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Dynamic Instability Evaluation of Coastal Saturated Loose Sands

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Abstract: *In this paper is presented a simplified liquefaction evaluation of coastal saturated loose sands. Dynamic excitation due to earthquakes results in relative displacements of soil grains. In conditions of water saturation and impossibility of fast drainage it is caused pore pressure increase as a result of gravity loads transferring to pore water. This process may induce the total reduction in initial effective stresses, which practically results in loss of soil strength frequently encountered in Adriatic coastal saturated loose sands. Such a state is defined as initial liquefaction occurrence expressed with strain levels increase causing severe consequences in touristic building structures realized in those areas.*

Definition of liquefaction depth, mostly influenced by relative density of saturated sands in high intensity seismic areas, it is an important factor to define the foundation type as a direct contact of structural load transmitting to the soil.

Keywords: *soil, liquefaction, saturated, effective stresses, pore pressure, seismic areas, gravity load*

1. Introduction

The Liquefaction phenomenon caused by earthquakes motions is observed in seismically active regions with typical form of settlement and tilting of civil engineering structures. The loss of soil strength or stiffness among the seismic ground motions is called Soil Liquefaction. This phenomenon is characteristic mainly of cohesionless water saturated sands, typical of coastal Vlora deposits. Shear waves induced due to earthquake, result in relative displacement of soil grains and a volume change tendency. In conditions of water-saturation and impossibility of fast drainage, the volume change tendency can not be realized. Thus, immediately increase of pore pressure cause the transfer of gravity loads from soil particles to pore water. The total reduction in initial effective stresses that takes places in soil, is known as loss of strength. This state is defined as liquefaction occurrence or cyclic mobility.

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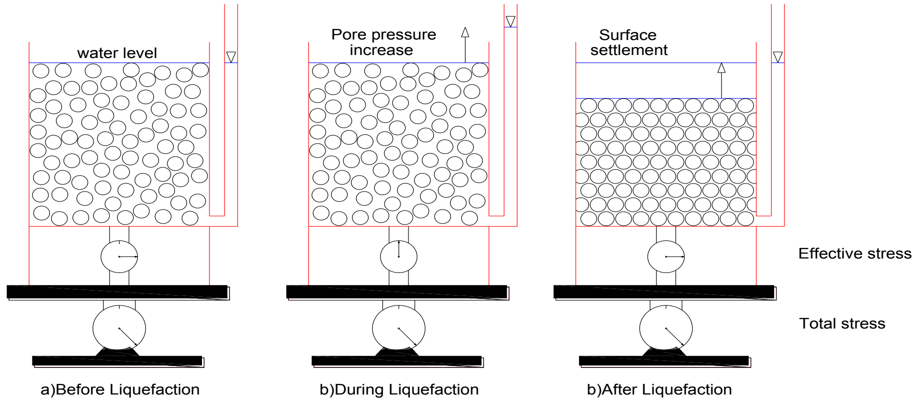


Fig. 1: Liquefaction mechanism of water saturated cohesionless materials (Ishihara 1985)

2. Mechanism and Definitions

Liquefaction is condition where soil undergo deformation with low residual resistance, due to the buildup of high water pressures (u) which reduce the effective pressure ($\sigma^0{}_1$) to a very low value. The liquefaction can occure due to static or cyclic stress applications. It depends on cohesionless sand relative density (Dr), effective presure or hydraulic gradient during upward flow of water in a sand deposit.

During the course of cyclic stress applications, the pore pressure becomes equal to the applied effective pressure. This condition denotes Peak Cyclic Pore Pressure Ratio of 100% that used for assessing subsequent soil behaviour.

The condition in which cyclic stress applications develop pore pressure equal to the applied effective pressure and subsequent cyclic stress cause limited strains is denoted as **Cyclic Mobility** or **Peak Cyclic Pore Pressure Ratio of 100% with Limited Strain Potential.**

