EVALUATION OF SOIL LIQUEFACTION IN SRI LANKA - A DYNAMIC APPROACH USING GEOTECHNICAL INVESTIGATION DATA

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Abstract: Soil Liquefaction is a process, where granular soils below the ground water table temporarily lose its strength and behaved as a viscous fluid rather than a solid. The porewater pressure is suddenly increased during an earthquake due to the cyclic loading. The increased porewater pressure is forced the soil particles to suspend in water. As a result, the buildings, utility services, natural substances and other structures are collapsed causing severe damage to the people and the nature. In Sri Lanka, it is rare to find the historical data of liquefaction or related incidents. Recent studies demonstrated that there is a potential for liquefaction in some places of the island. It could be evaluated by using the basic geotechnical investigation data, according to the simplified procedure proposed by Seed and Idriss (1971). This paper is intended to evaluate the liquefaction potential in Sri Lanka by identifying the liquefiable layer thickness, using an extensive geotechnical investigation data base. As per the analysis 384 locations were identified as susceptible for liquefaction with various layer thicknesses out of 3282 locations analysed. Further, the analysis has extended for varied ground water table.

Keywords: liquefaction, ground acceleration, magnitude, earthquake

1.0 Introduction

Liquefaction is a process by which granular soils below the water table temporarily lose their strength and behave as a viscous fluid rather than a solid. The types of granular soils most susceptible for liquefaction are cohesionless sand and silts. The process of liquefaction is as follows. Seismic event is generating shear waves and those waves are passing through saturated granular layers. Then the granular structure of soil is distorting and cause loosely packed groups of particles to collapse. These collapses increase the pore-water pressure between the grains if drainage cannot occur.

If the pore-water pressure rises to a level approaching the weight of the overlying soil, the granular layer temporarily behaves as a viscous liquid rather than a solid. Thus liquefaction has occurred.

In the liquefied condition, soil may deform with little shear resistance but deformations are large enough to cause damage to buildings and other structures. The easiness with which a soil can be liquefied depends primarily on the looseness of the soil, the amount of cementation between particles, and the amount of drainage restriction. The amount of soil deformation following liquefaction depends on the looseness of the material, the depth, thickness, and areal extent of the liquefied layer, the ground slope, and the distribution of loads applied by buildings and other structures.

Liquefaction could be observed in different forms. These include, flow failures, lateral spreads, ground oscillation, loss of bearing strength, settlement, sand boiling and increased lateral pressure on retaining walls.

2.0 Evaluation of Liquefaction

There are number of methods to evaluate the liquefaction potential. Most convenient method is Simplified procedure developed by Seed & Idriss (1971) which was accepted by many scientists in various regions. The procedure has evolved over time and still been used worldwide to analyze liquefaction resistance of soil.

2.1 Simplified Procedure

Seed & Idriss (1971) considered a soil column as a rigid body. When the seismic loading is excited at the base of the soil column, the shear wave propagates to the ground surface. Then the shear stress is generated in the soil column and could be estimated by using the empirical equations proposed.

2.2 Evaluation of Cyclic Stress Ratio (CSR)

Seed & Idriss (1971) suggested that a value of 65% of the maximum shear stress is reasonably accurate to find the liquefaction potential (Figure 1). The reason is the shear stress variation showing a jagged shape during an earthquake.

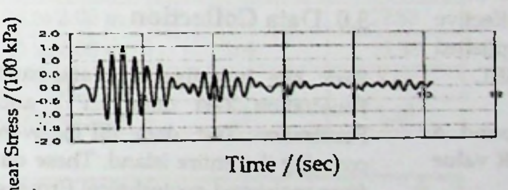

Figure 1 : Variation of Shear Stress With Time During an Earthquake

$$CSR = 0.65 \frac{a_{max}}{g} \frac{\sigma_{vo}}{\sigma'_{vo}} r_d \qquad (1)$$

Where,

a_{max} = maximum ground acceleration

σ_{vo} = Total stressat the depth Z

σ'_{vo} = Effective stress at the depth Z

r_d =a correction factor for depth given by,

$$r_d = \frac{1.0 - 0.4113 z^{0.5} + 0.04052 z + 0.001753 z^{1.5}}{1.0 - 0.4177 z^{0.5} + 0.05729 z - 0.006205 z^{1.5} + 0.00121 z^2} \qquad (2)$$

2.3 Maximum Ground Acceleration (a_{max})

Abayakoon (1998) showed the expected value of maximum ground acceleration in Sri Lanka could be 0.28g. With the historical records of earthquakes triggered in the close vicinity of Sri Lanka, the above value is quite reasonable. In the past, earthquakes of average magnitude 6.0, occurred in several places in and around Sri Lanka. Correlating to the Mercalli Intensity, it was matched with intensity group VII. Expected maximum ground acceleration for group VII lies between 0.15g and 0.25g. Thus it is more accurate to use maximum ground acceleration as 0.2g for the analysis.

