhe\ 0.6 \leq f \leq 0.8 \tag{22}$$

K_α →correction factor to the steep terrain

MSF→magnitude scale factor and is determined:

$$MSF = 87.2 M_w{}^{-2.215}$$

M_w →Earthquake's magnitude

A_s→maximum peak acceleration of ground surface

$A_s = F_{pga} \cdot PGA$

F_{pga} →The amplification factor for the period 0 in the spectrum of acceleration

PGA →Peak acceleration on radical formation (rock).

σ_{v0} →Total vertical strain conditions

σ'_{v0} → effective vertical strains for final conditions.

$\sigma'_{v0}, \sigma_{v0}$ can be calculated by determining huge weights with granulated material as follows:

$\gamma_{granular} = 0.095 \cdot N_m^{0.095}$ –For conditions on the level of groundwater

$\gamma_{granular} = 0.105 \cdot N_m^{0.07} - 0.0624$ –For the underground conditions

N_m –Number of N_{SPT}.

r_d→strain reduction factor

$$r_d = \frac{1 + \frac{-23.013 - 2.949A_s + 0.999M_w + 0.016V^*_{s,40}}{16.258 + 0.210 \cdot e^{0.104(-d+0.0785V^*_{s,40}+24.888)}}}{1 + \frac{-23.013 - 2.949A_s + 0.999M_w + 0.016V^*_{s,40}}{16.258 + 0.210 \cdot e^{0.104(0.0785V^*_{s,40}+24.888)}}} \text{ for } d \leq 20m \tag{23}$$

$$r_d = \frac{1 + \frac{-23.013 - 2.949A_s + 0.999M_w + 0.016V^*_{s,40}}{16.258 + 0.210 \cdot e^{0.104(-d+0.0785V^*_{s,40}+24.888)}}}{1 + \frac{-23.013 - 2.949A_s + 0.999M_w + 0.016V^*_{s,40}}{16.258 + 0.210 \cdot e^{0.104(0.0785V^*_{s,40}+24.888)}}} = -0.014(d-20) \text{ for } d \geq 20m \tag{24}$$

And

Where:

$V_{s.40}^*$ → The average speed of seismic waves spread no further than 12m from the surface of earth

$V_{s.40}{}^{*} = \frac{12}{\sum_1^n \frac{d}{V_{si}}}$

V_{si}→velocity of propagation of waves of a core cutter.

d→thickness of the layer

$V_s = 169 \cdot N_m{}^{0.516}$

N_m →uncorrect number of standart penetration

(25)

$(N_1)_{60CS} = \alpha + (N_1)_{60} \cdot \beta$

$\alpha = 0$ for $FC \leq 5\%$

$\alpha = e^{\left(1.76 - \frac{190}{FC^2}\right)}$ for $5\% \leq FC \leq 35\%$

$\beta = 1$ for $FC \leq 5\%$

$\beta = 0.99 + \frac{FC^{1.5}}{1000}$ for $5\% \leq FC \leq 35\%$

$\beta = 1.2$ for $FC \geq 35\%$

$(N_1)_{60} = N_m \cdot C_N \cdot C_E \cdot C_B \cdot C_R \cdot C_S$

N_m →number of hits for 30cm

C_N →correction factor for equal overload

$$C_N = \frac{2.2}{1.2 + \frac{\sigma'_{v0}}{2.12}} \leq 1.7$$

C_E →corret raport of the hammer's energy

$C_E = \frac{ER}{60}$ ER –The efficiency ratio of the hammer.

C_B →Correction factor for the diameter of the probe.

$C_B = 1$ for $d = (2.5 \div 4.5)inc$

$C_B = 1.05$ for $d \cong 6inc$

$C_B = 1.15$ for $d \cong 8inc$

C_R →correction factor on chilling founder

$$C_R = (-2.1033 \cdot 10^{-11})l^6 + (7.9025 \cdot 10^{-9}) \cdot l^5 - (1.2008 \cdot 10^{-6})l^4 + (9.4538 \cdot 10^{-5})l^3 - (4.0911 \cdot 10^{-3})l^2 + (9.3996 \cdot 10^{-2})l + 0.0615 \quad (30)$$

$0.75 \leq C_R \leq 1.0$

$C_S = 1 + \frac{C_N \cdot N_m}{100}$ and $1.1 \leq C_R \leq 1.3$

4.2 4.3. Evaluation of liquefaction potential

By analytical calculations liquefaction we can conclude if there is any possibility of liquefaction occurs but is not as areas that can undergo liquefaction.

-Enhancing cyclic acceleration caused by the vibration amplitude (earthquake)CSR.

-Resistance to liquefaction-in feature of the uniform amplitude (Figure 19) of cyclic strain capable of CRR produce liquefaction. The safety factor is given by the formula:

$$FS = \frac{CRR}{CSR} \geq 1 \qquad (26)$$

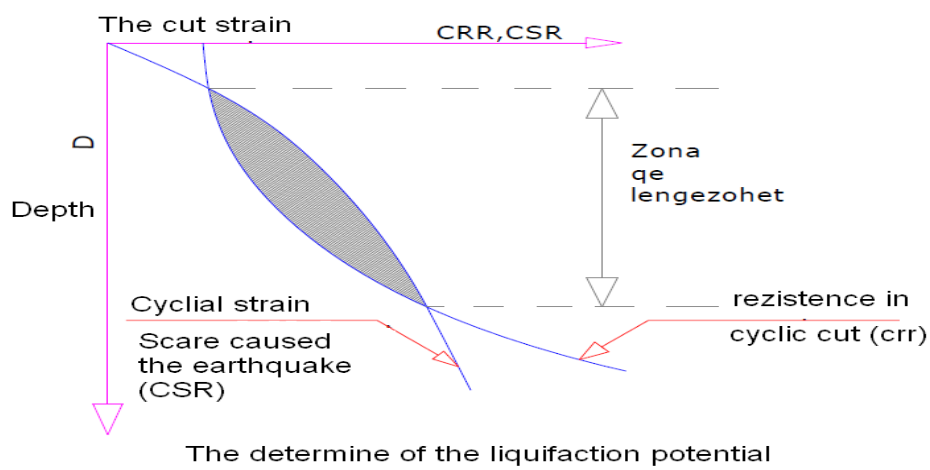

Figure 19.Determination of liquefaction potential

The safety factor against liquefaction depends on the type of liquefaction and report on water pressure of.

For a safety factor (FS) provided we can determine how effective pressure reduced is lower than the rigidity of the earth and chilling phenomenon.

2. Method of cyclic deformation approximations.

This method is based on real observations and records real soil behavior as a result of their dynamic response (earthquake action).

Action earthquake cycle which is provided through a number equivalent to the deforming

Cyclic uniform, given by the formula:

$$\gamma_C = 0.65 \cdot \frac{a_{max}}{g} \cdot \frac{\sigma_v \cdot r_d}{G(\gamma_{cl})} \gamma = \gamma_c \tag{27}$$

Under this approach Yc defined set of earthquake cycles for a number equivalent to function of magnitude. Also calculated deformation Ykuf and Ncikle cutting for causing liquefaction. If you build both these parameters in function of depth will determine how much is areas potentially liquefied.

4.3 Factors affecting resistance to cyclic shearing (CRR)

We cut resistance to cyclical that affect some factors, which are listed below:

• Status of current fetched. Coefficient is expressed through "K_0." In the case of the condition of the deformed fetched plans, an element in the earth, at the moment of equilibrium points exist a relationship between Vertical and horizontal strain.

$$\sigma_H = K_0 \cdot \sigma_V \qquad (28)$$

Where: K0 - coefficient of repression in silence. For a "K_0" particular we know that the more large is the number the more the number of cycles of loading - unloading and as much as σv increases, so the ratio becomes smaller τ_c / σ_v. On the other hand the growth of K0 increases resistance to cyclic CRR cut (Figure 20).

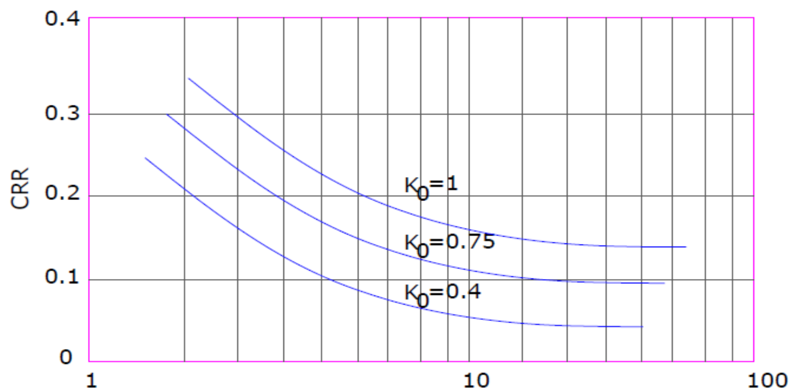

Figure 20.The number of cycles

The state of soil consolidation and strengthen its initial Stone. From tests carried out for some sand samples with specific densities that have been subjected to a certain number loading cycles - downloading, but before cyclic loading have had varying degrees consolidation. Tests were carried out in conditions without drainage (Figure 21).

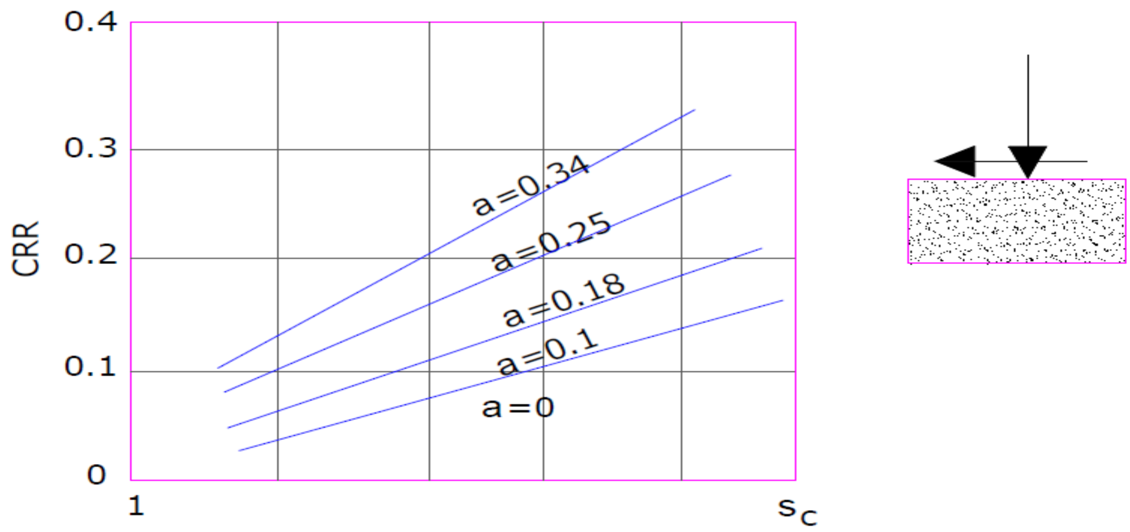

Figure 21. The initial force impact of CRR

- History fetched situation and deformed.

By way of forming sandstone formation and the number of charge - discharge, which has he suffered during its history, it has a certain degree of compactness before undergoing vibrations. The higher is the degree of compacting the (ID) the higher will become the CRR (Figure 22).

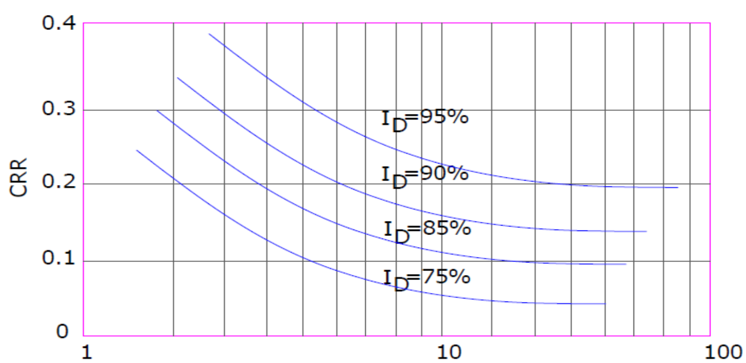

Figure 22.The cycle number

- Change of situation fetched.

