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1D NONLINEAR GROUND RESPONSE ANALYSIS OF SOILS IN IIT GUWAHATI AND LIQUEFACTION POTENTIAL IDENTIFICATION

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Abstract

Earthquakes are natural phenomena that have caused the loss of millions of lives and property over centuries. Thus, characterizing these strong ground motions, predicting their damage potential and suggesting measures to minimize their threat becomes essential. For any particular site, the response of the sub-soil strata under seismic excitations plays an important role in governing the safety of earth retaining structures and super-structures. For sandy layers, with a noticeable fines content and water table close to the ground surface, the excess pore water pressure (PWP) build up rapidly under repeated cyclic loading. This further lowers the effective strength of the soil increasing the chances of occurrence of liquefaction. Guwahati city, which is located on the banks of river Brahmaputra in North-East India, is a zone of high seismicity and numerous earthquakes have been recorded in and around this region in the past. Most of the earlier ground response analysis (GRA) studies that have been performed in this region have used the linear or equivalent linear approach, with very few studies focusing on nonlinear approach. However, we know that unlike linear/equivalent linear approach, nonlinear approach can account for the pore water pressure generation and degradation of soil stiffness with number of loading cycles that occurs in an actual earthquake scenario. In the present paper, 1-D nonlinear effective stress ground response analysis, incorporating non-Masing criteria, and pore water pressure dissipation are performed on three borehole sites located at IIT Guwahati region in North Guwahati. The Sikkim earthquake motion recorded at IIT Guwahati and three of its scaled-up components are used as input. Peak horizontal acceleration (PHA) profiles, amplification/attenuation characteristics of the ground motion in the soil column, amplification factor, and 5% damped surface response spectra are presented. Reduction in amplification factor with increase in peak bedrock acceleration (PBRA) value of input motion is observed. For input motions having PBRA values in the range of 0.18g to 0.36g, soils in this region will attenuate the ground motion. Liquefaction susceptibility has also been studied using cyclic stress approach. Soils in this region are found to have a tendency to liquefy under the effect of ground motions with PBRA values greater than 0.06g. The depths of liquefaction in the three soil profiles, subjected to input motions with different PBRA values, are also identified. Furthermore, a comparison is made between equivalent linear and nonlinear approach of GRA based on some response parameters.

Keywords: 1D Nonlinear GRA; Non-Masing; Effective Stress Analysis; Response Parameters; Liquefaction

1. Introduction

Due to the devastations caused by earthquakes, geotechnical engineers face a very pertinent challenge to identify the causes and characteristics of these strong ground motions and the extent to which they can wreck damage. The damage potential of any particular earthquake depends on the properties of soils through which they are transmitted. At a particular site, the safety of the structures is guided by the response of the underlying soil strata subjected to the propagating seismic waves. The response is evaluated based on parameters like peak ground acceleration or PGA (the peak horizontal acceleration at ground level), maximum shear strain in soil profile, spectral acceleration (S_a) to name a few. Thus, performing an extensive GRA study for any site becomes an essential task in characterizing the site.

Guwahati city and the entire North-East India as a whole have experienced several moderate to large magnitude earthquakes in the past and have been designated as seismic zone V (as per BIS 2002) which is the most active seismic zone in the country. Widespread damages ranging from settlement failures, structural failures and liquefaction related failures have been reported by Oldham [1] and Poddar [2] during the event of the major earthquake occurrences in this region like the Shillong plateau earthquake (1897) and the Assam earthquake (1950).

In the non-linear approach of one-dimensional GRA, the response of a soil deposit is analyzed by performing numerical integration in time domain in small steps. Numerical integration technique like Newmark β method [3] may be used. Any non-linear stress strain model following Masing [4] or non-Masing rules may be used during the integration process. At the beginning of each time step, the stress-strain relationship is referred to obtain the soil properties to be used in that time step. This approach can account for the degradation of stiffness that occurs with the number of loading cycles, and also the generation of pore water pressure. In non-linear time-domain analysis the soil profile is discretized into a number of layers using multi-degree-of-freedom lumped parameter model and the following equation of motion, Eq. (1), is solved.

$$[M]\{\ddot{u}\}+[C]\{\dot{u}\}+[K]\{u\}=-[M]\{I\}\ddot{u}_g \tag{1}$$

where, $[M]$ is the mass matrix, $[C]$ is the viscous damping matrix, $[K]$ is the stiffness matrix, $\ddot{u}$ is the vector of relative nodal accelerations, $\dot{u}$ is the vector of relative nodal velocities, u is the vector of relative nodal displacements, $\ddot{u}_g$ is the acceleration at the base of soil column, $[I]$ is an unit vector.

