Liquefaction Potential Assessment for Near Field and Far Field Ground Motion

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Abstract. Out of the various seismic hazards, soil liquefaction is a major cause of both losses of life and damage to infrastructures and lifeline systems. For establishment of various important structures, suitable identification, evaluation and analysis of the liquefaction should be done in soil sites. Present scope of work is evaluation of liquefaction based on SPT data of a site. Liquefaction potential for a soil site was evaluated using empirical as well as analytical method for available data of 38 bore holes. Under analytical method, non-linear and equivalent linear ground response analysis of site was carried out using DEEPSOIL for near field and far field ground motion for same PGA value. The results were compared and it was found that FOS against liquefaction potential by equivalent linear ground motion is less compare to non-linear ground motion, and FOS for near field earthquake ground motion is less compare to far field earthquake ground motion for same PGA value. Bore holes which is susceptible to liquefaction are located nearby to each other It justifies the applicability of different methods and importance of confirmation of results**.**

Keywords: Liquefaction, SPT, PGA, Near field and far field ground motion Introduction

1 Introduction

Liquefaction is defined as the transformation of a granular material from a solid to a liquefied state as a consequence of increase of pore water pressure and reduced effective stress. The change of state occurs most readily in loose to moderately dense uniformly graded saturated granular soil with poor drainage, such as silty sands or sands and gravel. Shear strength of such type of soil is only due to particle to particle inter granular friction.

When cyclic loading is subjected to soil excess positive pore water pressure is generate consequence of this water trapped between the soil particle try to separate soil particle i.e. strength which is gained by inter granular contact between particle is reduced to a significant value. Due to drastically reduction of strength, soil loses its shape and behaves like liquid. This phenomenon of soil is called liquefaction.

During liquefaction, structures tend to settle or sink into the ground. In many cases, some parts of the building may sink more than the others, leading to tilting of the building. In sloppy ground if lower level soil loses its strength, it causes landslide which is extending over hundreds of meters. Therefore, structures may sink and destabilize, if supported by such soil. Although engineering measures are available to avoid liquefaction. But if we apply this treatment methodology for whole site, it is too uneconomical. It is better if we first evaluate the critical area where chances of liquefaction would be more, then for that area, the Engineering measure needs to be applied for safety against liquefaction. For construction of nuclear structures, liquefaction potential of site needs to be evaluated [1].

In present study liquefaction potential is evaluated using empirical approach [2] and tools of ground response analysis [3]. Near field and far field earthquake affects the liquefaction potential significantly. Assumption of soil behavior like linear, equivalent linear, non-linear also affects the liquefaction potential result. Comparison between various methods of evaluation needs to be done. There is need for evaluation of liquefaction potential for a particular site taking into account for far field and near field ground motion. Hence, in the present study, coastal site is selected and soil geotechnical test results data is collected. Liquefaction is calculated by both empirical and analytical method for near field and far field ground motion considering two cases namely equivalent linear and non-linear behavior in ground response analysis.

1.1 Determination of liquefaction potential

Since liquefaction occurs due to cyclic loading therefore for assessment of liquefaction first to find out stress generated due to cyclic loading numerically cyclic stress ratio denoted cyclic loading which is the ratio of average shear stress subjected due to earthquake to effective stress, than resistance of soil against this cyclic stress, numerically cyclic resistance ratio is denoted cyclic resistance, it is inherent property of soil. Whenever cyclic stress ratio (CSR) is greater than cyclic resistance ratio (CRR) liquefaction will occur. Numeric value of CRR and CSR is estimated by using basic soil investigation data in empirical formula developed in [2].

$$\mathrm{CSR} = 0.65\left(\frac{a_{max}}{g}\right)\left(\frac{\sigma_v}{\sigma'_v}\right) r_d \tag{1}$$

Where
z = depth below the ground surface
a_{max} = peak ground acceleration (PGA)in terms of g,
g = acceleration due to gravity
r_d= stress reduction coefficient
$r_d = 1 - 0.00765z$ for $z \le 9.15$ m
$r_d = 1.174 - 0.0267z$ for $9.15\text{m} \le z \le 23$ m
σ_v and σ'_v are total and effective stress

For calculation of CRR first it is need to correct SPT blow count and apply fineness correction on it

$$(N_1)_{60} = N_m C_N C_E C_B C_R C_S \tag{2}$$

(N_1)60 is corrected blow count $C_{N,}$ $C_{E,}$ $C_{B,}$ C_R and C_S are the corrections for overburden, hammer energy, bore hole, rod length and sampler correction respectively.

