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EFFECTIVE STRESS v/s TOTAL STRESS GROUND RESPONSE ANALYSES FOR A TYPICAL SITE IN CHENNAI (INDIA)

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ABSTRACT

This paper presents the results of ground response analyses carried out for a typical sandy site in Chennai city by equivalent linear and nonlinear total and effective stress approaches. The soil profile at the site consists of 26m thick sandy layer with SPT blow count increases from 16 to above 50 with depth. The shear wave velocity profile measured using field Multichannel Analysis of Surface Wave (MASW) test is found to increase from 170 m/s to 400 m/s at 26m depth. The equivalent linear ground response analysis was carried out using SHAKE2000. The nonlinear total and effective stress analyses were performed using D-MOD2000. In the nonlinear total stress analysis, the Modified Kondner and Zelasko (MKZ) constitutive model was used. In the case of nonlinear effective stress analysis, modulus degradation and stress degradation models of Matasovic and Vucetic (1993) were used to incorporate the pore pressure parameters. The analyses were carried out for a time history of bedrock acceleration with PGA of 0.16g obtained from the seismic hazard analysis. The results of the analyses are presented in terms of ground acceleration, shear stress and shear strain. The results of the equivalent linear, nonlinear total and effective stress analyses show similar ground response characteristics except marginal variation in the period corresponding to the peak spectral acceleration due to low intensity of input motion.

INTRODUCTION

The seismic ground response analysis is commonly carried out by the equivalent linear method which is based on the wave propagation theory, due to its simplicity. However, non-linear, step-by-step integration, total stress methods provide more accurate results especially in the case, when significant nonlinearity and development of large shear strains are induced by strong earthquakes. Recently, effective stress methods, which also account the effects of excess pore water pressures on the ground response are also developed.

To study the effectiveness of these methods, the ground response analyses were carried out with reference to a typical sandy site at Chennai by the following three methods: equivalent linear analysis, nonlinear total stress and effective stress methods. The site predominantly consists of sandy strata with a thickness of 26m. The shear wave velocity profile for the ground response analyses was obtained from field MASW test. The analysis was carried out for a bedrock input acceleration time history with PGA of 0.16g. The comparison was carried out in terms of ground accelerations, shear stresses, shear strains and other relevant response properties.

GEOLOGY OF THE STUDY REGION

Chennai, India's fourth largest metropolitan city is located between 12.75° - 13.25° N and 80.0° - 80.5° E on the southeast coast of India as shown in Fig. 1. The city spreads over 19 km in length along the Coromandel coast and extends inland about 9 km and covers about 172 sq km. Chennai is trisected by two east flowing rivers: Cooum river and Adayar river that traverses along its width. The general geology of the city comprises of mostly sand, clay, shale and sandstone as shown in Fig. 2 (GSI, 1996). The study area has two distinct geological formations: the shallow bedrock (crystalline) on the east and south, and the Gondwanas (conglomerate, shale and sandstone) below the alluvium to the north and west. Almost the entire area is covered by the Pleistocene / Recent alluvium, deposited by the two rivers, Cooum and Adayar. Igneous / metamorphic rocks are found in the southern area; marine sediments containing clay-silt sands and Charnockite rocks are found in the eastern and northern parts, and the western parts are composed of alluvium and sedimentary rocks.

The study site, Mylopore as shown in Fig. 1 is located along the east coast of Chennai city. The coastal region of the city is fully covered by marine sediments. The alluvium is underlain by the crystalline rock complex of charnockite - granitic gneiss along the southern and eastern parts with depth ranging from 10 to 30m below ground level. It is seen that in general, the eastern coastal zone is predominantly sandy deposit, while the northwestern region is mostly clayey in nature. The seacoast is flat and sandy for about a km from the shore.

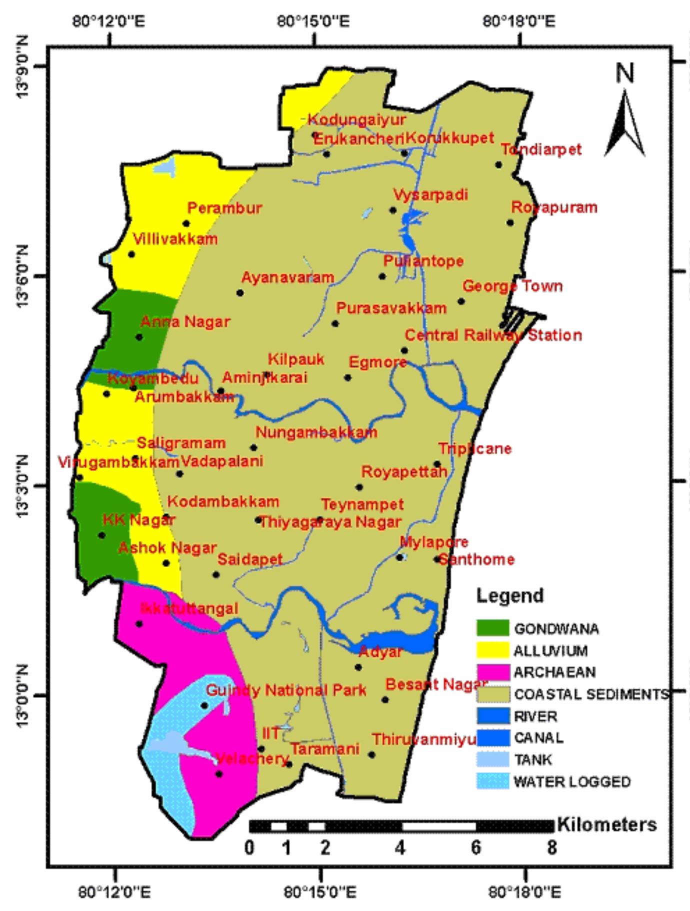


Fig. 1. Geology map of the Chennai city

SITE DESCRIPTION

The site is characterized predominantly by a thick layered deposit of medium dense to very dense silty sand. The details of borelog and the variation of SPT-N with depth are shown in Fig. 2. The SPT-N value varies from 16 to above 50 with depth. The top 1m layer consists of filled up soil. The relatively dense sand layer with SPT-N varies from 28 to 33 is up to a depth of 3m. This layer is followed by a 6m thick medium dense sand deposit with SPT-N value varies from 16 to 27. It is followed by a dense sand layer up to a depth of 21 m with SPT-N in the range of 30 to 50. This layer is followed by very dense silty sand with SPT-N above 50 up to a depth of 26m. The weathered bedrock encountered at a depth of 26m from the surface. The water table is encountered at a depth of 2m below the ground surface. The sand is classified as SW (well graded sand) as per IS: 1498 (1997) with fine content in the range of 30 to 70 %.