The hyperbolic stress-strain model developed by Kondner and Zelasko [5] and its subsequent modified forms developed by Matasovic and Vucetic [6], Hashash and Park [7] and others, are used to describe the initial backbone curve for the first loading cycle. The stress-strain behavior in the subsequent cycles is obtained by modeling the soil stiffness degradation, with pore water pressure that develops, as the parameter for modeling the degradation. The non-linear model developed by Phillips and Hashash [8] with hysteretic damping reduction factor referred to as MRDF procedure, which modifies the Masing and extended Masing rules, has been employed in the DEEPSOIL [9] code for performing non-linear non-Masing analyses. Eq. (2) and Eq. (3) are used to represent the loading and unloading/reloading conditions, respectively, to obtain the stress-strain curves.

$$\tau=\frac{G_0\gamma}{1+\beta\left(\gamma/\gamma_r\right)^s} \tag{2}$$

$$\tau=F(\gamma_m)\left[2\frac{G_0((\gamma-\gamma_{rev})/2)}{1+\beta((\gamma-\gamma_{rev})/2\gamma_r)^s}-\frac{G_0(\gamma-\gamma_{rev})}{1+\beta(\gamma_m-\gamma_r)^s}\right]+\frac{G_0(\gamma-\gamma_{rev})}{1+\beta(\gamma_m-\gamma_r)^s}+\tau_{rev} \tag{3}$$

where, γ : given shear strain, γ_r : reference shear strain, β : dimensionless factor, s : dimensionless exponent, γ_{rev} : reversal shear strain, τ_{rev} : reversal shear stress, γ_m : maximum shear strain, $F(\gamma_m)$: reduction factor, G_0 : initial shear modulus.

Eq. (4) and Eq. (5) are used to represent the dependency of reference shear strain and small-strain damping ratio on confining pressure.

$$\gamma_r = REF.\,strain\left(\frac{\sigma'_v}{REF.\,stress}\right)^b \tag{4}$$

$$\xi = \frac{Damping\ ratio}{(\sigma'_v)^d} \tag{5}$$

where, σ'_v is the effective vertical stress, ξ is the effective small-strain damping ratio, b and d are curve fitting constants. The values of b and d are taken as zero when the reference strain and damping ratio are considered to be confining pressure independent.

The pore water pressure generation model developed by Matasovic [10] for sands and Matasovic and Vucetic [11] for clays is implemented in the DEEPSOIL code. The pore water pressure dissipation model which is employed in DEEPSOIL [9], is based on Terzaghi's [12] 1D consolidation theory.

Identification of liquefaction potential may be done using the cyclic stress approach or the cyclic strain approach. The cyclic stress approach is more commonly used owing to its simplicity. In the cyclic stress approach, the earthquake induced loading is expressed in the form of a ratio known as the cyclic stress ratio (CSR), and the liquefaction resistance of the soil is expressed in the form of a ratio known as the cyclic resistance ratio (CRR). CSR can be estimated based on the formulation by Seed and Idriss [13]. The liquefaction resistance of the soil may be characterized based on laboratory tests or in-situ tests like Standard Penetration test, Cone Penetration test to name a few. CRR can be estimated from Seed et al. [14, 15, 16]. Finally, the factor of safety (FOS), which is the ratio of CRR to CSR, is evaluated. Regions where the FOS value falls below 1 may be susceptible to liquefaction.

2. Modeling of Soil Profile and Input Details

2.1 Regional geology and geotechnical data

For the present study, soil profiles from IIT Guwahati region, which is located on the banks of river Brahmaputra, have been considered. Soil data have been taken from available borehole log reports for this region. The sub-surface geology profile in this region consists of inter-bedded layers of clayey silts, silty sands and sands. Water table has been observed to be quite close to the ground surface (within 2 meters from the existing ground level) in almost all of the borehole locations under consideration. The soil up to 10-15 m depth is mostly soft or in loose state with SPT-N value much lesser than 30, thereby, making them highly susceptible to liquefaction. The shear wave velocities (V_s) of different layers in the soil profiles have been evaluated from the empirical correlation with SPT-N value given by Imai and Tonouchi [17].

2.2 Methodology

Non-linear GRA, incorporating non-Masing criteria, has been performed on the soil stratigraphic profiles using DEEPSOIL v6.0 [9] and, subsequently, liquefaction susceptibility identification has been done.

The modulus reduction and damping curves that need to be defined for different soil layers, have been obtained from the formulations of Ishibashi and Zhang [18]. These formulations take into account the soil properties like plasticity index, confining pressure, over-consolidation ratio, angle of internal friction to name a few. The obtained curves have been fitted using the MRDF procedure and, subsequently, the parameters for the stress-strain model get defined. The reference strain and small-strain damping ratio of the soil profiles have been considered to be independent of confining pressure. Frequency independent small-strain viscous damping formulation, given by Phillips and Hashash [8] has been used.

Liquefaction potential has been evaluated using the cyclic stress approach. The cyclic stress ratio (CSR) has been calculated based on the Seed and Idriss [13] formulation. Reduction factor required for calculating CSR has been estimated using the formulation provided by Liao and Whitman [19]. The cyclic resistance ratio (CRR) has been evaluated based on the recommendations by Youd and Idriss [20].

2.3 Description of soil profiles

Soil profiles from three borehole locations, referred to as BH1, BH2 and BH3, have been studied.

BH1 soil profile consists of a layer of sandy silt overlying a layer of fine to medium sand with a total depth of 23 m. Ground water table is observed at a depth of 1.7 m from the existing ground level.

BH2 soil profile consists of a top fill layer underlain by inter-bedded layers of clayey silt, sandy silt and sand up to a depth of 30 m. Ground water table is observed at a depth of 1.5 m from the ground surface.

BH3 soil profile consists of inter-bedded layers of clayey sand, silty sand and fine to medium sand up to a depth a 15 m. Ground water table is observed at a depth of 0.7 m from ground surface.

