15th Symposium on Earthquake Engineering

Indian Institute of Technology, Roorkee

December 11-13, 2014

**Paper No. A003**

# SITE SPECIFIC RESPONSE BASED ON GLOBAL DATA, LIQUEFACTION POTENTIAL ASSESSMENT AND DETERMINATION OF TARGET VALUES FOR GROUND IMPROVEMENT FOR SHALLOW REGION IN INDIA

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## ABSTRACT

Destructive earthquake damages can be evidenced almost every part of the world in various possible forms. Extents of damage are also different for direct and induced effects. Subsoil properties play a vital role in controlling these actual damage scenario. Depending upon the subsoil properties, the surface can experience moderate to severe level of ground shaking. It is the modified ground motion created on the surface which controls the response of buildings and other infrastructure. Further, induced effects such as liquefaction and landslide are also the function of surface ground shaking. Thus, the surface level ground motion should be known to quantify these induced effects. These need subsoil properties and input bedrock ground motion. In absence of recorded ground motion regional ground motions or synthetic data have been practiced in many region specific studies. In this work site response of shallow region is attempted based on globally recorded ground motions. Amplification factors obtained from earlier studies are very high. Taking in-situ condition and design requirements into consideration, a methodology was proposed to reduce these values of amplification factor by the author. Thus, obtained surface ground motions can be used for liquefaction assessment. Further, if the site is found susceptible to liquefaction, ground improvement if practiced. Hence, in this work following the standard methodology to assess liquefaction potential, a new correlation is proposed. This correlation will provide corrected N-SPT value to be achieved in the field after ground improvement at different depths considering known earthquake loading at surface. These target values will ascertain no liquefaction condition at the site.

Keywords: *Earthquake, PEER, ground motion, site response, liquefaction, ground improvement.*

## INTRODUCTION

Frequent earthquakes (EQ) can be evidenced in different regions of India starting from seismically highly active regions of North-East India and the Himalayan belt to low seismicity regions such as the Peninsular India. Construction activities related to heavy infrastructures and dams in low seismicity region has resulted an increase in the seismicity (induced seismicity e.g. 1993 Killari Earthquake) compared to the past. As a consequence, frequent earthquakes, microtremors are being observed in the North-East part of the country, in the Himalayas and in the stable parts of the country as well. Induced effects such as excessive ground shaking, liquefaction and landslides are the results of modified ground motions which are the attributes of subsoil available at the site of the interest. Presence of subsoil alters the ground motion developed due to earthquake. Damages evidenced during 1918 Srimangal EQ, 1999 Chamoli EQ (earthquake) in Delhi, 2001 Bhuj EQ in Ahmedabad in India, 2011 Sendai EQ in Japan and many more are the clear indication that induced effects can be triggered even at far distance from the epicenter. The change in ground motion characteristics are the function of subsoil properties. Site response analysis determines this change in ground motion characteristics between the bedrock and the surface. These modified ground motions trigger induced effects. Effective estimation of induced effects depends upon the accuracy in the site response analyses. Many of the earlier published site response studies were region specific where a broad picture of ground motion amplification, predominant period, period of highest spectral accelerations were highlighted. Such studies can be used either as preliminary studies for future site specific analyses or as guidelines in case a site specific study needs to be performed. In most of the cases, the results from such studies predicts comparatively higher amplifications and hence construction cost will increase considerably. For important structures such as dams and Nuclear power plants, where safety of the structure is the priority and not the cost, uneconomical design is possible. However for routine structures, keeping the construction cost and the design consideration into account, such studies cannot be used directly ascertaining the safety of site against induced effects.

Overcoming the above limitations, this paper discusses data filtering methodology for the site response analysis results and a new correlation is proposed to ascertain no liquefaction condition at the site.

## STUDY AREA

Area considered in the present analyses belong to shallow crustal region of India. The study area belongs to seismic zone IV as per IS: 1893 (2002). Literature review suggests repeated moderate to severe damages in the study area due to distance earthquakes either in the Himalayan seismic belt or due to regional earthquake from other nearby sources. Further, analyses procedure developed in this work will be helpful to apply same procedure for other similar regions of India as well. As the boreholes were drilled for a client’s driven project and not under any research work, the location of the site has not been disclosed here (Abhishek *et al.*, 2014).

