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# Liquefaction Analysis of Kakinada Region by Using Geotechnical Borehole Data

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***Abstract:*** *During an earthquake the soil structure may fail owing to various reasons, namely: fissures, differential movements, faults and loss of strength of soil. The reason pertaining to loss of strength is the loss of shear strength of soil. When there is a loss of shear strength, the soil will behave like a viscous liquid and this phenomenon of loss of strength is termed as 'LIQUEFACTION'. If this stress transfer is complete, there is total liquefaction. However, when only partial stress transfer takes place, there is a partial loss of strength resulting in partial liquefaction. 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. For example the damage due to liquefaction for the ports and harbor structure was of appreciable magnitudes in the Andaman port due to 2004 Sumatra earthquake. Kakinada coast is erosional type, fault controlled and hence vulnerable to tremors of low to moderate intensity .Therefore it is essential to take up seismic hazard studies at micro level in order to improve safety norms for the port structures, industrial structures and underground pipe lines. In present study, the factor of safety against liquefaction is evaluated for Kakinada region which as an area of 190 sq.km. The factor of safety against liquefaction varies in the range of 0.3 to 10 for corresponding Peak Ground Accelerations from 0.1 to 0.3g. In order to determine the ground response using a one-dimensional approach, several input parameters including soil profile, bedrock level and other geotechnical properties of the subsurface and the design earthquake are required. Using the collected standard penetration test (SPT) data along with available geotechnical information and synthetically generated ground motion, equivalent linear analysis was performed using the computer program SHAKE -2000. For the selected bore holes, the soil profile fundamental period, peak ground acceleration, and ground response spectrum at the surface are reported. Liquefaction study was done using simplified Seed and Idriss approach by considering the amplified PGA at the ground surface.*

***Keywords:*** *Earth Quake, Liquefaction, Peak ground acceleration, ground response analysis,SHAKE-2000*

## I. Introduction

Many engineering systems and models have to be designed without complete information and thus the performance can seldom be perfect. Moreover, many decisions that are required during the process of planning and designing are made invariably under conditions of uncertainty. All these leads to accumulation of error in the result obtained. Therefore, there is invariably some chance for failure and its associated consequences. In the case of a structure, its safety is clearly a function of load imposed on it and the load carrying capacity of it. But it is hard to predict the load which it will be exposed. This adds to uncertainties in the equations used in the analysis. In case of hydrology and water resources, the available supply of water relative to maximum demand or usage of water is of concern in planning and designing of a system. The available supply from different source may be highly variable, whereas the actual usage may also fluctuate significantly, such that predictions in either case may be subject to significant uncertainties. Thus uncertainties are present in all engineering fields. Thus the probability that the designed parameter holds good has to be found out. This leads to reliability, which is the probabilistic measure of performance. In geotechnical engineering most of the parameters are related with natural variations. Thus the relevance of reliability in geotechnical engineering is very high. In liquefaction of soil almost all variables used in the analysis bring uncertainties along with them. Also liquefaction is related with earthquake, for which the behavior cannot be predicted. The equations which we use to carry out analysis is being modified each time, when the database available for each parameter changes. Thus the probability that the analysis which we carry out holds good should be quantified. Thus the work is a probabilistic analysis of results obtained during deterministic analysis of liquefaction of soil

## II. Liquefaction

During an earthquake the soil structure may fail owing to various reasons, namely: fissures, differential movements, faults and loss of strength of soil. The reason pertaining to loss of strength is the loss of shear strength of soil. When there is a loss of shear strength, the soil will behave like a liquid. This phenomenon of loss of strength is termed as 'liquefaction'

#### 1.2.1 Liquefaction Potential

Liquefaction potential is the evaluation of resistance of soil to liquefy. It can be evaluated by field test and lab test. Field test includes standard penetration test, shear wave velocity test and static cone penetration test.lab test includes shake table test and cyclic plate load test. Generally field tests are used to evaluate liquefaction potential

### 1.3 Objective

In the present study, liquefaction analysis is carried out by using standard penetration test (SPT). The deterministic analysis of liquefaction is done by SPT using spreadsheet. By considering all the uncertainties like model uncertainties and parametric uncertainties, reliability analysis is done in spreadsheet. The main aim is to give a generalized, flexible model in which the distributions of variables can be varied. In this study the correlation between different variables is also considered. The main objectives of the study are,

- To determine factor of safety from deterministic analysis
- To analyse reliability index using reliability analysis
- To predict probability of liquefaction from reliability index.

