JOURNAL GEOLOGICAL SOCIETY OF INDIA
Vol.97, November 2021, pp.1365-1378

Development of Surface Level Seismic Hazard Maps Considering Local Soil Conditions for the State of Haryana, India

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ABSTRACT

An extensive ground response analysis has been carried out in order to consider the effect of local soil conditions on surface level strong ground motions in the state of Haryana, India. Non-linear approach has been adopted to analyze 81 sites drilled up to refusal covering the entire state. It has been observed that 75 sites fall under site class D (medium dense soils), 4 sites under site class C (very dense soils), whereas only 2 sites fall under site class E (loose soils) as per the provisions of National Earthquake Hazard Reduction Program (NEHRP). For the estimation of low strain shear modulus (G_{max}) using SPT N-value for different type of soils, various correlations have been used due to non-availability of in-situ measured values of shear wave velocity (V_s). The standard backbone curves for clays and sands have been used in the study in the absence of site-specific shear modulus degradation and damping ratio curves. The recorded acceleration time histories of the M_s 7.0 1991 Uttarkashi earthquake, having a focal depth of 10 km with PGA (g) ranging from 0.05g to 0.31g, have been used as input motion for the analysis. Amplification factors for peak ground acceleration (PGA) and spectral acceleration (Sa) for site class D have been calculated which are found to be comparable with those reported in NEHRP provisions. However, the site-specific hazard parameters determined for the tectonic setup of the seismic study region are quite different from that suggested by Indian standard code of practice.

INTRODUCTION

The subcontinent of India has withstood a number of great earthquakes documented from the times of great battle of Mahabharata to the recent 2001 Bhuj earthquake. Millions of people lost their lives, homes and families during these earthquakes. The main reasons of the disastrous consequences of these earthquakes in India are the outdated structural design practices and lax building by-laws. Earthquake is an unstoppable force of nature and its occurrence cannot be predicted. However, sound design of earthquake resistant structures, planning of rescue arrangements and implementation of mitigation measures, can greatly reduce vulnerability to earthquakes (Jain, 2016). The assessment of intensity of earthquakes, and earthquake induced hazards, e.g. wave amplification, soil liquefaction, landslides and tsunami, in the regions of high seismicity, is crucial. Landslides and tsunami occur in hilly and coastal areas respectively, making the other two, i.e. seismic wave amplification and soil liquefaction as the more observed ones in plain areas including the state of Haryana.

Seismic hazard is known to have considerable spatial variability (Iyengar and Ghosh, 2004). Therefore, it is necessary to develop regional seismic hazard scenario in line with local tectonic setting considering the effect of local soil conditions as opposed to the broad zoning with specified PGA values as suggested by the Indian Seismic Code (IS:1893-Part 1, 2016). In the present study, one dimensional nonlinear ground response analysis has been carried for the state of Haryana to account for the local site effects on earthquake ground motions. The soil formations in the top 30 m of the geological profile above bedrock, often act as a filter, and are generally responsible for modification in ground motions. Hence, the sites are classified on the basis of geotechnical properties for the top 30 m of the soil profile, e.g. SPT-value, shear wave velocity, CPT-value etc. The cyclic behaviour of soil can be modeled by using the dynamic properties, i.e. low strain shear modulus and low strain damping ratio, and the backbone curves for the variation of these parameters, i.e. modulus degradation (G/G_{max}-γ) curves and damping ratio (D-γ) curves. The low strain shear modulus (G_{max}) values for various sites have been estimated using suitable correlations between SPT N-value and G_{max}, and backbone curves have been selected based on the review of literature. The earthquake acceleration time histories recorded in the seismic study region have been used as input motion for representing the dynamic force. The outcome has been formulated in terms of amplification factors for peak ground acceleration and spectral acceleration. The plots for the variation of shear strains developed along the depth have also been studied to observe the deformation trend at various sites due to earthquake ground motions.

SEISMIC HAZARD SCENARIO IN HARYANA

Indian seismic code has classified the state of Haryana into three seismic zones: Zone II, Zone III and Zone IV (IS:1893-Part 1, 2016), making it prone to low to moderate damage risk from earthquakes. The potential seismic hazard of the state is controlled by different tectonic regimes, which include Himalayan Frontal Thrust on the north and north-eastern side, Aravalli-Delhi fold belt on the south and Sargodha-Lahore-Delhi ridge on the north-western side (see Fig. 1) (Puri and Jain, 2019). The region is seismically susceptible and vulnerable to great earthquakes, originating in the far field in Himalayan thrust system. The Aravalli Delhi Fold Belt is also a source of frequent moderate earthquakes in the southern parts of Haryana. The earthquake hazard in the study area is affected to some degree by seismicity of Sargodha-Lahore-Delhi ridge. However, no major earthquake has been attributed to this tectonic regime.

In the present study, the results of probabilistic seismic hazard analysis (PSHA) carried out for the state of Haryana by Puri and Jain (2019) have been adopted as primary input parameters for the selection of earthquake time histories in order to perform ground response analysis for various sites. In the study, the hazard has been calculated for rock site conditions at return periods of 475 years, 2475 years and 4975 years with 10%, 2% and 1% probability of exceedance respectively in 50 years. The PGA_{rock} ranging from 0.05g to 0.35g has been observed for a return period of 475 year with 10% probability of exceedance in 50 years (Figure 2). For a return period of 2475 year, it ranges from 0.1g to 0.6g with 2% probability of exceedance in 50 years, while for a return period of 4975 year, it ranges from 0.1g to

0016-7622/2021-97-11-1365/$ 1.00 © GEOL. SOC. INDIA | DOI: 10.1007/s12594-021-1875-z

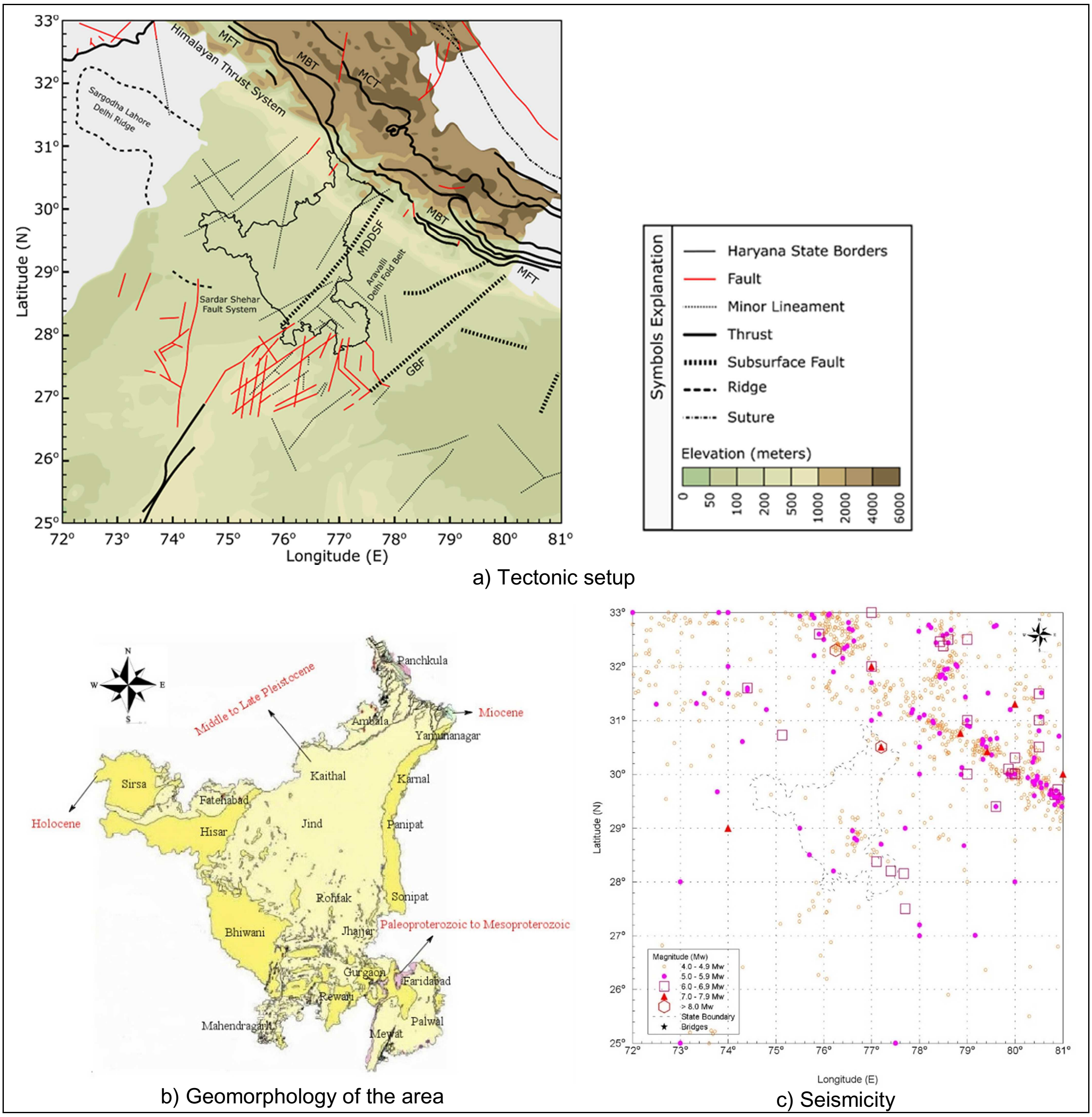

Fig.1. Map showing tectonic setup, Geomorphology and seismicity of the seismic study region (after Puri and Jain 2019)

0.7g with 1% probability of exceedance in 50 years. In this study, the hazard parameters obtained at 10% probability of exceedance have been considered for further analysis due to unavailability of the recorded earthquake acceleration time histories compatible with the expected ground motions at 2% and 1% probabilities of exceedance, the same representing rarely expected hazard scenario.