$$(N_1)_{60cs} = \alpha + \beta(N_1)_{60} \tag{3}$$

Where
α = 0 ; β=1 for FC ≤ 5%
α=exp(1.76-(190/FC2)); β= 0.99 + FC1.5/1000; for 5% < FC < 35%
α =5; β=1.2; for FC ≥ 35%

$$CRR_{7.5} = \frac{1}{34-(N_1)_{60cs}} + \frac{(N_1)_{60cs}}{135} + \frac{50}{[10(N_1)_{60cs}+45]^2} - \frac{1}{200} \tag{4}$$

Where $CRR_{7.5}$ = CRR at 7.5 magnitude earthquake
(N_1)60cs corrected value of blow count after applying fineness correction and other correction.
This equation is valid for (N_1)60 <30 for (N_1)60 ≥30 soil is classified as non-liquefiable.
Since this equation is for 7.5 magnitude earthquake for other magnitude of earthquake magnitude correction factor is applied for this Calculated value of CRR at 7.5 magnitude earthquake is multiplied by a magnitude scaling factor (MSF). This MSF is calculated from following equation.

$$MSF = \frac{10^{2.24}}{M_W^{2.56}} \tag{5}$$

CSR is also evaluated by analytical method by ground response analysis. There are various software tools like LS-DYNA, SHAKE, DEEPSOIL are available for ground response analysis. In present study, DEEPSOIL software is used which can perform 1-D site specific ground response analysis by time domain non-linear analysis with assuming excess pore water pressure and frequency domain equivalent linear analysis, further that various curves are available for time domain non-linear analysis but for our case data required for [5] are available. So [5] is selected as a reference curve. Water table is assumed at ground level and bedrock is assumed as elastic half space with 5% of damping value.
It is defined in Uniform Building Code (UBC) that if the epicentral distance of site is less than 15 KM it is near field earthquake and if it is more than 15 KM than it is far field earthquake. Input ground motions have been taken from web literature [6]. From the input ground motion 3 near field “Alum rock California (Distance from epicentre 10.9 KM), Greese (Distance from epicentre 13.7 KM), ChiChi(Distance from epicentre 10.7 KM)” and 3 far field “Bhuj (Distance from epicentre 239 KM), Gulf of California (Distance from epicentre 95 KM), Sumatra (Distance from epicentre 392 KM)” base line correction on these earthquake motion is applied than all earthquake motion is scaled to a PGA value of 0.1g because site lies in seismic zone factor (ii), Further frequency independent damping matrix type is chosen in which 15 no’s of iteration

have been done. And effective shear strain ration is taken as 0.65. Final input motion time history which is used in this study is shown in fig 1 to fig 6

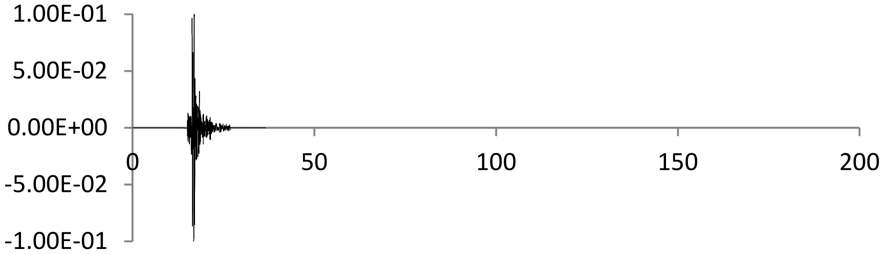

Fig. 1. Input bedrock motion (Alum Rock California)

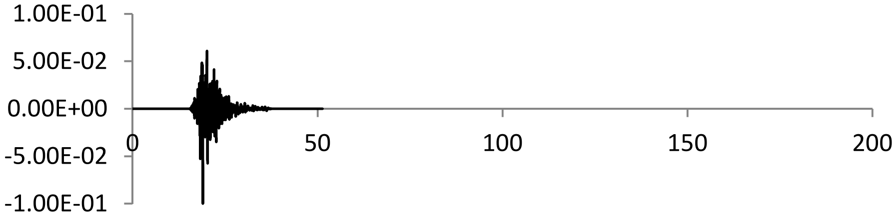

Fig. 2. Input bedrock motion (Greese)

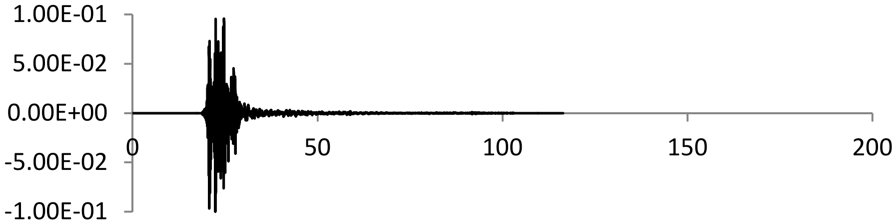

Fig. 3. Input bedrock motion (Chi Chi)