## III. Literature Review

### 2.1 General

The entire study is about deterministic and reliability analysis of liquefaction. It brings forth the previous studies conducted on deterministic and reliability analysis. The following is the literature review of deterministic and reliability analysis.

### 2.2 Liquefaction

Liquefaction is the phenomena where there is loss of strength in saturated 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.

#### 2.2.1 Types of liquefaction

As far as geotechnical earthquake engineering is concerned, the phenomenon of liquefaction may be divided into three main groups, namely:

- Flow liquefaction
- Cyclic mobility
- Ground level liquefaction

**Flow liquefaction**

It occurs when static shear stress in a soil deposit during earthquake excitation is greater than the steady state strength of the soil. It can produce devastating flow slide failures during or even after earthquake shaking. However flow liquefaction can occur only in loose soils.

**Cyclic mobility**

Cyclic mobility can occur when the static shear stress is less than the steady state strength and the cyclic shear stress is large enough, then the steady state is exceeded momentarily. Deformations produced by cyclic mobility develop incrementally but can become substantial by the end of a strong and or long-duration earthquake. Cyclic mobility can occur in both loose and dense soils but deformations decrease markedly with increased density.

**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 generally manifested by ground oscillation, post-earthquake settlement or development of sand boils. Level ground liquefaction can occur in loose and dense soils. This level ground liquefaction is a special case of cyclic mobility. Level ground liquefaction failures are caused by the upward flow of water when seismically induced excess pore pressure dissipates.

#### 2.2.2 Mechanism of Liquefaction

The shear strength of soil is mainly due to cohesion and frictional resistance. The inter-molecular attraction and the frictional resistance contribute to shear strength of soil. The shear strength of soil can be expressed as,

$$\boldsymbol{\tau = c + \sigma_n \tan \varphi} \qquad \text{……..(2.1)}$$

For cohesionless soils i.e., sands c = 0. Thus the above equation can be written as,

$$\tau = \sigma_n \tan \varphi \qquad \text{……..(2.2)}$$

Soil is a polyphase material consisting of water, air in pores and the solid soil skeleton. The pore water pressure 'u', does make reduction in effective normal stress, so that,

$$\tau = (\sigma_n - u) \tan \varphi \qquad \text{……..(2.3)}$$

During an earthquake owing to ground motion, there is an instantaneous increase in pore water pressure and a reduction in shear strength. In other words, during an earthquake, the propagation of shear waves causes the loose soil to contract, resulting in increase of pore water pressure. The increase in pore water pressure causes a reduction in shear strength. This loss of strength due to transfer of inter-granular 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. If this stress transfer is complete, there is total liquefaction. However, when only partial stress transfer takes place, there is a partial loss of strength resulting in partial liquefaction. 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. The shear strength of sand in saturated condition may be expressed as

$$\tau = (\sigma_n - u) \tan\varphi \qquad \text{……..(2.4)}$$

Where,

$\tau$ = shear strength

$\sigma_n$= normal stress on soil element at depth z

$u$ = pore pressure

$\varphi$ = angle of internal friction

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

$$\sigma_n = \gamma_{sat} Z \qquad \text{……..(2.5)}$$

where,

$\gamma_{sat}$ = unit weight of saturated soil.

Thus,

$$\sigma_{eff} = (\sigma_n - u) = \gamma_{sat} Z - \gamma_w Z$$
$$= (\gamma_{sat} - \gamma_w) Z \qquad \text{……..(2.6)}$$

During the ground motion due to earthquake, the static pore pressure may increase by an amount + $\Delta_n$, then

$$\Delta_n = \gamma_w . h_w'$$
$$\sigma_{eff} = \gamma_{sat} Z - \Delta_n$$
$$= \gamma_{sat} Z - \gamma_w h_w' \qquad \text{……..(2.7)}$$
$$\tau = (\gamma_{sat} Z - \gamma_w h_w') \tan\varphi \qquad \text{……..(2.8)}$$

Or it can be written as

$$\tau_{dyn} = (\sigma_n - \Delta_{u\,dyn}) \tan \varphi_{dyn} \qquad \text{………..(2.9)}$$

Forcomplete loss of strength,

$$\sigma_n - \Delta_{u\,dyn} = 0$$
$$\gamma_b z - \gamma_w h_w = 0$$
$$\frac{hw'}{z} = \frac{\gamma_b}{\gamma w} = \frac{G-1}{1+e} = i_{cr} \qquad \text{…...(2.10)}$$

Where,

**G** = specific gravity of soil solids

**e** = void ratio

$i_{cr}$ = critical hydraulic gradient

$h_w'$ = rise in water head due to $\Delta_n$ increase in pore pressure.