  Seismic rolling can cause an increase in σ_z or σ_x. Confirmed that the vibration that is increase σ_x are dangerous because CRR cut, then the possibility of liquefaction caused.
- The degree of saturation of sand with water.

  It is proven that the higher the degree of saturation with water and sand, the more reduced CRR and as the sand is likely to liquefy (Figure 23).

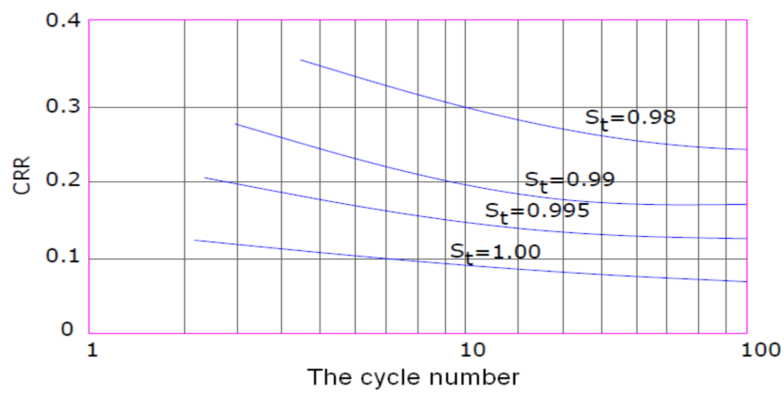

Figure 23. CRR dependency on the number of cycles and St

If sand is vibrated in advance it will have a higher CRR than the same as the unvibrated sand (Figure 24).

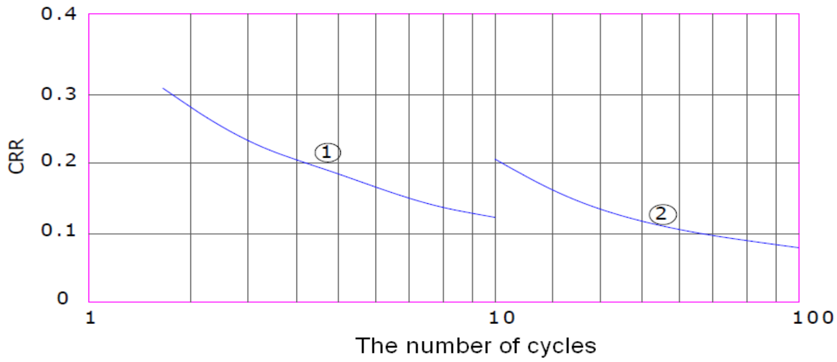

Figure 24. Dependence of CRR in respect of the number of cycles m and

Sand structure

1-unvibrated sample Id=55%

2-vibrated sample Id=55%

CHAPTER V

5 IMPACT ON LIQUEFACTION PHENOMENON SUBSTRUCTURE

5.1 Introduction

The destructions of structures as the result of liquefaction (Figure 25). One of the key factors for major injuries pilots are permanent displacement soil layer on the surface that does not liquefy. Not liquefy layer performs a passive pressure on pilots (crustal layer) with a Kh = horizontal push koificient Kp and layers that liquefy with a push koificient horizontal Kh = 0.3 (so 30% of the pressure exercised by the layer liquefy).

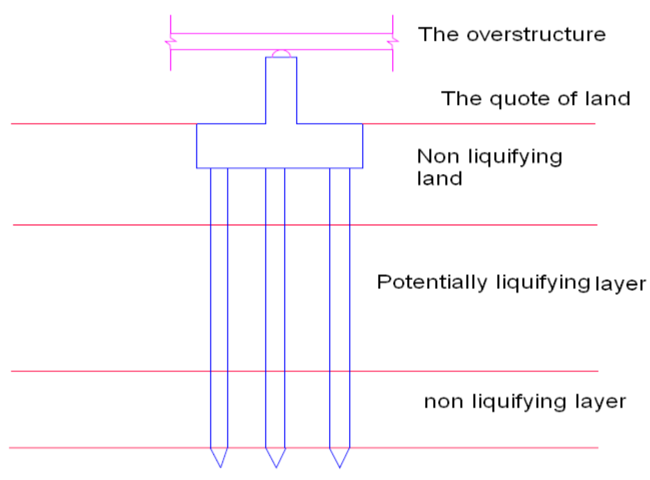

Figure 25.Destructions of structures as the result of liquefaction

Below (Figure 26) we present the diagram of pressures that develop as a result of the phenomenon of liquefaction.

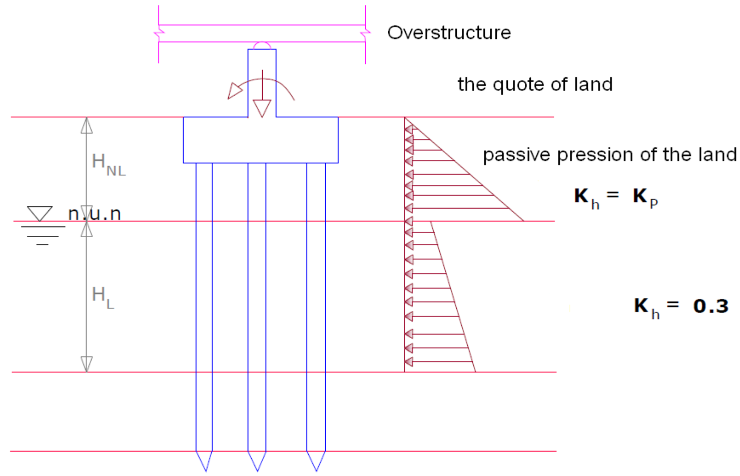

Figure 26.Diagram of lateral pressures

Distinction must be made between security holder ability to pilots in cutting and bending Axial loading and securing them as a result of the occurrence of the phenomenon of liquefaction's proved that although pilots were provided to bending and twisting but not cutting was taken into account passive pressure on pilots emerging from liquidation resulted in the destruction of them. To avoid destruction by bowing we should increase the resistance of fluency material which is achieved by increasing the use betinit grade, increasing the number of bars in etc. But pilots should know that avoiding destruction by not bowing avoids destruction as a result of the occurrence of the phenomenon of liquefaction, which avoided only if set a minimum diameter of pilots that depends on the depth of the beginning of liquefaction.

5.2 Evaluation using the beginning liquefaction semi-empirical method.

Semi-empirical method is used for the evaluation of liquefaction potential in soils without cohesion during earthquakes. To assess the onset of liquefaction we should set a parameter caused by regularities earthquakes and compare this with the resistance that shows ground during liquefaction. This method uses several parameters that are: rd, MSF, Kσ and CN. This kind of parameter determined by the ratio method is cutting cyclic strain (CSR) to ground movements caused by the earthquake, and this parameter (CSR) compared with during liquefaction resistance (CRR). To determine the ratio of cyclic strain cutter use the method proposed by

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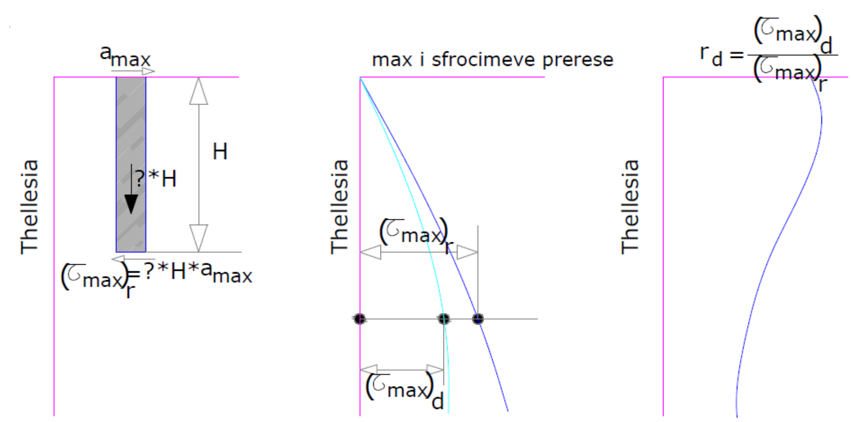

Figure 27.Acceleration assigned based on earthquake magnitude and maximum shearing stress

The proposed formula for calculating cutting cyclic strain caused by the earthquake is given as follows:

$$CSR = 0.65 \cdot \left(\frac{\sigma_{v0} \cdot a_{max}}{\sigma_{v0}^{'}}\right) r_d \qquad (29)$$

According Seed-Idris CSR magnitude calculated for M = 7.5 is given by the formula:

$$CSR_{M=7.5} = 0.65 \cdot \left(\frac{\sigma_{v0} \cdot a_{max}}{\sigma_{v0}^{'}}\right) \frac{r_d}{MSF} \qquad (30)$$

It is a graphic design that connects equivalent number of cycles (Nek) for measuring 0.65 τmax

Earthquake M as follows:

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MSF provides a representation of the effect of vibration duration or cycle number. For example, the recorded data to show that for Nigata 10 cycles uniform CRR = 0:45 to cause liquefaction, and 40 cycles should be uniform with CRR = 0.3 to cause liquefaction.

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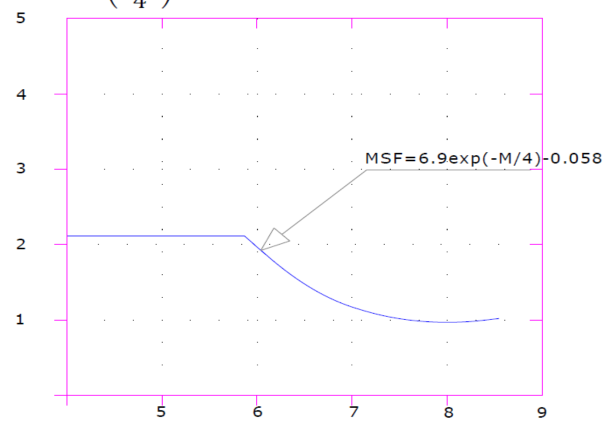

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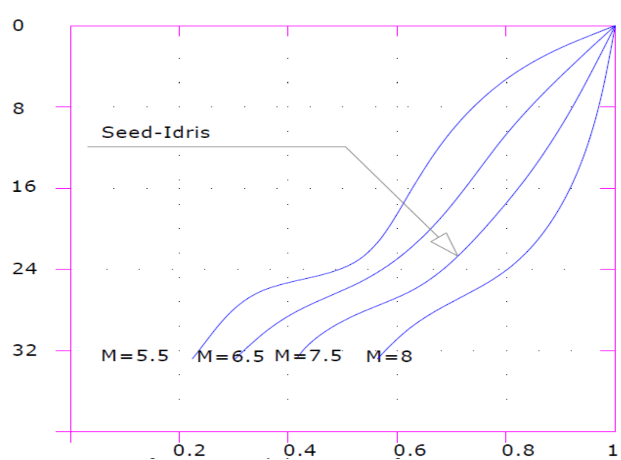

Figure 29.The relationship between the depth and the strain reducing coefficient

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$$CRR = CRR_{\sigma=1,\alpha=0} \cdot k_{\sigma} \cdot k_{\alpha} \tag{34}$$

Where:

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$$\alpha = \frac{\tau_{\text{horizontal statics}}}{\sigma'_V}$$

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$$k_{\sigma} = 1 - C_{\sigma} \ln\left(\frac{\sigma'_{V0}}{P_a}\right) \tag{35}$$

And:

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Or:

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37)

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$$CRR = CRR_{7.5} \cdot K_\sigma \cdot K_\alpha \cdot MSF$$

And

$$CSR = 0.65 \cdot \left(\frac{\sigma_{v0} \cdot A_s}{\sigma'_{v0}}\right) r_d$$

5.4 Evaluation of liquefaction potential

By analytical calculations liquefaction we can conclude if there is any possibility of liquefaction occurs but is not as areas that can undergo liquefaction. To solve this problem there are two main methods are:

1.The cyclic strain approximations method .