In the study, It has been observed that at 10% probability of exceedance in 50 years, the north and north-eastern parts of Haryana, which include districts of Panchkula, Ambala, Yamunanagar and some parts of Kurukshetra, are expected to experience high intensity ground motions during earthquakes. It can be attributed to the proximity of this region to Himalayan thrust system. The rest of area in Haryana is expected to have low seismic hazard. The PGA values in some areas are low to moderate, but it has been anticipated that the ground motions could get amplified due to local site effects.

Disaggregation of PSHA results has been carried out to obtain the PGA, magnitude and distance combination for the expected ground motion at every grid point. These values have been used further in the ground response analysis for selecting input acceleration time histories for each site.

GEOTECHNICAL SITE CHARACTERIZATION

The geotechnical data have been collected from several government and private organizations to assess ground water table and soil conditions in the state. The developed database has information for 1058 distinct locations covering various districts of Haryana for boreholes extended up to 50 m depth. The record of only 81 boreholes which were drilled up to refusal has been considered for ground response analysis. The sites, where positive pore water pressure can develop during earthquake, have not been considered in the analysis, as it can alter the soil response.

Site Class

The sites have been classified using average SPT value of the profile as per the provisions of National Earthquake Hazard Reduction Programme (NEHRP) (FEMA 450, 2003). The current NEHRP

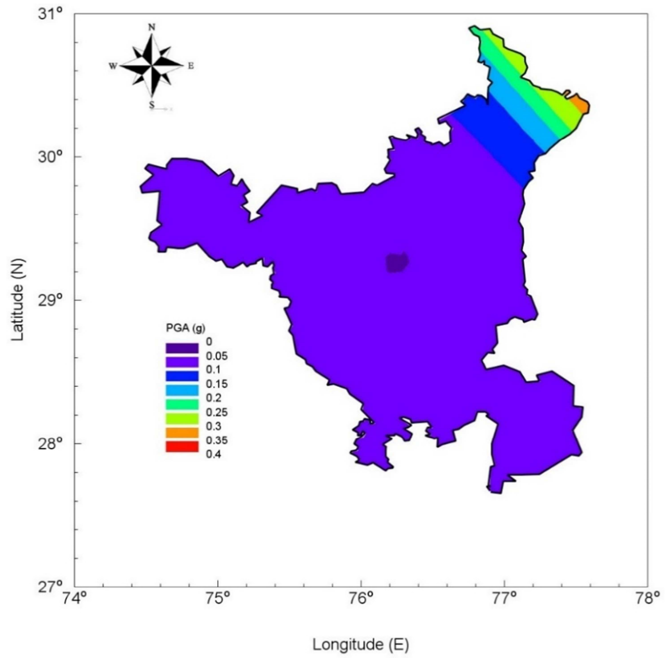

Fig.2. PGA map of Haryana for 10% probability of exceedance in 50 years (Puri and Jain 2019)

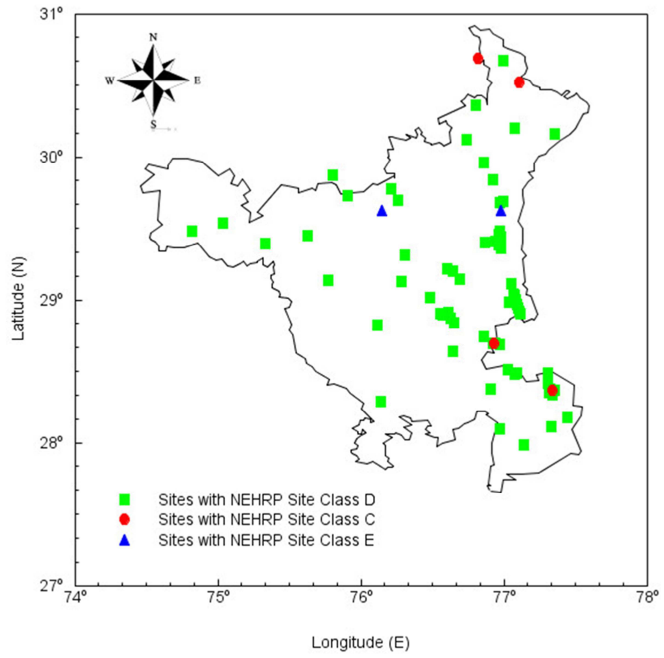

Fig. 3. Location and NEHRP site classification of selected sites

provisions categorize soils into A, B, C, D, E and F classes based on average SPT-value (N_{30}) of the profile which can be calculated using the following equation:

$$N_{30} = \frac{\Sigma_{i=1}^{n} d_i}{\Sigma_{i=1}^{n} (d_i / N_i)} \tag{1}$$

where N_{30} = average SPT N-value for 30 m depth, N_i = SPT N-value of the i^{th} layer and d_i = thickness of the layer. It has been observed that the average N-value for the profiles vary between 11 to 61, with minimum and maximum values observed at locations near Narwana in district Jind and Raipur Kalan in district Panchkula respectively. Also for 81 sites drilled up to refusal, 75 sites fall under site class D (medium soils), 4 sites fall under site class C (very dense soils), whereas only 2 sites fall under site class E (loose soils). Figure 3 shows the location and NEHRP site classification of the sites used in the ground response analysis.

Bedrock Definition

An elastic bedrock has been assumed at refusal, i.e. for blow count, N>50 for 15 cm penetration and N>100 for 30 cm penetration of SPT split-spoon sampler. Conventionally, the engineering bedrock is assumed as the uppermost layer of the soil column having a shear wave velocity (V_s) ≥ 760 m/s (Nath and Thingbaijam, 2011). In general, the shear wave velocity of the bedrock is greater than that of the overlying soil profile. The bedrock damping ratio has no effect in time domain analysis and only a negligible effect in frequency domain analysis (Hashash et al., 2016). For the present study, bedrock has been modelled as an elastic half space with 2% damping, 2.5 gm/cm^3 density and shear wave velocity (V_s) of 760 m/s.

ESTIMATION OF DYNAMIC SOIL PROPERTIES

The dynamic soil characteristics essential for modelling the cyclic behaviour of soils are, (i) maximum shear modulus or low strain shear modulus (G_{max}), (ii) modulus degradation (G/G_{max}- γ) curves, and (iii) damping ratio (D-γ) curves. These properties, in context with the present study, have been described below.

Low Strain Shear Modulus (G_{max})

Maximum shear modulus (G_{max}) plays fundamental role in the estimation of the ground response parameters in seismic microzonation studies. Characterization of the stiffness of an element of soil requires consideration of both maximum shear modulus (G_{max}), and the way the modulus ratio G/G_{max} varies with cyclic strain amplitude and other parameters. The field value of G_{max} is generally computed using shear wave velocity by the following equation.

$$G_{max} = \rho\, V_s^2 \tag{2}$$

where ρ = density of soil layer and V_s = shear wave velocity. The value of shear modulus calculated from shear wave velocity is quite reliable, as most of the geophysical tests conducted to determine V_s, induce shear strains < 3 × 10^{-4} %.

Due to non-availability of in-situ measured shear wave velocity (V_s), correlations for the estimation of G_{max} can be used using parameters determined from various field tests as reported in literature which include standard penetration test (SPT) (Imai and Tonouchi, 1982), cone penetration test (CPT) (Baldi et al., 1989), dilatometer test (DMT) (Hryciw, 1990) and pressure meter test (PMT) (Byrne et al., 1991). In the present study, several correlations between low strain shear modulus (G_{max}) and SPT N-value for different type of soils have been reviewed as reported in Table 1. The V_s - N correlations have not been used considering the unnecessary approximation in calculations, as V_s determined using SPT blow count would again be converted to G_{max} in the ground response analysis software. Suitable correlations for the soils in the study area have been shortlisted as per the recommendations of Anbazhagan et al. (2015) and Anbazhagan et al. (2016). The equations developed by Ohba and Toriumi (1970) for clays, Ohsaki and Iwasaki (1982) for cohesionless soils and intermediate soils, and Imai and Tonouchi (1982) for gravels have been used to calculate the G_{max} values.

