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Comparative Study of Equivalent Linear and Non-Linear Ground Response Analysis for Rapar District of Kutch, India

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Abstract**—**Earthquakes are considered to be the most destructive rapid-onset disasters human beings are exposed to. The amount of loss it brings in is sufficient to take careful considerations for designing of structures and facilities. Seismic Hazard Analysis is one such tool which can be used for earthquake resistant design. Ground Response Analysis is one of the most crucial and decisive steps for seismic hazard analysis. Rapar district of Kutch, Gujarat falls in Zone 5 of earthquake zone map of India and thus has high seismicity because of which it is selected for analysis. In total 8 bore-log data were studied at different locations in and around Rapar district. Different soil engineering properties were analyzed and relevant empirical correlations were used to calculate maximum shear modulus (Gmax) and shear wave velocity (Vs) for the soil layers. The soil was modeled using Pressure-Dependent Modified Kodner Zelasko (MKZ) model and the reference curve used for fitting was Seed and Idriss (1970) for sand and Darendeli (2001) for clay. Both Equivalent linear (EL), as well as Non-linear (NL) ground response analysis, has been carried out with Masing Hysteretic Re/Unloading formulation for comparison. Commercially available DEEPSOIL v. 7.0 software is used for this analysis. In this study an attempt is made to quantify ground response regarding generated acceleration time-history at top of the soil column, Response spectra calculation at 5 % damping and Fourier amplitude spectrum calculation. Moreover, the variation of Peak Ground Acceleration (PGA), Maximum Displacement, Maximum Strain (in %), Maximum Stress Ratio, Mobilized Shear Stress with depth is also calculated. From the study, PGA values estimated in rocky strata are nearly same as bedrock motion and marginal amplification is observed in sandy silt and silty clays by both analyses. The NL analysis gives conservative results of maximum displacement as compared to EL analysis. Maximum strain predicted by both studies is very close to each other. And overall NL analysis is more efficient and realistic because it follows the actual hyperbolic stress-strain relationship, considers stiffness degradation and mobilizes stresses generated due to pore water pressure.

Keywords**—**DEEPSOIL v 7.0, Ground Response Analysis, Pressure-Dependent Modified KodnerZelasko (MKZ) model, Response Spectra, Shear wave velocity.

I. Introduction

PERNICIOUS repercussions of earthquakes have been widely known for centuries but the hefty role of soils to the damage pattern and magnitude was not widely appreciated until recent. Ground response analysis is the process of calculating the response of a soil deposit to an earthquake in the absence of structures; i.e. calculating the free-field response. It is a nonlinear three-dimensional wave-propagation problem involving the excitation of a soil deposit by a wave-field comprising of body and surface waves. However, in practise, ground response analysis is perpetually computed using one-dimensional methods which are based on mainly three assumptions. 1) soil profiles are composed of horizontal soil layers bunched up on top of each other 2) along a given horizontal plane the soil layers are homogenous and 3) the ground motion incident at the bottom of soil deposit is comprised of vertically propagating shear waves [1]. These assumptions simplify the site response phenomenon to a one-dimensional problem and have enabled the use of simplified numerical methods for analysis. The concept of one-dimensional ground response analysis encompasses the excitation of a soil column using the higher horizontal component of the earthquake ground motion and computing the response of individual soil layers. Input of a rock outcrop motion at the soil-bedrock interface is used in this type of upward vertical wave propagation analysis. The bedrock is then replaced by a transmitting boundary to enable the emission of outgoing waves. The damage extent of a given earthquake depends on the properties of soils through which it is passing. Thus, performing an extensive Ground response analysis study for any site becomes an essential task in characterizing the site.

Identification and analysis of the soil engineering properties is required for dynamic modelling of soil. This requires obtaining the lateral soil profile in the form of bore log data and quantifying its properties such as type of sample, thickness of strata, soil type, SPT-N value recorded, plasticity index(PI), unconfined compressive strength(S_u), bulk and dry density, effective confining stress, angle of internal friction etc. layer by layer. These properties are then correlated with shear modulus (G) and shear wave velocity (V_S) using empirical equations developed for different types of soils. Once the values of dynamic soil parameters are obtained a suitable soil model can be adopted to perform the analysis. Finally, reference curves which simulate modulus reduction and damping as function of strain are chosen. Now, this analysis can be performed by two approaches, equivalent linear (EL) and nonlinear (NL). Both of these methods have different types of assumptions and methodologies for carrying out the analysis. The NL method is considered more realistic as it follows the actual hysteretic

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stress-strain relationship, considers stiffness degradation in given cycle, allows for the application of pore-water dissipation models and has concepts of residual strain and shear strength at which failure occurs [1]. These are not quite present in EL method but still, it gives fairly close and accurate results as of NL method for given strain range and shear stress values. In [1] 1-D ground response analysis was performed using both the methods for the borehole sites in Sikkim. Variation of peak ground acceleration (PGA) with depth for low (0.02g and 0.06g) amplitude and high (0.18g and 0.36g) amplitude strong ground motion was studied. They observed that for low amplitude motion the difference between the PGA values computed by EL and NL approaches is quite less but for high amplitude motion, a considerable difference between the PGA values was obtained with EL analysis resulting in much higher values as compared to NL analysis. Rapar district of Kutch falls in zone V classification of the seismic map of India with high possibility of getting a major earthquake of the order of magnitude 7.5-7.8. In order to account for the degree of variation in parameters such as PGA, maximum displacement, maximum strain (%), mobilized shear stress and spectral acceleration values for response spectrum a comparative study is put forward by evaluating both the methods.

II. Types of Analysis

A Equivalent Linear Method

The EL method involves the calculation of an approximate nonlinear response using a linear analysis with the soil properties adjusted to account for displacement during earthquake shaking. An iterative process is used to execute a series of linear analysis for calculating the layer properties of soil which can be carried out either in frequency or time domain. Using initial values of shear modulus and damping ratio, a linear analysis is started to compute peak strains in the soil layers. An effective shear strain is computed for each layer by multiplying the peak shear strain by an effective shear strain ratio. Along with modulus reduction and damping curves, this calculated strain value is used to update the shear modulus and damping ratio of given cycle for each layer. Computed new values are then used to perform next iteration and this process is continued till the shear strains from consecutive analysis fall within a predefined tolerance. Now, the loading conditions occurring during the earthquake needs to be related to the laboratory tests (that are used to calculate modulus reduction and damping curves) for which a parameter known as effective shear strain ratio is used. The SHAKE user's manual recommends a value of 0.65, which has traditionally been used in practice [2].