2.4 Description of seismic loading

The Sikkim earthquake (2011) strong motion recorded at IIT Guwahati with peak bedrock level acceleration (PBRA) of 0.02g, and three scaled-up components of the motion with peak bedrock level accelerations of 0.06g, 0.18g and 0.36g, have been used as input for analyses. The 0.36g PBRA motion (extreme case) has been selected based on the peak accelerations recorded at some of the sites around the hypocenter of the earthquake in Sikkim, where peak bedrock level accelerations as high as almost 0.4g were recorded. These motions have been applied at the base of the three soil profiles BH1, BH2 and BH3. Fig. 1, Fig. 2, Fig. 3 and Fig. 4 shows the acceleration time history of the four motions with PBRA of 0.02g, 0.06g, 0.18g and 0.36g respectively.

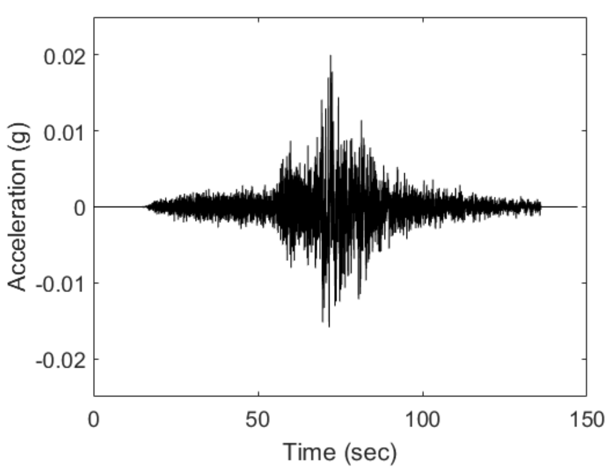

Fig. 1 - PBRA=0.02g

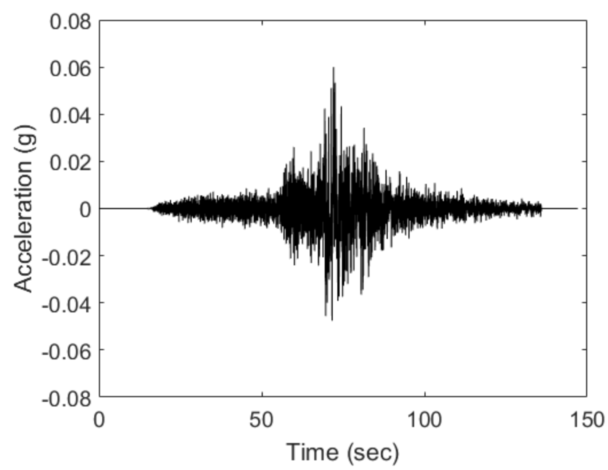

Fig. 2 - PBRA=0.06g

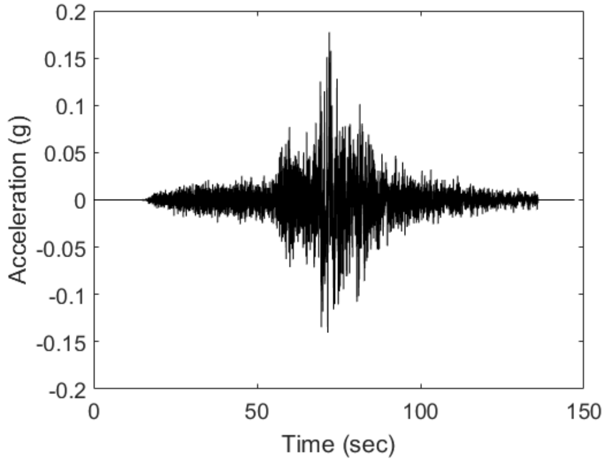

Fig. 3 - PBRA=0.18g

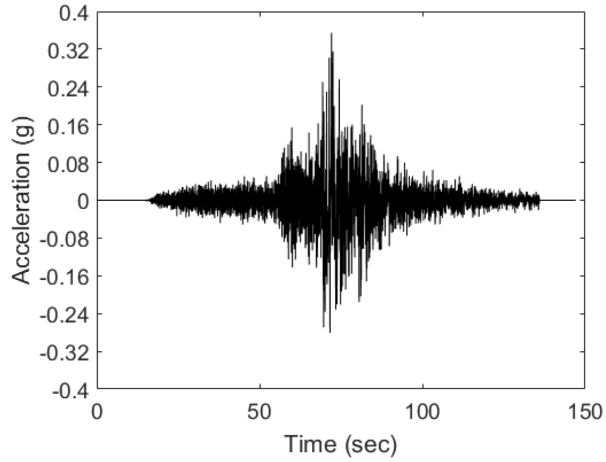

Fig. 4 - PBRA=0.36g

3. Results and Observations

Each of the three soil profiles has been analyzed with all the four input motion components and the response has been expressed in terms of PHA profile, PGA, amplification factor, spectral acceleration (S_a). Subsequently, liquefaction susceptibility of the three sites has also been looked into and the zone of liquefaction has been identified.

3.1 Site-BH1

It is observed that the amplification factor, i.e. the ratio of PGA to PBRA, decreases with increase in PBRA value of input motion. For the 0.02g and 0.06g PBRA input motions, amplification of motion is observed from bottom of the soil profile to the top whereas for 0.18g and 0.36g PBRA motions, PGA value at the top of the soil profile is less than the PBRA value. PHA vs. Depth profiles for site BH1 are shown in Fig. 5 for the four input motion components. Fig. 6 shows the 5% damped surface response spectra plots for the four motion components.