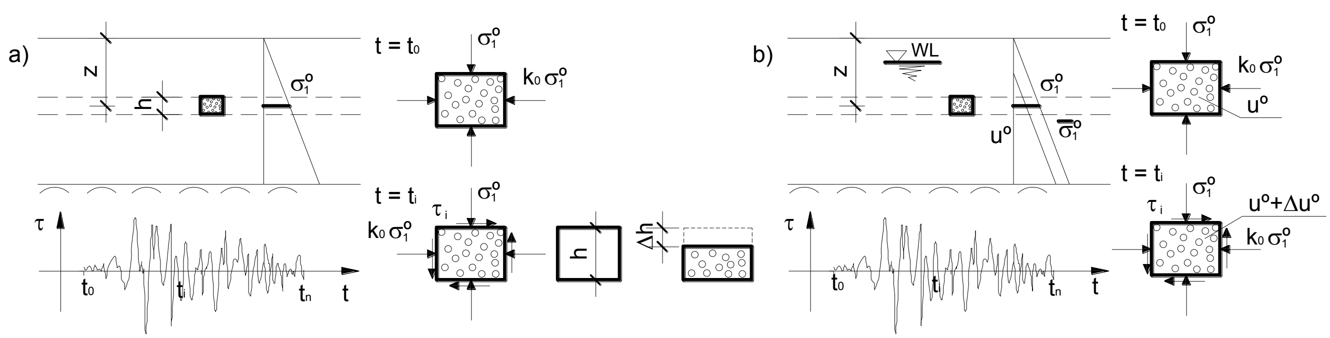


Fig. 2: a) Compaction Settlement; b) Liquefaction , Cyclic Mobility

a) $t \leq t_0 \quad \tau_f = \sigma^0{}_1 \mathrm{tg}\,\phi$ (1)

$t = t_i$; Compaction Settlement

b) $t \leq t_0 \quad \overline{\sigma}_1^{\,0} = \sigma_1^0 - u^0 \quad \tau_f^0 = \overline{\sigma}_1^{\,0} \tan\phi'$ (2) $t = t_i$

$$\overline{\sigma}_1^{\,i} = \sigma_1^0 - (u^0 + \Delta u^i) \quad \tau_f^i = \overline{\sigma}_1^{\,i} \tan\phi' \quad (3)$$

$$t = t_l \quad \Delta u^l = \overline{\sigma}_1^{\,0} \;;\; \overline{\sigma}_1^{\,l} = \sigma_1^0 - (u^0 + \Delta u^l) = \sigma_1^0 - u^0 - \overline{\sigma}_1^{\,0} = \overline{\sigma}_1^{\,0} - \overline{\sigma}_1^{\,0} = 0 \quad (4)$$

$\tau_f^l = \overline{\sigma}_1^{\,l} \tan\phi' = 0$; Initial Liquefaction. (5)

Cyclic stress aplications can result either in Liquefaction or in Cyclic Mobility. It depend of saturated sand type. Generally liquefaction can occure in loose saturated sands and cyclic mobility in medium to dense sands with relative density below a critical value and low values of effective pressure.

The cyclic stresses induced by earthquake shear waves in saturated sands causes excess or build up of hydrostatic pressures. During earthquake waves, the structure of sands tends to become more compact transferring stresses to the pore water reducing stresses on the soil grains.

As the pore presure becomes equal to effective pressure, the sands begin to undergo deformations

As a consecuence in loose saturated sands occure large deformations that may exceed ±20%. So, unlimited deformation of loose saturated sands without mobilizing resistance to deformation is called Liquefaction.

In case of dense sands after cyclic pore pressure becomes equal to effective pressure, the soil tend to dilate following with pore pressure drop and developing resistance to cyclic applied stress. As the cyclic stresses continues, the amount of deformation to produce a stable condition will be increased. So, for any subsequent induced stress cycle soil withstand without further deformation increase. This behaviour is called “Cyclic Mobility” or “Peak Cyclic Pore Pressure Ratio of 100% with Limited Strain Potential” .

3. Liquefaction or Cyclic Mobility Evaluation

Two base methods are used to evaluate Liquefaction or Cyclic Mobility in saturated sands subjected to earthquake waves.

3.1 Methods based on Previous Earthquake Observations

Collection of site conditions at various locations where some evidence of liquefaction or no liquefaction are developed from different authors using the field values of cyclic stress ratio $R\tau = \tau / \sigma^0_1$

(in which τ-horizontal shear stress induced by earthquake; σ^0_1- initial effective vertical pressure on the soil layer), relative density Dr, σ_0- total vertical stress on sand layer considered, Standart Penetration Resistance N,

a_{max}-maximum acceleration at the ground surface, r_d - stress reduction factor etc.