B Non-Linear Method

NL method simulates the hysteretic stress-strain response of the soil and is therefore potentially more realistic than EL method. Generally two approaches are used while modelling a soil profile by NL method; 1) lumped-mass approach and 2) finite element based approach. In the former, the soil layers are bunched into adjacent nodal masses, which are joined together by springs that model the soil stress- strain behaviour in shear. Dynamic equations of motion are then mathematically solved to get the response of the soil layer subjected to input motion at the bottom of soil column. In finite element approach the principle used for modelling is of solid elements stacked on top of each other and restrained to move only in shear. One-dimensional nonlinear material models are typically characterized by the backbone curve and a set of rules that govern the hysteresis path under an irregular cyclic loading [2]. The most well-known set of hysteresis rules are the extended Masing rules, which are used in DEEPSOIL. Because they define the shape of the loops, the hysteresis rules significantly affect the nonlinear response and especially hysteretic damping.

III. Setting up of Database

Rapar district of Kutch has high possibility of getting earthquake from the hypocentre of previously occurred Bhuj earthquake and the South Wagad fault (SWF) which passes through its vicinity. Now for given earthquake parameters such as plate regimes, moment magnitude, hypocentral distance, fault type, etc. a real accelerogram data needs to be found out from previously occurred earthquakes in order to match our site-specific parameters. This actual acceleration time history can then be used to predict the ground response after passing it through soil column. Details about Bhuj earthquake as recorded by United States Geological Survey (USGS) [3], [4] are; Magnitude: $7.7M_w$, Location: 23.420^oN, 70.230^oE, Depth of hypocentre: 16 km, Type of fault: Strike slip + Reverse. The co-ordinates of Rapar district are 23.5730^oN and 70.6447^oE. So using the principles of great trigonometric circle the epicentral and hypocentral distance are 45.58 km and 48.309 km respectively.

The web-based Pacific Earthquake Engineering Research Centre (PEER) [5] ground motion database was used to download unscaled recorded acceleration-time histories along with horizontal and vertical response spectra at 5% damping. The Chi-Chi earthquake of Taiwan in 1999 most closely resembles the earthquake parameters of Bhuj earthquake with magnitude of 7.62, depth of hypocentre: 30 km and fault type being strike slip + reverse. Therefore data of recording stations of chi-chi earthquake with hypocentral distance in range of 35-55 kms were chosen. For BH-1 and BH-2 Vs30 is 377 ms-1 and 365 ms-1 respectively so input motion of RSN 1479 station east component (with Vs30= 380 ms-1) is used and for BH-3, BH-4, BH-5 Vs30 is 423 ms-1, 413 ms-1 and 410 ms-1 respectively so input motion of RSN 1478 station east component (with Vs30= 420 ms-1) is used. The input acceleration time history for all the boreholes is as shown in Fig. 1.

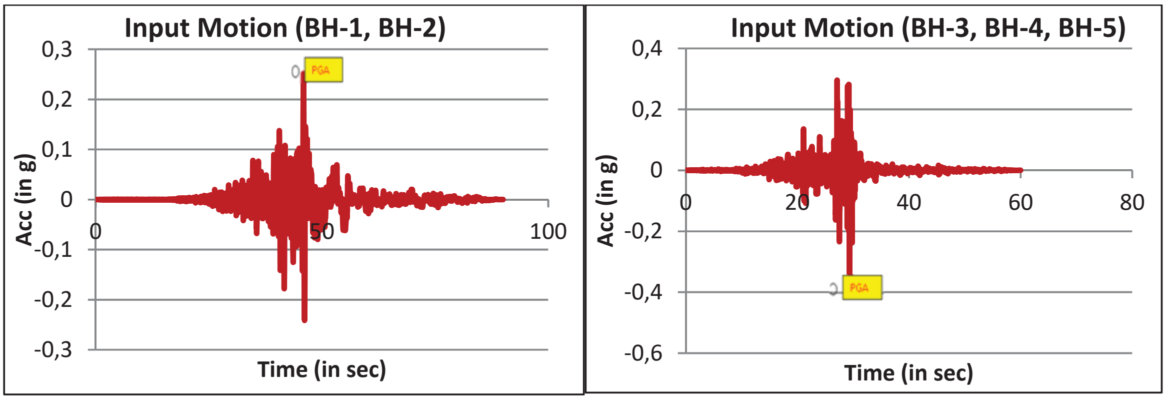

Fig. 1 Input motion for Boreholes

IV. Bore Log Data Interpretation

In total five bore log data were interpreted for ground response analysis out of which four were from Gagodhar sub-branch canal data and one was from school building in old school compound at Rapar. Total depth for exploration was 15 m for Gagodhar sub-branch canal data and 10 m for old school compound data. The location marked on Google maps is as shown in Fig. 2. Two boreholes were dug at chainage 53.769 km with 20 m spacing and latitude of 23°23′21"and longitude of 70°47′10", two boreholes were dug at chainage of 57.769 km with 20 m spacing and latitude of 23°20′32"and longitude of 70°44′32" and one borehole was dug at school compound with latitude of 23°34′12"and longitude of 70°38′24".

After interpreting the bore logs of Gagodhar sub- branch canal, lateral soil profiles were generated showing the stratification of layers where there is change in property or type of soil. Auto-CAD drawings at both the chainages are shown in Fig. 3. The reduced levels are marked on the straight depth of borehole exploration so that thickness of each layer can be easily understood. The soil classification is done as per IS 1498 (1970) and is also shown in Fig. 3.