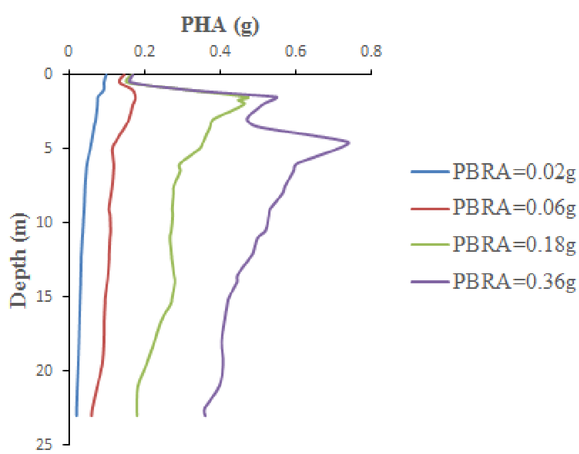

Fig. 5 - PHA vs. Depth

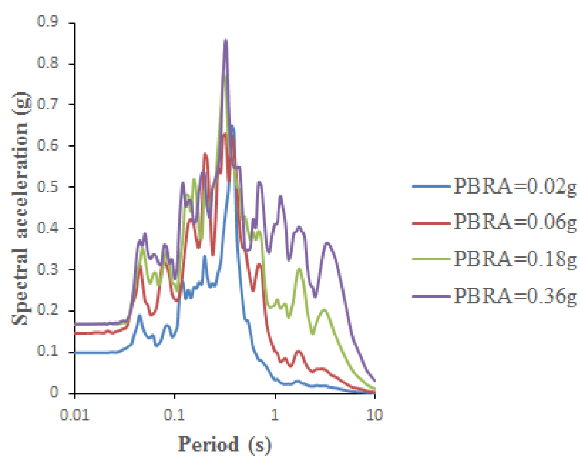

Fig. 6 - Response spectra

For the motion with PBRA of 0.02g, chances of liquefaction of the soil column are minimal, as shown in Fig. 7. However, for the motions with PBRA of 0.06g, 0.18g and 0.36g about 7 m of soil column from the ground surface, where the FOS falls below 1, is susceptible to liquefaction (Fig. 7).

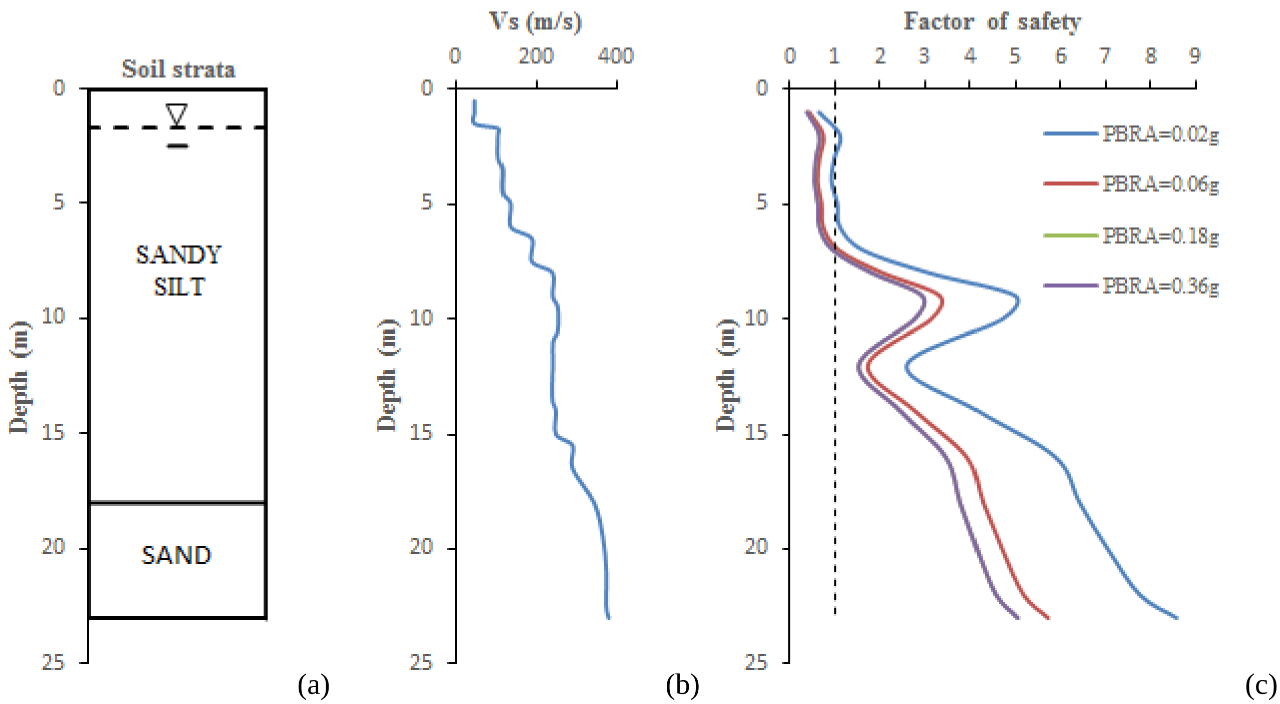

Fig. 7 – (a) Sub-surface soil profile at site BH1 (b) Shear wave velocity profile at site BH1 (c) FOS vs Depth for the four input motion components