Thus, the gradient at which the effective stress is zero is called the critical hydraulic gradient.

From the above equation it is evident 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 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. When a natural surface or soil deposit is in quick sand condition, it cannot support the weight of a person or an animal.

**2.3 Liquefaction Analysis**

The basic objective of the liquefaction analysis is to ascertain if the soil has the ability to liquefy during an earthquake. It can be determined by using factor of safety (F.S), which is the ratio of cyclic resistance ratio (CRR) to the cyclic stress ratio (CSR).Thus, the factor of safety (**FS**) against liquefaction may be defined as,

**FS = CRR/CSR**

Let usconsider a soil column of unit width and length as shown in Fig., 2.1.

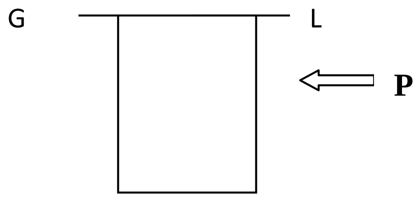


Fig., 2.1 Seismic force acting on a soil column

It is assumed that the soil column will move horizontally as a rigid body in response to the maximum horizontal acceleration $\mathbf{a_{max}}$ exerted by the earthquake at ground surface. If 'P' denotes the horizontal seismic force acting on a soil column of unit width and unit length, then

$$\mathbf{P = mass * acceleration} \qquad \ldots\ldots(2.11)$$

where

$$\mathbf{mass = W/g = (\gamma_{sat} Z /g)}$$

Substituting the value of mass in Eq (2.11),

$$\mathbf{P = (\gamma_{sat}\ Z /g).\ a_{max} = \sigma_{vo}\ .\ (a_{max} / g)} \qquad \ldots\ldots(2.12)$$

where,

**W** = total weight of soil column
$\mathbf{\Upsilon_{sat}}$= total unit weight of soil (kN/m$^3$)
**Z** = depth below ground level as shown in fig
**a**= acceleration which is equal to maximum horizontal at ground surface, $\mathbf{a_{max}}$
$\mathbf{a_{max}}$= maximum horizontal acceleration at ground surface due to earthquake
$\mathbf{\sigma_{v0}}$ = total vertical stress at bottom of soil column (kPa)

Considering the force equilibrium of the soil column the force '**P**', acting on the rigid soil column is equal to the maximum shear force at the base of the soil column.

Since the soil column is assumed to have a unit base width and length, the maximum shear force '**P**' is equal to the maximum shear stress $\mathbf{\tau_{max}}$. The equation of the force equilibrium may be written as

$$\mathbf{\tau_{max} = P = \sigma_{v0}\ (a_{max} / g)} \qquad \ldots\ldots(2.13)$$

Let $\mathbf{\sigma_{v0}'}$ be the vertical effective stress, then dividing both sides of Eq. (2.13) by $\mathbf{\sigma_{v0}'}$,

$$\mathbf{(\tau_{max} / \sigma_{v0}') = (\sigma_{v0} / \sigma_{v0}').(\ a_{max} / g)} \qquad \ldots\ldots(2.14)$$

Since the soil column does not act as a rigid body in reality during the earthquake, Seed and Idriss (1971) proposed a depth reduction factor'$\mathbf{r_d}$' into the right- side as the soil is deformable,

So,

$$\mathbf{(\tau_{max} / \sigma_{v0}') = r_d.\ (\sigma_{v0} / \sigma_{v0}').(\ a_{max} / g)} \qquad \ldots\ldots(2.15)$$

Further Seed et al., (1975) proposed a simplified method by converting the typical irregular earthquake record as shown in Fig.,2.2 to an equivalent series of uniform cycles by assuming the following.