Standard G/G_{max}-γ and D-γ Curves

The G/G_{max}-γ and D-γ curves are extremely important for ground response analysis as the variation of parameters strongly influences the extent to which a soil deposit would amplify or attenuate seismic

Table 1. Correlation between shear modulus (G) and SPT N-value

Sl. No.	Author(s) name	Correlation	Unit	Soil type	N-value used	Correlation coefficient
1.	Imai and Yoshimura (1970)	$G = 1000\ N^{0.78}$	t/m^2	Mixed soil type	1-100	-
2.	Ohba and Toriumi (1970)	**$G = 1220\ N^{0.62}$**	**t/m^2**	**Alluvial sand, clay**	**1-40**	-
3.	Ohta et al. (1972)	$G = 1390\ N^{0.72}$	t/m^2	Tertiary, diluvial sandy, cohesive soil	0-150	-
4.	Ohsaki and Iwasaki (1973)	$G = 1218\ N^{0.78}$	t/m^2	All soil types	0-100	0.888
5.	Ohsaki and Iwasaki (1973)	**$G = 650\ N^{0.94}$**	**t/m^2**	**Cohesionless soils**	**5-100**	**0.852**
6.	Ohsaki and Iwasaki (1973)	**$G = 1182\ N^{0.76}$**	**t/m^2**	**Intermediate soils**	**1-100**	**0.742**
7.	Ohsaki and Iwasaki (1973)	$G = 1400\ N^{0.71}$	t/m^2	Cohesive soils	0-50	0.921
8.	Hara et al. (1974)	$G = 158\ N^{0.668}$	kg/cm^2	Alluvial, diluvial and tertiary soils	0-90	0.88
9.	Imai and Tonouchi (1982)	$G = 154\ N^{0.557}$	kg/cm^2	Clay fill	0-35	0.582
10.	Imai and Tonouchi (1982)	$G = 224\ N^{0.383}$	kg/cm^2	Special soils-Loam, Sirasu	1-50	0.497
11.	Imai and Tonouchi (1982)	$G = 142\ N^{0.5}$	kg/cm^2	Sand fill	0-100	0.606
12.	Imai and Tonouchi (1982)	$G = 204\ N^{0.668}$	kg/cm^2	Tertiary clay and sand	7-500	0.682
13.	Imai and Tonouchi (1982)	$G = 176\ N^{0.607}$	kg/cm^2	Alluvial clay	0-40	0.715
14.	Imai and Tonouchi (1982)	$G = 53.7\ N^{1.08}$	kg/cm^2	Alluvial peat	0-5	0.769
15.	Imai and Tonouchi (1982)	$G = 251\ N^{0.555}$	kg/cm^2	Diluvial clay	1-170	0.712
16.	Imai and Tonouchi (1982)	$G = 125\ N^{0.611}$	kg/cm^2	Alluvial sand	1-140	0.871
17.	Imai and Tonouchi (1982)	$G = 177\ N^{0.631}$	kg/cm^2	Diluvial sand	2-300	0.729
18.	Imai and Tonouchi (1982)	**$G = 82.5\ N^{0.767}$**	**kg/cm^2**	**Alluvial gravel**	**10-200**	**0.798**
19.	Imai and Tonouchi (1982)	**$G = 319\ N^{0.526}$**	**kg/cm^2**	**Diluvial gravel**	**25-380**	**0.552**
20.	Imai and Tonouchi (1982)	$G = 144\ N^{0.68}$	kg/cm^2	All soil types	0-300	0.867
21.	Seed et al. (1983)	$G = 65\ N$	t/ft^2	All soil types	-	-
22.	Kramer (2013)	$G = 325\ (N_{60})^{0.68}$	$kips/ft^2$	Sandy soil	-	-
23.	Anbazhagan and Sitharam (2010)	$G = 24.28\ N^{0.55}$	MPa	Silty sand with less percentage of clay	2-109	0.938
24.	Anbazhagan et al. (2012)	$G=16.40\ N^{0.65}$	MPa	All soil types	1-100	0.922

*G = Shear modulus, N = SPT N-value and N_{60} = N-value corrected for 60% hammer efficiency

waves. Often due to time and practical constraints, ground response studies are done using the standard curves, as developing the site-specific curves for soils is a tedious process which requires advanced dynamic soil tests. Many investigators, e.g. Seed and Idriss (1970), Seed et al. (1986), Sun et al. (1988), Vucetic and Dobry (1991), Darendeli (2001), Menq (2003), and Vardanega and Bolton (2011), have developed standard curves for different type of soils. The G/G_{max}-γ and D-γ curves developed by Vucetic and Dobry (1991), based on plasticity index (PI) values, have been used for clays and plastic silts. As proposed in the study carried out on sandy and silty soils of Delhi and nearby areas by Hanumantharao and Ramana (2008), the curves developed by Seed and Idriss (1970) have been used for sandy and silty soils. The curves developed by Seed and Idriss (1986) have been used for gravels.

Selection of Input Earthquake Motion

The last step in wave amplification analysis is generating or getting an acceleration time history, which is compatible with the maximum dynamic loading expected at the site of interest. This time history is then used, as an input motion assuming it to be originating from the engineering bedrock as an incident wave. As per the recommendations of Pacific Earthquake Engineering Research Center (PEER), a rock outcropping motion should be applied without any modification for an elastic base for time-domain analysis, or a within motion should be used without modification for a rigid bedrock (Stewart and Kwok, 2008).

Modern seismic codes, e.g. UBC 1997 and IBC 2000, motivate the use of real records, at the same time allowing the design engineer to supplement these with simulated motions where sufficient suitable real records are not available (Bommer and Acevedo, 2004). Suitable acceleration time histories can be selected on the basis of PGA value, magnitude of controlling earthquake, source to site distance and site class.

In the study, PGA values for rock sites obtained from PSHA at 10% probability of exceedance for a time frame of 50 years have been used for the selection of input motions for the sites analyzed. The rock PGA values for the selected 81 sites vary from 0.05g to 0.35g. The records of acceleration time histories for several earthquakes accessed from virtual repositories of strong motion data, COSMOS and PESMOS, have been analyzed to select an appropriate input motion (Table 2).

Table 2. Earthquake events considered for the selection of input motion

Sl. No.	Earthquake	Magnitude	Depth (m)	Site type
1.	Uttarkashi (19-10-1991)	7.0 M_s	10.0	Rock
2.	Chamba (24-03-1995)	4.9 M_L	33.0	Soil
3.	Chamoli (28-03-1999)	6.6 M_w	15.0	Rock
4.	Bhuj (26-01-2001)	7.0 M_L	16.0	Soil (in a building)
5.	Sikkim (18-09-2011)	6.9 M_w	19.7	Rock

Table 3. Strong motion characteristics of M_s 7.0 1991 Uttarkashi earthquake

Sl. No.	Recording station	Site type	Hypocentral distance (km)	Horizontal component of PGA (g)	
				Longitudinal component	Transverse component
1.	Bhatwari	Rock	21.7	0.253	0.247
2.	Uttarkashi	Rock	34.0	0.242	0.310
3.	Ghansiali	Rock	39.3	0.118	0.117
4.	Tehri	Rock	50.6	0.073	0.062
5.	Barkot	Rock	55.8	0.095	0.082
6.	Rudraprayag	Rock	56.2	0.053	0.052
7.	Srinagar	Rock	58.8	0.067	0.050
8.	Koteshwar	Rock	61.3	0.101	0.067
9.	Karnprayag	Rock	69.6	0.062	0.079
10.	Purola	Rock	70.0	0.075	0.094

There is no natural earthquake time history recorded in Haryana that can justify the expected seismic loading determined based on PSHA. Moreover, use of recorded earthquake time histories of low magnitude earthquakes as seed record to simulate spectrum compatible artificial time histories would lead to unrealistic changes in the velocity, displacement and energy components of the recorded ground motion.

The earthquake hazard in the state is mainly related to the proximate Himalayan thrust system. Therefore, recorded earthquake time histories of the Himalayan earthquake that occurred on 19th October, 1991 at Uttarkashi (epicenter) has been used in the present study. The acceleration time histories of the Uttarkashi earthquake have the source-to-site distance, which is in line with the PSHA based scenario developed for the study area. The focal depths of the target and selected earthquake motions are reasonably comparable. Also, the records are from the same seismic region as has been adopted for the study. Therefore, the recorded acceleration time histories of the M_s 7.0 1991 Uttarkashi earthquake measured at rock outcrop, having a focal depth of 10 km, with PGA (g) ranging from 0.05g to 0.31g, have been used as input motion for the analysis (Figure 4). The strong motion characteristics of the earthquake have been reported in Table 3.