## SUBSOIL LITHOLOGY

Understand the subsoil lithology, 41 boreholes up to 30 m depth are drilled at the site under consideration. All the boreholes are of 150 mm diameter as per IS: 1892 (1974) and N-SPT values are measured regularly at 1.5 m interval as per IS: 2131 (1981). Data of N-SPT values, depth of sample collection and soil type identification etc. are logged during field testing. The physical properties are measured during the laboratory tests based on disturbed soil samples as per IS: 1498 (1970) and are used for soil classification in this paper. One typical borelog is shown in the Figure 1. It can be observed from Figure 1 that surface layer consists of filled up soil. Deeper layers consist of alternate beds of silty sand (SM) and medium to low compressibility clays (CI-CL). These soils are available in thickness ranges from 1 m to 6 m till a depth of 30m below the ground level. Disturbed and undisturbed soil samples are collected during the borehole drilling at various depths as mentioned in Figure 1. Water table level is reported after observing for 24 hours to ascertain no further variation. As mentioned in Figure 1, the water table for BH No 1 is at 0.9m below the ground surface. Plasticity index of the *in-situ* soil (CI-CL) varies between 10 % and 24 %. Overall observation by combining all the borelog suggested the presence of silty sand at most of the locations at various depths followed by layers of medium to low compressibility clays. Soil characterization as per National Earthquake Hazard Reduction Program (NEHRP) classification system BSSC (2003) using N-SPT values suggests the presence of soft soil (N < 15) up to 4 to 5m below the ground surface. Stiff soil (15 < N < 50) can be found in the depth range of 5m to 15m. At deeper depths (>15m), dense soils (N > 50) are encountered till the depth of 30m. NEHRP based soil classification based on 30m average N-SPT (N30) suggest site class E (N30 < 15) and D (15 < N30 < 50) for all the boreholes.

| BH No B1 | Ground Water Table at 0.9m below Ground Level (GL) | | | | | | |
|---|---|---|---|---|---|---|---|
| Depth below GL(m) | Soil Description | Thickness of Layer | Legend | Soil Classification | Sample | Depth(m) | SPT-N values |
| 1.0 | Fill | 1.0 | | - | - | 1 | 2 |
| 2.0 | Silty Sand | 1.0 | | SM | UDS | 2 | 4 |
| 3.0 | Low Compressibility Clay | 1.0 | | CL | SPT | | |
| 4.0 | | | | | SPT | 3.5 | 8 |
| 5.0 | | | | | SPT | 5 | 14 |
| 6.0 | | | | | SPT | 6.5 | 18 |
| 7.0 | | | | | UDS | | |
| 8.0 | | | | | SPT | 8 | 22 |
| 9.0 | Silty Sand | 6.0 | | SM | UDS | | |
| 10.0 | | | | | SPT | 9.5 | 24 |
| 11.0 | | | | | SPT | 11 | 26 |
| 12.0 | Medium Compressibility Clay | 2.0 | | CI | SPT | 12.5 | 33 |
| 13.0 | | | | | UDS | | |
| 14.0 | | | | | SPT | 14 | 37 |
| 15.0 | | | | | SPT | 15.5 | 45 |
| 16.0 | | | | | UDS | | |
| 17.0 | | | | | SPT | 17 | 55 |
| 18.0 | | | | | SPT | 18.5 | 62 |
| 19.0 | | | | | UDS | | |
| 20.0 | Silty Sand | 8.0 | | SM | SPT | 20 | 71 |
| 21.0 | | | | | SPT | 21.5 | 66 |
| 22.0 | | | | | UDS | | |
| 23.0 | | | | | SPT | 23 | 69 |
| 24.0 | Medium Compressibility Clay | 4.0 | | CI | SPT | 24.5 | 79 |
| 25.0 | | | | | UDS | | |
| 26.0 | | | | | SPT | 26 | 84 |
| 27.0 | Silty Sand | 3.0 | | SM | SPT | 27.5 | 95 |
| 28.0 | | | | | UDS | | |
| 29.0 | | | | | SPT | 29 | 96 |
| 30.0 | Medium Compressibility Clay | 3.0 | | CI | SPT | 30 | 101 |
| Note | Borehole terminated at 30.0 m | | | | | | |
| | DS-Disturbed Sample | SPT-Standard Penetration Test | | | | | |
| | UDS-Undisturbed Sample | | | | | | |

