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A Theoretical Study of Liquefaction Process

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Abstract—*Liquefaction is the phenomenon 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 earthquake loading. In liquefaction the stiffness and strength of the soils is reduced due the earthquake ground motion and any other dynamic vibration. Liquefaction exists in saturated soils, in which the space between individual's particles is completely filled with water. This water exerts a pressure on the soils particles that influences how closely the particles themselves are pressed together.*

This paper generally presents a common description of liquefaction process along with their types and assessment. It is acknowledged that several analytical and experimental research studies have been done to study liquefaction. The following paper is an attempt to provide a brief presentation of these attempts and to formulate new research ideas and approach to extend and complement the under studies of this phenomenon.

Keyword*:* *Liquefaction, pore pressure, effective stress, shear strength*

Introduction:-

Liquefaction is the rapid loss of shear strength of cohesion less soils subjected to dynamic loading, such as from an earthquake and blasting. In some cases the shear strength tends to zero, but in some cases it falls to a lower-than normal value. In either case, liquefaction can lead to many kind of failure, so its evaluation is one of the most important aspects of geotechnical engineering. The reason to loss of strength means loss of shear strength of soil. The shear strength of soil is mainly due to cohesion and frictional resistance. The intermolecular attraction and the frictional resistance contribute to shear strength of soil. The shear strength is expressed as

$$\tau = c + \sigma \tan \varphi$$

During an earthquake due to ground motion, there is instantaneous enhance in pore water pressure and so reduction in shear strength. In other hands during an earthquake, the propagation of shear waves causes the loose soil to contract, resulting in an increase in pore water pressure. The loss of strength is more pronounced in sandy soil due to increase in pore pressure. This phenomenon of loss of strength owing to rise in pore pressure is called liquefaction. As the term suggests the sand no longer behaves like a solid, rather it acts like a viscous fluid. Loose saturated sand deposits may lose a part of their shear strength when subjected to sudden earthquake excitation. It is evident that owing to increase in pore water pressure, the effective stress reduces, resulting in loss of strength. After sudden rise in pore pressure and therefore, stress transfer take place and the resulting effective stress controls the shear strength. If this stress transfer is complete, there is total liquefaction. However, when only partial stress transfer takes place,

$$\lim (\sigma_n - u) \rightarrow 0$$

There is a partial loss of strength resulting in partial liquefaction. Apart from earthquake, strength may be reduced due to sudden shock or dynamic load due to pile driving, explosive blasting, bomb blast loading, and vibration in machinery or even rapid draw-down in dams. Thus, owing to total loss of strength, the soil is said to have liquefied. Indeed, liquefaction is an external manifestation of decrease in shear strength, due to the cyclic pore-pressure generation mechanism. In other words if a loose saturated sand is rapidly deformed due to an earthquake, the grains tend to become closely spaced and compact. For, this to happen, water must flow out of the voids. If the loading is so rapid that there is no time for drainage of water, the entire load on the soil must be carried by pore water. Pore water is more compressible than the soil skeleton. Thus, the intergranular stress is largely reduced due to increase in pore water pressure. The increase in pore water pressure causes a reduction in shear strength. This loss of strength due to transfer of intergranular stress from soil grain to pore water due to dynamic load is known as liquefaction. When loss of strength occurs, the soil behaves like a viscous fluid. As the bearing capacity of soil to sustain foundation loads is directly related to strength, liquefaction poses a serious hazard to structures and must be assessed in areas where liquefaction prone deposits exist.

There are two types of liquefaction:

(1) Flow Liquefaction: - It occurs when the static shear stresses in the soil exceed the shear strength of the liquefied soil. This usually leads to large and sudden shear movements in the soil. However, flow liquefaction can occur only in loose soil.

(2) Cyclic Mobility: - It generally occurs when the static shear stresses are somewhat less than the liquefied shear strength, but the static plus dynamic stresses are higher than the liquefied shear strength. It creates incremental shear movement that are usually not as dramatic, as flow liquefaction, but still can be a source of significant damage.

