Analysis of Liquefaction of Soil using SPT Data

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Abstract: Soil liquefaction is a phenomenon whereby saturated or partially saturated loose cohesionless soil substantially losses strength and stiffness in response to a applied stress, usually earthquake shaking or other sudden change in stress condition, causing it to behave like liquid. All the structures which are built over the soil which is susceptible to Liquefaction can be damaged during an Earthquake even if they are structurally strong. As one of the most hazardous events is discussed, certain analysis for soil are to be performed to understand the behavior of soil and its stability towards such actions on different sites and determining the liquefaction susceptibility.In this paper we have analysed liquefaction potential of ten sites (2 from Visakhapatnam, 4 from Bangalore and 4 from Delhi region) as per the simplified procedure provided in Annexure-F of IS 1893-Part1(2016). This analysis will help in Liquefaction Mapping of our country (India), which finally help in preliminary understanding of liquefaction susceptibility of the site chosen for construction. Mitigation measures to reduce liquefaction susceptibility of soil such as Deep Dynamic compaction, Vibro-Compaction and Stone columns and suitability of each method are also discussed in this paper. As per our analysis Visakhapatnam is very safe, Bangalore is moderately safe and Delhi region is not safe against liquefaction.

Keywords: soil, liquefaction

I GENERAL INTRODUCTION

A. Definition

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.

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.

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.

Soil liquefaction can also exert higher pressure on retaining walls, which can cause them to slide or tilt. This movement can cause destruction of structures on the ground surface and settlement of the retained soil.

B. Cause behind liquefaction

It is required to recognize the conditions that exist in a soil deposit before an earthquake in order to identify liquefaction. Soil is basically an assemblage of many soil particles which stay in contact with many neighboring soil. The contact forces produced by the weight of the overlying particles holds individual soil particle in its place and provide strength.

Occurrence of liquefaction is the result of rapid load application and break down of the loose and saturated sand and the loosely-packed individual soil particles tries to move into a denser configuration. However, there is not enough time for the pore-water of the soil to be squeezed out in case of earthquake. Instead, the water is trapped and prevents the soil particles from moving closer together. Thus, there is an increase in water pressure which reduces the contact forces between the individual soil particles causing softening and weakening of soil deposit. In extreme conditions, the soil particles may lose contact with each other due to the increased pore-water pressure. In such cases, the soil will have very little strength, and will behave more like a liquid than a solid - hence, the name "liquefaction".

C. Past records of liquefaction

Earthquakes accompanied with liquefaction have been observed for many years. In fact, written records dating back hundreds and even thousands of years have descriptions of earthquake effects that are now known to be associated with liquefaction. However, liquefaction has been so common in a number of recent earthquakes that it is often considered to be associated with them. Some of those earthquakes are

(1) Niigata, Japan (1964)
(2) Alaska, USA (1964)
(3) Loma Prieta, USA (1989)
(4) Kobe, Japan (1995)
(5) Bhuj Earthquake, India (2001)

D. Methods of reducing liquefaction hazards

There are basically three methods of reducing hazards liquefaction hazards:

1) By Avoiding Liquefaction Susceptible Soils
Construction on liquefaction susceptible soils is to be avoided. It is required to characterize the soil at a particular building site according to the various criteria available to determine the liquefaction potential of the soil in a site

2) Build Liquefaction Resistant Structures
The structure constructed should be liquefaction resistant i.e., designing the foundation elements to resist the effects of liquefaction if at all it is necessary to construct the structure on liquefiable soil because of favorable location, space restriction and other reasons.

3) Improve the Soil
This involves mitigation of the liquefaction hazards by improving the strength, density and drainage characteristics of the soil. This can be done using variety of soil improvement techniques.