**Fig. 1:** Typical borelog from the study area considered in the present work (Abhishek *et al.*, 2014)

## SITE RESPONSE ANALYSIS

Induced effects such as amplified ground motion, liquefaction and landslides are the consequences of modified ground motion as reached to the surface. Recent example of site effect leads to 2011 Sendai earthquake in Japan and 2011 Sikkim earthquake in India. 2011 Sendai earthquake (Mw=9.0), even though the epicenter was 130 km off from the eastern causes massive liquefaction and foundation failure due to differential settlement at Maihama and Tokai Mora which were located 150 km from the epicenter Nihon (2011). 2011 Sikkim earthquake (Mw=6.8) causes several building collapse in Mangam, Jorethang and lower Zongue located 150 km away from the epicenter. Ground motions due to 2011 Sikkim earthquake were felt at many places in west Bengal and Bihar as well EERI (2012). Thus, irrespective of the magnitude of the earthquake, the damages can be spread in a wider area depending upon the subsoil properties.

In this work, the site response of a typical construction site is attempted. The site is selected by the client for important public utility. Equivalent linear site response approach is considered in this work using SHAKE2000 (Schnabel *et al*., 1972).

## GROUND MOTION SELECTION

In India, recorded ground motions are available only after 1986. Since then no major or great event has occurred in the Himalayan belt. Ground motion characteristics which controls the response of the soil during an earthquake are frequency content, duration and amplitude of the earthquake ground motion. To account for uncertainty about these ground motion characteristics during the future earthquake a large set of bedrock motions are needed which should cover a wider range of ground motion characteristics. In the absence of recorded data, globally recorded ground motions are taken from PEER (Pacific Earthquake Engineering Research) database as given in SHAKE2000. Selected ground motions are not concentrated to any specific region thus this problem is not dealt like any region specific study. In total 30 ground motions, all recorded at bedrock level are considered in this work. The bedrock PGA can be obtained from the seismic hazard analysis (SHA) for the study area as published by earlier researchers. However, the local site effects would be determined here. The end result is the amplification factor [ratio of surface PGA (Peak Ground Acceleration) to the bedrock PGA]. Thus, amplitude of selected ground motions will not affect the magnitude of surface PGA directly. Keeping this in mind ground motions are selected. A wider range of amplitude (0.036g to 1.03g) are covered. Also, the selected data covers a wide range of duration (6.8 s to 140 s) and frequency content (1.2 Hz to 50 Hz). Thus, a large variation of ground motion characteristics have been considered to account for the future earthquakes in the region.

## DYNAMIC SOIL PROPERTIES

The stress-strain behavior of soil is nonlinear. The modulus reduction ($G/G_{max}$) and damping of the soil are the function of the level of strains and is different for different soils. In equivalent linear approach, an initial value of shear modulus and damping is assumed which keeps on updating after every iteration to perform the site response analysis. Thus, the soil response is a function of the modulus reduction ($G/G_{max}$) and damping properties of the soil. These modulus reduction ($G/G_{max}$) and damping ratio curves for each of the material are obtained from laboratory tests such as simple shear, torsional shear, cyclic triaxial and resonant column tests (Doroudian and Vucetic, 1995; Stewart *et al*., 2001). However due to limited resources and the standard curves available for each type of material based on large number of tests. Such curves are being used in most of the site response studies (Stewart *et al*., 2001). These curves can be selected depending upon the soil type, it’s over consolidation ratio (OCR), plasticity index (PI) and many other properties which are resemblance of that particular soil. For the present work, three types of soils are considered as given in Figure 1. These include silty sand, low compressibility clays and medium compressibility clays. Since the client recommendations are not to place foundation in the fill layer, any fill layer has not been modelled while performing site response analysis. Thus three types of soils are considered from the SHAKE database as 1) Average sand for silty sand; 2) Clay with PI 0 to 10 and 3) Clay with PI 10-20. $G/G_{max}$ and damping curves for sandy soil given by Seed and Idriss (1970), is used for silty sand layers. Similarly, Sun *et al.*, (1988) studied $G/G_{max}$ ratio of clay with different PI with over consolidation ratio (OCR) of 5 to 15. Sun *et al.*, (1988) found that low value of PI has considerable effect on position of $G/G_{max}$ curve when compared to high PI clays. Authors [13] proposed different $G/G_{max}$ curve for clay with different plasticity Index (PI) values. Hence, $G/G_{max}$ curve for clay soil is selected from Sun *et al.*, (1988) based on PI values. The average damping curve for clay is independent of PI of the clay. For this reason one damping curve as per Seed and Idriss (1970) is used for both the clays (CI and CL). Selected damping curves and the $G/G_{max}$ for various soil types in this work are shown in Figure 2 and 3 respectively (Abhishek *et al*., 2014). Depth of water table obtained