Ground level liquefaction: - It can occur when cyclic loading is sufficient to produce high excess pore pressure even when static driving stresses are absent. Its occurrence is usually demonstrated by ground oscillation, post-earthquake settlement or development of sand boils. Level liquefaction can occur in loose as well as in dense soils. This level ground liquefaction is a special case of cyclic mobility. Resulting level-ground liquefaction damages are caused by the upward flow of water that occurs when seismically induced excess pore pressure dissipates.

Liquefaction is often accompanied by sand boils, which are made of liquefied sand ejected from the ground. If the sand boils are observed, we are sure liquefaction has occurred.

As a competent consulting geotechnical engineers you need to be familiar with the following aspects of liquefaction:

(1) What are the different kinds of liquefaction induced damage?

(2) What types of soils are susceptible to liquefaction?

(3) What is the theory of liquefaction?

(4) How to assess the site-specification susceptible?

(5) How to mitigate the liquefaction hazard?

Theory of Liquefaction:-

Liquefaction is a process related with weak deposits saturated, fine, uniform sands. If such a deposit is subjected to a sudden disturbance or shock, e.g., due to the vibrations of heavy machinery (pile driver), blasting or earthquakes, etc., the sand initiate to decrease rapidly in volume under the suddenly developed shearing stresses. This sudden tendency to volume decrease causes a temporary increase in pore pressure which may be sufficient to reduce the effective stresses to such a small value (or to zero) that a quick condition develops. The soil gets temporarily transformed into a fluid mass with negligible shear strength. Liquefaction is an example of quick sand which occurs even when seepage chiefly of rounded grains, with porosity greater than 44%, and have the effective size smaller than 0.1 mm and the uniformity coefficient less than 5.

The shear strength of sand in saturated condition may be expressed as-

$$\tau = (\sigma_n - u)\tan\varphi$$

Where,

τ = shear strength

σ_n= normal stress on a soil element at depth z

u = pore pressure

$\emptyset$ = angle of internal friction

The vertical stress on a horizontal plane of elemental soil under consideration at a depth z is given by

$$\sigma_n = \gamma_{sat}\, z$$

Where, = unit weight of saturated soil.

Thus,

$$\sigma_{effective} = (\sigma_n - u) = \gamma_{sat} z - \gamma_w z = (\gamma_{sat} - \gamma_w)z$$

During the ground motion due to earthquake, the static pore pressure may increase by an amount ∇_n, then

$$\nabla_n = \gamma_w . hw'$$

$$\sigma_{effective} = (\gamma_b\, z - \nabla_n)$$

$$= (\gamma_b - \gamma_w hw')$$

$$\tau = (\gamma_b z - \gamma_w hw')\tan\varphi$$

Or in other form it can be written as

$$\tau_{dyn} = (\sigma_n - \nabla_{u\, dyn})\tan\emptyset dyn$$

For complete loss of strength,

$$\sigma_n - \nabla_{\text{n dyn}} = 0$$

$$\gamma_b z - \gamma_w hw' = 0$$

$$\frac{hw'}{z} = \frac{\gamma_b}{\gamma_w} = \frac{G-1}{1+e} = i_{cr}$$

Where,

G = specific gravity of soil solids

e = void ratio

i_{cr}= critical hydraulic gradient

hw' = rise in water head due to ∇_n increase in pore pressure.

Thus, the gradient at which the effective stress is zero is called the critical hydraulic gradient. From above equation it is clear that liquefaction of sand may develop at any zone of deposit at any depth. Liquefaction may also result in the absence of ground motion or such motions if the underlying zones of the deposit have liquefied. Once liquefaction develops at some depth, the excess pore water pressure will dissipate by flow of water in an upward direction. For this the hydraulic gradient may be large enough to induce a quick sand condition.