$$\mathbf{\tau_{av} = \tau_{cyc} = 0.65\ \tau_{max}}$$

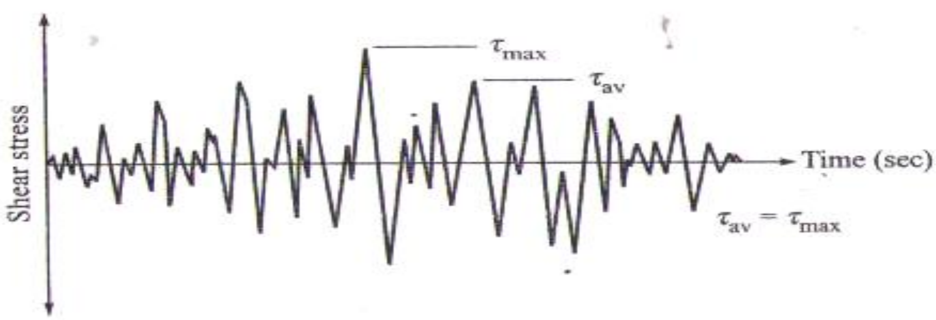


Fig., 2.2 Time history of shear stress during earthquake for liquefaction analysis

**2.3.1 Cyclic Stress Ratio**

For the liquefactionanalysis, a dimensionless parameter **CSR** (Cyclic Stress Ratio) is defined. This ratio is defined as

$$\mathbf{CSR = \tau_{cyc}} / \sigma_{v0}' \qquad \ldots\ldots(2.16)$$

Thus,

$$\mathbf{CSR = 0.65\ r_d.\ (\sigma_{v0} / \sigma_{v0}').(\ a_{max} / g)} \qquad \ldots\ldots(2.17)$$

Where,

$\mathbf{a_{max}}$ = maximum horizontal acceleration at ground surface (m/sec$^2$)
**g** = acceleration due to gravity
$\boldsymbol{\sigma_v}$ = total vertical stress at a particular depth where liquefaction analysis is done
$\boldsymbol{\sigma_v'}$ = vertical effective stress at the same depth
$\mathbf{r_d}$ = stress reduction factor that accounts for the flexibility of soil column

**2.3.2 Cyclic Resistance Ratio (CRR)**

The cyclic resistance ratio represents the liquefaction resistance of the soil. The most commonly used method for determining the liquefaction resistance is to use the data obtained from the SPT test. In general, the factors that increase the liquefaction resistance of a soil will also increase the corrected **N** values from the SPT test.

**2.3.3 Factor of Safety against Liquefaction**

The liquefaction analysis culminates in determining the factor of safety against liquefaction. If cyclic stress ratio (**CSR**) caused by the anticipated earthquake is greater than the cyclic resistance ratio (**CRR**) of the insitu soil, then liquefaction could occur during the earthquake. Thus, the factor of safety (**FS**) against liquefaction may be defined as

$$\mathbf{FS = CRR/CSR}$$

Where,**CRR** represents the resistance of soil to liquefaction and **CSR** represents the stress caused by the earthquake load.

The higher the factor of safety, the more resistant is the soil to liquefaction. In general if the **FS ≤ 1**, then the soil is considered to liquefy otherwise the soil is safe against liquefaction.

**2.4factors Affecting Liquefaction**

The following are the factors which affects liquefaction

- **Particle Size Distribution of Soil**

Fine and uniform sands are more prone to liquefaction. As the pore pressure is dissipated more quickly in coarse-sand, hence the chances of liquefaction are reduced in coarse-sand deposits. Further, uniformly graded non-plastic soil is more susceptible to liquefaction than the well-graded soil. Well-graded soils will have small particles that fill in the void space. This tends to reduce the potential contraction of the soil, resulting in less excess pore pressure generation during an earthquake.

- **Relative Density**

The relative density is a measure of denseness of sand deposit and it is one of the most important factors controlling liquefaction. The chance of liquefaction is much reduced if the relative density is high. Based on field studies, cohesionless soils in loose relative density state are susceptible to liquefaction.

- **Drainage Facility**

If the excess water pressure can quickly dissipate, the soil may not liquefy. Thus highly permeable gravel drains or gravel layers can reduce the liquefaction potential of adjacent soil.

- **Confining Pressures**

The greater the confining pressure, the less susceptible the soil is to liquefaction. The condition that can create a higher confining pressure on a deeper groundwater table is the soil that is located at a depth below ground surface and a surface charge pressure applied at the ground surface. Day states that case studies have shown that the possible zone of liquefaction usually extends from ground surface to a maximum depth of about 15 m. Deeper soils generally do not liquefy because of the higher confining pressure.