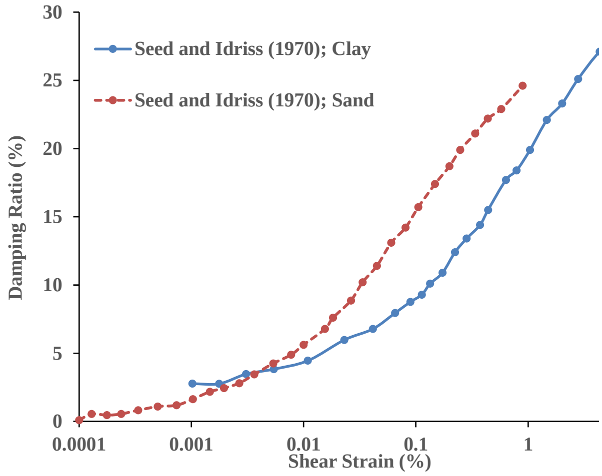


**Fig. 2:** Damping ratio curves for different soil types

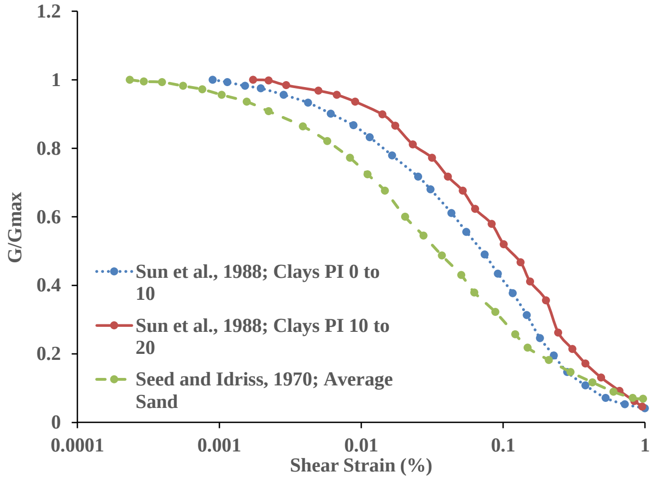


**Fig. 3:** $G/G_{max}$ curve for different soil types

from borelog reports are considered while modelling soil column in SHAKE2000. Subsurface soil properties and soil dynamic model curves discussed above are used as inputs modelling of soil columns. Further, each of the soil column is subjected to all the selected 30 ground motions and the response in terms of amplification factor are observed.

## ANALYSIS AND RESULTS

Equivalent linear site response analyses is performed as using SHAKE2000 (Schnabel *et al.*, 1972). All the soil columns defined as per the selected soil types and dynamics soil properties are assigned to each of the soil layer. Thicker layers (> 3m) are subdivided into 3 m thickness. N-SPT obtained from the *in-situ* test are used to determine the initial shear modulus of the soil. Each of the soil column is subjected to all the 30 selected ground motions. Outputs in the form of PGA variation with depth, acceleration time history, stress-strain time histories at selected layers are obtained. The surface PGA

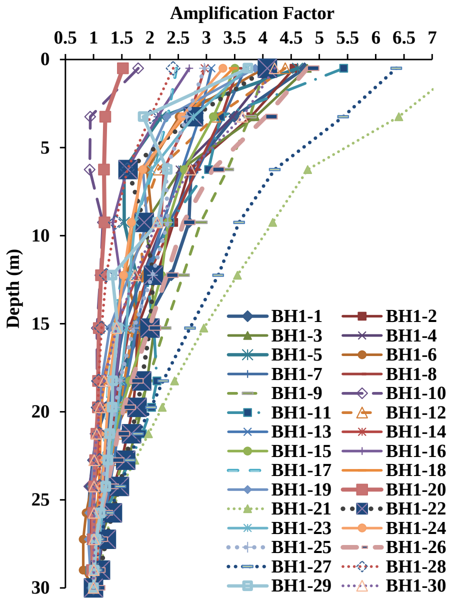


**Fig 4:** Typical plot showing the variation in amplification factor corresponding to selected ground motions (Abhishek *et al.*, 2014)