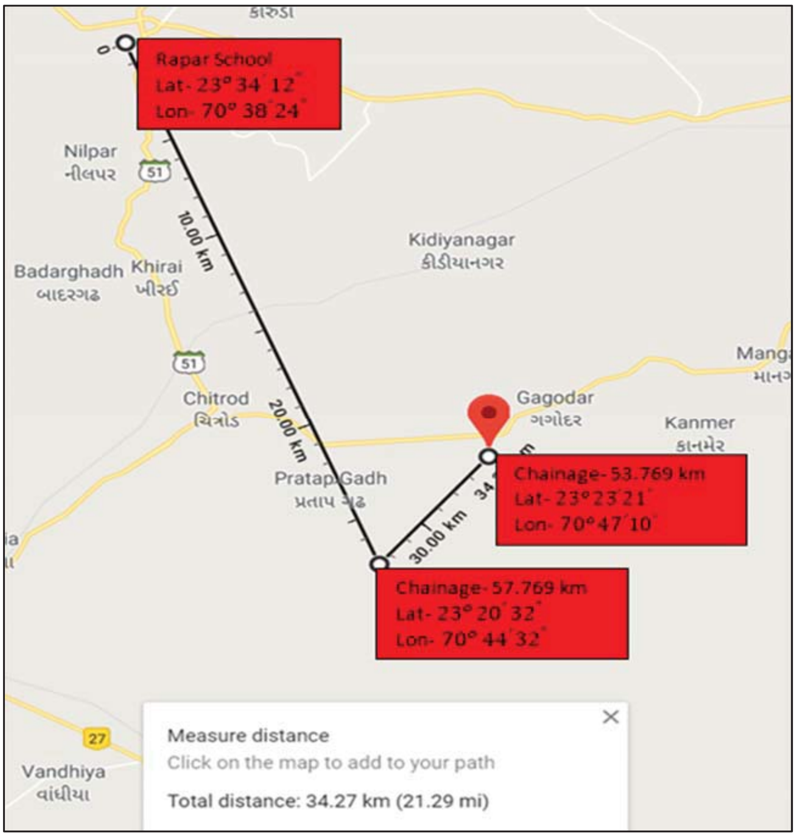

Fig. 2 Location of Boreholes

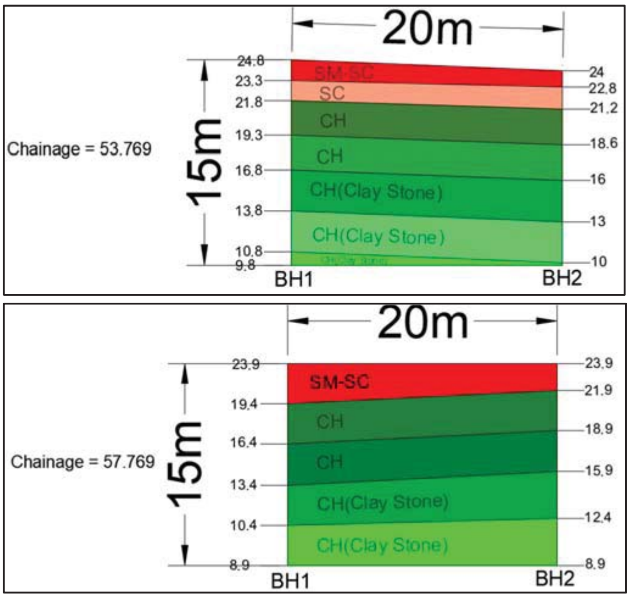

Fig. 3 Lateral Soil Profile

From the litho log, for chainages 53.769 km and 57.769 km it can be clearly seen that top 2-3 m of soil is of type sandy silt and silty clay. For next 5-6 m dark brownish yellow coloured clay of high plasticity is found with SPT- N value in the range of 35-50. Finally for next 6-7 m compacted clay stone recovered as clay core of high plasticity is found with SPT- N value greater than 50 suggesting hard rock.

For Rapar School, top 3 m of soil comprise of silty sand with SPT- N value of 13-20 and next 3 m is also of silty sand but SPT- N value ranges from 40-50 suggesting higher penetration resistance due to increasing overburden pressure. Finally next 5-7 m is made up of rock with SPT- N value greater than 50. Water table is not present in top 15 m of the soil column in all the bore logs investigated.

V. G_{max} AND V_s CALCULATIONS

Existing methods such as cross-hole or down-hole testing can be used to measure the in-situ shear wave velocity. But these techniques include usage of geophones, down-hole shear wave hammer, wooden plank weighted vehicle, seismograms etc. which are out of the scope of this project and moreover data analysis and interpretation of the recorded results also requires fine technical know how about the methods.

To overcome these problems, empirically derived shear wave velocities can be used for preliminary assessment and calculation of response spectra and ground response analysis.

One of the widely used in situ parameter for estimation of shear wave velocity is standard penetration number(N) and a variety of empirical relations taking into consideration factors such as effective confining stress, coefficient of earth pressure at rest, type of soil along with corrected SPT- N value have been developed. Empirical relations by Wair et al. [6] are commonly used as they are the most recent and encapsulates mostly all the parameters that affect the shear wave velocity. They are as follows:

$$V_s = 30\,(N_{60})^{0.215}(\sigma_v)^{0.275}\text{ASF [Allsoils]} \tag{1}$$

$$V_s = 26\,(N_{60})^{0.17}(\sigma_v)^{0.32}\text{ ASF [Clays and Silts]} \tag{2}$$

$$V_s = 30\,(N_{60})^{0.23}(\sigma_v)^{0.23}\text{ ASF [Sands]} \tag{3}$$

$$V_s = 115(N_{60})^{0.17}(\sigma_v)^{0.12}\text{ ASF [Gravels]} \tag{4}$$

where, V_s: Shear wave velocity in m/s, N_{60}: Corrected N-value for 60 % hammer efficiency, σ_v: Effective vertical stress in kN/m^2, ASF: Age scaling factor (generally taken as 1).

The equipments used in our testing apparatus are rope and pulley type of safety hammer, sampler with liner of clay, borehole diameter of 150 mm and accordingly the correction factors are applied as per Skempton's SPT-N value corrections [7]for N_{60} calculation. From 1-D wave propagation theory,

$$V_s = \sqrt{\frac{G}{\rho}} \tag{5}$$

where, G: Shear modulus in kN/m^2, ρ: Density of soil layer in gm/cm^3, V_s and G_{max} for all the boreholes at chainage of 53.769 km, 57.769 km and School compound are calculated and shown in Tables I-V, respectively.

VI. SOIL MODEL AND REFERENCE CURVES

Some important aspects of soil behaviour such as building up of pore water pressure, anisotropy, dilation etc. are very accurately captured by using advanced constitutive models.