The limitations of those procedures are:

1) The lower bound causing cyclic mobility or liquefaction at high values of $R\tau = \tau / \sigma^0_1$ can not be defined.

2) The factors affecting cyclic mobility or liquefaction, such as shaking duration, earthquake magnitude can not take into account.

3) The penetration resistance may not be an appropriate as a cyclic mobility characteristic of soil and its value may vary significantly depending on the boring and sampling conditions.

With regard to the possibility that penetration resistance of sand may be not appropriate as liquefaction characteristic it is important to know the different factors as:

Relative density, grain structure, lateral earth presure coefficient K_0 value, prior seismic or shear strains to which the sand may have been subjected.

Generally the factors tending to increase the resistance to liquefaction or cyclic mobility tend to increase the penetration resistanceof sands.

The results based on stress evaluations using ground response analyses and detailed soil testing programs will be more realistic in evaluation of Cyclic Mobility or Liquefaction Potential at any particular site conditions.

3.2 Methods based on Analytical Evaluation of Stress Conditions in Field and Laboratory

Analytical evaluations of Liquefaction or Cyclic Mobility proposed by Seed and Idriss (‘74) consist in

1) Evaluation of cyclic shear resistance and expected cyclic shear stress induced by earthquake at different soil levels

2) Field and laboratory investigation to define for effective pressure at specific depth, peak cyclic pore presure ratio of 100% or peak cyclic presure ratio of 100% with limited strain.

Through these methods are defined analitical approaches to evaluate the stresses on potentially liqueifable layer during earthquake, procedures to convert irregular stress history by an equivalent uniform cyclic stress history, procedures for measuring the cyclic stress causing a peak pore pressure ratio of 100% or peak pore pressure ratio of 100% with limited strain, factors that influence on liquefaction or cyclic mobility characteristic of soils.

4. Evaluation of Liquefaction Potential Depth in coastal saturated cohesionles sands in Vlora, Albania

In building structures realised on coastal saturated sands it seems practical to evaluate Liquefaction Potential Depth z_L. It's definition can help structural designer in appropriate choice of foundation type. One practical method in evaluation of Liquefaction potential depth based on Seed and Idriss proposal ('74) is developed shortly in this paper presenting results for two cases.

- Saturated loose sands subjected to cyclic stresses induced by earthquake ground motion **without structure presence.**
- Saturated loose sands subjected to cyclic stresses induced by earthquake ground motion **with structure presence.**

This method is based on experimental and analitical comparison of cyclic shear stresses expected $\tau(e)$ and cyclic shear strength $\tau(r)$ in cohesionles saturated soils. Liquefaction Potential exist if $\boldsymbol{\tau(e)/\ \tau(r) > 1}$

$$\tau(e)=C_2\times C_3\times(a_{max}/g)\times\sigma_i\times r_d; \qquad (6)$$

$$\tau(r)=C_1\times R\tau\times\sigma^0_i; \qquad (7)$$

C_1- parameter based on laboratory test data $C_1= \tau_R/\ \tau_{lab}$; $C_1=1.5\div2.0$
C_2- parameter based on earthquake magnitude M $C_2=0.55, 0.65, 0.75$
C_3- parameter based on multidirectional shaking effect $C_3=1.1\div1.2$;

The Liquefaction Potential zone induced by earthquakes in cohesionless saturated sands under the building structures is analysed based on seismic and geological investigation for a building site situated in Vlora, Albania. The analitical procedure presented here involve the following steps:

1) Determination of the existing static stress conditions in the soil below and adjacent to a structure using an appropriate stress distribution analysis(e.g; finite element analyses).
2) Determination of cyclic shear stresses induced by earthquake motions on soil layers.
3) Determination of cyclic shear strength of soil layers.
4) Laboratory observations in test samples of the effects of the superimposed cyclic stresses in terms of pore water presures and strains they produce and stress-strain assesment of soil-structure system involved as well.