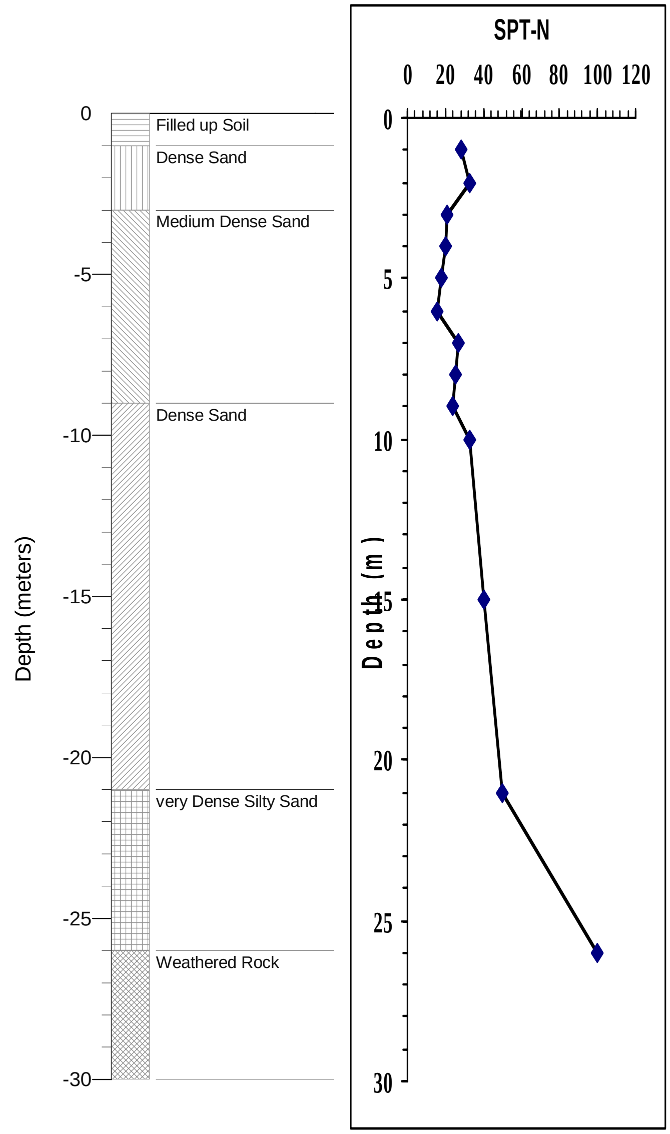


Fig. 2. Borelog and variation of SPT-N with depth

SHEAR WAVE VELOCITY PROFILE

In the seismic site response studies, the shear wave velocity profile is an important characteristic of the soil to represent the low stiffness properties of the soil. Currently Multichannel Analysis of Surface Wave (MASW) method is widely used to obtain the shear wave velocity (V_s) profile due to its accuracy, coherency and relatively less time consumption for investigation a large area. The shear wave velocity profile obtained from surface wave method involves three steps: acquisition of ground roll, construction of dispersion curve (phase velocity vs. frequency) and back calculation (inversion) of the V_s profile from the calculated dispersion curve (Park et al. 1999).

In the present study the MASW tests were carried out using Geometrics make 24 channels Geode seismic recorder with single geode operating software (SGOS). The vertical geophones with natural frequency 4.5 Hz (24 nos.) were used to receive the wave fields generated by the active source of 8 kg sledgehammer. Twenty four geophones were deployed in a linear pattern with equal receiver spacing in the range of 0.5 to 1 m interval with the nearest source to geophone offset in the range of 5 to 15m to meet the requirement of different types of soil as suggested by Xu et al. (2006). The source and each receiver are connected to an individual recording channel as shown in Fig. 3.

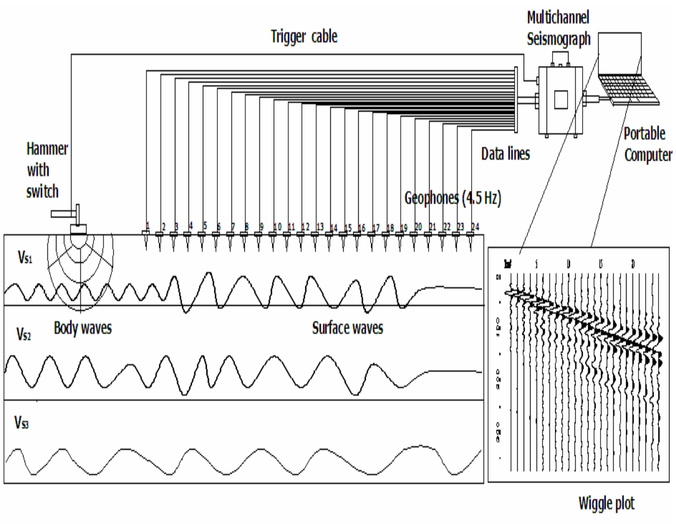


Fig. 3. Field test setup of MASW test

The acquired wave data were processed using the SurfSeis software to develop experimental dispersion curve. The experimental dispersion curve was subjected to inversion analysis to develop one-dimensional (1D) shear wave velocity profile. The shear wave velocity profile obtained from MASW test is shown in Fig. 4. It indicates that V_s of the upper dense sand layer varies from 175 to 200 m/s. This layer is followed by the relatively medium dense sand layer with V_s of 150 m/s up to a depth of 7 m and followed by higher velocity of 190 m/s up to depth of 9 m. The V_s of the dense sand layer is around 280 m/s. This layer is followed by very dense silty sand with Vs of 400 m/s up to a depth of 26m. The weathered rock with V_s of 750 m/s is encountered at a depth of 26m. The weighted average shear wave velocity over 30m depth $(Vs)_{30}$ is about 260 m/s. The study site is classified as site D as per NEHRP (2000) using $(Vs)_{30}$ procedure.