II SIMPLIFIED PROCEDURE

Step 1- The subsurface data used to assess liquefaction susceptibility should include the location of the water table , either SPT blow count N or tip resistance q_c of a CPT cone or shear wave velocity V_s, unit weight and fines content of the soil (percent by weight passing the IS standard sieve no 75µ) .
Step 2- Evaluate total vertical overburden stress σ_{vo} and effective vertical overburden stress σ'_{vo} at different depths for all potentially liquefiable layers within deposit

Step 3- Evaluate stress reduction factor reducing:

r_d= 1-0.00765*Z for 0<Z<9.15m

=1.174-0.0267*Z for 9.15m<Z<23m

Where,

Z = depth in metres

Step 4 - Calculate cyclic stress ratio CSR induced by the earthquake using:

$$\mathbf{CSR = 0.65 \times (a_{max}/g) \times (\sigma_{vo}/ \sigma'_{vo}) \times r_d}$$

Where,
a_{max}=peak ground acceleration preferably in terms of g,

G=acceleration due to gravity, and

R_d =stress reduction factor

If value of PGA is not available the ratio**a_{max}/g**may be taken equal to seismic zone factor, Z

Siesmic Zone Factor, Z				
Zone	**II**	**III**	**IV**	**V**
Z	**0.10**	**0.16**	**0.24**	**36**

Step 5- Obtain cyclic resistance ratio CRR by correcting standard cyclic resistance $CRR_{7.5}$ for earthquake magnitude, high overburden stress level and high initial static shear strength using:

$$\mathbf{CRR=CRR_{7.5}*(MSF)*K\sigma*K\alpha}$$

Where,$CRR_{7.5}$=standard cyclic resistance ratio for 7.5 magnitude earthquake obtained using values of SPT
MSF= magnitude scaling factor given by following equation

$$\mathbf{MSF = 10^{2.24}/m_w^{2.56}}$$

This factor is required when the magnitude is different than 7.5 the correction for high overburden pressure is high and can be found using following equation

$$\mathbf{K\alpha=(\sigma'_{vo} /P_a)^{(f-1)}}$$

Where σ'_{vo} effective overburden pressure P_a atmospheric pressure are measured in the same units and f is an exponent and its value depends on the relative density D_r for D_r=40 percent ~60 percent, f=0.8~0.7 and for D_r =60 PERCENT ~80 PERCENT, f=0.7~0.6. The correction for static shear stresses K**α** is required only for sloping ground and is not required in routine engineering practice. Therefore, in the scope of this standard, value of Kα shall be assumed unity.

STEP 6- Obtain cyclic resistance ratio CRR7.5

Evaluate the SPT blow count N_{60} , for a hammer efficiency of 60 percent .

$$\mathbf{(N_1)_{60} = C_N N_{60},}$$

Where,

$$\mathbf{C_N = \sqrt{(P_a / \sigma'_{vo})} \leq 1.7,}$$

The cyclic resistance ratio $CRR_{7.5}$ is estimated from fig 4.1, using $(N_1)_{60}$ and finding $(N_1)_{60cs}$ as follows :
$$\mathbf{(N_1)_{60cs} =\alpha+\beta (N_1)_{60},}$$

Where,
$$\mathbf{\alpha=0 \quad \beta = 1 \text{ For FC} \leq 5\%}$$

$\alpha = e^{1.76-(190/F.C*2)}$ $\beta = 0.99 + FC^{1.5}/1000$ for 5% <FC< 35% and
$\alpha = 0.5$ $\beta = 1.2$ for FC $\geq$ 35 %

$$CRR_{7.5} = \frac{1}{34-(N1)60CS} + \frac{(N1)60CS}{135} + \frac{50}{(10*(N1)60CS+45)2} - \frac{1}{200}$$

Step 7 – Calculate the factor of safety FS against liquefaction using:

FOS = CRR/CSR

Where CSR is as estimated in step 4 and CRR in step 5.When the design ground motion is conservative, earthquake related permanent ground deformation is generally small , if FS≥1.2.

Step 8 - if FS<1, then the soil is said to be liquef

III. CALCULATIONS

A. Hyderpur, Delhi.

S.No.	Depth(m)	σ(kN/m^2)	σ'(kN/m^2)	r_d	CSR	N_{60}	FC (%)	$(N_1)_{60CS}$	$CRR_{7.5}$	MSF	CRR	FOS	Comments
1.	2.0	37.2	37.2	0.984	0.1532	15	60.2	30.2	0.482	1.0	0.482	3.15	**SAFE**
2.	4.0	75.2	75.5	0.969	0.1514	28	27.5	55.5	0.359	1.0	0.359	2.37	**SAFE**
3.	6.0	114	114	0.954	0.1481	38	22.0	56.6	0.370	1.0	0.370	2.50	**SAFE**
4.	8.0	152	132	0.938	0.1686	41	56.9	43.5	0.213	1.0	0.213	1.26	**SAFE**
5.	10.0	189.2	149.2	0.907	0.1795	45	56.9	44.9	0.236	1.0	0.236	1.31	**SAFE**
6.	12.0	228.4	168.4	0.853	0.1803	59	22.0	71.3	0.496	1.0	0.496	2.75	**SAFE**
7.	14.0	267.6	187.6	0.800	0.1787	61	15.0	56.8	0.372	1.0	0.372	2.08	**SAFE**