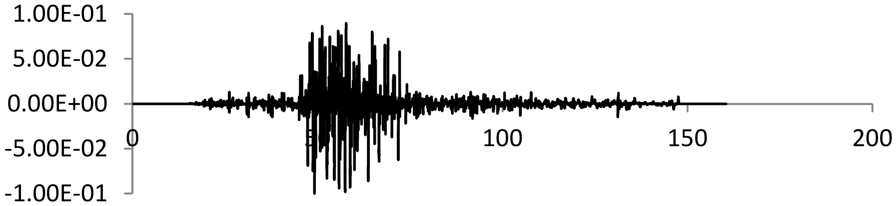

Fig. 4. Input bedrock motion (Bhuj)

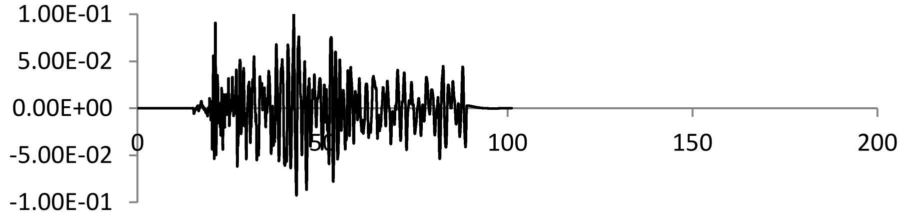

Fig. 5. Input bedrock motion (Gulf of California)

Input data of various soil parameters like total stress, effective stress SPT blow count is collected for 38 bore hole at site. Since shear wave velocity data is not measure at all 38 bore hole it is calculated from empirical relation between SPT blow count and shear wave velocity for a local site given by [4].

$$\text{Shear wave velocity } V_s = 84.87N^{0.26} \tag{6}$$

2 Sample calculation for bore hole

Since site lies in seismic zone factor II, therefore value of a_{max} is taken as 0.1. To account for fluctuation in water table it is assumed that water table is at ground level.
Since design earthquake magnitude 6.5, Magnitude scaling factor is calculated from eq (7)

$$\text{MSF}= 10^{2.24}/M_W^{2.56}= 10^{2.24}/6.5^{2.56} = 1.44 \tag{7}$$

Since according to IS 1893 [7], factor of safety against liquefaction should be more than 1.2 than graph between CRR/1.2 and CSR Vs Depth by various methods is plotted in Fig 7and fig 8 for liquefaction assessment. Data of National Disaster Management Authority (NDMA) [8] is used for consideration of earthquake.

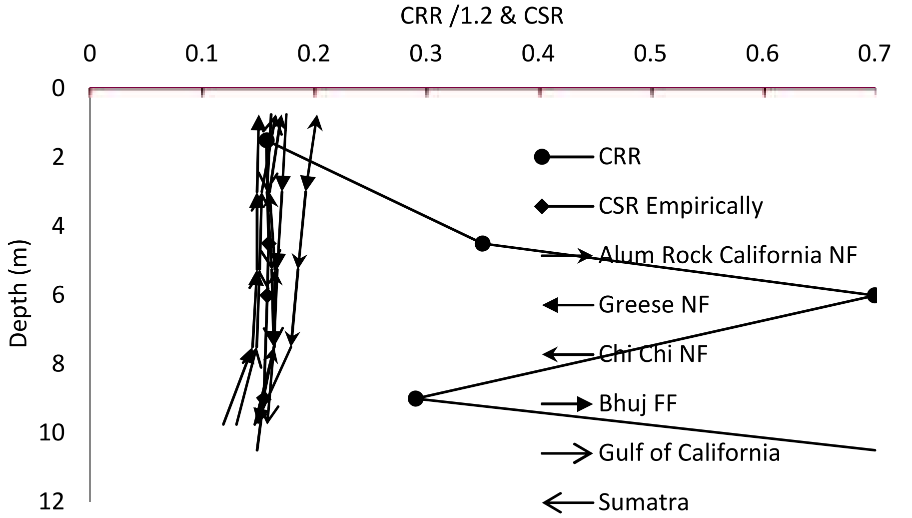

Fig. 7. Depth Vs CRR/1.2 & CSR in Non-linear analysis for finding out the zone where treatment is required