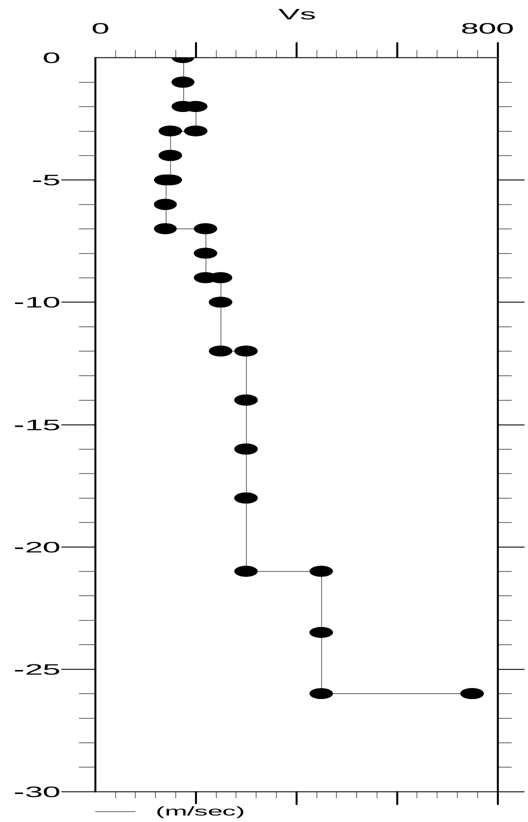
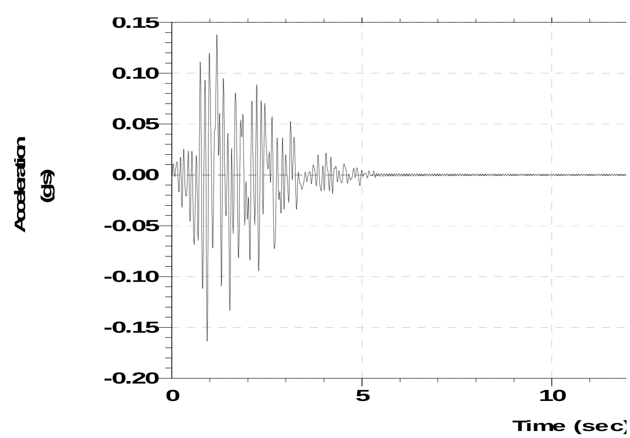


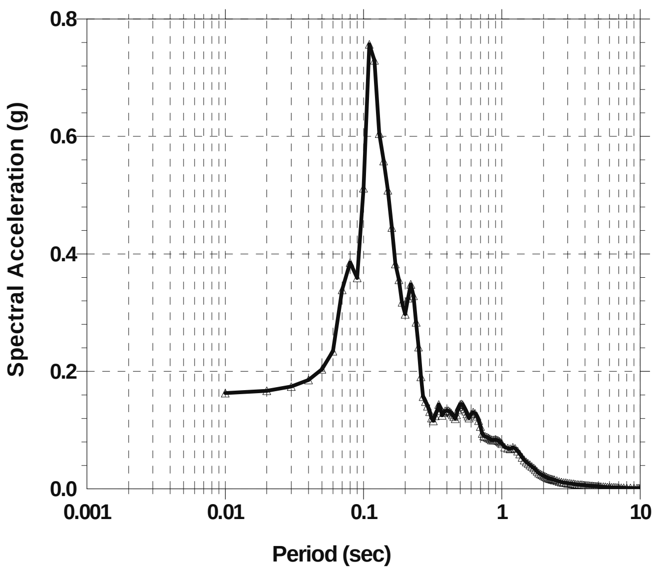
Fig. 4. Shear wave velocity profile from MASW test

INPUT MOTION

The seismic hazard analysis was carried out for the city considering 300 km radius of influence considering all faults and seismicity. Based on the deterministic seismic hazard assessment, the fault 24 at a distance of 22 km was identified as vulnerable fault for the city having moment magnitude of 5.5 with PGA of 0.16g. In order to carry out the ground response analyses for the study site, a synthetic bedrock motion was developed using stochastic method considering major fault as shown in Fig. 5 (a). Fig. 5 (b) shows the response spectra for the input motion.



(a)



(b)

Fig. 5 (a) Acceleration time history and (b) Response Spectra for the input motion

METHODS OF GROUND RESPONSE ANALYSES

Three different methods of analyses by total stress and effective stress methods were used to evaluate the seismic response of the considered profile. The main characteristics of the employed methods are discussed below.

Equivalent Linear Method

The equivalent linear method uses total stress procedure for ground response analysis using SHAKE2000, in which the soil is represented by a damped equivalent linear model. An equivalent shear strain, i.e 65% of the maximum shear strain is considered to select the corresponding shear modulus and damping according to a selected strain-dependent soil properties. These shear modulus and damping are constant and assumed to be relevant at any stage of the load application. Since this is an elastic analysis with degraded shear modulus, permanent deformation can not be accounted for, i.e., upon cessation of the motion, the system returns to its initial, non-deformed, position. Equivalent viscous damping simulates the effects of hysteretic material damping. The following soil parameters such as small-strain shear wave velocity, modulus reduction curves, hysteretic damping curves and material density are considered for the equivalent linear analysis.

In the present study the sandy strata of the site considered is divided into 17 layers with varying thickness of 1 to 3m. The input motion was specified as outcropping at the top of the elastic halfspace. The measured shear wave velocity is used to characterize the low stiffness properties of the soil and standard modulus reduction and damping curves (Seed and Idriss, 1970) were adopted.

Nonlinear Effective and Total Stress Methods

The nonlinear analysis is required in case of strong ground shaking and/or for week soil deposits. The nonlinear total stress method has been carried out using D-MOD2000 which is descendant of the computer program DESRA-2 (Lee and Finn, 1978). The total and effective stress analysis is executed incrementally in time domain, in which a nonlinear stress strain relation of the soil is used. In these methods, the stiffness and hysteretic damping of soil are represented with nonlinear hysteretic springs connected to lumped masses. It uses the dynamic response model which is the numerical solution for the dynamic equation of motion in the time domain, developed by Lee and Finn (1978). Additional viscous damping is included through the use of viscous dashpots.

Layer Stratification and Thickness. In this nonlinear ground response analysis, the layer stratification was achieved based on maximum frequency of a layer (f_{max}) which is the highest frequency that the layer can propagate. The f_{max} is calculated as: $f_{max} = V_s/4H$ where, V_s and H are the shear wave velocity and thickness of the layer, respectively. If a layer is too thick, the maximum frequency that a layer can propagate is small. The most commonly used f_{max} to calculate an adequate thickness of a layer for site response analysis is 25 Hz. In this case the layer thickness is calculated as $h_i = (V_s)_i/100$. The minimum layer thickness is controlled by wavelength of incoming motion and it is assumed as 1/8th of shortest wavelength. The sandy layers up to a depth of 26 m were divided with 17 layers with thickness in the range of 1 to 3m. each layer is divided into two subdivisions. The width is considered as 0.3 m for all the layers. The input motion was specified as outcropping at the top of the elastic halfspace (transmitting boundary).

Evaluation of the Viscous Damping Model Parameters. The full Rayleigh damping formulations (which match a target damping ratio at one or two frequencies, respectively) was specified for this analysis as per the following equation:

$$\alpha_R = \xi_{tar} \cdot \left(4\pi/T\right)\left[\frac{n}{n+1}\right] \tag{1}$$

$$\beta_R = \frac{\left(\xi_{tar} \cdot T\right)}{\left[\pi(1+n)\right]} \tag{2}$$

where n is an integer (1,3,5…11) and ξ_{tar} is the target viscous damping ratio. Here, n and ξ_{tar} parameters were selected by calibrating against SHAKE analysis. The different n and ξ_{tar} parameters were used until satisfactory match of spectra was achieved between SHAKE and D-MOD2000. The best match parameters used in this analysis are: n = 5 and ξ_{tar} = 0.5 %.