Assessment:-

Liquefaction research also has produced methods of assessing the susceptibility of soils to liquefaction. Most of these methods use the cyclic stress approach, which describes earthquake loading in terms of the cyclic stress ratio, τ_{cyc}/σ'_{z0}, where τ_{cyc} is the cyclic shear stress and σ'_{z0}, is the initial vertical effective stress. This method assesses the cyclic stress ratio anticipated at the site during a certain design earthquake and compares it to that required to produce liquefaction. Both of these values depend on many factors, and very detailed investigations and analysis can be employed to define them. However, for many projects, a simplified analysis (Seed et al., 1985) is sufficient.

The cyclic stress ratio induced in the soil by the design earthquake may be estimated by using the following simplified formula (Seed and Idris, 1971):

$$\frac{\tau_{cyc}}{\sigma'_{z0}} = 0.65\frac{a_{max}}{g}.\frac{\sigma_{z0}}{\sigma'_{z0}}r_d$$

Where,

τ_{cyc}/σ'_{z0}= cyclic stress ratio (CSR) produced by an earthquake

a_{max}/g = peak horizontal ground acceleration divided by acceleration of gravity

σ_{z0} = initial vertical total stress

σ'_{z0} = initial vertical effective stress

r_d = stress reduction factor (from figure 2)

The design value of a_{max}/g is determined from an assessment of the magnitude of the design earthquake and its distance from the project site, as well as site response effects. The cyclic stress ratio required to produce liquefaction depends on many factors, but the simplified analysis considers only the following:

(1) The standard penetration test (SPT) or cone penetration test (CPT) resistance, which reflects the relative density. Dense soils have much more resistance to liquefaction.

(2) The grain size distribution, expressed either as percentage of fines (i.e., percent passing a # 200 sieve) or as D_{50} (the mean grain size). Soils with less than 5 percent fines are most susceptible to liquefaction. If more than 5% fines are present, the liquefaction resistance becomes much greater.

(3) The earthquake magnitude, which reflects its duration. Long duration earthquakes are more likely to cause liquefaction.

This method was originally developed using SPT data from sites that had experienced significant earthquakes and whose liquefaction history was known. Figure 1 shows cyclic stress ratios for these sites, with closed symbols representing those that had liquefied, and open symbols representing those that had not. The curves on this plot are thus empirical divisions between liquefiable and non-liquefiable soils. When using this figure, be sure to adjust the field N-value to $(N_1)_{60}$ using below equation (Liao and Whitman, 1986):

$$(N_1)_{60} = N_{60}\sqrt{\frac{100kPa}{\sigma'_z}}$$

Where,

$(N_1)_{60}$ = SPT N- value corrected for field procedures and overburden stress

σ'_{z0} = initial vertical effective stress

N_{60} = SPT-N value corrected for field procedure

The variations in testing procedures may be at least partially compensated by converting the N- recorded in the field to N_{60} as follows (Skempton, 1986):

$$N_{60} = \frac{E_m C_B C_S C_R N}{0.60}$$

Where,

N_{60} = SPT-N value corrected for field procedure

E_m = hammer efficiency

C_B = borehole diameter correction

C_s = sampler correction

C_R = rod length correction

N = SPT N- value recorder in the field

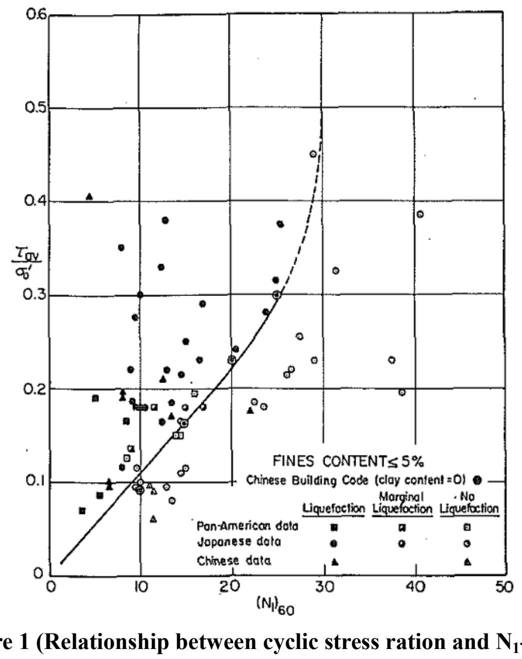

Figure 1 (Relationship between cyclic stress ration and N_1-Value)