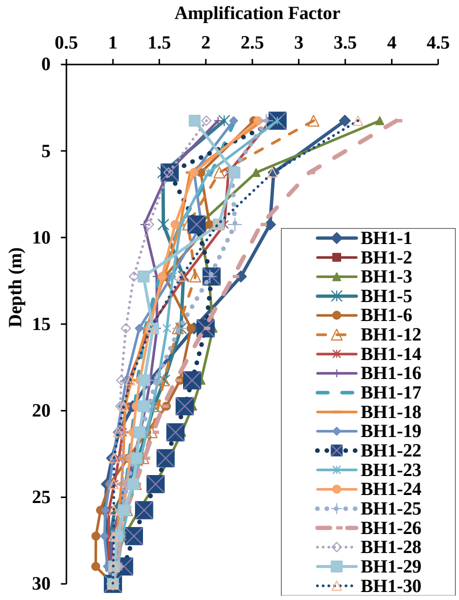


**Fig. 5:** Revised amplification factor variation corresponding to Figure 1 after applying the three observations (Ref: Abhishek *et al.*, 2014)

obtained from the analyses are used to determine the amplification factor. Figure 4 shows the variation of amplification factor considering various ground motions as bedrock motion (Abhishek *et al*., 2014). Since PGA values at various depths are normalized with respect to the bedrock PGA, for this reason all the graphs are narrowed to amplification factor of 1.0 at 30 m depth. Numbers given in the legend are the nomenclature given to the input ground motions. Analysis results shows a wide scatter in the amplification factor. Further, the variation in amplification factor below 25 m depth is minimal. In order to narrow down this range of amplification actor and to enhance confidence on selected value of amplification factor, a detailed procedure using the same data set was proposed by Abhishek *et al*., (2014). As per Abhishek *et al*., (2014), minimal to no amplification was found for ground motions having bedrock PGA ≥0.52g. Further, lower amplitude input motions shows higher values of amplification factor. Similarly to provide guidelines on borehole termination, Abhishek *et al*., (2014) proposed lower value of amplification in case of sandy soil with N-SPT ≥ 50 beyond 25 m depth. For clays of low to medium compressibility as observed from Figure 1 and having N-SPT ≥70 at depths below 25 m shows no to minimal amplification. Hence, in case of similar site conditions, drilling of boreholes can be terminated at shallower depth compared to standard 30 m practise. Considering the limitation of the length of the paper, the detailed procedure proposed by Abhishek *et al.*, (2014) has not been repeated here. As per Abhishek *et al*., (2014), the wide variation in amplification factor was narrowed based on revised observations and design considerations as shown in Figure 5. This was proposed as post filtering procedure by Abhishek *et al*., (2014) and would have considerable impact on the construction cost. Further, the revised range of amplification factor was confined to a value of 2.5 based on statistical analysis as given in Abhishek *et al.,* (2014). The same value has been used to determine the surface PGA.

National Disaster Management Authority (NDMA), Government of India, developed the probabilistic seismic hazard maps for entire country. The entire country was divided into 7 tectonic zones based on the seismotectonic parameter characterization as per Seeber *et al*., (1999) considering 32 aerial sources. Ground motion prediction equations (GMPEs) for each of the tectonic regions were developed separately based on the synthetic ground motions. Finite Fault models considering regional seismotectonic parameters were used to develop synthetic ground motion for each zone. Considering the proposed GMPEs and the past seismicity, detailed Probabilistic seismic hazard maps for entire country were developed. These maps show spectral acceleration for various periods. Further, seismic hazard maps for different return period were also developed. This work is referred NDMA (2010) in this paper. Limitation of IS: 1893 (2002) in predicting the present seismicity of the country has been highlighted by number of earlier studies. Overcoming the shortcoming of codal recommendations for the study area, bedrock PGA as per NDMA (2010) is considered in this work. Bedrock PGA for the site considering 10% probability of exceedence in 50 years as per NDMA (2010) is 0.08g which is used in this work. Using this value of bedrock PGA and an amplification factor of 2.5, the surface PGA was proposed as 0.20g (Abhishek *et al*., 2014).