B. Rohini, Delhi.

S.No.	Depth(m)	σ(kN/m^2)	σ'(kN/m^2)	r_d	CSR	N_{60}	FC (%)	$(N_1)_{60CS}$	$CRR_{7.5}$	MSF	CRR	FOS	Comments
1.	2.0	39.40	39.40	0.984	0.1531	16	70.2	31.2	0.594	1.0	0.594	3.88	**SAFE**
2.	4.0	77.71	77.71	0.969	0.1514	26	37.0	36.1	0.555	1.0	0.555	3.67	**SAFE**
3.	6.0	114.96	114.96	0.954	0.1487	32	18.0	35.2	0.556	1.0	0.556	3.74	**SAFE**
4.	8.0	153.22	153.22	0.938	0.1464	39	60.0	38.5	0.059	1.0	0.059	0.40	**UNSAFE**
5.	10.0	193.24	153.46	0.907	0.1570	44	56.0	40.8	0.152	1.0	0.152	0.97	**UNSAFE**
6.	12.0	223.00	193.00	0.853	0.1602	54	52.0	47.4	0.271	1.0	0.271	1.69	**SAFE**
7.	14.0	253.08	173.08	0.800	0.1822	65	12.0	52.8	0.330	1.0	0.330	1.81	**SAFE**

C. Noida Sector-1, Delhi.

S.No.	Depth(m)	σ(kN/m^2)	σ'(kN/m^2)	r_d	CSR	N_{60}	FC (%)	$(N_1)_{60CS}$	$CRR_{7.5}$	MSF	CRR	FOS	Comments
1.	1.5	27.1	27.1	0.9885	0.1542	10	60.2	20.9	0.227	1.0	0.227	1.47	**SAFE**
2.	3.0	55.0	55.0	0.9771	0.1524	18	56.9	29.9	0.461	1.0	0.461	3.02	**SAFE**
3.	4.5	83.3	83.3	0.9656	0.1506	19	27.5	28.2	0.377	1.0	0.377	2.50	**SAFE**
4.	6.0	112.4	112.4	0.9541	0.1488	14	22.0	18.4	0.196	1.0	0.196	1.32	**SAFE**
5.	7.5	141.5	141.5	0.9426	0.1470	20	15.0	20.3	0.219	1.0	0.219	1.49	**SAFE**
6.	9.0	170.8	165.8	0.9321	0.1498	28	16.5	25.9	0.311	1.0	0.311	2.08	**SAFE**
7.	10.5	200.5	180.5	0.8937	0.1549	32	13.6	27.1	0.341	1.0	0.341	2.20	**SAFE**
8.	12.0	230.2	195.2	0.8536	0.1570	47	12.2	36.5	0.134	1.0	0.134	0.85	**UNSAFE**
9.	13.5	260.4	210.4	0.8136	0.1571	56	12.0	41.4	0.167	1.0	0.167	1.06	**SAFE**
10.	15.0	290.5	225.5	0.7735	0.1554	62	12.0	44.4	0.228	1.0	0.228	1.47	**SAFE**

D. Noida Sector-6, Delhi.

S.No.	Depth(m)	σ(kN/m^2)	σ'(kN/m^2)	r_d	CSR	N_{60}	FC (%)	$(N_1)_{60CS}$	$CRR_{7.5}$	MSF	CRR	FOS	Comments
1.	1.5	26.3	26.1	0.9885	0.1542	6	60.5	12.7	0.138	1.0	0.138	0.89	**UNSAFE**
2.	3.0	54.2	54.2	0.9770	0.1524	8	60.0	13.5	0.145	1.0	0.145	0.95	**UNSAFE**
3.	4.5	82.1	82.1	0.9655	0.1506	7	55.5	9.8	0.111	1.0	0.111	0.74	**UNSAFE**
4.	6.0	110.8	110.8	0.9541	0.1488	8	40.0	9.6	0.109	1.0	0.109	0.73	**UNSAFE**
5.	7.5	139.9	139.9	0.9426	0.1470	12	38.0	12.7	0.138	1.0	0.138	0.94	**UNSAFE**
6.	9.0	169	169	0.9311	0.1498	20	35.0	18.9	0.203	1.0	0.203	1.36	**SAFE**
7.	10.5	198.7	183.7	0.8936	0.1507	16	32.0	18.0	0.192	1.0	0.192	1.27	**SAFE**
8.	12.0	228.7	198.4	0.8536	0.1532	15	20.0	14.2	0.152	1.0	0.152	0.99	**UNSAFE**
9.	13.5	258.1	213.1	0.8135	0.1537	17	14.0	13.1	0.142	1.0	0.142	0.92	**UNSAFE**
10.	15.0	288.1	228.1	0.7735	0.1524	20	12.0	13.7	0.147	1.0	0.147	0.96	**UNSAFE**