TABLE I
G_{max} AND V_s CALCULATIONS FOR CHAINAGE 53.769 KM [BH-1]

Chainage	Bore hole no	Soil Type	H (m) Thickness	N (observed)	N (corrected)	σ kN/m^2	PI	Unit wt kN/m^3	Density gm/cm^3	K_o	σ_m	G_{max} kN/m^2	V_s ms^{-1}	V_{s30} ms^{-1}
53.769 km	1	SC	1.50	~	~	13.65	15	~	~	~	~	9539.17	72.40	377.13
		SC	1.50	3	2.1	40.95	17	18.2	1.82	0.66	0.32	17455.32	97.93	
		CH	2.50	30	24.1	77.98	44	18.7	1.87	0.76	0.65	72595.14	197.03	
		CH	2.50	38	34.1	125.48	46	19.3	1.93	0.74	1.04	113023.8	241.99	
		CH	3.00	>50	100.0	179.45	46	19.9	1.99	0.72	1.46	225301.7	336.48	
		CH	3.00	>50	100.0	239.30	47	20.0	2.00	0.74	1.98	265271.7	364.19	
		CH	1.00	>50	100.0	279.45	41	20.3	2.03	0.72	2.28	293228.2	380.06	
		CH	15.0	>50	100.0	469.60		24.0	2.40			1176000	700.00	

TABLE II
G_{max} AND V_s CALCULATIONS FOR CHAINAGE 53.769 KM [BH-2]

Chainage	Bore hole no	Soil Type	H (m) Thickness	N (observed)	N (corrected)	σ kN/m^2	PI	Unit wt kN/m^3	Density gm/cm^3	K_o	$σ_m$	G_{max} kN/m^2	V_s ms^{-1}	V_{s30} ms^{-1}
53.769 km	2	SM	1.20	~	~	10.74	NP	~	~	~	~	6907.21	62.12	363.15
		SC	1.60	2	1.4	35.80	12	17.9	1.79	0.59	0.26	13393.25	86.50	
		CH	2.60	38	30.5	74.69	42	18.9	1.89	0.76	0.63	79321.59	204.86	
		CH	2.60	48	43.1	124.48	44	19.4	1.94	0.75	1.04	125066.0	253.90	
		CH	3.00	>50	100.0	180.15	44	20.3	2.03	0.74	1.49	230323.0	336.84	
		CH	3.00	>50	100.0	241.05	43	20.3	2.03	0.74	1.99	270332.0	364.92	
		CH	1.00	>50	100.0	281.75	41	20.5	2.05	0.72	2.28	297455.1	380.92	
		CH	15.0	>50	100.0	465.85		24.0	2.40			1176000	700.00	

TABLE III
G_{max} AND V_s CALCULATIONS FOR CHAINAGE 57.769 KM [BH-1]

Chainage	Bore hole no	Soil Type	H	N (observed)	N (corrected)	σ kN/m^2	PI	Unit wt kN/m^3	Density gm/cm^3	K_o	$σ_m$	G_{max} kN/m^2	V_s ms^{-1}	V_{s30} ms^{-1}
57.769km	1	SM-SC	1.5	~	~	13.13	5	~	~	~	~	7540.33	65.64	423.36
		SM-SC	1.5	2	1.4	39.38	7	17.5	1.75	0.55	0.27	13797.70	88.79	
		CH	3.0	28	22.5	80.85	40	18.9	1.89	0.72	0.66	72659.45	196.07	
		CH	3.0	38	34.1	138.75	42	19.7	1.97	0.76	1.16	121927.2	248.78	
		CH	3.0	44	41.6	198.45	43	20.1	2.01	0.72	1.62	164917.6	286.44	
		CH	3.0	>50	100.0	259.35	40	20.5	2.05	0.71	2.09	284206.3	372.34	
		CH	15	>50	100.0	470.10		24.0	2.40			1176000	700.00	

TABLE IV
G_{max} AND V_s CALCULATIONS FOR CHAINAGE 57.769 KM [BH-2]

Chainage	Bore hole no	Soil Type	H	N (observed)	N (corrected)	σ kN/m^2	PI	Unit wt kN/m^3	Density gm/cm^3	K_o	$σ_m$	G_{max} kN/m^2	V_s ms^{-1}	V_{s30} ms^{-1}
57.769 km	2	SM-SC	2.0	~	~	18.50	5	~	~	~	~	11163.53	77.68	413.60
		CH	3.0	28	22.5	64.75	40	18.5	1.85	0.74	0.54	62944.80	184.46	
		CH	3.0	38	34.1	121.00	41	19.0	1.90	0.76	1.01	109066.6	239.59	
		CH	3.5	38	35.9	184.15	45	19.8	1.98	0.79	1.59	146384.3	271.90	
		CH	3.5	>50	100.0	254.68	37	20.5	2.05	0.71	2.05	281377.1	370.48	
		CH	15	>50	100.0	470.55		24.0	2.40			1176000	700.00	

TABLE V
G_{max} AND V_s CALCULATIONS FOR BH RAPAR SCHOOL

Chainage	Bore hole no	Soil Type	H	N (observed)	N (corrected)	σ kN/m^2	PI	Unit wt kN/m^3	Density	K_o	$σ_m$	G_{max} kN/m^2	V_s ms^{-1}	V_{s30} ms^{-1}
Rapar School Building	1	SM	1.0	13	8.8	9.25	10	18.5	1.85	0.61	0.07	14400.86	88.23	410.76
		SM	1.5	20	13.5	32.38	12	18.5	1.85	0.61	0.24	34520.19	136.60	
		SM	1.5	42	28.4	60.35	12	18.8	1.88	0.61	0.45	67978.36	190.15	
		SM	1.5	46	35.2	88.70	12	19.0	1.90	0.59	0.65	93178.21	221.45	
		Rock	1.5	100	85.5	120.95	~	24.0	2.40	1.00	1.21	752640.0	560.00	
		Rock	1.5	100	85.5	156.95	~	24.0	2.40	1.00	1.57	864000.0	600.00	
		Rock	1.5	100	90.0	192.95	~	24.0	2.40	1.00	1.93	1014000	650.00	
		Rock	20	100	100	450.95		24.0	2.40			1176000	700.00	

The use of these soil constitutive models is, however, only appropriate when detailed information on soil behaviour is available. However, for majority of applications the only information available is about the modulus reduction and damping curves. Therefore, Pressure dependent Modified Konder and Zelasko hyperbolic model (MKZ) [8]which defines the stress-strain relationship for loading and unloading conditions is used.