2.4 Evaluation of Cyclic Resistance Ratio (CRR)

Several field tests can be used to evaluate the CRR, including the Standard Penetration Test (SPT) and the Cone Penetration Test (CPT).

2.4.1 CRR in Terms of SPT

Seed and Idriss (1971) developed correlations between the SPT 'N' value and the cyclic stress ratio to cause liquefaction during earthquakes of magnitude, M = 7.5 on the basis of field data from many earthquake events. Seed and Idriss (1971) drew a curve separating liquefied from non-liquefied sites. They noted that this curve corresponded to cyclic resistance ratios, CRR, for sands with 5% or less fines content (FC). These increases in CRR were approximated by two additional CRR curves for FC = 35% and FC = 15% (Figure 2). These critical CRR curves are also shown in all curves which appear as dotted lines where they are not adequately restrained by field data.

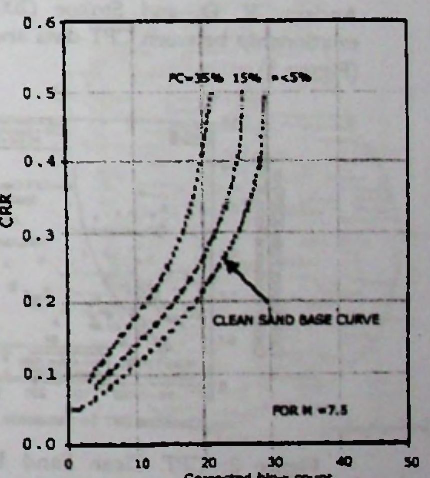

Figure 2 : *SPT* Clean Sand Base Curve for Earthquake Magnitude of 7.5

Here the corrected blow count is taken as $(N1)_{60}$.

The equivalent clean sand value of $(N1)_{60cs}$ can be calculated by applying constants, α and β, which are functions of fines content (Table 1).

$$(N_1)_{60cs} = \alpha + \beta \times (N_1)_{60} \tag{5}$$

The values of α and β can be calculated by using the following equations.

Table 1: Values of α and β against Fines Content

Description	α	β
for FC ≤ 5%	0	1.00
for 5% ≤ FC ≤ 35%	exp [1.76 – (190/FC²)]	[0.99 + (FC^1.5/1000)]
for FC ≥ 35%	5.0	1.2

It was noted that as per the equation 03, $(N1)_{60cs}$ should be less than 34. If $(N1)_{60cs}$ is exceeding 34, the value of CRR is negative. However with the historical data, it was observed that the soils with higher $(N1)_{60cs}$ values are not susceptible for liquefaction.

2.4.2 CRR in Terms of CPT

Robertson P.K. and Wride C.E. (1997) developed that the relationships between CPT tip resistance and the CRR. Normalization of the cone penetration resistance to an effective confining pressure of 100 kPa is accomplished in a manner similar to that used for the SPT.

Andrus, R. D. and Stokoe (2000) found a relationship between CPT data and CRR value (Figure 3).

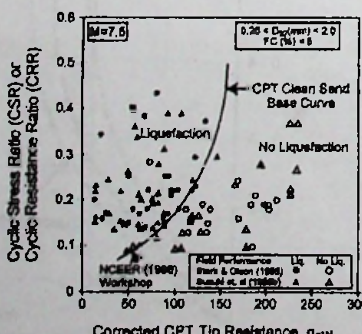

Figure 3: *CPT* Clean Sand Base Curve for Earthquake Magnitude of 7.5

2.5 Magnitude Scaling Factor (MSF)

The SPT and CPT clean sand base curves can only be applied to earthquake with magnitude (M) of 7.5. For earthquake magnitudes other than 7.5, a magnitude scaling factor (MSF) should be applied. Youd et al (2001) suggested the following correlations to determine the MSF.

For earthquake magnitudes (M) other than 7.5,

Lower bound

$$MSF = 173.78 \times M^{-2.56} \tag{8}$$

Upper bound

$$MSF = 173.84 \times M^{-2.56} \tag{9}$$

Then the corrected CRR value,

$$CRR = CRR_{M=7.5} \times MSF \tag{10}$$

Finally the factor of safety against soil liquefaction can be written as,

$$FOS = CRR \div CSR \tag{11}$$

If the factor of safety is greater than 1.0, liquefaction may not be triggered. Thus, liquefaction is likely to be occurred when the factor of safety is less than 1.0.

3.0 Data Collection

Basic site investigation data, such as Cone Penetration Test data (CPT) and Standard Penetration Test data (SPT), were collected covering the entire island. These data collected from registered geotechnical firms, government organizations on their acknowledgement.