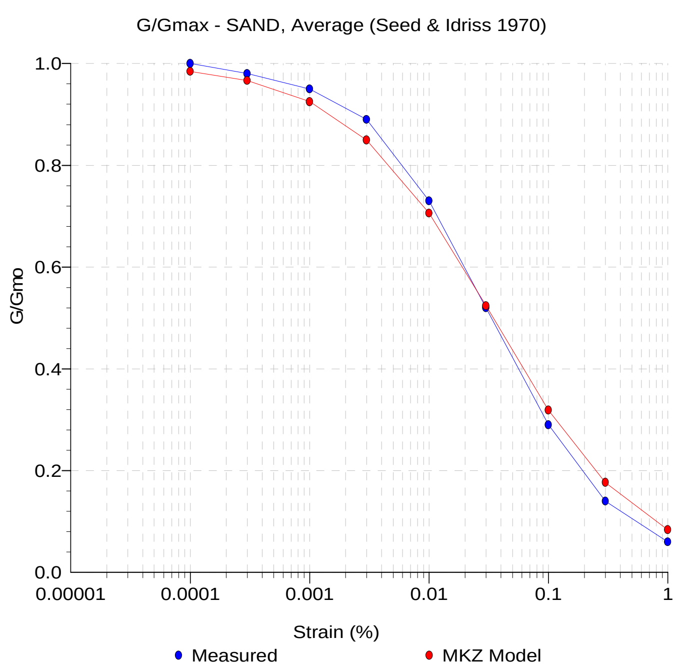
Evaluation of Material and Model Parameters. The required material parameters are small-strain shear wave velocity obtained from MASW test, reference strain (τ_{mo} / G_{mo}) and unit weight of the soil layers. D-MOD2000 incorporates constitutive model for sand, clay and enables analysis of composite soil deposits. In addition to the representation of stiffness and damping with nonlinear hysteretic springs and viscous dashpots, an energy transmitting boundary is included at the model's base.

In the total stress analysis, the soil behaviour is represented by a nonlinear backbone curve (which is a curve fit to match G/G_{max} curves) coupled with extended massing rules that describe unload – reload behaviour and establish the level of hysteretic damping. The Modified Kondner and Zelasko (MKZ) constitutive model (Matasovic and Vucetic, 1993, 1995) is used to define the initial backbone curve and is given as

$$\tau^* = f^*(\gamma) = \frac{G^*_{m0}\gamma}{1 + \beta\left(\frac{G^*_{mo}}{\tau^*_{mo}}\gamma\right)^S} \tag{3}$$

where $\tau^*_{mo} = \frac{\tau_{mo}}{\sigma'_{vc}}$, $G^*_{mo} = \frac{G_{mo}}{\sigma'_{vc}}$ and β, S – constants

In the curve fitting constants β and S adjust the position of the curve along the ordinate and control the curvature. The typical Seed and Idriss (1970) curves for sand with the adjusted MKZ model is shown in Fig. 6.



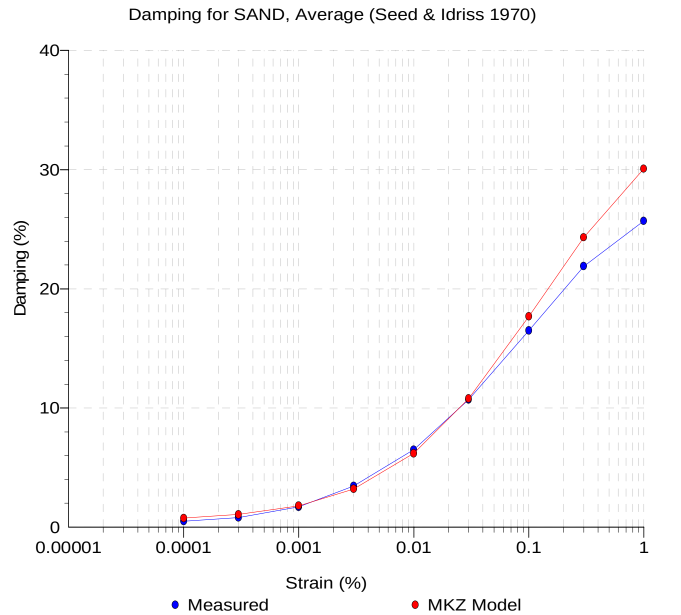


Fig. 6. Comparison between Seed and Idriss (1970) curve and fitted MKZ model curve

In the effective stress analysis, in addition to the soil nonlinearity, the effect of excess pore pressure generation is also considered by using the modulus degradation and the stress degradation models in normalized form, expressed by the following equations (Matasovic and Vucetic, 1993):

$$G^*_{mt} = G^*_{mo}\sqrt{\frac{(\sigma'_{vc} - u)}{\sigma'_{vc}}} = G^*_{mo}\sqrt{1 - u^*} \quad \text{and}$$

$$\tau^*_{mt} = \tau^*_{mo}\frac{(\sigma'_{vc} - u)}{\sigma'_{vc}} = \tau^*_{mo}\left(1 - u^*\right) \tag{4}$$

where u^* = normalized residual excess pore water pressure. The equation for the initial backbone curve and then the associated degraded backbone curve corresponding to different values of u^* is

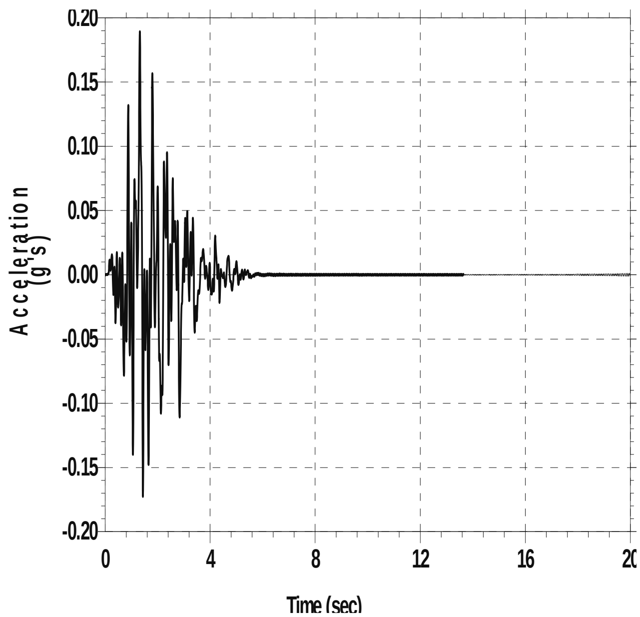
$$\tau^* = f^*(\gamma) = \frac{G^*_{mt}\gamma}{1 + \beta\left(\frac{G^*_{mt}}{\tau^*_{mt}}|\gamma|\right)^S} \tag{5}$$