2. The cyclic deformation approximations method.

Through the first method we determine;

-Enhancing cyclic acceleration caused by the vibration amplitude (earthquake) CSR.

-Resistance to liquefaction-in feature of the uniform amplitude of cyclic strain capable of CRR produce liquefaction. The safety factor is given by the formula:

$$FS = \frac{CRR}{CSR} \geq 1 \quad (38)$$

The safety factor against liquefaction depends on the type of liquefaction and report on water pressure. For a safety factor (FS) provided we can determine how effective pressure reduced is lower than the rigidity of the earth and chilling phenomenon. Method of cyclic deformation approximations. This method is based on real observations and records real soil behavior as a result of their dynamic response (earthquake action). Action earthquake cycle which is provided through a number equivalent to the deforming cyclic uniform, given by the formula:

$$\gamma_C = 0.65 \cdot \frac{a_{max}}{g} \cdot \frac{\sigma_v \cdot r_d}{G(\gamma_{cl})} \gamma = \gamma_c \quad (39)$$

Under this approach Yc defined set of earthquake cycles for a number equivalent to function of magnitude. Also calculated deformation Ykuf and Ncikle cutting for causing liquefaction. If you build both these parameters in function of depth will determine how much is areas potentially liquefied, as follows:

5.5 Factors affecting resistance to cyclic shearing (CRR)

We cut resistance to cyclical that affect some factors, which are listed below:

• Status of current fetched.

Coefficient is expressed through "K0." In the case of the condition of the deformed fetched plans an element in the earth, at the moment of equilibrium points exist a relationship between Vertical and horizontal strain.

$$\sigma_H = K_0 \cdot \sigma_V \quad (40)$$

Where: K0 - coefficient of repression in silence. For a "K0" particular we know that the more large is the number the more the number of cycles of loading - unloading and as much as σv increases, so the ratio becomes smaller τc / σv. On the other hand the growth of K0 increases resistance to cyclic CRR cut. The state of soil consolidation and strengthen its initial Stone. From tests carried out for some sand samples with specific densities that have been subjected to a certain number loading cycles - downloading, but before cyclic loading have had varying degrees consolidation. Tests were carried out in conditions without drainage (Figure 30).

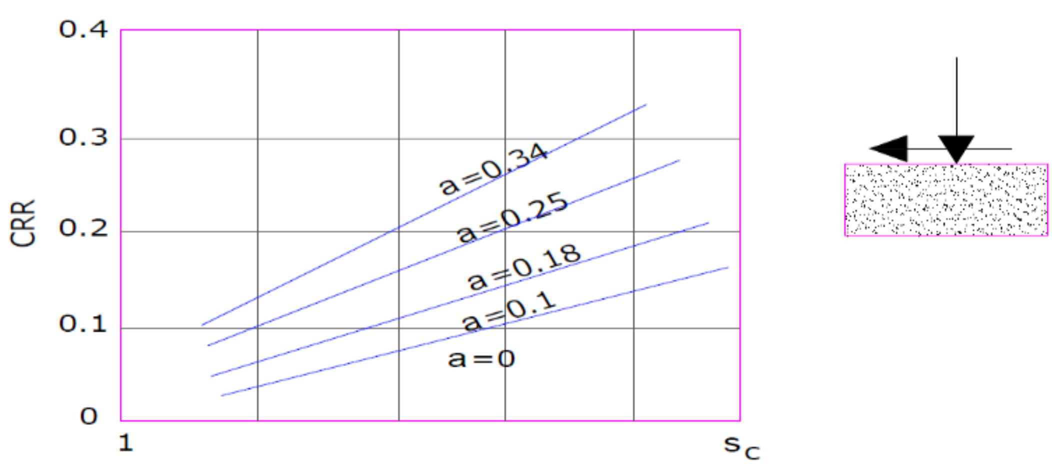


Figure 30. The initial force impact of CRR

History fetched situation and deformed.

By way of forming sandstone formation and the number of charge - discharge, which has he suffered during its history, it has a certain degree of compactness before undergoing vibrations. The higher is the degree of compacting the (ID) the higher will become the CRR.

The cycle number Change of situation fetched.

Seismic rolling can cause an increase in σz or σx. Confirmed that the vibration that is increase σx are dangerous because CRR cut, then the possibility of liquefaction caused.

• The degree of saturation of sand with water.

It is proven that the higher the degree of saturation with water and sand, the more reduced CRR and as the sand is likely to liquefy.

If sand is vibrated in advance it will have a higher CRR than the same as the unvibrated sand.

1-unvibrated sample Id=55%

2-vibrated sample Id=55%

CHAPTER VI

6 LIQUEFACTION AND PROTECTIVE MEASUREMENT'S

6.1 Introduction

Liquefaction is a very dangerous phenomenon because of the catastrophic damage in buildings, bridges, pipelines and many other constructions. In the form of flow liquefaction can cause massive sliding, swinging and tumbling heavy structures, grading of light structures and demolition of structures holder (walls, curtains, piled). Cyclical movement causes a reduction in buildings, spread side (Pressure side greater), destruction and loss of retaining structures

Stability of slopes. The effects of liquefaction can be classified as follows:

6.2 Alternation and the motion of earth

Developing a positive pressure in the pore water causes decrease in arresting the earth shaking. As result the amplitude and frequency of oscillation of the surface soil may change significantly during the earthquake. But the acceleration amplitude reduction for lower frequencies can cause very large displacements, which are found in structures embedded in the ground.

6.3 "Boiling sands" (fountains of sand)

During earthquakes often pore water pressure too high causing explosions water with sand particles on the surface of the phenomenon known free "boiling sand. Happens when we grow Pu surface layers and layers shield is thin. Then arises an intermediate layer of water, do not let it go down because the ground has little refinement But he explodes up because there's less resistance. From the observed cases show that there is close relation between the layer thickness to liquefy, and vibrational acceleration (Pilots, wells, etc.) can completely destroy them (the part where more rigidity varies). Thickness coverage that arise cooking phenomenon causing significant damage (Figure 31).

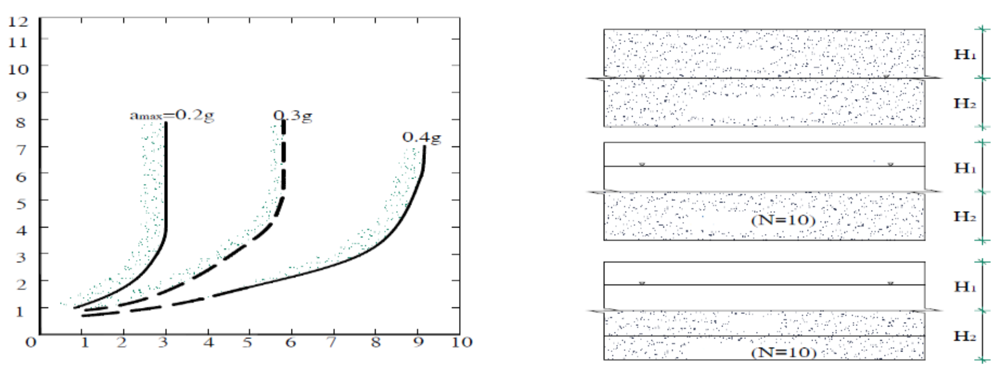


Figure 31.The relationship between layer thickness and thickness liquescent layer on the (Ishihara, 1985)

6.3.1 Decreases

Tendency to be tight sands during earthquakes is well documented in major form of reductions that can cause very large deformations shallow foundations, damage to structures supported on pilots, damages shallow pipelines etc. Dry sands off the end very quickly and practically we can say that they have completed at the end of the earthquake.

Reduce saturated sands that wraps completely extinguished when pressure in the pore water which depends on refinement and ability to distort the massif. Compaction of sands to saturate by the earthquake depends on the ID, "γmax", the percentage of pore water pressure in Pu (from earthquake). Laboratory tests show that εc (volumetric deformation after the onset of liquefaction) depends the ID and γmax. Seed and connectivity through correlative Tokimatsa N1 (60) - CSR - CSR extract ID for M = 7.5. Ishihara has built a graph that provides connectivity εv the factor of safety (FSL) and density Relative ID or N1 or qc (Figure 32).

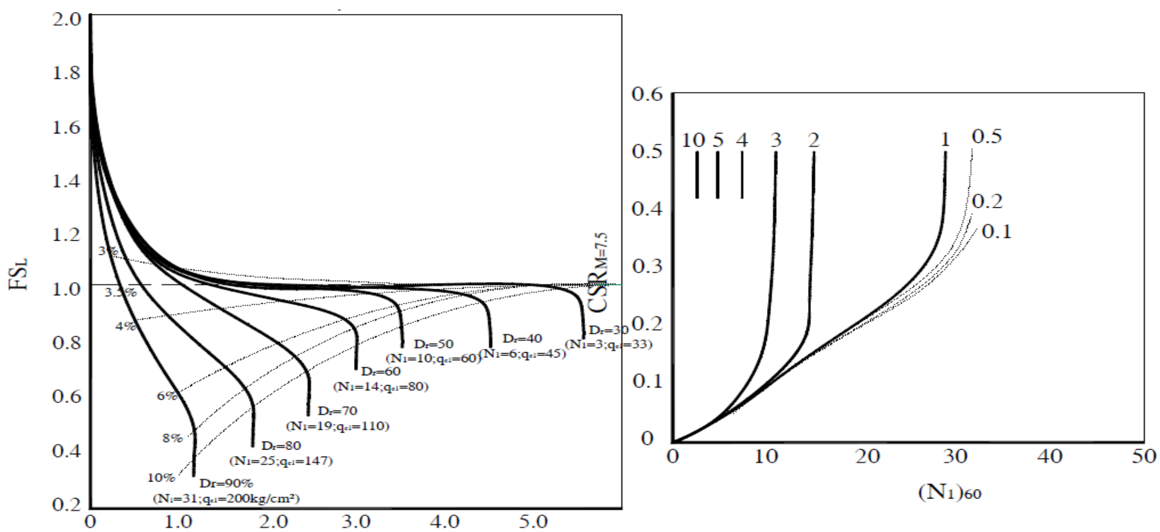


Figure 32. Assessment deformation after liquefaction volumetric εv

6.3.2 Instability

Liquefaction causes greater instability from all other earthquake hazards as leak-shaped slide, lateral spreading, destroying walls, destruction the foundations of basements, etc. slide slopes. Resistance to cutting fluid earth. Instability occurs when the shearing strains are needed to keep the earth balance exceeding the cutting resistance (hardness) of the deposit, ie "τ" by vibration exceeds the ability of the soil resistance. Soil deformed until occupies such a position that these constraints should not exceed this resistance. The size of the deformation to achieve this new configuration of more sustainable affected by the difference in the shearing strain hold in equilibrium (τ) with resistance to cutting of the liquefied soil (CSR). If this difference is small permanent deformation will be small and it is great that you can develop very large deformations. Therefore, more importance is the accurate determination of resistance the cutting of the liquefied soil. Has three estimates for the evaluation of this resistance. There exists three approaches for determining the resistance.

