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A STUDY ON LIQUIFACTION

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ABSTRACT: Liquefaction is the phenomena when there is loss of strength in saturated and cohesion-less soils because of increased pore water pressures and hence reduced effective stresses due to dynamic loading. It is a phenomenon in which the strength and stiffness of a soil is reduced by earthquake shaking or other rapid loading. It is therefore important to understand the conditions under which different types of soil liquefy. The liquefaction potential of a soil layer can be determined through either laboratory tests on undisturbed soil samples or from in situ tests. A combination of these two methods can also be used in soil liquefaction analysis. Initially, the soil is tested for vulnerability to liquefaction. Geological studies identify the landfill sediments, and water table in a seismic region. Soil composed of a mix of small and big grains, can safely endure liquefaction, since the smaller grains fill the pores between the larger grains. By conducting the Atterberg limits (Liquid limit, Plastic limit, Shrinkage limit), SPT, CPT tests we observe that this soils are not susceptible to soil liquefaction.

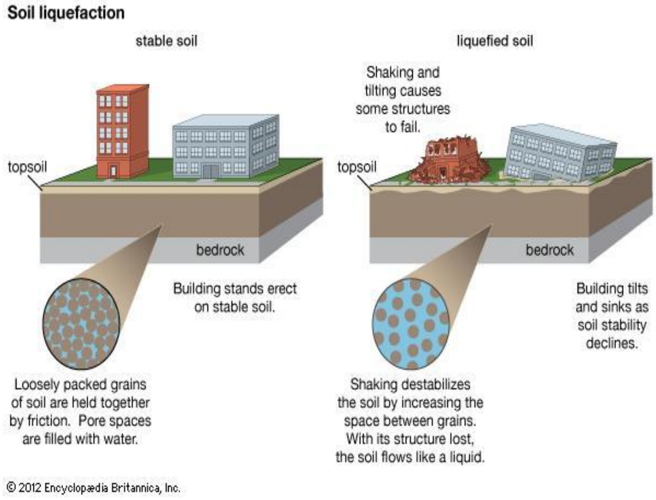
I. INTRODUCTION

Liquefaction occurs in saturated soils and saturated soils are the soils in which the space between individual particles is completely filled with water. This water exerts a pressure on the soil particles that. The water pressure is however relatively low before the occurrence of earthquake. But earthquake shaking can cause the water pressure to increase to the point at which the soil particles can readily move with respect to one another. Examples of such soils are silt sands, clay sands, and sands containing impermeable sediments. Although earthquakes often triggers this increase in water pressure, but activities such as blasting can also cause an increase in water pressure. When liquefaction occurs, the strength of the soil decreases and the ability of a soil deposit to support the construction above it.

Fig: Liquifation in Nigata ,Japan (1964) and in Adhapazari Turkey(1999)

Overburden correction factor CN for penetration resistances

Cause behind liquefaction



DATA USED:

The datas used were the SPT case datas from two major earthquakes, namely Chi-Chi, Taiwan earthquake (magnitude Mw =7.6) and Kocaeli, Turkey earthquake (magnitude Mw = 7.4) in 1994 as given by Adel M. Hanna, Derin Ural, . 40 numbers of datas were analyzed using semi-empirical procedures for evaluating the liquefaction potential

Calculation and Table

The following three tables gives the calculation for the values of CSR and CRR, followed by the assessment of liquefaction potential from these values found out by the semi-empirical method. Table 3 gives a comparison between these results and the actual on-field results as given in the paper by Soil grains in a soil deposit. The height of The blue column to the right represents the Level of pore-water pressure in the soil. The length of the arrows represents the size of the contact forces between individual soil grains. The contact forces are large when the pore-water pressu

II. SPT-BASED PROCEDURE FOR EVALUATING LIQUEFACTION POTENTIAL OF COHESIONLESS SOILS

Semi-empirical procedures for the liquefaction potential analysis was developed using the Standard Penetration Test (SPT) for differentiating between liquefaction and non-liquefaction conditions in the 1964 Navigate earthquake, Japan. In this paper we have used the semi-empirical approach for differentiating between liquefaction and non-liquefaction conditions for 40 SPT cases the two major earthquakes, namely Chi-Chi, Taiwan earthquake (magnitude MW =7.6) and Kochab, Turkey earthquake (magnitude MW = 7.4) in 1999. Thus following the semi-empirical approach, the CSR and (N1)60 values were re-calculated using the revised rd, MSF, K and CN relations recommended herein.