TABLE I: Liquefaction Potential Depth in Correlation with Cyclic Shear Strength $\tau(r)$ and Cyclic Shear Expected
No Building Structure Presence; Earthquake Magnitude M=6.75; WL=1.5m;

Soil Layers	γsat (under WL)	Layer Thickness under WL	Relative Density Dr	Effective Vertical Pressure σ^0_i	Depth z	Total Vertical Pressure σi	Cyclic Stress Ratio $R\tau=\tau/\ \sigma^0_i$ (Seed . Fig.4)	Cyclic Shear Strength τ(r)	Reduction factor rd (Tab.2)	Cyclic Shear Expected τ(e)
	(kN/m3)	(m)	(%)	(kN/m2)	(m)	(kN/m2)		(kN/m2)		(kN/m2)
1.00	19.10	1.00	54.00	31.65	2.00	36.55	0.14	6.65	1.00	7.27
2.00	19.10	0.50	60.00	38.61	2.75	50.88	0.16	9.27	1.00	10.11
3.00	19.60	2.00	60.00	50.73	4.00	75.25	0.16	12.17	0.98	14.66
4.00	19.50	5.00	60.00	84.74	7.50	143.60	0.18	22.88	0.94	26.83
5.00	20.00	10.00	70.00	159.92	15.00	292.35	0.23	55.17	0.70	40.68
6.00	20.00	10.00	70.00	261.82	25.00	492.35	0.23	90.33	0.56	54.81

$\tau(r)$- Cyclic Shear Strength; $\tau(e)$- Cyclic Shear Expected (kN/m^2)

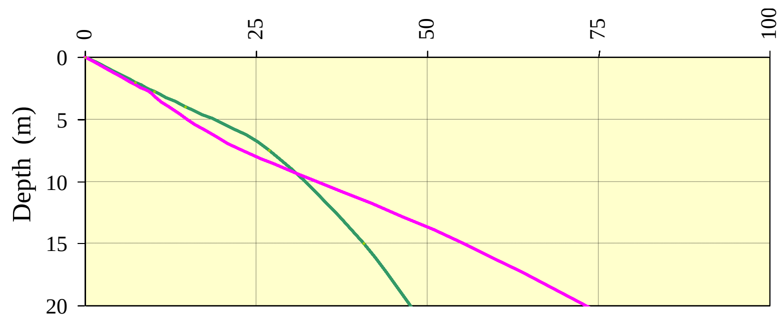


Fig. 3: Liquefaction Potential Depth; **No Building Structure Presence**

TABLE II: Correlation between Earthquake Magnitude and Cyclic Number required to cause t=0.65 tmax (Talaganov K. "Dynamic Soil Instabilities", 2001

Earthquake Magnitude	Cyclic Number
5.5	2÷3
6	5÷6
6.75	10
7.5	15
8.5	26

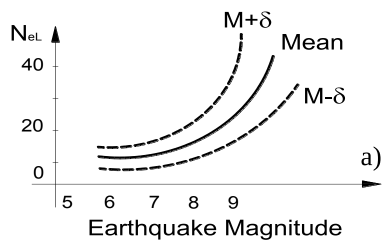
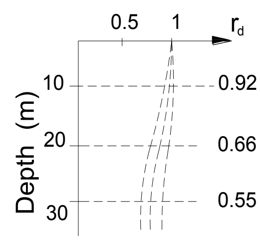


TABLE III: Stress Reduction Coefficient r_d (Seed H.B. 1979)

Depth (m)	Reduction factor; r_d $=\tau_{max(fl.)}/\tau_{max(rig)}$
0.00	1.00
5.00	0.98
10.00	0.92
15.00	0.82
20.00	0.66



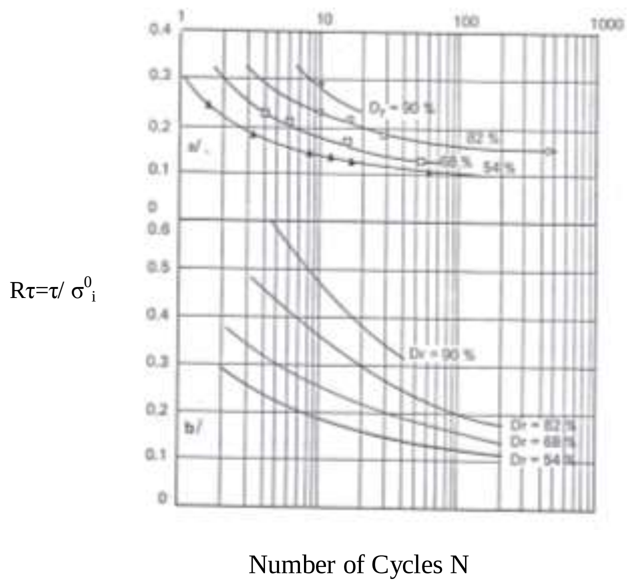


Fig. 4: Typical results of laboratory testing by cyclic shear stress: relationship between (Rτ), number of cycles (N) for different relative densitites (Dr); a) Peak Cyclic Pore Pressure Ratio of 100% b) Peak Cyclic Pore Pressure Ratio of 100% with Strain Potential 5%. (Seed 1979)