6.3.3 Approximation by laboratory tests

Based on the steady state resistance τq, which depends largely on the density of the earth. We propose that way for sandy soils: Determined in situ size and porosity "of" the samples with intact structure (with some way). SSL lines of evidence determined in samples 5-6 to restore the structure to be as close to reality, and compressed in the final triaksial without drainage. Tq determined resistance of samples with intact structure to a reasonable level deformation (parallel is with SSL). Correct τq regarding resistance "and" determined in situ. With this line can be corrected to assess the risk of liquefaction. Approximation by the evidence in situ (residual resistance). Have issued a conclusion between resistance remained τ mbt and SPT resistance. For sands ≥ 10% finer material, SPT equivalent to fine sand related to N1 (60) and N corr (Figure 33).

$$(N_1)_{60\text{-}cs} = (N_1)_{60} + N_{corr} \quad (2)$$

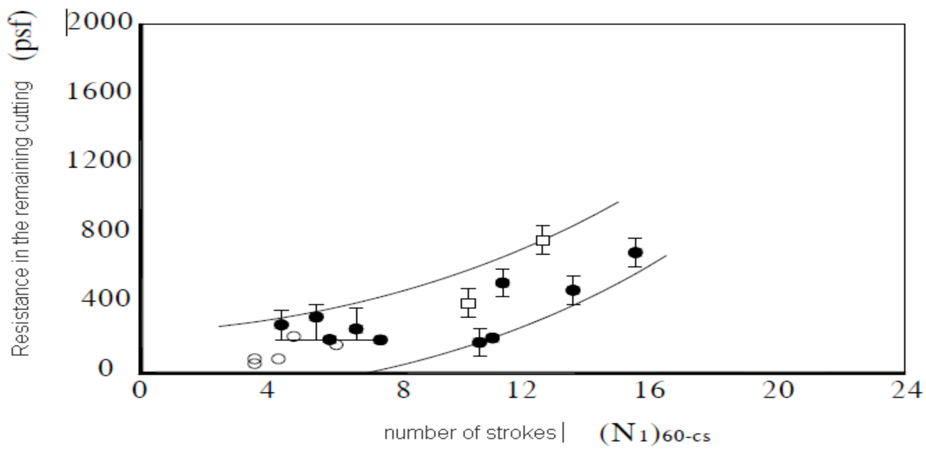


Figure 33. The relationship between resistance remained and Ni60

Approximation of normalized resistance (resistance ratio remained effective strain initial vertical τmbt / σic. Valid for cohesive soils and for those who liquefy. Evaluation of resistance remained liquefied sand is one of the most difficult problems. Concept of τmbt = τ const is very useful for understanding the behavior of soils that liquefy But it (τmbt) depends on the trajectory of the latter strain and is very different in breaking the surface of the three approaches in, the latest report is practical because τmbt / σic≈ constant for a soil type.

6.3.4 Destruction in the form of leakage

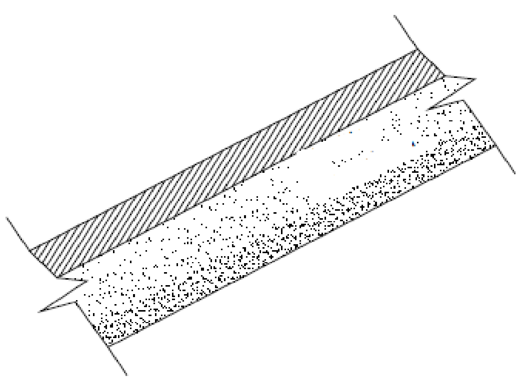
This kind of destruction occurs when the resistance to cutting required for static equilibrium is greater than the cut resistance of the liquefied soil. This road is reached different, but are now recorded 4 - mechanisms.

A. Mechanism

Liquefaction occurs in the form of leakage in total without drainage conditions (Pu does not change, does not change the porosity). It begins when the pressure generated in the pore water (Pu) becomes ≥ effective pressure earth element back in the floating state loses stability. It appears more during the earthquake and causes great movements of the earth.

Mechanism B.

Local leak (because of the release). Issue (release) of soil can lead to a τmbt = τconst <τ, the necessary for static equilibrium. Then flow begins. This happens if the layer of sand has a layer less filtrueshme (no drainage). During oscillation remains constant volume of sand, but restored with dense particles below the porous so the above. If this concession (increase porosity) leads to a reduction of such τmbt as τmbt <τsf it will appear in the local leak form of an intermediate water layer (Figure 34).



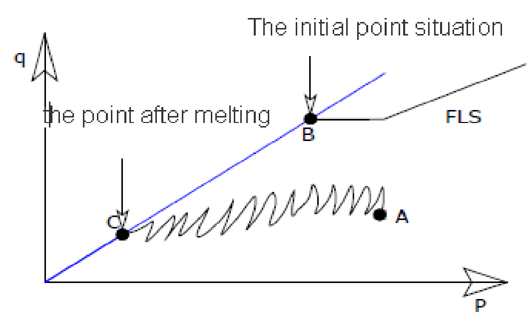


Figure 34.FigSlipped as a result of melting (increasing porosity)

Side note leak when shaking, when porous ID <50% and report τav / σl relatively small value (in hundredths) for a certain number of loading cycles, but on a minimum amount necessary. Lateral flow after the action of cyclic load explained by the presence of a thin dust layer on layer of clay or sand. Layers of dust or clay and waterproof makes you immediately increase water pressure in the pores, displayed as a new gradient "im" that did not exist before. So the liquefied sand is now based on a membrane water or thin layer of water that formed between sand and dust. This membrane water sit for such a long time and layers in sandwich form that is under the action of the gradient " im " of causes a hardening but it already begins to slide on the water membrane. Flow side and lateral deformations are the result of combines sandy layers disposed of liquefy with waterproof layers thus enabling a redistribution in pore water pressure, which may cause major side shift, slide to the very small slopes after earthquake.

C. Mechanism

Global Issue is generated when the pore pressure rapidly and totally lacking Pu>>drainage. This reduces even more τmb.

D. Mechanism

Leakage between the two contact surfaces between structure and occurs when the liquefied soil resistance to cutting is less than the static resistance

$\tau < \tau_s$

6.3.5 Destruction in the form of deformation

Occurs by diffusion side (lateral Spreading). This is a surface deformation that the split causes upper layer and its division into blocks that move horizontally and vertically. We show the surface cracks and buildings, bridges, pipelines, etc., that located in the area destroyed.

In this category fall liquefaction effects and increases above the underground part objekteve.Kjo transmits pressure occurs when small structures on earth such as: pipes, Sewers, etc. fuel deposits. Building up these structures during earthquakes, for because liquefaction occurs when Pu = 100%, or at least the pressure rises above 70% and the The earthquake tremor lasting enough (no more earthquake aftershocks). Rise above can go of 1-2m.

6.4 Measures for the phenomenon of liquefaction

Before we intervene and take steps to prevent or avoid the phenomenon of liquefaction should verify the reliability of the occurrence of the phenomenon of liquefaction and if as it happens liquefaction potential (ie areas that liquefy). Both of these factors have a crucial importance for:

-Intervention in the treatment of soils.

-The area to be treated, which depends on the liquefaction potential.

-Methods of intervention for treating soils which are different.

Measures relating to the liquefaction problem can be:

-Abandoning (drop) the construction in the area that can liquefy.

The acceptance of risk without interfering in the design of liquefaction.

Protective measures to prevent the phenomenon of liquefaction can include:

1. Reducing the risk of soil liquefaction with modification in construction sites.

2. Reducing the risk of soil liquefaction treatment to construction sites.

3. Reducing the risk of liquefaction in the design of substructure (columns of stone).

4. Reducing the risk of liquefaction through the drainage system.

A crucial role in solving measures against liquefaction is the economic factor.

6.5 Reducing the risk of soil liquefaction with modification in construction sites

a) Soil excavation and replacement of potentially deliquescent. Under this method, knowing the liquefaction potential, all material within the area excavated and is cleared away. This area is replaced with suitable material, material with a certain compactness, with a certain granulometri and with a certain refinement able to not liquefy during earthquake. Material associated with laboratory evidence and proof in the field (in situ) in order to be a material suitable to withstand the phenomenon. Given that soil liquefaction are disabled under the groundwater level should measures throughout the drainage during working. The time is a factor that reduces effectiveness of the work significantly and this method is generally uneconomic.

b) Compacting the ground (in situ).

According to this method is in place compacting all with a measure that liquefy. Earths with certain compactness are less likely to be liquid because: Soil-compacted are not able to increase its volume during the earthquake. - Reduce the possibility of adding more pressure in the pore water.

Field compaction methods are different:

1. Method surface realization these include:

Performance-compression method with explosives.

- Methods of realization of the shock compression.

- Method of compaction with vibration performance.

- Implementation of the compression method with hidrovibration.

2. Method the realization of compression at depth.

Conduct in-depth compression is a very powerful technique used to treat soil type’s pores. According to this method a measure of 40ton falls free from a height of 40m, there are examples that a measure of 200ton falls from a height of 25m by ground. Except enable intensification of height and mass firing of some importance great effectiveness of this method has and distance between those stroke points .Usually placed by a quadratic network by calculating the impact of the collision zone at each quadratic point .In sandy soils, silt and gravel compaction depth of this approach calculated with the following formula:

c) Vibration techniques.

-Vibroflotacioni. Vibroflotacioni is a vibration with owning process through which a car is down to earth and unrelated compacts soils by vibration and saturation of simultaneously. While the machine vibrates, water is pumped with such speed that absorbed (absorbed) from the ground (earth). -Vibro-replacement. Vibro - replacement is a process by which soil can improved and is particularly suitable when large quantities of shallow soils difficulties which respond to vibratory compaction. With vibro - replacement, a vibrator is used to penetrate up to a desired depth and cavities (holes) that formed, are filled with thick sand material which may consist of stones.

Vibro-Replacement is quite suitable for soil stabilization, for the construction of shallow foundations, And to reduce liquefaction (Figure 35).

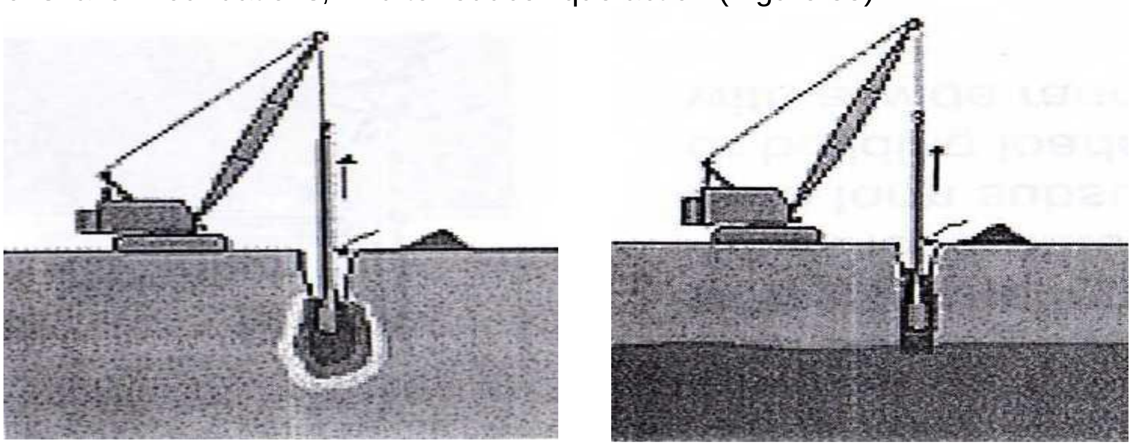


Figure 35.Vibration methods [22]

6.5.1 Reducing the risk of soil liquefaction treatment to construction sites

This method firms' use of chemical methods of reinforcement as:

-Silikatizim.

-Cementing.

Regarding silikatizim authors have proposed different methods which consist of injecting into the earth silicate salts by reaction of a material which is derived in connection gel that surrounds the particle and earthquakes ground .During reinforces this jelly is highly resistant, it is not solidify method that can destroied.This method use less refinement. Regarding the filling grout a low viscosity chemical penetrates the pressure on enabling granular soil increase its rigidity and reduced landings during construction. Chemical method is very effective in Non penetration of water into underground structures.