RESULTS AND DICUSSION

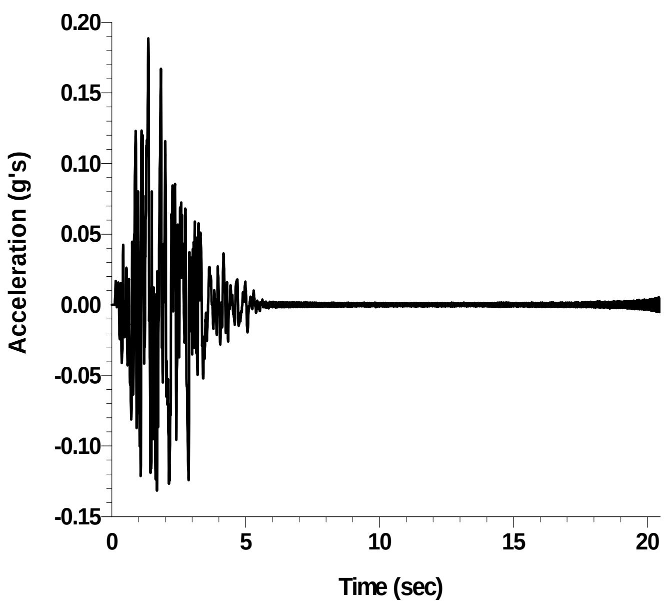
The results of the ground response analyses carried out by three methods are compared in terms of the computed accelerations, shear stresses, shear strains and other response properties.

Time History of Acceleration at the Surface

The acceleration time history at the surface obtained from equivalent linear, nonlinear total stress and nonlinear effective stress analyses is shown in Fig. 7(a), (b) and (c) respectively. Fig. 7 (a) and (b) shows the PGA of 0.19g except some minor variation in acceleration time history indicates a very good agreement between the equivalent linear and nonlinear total stress analyses. All the analysis reveals amplification of the ground motion with the PGA of about 0.19g. This is due to the fact that the site has relatively higher shear wave velocity and is subjected to low intensity shaking.



(a)



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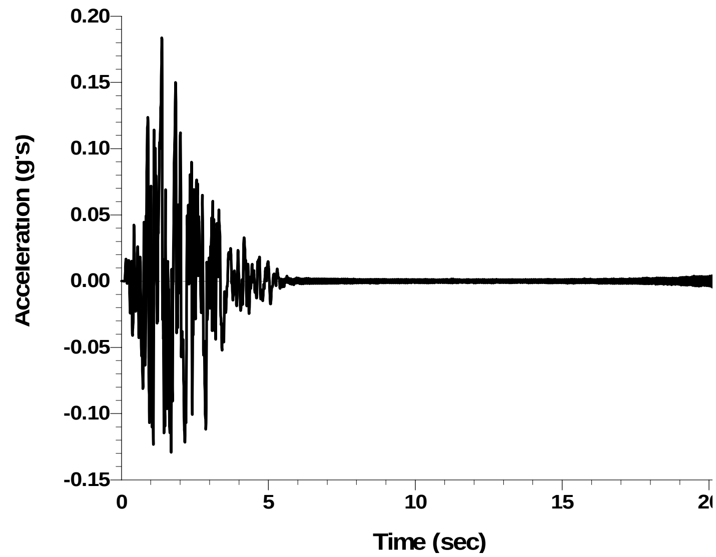
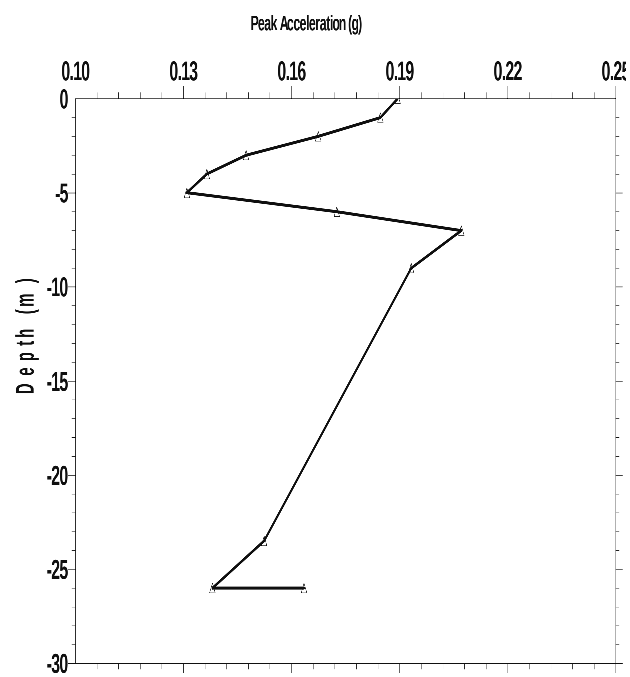


(c)

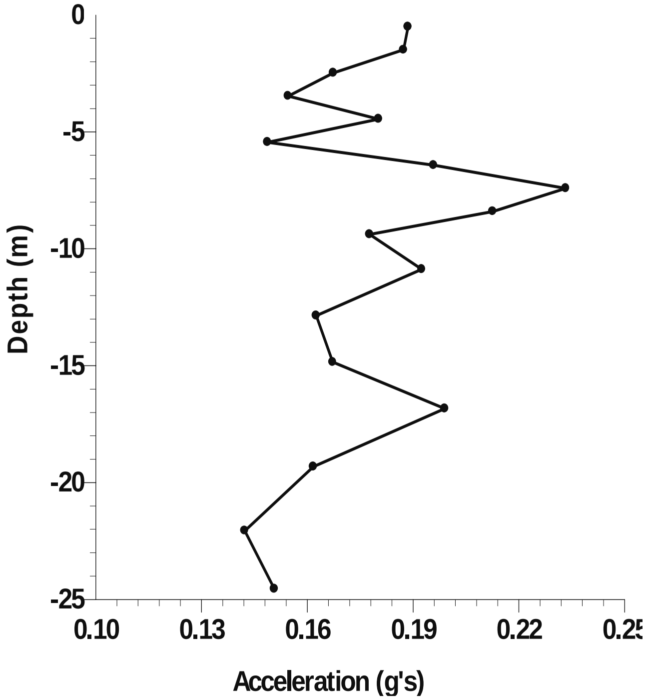
Fig. 7 Surface acceleration from (a) Equivalent linear (b) Nonlinear total stress and (c) Nonlinear effective stress

Variation of Maximum Acceleration with Depth

The variation of maximum acceleration with depth obtained from equivalent linear, nonlinear total stress and nonlinear effective stress analyses are shown in Fig. 8 (a), (b) and (c) respectively. Fig. 8 (a) shows the increase in acceleration due to the impedance contrast between the layers. The results of the total stress analyses as shown in Fig. 8 (a) and (b) indicates that the accelerations which mostly increases monotonically from base to ground surface.