TABLE IV: Liquefaction Potential Depth in Correlation with Cyclic Shear Strength τ(r) and Cyclic Shear Expected Building Structure Presence; Earthquake Magnitude M=6.75; WL=1.5m; Building Structure Vertical Pressure **p=100kN/m²**; Foundation Depth=3m; B=20m; L=30m; L/B= 1.5;

Soil Layers	γ_{sat} (down WL)	K_0 (Tab.4)	Building Structure Overburden Vertical Pressure $\sigma(+)=K_0 \times p$	Layer Thickness under WL	Relative Density Dr	Effective Vertical Pressure σ^0_i	Depth z	Total Vertical Pressure σ_i	Cyclic Stress Ratio Rτ (Seed Fig.4)	Cyclic Shear Strength τ(r)	Reduction factor rd (Tab.2)	Cyclic Shear Expected τ(e)
	(kN/m3)		(kN/m2)	(m)	(%)	(kN/m2)	(m)	(kN/m2)		(kN/m2)		(kN/m2)
1.00	19.10	1.00	44.35	1.00	54.00	31.65	2.00	36.55	0.14	6.65	1.00	7.27
2.00	19.10	1.00	44.35	0.50	60.00	38.61	2.75	50.88	0.16	9.27	1.00	10.11
3.00	19.60	1.00	43.60	2.00	60.00	94.33	4.00	118.85	0.16	22.64	0.98	23.15
4.00	19.50	0.70	30.63	5.00	60.00	115.37	7.50	174.23	0.18	31.15	0.94	32.55
5.00	20.00	0.40	17.20	10.00	70.00	177.12	15.00	309.55	0.23	61.10	0.70	43.07
6.00	20.00	0.20	8.60	10.00	70.00	270.42	25.00	500.95	0.23	93.29	0.56	55.76

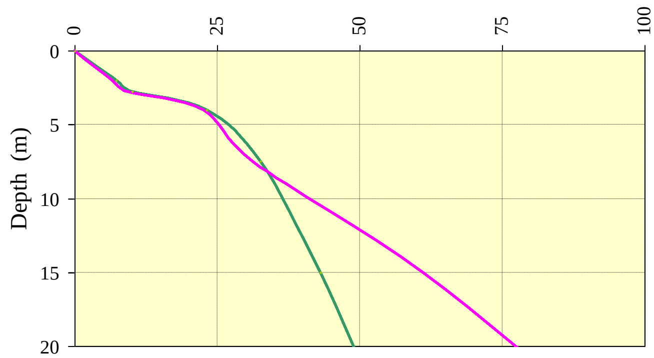
TABLE V: Liquefaction Potential Depth in Correlation with Cyclic Shear Strength τ(r) and Cyclic Shear Expected Building Structure Presence; Earthquake Magnitude M=6.75; WL=1.5m; Building Structure Vertical Pressure **p=200kN/m²**; Foundation Depth=3m; B=20m; L=30m; L/B= 1.5;

Soil Layers	γ_{sat} (down WL)	K_0 (Tab.4)	Building Structure Overburden Vertical Pressure $\sigma(+)=K_0 \times p$	Layer Thickness under WL	Relative Density Dr	Effective Vertical Pressure σ^0_i	Depth z	Total Vertical Pressure σ_i	Cyclic Stress Ratio Rτ (Seed Fig.4)	Cyclic Shear Strength τ(r)	Reduction factor rd (Tab.2)	Cyclic Shear Expected τ(e)
	(kN/m3)		(kN/m2)	(m)	(%)	(kN/m2)	(m)	(kN/m2)		(kN/m2)		(kN/m2)
1.00	19.10	1.00	144.35	1.00	54.00	31.65	2.00	36.55	0.14	6.65	1.00	7.27
2.00	19.10	1.00	144.35	0.50	60.00	38.61	2.75	50.88	0.16	9.27	1.00	10.11
3.00	19.60	1.00	143.60	2.00	60.00	194.33	4.00	218.85	0.16	46.64	0.98	42.63
4.00	19.50	0.70	100.63	5.00	60.00	185.37	7.50	244.23	0.18	50.05	0.94	45.63
5.00	20.00	0.40	57.20	10.00	70.00	217.12	15.00	349.55	0.23	74.90	0.70	48.64
6.00	20.00	0.20	28.60	10.00	70.00	290.42	25.00	520.95	0.23	100.19	0.56	57.99