6.5.2 Reducing the risk of liquefaction in the design of substructure (columns of stone)

Building stone columns is the most efficient and useful for avoiding liquefaction. Given that the main reason for the occurrence of the phenomenon of liquefaction is the rapid increase of pore water pressure, namely the construction of columns will play Vertical drena role as enabling suppression of pore water pressure

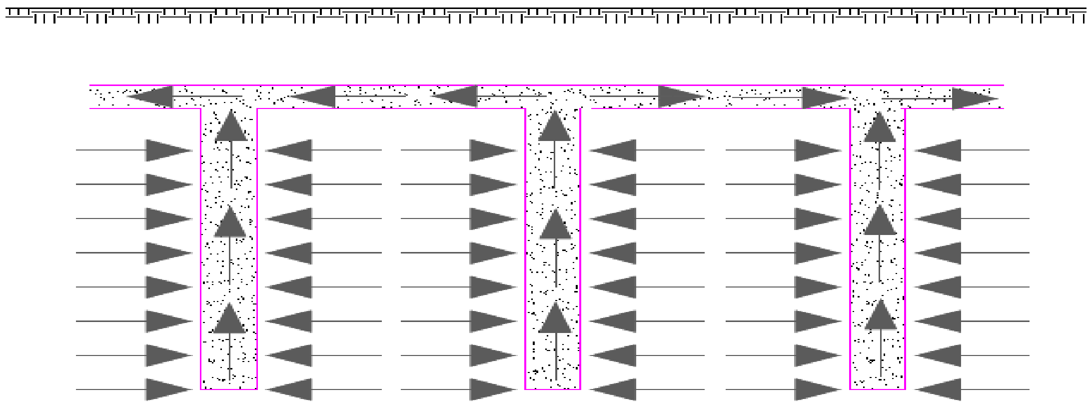


Figure 36.Role of stone columns as vertical drain

To make possible the role of a horizontal drainage columns connected in their heads with a pillow which consists from the same material as the construction of columns. Stone columns being the most used and most effective, below are some aspects of construction and design of these structures.

CHAPTER VII

7 SCIENTIFIC RESULTS AND DISCUSSION

7.1 Introduction

Albania possesses a coastline of about 430 km in length with a very diverse natural conditions as in terms of beauty and tourism development opportunities as the engineering standpoint (Figure 37).

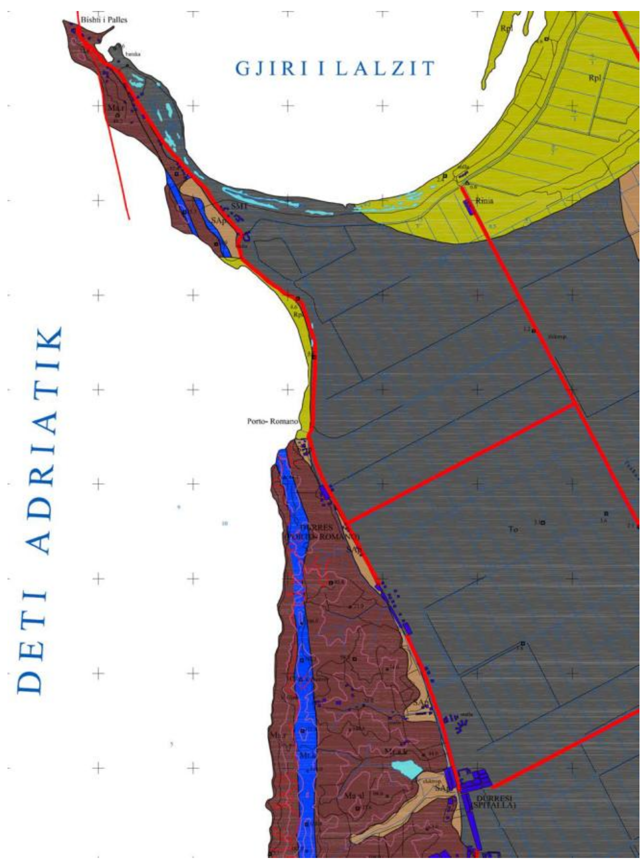


Figure 37.representation of study area (muceku at al.2008)

Geotechnical assessment of Albania's coastal land about tourism development is an important factor in the emergent and current conditions of market economy development .

In this context, during 2012 and 2013 was undertaken by us on a geotechnical study to evaluate the sands during the Adriatic coast for the purposes of planning and development of tourist areas , perspective " 1:10000 scale . For this purpose, initially work in the coast of Durres from the delta of the River Erzeni to Darcie River spilling through surface geotechnical surveys , and conducting field and laboratory puneme . So we are committed field study with 26 drilling through and through 15.0-30.0 , were taken from 78 test soils (5 evidence and structure without destroying the structure 28 spoiled evidence) to be analyzed in the laboratory . Also in the works of each one meter drilling are performed insitu evidence SPT - Standard Penetration Test , and geophysical measurements are made - seismic methods (Vs and Vp) .

Geotechnical data to benefit from the above mentioned works were sufficient to assess soil liquefaction in connection with the phenomenon of sands in the area taken for the study, which we summarized menure I tried to give in this study .

7.2 Geology

7.2.1 General Data

Studied region which extends from the river overflow spilling Erzeni to Darcie River . Geological structures that build the region stretching from south east towards the northwest .

They are represented by anticline which create wrinkles verses hilly country that often constrain creation and hatched kepash coastal accumulative coast abrazive.Ndersa parts built by filling mostly alluvial , alluvial - marine ,

marine , marine - swampy and marshy . Historically filling process has started in the interior of the Plain at the end of the Pliocene . Tectonic movements during the Quaternary era of old time constrain retreat of the sea and erosion and flooding of sea time toward the mainland and accumulation . Have a great role to play and are playing with the accumulative nature of parts of the coast rivers like Buna , Drin , Mat , Ishmi , Erzeni and others with small , where the material is washed and eroded and transported accumulated in this region . Neotectonic movements that have conditioned floating attendance gjeomorfo - structural differentiation , and along with this the areas of corrosion processes are ongoing accumulation and cause vijueshmerisht changing coastline which is expressed through the creation of new lagoon (between the Mat River delta and delta Ishem river) , reducing the size of the old lagoons , diving fundamentals of engineering facilities (bunkers) etc. . In these conditions, the accumulative parts of the coast is characterized by a rugged coastline and quite unstable .

7.3 Building lithology of the studied area

Participate in constructing geological rock formations which are describe below :

7.3.1 quaternary deposits (Q4)

The Quaternary deposits are represented :

Coastal deposits (Q4dt)

These deposits consist of sand and coastal surer that extend throughout the coastal area studied . Slightly thick rivers meet in grykderdhje Erzeni and Durch . Sands meals constructed from quartz , carbonate , feldshpati and heavy minerals . Generally placed on the thick deposits represented by gravel . Also within them often meet ndershtresezime suargjilash , sometimes gravel floodplain and lentils . Towards the mainland RERAT suargjilore covered by a layer , which is clayey alluvial origin deluviale . The thickness of these deposits ranges from 5-6 m to 40-50 m , and sometimes more .

7.3.2 Swamp deposits (Q4kt)

Consisting of clay , and peat swamp lyme . Origin are marshy lagoon , sea - marsh . Was spread from the south bay beach Lalzi to Durres) . Are represented mainly by tiny layers and lenses , surer suargjila , clay and peat which alternate with each other. Placed on the sand and creating their uppers . The thickness of these deposits ranges from 1 to 10- 15m 2m .

7.3.3 Marine deposits - lagoon (Q4dt + lg)

Marine deposits are fused (the ones lagoon . Stretching in the form of a belt between sea and marsh deposits and towards the mainland . Consist of sand and lyme petty . Jane slightly to moderately compressed . Kane thickness from 1 - 2m to 50 - 100m .

7.3.4 Deposits eluviale - deluviale (G4 Al + dl)

Meet us Palles peninsula Tail of Durres . Placed directly to the radical molassic rocks . Suargjila consist of gray to yellow , yellow mixed with snippets shkembenjsh with alevroliti composition and size ranori from 2-3 cm to 30-50 cm . Their thickness varies from 0.5-1.5 to 6.0-8.0 m (Durres - Porto Romano) .

7.3.5 Alluvial deposits (Q4al)

Suargjilat consist of clays and sands which interlink with thick, petty and surer. In the lower parts of cutting and ndershtresa rerash meet with gravel jury. These deposits found in river deltas and Durch Erzeni. Their thickness varies from 20-30 m to 150-200m.

7.3.6 Neogenit deposits (N)

Rates are represented by combination of sandy clays, clays and conglomerates. These lithological units sometimes form packs of special packages and other times forming frequent alterations between them. Build wrinkled structure (anticline-syncline) with NW-SE extension

7.4 Geotechnical assessment of geotechnical units

Below I will address in detail the geotechnical units based on lithological types, their.

7.4.1 Sands and surer Coast (Q4 dt)

With few exceptions only build a cordon across the Adriatic coast cumulative study area . Generally represented by Koker tiny to medium sand , but also by the surer powdery sand . Only the upper part of their on zero quota is dry , and the rest is below the water table and of course the compressed and saturated with water , and in some parts are likely to be liquefied eg Lalzi Bay and Porto Romano . Strong earthquakes around 4.5-5.5 Richter in the area have caused the eruption of liquefied sands of shaking and creating pseudo forms volcanoes of sand flowing to flat surfaces supine’s clay soil . Building conditions in these deposits are suspicious.

7.4.2 Clays, suargjilat , sludges , peat swamp (Q4kt)

These deposits lie in the former land of the former swamps or on the soles of swamps today. In some cases we meet as strata deposits of sand. These deposits constitute a thickness from 0.5-3 m to 7-10 m and placed on deposits of sand or of gravel. Sludge’s, silt and peat layers be soft layers of weak. Therefore, in assessing the geological-engineering sectors, must determine their spatial placement. The main dissemination have former swamp area of Durres.

Clay and sand and tiny marine - swamp (Q4dt + kt)

In fact, from clay to sand meet intermediate fractions from the light suargjilat to surer of tiny powdery sand and in any case they surer suargjila the floodplain. These factions appear quite mixed media, create time - time massive mix, and occasionally with differentiated layers. Thickness ranges from 2-3 m to 10-12 m lie on the sand and gravel.

The land comprising these deposits appear in difficult conditions gjeologjo - engineering, so long as the building construction phase etc. need be carried out detailed studies to determine the presence of weak siltstone layers .

7.4.3 Suargjilat sand and gravel alluvial (Q4al)

These deposits are alluvial whole but can also be partially aluvialo - coastal. Meet the layer clays, surer and jury. In the land of the spread of the stockpile in general has good conditions gjeologjo - engineering

These deposits are interwoven with each other and create specific hydrogeological conditions in some aquifers but without important.

molassic rocks

Molassic depositions of the study Neogenit band stretching from the peninsula where Kavaja Radon build up in the hill ranges (anticline) and the foundation of the Quaternary deposits , which constitutes the general manger of the syncline other morfostrukturave submerged under the action of neotectonic movement . Generally constructed by combination of rocky layers represented by soft rocks such as sandstone rocks, clay and conglomeratic

7.5 Zoning geological-engineering study area

On the basis of the data , obtained from the work done in the area of study in relation to :

• Building lithological

• physico - mechanical properties of soils and rocks

• geomorphological conditions

• hydrogeological conditions

• physico - geological processes

It became possible zoning geological-engineering study area , which are elaborate below :

The area represented by marine soil , sand bedding Koker to medium Koker , recent color yellow gray , 2-6 m thick .