E. Shanthi Nagar, Bangalore.

S.No.	Depth(m)	σ(kN/m^2)	σ'(kN/m^2)	r_d	CSR	N_{60}	FC (%)	$(N_1)_{60CS}$	$CRR_{7.5}$	MSF	CRR	FOS	Comments
1	1.5	28.95	28.95	0.9885	0.0642	9	24.8	21.3	0.232	1.442	0.335	5.21	**SAFE**
2	3	57.45	47.45	0.9770	0.0768	5	65.9	7.4	0.091	1.442	0.131	1.71	**SAFE**
3	4.5	86.7	61.7	0.9655	0.0881	9	27.5	12.9	0.139	1.442	0.200	2.28	**SAFE**
4	6	115.5	75.5	0.9541	0.0948	7	5.0	4.5	0.068	1.442	0.098	1.03	**SAFE**
5	7.5	144.6	89.6	0.9426	0.0988	5	3.0	2.6	0.056	1.442	0.081	0.82	**UNSAFE**
6	9	173.7	103.7	0.9311	0.1013	16	2.0	7.0	0.087	1.442	0.125	1.24	**SAFE**

F. RK Hegde Nagar, Bangalore.

S.No.	Depth(m)	σ(kN/m^2)	σ'(kN/m^2)	r_d	CSR	N_{60}	FC (%)	$(N_1)_{60CS}$	$CRR_{7.5}$	MSF	CRR	FOS	Comments
1.	1.5	28.95	28.95	0.9885	0.0643	7	31.41	18.7	0.201	1.442	0.290	4.51	**SAFE**
2.	3	87.45	77.45	0.9771	0.0717	9	22.02	15.1	0.160	1.442	0.231	3.22	**SAFE**
3.	4.5	165.45	145.45	0.9656	0.0714	10	17	11.8	0.132	1.442	0.190	2.67	**SAFE**
4.	6	286.65	246.65	0.9546	0.0721	13	11	9.7	0.113	1.442	0.163	2.26	**SAFE**
5.	7.5	442.65	392.15	0.9426	0.0692	8	15	6.7	0.092	1.442	0.133	1.92	**SAFE**
6.	9	629.85	559.85	0.9321	0.0682	9	4	3.7	0.060	1.442	0.087	1.27	**SAFE**

G. Halasuru, Bangalore.

S.No.	Depth(m)	σ(kN/m^2)	σ''(kN/m^2)	r_d	CSR	N_{60}	FC (%)	$(N_1)_{60CS}$	$CRR_{7.5}$	MSF	CRR	FOS	Comments
1	1.5	28.95	28.95	0.9885	0.0642	9	65.4	36.4	0.150	1.442	0.216	3.37	**SAFE**
2	3.0	58.2	48.2	0.9770	0.0766	5	24.5	23.5	0.266	1.442	0.384	5.01	**SAFE**
3	4.5	87	62	0.9655	0.0880	9	4.9	17.8	0.190	1.442	0.274	3.11	**SAFE**
4	6.0	115.8	75.8	0.9541	0.0947	7	3.2	15.0	0.160	1.442	0.231	2.44	**SAFE**
5	7.5	144.6	89.6	0.9426	0.0988	5	4.6	39.3	0.100	1.442	0.144	1.46	**SAFE**
6	9.0	173.4	103.4	0.9311	0.1014	16	1.5	44.5	0.230	1.442	0.332	3.27	**SAFE**