The use of SPT correction factors is often a confusing issue. Corrections for field procedures (N_1) are always appropriate, but the overburden correction may or may not be appropriate depending on the procedures used by those who developed the analysis method under consideration.

Based on standard penetration test (corrected N values) and field performance data, Seed et al., (1985) concluded that there are three approximate potential damage ranges than can be identified and they are listed in Table (1).

Table 1 (Potential damages according to ranges of SPT $N_{corrected}$)

S. NO.	Ranges of SPT $N_{corrected}$	Potential Damage
1.	0-20	High
2.	20-30	Medium
3.	> 30	No significant

Factors governing liquefaction:-

There are many factors which are responsible for liquefaction-

(1) Earthquake intensity and duration:-

The main characteristics of ground motion are acceleration and duration of shaking. Earthquake of highest magnitude produces largest ground acceleration and largest duration of ground shaking.

(2) Ground water table:-

Condition is most conductive to liquefaction near the surface of ground water table. If the ground water fluctuates, liquefaction potential will also fluctuate. Generally the historic high ground water level should be used in liquefaction analysis.

(3) Soil type:-

Liquefaction occurs in deposits consisting of fine to medium sand and sand consisting low plasticity fines. Greater majority of cohesive soils do not liquefy.

(4) Relative density:-

Less relative density is more susceptible to liquefaction. Loose non-plastic soil contracts during seismic loading causing development of excessive pore pressure.

(5) Particle size gradation:-

Well graded soils are less susceptible to liquefaction than uniformity graded non-plastic soils. Uniformly graded non-plastic soils tend to form more unstable particle arrangement.

(6) Deposition Environment:-

Lacustrine, alluvial and marine deposits are most liquefiable deposit.

(7) Drainage condition:-

If excess pore water pressure can quickly dissipate then soil may not liquefy. A highly permeable gravel drain or gravel layer can reduce the liquefaction potential of adjacent soil.

(8) Confining pressure:-

Greater confining pressure shows less liquefaction susceptibility. Normally soil up to maximum depth of 15 m liquefies because confining pressure increases with depth.

(9) Particle shape:-

Soil contains round soil particles is more susceptible to liquefaction than soil containing angular soil particles.

(10) Building load:-

Construction of a heavy building on top of a sand deposit can decreases the liquefaction resistance of soil.

Effects of Liquefaction:-

Typical effects of liquefaction:

1. Loss of bearing strength
2. Lateral spreading
3. Sand boils
4. Flow Liquefaction
5. Ground oscillation
6. Floatation
7. Settlement

Methods to reduce liquefaction:-

1. Avoid liquefaction-susceptible soil from sites
2. Formation Liquefaction –resistant structure
3. Deep foundation aspects
4. Mat foundation is case of shallow foundation
5. Improve the soil quality (Soil improvement techniques)
6. A suitable drainage techniques

Conclusion:-

Liquefaction is a loss of bearing strength that occurs when saturated cohesion less sediment is subjected to strong shaking or cyclic loading. For liquefaction to occur soil must be cohesion less, loose, saturated and there must be dynamic force like earthquake. The liquefaction resistance increases with a decrease in the initial degree of saturation and increases in relative density and initial effective confining pressure. When the degree of saturation is near 100%, the effect of the degree of saturation on the liquefaction resistance is more significant.

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