- **Groundwater Table**

The condition most conducive to liquefaction is a near surface groundwater table. Unsaturated soil above the groundwater table will not liquefy. At sites where the groundwater table fluctuates, liquefaction potential also fluctuates.

- **Dynamic Characteristics of Soil**

Seed(1976) states that multidirectional shaking as in earthquake excitation is more severe than that in one-directional vibrations. Under multi-directional shaking, pore pressure builds up faster and consequently the loss of the strength of the soil is much faster and the stress ratio required is about ten percent less than that required for uni-directional shaking.

- **Strain History**

The studies on liquefaction characteristics of fresh deposits or of recent origin and their comparison with similar soil deposits previously subjected to some strain history reveals that there is no significant change in relative density owing to the previous strain history. Seed (1995) showed that although the prior strain history caused no significant change in the density of sand, but it increased the stress that causes liquefaction by a factor of about 1.5.The older soil deposits that have already been subjected to seismic shaking in the past have increased liquefaction resistance compared to a newly formed specimen of the soil having an identical density.

- **Influence of Superimposed Load and Overburden Pressure**

In soil deposits at any depth the effective stress depends upon the magnitude of superimposed load and the overburden pressure. In the field the initial stress conditions are not isotropic, i.e., the lateral stress is not equal to the normal stress, rather the lateral stress depends upon the coefficient of earth pressure at rest $\mathbf{K_o}$ which is usually defined as

$$\mathbf{K_o = (\mu / 1\text{-}\mu)}$$

Where'**μ**' is the Poison's ratio.
The stress conditions causing liquefaction depend upon the value of $\mathbf{K_o}$.

- **Entrapped Air**

The soil is a polyphase material consisting of solid soil grains, water and air entrapped in voids and the pores. If the air is entrapped, during the rise of pore pressure due to earthquake excitation, part of it is consumed and is dissipated due to compression of air. Hence, the trapped air helps in reducing the chances of liquefaction.

- **Particle Shape**

The soil particle shape can also influence liquefaction potential. For example, soils having rounded particles tend to densify more easily than granular shape soil particles.

- **Building Load**

The construction of a heavy building on top of a sand deposit can decrease the liquefaction resistance of the soil. The reason is that a smaller additional shear stress will be required from the earthquake in order to cause contraction and hence liquefaction of soil.

## IV. Data Collections

The borehole and Standard Penetration Test (SPT) needed for the liquefaction analysis has been obtained from a site in Kakinada was presented in Table 3.1.

Table 3.1Boreholeand SPT data

| DEPTH (m) | UNIT WT (kn/m$^3$) | OBSERVED N value | FINES (%) | REMARKS |
|---|---|---|---|---|
| 0 | 0 | 0 | 0 | Water level was assumed to be at ground level |
| 1.5 | 16.87 | 9 | 7 | |
| 3 | 17.75 | 6 | 8 | |
| 5 | 17.75 | 12 | 10 | |
| 6 | 18.35 | 14 | 4 | |
| 8 | 18.35 | 21 | 1 | |
| 10 | 19.13 | 24 | 3 | |
| 12 | 19.62 | 20 | 2 | |
| 15 | 19.72 | 27 | 5 | |
| 18 | 19.72 | 28 | 1 | |
| 20 | 19.82 | 31 | 8 | |
| 22 | 19.92 | 31 | 5 | |

### 3.1 Deterministic Analysis Of Liquefaction Of Soil

The conventional liquefaction analysis follows the determination of factor of safety (F.S.). If factor of safety is greater than one, then the soil is said to be non-liquefiable.

$$\mathbf{F.S.} = \frac{CRR}{CSR} \qquad \text{......(3.1)}$$

If F.S. > 1, soil will not liquefy
If F.S. ≤ 1, soil will liquefy

### 3.1.1 Cyclic Resistance Ratio

As described in literature review different authors has proposed different equations for CRR. Among them Cetin et. al (2000) has considered many case studies and the equation suggested is more reliable.

$$\mathbf{CRR = exp\left\{\frac{(N_1)_{60cs}}{14.1} + \left[\frac{(N_1)_{60cs}}{126}\right]^2 - \left[\frac{(N_1)_{60cs}}{23.6}\right]^3 + \left[\frac{(N_1)_{60cs}}{25.4}\right]^4 - 2.8\right\}} \quad \text{.......(3.2)}$$

Where, $\mathbf{(N_1)_{60cs}}$ isthe SPT number corrected for overburden, fines and other corrections as shown in Table 3.1. The SPT number obtained from the field is corrected for overburden and fines content. The following are the most reliable equations used for the corrections.