H. Hesaragatta, Bangalore.

S.No.	Depth(m)	σ(kN/m^2)	σ'(kN/m^2)	r_d	CSR	N_{60}	FC (%)	$(N_1)_{60CS}$	$CRR_{7.5}$	MSF	CRR	FOS	Comments
1.	1.50	30	30	0.9885	0.0643	19	48	39.3	0.098	1.442	0.141	2.20	**SAFE**
2.	3.50	70	57	0.9732	0.0777	28	43	45.1	0.239	1.442	0.345	4.44	**SAFE**
3.	4.50	90	67	0.9656	0.0843	26	60	38.9	0.079	1.442	0.114	1.35	**SAFE**
4.	6.00	120	82	0.9541	0.0908	41	48	55.1	0.356	1.442	0.513	5.65	**SAFE**
5.	7.50	150	97	0.9426	0.0947	55	37	67.8	0.468	1.442	0.675	7.13	**SAFE**
6.	9.00	180	112	0.9312	0.0973	100	28	109.1	0.790	1.442	1.139	11.71	**SAFE**
7.	10.50	210	127	0.8937	0.0961	100	28	102.5	0.740	1.442	1.067	11.10	**SAFE**
8.	12.50	250	147	0.8403	0.0929	100	28	95.9	0.690	1.442	0.995	10.71	**SAFE**

I. Med. Tech. Zone, Visakhapatnam.

S.No.	Depth(m)	σ(kN/m^2)	σ'(kN/m^2)	r_d	CSR	N_{60}	FC (%)	$(N_1)_{60CS}$	$CRR_{7.5}$	MSF	CRR	FOS	Comments
1	2	39.5	39.5	0.9847	0.0640	26	43.4	50.4	0.308	1.442	0.444	6.94	**SAFE**
2	4	79.5	65.6	0.9694	0.0760	42	38.5	62.9	0.427	1.442	0.616	8.10	**SAFE**
3	5	99.6	75.6	0.9617	0.0820	48	38.6	66.7	0.459	1.442	0.662	8.07	**SAFE**
4	6.5	130.35	91.3	0.9502	0.0880	55	64.7	69.8	0.484	1.442	0.698	7.93	**SAFE**
5	8	161.1	107.1	0.9388	0.0910	62	59.5	72.6	0.507	1.442	0.731	8.03	**SAFE**
6	9	181.3	117.3	0.9311	0.0930	78	3.63	71.7	0.500	1.442	0.721	7.75	**SAFE**

J. SBI Colony, Visakhapatnam.

S.No.	Depth(m)	σ(kN/m^2)	σ'(kN/m^2)	r_d	CSR	N_{60}	FC (%)	$(N_1)_{60CS}$	$CRR_{7.5}$	MSF	CRR	FOS	Comments
1.	2.0	38.10	38.10	0.9847	0.0640	35	39	68.9	0.477	1.442	0.688	10.75	**SAFE**
2.	4.0	76.98	58.48	0.9694	0.0829	> 100	4.7	----	----	1.442	----	----	**VERY SAFE**
3.	6.5	125.56	82.06	0.9503	0.0945	63	45.9	84.4	0.600	1.442	0.865	9.16	**SAFE**
4.	8.0	154.13	95.53	0.9388	0.0985	71	48.7	88.2	0.631	1.442	0.910	9.24	**SAFE**
5.	9.0	173.66	105.16	0.9321	0.1001	78	4.6	76.4	0.538	1.442	0.776	7.75	**SAFE**
6.	10.0	193.66	115.16	0.9070	0.0991	> 100	4.2	----	----	1.442	----	----	**VERY SAFE**

IV. CONCLUSION

Thus it can be concluded that Visakhapatnam is very safe, Bangalore is moderately safe and Delhi region is not safe against liquefaction. Hence, from the limited studies done in this paper we may state the above but for more accurate results more bore hole log reports should beanalyzed.

Liquefaction Susceptiblesites

- Rohini, Delhi
- Sector-1Noida
- Sector-6Noida
- Shanthinagar,Bangalore

Moderately Safe sites

- Hyderpur, Delhi
- Halasuru, Bangalore
- Hesaraghatta, Bangalore
- R.K Hegdenagar, Bangalore

Very Safesites

- Med .Tech zone,Visakhapatnam
- SBI Colony,Visakhapatnam

Deep dynamic compaction should be the first option for increasing liquefaction resistance of soil if the soils fail up to 10 m deep during analysis, as it is very cheap and also it will increases the bearing capacity, reduces the settlement and reduces the substructure cost. Vibro-Compaction is preferred in pure sands. Vibro-Stone columns are suggested when there is low drainage due to presence of silts and clays in sand.