Area marine soils consisting of finely ground sand to gray powdery blue that we spend time and you having some LINZA surer and layers suargjile . Have a thickness of 4-6 m to 10 m .Areas of alluvial soils (Erzeni river delta and Barçi) , medium sand up Koker Koker thick . 10-20 m in thickness .Zone - alluvial soil and consist of suargjila deluviale with brown Beige 1-4 m in thickness .Area marine soils - lagoon , sand grain Petite , medium grain to surer with beige and 1.5-2.5 m in thickness .Area marine - lagoon soil , sand grain Petite , medium grain gray ash and content in small quantities of broken shells , have a thickness of 2.5 m to 10-15 m .

The area of wetland soils - marshy consisting of finely ground sand grain less lymezim , gray to blue . Containing waste timber and finely chopped 4-5 cm LINZA TORF . Have a thickness of 2-5 m . to 30 m .

Area detaro - marshy soil consisting of clay, silt content in hazel Torfi to millezuar with dark brown . 10-15 m in thickness .Detaro - area soils consist of suargjila lagoon and quite powdery sand and surer we ndershtresa yarn containing seaweeds which have 10-30 m thick .Area of soft rocks - rocks shale , and conglomeratic sandstone .

Physico - mechanical properties (properties geotechnical) engineering - geological units treated with the above and given in engineering - geological map (FEV : 1:25000) are analyzed in detail in the chapter " geological-engineering classification of the coastal zone "

7.6 Geotechnical model of the studied area

We are based in a very big field and laboratory we show two main geotechnical models which represent the north and at ejugore. Geotechnical model of the studied area, the northern sector (Edison have to estimate liquefaction) . Geotechnical model of the studied area, the southern sector

Layer No.1

Koker represented by tiny powdery sand that gradually move on surer, with beige-gray, with high humidity so saturated with water, the extent seaweeds contain 25 to 30%. Are less crowded. Meets at 1:00 depth - 4.80 m.

Physico-mechanical characteristics of this layer are**:**

<u>The composition Granulometry</u>

clay fraction	< 0.002 mm	18.80 %
fraction powdery	0.002-0.075 mm	57.6 0%
sand fraction	> 0.075 mm	23.60 %
Weight volumore in natural state	γ=1.86 gr/cm^3	
Internal friction angle	$\varphi = 22^o$	
Compression Module	$E = 65$ kg/cm^2	
Compressive loads allowed	$\sigma = 1.5$ kg/cm^2	

Layer No. 2

Koker sand is represented by small to tiny Koker powdery gray blue color, are saturated with water, containing rare seaweeds in small amounts, are slightly to moderately crowded. Meet the depth: 4:50 to 6:20 m.

Physico-mechanical characteristics of this layer are (Figure 37):

Figure 38.representation of sand moisture

Indicators of physical-mechanical properties of this layer are:

Granulometry:

sand fraction (2 mm - 0.5 mm)	58,7 %
fraction powder (0.05 - 0.002 mm)	33,7 %
clay fraction (< 0. 002mm)	7,6 %

Indicators physico-mechanical:

The upper limit of plasticity	W_l = 34,2 %
The lower limit of plasticity	W_p = 20,6 %
The plasticity index	I_p = 13,8
natural humidity	W_n = 32,1 %
specific weight	= 2,64 gr/cm^3
volumetric weight	γ = 1,80 gr /cm^3
Volumetric weight of the shell	γ_s = 1, 36 gr/cm^3
porosity	n = 48,45 %
Index of porosity	e = 0,94
Internal friction angle	φ = 26º
cohesion	c = 0,01 kg / cm^2
Module deformacionit	E_{1-3} = 65 kg/cm^2
Compressive loads allowed	σ = 1,6 kg/cm^2
Nr. SPT	8 - 10

Layer No.-6

Thellsia met from 6.2-13.0m. Suargjila represented by gray siltstone with hazel. E loose, more moisture, fluid plastic. Physico-mechanical characteristics of this layer are:

Indicators of physical-mechanical properties of this layer are:

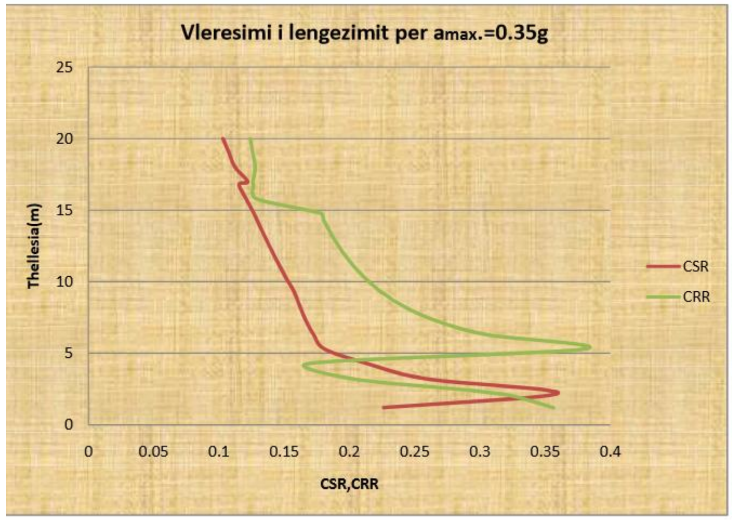
Granulometry:

gravel fraction	(> 2.0mm)	-
sand fraction	(2mm - 0.5mm)	27,8 %
fraction powder	(0.05-0.002mm)Silt	61,8 %
clay fraction	(< 0. 002mm)	10,4 %

Physico-mechanical INDICATORS:

The upper limit of plasticity	W_l= 41,6%
The lower limit of plasticity	W_p = 22,7%
The plasticity index	I_p = 18,9
natural humidity	W_n = 40,84%
Index of consistency	I_c = 0,96
specific weight	γ_o = 2,68 gr/cm^3
volumetric weight	γ = 1,70 gr /cm^3
Volumetric weight of the shell	γ_s = 1, 2 gr/cm^3
porosity	n = 55,15%
Index of porosity	e = 1,23
Internal friction angle	φ = 10^o
Cohesion	C = 0,05kg / cm^2

Module deformacionit	$E_{1\text{-}3}$ = 12,0 kg/cm^2
Compressive loads allowed	σ = 0,7 kg/cm^2
Nr. SPT	3 - 4



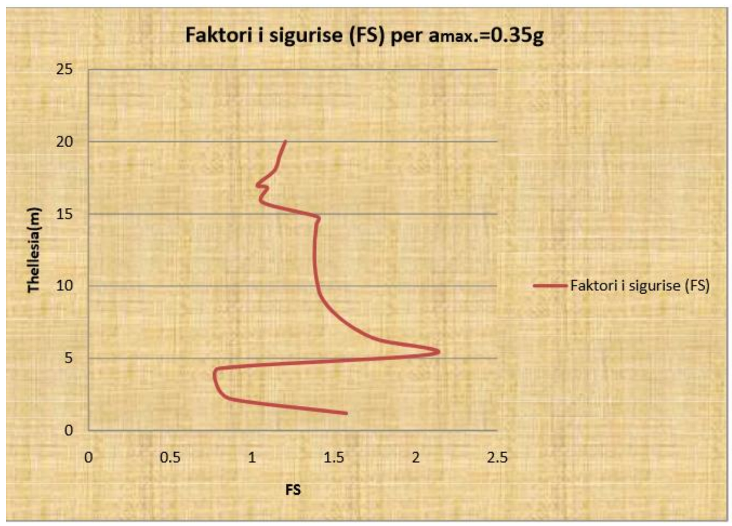


Figure 39. Assessment of liquefaction for 0.35g acceleration

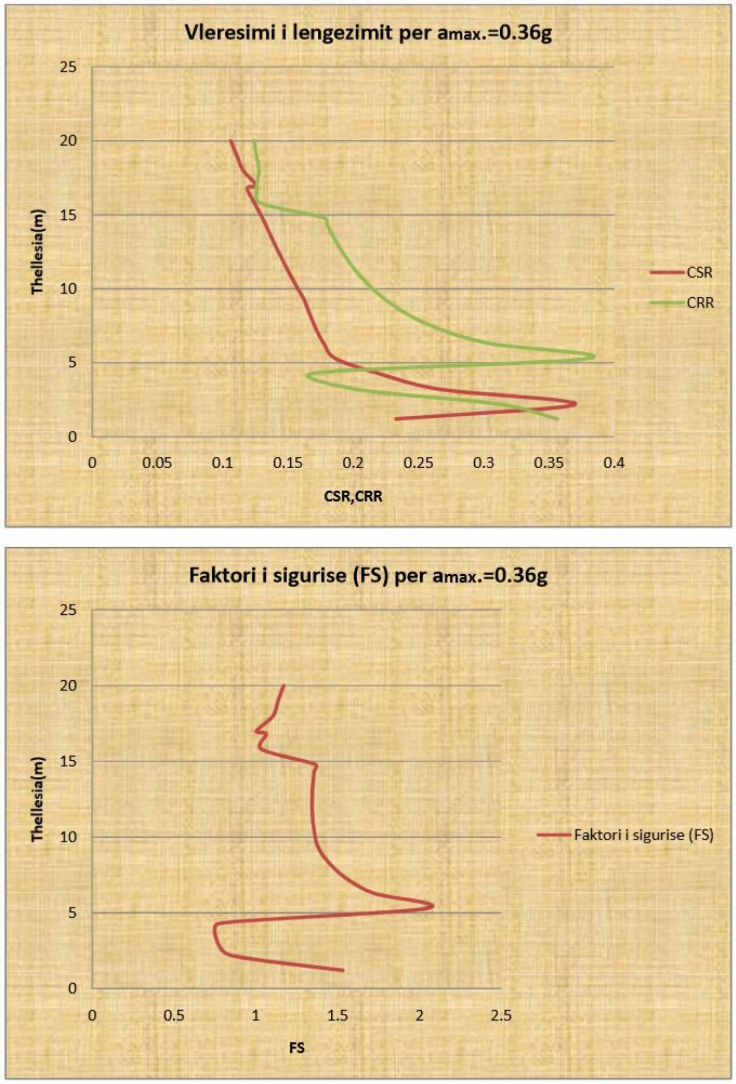
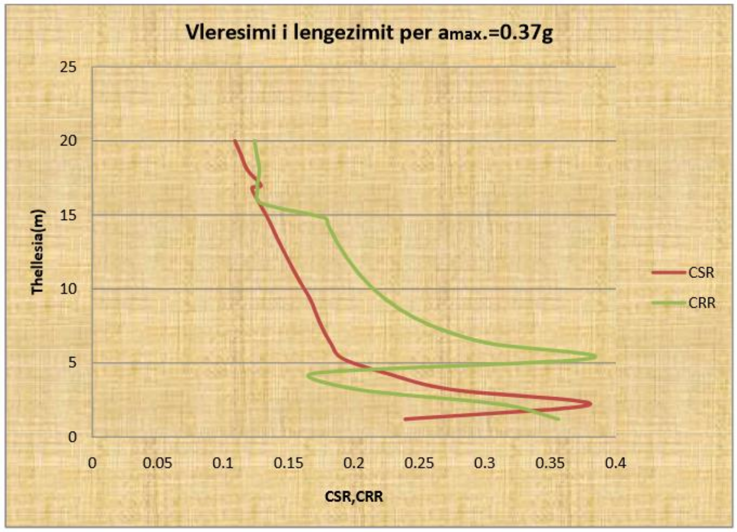