Figure 4: Mapped Data Collection

4.0 Analysis and Results

Initial analysis was done by assuming an earthquake event of 6.0 in Richter scale, which is feasible to trigger in Sri Lanka. Among the analyzed locations, 35% locations (Table 2 & 3) were found to be susceptible for soil liquefaction as per the simplified procedure.

Table 2: Details of Boreholes Analyzed.

Description	Value
Total No of Boreholes	3282
Disregarded No of Boreholes due to non-availability of cohesionless soil layer.	1797
Disregarded No of Boreholes due to non-availability of ground water table above the granular layers considered.	400
Effective No of Boreholes subjected to analyze.	1085

Table 3: Liquefiable Locations with Layer Thickness.

Granular Soil Layer Thickness (m)	No of Locations susceptible for liquefaction	Percentage (%)
0 to 1.00 m	17	1.57
1.00 to 2.00 m	137	12.63
2.00 to 4.00 m	169	15.58
4.00 to 6.00 m	48	4.42
Above 6.00 m	13	1.20

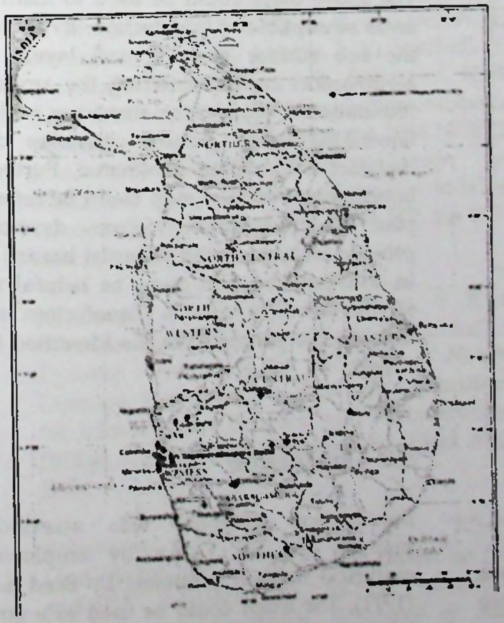

Figure 5: Liquefiable Layer Thickness up to 1 m

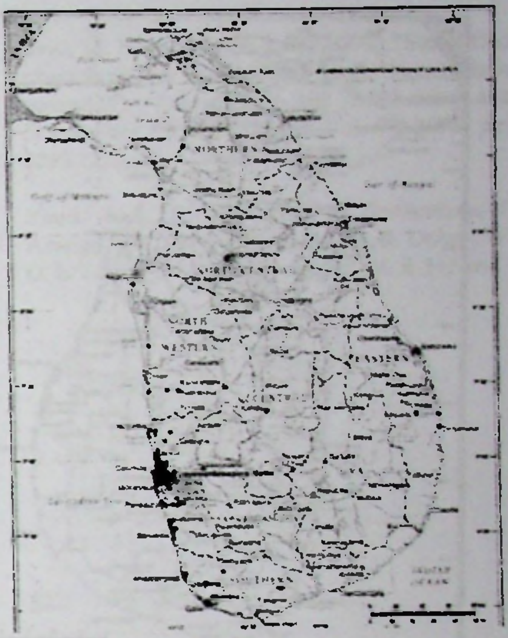

Figure 6: Liquefiable Layer Thickness 1 to 2 m

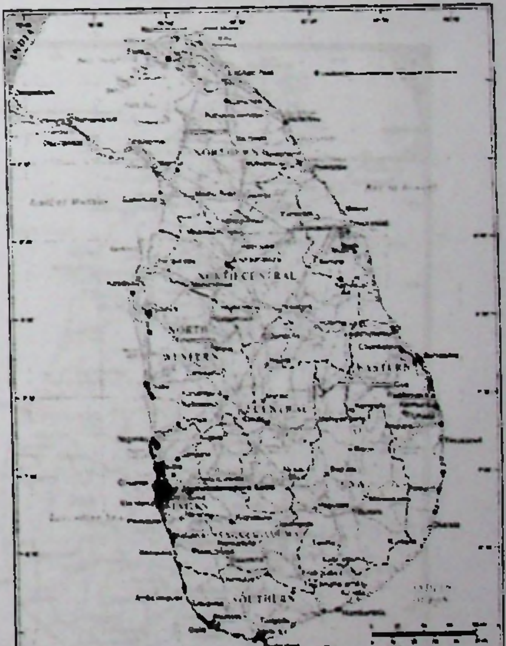

Figure 7: Liquefiable Layer Thickness 2 to 4 m

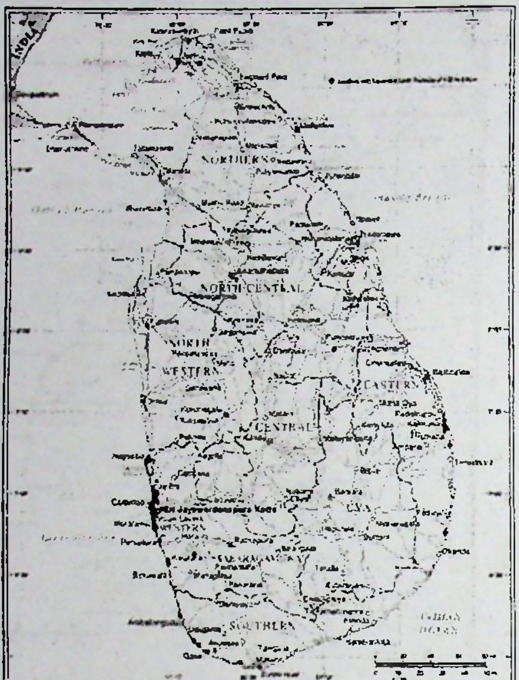

Figure 8: Liquefiable Layer Thickness 4 to 6 m

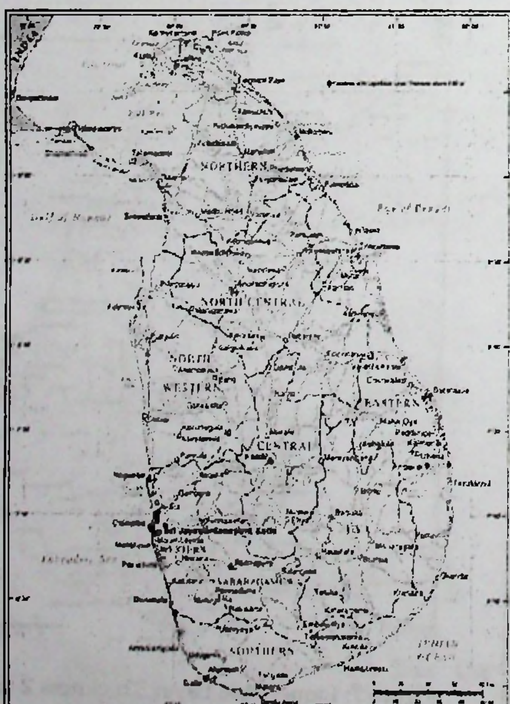

Figure 9. Liquefaction Layer Thickness > 6m