REFERENCES

1. IS 1893:Part-1(2016), Criteria for Earthquake Resistance Design of Structures.
2. IS 2131 (1981): Method for standard penetration test for soils [CED 43: Soil and Foundation Engineering]
3. "Geotechnical investigation report", Andhra Pradesh Med. Tech Zone, Visakhapatnam, Andhra Pradesh. By IDAX CONSULTING & RESEARCH PVT. LTD.
4. "Geotechnical investigation report", SBI Colony, Visakhapatnam, Andhra Pradesh. By IDAX CONSULTING & RESEARCH PVT. LTD.
5. "Geotechnical investigation report", Halasuru, Banglore. By PATIL ENGINEERS & CONTRACTORS.
6. "Geotechnical investigation report", Hesaraghatta, Banglore. By PATIL ENGINEERS & CONTRACTORS.
7. "Geotechnical investigation report", R.K. Hegde Nagar, Banglore. By PATIL ENGINEERS & CONTRACTORS.
8. "Geotechnical investigation report", Shanthi Nagar, Banglore. By PATIL ENGINEERS & CONTRACTORS.
9. "Liquefaction Hazard Assessment Using SPT and V_S for Two Cities in India", By K S RAO, Dr D NeelimaSatyam , IIIT Hyderabad.
10. Sladen, J. A., D□Hollander, R. D., and Krahn, J. _1985_. „„,The liquefaction of sands, a collapse surface approach.□□ Can. Geotech. J., 22, 564– 578.
11. Finn, W. L., Ledbetter, R. H., and Wu, G.: Liquefaction in silty soils: design and analysis, Ground failures under seismic conditions, Geotechnical Special Publication No 44, ASCE, Reston, 51–79, 1994
12. Castro, G., (1975) Liquefaction and cyclic mobility of saturated sands. Journal of the Geotechnical Engineering Division, ASCE, 101 (GT6), 551-569.
13. Seed, H. B. 1979. "Soil Liquefaction and Cyclic Mobility Evaluation for Level Ground During Earthquake," Journal of Geotechnical Engineering Division, ASCE, Vol 105, No. GT2, pp 201-225.
14. Selig, E.T., and Chang C.S.(1981), "soil failure modes in undrained cyclic loading" J. Geotech. Engg. Div.,ASCE, Vol.107, No.GT5, May, pp 539-551
15. Robertson, P.K.1994, "suggested terminology for liquefaction":An Internal CANLEX Report
16. Robertson, P.K.and Fear, C.E. (1996), "Soil liquefaction and its evaluation based on SPT and CPT", Liquefaction Workshop, January 1996
17. National Research Council□s Committee on Earthquake Engineering (1985) Krinitzky et al.1993
18. M. Idriss and R. W. Boulanger, "Semi-empirical Procedures for Evaluating Liquefaction Potential During Earthquakes", Proceedings of the 11th ICSDEE & 3rd ICEGE, pp 32 – 56, January 7 – 9, 2004.
19. Jin-Hung Hwangand Chin-Wen Yang, "A Practical Reliability-Based Method for Assessing Soil Liquefaction Potential", Department of Civil Engineering, National Central University.
20. Adel M. Hanna, Derin Ural and GokhanSaygili, "Evaluation of liquefaction potential of soil deposits using artificial neural networks".
21. Poulos, S.J., Castro, G., and France, W., 1985. Liquefaction evaluation procedure, J. Geotechnical Engineering Div., ASCE, Vol. 111, No.6, pp. 772-792.
22. Ishihara, K., "Liquefaction and Flow Failure during earthquakes (Rankine Lecture)". Geotechnique, 43 (3): 351-415, 1993
23. Koester, J.P. (1994) "The Influence Of Fine Type And Content On Cyclic Strength" Ground Failures Under Seismic Conditions, Geotechnical Special Publication No. 44, ASCE, pp. 17-33.
24. Robertson, P.K., Woeller, D.J. & Finn, W.D.L. 1992. Seismic cone penetration test for evaluating liquefaction potential under cyclic loading. Canadian Geotech. Jnl, 29, 686695.

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