3.2 Site-BH2

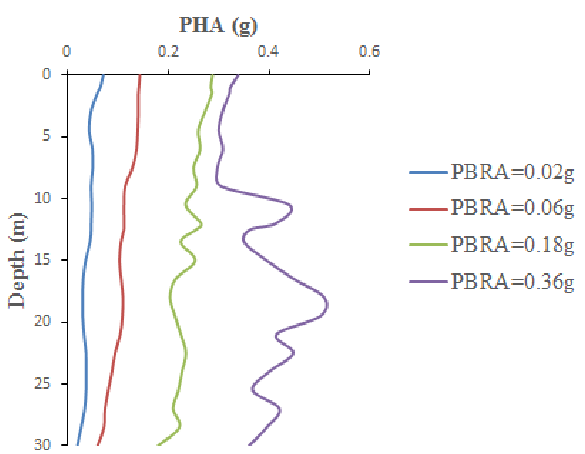

Fig. 8 - PHA vs. Depth

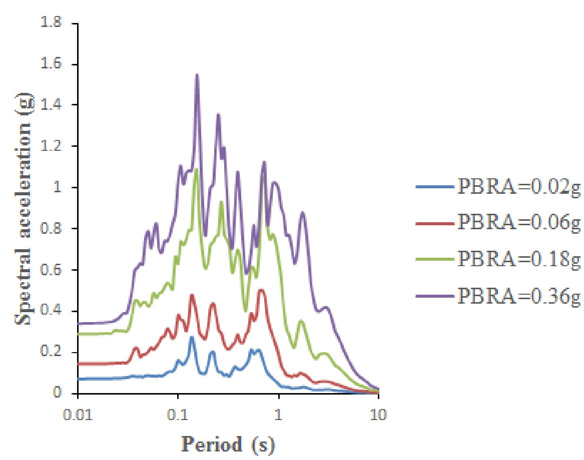

Fig. 9 - Response spectra

The amplification factor is observed to decrease with increase in PBRA value of input motion. Amplification of ground motion is observed from bottom of the soil profile to the top for the motions with PBRA values of 0.02g, 0.06g and 0.18g, whereas for the 0.36g PBRA input motion, PGA value at the top of the soil profile is less than the PBRA value. PHA vs. Depth profiles for site BH2 are shown in Fig. 8 for the four input motion components. Fig. 9 shows the 5% damped surface response spectra plots for the four motion components.

For the two motion components with PBRA of 0.02g and 0.06g, chances of liquefaction of the soil column are less (Fig. 10). However, for the motion components of 0.18g and 0.36g PBRA, it is seen that liquefaction is likely to occur in the soil column up to a depth of 25 m from the ground level (FOS < 1).

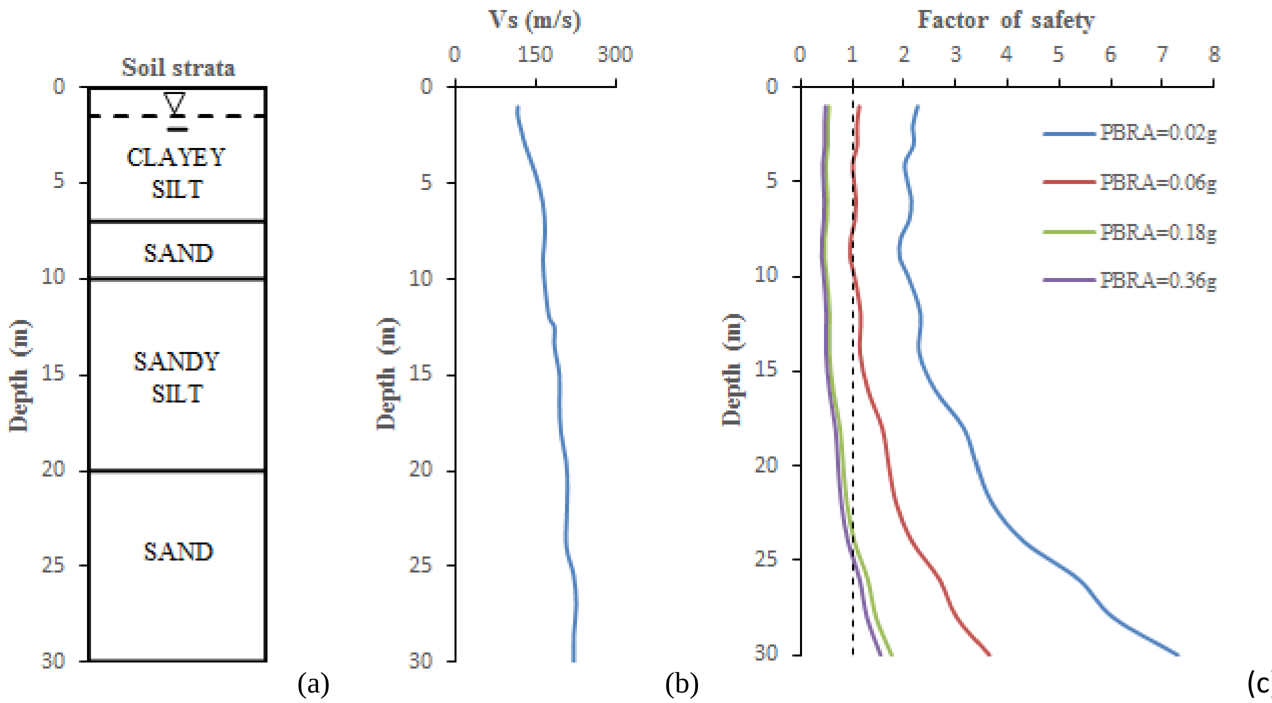

Fig. 10 - (a) Sub-surface soil profile at site BH2 (b) Shear wave velocity profile at site BH2 (c) FOS vs Depth for the four input motion components