**Cetin et al. (2000)** have suggested the following equations to adjust the SPT penetration resistance to clean sand value as:

$$\mathbf{(N_1)_{60cs} = (N_1)_{60} + \Delta(N_1)_{60}} \quad \text{........(3.3)}$$

$$\mathbf{\Delta(N_1)_{60} = exp\left[1.63 + \frac{9.7}{FC} - \left(\frac{15.7}{FC}\right)^2\right]} \quad \text{........(3.4)}$$

Where FC is fines content, $\mathbf{(N_1)_{60}}$ is SPT number corrected for overburden pressure

**Robertson and Wride,(1998)** suggested the following correction for SPT number as shown in table 3.1 and suggested the following equation.

$$\mathbf{(N_1)_{60} = N_m C_N C_E C_B C_R C_S} \quad \text{.......(3.5)}$$

Where$\mathbf{N_m}$is the SPT value observed in the field.

**Boulanger and Idriss (2004)** suggested the following expressions for overburden correction

$$\mathbf{C_N = \left(\frac{P_a}{\sigma'_{vo}}\right)^{\alpha} \leq 1.7} \quad \text{........(3.6)}$$

$$\boldsymbol{\alpha = 0.784 - 0.0768\sqrt{(N_1)_{60}}} \quad \text{........(3.7)}$$

$\mathbf{(N_1)_{60}}$ is limited to a maximum of 46.

Table 3.2 Correction factors given by Robertson and Wride (1998)

| Factor | Equipment variable | Term | Correction |
|---|---|---|---|
| Energy ratio | Donut Hammer<br>Safety Hammer<br>Automatic-Trip Donut Hammer | $C_E$ | 0.5 to 1.0<br>0.7 to 1.2<br>0.8 to 1.3 |
| Borehole diameter | 65 mm to 115 mm<br>150 mm<br>200 mm | $C_B$ | 1.0<br>1.05<br>1.15 |
| Rod length | 3 m to 4 m<br>4 m to 6 m<br>6 m to 10 m<br>10 m to 30 m<br>>30 m | $C_R$ | 0.75<br>0.85<br>0.95<br>1.0<br><1.0 |
| Sampling Method | Standard sampler<br>Sampler without liners | $C_S$ | 1.0<br>1.1 to 1.3 |

### 3.1.2 Cyclic Stress Ratio

**Seed and Idriss (1975)** suggested the following equation for CSR

$$\mathbf{CSR = 0.65\, r_d . (\sigma_{v0} / \sigma_{v0}').( a_{max} / g)} \quad \text{........(3.8)}$$

Where $\mathbf{r_d}$is depth reduction factor.

**Idriss (1999)**, suggested following equation for $\mathbf{r_d}$

$$\mathbf{Ln(r_d) = \alpha(z) + \beta(z)M} \quad \text{........(3.9)}$$

Where **M** is the moment magnitude

$$\mathbf{\alpha(z) = -1.012 - 1.126 sin\left(\frac{z}{11.73} + 5.133\right)} \quad \text{......(3.9a)}$$

$$\mathbf{\beta(z) = 0.106 + 0.118 sin\left(\frac{z}{11.28} + 5.142\right)} \quad \text{.......(3.9b)}$$

These equations are considered appropriate to a depth **Z** ≤ 34 m. For **Z**> 34 m, the following equation is used.

$$\mathbf{r_d = 0.12*exp(0.22M)} \quad \text{.......(3.10)}$$

### 3.1.3 Magnitude Scaling Factor

All the equations proposed are for earthquake magnitude of 7.5. To suit these equations for other magnitudes MSF was proposed.