Nonlinear Ground Response

A nonlinear model has been adopted considering the inherent nonlinearity of soil with its tendency to have constant variations in shear modulus during cyclic loading. Moreover, an equivalent linear model is incapable to represent true variation in soil stiffness that actually occurs during cyclic loading as highlighted by many investigators, e.g. Finn et al. (1978), Arslan and Siyahi (2006), Hosseini and Pajouh (2012) and Kramer (2013). Nevertheless, the nonlinear models can better represent the response of soils undergoing cyclic loading (Hosseini and Pajouh, 2012) as well as for the sites with deep soft soils or sites where strong earthquakes are expected (Hashash et al., 2010).

Nonlinear analysis has been carried out using DEEPSOIL v.6.1 (Hashash et al., 2016). The pressure dependent hyperbolic model (MKZ) relates shear modulus and damping ratio of the soil layers to shear strains developed during earthquakes. For each layer of the soil deposit, reference strain (γ_r), stress-strain curve parameter-β, stress-strain curve parameter-s, pressure dependent parameter-b, reference stress (σ_{ref}) and pressure dependent parameter-d, need to be defined. A curve fitting procedure, MRDF-UIUC (a curve fitting model available in DEEPSOIL), is then adopted for each layer to find the above parameters that provide the best fit for both modulus reduction and damping ratio. For sandy soils, effective vertical stress is required for defining the variation of shear modulus with shear strain at a particular depth from modulus reduction curves, whereas for clayey soils, effective vertical stress and plasticity index are required. The hysteretic behaviour, i.e. the variation of cyclic shear stress with cyclic shear strain, is governed by Masing and extended Masing criteria. The thickness of the layers is so adjusted that the maximum frequency that a layer can propagate is always above 25 Hz. The number of iterations in the software is kept at 15.

The nonlinear analysis of the wave propagation in soils allows the soil properties to change with the time for the variation in strain. All the sites are assumed to have horizontal soil layers, which extend infinitely. The soil profiles have been modelled as a series of lumped masses connected by springs and dashpots making a multiple degree freedom system (MDOF) as shown in Figure 5. The following incremental dynamic equation of motion is solved to carry out the nonlinear dynamic analysis of the soil column.

$$M\Delta\ddot{u} + C\Delta\dot{u} + K\Delta u = -M\Delta\ddot{u}_g \tag{3}$$

where the coefficients M, C and K represent mass, viscous damping and stiffness respectively, and $\ddot{u}$, $\dot{u}$, u and $\ddot{u}_g$ represent acceleration, velocity, displacement and exciting acceleration at the elastic base respectively.

The soil response is obtained from a constitutive model that describes the cyclic behaviour of soil. For modelling the hysteretic behaviour, most widely used software use a variation of hyperbolic model, to represent the backbone curve of the soil with the extended unload-reload Masing rules (Masing, 1926). The loading and unloading equations of modified Konder-Zelasko (MKZ) model (Matasovic, 1993), further modified by Hashash and Park (2001) used in DEEPSOIL software are respectively as follows.

$$\tau = \frac{\gamma G_{max}}{1 + \beta(\gamma / \gamma_r)^S} \tag{4}$$

$$\tau = \frac{2G_{max}((\gamma - \gamma_{rev})/2)}{1 + \beta((\gamma - \gamma_{rev})/2\gamma_r)^S} + \tau_{rev} \tag{5}$$

where τ = shear strength, G_{max} = low strain shear modulus, γ = shear strain, γ_r = reference shear strain, τ_{rev} = shear stress at reversal, γ_{rev} = shear strain at reversal, β and S are model fitting parameters.

Loading and unloading (cyclic loading) is introduced by extended Masing rules, which are as follows (Kramer, 2013) (Fig. 6):

(i) The stress-strain curve follows the backbone curve for the initial loading.

(ii) The stress-strain curve tracks a path given by Equation 5, as stress reversal occurs at a point B. This means that the unloading, reloading curves would have the same shape as the backbone curve, and the origin is shifted to load reversal point. The path is enlarged by a factor of two.

The first two rules are called Masing Rules (Masing, 1926), but are insufficient for describing the soil response under general cyclic loading. Hence, the following additional rules are required.

(iii) If unloading or reloading curve exceeds maximum previous strain and intersects backbone curve, it follows backbone curve until next stress reversal.

(iv) The stress strain curve follows the stress-strain curve of previous cycle, if unloading or reloading curve of the present cycle intersects unloading or reloading curve of previous cycle.

The modification in MKZ model allows the effect of confining pressure on secant shear modulus of soil. In addition, there is no coupling between the confining pressure and shear stress. The coupling is introduced by making reference shear strain (γ_r) dependent on effective stress by using the following equation:

$$\gamma_r = a\,((\sigma'_v) / (\sigma_{ref}))^b \tag{6}$$

where *a* and *b* are curve fitting parameters, σ'_v = vertical effective stress, σ_{ref} = reference shear stress of 0.18 MPa.

However, the modified model is almost linear at low strains, and hence provides zero hysteretic damping at lower strains. Low strain damping (ξ) is added separately by the following equation in order to simulate the actual soil behaviour, which exhibits damping even at very small strains.

$$\xi = c / (\sigma'_v)^d \tag{7}$$

where c and d are curve fitting parameters. The parameter 'd' can be set to zero in case a pressure independent small strain damping is desired.

It is observed that overestimation of damping at large strain can result, when the hysteretic damping (ξ_{Masing}) is calculated using unload-

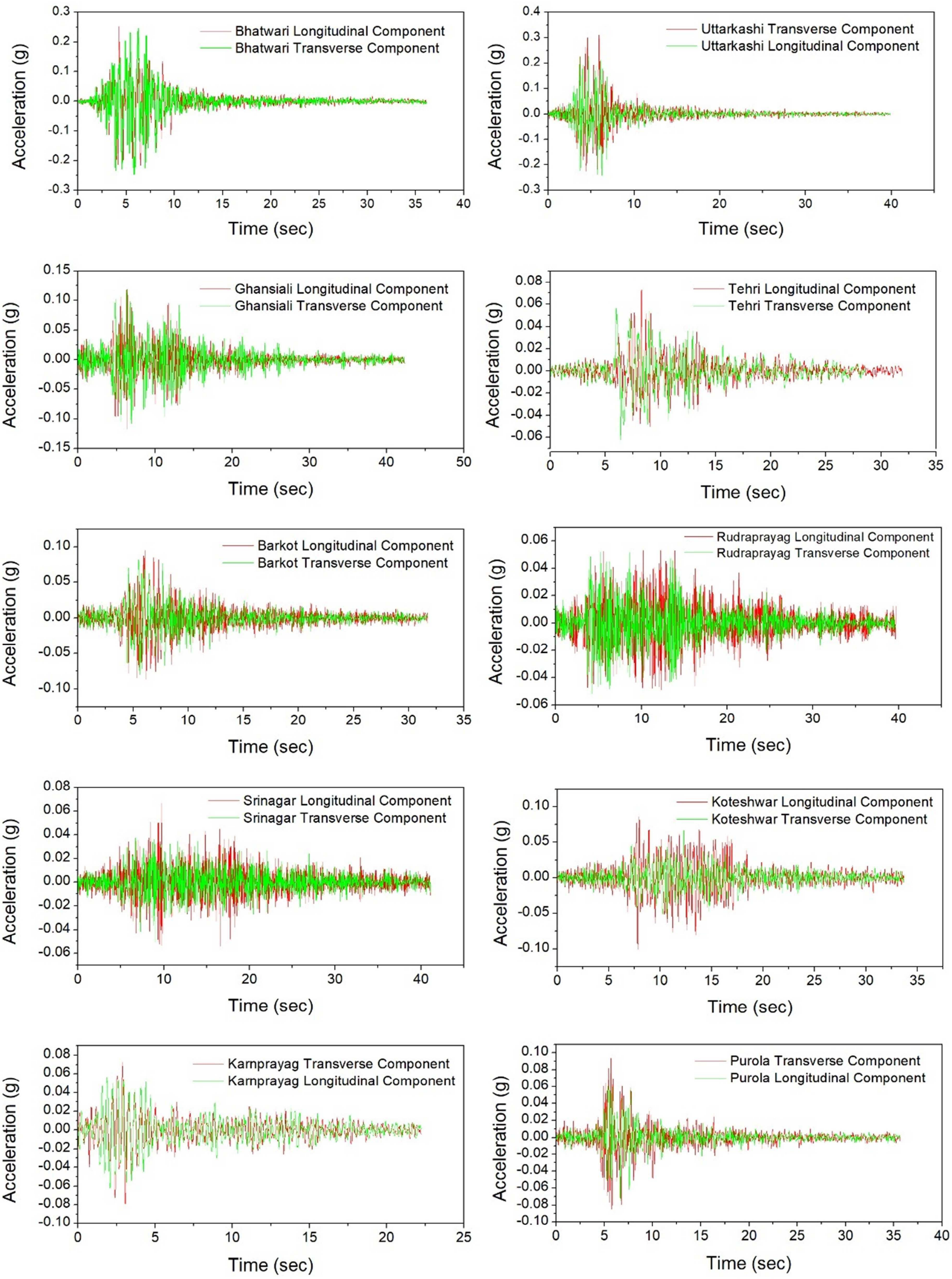

Fig. 4. Recorded acceleration time histories of M_s 7.0 1991 Uttarkashi earthquake

reload cycles as per using Masing rules based on modulus reduction curves. This overestimation can be avoided by multiplying ξ_{Masing} with a damping reduction factor $F(\gamma_m)$, which is given by the following equation.