3.3 Site-BH3

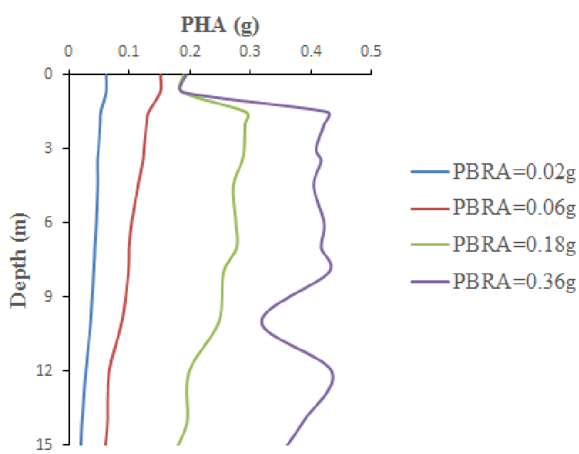

Fig. 11 - PHA vs. Depth

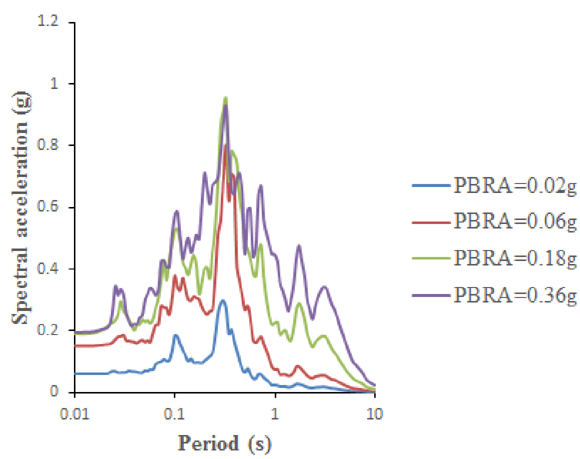

Fig. 12 - Response spectra

It is observed that ground motion amplification takes place from bottom to the top of the soil profile for 0.02g, 0.06g and 0.18g PBRA motion components, whereas for the 0.36g PBRA motion, the PGA value at the ground surface is less than 0.36g, i.e. the PBRA value of the motion. Amplification factor can be seen to be decreasing with increase in PBRA value of the input motion. Fig. 11 and Fig. 12 shows the PHA vs. Depth profiles and the 5% damped surface response spectra plots for all the four input motion components.

No liquefaction is expected to occur in the soil column for the 0.02g PBRA motion component as shown in Fig. 13. For the motion component with PBRA of 0.06g, liquefaction is likely to occur in some of the intermediate layers in the soil column (Fig. 13). For the 0.18g and 0.36g PBRA motions, soil layers in between 2 m to 11 m from the ground surface are likely to liquefy (Fig. 13).