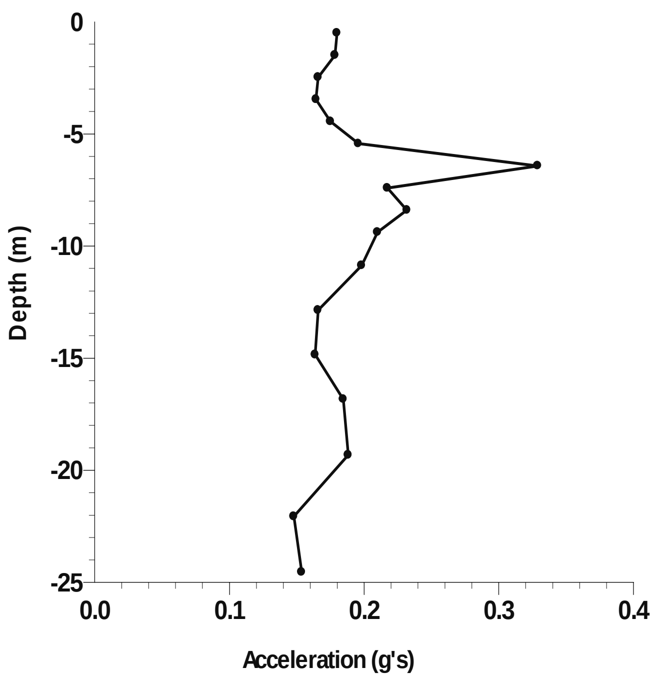


(a)



(b)

But, in the case of effective stress analysis (Fig. 8 c), gives a bulge in the maximum acceleration profile. This bulge usually coincides with the zone of high excess pore pressure ratio. The higher acceleration is observed in the case of effective stress analysis as compared to total stress analysis due to high excess pore pressure ratio.

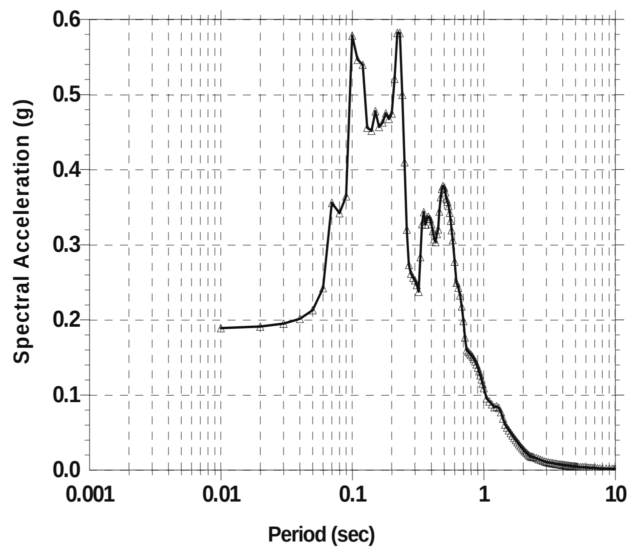


(c)

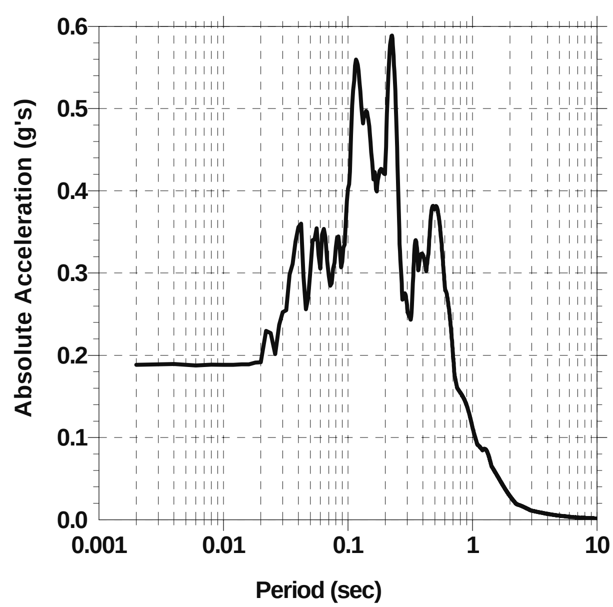
Fig.8. Variation of acceleration with depth from (a) Equivalent linear (b) Nonlinear total stress and (c) Nonlinear effective stress

<u>Response Spectra</u>

The response spectra obtained from equivalent linear, nonlinear total stress and nonlinear effective stress analyses is shown in Fig. 9 (a), (b) and (c) respectively. By comparing Fig. 9 (a) and (b) shows that the multi peaks are observed in the low period range in case of nonlinear total stress analysis which may be due to the higher peak accelerations observed in the acceleration time history in the short period (Fig. 8 b). In the case of equivalent linear analysis the maximum spectral acceleration of 0.58g occurs in the range of 0.1 to 0.2s. In case of nonlinear total stress analysis the maximum spectral acceleration of 0.58g occurs at 0.2s where as for nonlinear effective stress analysis is at 0.1s.



(a)



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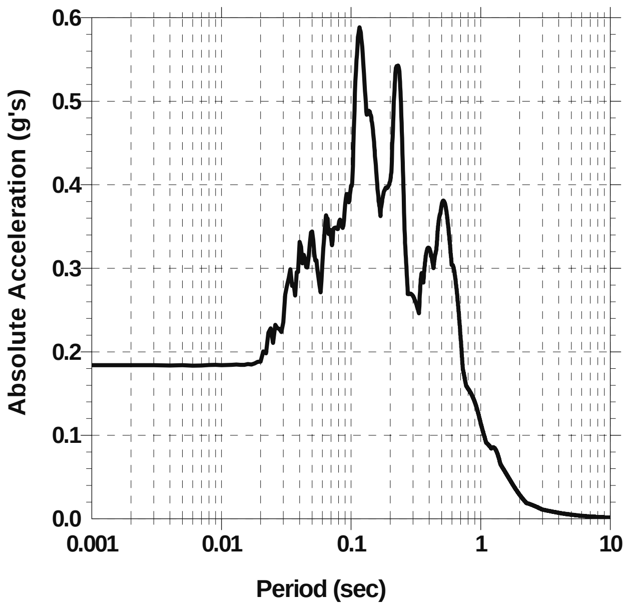


(c)

Fig. 9. Ground response spectra from (a) Equivalent linear (b) Nonlinear total stress and (c) Nonlinear effective stress

Variation of PWP Ratio with Depth

The variation of pore water pressure (PWP) ratio obtained from nonlinear effective stress is shown in Fig. 10. It indicates the maximum PWP ratio of 0.25 at a depth of 6 m from the surface. At this level, the acceleration is also high as seen in Fig. 8 (c).