Evaluation of CSR

The K factor is usually applied 10th capicity side of the analysis during design but it must also be used to convert the CSR [Boulanger and Idriss (2004) It is given as follows:

$$(CSR)_{M-7.5} = 0.65\left(\frac{\sigma_{vo} a_{max}}{\sigma'_{vo}}\right)\frac{r_d}{MSF}\frac{1}{K_\sigma}$$

Evaluation of CRR

For the CRR value, at first the SPT penetration resistance was adjusted by Boulanger and Idriss (2004) [to an equivalent clean sand value: The value of the CRR for a magnitude of earthquake=7.5 and an effective vertical stress of 1 at can be calculated on the basis of the value of (N1)60cs using the following expression:

DATA Used

$$CRR - exp\left\{\frac{(N_1)_{60cs}}{14.1} + \left(\frac{(N_1)_{60cs}}{126}\right)^2 - \left(\frac{(N_1)_{60cs}}{23.6}\right)^3 + \left(\frac{(N_1)_{60cs}}{25.4}\right)^4 - 2.8\right\}$$

$$(N_1)_{60cs} = (N_1)_{60} + \Delta(N_1)_{60}$$

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

The datas used were the SPT case datas from two major earthquakes, namely Chi-Chi, Taiwan earthquake (magnitude Mw =7.6) and Kocaeli, Turkey earthquake (magnitude Mw = 7.4) in 1994 as given by Adel M. Hanna, Derin Ural, Gokhan saygili Neural network model for liquefaction potential in soil deposists using Turkey and Taiwan eartquake data Soil Dynamics and Earthquake Engineering 27 (2007) 521 540. 40 numbers of datas were analyzed using semi-empirical procedures for evaluating the liquefaction potential.

Calculation and Table

The following three tables gives the calculation for the values of CSR and CRR, followed by the assessment of liquefaction potential from these values found out by the semi-empirical method. Table 3 gives a comparison between these results and the actual on-field results as given in the paper by Adel M. Hanna, Derin Ural, Gokhan Saygili, Neural network model for liquefaction potential in soil deposits using Turkey.

Calculation of CSR by semi-empirical method using SPT case datas

SI NO	Z	rd	MSF	(NI)60	σ'_{vo}/p_a	σ_{vo}	K	σ_{vo}/σ'_{vo}	CSR
1	1	0.77	1.03	6	0.08	0.14	1.16	1.16	0.2
2	1.8	0.78	1.03	8	0.09	0.2	1.14	1.5	0.26
3	2.6	0.77	1.03	7	0.08	0.27	1.11	1.67	0.3
4	3.4	0.78	1.03	5	0.08	0.34	1.08	1.77	0.32
5	4.2	0.77	1.03	5	0.08	0.41	1.07	1.83	0.33
6	5	0.78	1.03	3	0.07	0.47	1. 05	1.89	0.35
7	6	0.77	1.03	3	0.07	0.55	1.04	1.94	0.36
8	7	0.77	1.03	19	0.13	0.65	1.06	1.95	0.36
9	8	0.76	1.03	26	0.17	0.75	1.05	1.96	0.36
10	9	0.76	1.03	48	0.81	0.84	1.14	1.97	0.34

Calculation of CRR by semi-empirical method using SPT case datas

SI NO	FC	$\Delta(N_1)_{60}$	$(N_1)_{60}$	$(N_1)_{60CS}$	CRR
1	90	5.51	6	11.51	0.129
2	94	5.50	8	13.5	0.144
3	100	5.49	7	13.49	0.136
4	87	5.52	5	10.52	0.122
5	74	5.56	5	10.56	0.122
6	92	5.51	3	8.51	0.108
7	97	5.49	3	8.49	0.108
8	70	5.57	19	54.57	0.28
9	58	5.61	26	31.61	0.607
10	5	0.0019	48	48.0019	162.26