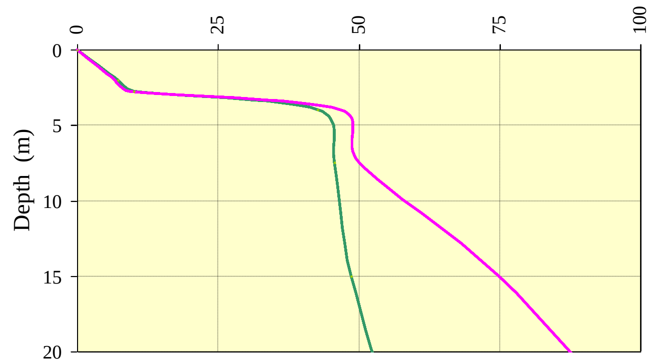
TABLE VI: Liquefaction Potential Depth in Correlation with Cyclic Shear Strength τ(r) and Cyclic Shear Expected Building Structure Presence; Earthquake Magnitude M=6.75; WL=1.5m; Building Structure Vertical Pressure **p=300kN/m²**; Foundation Depth=3m; B=20m; L=30m; L/B= 1.5;

Soil Layers	γ_{sat} (down WL)	K_0 (Tab.4)	Building Structure Overburden Vertical Pressure $\sigma(+)=K_0 \times p$	Layer Thickness under WL	Relative Density Dr	Effective Vertical Pressure σ^0_i	Depth z	Total Vertical Pressure σ_i	Cyclic Stress Ratio Rτ (Seed Fig.4)	Cyclic Shear Strength τ(r)	Reduction factor rd (Tab.2)	Cyclic Shear Expected τ(e)
	(kN/m3)		(kN/m2)	(m)	(%)	(kN/m2)	(m)	(kN/m2)		(kN/m2)		(kN/m2)
1.00	19.10	1.00	244.35	1.00	54.00	31.65	2.00	36.55	0.14	6.65	1.00	7.27
2.00	19.10	1.00	244.35	0.50	60.00	38.61	2.75	50.88	0.16	9.27	1.00	10.11
3.00	19.60	1.00	243.60	2.00	60.00	294.33	4.00	318.85	0.16	70.64	0.98	62.11
4.00	19.50	0.70	170.63	5.00	60.00	255.37	7.50	314.23	0.18	68.95	0.94	58.71
5.00	20.00	0.40	97.20	10.00	70.00	257.12	15.00	389.55	0.23	88.70	0.70	54.20
6.00	20.00	0.20	48.60	10.00	70.00	310.42	25.00	540.95	0.23	107.09	0.56	60.22

τ(r)- Cyclic Shear Strength; τ(e)- Cyclic Shear Expected (kN/m²) τ(r)- Cyclic Shear Strength; τ(e)- Cyclic Shear Expected (kN/m²)

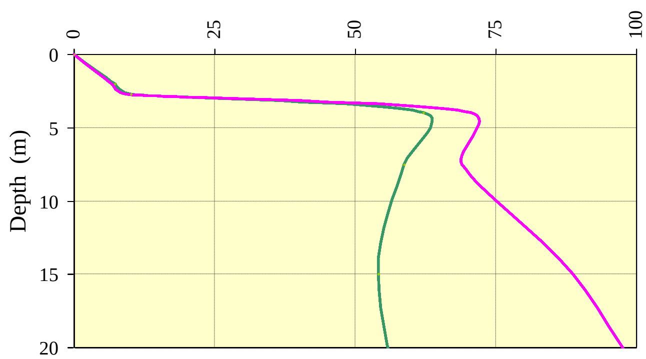


Structure Pressure **p=100kN/m^2**



Structure Pressure **p=200kN/m^2**

τ(r)- Cyclic Shear Strength; τ(e)- Cyclic Shear Expected (kN/m^2)