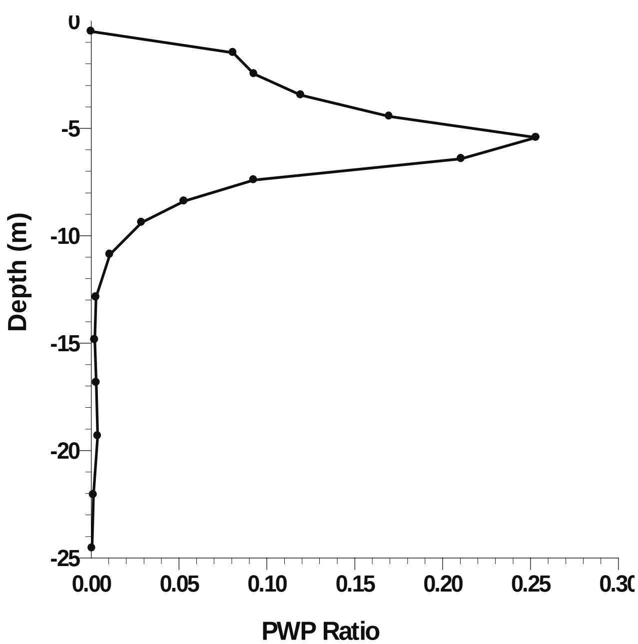
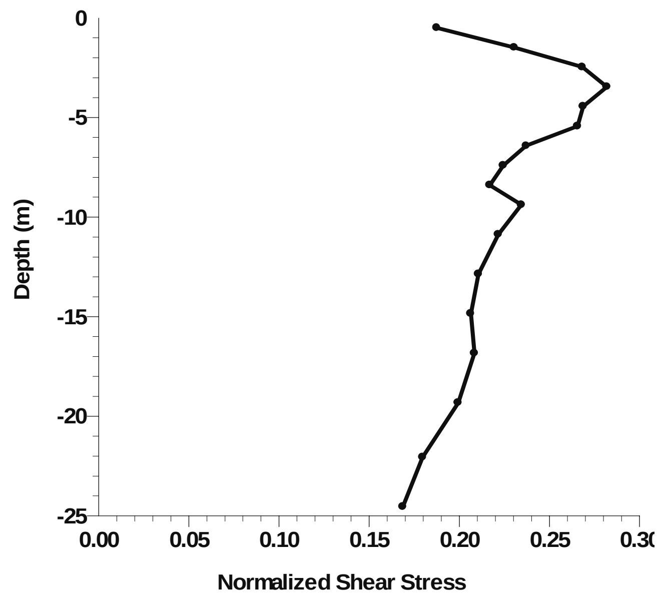


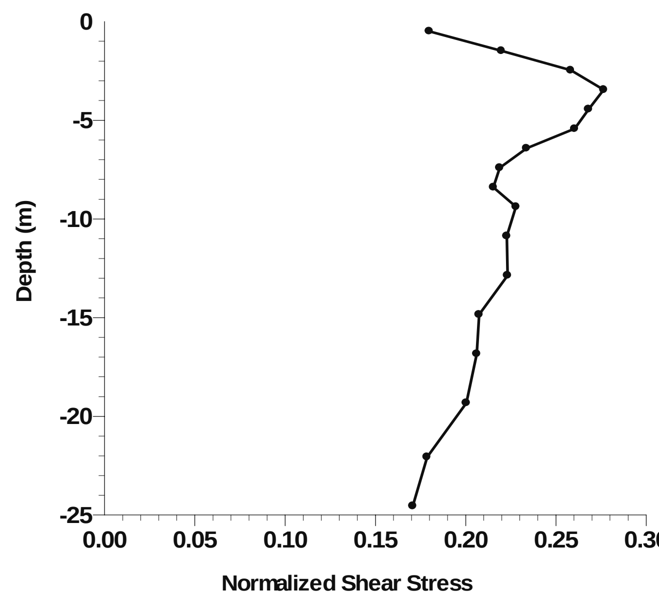
Fig.10. Variation of PWP ratio with depth from nonlinear effective stress

Variation of Maximum Shear Stress and Strain with Depth

The variation of normalized shear stress with depth obtained from nonlinear total stress and nonlinear effective stress analyses is shown in Fig. 11 (a) and (b) respectively. The variation of maximum shear strain with depth obtained from equivalent linear, nonlinear total stress and nonlinear effective stress analyses is shown in Fig. 12 (a), (b) and (c) respectively. The maximum shear stress and strain observed from all the three methods show similar value because of low intensity shaking.

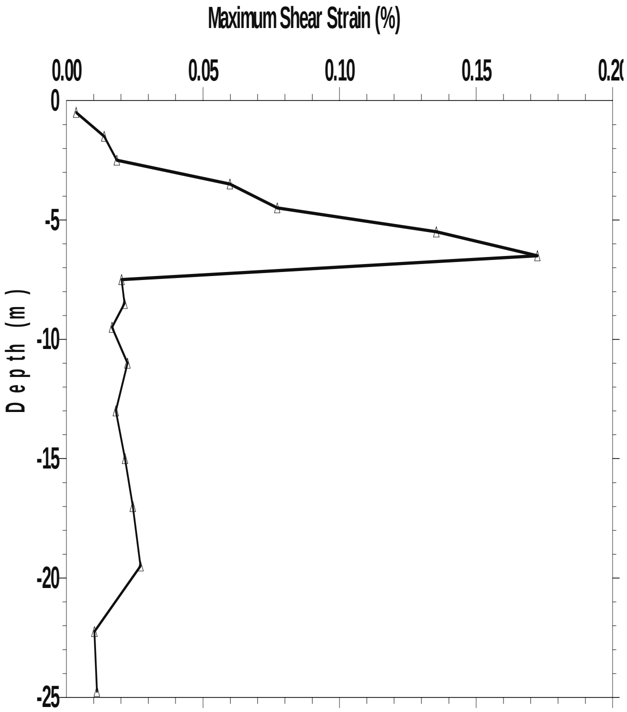


(a)