Figure 40. Assessment of liquefaction for 0.36g acceleration



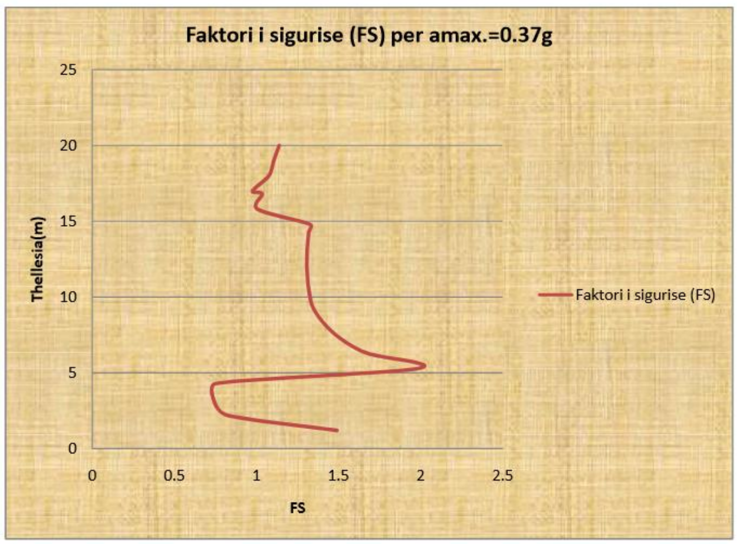
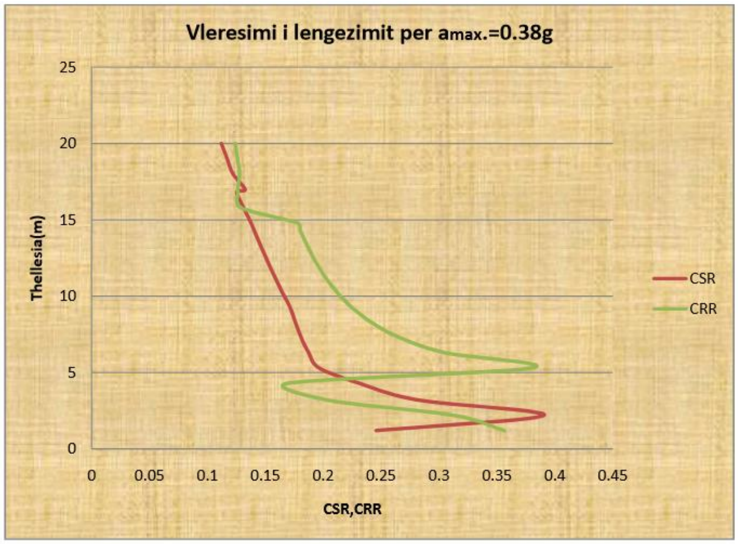


Figure 41. Assessment of liquefaction for 0.37g acceleration



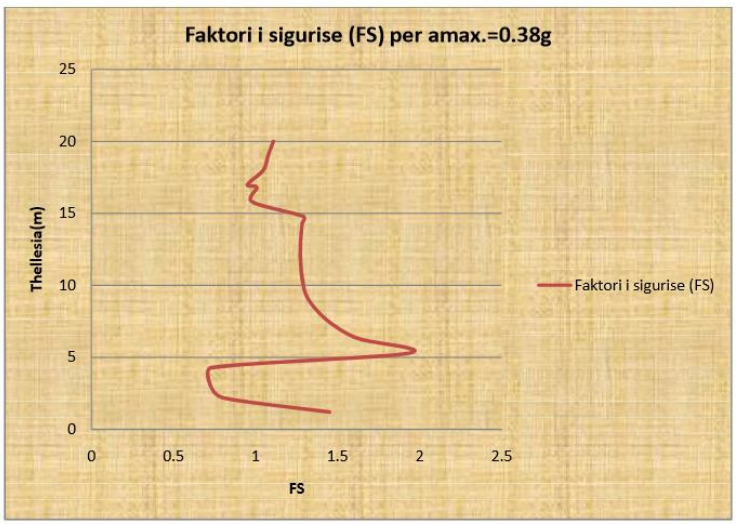
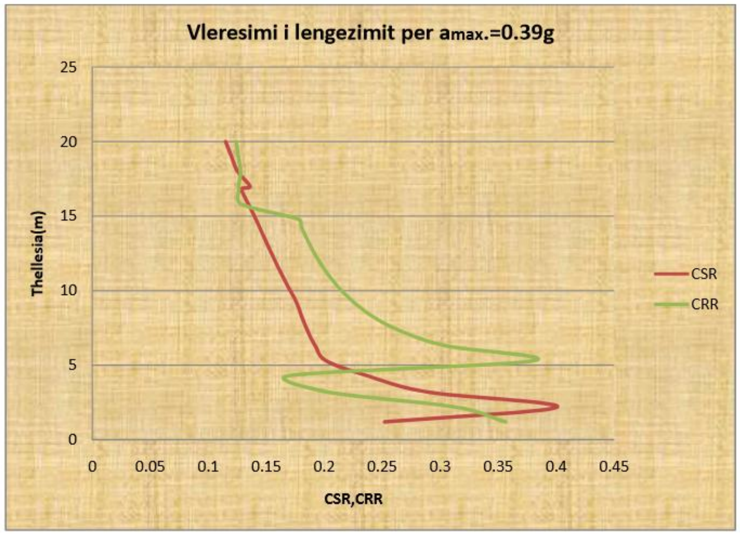


Figure 42. Assessment of liquefaction for 0.38g acceleration



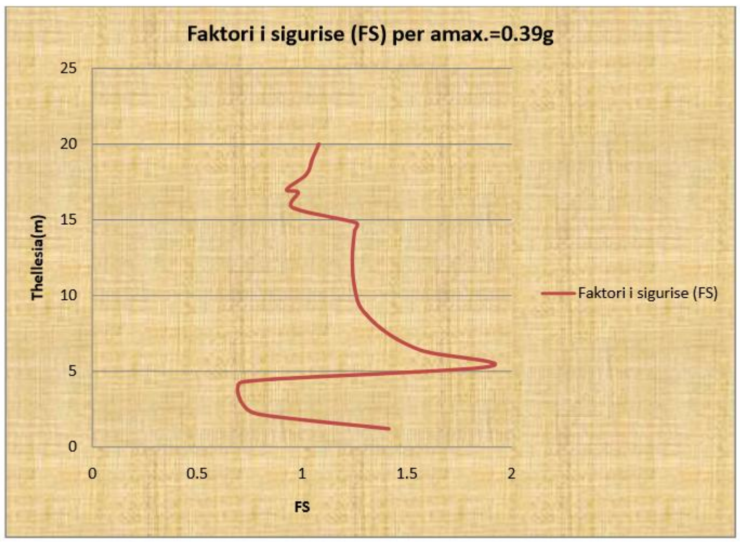


Figure 43. Assessment of liquefaction for 0.39g acceleration

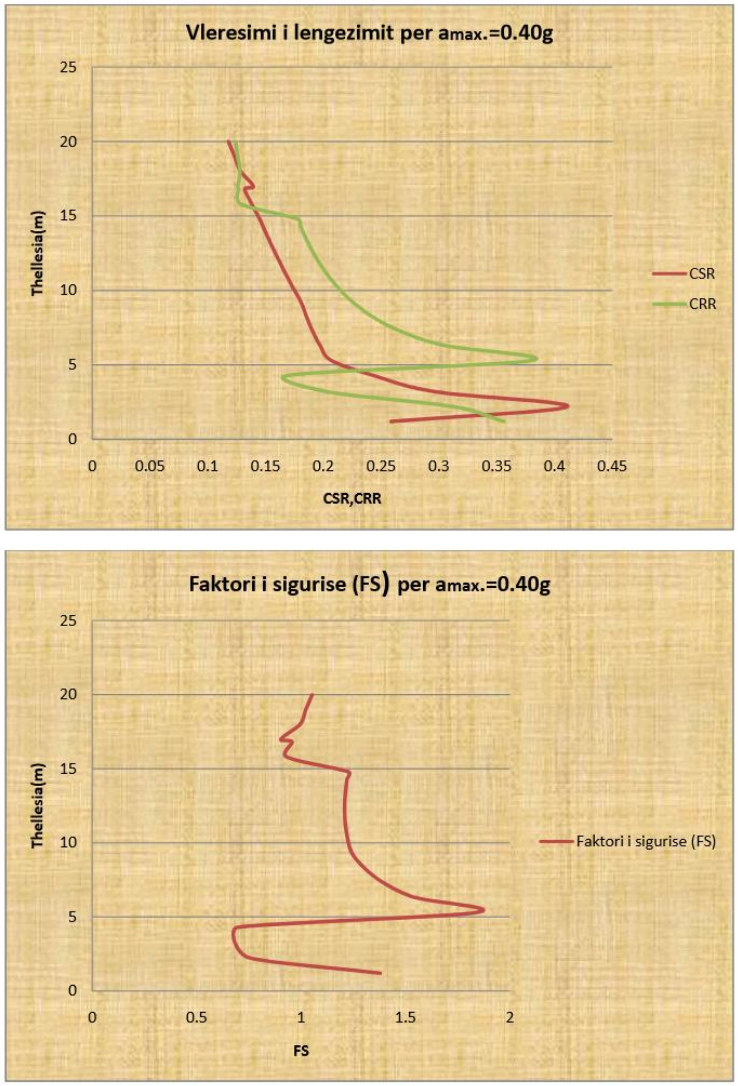
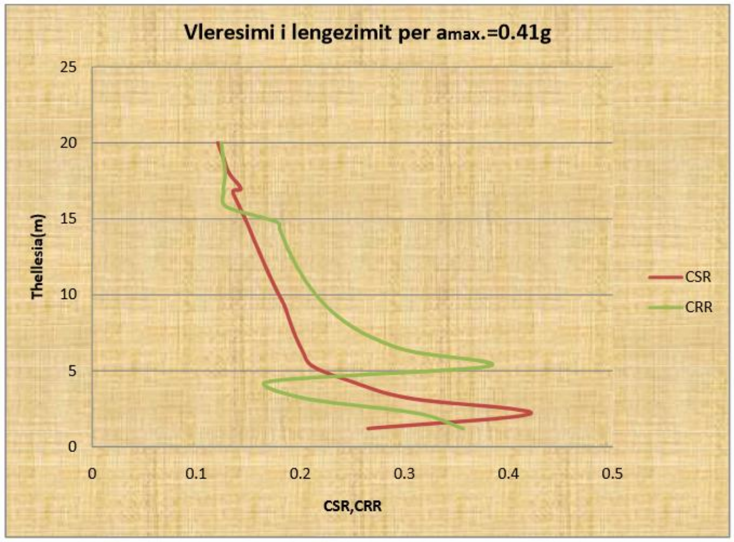


Figure 44. Assessment of liquefaction for 0.40g acceleration



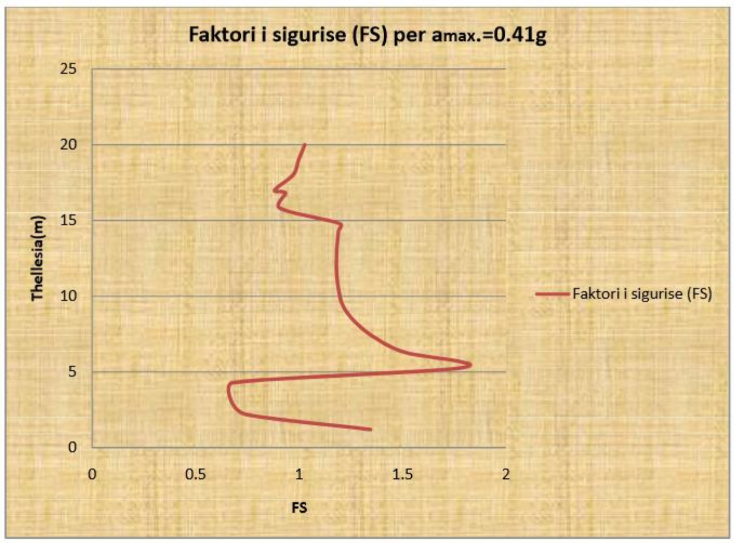
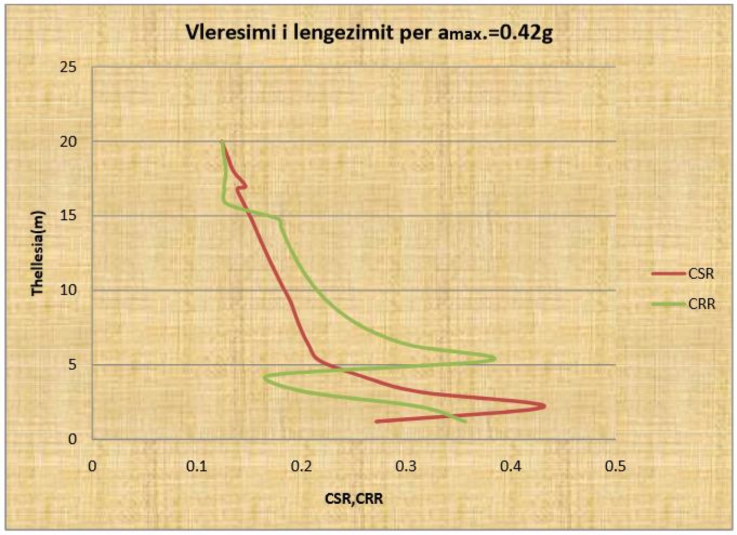