**Idriss (1999)** proposed following equation for MSF as

$$\mathbf{MSF = 6.9exp(-M/4)-0.058} \quad \text{.....(3.11)}$$

**MSF ≤ 1.8**

## V. Deterministic Analysis

The data in Table 3.1 is used for the deterministic analysis. All the equations and correction factors required for the deterministic analysis are formulated in the excel sheet. The main of the deterministic analysis is to calculate the factor of safety. Liquefaction potential assessment can be determined based on the factor of safety. In general if the factor of safety is less than or equal to one, then the soil will liquefy. As the parametric uncertainties are not considered in the deterministic analysis, certain amount of risk is involved in the result. Reliability analysis predicts the risk involved in the result.

Table 4.1 Abstract of boreholes

| S.No | Bore log location | Factor of safety against liquefaction | Possibility of Liquefaction(Yes or No) |
|---|---|---|---|
| 1 | water table is at 1.1m depth, near LB Nagar,kakinada | >1 | No |
| 2 | water table is at 1.5 m depth,l.b.nagar,road no:8,kakinada | >1 | No |
| 3 | water table is at 1.2m depth,bhanugudi ,Kakinada | >1 | No |
| 4 | water table is at 1.2 m depth,chairminar café,kakinada | >1 | No |
| 5 | water table is at 1.6 m depth, railway kalyana mandapam, kkd | >1 | No |
| 6 | water table is at 1.2 m depth,vidyut nagar,kakinada | >1 | No |
| 7 | water table is at 1.5 m depth,vidyut nagar,kakinada | >1 | No |
| 8 | water table is at 0 m depth, vidyut nagar,kakinada | >1 | No |
| 9 | water table is at 1.5 m depth, vidyut nagar,kakinada | >1 | No |
| 10 | water table is at 0 m depth, Kondyapallem,kakinada | >1 | No |
| 11 | water table is at 0 m depth, Kondyapallem,kakinada | >1 | No |
| 12 | water table is at 1.5 m depth, Kondyapallem,kakinada | >1 | No |
| 13 | water table is at 1.5 m depth, vidyut nagar,kakinada | >1 | No |
| 14 | water table is at 1.2 m depth, vidyut nagar,kakinada | >1 | No |
| 15 | water table is at 1.2m depth, ramakrishna rao peta,kakinada | >1 | No |
| 16 | water table is at 1.2m depth, ramakrishna rao peta,kakinada | >1 | No |
| 17 | water table is at 1.2m depth, Rameswaram peta,kakinada | >1 | No |
| 18 | water table is at 1.2m depth, Rameswaram,kakinada | >1 | No |
| 19 | water table is at 1.2m depth, RTC Complex Road,kakinada | >1 | No |
| 20 | water table is at 1.2m depth, S Atchutapuram,kakinada | >1 | No |
| 21 | water table is at 1.5m depth, NFCL Road,kakinada | >1 | No |
| 22 | water table is at 1.5m depth, NFCL Road,kakinada | >1 | No |
| 23 | water table is at 1.5m depth, NFCL Road,kakinada | >1 | No |
| 24 | water table is at 1.5m depth, NFCL Road,kakinada | >1 | No |
| 25 | water table is at 1.5m depth, NFCL Road,Kakinada | >1 | No |

Finally Liquefaction analysis is carried out for Kakinada region and it gives that no Liquefaction is encountered in my study area.

**Methods of Reducing Liquefaction Hazards**

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.

1. **By Avoiding the zone of Liquefaction Susceptible Soils**
2. **Ground Improvement techniques**

## VI. Conclusions

Following are the conclusions based on deterministic and reliability analysis of liquefaction based on the Standard Penetration Test.

- After examining the factor of safety's calculated no area is proned for liquefaction, and also there are no chances to get liquefaction for the earth quake magnitudes up to 6.0. If earthquake magnitude for more than 6.0, one has to think about the occurring of liquefaction
- Recent studies conducted are based on enlarged data set. Hence equations and correction factors developed recently will have higher degree of accuracy. Hence the results obtained considering the recent equations and correction factors will have better accuracy.
- From the liquefaction analysis done in this work shows by and large there is no possibility of getting liquefaction in most of the areas.
- Parametric uncertainties are not considered in the deterministic analysis.
- From the deterministic analysis, if the factor of safety is greater than one, soil should not liquefy. But from reliability analysis, it is found that there are 10-40% chances for the liquefaction to occur because of the parametric uncertainties.
- Uncertainties in the parameters will have considerable effect on the final result (F.S)
- When the uncertainties in the parameter increase, uncertainty in the final result increases.
- Design engineers should be cautions while designing the structures with all precautionary methods.

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