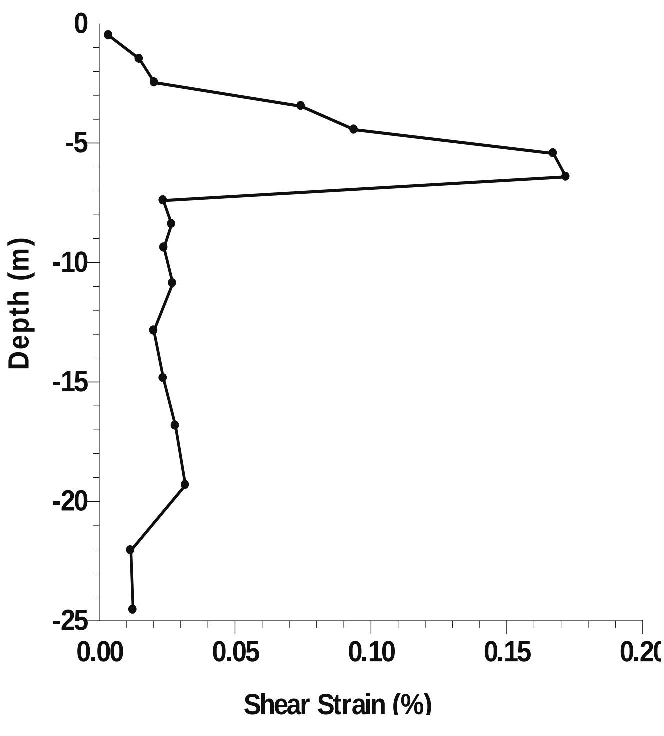


(b)

Fig.11. Variation of normalized shear stress with depth from (a) Nonlinear total stress and (b) Nonlinear effective stress



(a)



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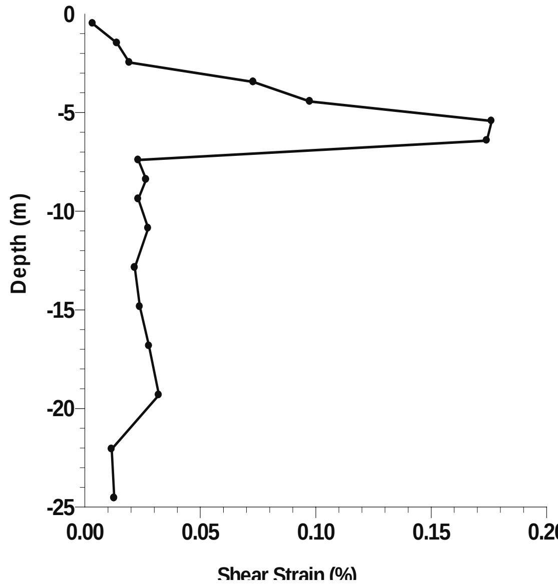
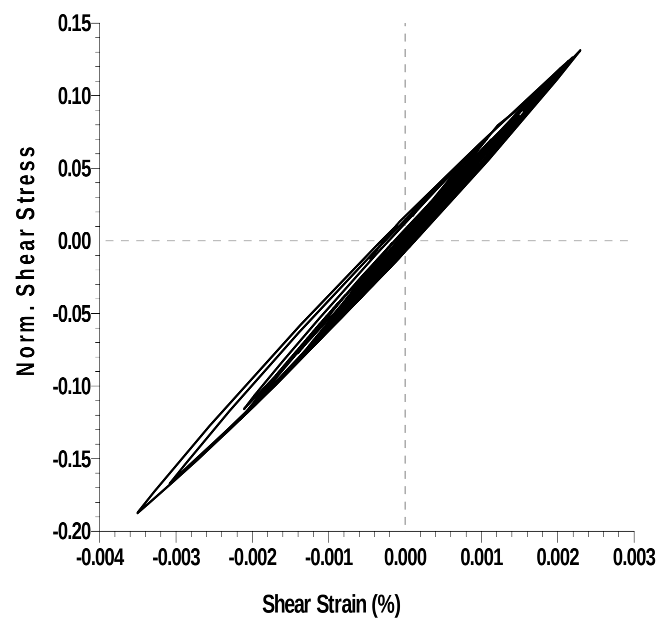


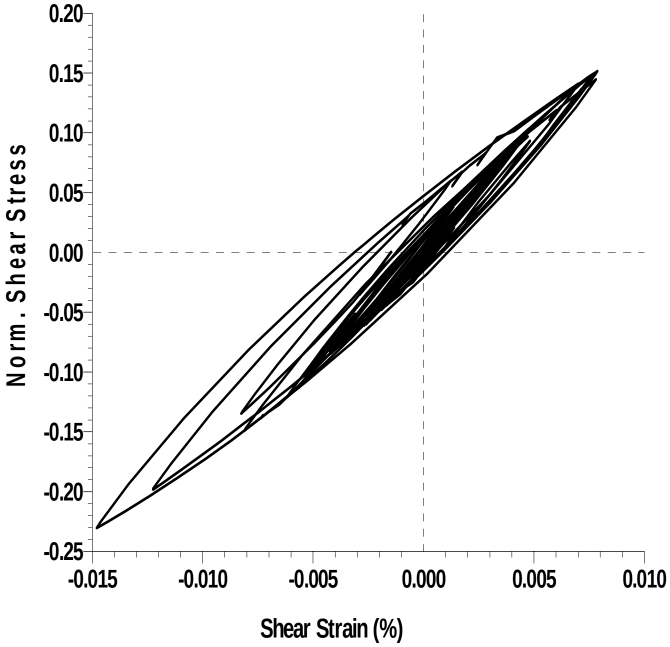
(c)

Fig.12. Variation of shear strain with depth from (a) Equivalent linear (b) Nonlinear total stress and (c) Nonlinear effective stress

The stress strain loops obtained from nonlinear effective stress method at the surface and at the zone of saturated layer is shown in Fig. 13 (a) and (b) respectively. It indicates the increase in shear strain in the zone of saturated layer



(a)



(b)

Fig. 13. Stress strain loops observed at the (a) surface and (b) at the zone of saturated layer

CONCLUSIONS

Seismic response analysis was carried for a sandy soil deposit of 26m thick subjected to input bedrock motion having a PGA of 0.16g by three methods: equivalent linear, nonlinear total stress and nonlinear effective stress analysis. It is observed that all the above methods yield practically the same ground surface PGA and peak spectral acceleration due to low intensity of input motion and relatively higher shear wave velocity of the sandy strata. However, the equivalent linear analysis predicts peak spectral acceleration in the range of 0.1-0.2s, where as nonlinear total and effective stress analyses predict the peak spectral acceleration at 0.2 and 0.1s respectively. The effective stress analysis indicates the occurrence of maximum pore pressure at a depth where the maximum acceleration was encountered. The variation of shear stress and strain with depth for the sand deposit is found to be the same for all the three methods of analyses.

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