$$F(\gamma_m) = p_1 - p_2\left(1 - \frac{G_{\gamma_m}}{G_{max}}\right)^{p_3} \tag{8}$$

where $G_{\gamma m}$ = shear modulus at maximum strain, and p_1, p_2, p_3 are fitting parameters. This factor provides the best fit for both modulus reduction and damping ratio curves. The reduction factor modifies the reloading cycle and the expression is as follows:

$$\tau = F(\gamma_m)\left[\frac{2G_{max}\left(\frac{\gamma-\gamma_{rev}}{2}\right)}{1+\beta\left(\frac{\gamma-\gamma_{rev}}{2\gamma_r}\right)^S} - \frac{G_{max}\,(\gamma-\gamma_{rev})}{1+\beta\left(\frac{\gamma_m}{\gamma_r}\right)^S}\right] + \frac{G_{max}\,(\gamma-\gamma_{rev})}{1+\beta\left(\frac{\gamma_m}{\gamma_r}\right)^S} + \tau_{rev} \tag{9}$$

where γ_m = maximum shear strain. The β method (Newmark, 1959)

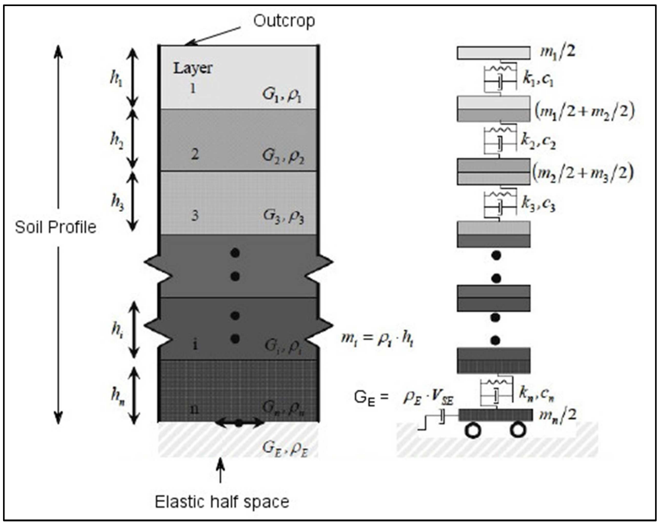

Fig. 5. Representation of horizontally layered soil column as MDOF (after Hashash et al., 2010)

is then used to solve the system of equations and to obtain response of the soil column.

RESULTS AND DISCUSSION

Peak Ground Acceleration

The results of ground response analysis for peak ground acceleration have been presented in Table 4. The average site periods for site class C, D and E have been observed as 0.34 sec, 0.49 sec and 0.67 sec respectively. The variation of amplification factors for the state has been shown in Fig.7. It has been observed that for $PGA_{rock} \leq 0.1g$, the amplification factor varies from 1.5 to 2.0 for most of the study area, while for $PGA_{rock} > 0.1g$, the amplification factor varies from 1.0 to 1.5. For site class C, D and E, the average amplification factors have been calculated as 1.48, 1.79 and 1.51 respectively. The average amplification factors of 1.81 for $PGA_{rock} \leq 0.1g$ and 1.30 for $PGA_{rock} > 0.1g$ have been calculated for site class D. As most of the area in the state belongs to this site category, these factors have been used to modify the seismic hazard map for PGA developed for rock sites at 10% probability of exceedance in 50 years as shown in Fig.8. It has been observed that in central Haryana, areas including the district of Jind and nearby are prone to low hazard due to earthquakes with PGA ranging from 0.08g to 0.1g. The north and north-eastern parts of the state are vulnerable to high shaking during earthquake with maximum calculated PGA value of 0.5g. The rest of the region is prone to low to moderate hazard with PGA ranging from 0.1g to 0.2g.

In order to study the effect of various factors such as N_{30}, D (depth of refusal for SPT sampler) and PGA of input motion on amplification factors (AF) for PGA during earthquakes, a regression analysis has been carried out. Results of regression analysis has been reported in Table 5. The predicted values of amplification factor have been plotted against values obtained from ground response analysis (see Fig. 9). Also, based on the regression analysis, an equation has been developed, which is as follows:

$$AF = 0.005\, N_{30} - 0.013D - 2.248\, PGA_{input\ motion} + 2.036 \qquad (10)$$

where AF = amplification factor, N_{30} = average SPT N-value for 30 m depth, $PGA_{\text{input motion}}$ = PGA of input motion (g). It has been observed that factors like significance F and P-value have values less than 0.005, which confirms the dependence of AF on the tested variables. Also, coefficient of correlation (R) has been observed to be 0.705, which shows the correlation if moderately strong. However,

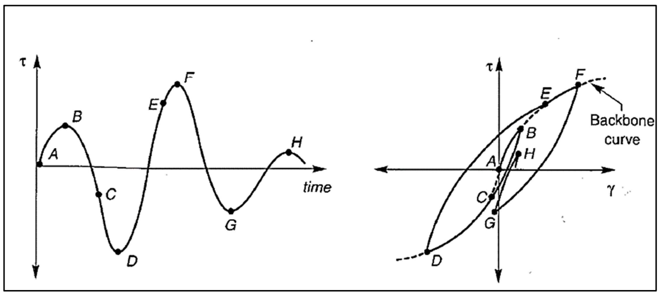

Fig. 6. Extended Masing rules (after Kramer, 2013). **(a)** Variation of shear stress with time. **(b)** Resulting stress strain behaviour.

coefficient of determination (R^2) shows that only 49.8% of variation in the value of the amplification factor can be explained by this correlation and the remaining 52.3% variation is unexplained.

The most of sites analyzed in the study area belong to site class D as per NEHRP provisions and also, geomorphology of these sites shows that age of these deposits range from middle to late Pleistocene. Hence, dependence of amplification factor on geology and geomorphology has not been investigated.

Spectral Acceleration

Amplification factors for spectral acceleration have been reported in Table 6. The seismic hazard maps for Sa (g) at 10% probability of exceedance in 50 years for rock sites at periods of 0.1 sec, 0.2 sec, 1.0 sec and 2.0 sec have also been modified for site class D using the respective amplification factors. These maps have been shown in Figure 10 to Figure 13.

It has been observed from the trend of spectral acceleration across the state that at low periods, i.e. for 0.1 sec and 0.2 sec, the whole of north-eastern part is prone to moderate to very high spectral accelerations reaching up to 0.9g. Also, for some areas in western Haryana, moderate values of Sa have been observed, going up to 0.4g. This shows that low period structures in these areas could receive moderate to high damage during earthquakes. At higher periods, only the north-eastern part of Haryana is expected to experience moderate Sa reaching up to 0.4g, and for rest of the areas, values of Sa at higher periods is quite low. Hence, except for north-eastern part, multi-storey buildings (more than 10-storey) constructed in the state are expected to remain safe during an earthquake. However, the old structures designed without any seismic consideration may sustain damage even in low risk zones.

Comparison of Calculated PGA and Sa with Code Provisions

The calculated amplification factors for PGA and Sa are comparable with factors reported in NEHRP provisions (FEMA P-750, 2009). The results from the study have also been compared with the specifications given in Indian standard code of practice (IS 1893-Part 1, 2016). It has been observed that the Indian seismic code underestimates the hazard for the districts of Ambala, Panchkula and Yamunanagar. For the districts of Gurgaon, Faridabad, Palwal, Mewat, Rewari, Jhajjar, Jind, some parts of Rohtak, Sonipat, Panipat, Karnal and Kurukshetra, it slightly overestimates the hazard. For districts of Sirsa, Fatehabad, Hisar and Bhiwani and some parts of Mohindragarh, it slightly underestimates the hazard. The deviation could be attributed to that the parameters reported in seismic code of India are not site specific hazard parameters. These values have been suggested on the basis of the earthquake events already occurred. Moreover, the national seismic code of India itself suggests a site specific study in case of important structures.