When a typical soil is subjected to cyclic loading, it will undergo hysteretic stress-strain relationship. This relationship can be effectively illustrated in two ways; first by following the actual path of loop itself, and second by parameters that conform to its general shape. The two important characteristics by which the shape of this hysteresis loop can be described are its inclination and breadth. The nature of inclination of the loop is contingent on the stifness of the soil, which at any point during the loading process can be described by the secant shear modulus (G) and the breadth of the loop is related closely to its area, which being a measure of energy dissipation can appropriately be described by the damping ratio (λ) [9]. Standard curves are developed by different researchers by doing laboratory tests on a number of specimens of different soil types and with variability in parameters such as effective confining pressure, plasticity index, void ratio, relative density, number of loading cycles and frequency, over consolidation ratio etc which could directly be used. Darendeli (2001) [10] reference curves are used in this study as tested samples ranged from broad intervals of sampling from sands to clays and with depth of 3-263 m. Confining pressure was also varied from 0.3-27.2 atm, plasticity index from 0-132% and over consolidation ratio from 1-8 which is one of the most exhaustive and well known studies till date. Commercially available software DEEPSOIL v 7.0 [11] was used to perform both equivalent linear and non-linear analysis with the standard real time analysis control parameters. MKZ soil model with default hysteretic formulation by Masing Re/Unloading is applied and the units system chosen is metric. Input motion of RSN 1479 and RSN 1478 station east component for chi-chi earthquake as mentioned above is chosen for analysis. Finally results in the form of time-history plots, stress strain plots, spectral plots, profile plots, response spectra and mobilised strength are calculated for given input motion for both EL and NL analysis.

VII. Comparative Study

A. PGA

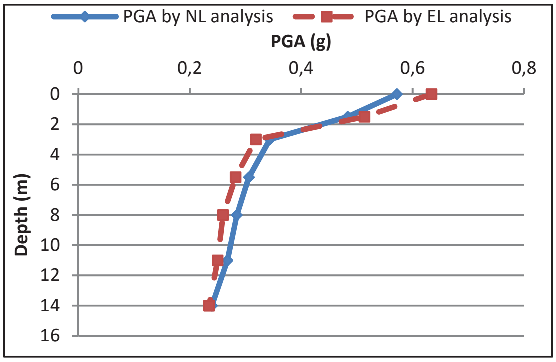

Fig. 4 PGA with Depth of soil column BH-1 CH-53.769 km

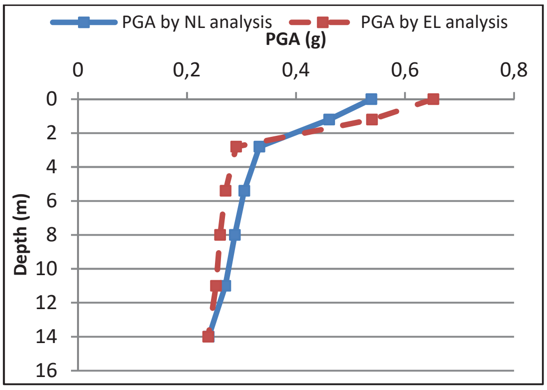

Fig. 5 PGA with Depth of soil column BH-2 CH-53.769 km

It is observed that the PGA is increasing as we move up the soil layers from bedrock towards the ground surface. This is as per the required trends because site amplification of bed-rock motion i.e. ≈ 0.27g (for BH-1 and BH-2) and ≈0.4g (for BH-3, BH-4 and BH-5) takes place due to the presence of soil column. For a depth of 3 to 15 m below the ground surface, the PGA calculated by both EL and NL analysis are in close proximity for all the bore logs modelled. For the top 3 m soil layer which mainly comprises of silty sands and clayey sands the NL analysis predicts PGA of 0.55g (for BH-1 and BH-2) and 0.4g-0.5g (for BH-3, BH-4 and BH-5) whereas the EL analysis predicts PGA of 0.65g-0.7g for all the bore logs. It is evident that an overestimation of ≈0.1g is made by EL analysis as compared to NL analysis.

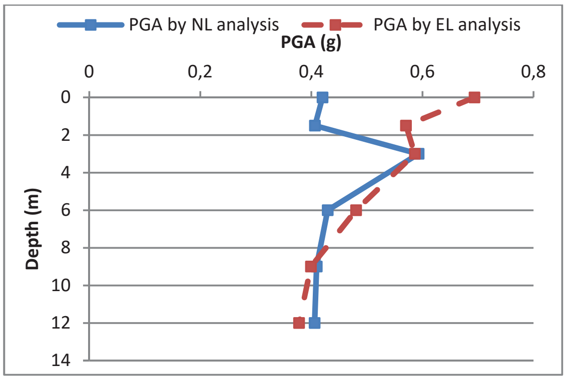

Fig. 6 PGA with Depth of soil column BH-1 CH-57.769 km

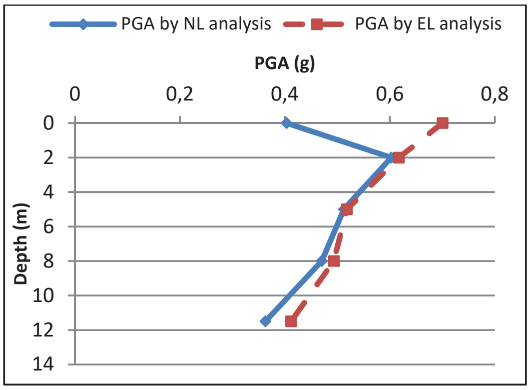

Fig. 7 PGA with Depth of soil column BH-2 CH-57.769 km

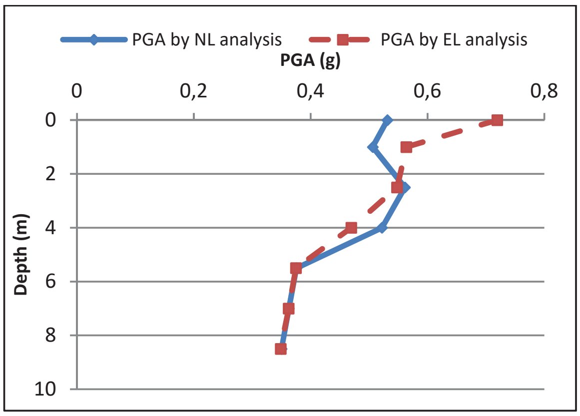

Fig. 8 PGA with Depth of soil column BH Rapar School

B. Maximum Displacement

It can be observed from almost all the bore log data that the upper layers (2-3 m below ground surface) which are of SM, SC or SM-SC have undergone a markable displacement from their mean position of up to 30-35 cm (for BH-1, BH-2 and BH-3), ≈25 cm for BH-4 and ≈10 cm for BH-5 by NL analysis. The EL analysis predictions are also close, with displacement values in these layers ranging from 25-30 cm (for BH-1, BH-2, BH-3 and BH-4) and ≈11 cm for BH-5. The lower layers that are of clay stone with high plasticity have not undergone much displacement (less than 10 cm from mean position) in total.