## PROPOSED EMPIRICAL CORRELATION FOR NO LIQUEFACTION CONDITION

*In-situ* failure of soil occurs in the form of loss of its shear strength. The soil loses all its shear strength and apparently behaves like a liquid under a specific earthquake loading. This phenomena is called as Liquefaction. A liquefied soil can have minimal shear strength of 1.5-2.0 kN/m$^2$ which is negligible compared to the overcoming load on the soil. Large scale damage due to liquefaction are usually evidenced. Since the soil become a liquid, anything it is supporting will undergo severe damages. This can be in the form of excessive uniform settlement and large amount of differential settlement. As a result, it may lead to failure of utilities such as drainage lines, sewer lines, building and bridge collapse etc. Few examples of large scale liquefaction include; 1869 Cachar EQ, 1964 Niigata EQ, 1971 San Fernando EQ, 1977 Argentina, 1989 Loma Prieta, 1995 Great Hansin EQ, 2001 Bhuj EQ, 2004 Niigata EQ and 2011 Sikkim EQ. Paleo-liquefaction studies in Assam also confirm liquefaction failures during Assam earthquake (Sukhija *et al*., 1999). These are the classical examples where the damages due to liquefaction were reported far away from the epicentre during an earthquake (Abhishek *et al*., 2013). Such examples clearly highlights the presence of softer medium at the shallow depth can cause the scenario more catastrophic even for distant regions. Since, the external loading triggers the phenomena, Liquefaction is classified as induced hazard of earthquake. It can be quantified once the surface level of PGA and subsoil shear strength properties are known. Standard methodology to estimate the liquefaction potential of the *in-situ* soil includes determination of the Cyclic Resistance Ratio (CRR). It resembles the resistance of soil against liquefaction. Cyclic Stress Ratio (CSR) which is the measure of external loading triggering liquefaction at the site. Different standard correlations are available to estimate CRR and CSR for the given site and to calculate the Factor of Safety of the soil against liquefaction (Seed and Idriss, 1971; Ambraseys, 1988; Roberston and Wride, 1997; Idriss, 1999; Youd and Idriss, 2001; Idriss and Boulanger, 2010). These methodologies are evolved for various fine contents and considering field observations from different regions. The most recent methodology by Idriss and Boulanger (2010) based on large number of data set from SPT chamber tests was developed. In this work, this methodology has been adopted and by performing a regression analysis, empirical correlation has been proposed for no liquefaction condition. As per Idriss and Boulanger (2003) the overburden correction was correlated to corrected N-SPT [$(N_1)_{60CS}$] as below;

$$C_N = \left(\frac{P_a}{\sigma'_{vo}}\right)^m \leq 1.7 \tag{1}$$

$$m = 0.784 - 0.0768\sqrt{(N_1)_{60CS}} \tag{2}$$

Where, $\sigma_{vo}{}'$ is the effective overburden pressure in kN/m$^2$, $P_a$ is the atmospheric pressure equal to 100kN/m$^2$. It can be observed from equation 1 and 2 that an iterative process is needed to determine the value of $C_N$. Once the value of "$m$" is known through iteration, the safety of site against Liquefaction can be determined since other available empirical correlations are also available to determine the CRR and CSR. In case the site is found liquefiable, it is treated by suitable ground improvement methods so that the *in-situ* resistance can be enhanced. In this section, a new correlation has been proposed which will determine $(N_1)_{60CS}$ values ascertainiung no liquefaction following the methodology of Idriss and Boulanger (2003). These can be called as target values to be achieved after ground imporovement.