Deformation Trend

The development of high shear strains ranging from 0.01% to 2%

Table 4. Results of nonlinear ground response analysis

Site No.	Location	PGA rock (g)	Depth of profile (m)	N_{30}	Site class	Site period (sec)	Amplification factor	Surface PGA (g)
1.	Bridge on river at Raipur Rani to Naraingarh road, Ambala	0.224	20.0	51	C	0.26	1.336	0.299
2.	Bridge at Adhoya Chhapar road, Ambala	0.140	35.0	27	D	0.51	1.641	0.230
3.	Jandali flyover bridge, Ambala	0.120	30.0	38	D	0.40	1.650	0.198
4.	Lajpat Nagar, Tosham-Dadri bypass, Bhiwani	0.059	25.0	16	D	0.42	1.742	0.103
5.	Bridge near railway station, Faridabad	0.064	30.0	33	D	0.38	2.070	0.132
6.	Bridge near Ballabhgarh railway station, Faridabad	0.064	30.0	21	D	0.44	1.806	0.116
7.	NHPC chowk Metro station, Faridabad	0.065	30.0	42	D	0.35	2.129	0.138
8.	Near Super Seals, Faridabad	0.065	30.0	42	D	0.28	2.113	0.137
9.	Sector 76, Faridabad	0.064	30.0	27	D	0.42	1.677	0.107
10.	Sector 10, Faridabad	0.064	30.0	39	D	0.36	1.940	0.124
11.	CGHS apartments, Sector 2, Faridabad	0.064	25.0	23	D	0.36	2.113	0.135
12.	Bridge over Agra canal, Faridabad	0.064	25.0	50	C	0.27	1.952	0.125
13.	Bridge at MDR 103, Bhattu Mandi, Fatehabad	0.067	25.0	15	D	0.46	1.612	0.108
14.	Gurney, Fatehabad	0.070	25.0	20	D	0.41	1.746	0.122
15.	Nuclear power plant site, Gorakhpur village, Fatehabad	0.066	20.0	15	D	0.40	2.500	0.165
16.	Bridge at Tohana, Fatehabad	0.067	35.0	24	D	0.56	1.532	0.103
17.	IMT Manesar, Gurgaon	0.061	15.0	17	D	0.24	2.032	0.124
18.	Bridge, Palam Vihar railway crossing, Gurgaon	0.063	25.0	16	D	0.46	2.468	0.155
19.	Sikandarpur Metro station, Gurgaon	0.063	30.0	48	D	0.45	1.746	0.110
20.	IFFCO chowk Metro station, Gurgaon	0.063	30.0	31	D	0.49	1.881	0.119
21.	Bridge near Jindal factory, Hisar	0.066	25.0	24	D	0.37	1.952	0.129
22.	Bridge on canal at Bass-Badchhapar Road, Hisar	0.051	20.0	18	D	0.53	1.673	0.085
23.	Nehru Park, Jhajjar	0.064	30.0	47	D	0.40	1.726	0.110
24.	Basant Vihar, Jhajjar	0.064	30.0	51	C	0.41	1.360	0.087
25.	Ram Nagar, Jhajjar	0.064	30.0	22	D	0.45	2.590	0.166
26.	Tikri Kalan, Jhajjar	0.064	30.0	28	D	0.48	1.836	0.118
27.	Asauda railway station, Jhajjar	0.064	20.0	19	D	0.46	2.210	0.141
28.	Bridge at Dhigal-Jhajjar section, Jhajjar	0.061	40.0	20	D	0.56	1.716	0.105
29.	Bridge at Jind - Rohtak - Delhi section, Jind	0.050	30.0	31	D	0.47	1.553	0.078
30.	Bridge on Dhantan distributary on Narwana road, Jind	0.058	31.0	11	E	0.63	1.774	0.103
31.	Bridge on Kurar-Hari Pura- Sanghan road, Kaithal	0.063	31.0	23	D	0.45	1.567	0.099
32.	Bridge on Sirsa canal on Kalayat - Sajuma road, Kaithal	0.059	31.0	18	D	0.51	1.677	0.099
33.	Bridge at Nilokheri, Karnal	0.087	31.0	17	D	0.60	1.585	0.138
34.	Bridge on Karnal Munak road, Karnal	0.080	30.0	22	D	0.47	1.512	0.121
35.	Sector 13, Karnal	0.083	20.0	26	D	0.31	2.405	0.200
36.	Bridge on Western-Jamuna canal on NH-1, Karnal	0.079	40.0	13	E	0.71	1.253	0.099
37.	Bridge, Amin road chowk, Kurukshetra	0.092	30.0	22	D	0.49	1.372	0.126
38.	Markanda bridge, Jhansa, Kurukshetra	0.094	30.0	16	D	0.53	1.543	0.145
39.	Bridge on Dadri Narnaul Road, Mohindragarh	0.061	30.0	30	D	0.39	1.642	0.100
40.	Government Medical College, Nuh, Mewat	0.061	13.5	18	D	0.23	2.265	0.138
41.	Mini Secretariat, Palwal	0.061	15.0	17	D	0.25	2.081	0.127
42.	PHC Rampur Khor, Palwal	0.062	10.5	20	D	0.17	1.702	0.106
43.	PHC Alwalpur, Palwal	0.061	15.0	20	D	0.26	2.548	0.155
44.	Bridge at Sabilpur to Khet Purali road, Panchkula	0.235	30.0	32	D	0.47	0.700	0.165
45.	Steel plate girder bridge, Panchkula	0.179	41.0	61	C	0.41	1.279	0.229
46.	Bridge, Jattal road, near railway station, Panipat	0.068	24.0	16	D	0.44	1.851	0.126
47.	Bridge, Jind-Panipat Line, near railway station, Panipat	0.069	40.0	16	D	0.72	2.105	0.145
48.	220 KV Power house, Panipat	0.068	40.0	23	D	0.56	1.685	0.115
49.	Kabri TDI road, Panipat	0.070	30.0	22	D	0.46	1.582	0.111
50.	Thermal power plant, Panipat	0.065	20.0	15	D	0.35	2.260	0.147
51.	Sector 25, Panipat	0.069	45.0	15	D	0.64	1.716	0.118
52.	Kishanpura, Panipat	0.069	45.0	18	D	0.64	1.478	0.102
53.	Paliwal, Panipat	0.069	45.0	23	D	0.59	1.534	0.106
54.	Sector 6, Panipat	0.070	45.0	27	D	0.55	1.497	0.105
55.	Simla-Molana, Panipat	0.071	45.0	20	D	0.65	1.567	0.111
56.	Badauli, Panipat	0.072	45.0	29	D	0.53	1.573	0.113
57.	Sector 18, Panipat	0.071	30.0	25	D	0.35	1.731	0.123
58.	Bridge on canal, Jawahar Nagar, Panipat	0.068	30.0	18	D	0.53	1.687	0.115
59.	Bridge at Babbarpur- Kachroli road, Panipat	0.071	30.0	25	D	0.42	1.627	0.116
60.	Kharanti railway station, Rohtak	0.057	30.5	27	D	0.47	1.962	0.112
61.	Bridge at Rohtak-Bhiwani road, Rohtak	0.060	35.0	21	D	0.50	1.661	0.100
62.	Bridge at city Road, Gopal colony, Rohtak	0.059	30.0	19	D	0.55	1.919	0.113
63.	Bridge, Kath Mandi, Rohtak	0.060	40.0	20	D	0.65	1.839	0.110

Table 4. Contd...

Site No.	Location	PGA rock (g)	Depth of profile (m)	N_{30}	Site class	Site period (sec)	Amplification factor	Surface PGA (g)
64.	Bridge, near Suncity extension, Rohtak	0.060	30.0	25	D	0.46	1.565	0.094
65.	Bridge on road dividing sectors 26,27,28, Rohtak	0.061	35.0	21	D	0.62	1.726	0.105
66.	Minor bridge, near Rohtak city, Rohtak	0.061	40.0	23	D	0.60	1.618	0.099
67.	Bridge on Ghaggar river on Rania to Kutabudh road, Sirsa	0.065	30.0	19	D	0.55	1.629	0.106
68.	Bridge, near Swami Dayanand High School, Sirsa	0.066	30.0	43	D	0.36	2.065	0.136
69.	Bridge on Bhutana branch canal, Sonipat	0.054	40.0	26	D	0.57	1.789	0.097
70.	Bridge on Sunder branch canal, Sonipat	0.055	41.0	22	D	0.57	2.346	0.129
71.	Rasoi, Sonipat	0.070	45.0	27	D	0.70	1.836	0.129
72.	Ferozepur Khadar, Sonipat	0.070	45.0	27	D	0.52	1.597	0.112
73.	Rai, Sonipat	0.070	45.0	22	D	0.68	1.657	0.116
74.	Sector 31, Sonipat	0.070	45.0	34	D	0.68	1.672	0.117
75.	Sector 8, Sonipat	0.071	45.0	24	D	0.71	1.582	0.112
76.	Murthal, Sonipat	0.071	45.0	18	D	0.76	1.567	0.111
77.	Gulshan Dhaba, Sonipat	0.071	45.0	16	D	0.63	1.612	0.114
78.	Bridge on Sonipat Murthal road, Sonipat	0.069	32.5	26	D	0.47	1.284	0.089
79.	Barhi, Sonipat	0.070	45.0	30	D	0.66	1.612	0.113
80.	Bridge near Gohana railway station, Sonipat	0.057	30.0	23	D	0.49	2.000	0.114
81.	Buria Village, Yamunanagar	0.197	45.0	34	D	0.60	1.193	0.235