Assessment of liquefaction potential using semi-empirical method for SPT case

SI NO	CRR	CSR(S)	Performance function Z=R-S	Liquefaction result	Actual liquifaction
1	0.129	0.199	-0.070	YES	NO
2	0.144	0.26	-0.166	YES	NO
3	0.136	0.297	-0.161	YES	NO
4	0.122	0.319	-0.197	YES	NO
5	0.122	0.332	-0.210	YES	YES
6	0.108	0.349	-0.241	YES	NO
7	0.108	0.361	-0.253	YES	NO
8	0.280	0.359	-0.079	YES	YES
9	0.607	0.362	0.245	YES	NO
10	162.260	0.335	161.930	YES	NO

III. CPT-BASED PROCEDURE FOR EVALUATING LIQUEFACTION POTENTIAL OF COHESIONLESS SOILS

Seed and Idriss (1981) as well as Douglas et al (1981) proposed the use of correlations between the SPT and CPT to convert the then available SPT-based charts for use with the CPT. The CPT-based liquefaction correlation was re-evaluated by Idriss and Boulanger (2003) using case history data compiled by Shibata and Teparaksa (1988), Kayen et al (1992), Boulanger et al (1995, 1997), Stark and Olson (1995), Suzuki et al (1997) and Moss (2003).

The re-evaluation of CPT cases will include the same adjustments and perimeter revisions as in case of SPT re-evaluation. The CSR adjustment remains same as in case of the SPT cases but the CRR qC1N will be adjusted according to the different values of tip resistance (qc).

3.3.1. Evaluation of CSR
The K Factor is usually applied to the "capacity" side of the analysis during design but it must also be used to convert the CSR as given by Boulanger and Idriss (2004)[1]. It is given as follow

Evaluation of CRR
The revised CRR qC1N relation, derived using the considerations can be expressed as follows:

$$CRR = exp\left\{\frac{q_{c1N}}{540} + \left(\frac{q_{c1N}}{67}\right)^2 - \left(\frac{q_{c1N}}{80}\right)^3 + \left(\frac{q_{c1N}}{114}\right)^4 - 3\right\}$$

Where, $q_{c1}=c_N q_c$
The datas used were the CPT case datas from two major earthquakes, namely Chi-Chi, Taiwan earthquake (magnitude Mw =7.6) and Kocaeli, Turkey earthquake (magnitude Mw=7.4)in 1994 as given by Adel M. Hanna Derin Ural Gokhan Saygili "Evalutuion of liquefaction potential of soil deposit using artificial neural networks"[3]. 28 Numbers of datas we are analyzed using semi-empirical procedures for evaluating the liquefaction potential.

Calculation and Table
The following three table's gives the calculation for the values of CSR and CRR, followed by the assessment of liquefaction potential from these values found out by the semi-empirical method. Table 6 gives a comparison between these results and the actual on-field results as given in the paper by Adel M. Hanna Derin Ural Gokhan Saygili."Evalutuion of liquefaction potential of soil deposit using artificial neural networks".

$$(CSR)_{M=7.5} = 0.65\left(\frac{\sigma_{vo} a_{max}}{\sigma'_{vo}}\right)\frac{r_d}{MSF}\frac{1}{K_\sigma}$$

Calculation of CSR by semi -empirical method using CPT datas

SI NO	Z	Rd	MSF	$\sigma_{Vo}\ \sigma^{l}_{Vo}$	CSR
1	3.60	0.780	1.027	1.76	0.348
2	4.80	0.780	1.027	1.97	0.389
3	5.80	0.780	1.027	1.65	0.326
4	3.60	0.796	0.970	1.42	0.136
5	17.80	0.793	0.970	1.75	0.355
6	7	0.796	0.970	1.75	0.355
7	3.20	0.797	0.970	1	0.203
8	9.60	0.795	0.970	1.82	0.651
9	8.60	0.795	0.970	1.81	0.357
10	4.4	0.778	1.03	1.81	0.357

Calculation of CRR by semi -empirical method using CPT datas

SI NO	C_n	qc1	qc1N	CRR
1	1.63	941.33	9.32	0.0520
2	1.27	1533.53	15.18	0.0540
3	1.37	1013.66	10.04	0.0517
4	1.54	2808.96	27.80	0.0599
5	0.74	1030.45	10.20	0.0518
6	1.16	2195.53	21.74	0.0565
7	1.40	1400.42	13.87	0.0531
8	1	5285.30	52.33	0.0798
9	1.02	4831.33	47.80	0.0754
10	1.50	2283.75	22.60	0.0565