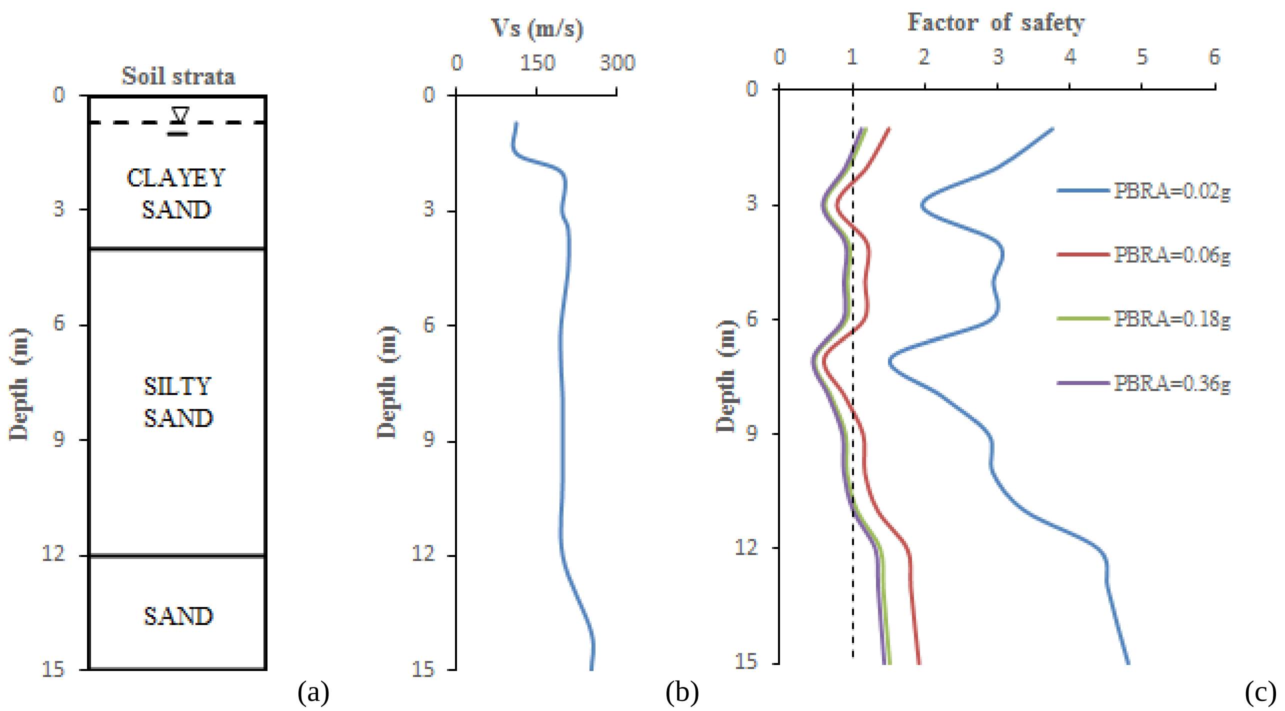

Fig. 13 - (a) Sub-surface soil profile at site BH3 (b) Shear wave velocity profile at site BH3 (c) FOS vs Depth for the four input motion components

Table 1 summarizes the PGA values, amplification factors obtained for the combination of all the four input motions for the three sites BH1, BH2 and BH3.

Table 1 – Summary of PGA values and amplification factors

	PBRA=0.02g	PBRA=0.06g	PBRA=0.18g	PBRA=0.36g
BH1	PGA=0.1g Amp. factor=5	PGA=0.15g Amp. factor=2.5	PGA=0.17g Amp. factor=0.94	PGA=0.17g Amp. factor=0.47
BH2	PGA=0.07g Amp. factor=3.5	PGA=0.14g Amp. factor=2.33	PGA=0.29g Amp. factor=1.61	PGA=0.33g Amp. factor=0.92
BH3	PGA=0.06g Amp. factor=3	PGA=0.15g Amp. factor=2.5	PGA=0.19g Amp. factor=1.06	PGA=0.2g Amp. factor=0.56

For the four selected input motions, the variation in amplification factors obtained at all the three sites is shown in Fig. 14. As stated earlier, it is evident from the figure that the amplification factor at any site decreases with an increase in the intensity (higher PBRA) of input motion. This characteristic has also been reported in past literature by Warnitchai and Lisantono [21], Aashford et al. [22] and Kumar et al. [23].

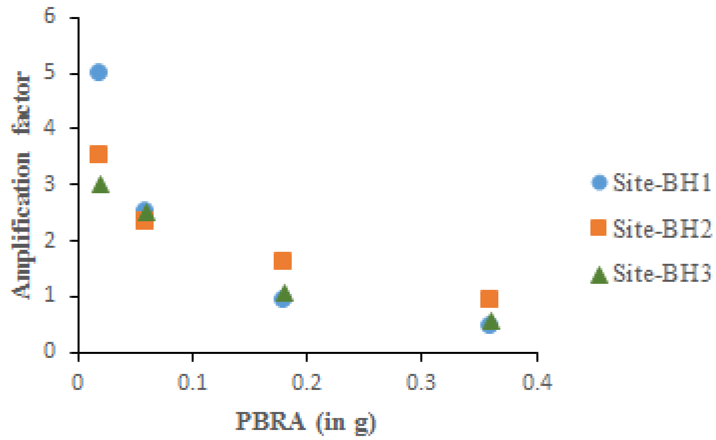

Fig. 14 - Effect of varying input motion PBRA on the amplification factor estimated for the three sites

3.4 Comparative study of nonlinear (NL) and equivalent linear (EL) GRA

1D GRA using EL approach has been performed for all the three borehole sites using the four Sikkim earthquake motion components (one recorded and three scaled-up components described earlier). Fig. 15 and Fig. 16 shows the difference in PHA profiles obtained using EL and NL approaches for sites BH1 and BH3 respectively.

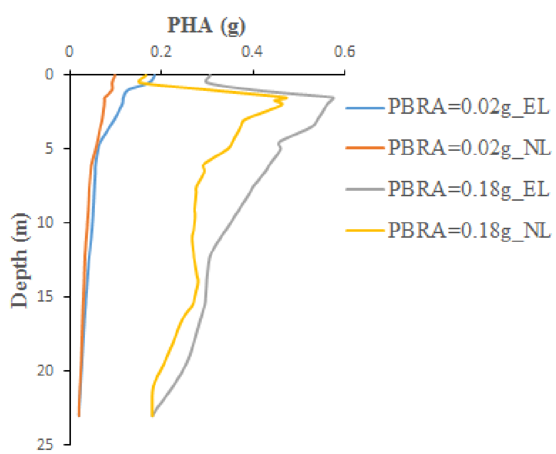

Fig. - 15 PHA vs Depth for site BH1

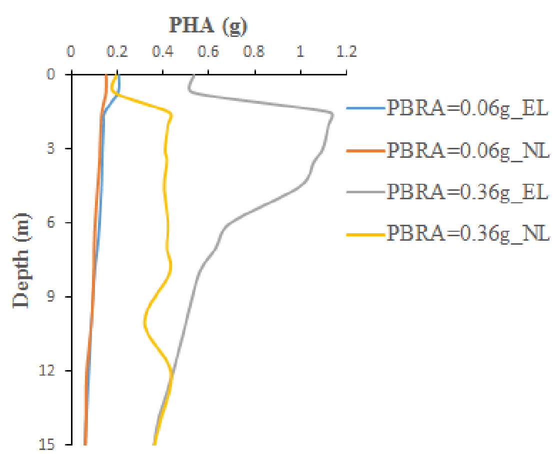

Fig. - 16 PHA vs Depth for site BH3

From Fig. 15 and Fig. 16, it is seen that for the two low amplitude motion components of 0.02g and 0.06g the difference between the PHA profiles computed for EL and NL approaches are almost similar. However, for the two high amplitude motion components of 0.18g and 0.36g, considerable difference between the PHA profiles computed for EL and NL approaches is observed, with EL method resulting in much higher PGA value as compared to NL method. Fig. 17 exhibits a typical consistently increasing difference between the PGA values obtained from EL and NL approaches for varying input motion PBRA.