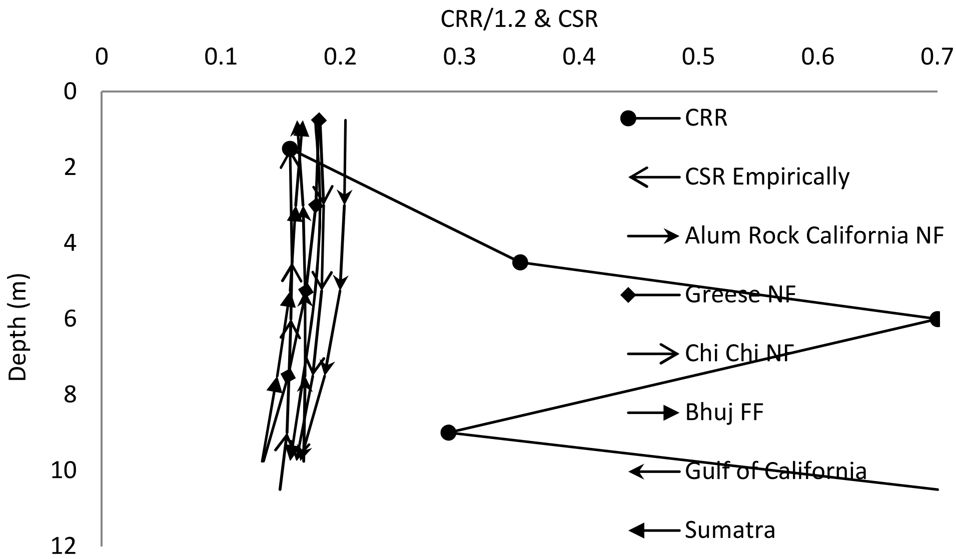

Fig.8. Depth Vs CRR/1.2 & CSR in Equivalent linear analysis for finding out the zone where treatment is required

Similar type of analysis has been done for remaining 17 Bore holes by equivalent linear and non-linear analysis for 1near field earthquake (Alum rock California and one Far field Earthquake (Bhuj). The results obtain for assessment of liquefaction potential is shown in Table 1

Table1 Summary of liquefaction Resistance at SPT Bore hole

BORE HOLE	Liquefaction Zone where treatment is required				
	Empirical Method	Alum rock California (Near field)		Bhuj (Far field)	
		Non-linear	Equivalent linear	Non-linear	Equivalent linear
BH 14	0 to 1.8 m	0 to 2.1 m	0 to 2.1 m	0 to 1.9 m	0 to 2 m
BH 26	0 to 1.9 m	0 to 2.2 m	0 to 3m	0 to 2m	0 to 2.2 m
BH 20	NIL	NIL	NIL	NIL	NIL
BH 28	NIL	NIL	NIL	NIL	NIL
BH 29	NIL	NIL	NIL	NIL	NIL
BH 31	0 to 4 m	0 to 5.5 m	0 to 5.4 m	0 to 5.2 m	0 to 5.2 m
BH 33	4.1 to 8 m	4 to 7.8 m	4 to 8.1 m	4 to 8.2 m	4.2 to 8 m
BH 9	NIL	NIL	NIL	NIL	NIL
BH 17	NIL	NIL	NIL	NIL	NIL
BH 23	NIL	NIL	NIL	NIL	NIL
BH 25	NIL	NIL	NIL	NIL	NIL
BH 32	NIL	NIL	NIL	NIL	NIL
BH 34	NIL	NIL	NIL	NIL	NIL
BH 35	NIL	NIL	NIL	NIL	NIL
BH 36	NIL	NIL	NIL	NIL	NIL
BH 37	NIL	NIL	NIL	NIL	NIL
BH 38	NIL	NIL	NIL	NIL	NIL

3 Results and Conclusions

In the present study, following conclusions have been made:
In the present study, liquefaction potential assessment for near field and far field ground motion is carried out. CSR has been evaluated using deterministic method and

ground response analysis using DEEPSOIL v 7.0. Equivalent linear and non-linear ground response analysis (GRA) is done.

Soil layers have reasonable influence in modifying the ground response either in amplification or de-amplification. Surface amplification/de-amplification has been observed in BH14 and BH26

Variation of PGA and CSR along the depth is obtained for 18 boreholes subjected to three far field and three near field ground motions. The CSR values obtained from empirical and GRA have been compared. It is observed that deterministic method underestimates CSR as compared to GRA carried out by DEEPSOIL. Hence factor of safety estimated from DEEPSOIL is lesser as compared to deterministic method. CSR value is more in case of near field ground motion as compared to far field ground motion. Hence it is necessary to carry our GRA for critical structures in order to know the accurate response of soil layers to the ground motion.

The effective comparison is carried out for 18 boreholes using non-linear and equivalent linear GRA using DEEPSOIL. It is observed that surface acceleration and spectral acceleration are considerably different for equivalent linear and non-linear GRA. The variation for near field motion is much larger as compared to far field motion. It is concluded that non-linear analysis provides more accurate results as it considers the actual non-linear behavior of soil.

Factor of safety is also estimated. It is observed that non-linear GRA provide more accurate results of soil liquefaction as compared to equivalent linear GRA which under predicts the factor of safety.

The results were compared and it was found that FOS against liquefaction potential by equivalent linear ground motion is less compare to non-linear ground motion, and FOS for near field earthquake ground motion is less compare to far field earthquake ground motion for same PGA value. Bore holes which is susceptible to liquefaction are located nearby to each other It justifies the applicability of different methods and importance of confirmation of results**.**

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