Assessment of liquefaction potential using semi-empirical method for CPT case datas

SI NO	CRR(R)	CSR	FS=CRR/CSR	Liquefaction result	Actual liquifaction
1	0.0520	0.348	0.149	yes	No
2	0.0540	0.389	0.139	Yes	Yes
3	0.0517	0.326	0.159	Yes	No
4	0.0599	0.136	0.440	Yes	yes
5	0.0518	0.174	0.298	Yes	No
6	0.0565	0.355	0.159	Yes	yes
7	0.0531	0.203	0.262	Yes	No
8	0.0798	0.651	0.117	Yes	yes
9	0.0754	0.646	0.117	Yes	Yes
10	0.0569	0.357	0.159	yes	Yes

Laboratory tests for soils

- Grain size analysis
- Liquid Limit Test
- Plastic limit Test
- Shrinkage limit Test Grain Size Analysis

Determination of quantitative size distribution of particles of soil down to fine grained fraction.

Apparatus: Sieves are 4.75(mm), 2.36(mm), 1.18(mm), 600(μ), 425(μ), 300(μ), 150(μ), 75(μ), pan

Procedure:
The portion retained on the No. 10 sieves is tested for grain-size distribution by passing the sample through a number of sieves are stacked in order, with the sieve with the largest size opening at the top. the sieves should be agitated. By when the sieving operation has been completed, the weight of the soil particles retained on each sieve is determined, from which the percentage passing each sieve can be computed.

Sieve size	Weight of soil retained	% Weight retained on each sieve	Cumulative Retained %	Percent Finer (%)
4.75 (mm)	163	4.075	4.075	95.925
2.36 (mm)	183	4.575	8.65	91.35
1.18 (mm)	334	8.35	17.00	83.00
600 (μ)	511	12.775	29.775	70.225
425 (μ)	772	19.3	49.075	50.925
300 (μ)	457	11.425	60.5	39.5
150 (μ)	977	24.425	84.925	15.075
75 (μ)	386	9.65	94.575	5.425
Pan	110	2.75	97.325	2.675
Total	3893			

Water content(W)(ml)		No. of blows(N)
30	(W1)	41 (N1)
52.5	(W2)	35 (N2)

% finer
Sieve sizes: Grain-size Analysis: % weight retained = (163/3893) x100 =4.2%

LIQUID LIMIT: To find the liquid limit of a given sample of soil this test has been using Formula:
Liquid limit of soil sample = Mass of water/ Mass of dry soil
Flow Index If = W1-W2/log(N1/N2) 52.5-30/log(41-35)

28.9 Liquid limit (LL)=52.5/150=35%
LP PLASTIC LIMI TEST: To determine the plastic limit of a given sample this test will be use full.
Formula: Plastic limit = Mass of water/ Mass of dry soil = 30/100=0.3%
Plastic index = (LL PL) =(0.35-0.3)=0.05 < 5

Lliquid limit PL - Plastic limit
SHRINKAGE LIMIT: This test is used to determine the shrinkage limit of soi
Formula: Shrinkage limit (Ws) = [W-(V-V0) x ϒw/W0] x 100= [100-(23-12)x 9.81/36]*100 = 243%

IV. CONCLUSIONS

We observe the results from the laboratory tests in that we can see the soil behavior and characteristics of the soil. From that we can conclude the soil is not susceptible to liquefaction
[1] Thus it can be concluded that the Reliability liquefaction probability analysis model gives us lower error percentage for the SPT case data's(37.5%) and
[2] olsen method gives lower percentage for the CPT case data's(35.7%).Hence,from the limited studies done in this paper wemay state the above but for more accurate results more earthquake case datas and other methodologies are to implemented.
[3] From the error percentages of SPT and CPT case datas studied in this paper, it can be said that CPT datas gives better results concerning liquefaction potential but for practical purposes the above cant be surely concluded.For accurate results,more earthquake case datas and other methodologies are to be implemented.
[4] From the grain size analysis test the % of weight retained in the sample of soil is 4.2%.From the atterberg limits the values of L.L= 0.35%,P.L=0.3%, S.L=24.3% ,so this values are not susceptible to soil liquefaction

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