These observations are primarily attributed to the two major differences in the analysis approach of the two methods. (a) In equivalent linear approach, the stiffness of any soil layer remains constant throughout the duration of a seismic loading. Thus, high amplification resulting in resonance might occur if a strong component of the input motion coincides with the natural frequency of any soil layer. However, in nonlinear approach, since the stiffness of soil changes continuously within the time-span of the seismic loading, this type of resonance in any particular soil layer is not possible. (b) In nonlinear approach (effective stress analysis), degradation of soil stiffness is taken into account due to the rise in excess pore-water pressure with number of loading cycles. Equivalent linear approach (total stress analysis), however, does not take into account any effect of excess pore-water pressure (PWP) build up in the soil. Thus, the resulting shear stresses computed in the soil profile are much higher in case of equivalent linear approach. Since, the PGA value is directly computed from the maximum shear stress developed, equivalent linear approach gives higher PGA values. As the PBRA of the input motion increases the difference in PGA values obtained from equivalent linear and nonlinear approach will increase. This is because, for input motion with low PBRA value, excess PWP developed is quite less. Thus, the shear stresses and peak accelerations computed will be similar for both the methods. Similar observations have been reported by Ghaboussi and Dikmen [24], Joyner and Chen [25].

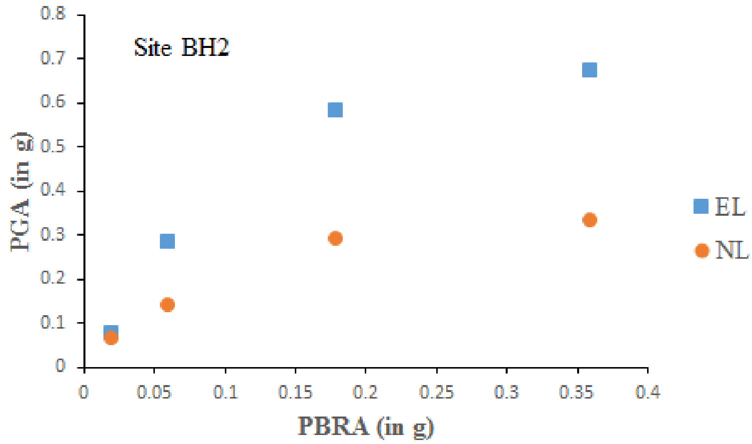

Fig. - 17 Effect of varying input motion PBRA on the PGA estimated from EL and NL approaches for site BH2

4. Conclusions

Different sets of 1D nonlinear ground response analyses, incorporating non-Masing criteria, have been performed on three borehole sites in the IIT Guwahati region, subjected to input motions having four different peak bedrock level acceleration (PBRA) values. It has been observed that the amplification factor decreases with increase in PBRA value of input motion for all the three sites which means the amplification of ground motion waves, as it travels upwards from the base of the profile to the top, decreases as the PBRA value increases. For the 0.18g PBRA motion, PGA value that has been obtained is almost same as the PBRA value, i.e. an amplification factor of 1, for the two borehole sites BH1 and BH3. For the site BH2, ground motion amplification has been observed for the input motion with PBRA of 0.18g. For the motion with PBRA of 0.36g, amplification factor lesser than 1 has been observed for all the three sites. Thus, it can be stated that attenuation of ground motion is expected to occur in the soils found in this region, for input motions having PBRA in the range 0.18g-0.36g. The 5% damped surface response spectra plots for the three sites, subjected to four input motions having different PBRA values, have also been presented.

Furthermore, liquefaction study has been performed for the three sites using cyclic stress approach. It has been observed that for the 0.02g PBRA input motion, liquefaction is not expected to occur in any of the sites. However, for motions having PBRA values greater than 0.06g, all the three soil sites are susceptible to liquefaction. The depth and zone of liquefaction in the soil columns have been observed to increase with increase in PBRA value of input motion. Thus, it is seen that widespread liquefaction is likely to occur in the entire region. Similar cases of widespread sand boils, and settlement of houses, levees and embankments in liquefied soils all across the Brahmaputra river plains have been reported during the earthquakes of Shillong plateau in 1897, and Assam in 1950, by Oldham [1] and Poddar [2]. A more recent liquefaction potential study of Guwahati city by Raghu Kanth and Dash [26] has also indicated high chances of liquefaction in this region. Finally, a comparative study of equivalent linear and nonlinear approach of GRA has been presented. It has been observed that equivalent linear approach results in a higher PGA value as compared to nonlinear approach and this difference in computed PGA for the two approaches increases with increase in the PBRA of input motion.

5. Acknowledgements

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6. Copyrights

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