In addition to overburden correction "$C_N$", other corrections to be applied are rod length correction "$C_R$", borehole diameter correction "$C_B$", liner correction "$C_S$", hammer energy correction "$C_E$" and Fine content correction "$\Delta(N_1)_{60}$". In standard practise in India, boreholes are drilled in 150 mm diameter, thus the value of $C_B$ would be 1.05 as per Robertson and Wride (1997). In absence of any liner in the borehole, the value of $C_S$ would be 1.05 as per Robertson and Wride (1997). Again the value of $C_R$ would be vary from 0.85 to 1.0 as per Robertson and Wride (1997). The average value of $C_E$ of 0.70 is considered as per Robertson and Wride (1998) for safety hammer type. For a known value of F.C. (Fine content), the value of $K_S$ can be calculated as per Robertson and Wride (1997). Again a magnitude value of 7.5 is considered to determine Magnitude Scaling Factor (MSF) as this is the standard magnitude value. In liquefaction analyses, to consider the worst scenario, the water table is generally considered at the surface. In the present work also, the water table has been considerd at the ground surface to calculate effective stress "$\sigma_{vo}{}'$"and total stress "$\sigma_{vo}$". Further, for Liquefaction to occur, CSR should be slightly greater than CSR. Further, for ease in the regression analysis, CSR = CRR has been considered in the present work. After so much of observations discussed earlier, and considering the value of $P_a$ as 100 atm, the problem is reduced to CRR or CSR, $\sigma_{vo}{}'$ and $(N_1)_{60CS}$. The value of CSR can be obtained from earthquake loading. Hence, by performing an inverse step by step regression analyses a new empirical correlation has been proposed as given in equation (3). The proposed correlation will predict $(N_1)_{60CS}$ for a known earthquake loading at various depth. Since this value ascertains no liquefaction condition, the proposed equation yields target values of corrected N-SPT which should achieved from ground improvement technique.

$$CRR = -0.144\ln(\sigma'_{vo}) + \left\{\frac{(N_1)_{60CS}}{20.9046\,(\sigma'_{vo})^{0.0866}}\right\}^3 - \left\{\frac{(N_1)_{60CS}}{17.3507\,(\sigma'_{vo})^{0.1263}}\right\}^2 + \left\{\frac{(N_1)_{60CS}}{24.5475\,(\sigma'_{vo})^{0.2048}}\right\} - \left\{\frac{1.6941}{(\sigma'_{vo})^{0.2241}}\right\} + 1.3073 \quad (3)$$

Where, $(N_1)_{60CS}$ is the corrected N-SPT or the target value, $\sigma_{vo'}$ is the effective stress obtained from borehole report and CRR is equal to CSR and can be obtained from the earthquake record. can be obtained. In can be observed from equation (3) that proposed correlation is of order three. There will be only one real root of this equation which will be the target value to be achieved in field. Other two values thus can be simply ommited. Further, as the foundations are always rested at certain depth below the ground surface, there should be always some value of effective stress "$\sigma_{vo'}$" while using equation (3). So, if the value of earthquake magnitude and Fine content of the *in-situ* soil is known, the uncorrected N-SPT can be calculated. This would be the in-situ value to be achieved during the test. This kind of correlation is very important particularly for geotechnical industry involved in ground improvement works. Such a correlation will give in advance the N-SPT values to be achieved at the end of the treatment before the actual treatment.

## CONCLUSION

Site response is a very crucial phenomena as it controls the damage scenario during an earthquake. The kind of damages cannot be only a function of earthquake magnitude and its distance from the site but also depends upon the subsoil characteristics available at the site under consideration. Large number of field tests both geotechnical and geophysical are available to determine the *in-situ* subsoil properties. The subsoil properties change drastically and thus planning for field testing for site characterization is a challenging objective. This paper presents two outputs. Once is on data filtering for site response analysis which will reduce the amplification factor and corresponding cost saving in the construction phase. Land available for the construction activities is very limited. Thus, if a site is found as liquefaction susceptible from the detailed analysis, then ground improvement is usually adopted to treat the site. Standard methodology are available to perform liquefaction susceptibility of any site. Following the standard methodology, an inverse approach has been attempted where many variables have been narrowed and the problem is reduced to a simpler form. A new empirical correlation has been proposed in this work. The proposed correlation will yield *in-situ* soil N-SPT which should be achieved from ground improvement. These target values will ascertain no liquefaction condition for that site under a given earthquake loading and subsoil properties. Keeping the design consideration, a minimum value of effective stress should be there while using the above equation. With this correlation, a cost effective study can be made before hand to adopted one particular ground improvement technique. Further, once target values are known in advance, it is relatively easier to achieve the same in the field.

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