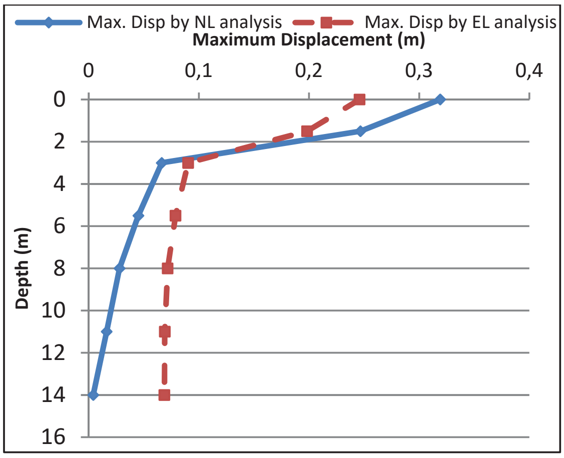

Fig. 9 Maximum displacement with Depth of soil column BH-1 CH-53.769 km

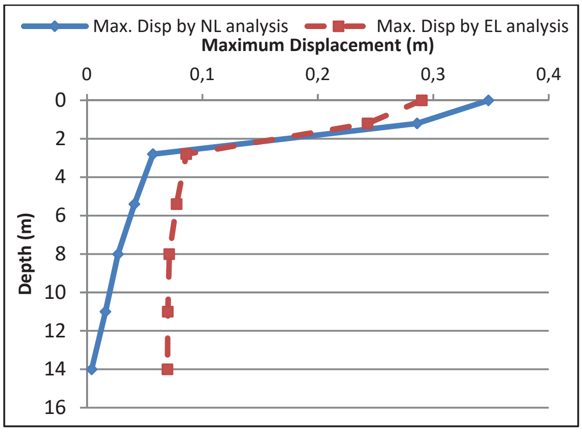

Fig. 10 Maximum displacement with Depth of soil column BH-2 CH-53.769 km

The maximum strain recorded for layers below 4 m of ground surface (for BH-1 BH-2, BH-3 and BH-4) and 6 m of ground surface for BH-5 is very small of about 0.1 % only (i.e. 0.001 m or 0.1 cm). This is because they comprise of clay stone with modulus of rigidity in the order of 0.4-1.6 GPa which is very high and therefore do not undergo much shear strain. Whereas the top layers are made of SM-SC layers which are elastic as compared to rocks and have undergone shear strains of about 0.5-1.5 % (for BH-1, BH-2, BH-3 and BH-4) and 0.2-0.4% for BH-5. The EL and NL analysis are quite close in estimation of maximum shear strain as can be seen from the charts.