have been observed for the regions with higher expected earthquake hazard, i.e. for north and north-eastern parts of the state, as shown in Figure 14. This can be attributed to the fact that ground motions with high PGA deform the soft soil layers, which leads to the development of high strains. Further, most of the energy of ground motions is dissipated in deforming the soft soil layers and, therefore, lesser amplification is often observed in such areas. For an area falling in north-western Haryana, in and around Fatehabad district, high strains between 0.25 - 0.5% have been observed. For this area, as per PSHA, a very low PGA between 0.05g - 0.1g is expected during earthquakes. However, due to presence of soft/loose soil deposits, a high amplification factor ranging between 1.53 - 2.5 has been observed from ground response analysis, and the motions could get amplified to introduce high strains in the area. In high strain areas, possibility of earthquake induced settlements and liquefaction are indicated.

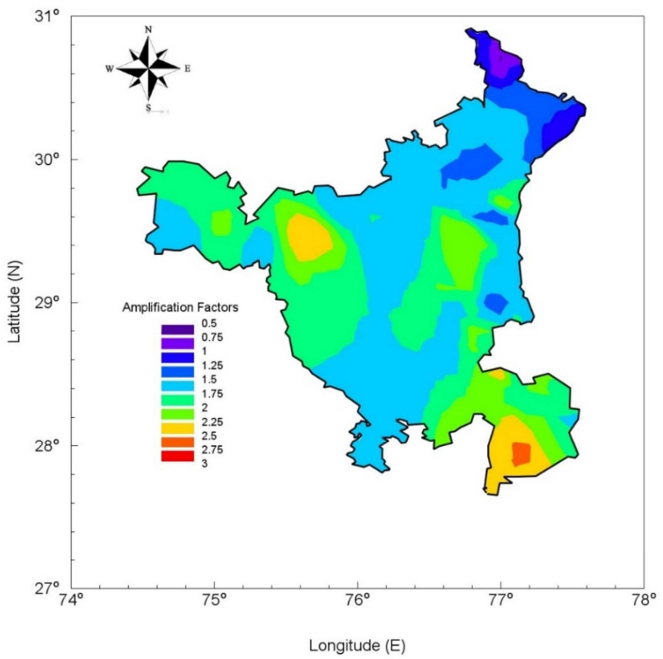

Fig.7. Variation of amplification factors across Haryana

Implications of the Input Parameters on Results

There are a number of factors which can impact the outcome of a computer based ground response analysis, such as, type of analysis (equivalent linear or non-linear), dimensionality, use of correlations for estimating G_{max} or using G_{max} from MASW field test results, use of standard G/G_{max}-γ and D-γ curves or site specific curves based on dynamic tests, type of bedrock and type of input motion used (recorded or simulated). Investigators often choose these input parameters based on their availability as well as feasibility.

There are a number of approaches available to estimate the degree of wave amplification, e.g. linear, equivalent linear and nonlinear analysis offering varying dimensionality (1-D, 2-D and 3-D). Schnabel et al. (1972) approximated the nonlinear hysteretic behavior of cyclically loaded soils by an equivalent linear model and developed SHAKE program. The computer program is now widely used for 1-D equivalent linear ground response analysis. Since soil is a nonlinear

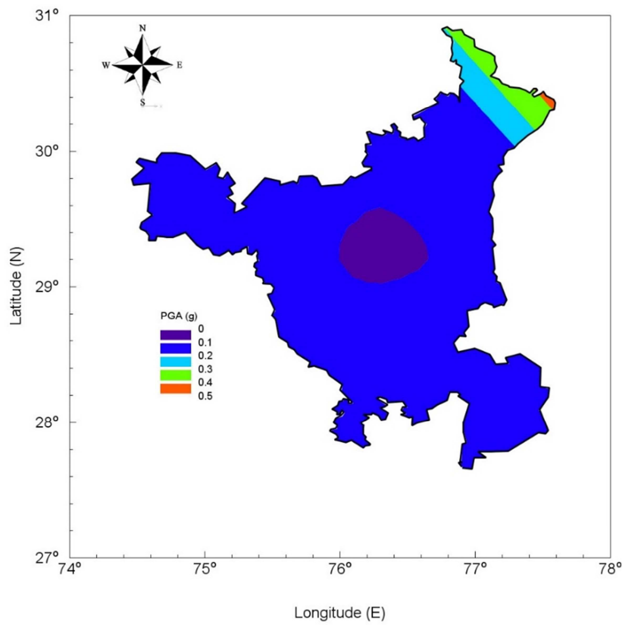

Fig.8. PGA map of Haryana for 10% probability of exceedance in 50 years for site class D

Table 5. Results of regression analysis

Summary Output								
Regression Statistics								
R	0.705							
R^2	0.498							
Adjusted R^2	0.477							
Standard Error	0.229							
Observations	77							
ANOVA								
	df	***SS***	***MS***	***F***	***Significance F***			
Regression	3.000	3.779	1.260	24.093	6.068E-11			
Residual	73.000	3.816	0.052					
Total	76.000	7.595						
	Coefficients Error	***Standard***	***t Stat***	***P-value***	***Lower 95%***	***Upper 95%***	***Lower 95%***	***Upper 95%***
Intercept	2.036	0.121	16.766	9.680E-27	1.794	2.278	1.794	2.278
X Variable 1	0.005	0.003	1.956	5.424E-02	-1.010E-04	0.011	-1.010E-04	0.011
X Variable 2	-0.013	0.003	-4.484	2.664E-05	-0.019	-0.007	-0.019	-0.007
X Variable 3	-2.248	0.337	-6.667	4.293E-09	-2.920	-1.576	-2.920	-1.576

material, shear modulus of the soil would vary constantly during cyclic loading. The incapability of equivalent linear model to represent true variation in soil stiffness that actually occurs during cyclic loading has been highlighted by many investigators, e.g. Finn et al. (1978), Arslan and Siyahi (2006), Hosseini and Pajouh (2012) and Kramer (2013). The nonlinear models have been found to better represent the response of soils to earthquake ground motions (Hosseini and Pajouh 2012). Also, for the sites with deep soft soils or sites where strong earthquakes are expected, the use of equivalent linear model is not considered as good practice (Hashash et al. 2010). Hence, the nonlinear approach for the assessment of ground response is preferred by most of the investigators. The standard nonlinear models that are being popularly used for analysis have been developed by Ramberg and Osgood (1943), Matasovic and Vucetic (1993) and Hashash and Park (2001).

The amplification of seismic waves for level or gently sloping sites with parallel material boundaries is generally evaluated using one-dimensional model, which assumes that the horizontal shear waves originating from bedrock propagate in a vertical direction through several layers of the soil profile. For some specific problems, the assumptions of 1-D wave propagation are not applicable, e.g. for steep or uneven ground surfaces, sites with large structures, buried structures, or walls and tunnels or pipelines, all require two-dimensional ground response analysis. The cases in which one dimension is significantly greater than the other can be solved as 2-D plane strain problem. Some important 2-D nonlinear ground response studies have been carried out by Larkin and March (1992), Takemiya and Adam (1998) and Soltani and Bagheripour (2018). The situations where soil conditions varies in 3-D and boundaries change in 3-D, e.g. earthfill dam in a narrow valley, soil structure interaction problems, and where response of one structure may influence response of another, a three-dimensional approach is more appropriate. 3-D ground response analysis is carried out just like 2-D analysis. Some important 3-D nonlinear ground response studies have been carried out by Chen et al. (2011) and Ichimura et al. (2014). Ground response problems involving 2-D and 3-D are most commonly solved using dynamic finite-element analysis and shear beam approach. Advanced constitutive models proposed by Mroz (1967), Momen and Ghaboussi (1982), Dafalias (1986), Kabilamany and Ishihara (1990) Gutierrez *et al.* (1993), Cubrinovski and Ishihara (1998) are generally preferred for the rare cases, dealing with 2D or 3D problems.