When the analysis was extended to various ground water levels, particularly at zero ground water level, the locations found as susceptible for liquefaction, dramatically increased. This could be happened during a flood.

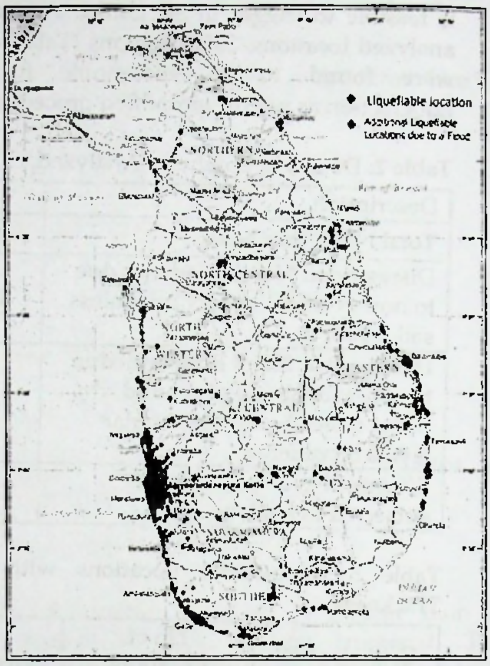

Figure 10: Liquefiable Locations during a Flood

Thickness of the liquefiable soil layers are varying from 0.50 m to several meters. However historical records shown that the damage due to liquefaction at places with thin sand layers as insignificant.

The above maps could be used to identify the areas susceptible to liquefaction. By improving the sub surface granular soil layers before construction and by selecting the appropriate foundation design for the structures within the liquefiable zones, possible damages due to liquefaction could be eliminated. Further, the liquefaction potential map could be referred to plan and implement future development projects away from the potential hazard zones. In addition, the map could be helpful to find secure locations against liquefaction and to decentralize people from the identified hazard zones.

5.0 Conclusion

Liquefaction potential was assessed and mapped for Sri Lanka by employing an empirical method proposed by Seed & Idriss (1971). The maps could be used as a guideline when designing and constructing structures, to minimize the damage due to liquefaction. It was revealed from the results that the Colombo city and coastal areas in Western, Sothern and Eastern provinces are highly susceptible to liquefaction.

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