Structure Pressure **p=300kN/m^2**

Fig.5: Liquefaction Potential Depth;
Building Structure Presence

5. Conclusions

1) The Liquefaction Potential zone in loose saturated sand below center of building structure is smaller compare to case of no building presence. It can be explained by higher values of initial effective vertical stresses of soil layers under the building structure conditions than in soil layers in natural conditions. So, the cyclic shear stress required to cause liquefaction increase significantly with increase of initial effective pressure.
2) Increase of the building structure weight founded on the loose saturated sands decrease the Liquefaction potential zone of soil layers under the center the building structure.
3) Increase of relative density in saturated sands increase the cyclic shear stresses required to cause Liquefaction induced by earthquake.
4) Increase of relative density in saturated sands increase the number of cycles for development of a peak cyclic pore presure ratio of 100%
5) Increase of Earthquake shaking intensity and maximum ground acceleration, increase the Liquefaction potential zone in saturated sand layers.
6) Increase of the lateral earth pressure coefficient value, increase the cyclic shear stress required to cause liquefaction.

Based also in recent investigations it can conclude that the Liquefaction Potential is influenced also by such important factors as:

- characteristic of the grains composing the cohesionless sands.
- structure of the grains.
- seismic history of cohesionless sands.
- age of the cohesionless sands.

6. References

[1] Seed, H.B., and Lee, K.L., "Liquefaction of Saturated Sands During Cyclic Loading", *Journal of the Soil Mechanics and Foundations Division*, *ASCE*,Vol.92,No.SM6, Nov. 1966.

[2] Seed, H.B.,and Idriss, I.M., "Simplified Procedure for Evaluating Soil Liquefaction Potential ", *Journal of the Soil Mechanics and Foundations Division, ASCE*, Vol.97, No. SM9, Sept. 1971.

[3] Talaganov K., "Dynamic Soil Instabilities", *Lecture Notes for the Postgraduate Studies*, Skopje 2001

[4] Casagrande, A., "Liquefaction and Cyclic Deformation of Sands- A Critical Review", *Harvard Soil Mechanics Series No.88,* Harvard University , Cambridge, Mass., 1976

[5] Castro, G., "Liquefaction and Cyclic Mobility of Saturated Sands", *Journal of the Geotechnical Engineering Division, ASCE*, Vol. 101, No. GT6, 1975

[6] Castro, G., and Poulos, S.J., "Factors Affecting Liquefaction and Cyclic Mobility", *Journal of the Geotechnical Engineering Division, ASCE*, Vol. 103, No. GT6, 1977

[7] Ishihara, K.and Yasuda, "Sand Liquefaction Due to Irregular Exitation", *Soil and Foundation*, Tokyo, Japan, 1972

[8] Lee,K.L., and Seed, H.B., "Cyclic Stress Conditions Causing Liquefaction of Sand", *Journal of the Soil Mechanics and Foundations Division*, *ASCE,* Vol.93, No.SM1, Jan. 1967.

[9] Martin, G.R., Finn, W.D., and Seed, H.B., "Fundamentals of Liquefaction under Cyclic Loading", *Journal of the Geotechnical Engineering Division, ASCE*, Vol. 101, No. GT5, May 1975.

[10] Seed, H.B., "Some Aspects of Sand Liquefaction Under Cyclic Loading ", *Proceedings Conference on Behaviour of Off-Shore Structures,* The Norwegian Institute of Technology, Norway, 1976.

[11] Seed, H.B., Mori, K., and Chan, C.K., "Influence of Seismic History on Liquefaction of Sands", *Journal of the Geotechnical Engineering Division, ASCE*, Vol. 103, No. GT4, April 1977.

[12] Tanimoto, K., and Suga, Y., "On the Dynamic Strength of Saturated Sand", *Proceedings, Third Japan Earthquake Engineering Symposium*, Tokyo, Japan 1970.

[13] Annaki, M., "Liquefaction of Sand in Triaxial Tests using Uniform and Irregular Cyclic Loading," *thesis presented to the University of California,* at Los Angeles, *1975*

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