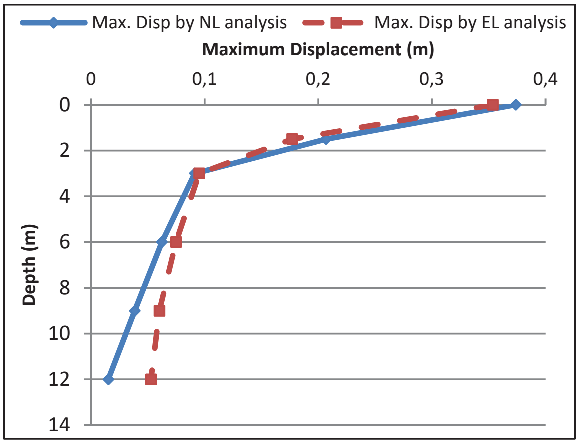

Fig. 11 Maximum displacement with Depth of soil column BH-1 CH-57.769 km

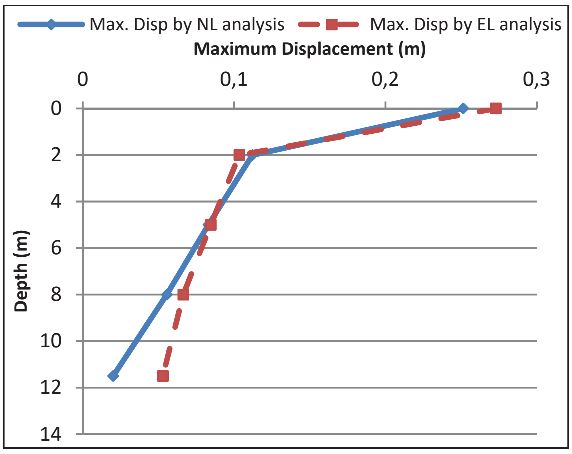

Fig. 12 Maximum displacement with Depth of soil column BH-2 CH-57.769 km

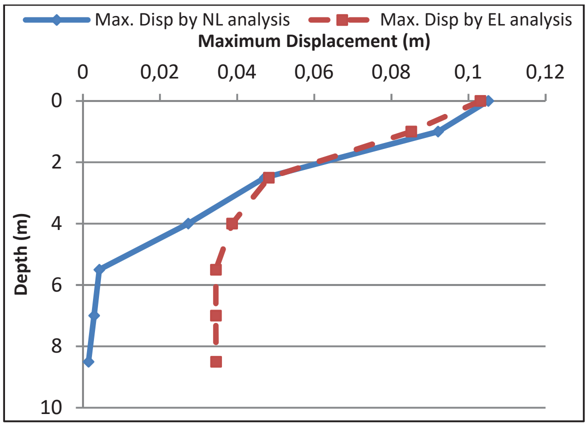

Fig. 13 Maximum displacement with Depth of soil column BH Rapar School

C. Maximum Strain

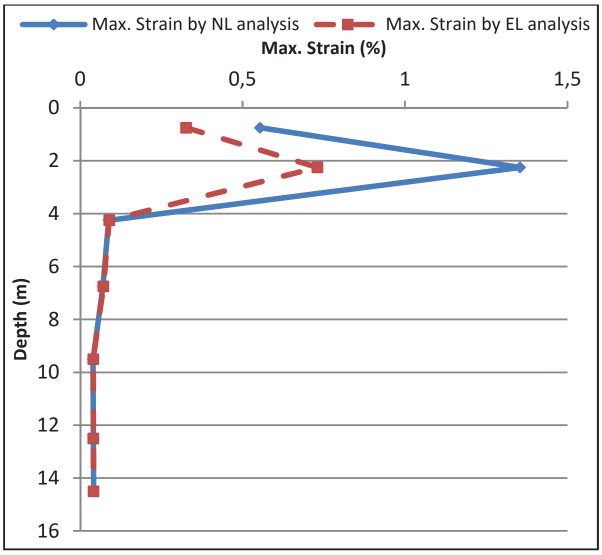

Fig. 14 Maximum Strain with Depth of soil column BH-1 CH-53.769

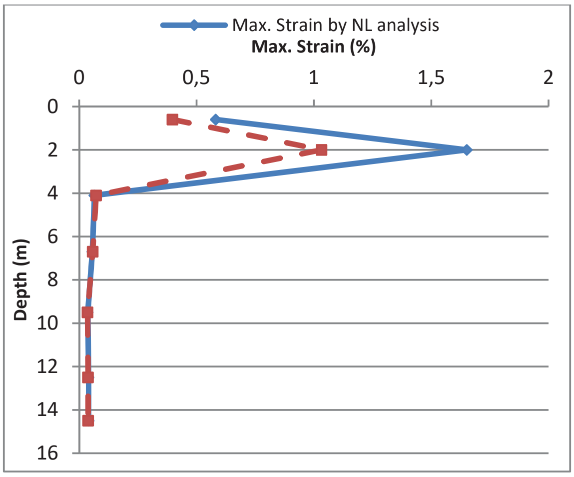

Fig. 15 Maximum Strain with Depth of soil column BH-2 CH-53.769

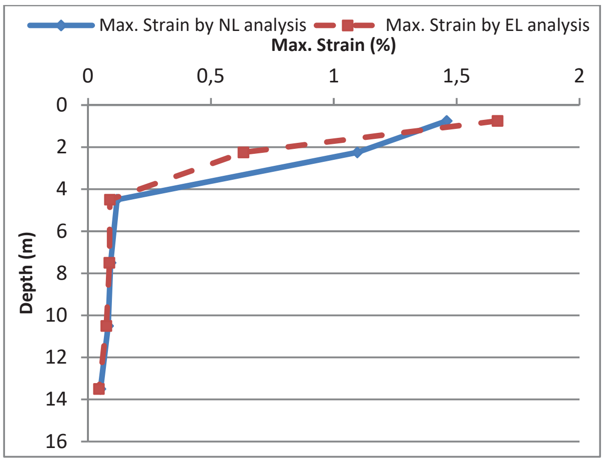

Fig. 16 Maximum Strain with Depth of soil column BH-1 CH-57.769

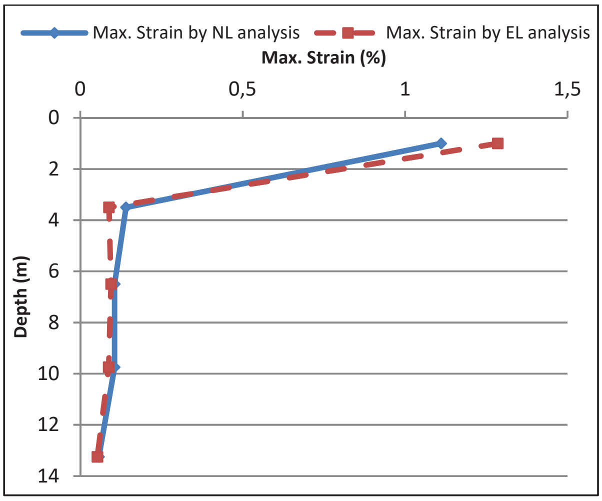

Fig. 17 Maximum Strain with Depth of soil column BH-2 CH-57.769

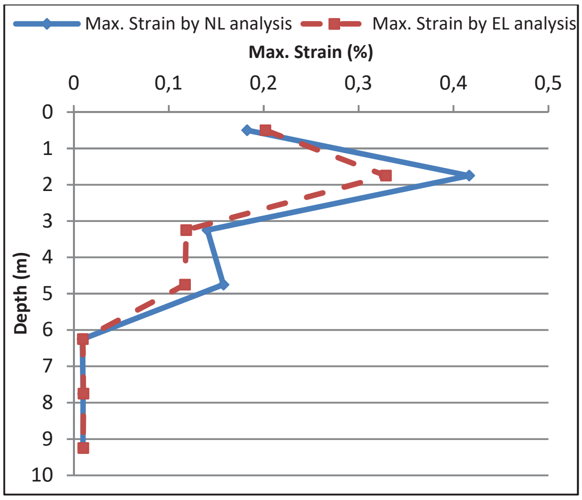

Fig. 18 Maximum Strain with Depth of soil column BH Rapar School

D. Response Spectrum

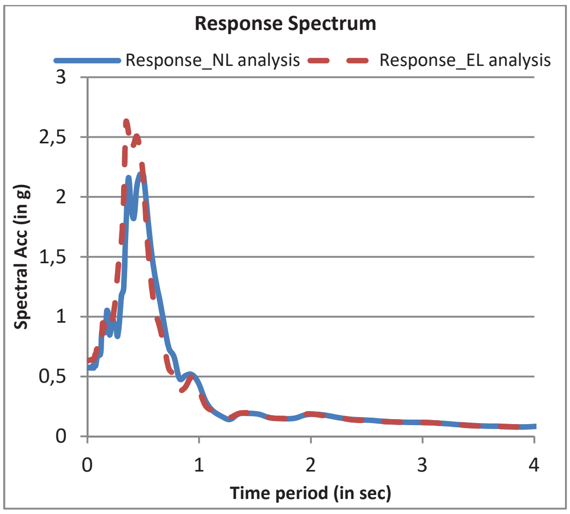

Fig. 19 Response spectrum for BH-1 CH-53.769 km

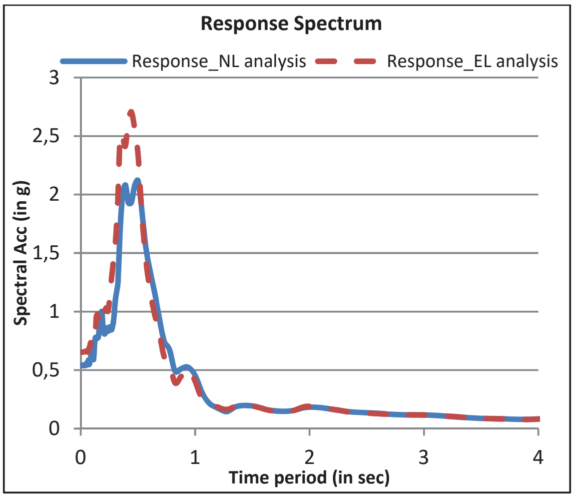

Fig. 20 Response spectrum for BH-2 CH-53.769 km

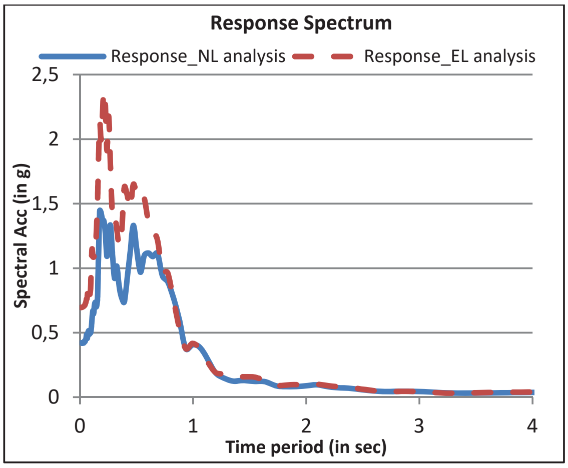

Fig. 21 Response spectrum for BH-1 CH-57.769 km

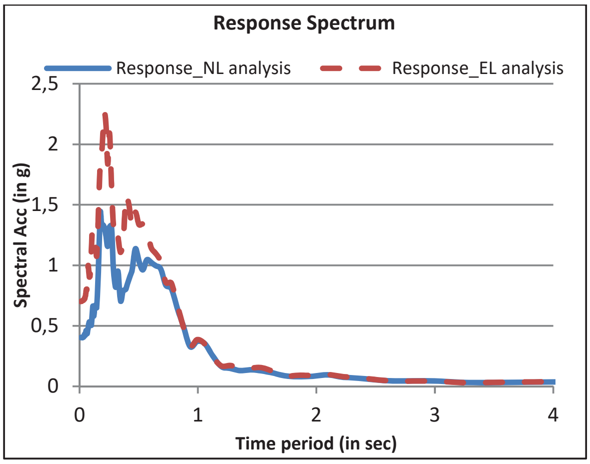

Fig. 22 Response spectrum for BH-2 CH-57.769 km

It is observed that for all the boreholes the nature of curve and time-periods for which maximum spectral acceleration occurs are predicted nearly the same by both the types of analysis. The major difference observed was in the magnitude of these spectral accelerations. The spectral accelerations predicted by EL analysis are 2.6g for BH-1, BH-2(at T≈0.4 sec), 2.4g for BH-3, BH-4(at T≈0.2 sec) and 2.5g for BH-5 (at T≈0.2 sec). Whereas, according to NL analysis spectral accelerations predicted are 2g for BH-1, BH-2(at T≈0.4 sec), 1.5g for BH-3, BH-4(at T≈0.2 sec) and 2.4g for BH-5 (at T≈0.2 sec). This shows that an overestimation of 0.5- 1 g is done by EL analysis as compared to NL analysis.

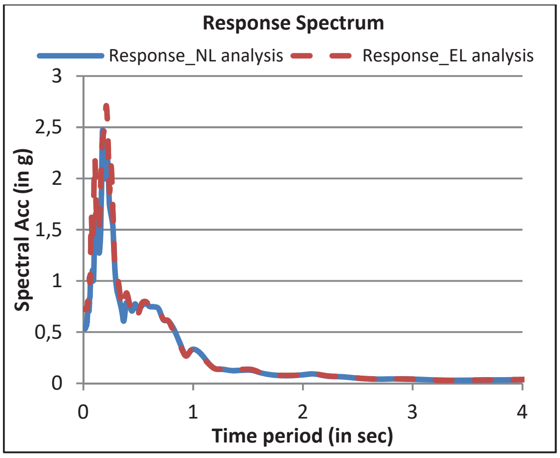

Fig. 23 Response spectrum for BH Rapar School

VIII. Conclusions

1. Equivalent linear analysis considers the value of G_{sec} as the value corresponding to maximum strain a soil layer has undergone under given seismic loading. But this is not accurate as the soil undergoes variety of strains in given cycle where the value of G_{sec} is constantly changing. Due to this assumption there is an anomaly in the results calculated by equivalent linear analysis and non-linear analysis.