Figure 45. Assessment of liquefaction for 0.41g acceleration



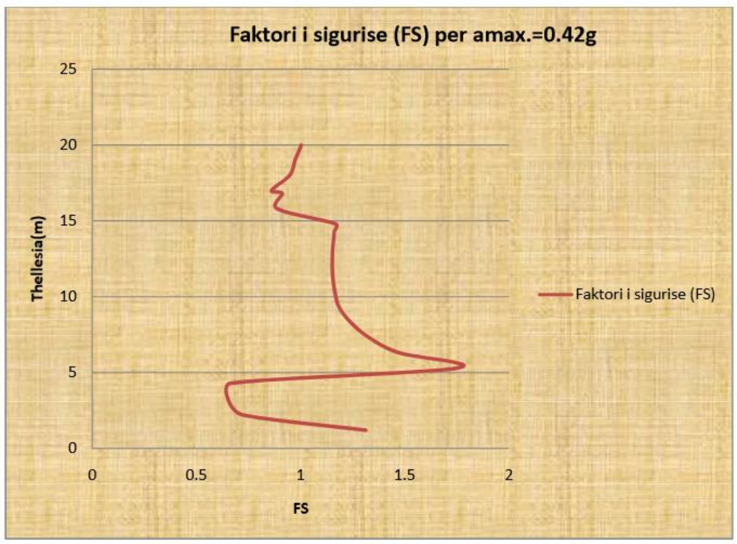
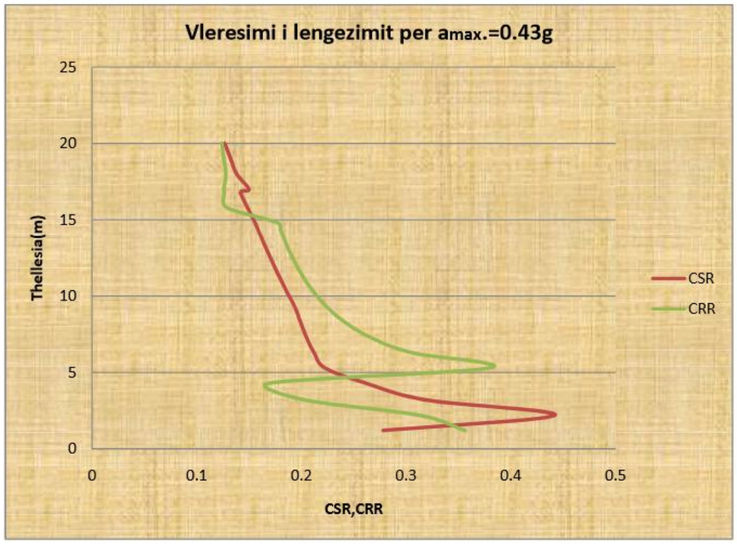


Figure 46. Assessment of liquefaction for 0.42g acceleration



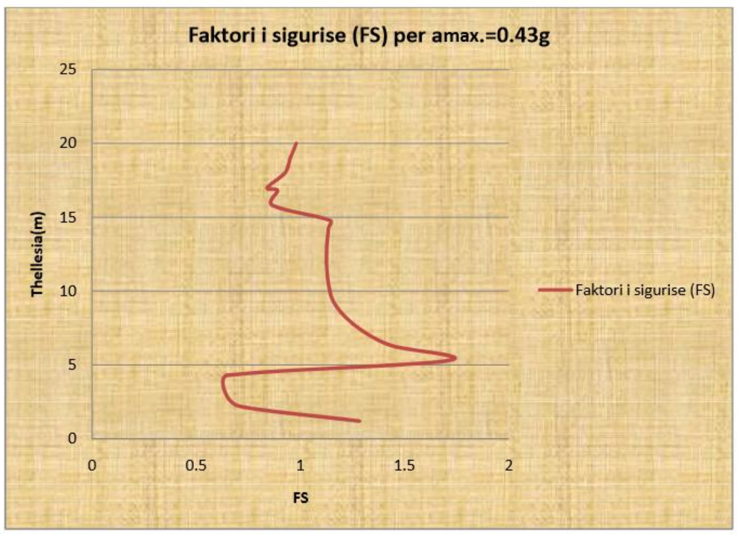


Figure 47. Assessment of liquefaction for 0.43g acceleration

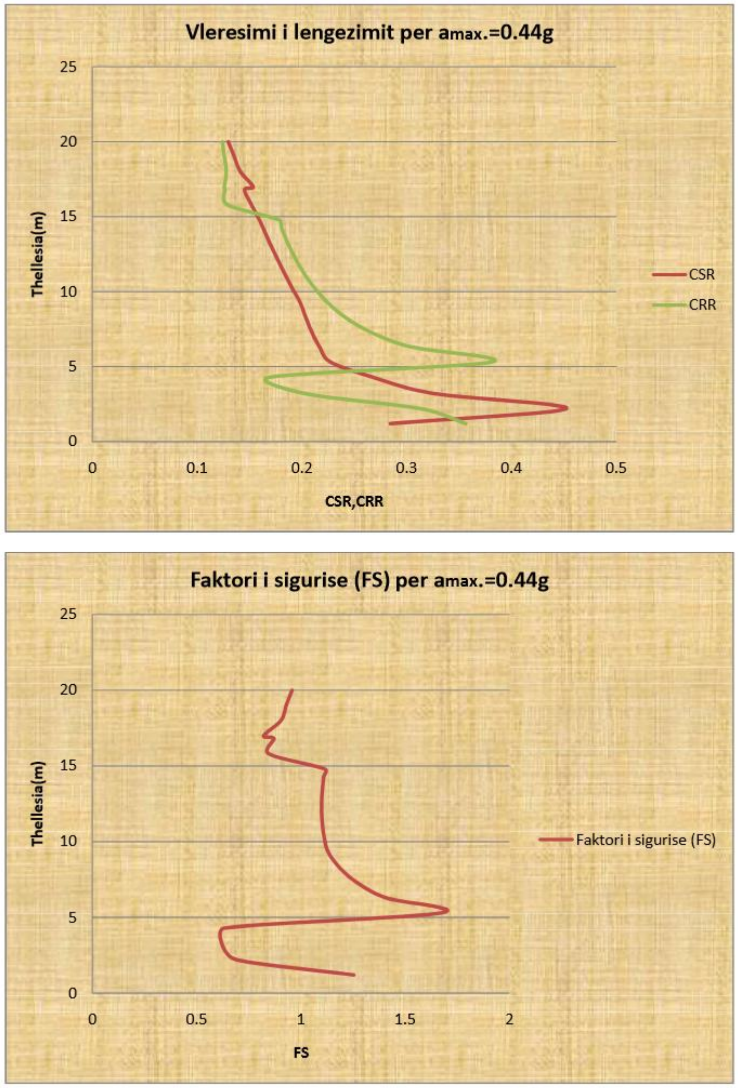
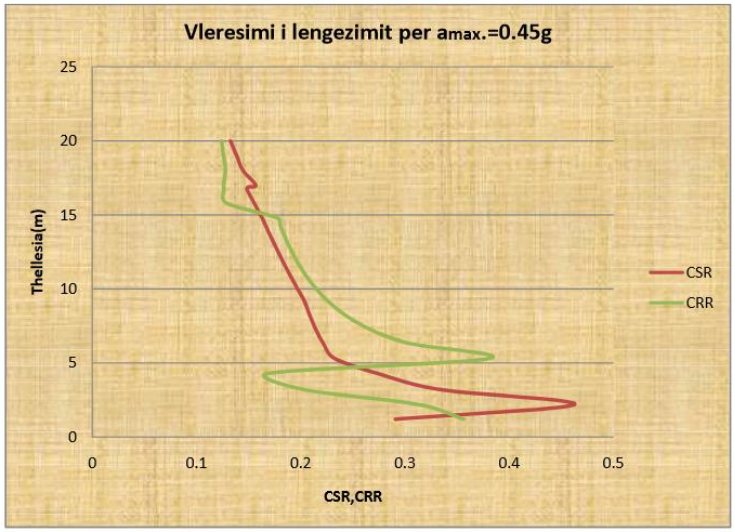


Figure 48. Assessment of liquefaction for 0.44g acceleration



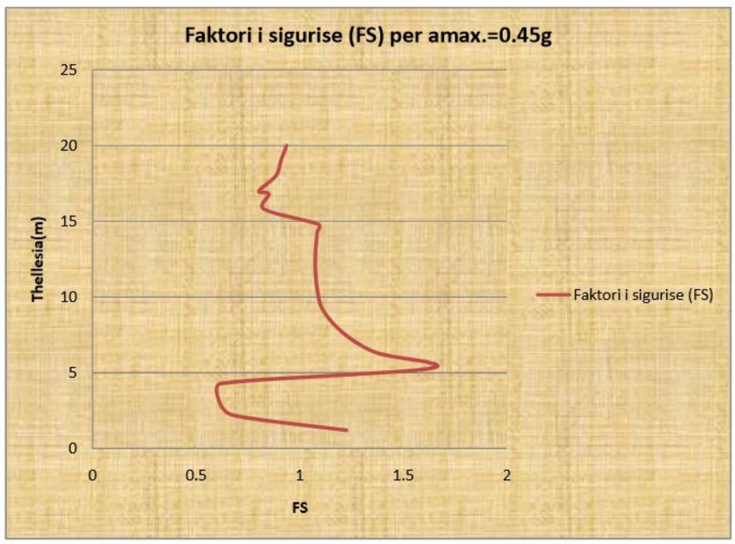


Figure 49. Assessment of liquefaction for 0.45g acceleration

Conclusions and recommendations

Being that liquefaction phenomenon is a very dangerous and catastrophic phenomenon, we should give a big importance to the measures for its projection there must be field and laboratory investigations and the information must be safe in order to evaluate right the potential of liquefaction. In the absence of the information from laboratory proofs (missing of equipment), may be considered enough representative during the fielding investigation.

It is recommended not to make calculations for liquefaction if NSPT>30 which has a water level higher than 15m, in all other cases it is obliged to make these calculations after it is decided in a safety way the potential of liquefaction, it is passed in the solution of the type of construction. In absence of the information from laboratory proofs (absence of above equipment) may be considered enough and representative the proofs during fielding investigations usage of vibration and compact techniques usage of chemical techniques after using the methods of improvement in land, it is recommended to make the field proof SPT, CPT and VS and the comparison of their results before the intervention in case if these results are negative we have to intervene again in the methods of improvement, in each case the control of quality of improvement is made from the proofs of fielding investigations.

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