2. In EL analysis, the stiffness of all the soil layers remains constant throughout the duration of a seismic loading. Thus, high amplification resulting in a possibility of resonance might occur if a strong component of the input motion coincides with the natural frequency of any soil layer. However in NL analysis, since the stiffness of soil updates continuously in the given period of the seismic loading, this type of resonance in any particular soil layer is not possible [1].
3. As can be seen from charts of PGA value of all the five boreholes, the values predicted by both EL and NL analysis for greater depths (>5m) are in close proximation as the stratum mainly present there is rocky or of stone which generally undergoes smaller strains. Since the stress-strain relationship is quite linear in that range, values predicted by both analyses are close. But in the upper stratum (<5m) where silty sand or clayey sand are present there is marginal amplification of PGA by EL analysis as it does not follow the actual stress-strain relationship which is not linear.

4. It can also be observed that the NL analysis gives conservative results in terms of maximum displacement as compared to EL analysis. The main reason for this is, EL calculates max. displacement corresponding to max. strain experienced by soil layer whereas NL analysis calculates it with respect to the final strain experienced by the soil layer. It can be greater or lesser as compared to maximum strain.
5. The values of max. strain predicted by both the analysis are very close to each other as seen from all the borehole results. This totally complies with authenticity of EL analysis as a close approximation to NL analysis. Moreover the mobilised shear stress values are also very close because these are the values of stress corresponding to maximum strains. Since the max. strains predicted by both analysis are close so are the values of mobilised stresses.
6. Overall NL analysis is more efficient and realistic because it follows the actual hyperbolic stress-strain relationship, considers stiffness degradation of soil layer in given loading cycle, mobilises stresses generated due to pore water pressure and is suitable up to shear strength of soil and even after failure.

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