The accurate selection of G/G_{max}-γ and D-γ curves is extremely important for ground response analysis as they strongly influence the extent to which a soil deposit will amplify or attenuate seismic waves. However, determination of these dynamic soil parameters is quite tedious as equipment and expertise required for the same are not readily available. Moreover, for a big state like Haryana, it was also beyond the scope of the study to obtain site specific modulus degradation and damping ratio curves. Also, due to lack of availability of MASW equipment, Vs-N correlations have not been established. From the

Table 6. Amplification factors for Sa (g) corresponding to site class D

Period (sec)	Amplification factor for Sa (g)		Period (sec)	Amplification factor for Sa (g)	
	≤0.1g	> 0.1g		≤0.1g	> 0.1g
0.01	1.8	1.4	0.4	2.1	1.2
0.02	1.8	1.4	0.5	2.3	1.3
0.03	1.9	1.5	0.6	2.1	1.2
0.04	2.2	2	0.7	2.1	1.4
0.05	2.3	2.3	0.8	1.9	1.6
0.06	2.1	2.2	0.9	1.8	1.5
0.07	2	1.8	1.0	1.7	1.5
0.08	2.1	1.5	2.0	1.2	1.5
0.09	1.9	1.2	3.0	1.1	1.3
0.1	2	1.1	4.0	1.1	1.3
0.2	1.9	1	5.0	1.1	1.3
0.3	1.9	0.9			

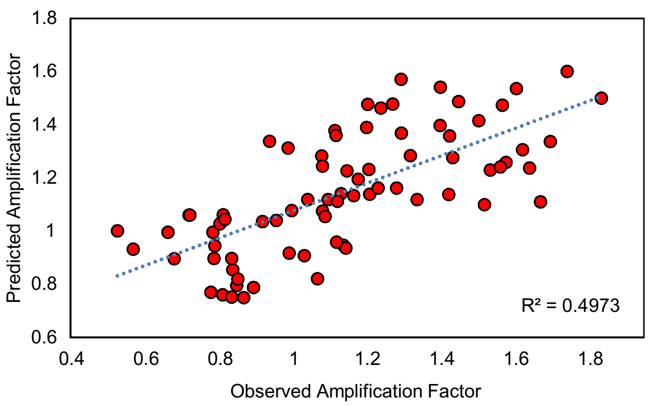

Fig. 9. Predicted vs. observed values of Amplification Factor

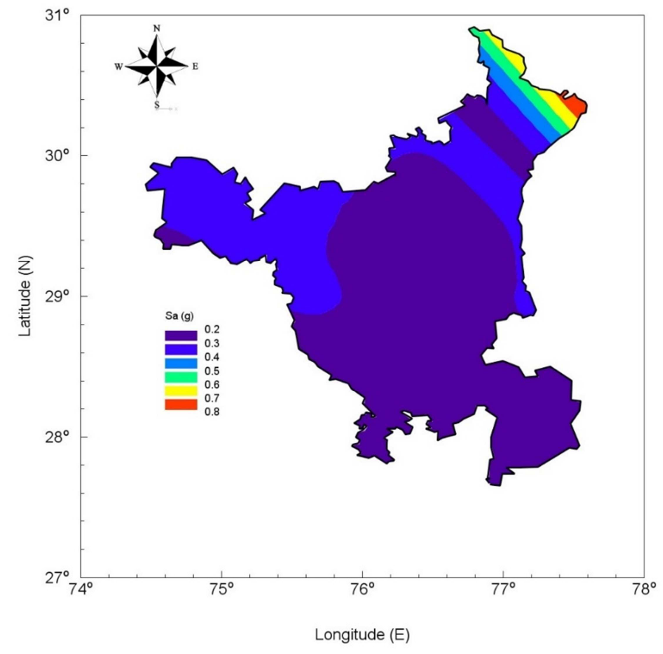

Fig. 10. Spectral accelerations (g) across Haryana for a return period of 475 year at 0.1 sec corresponding to site class D

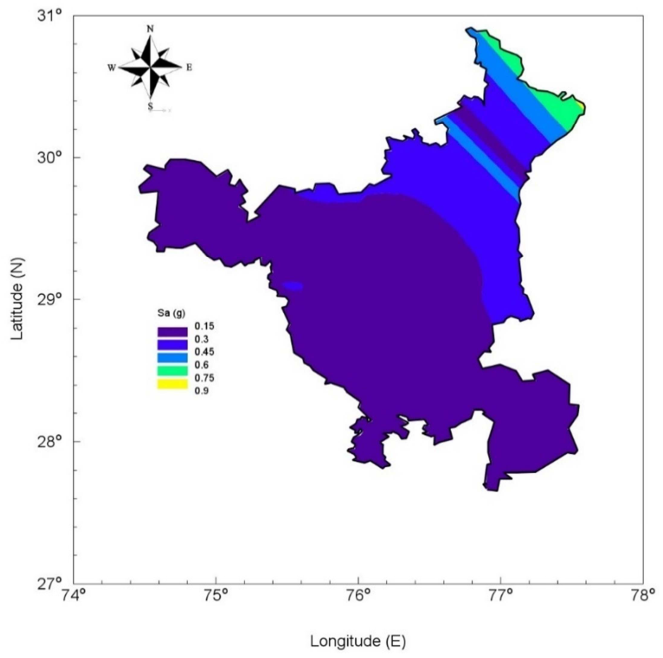

Fig. 11. Spectral accelerations (g) across Haryana for a return period of 475 year at 0.2 sec corresponding to site class D

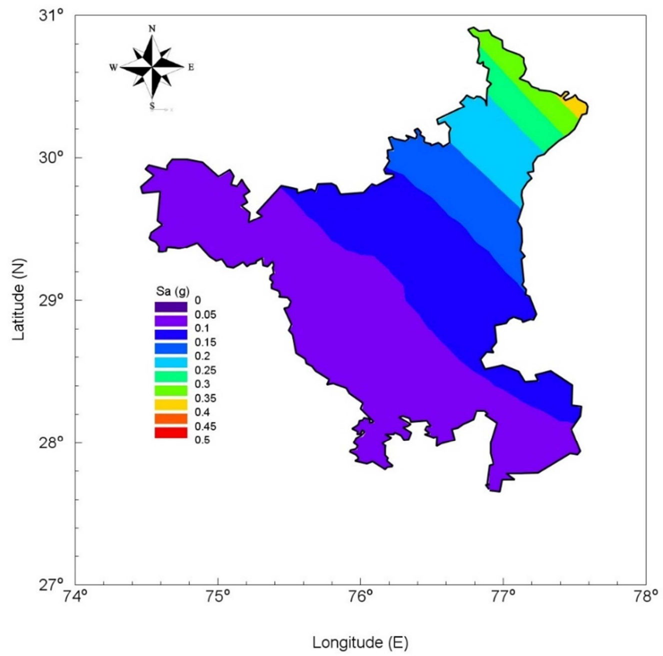

Fig. 12. Spectral accelerations (g) across Haryana for a return period of 475 year at 1.0 sec corresponding to site class D

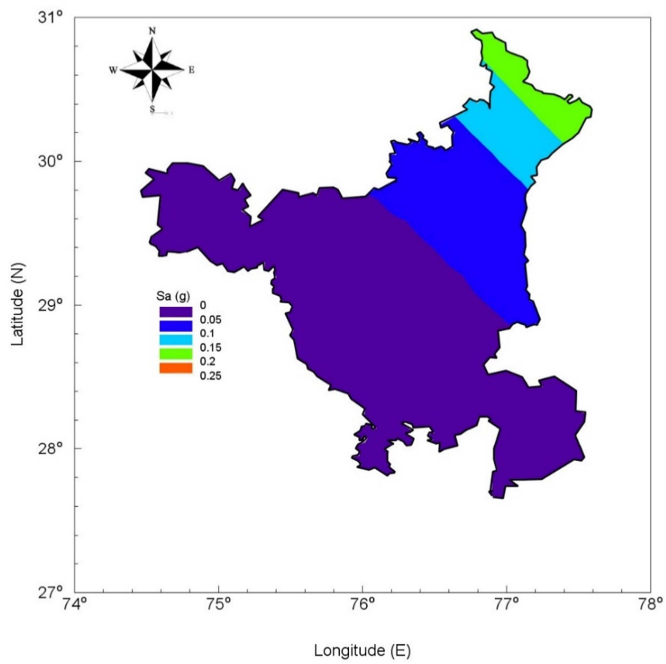

Fig. 13. Spectral accelerations (g) across Haryana for a return period of 475 year at 2.0 sec corresponding to site class D

available literature, it has been observed that the shear modulus of clays degrades much slowly than that of sands. The plasticity of clays has profound influence on the shape of shear modulus degradation curves, and it was first reported by Zen et al. 1978. Many other investigators, e.g. Sun et al. (1988), Vucetic and Dobry (1991), have also reported considerable influence of the plasticity index in comparison to void ratio, on the shape of G/G_{max}-γ curve. On the other hand, for the soils of low plasticity, effective confining stress influences the degradation behaviour of shear modulus (Iwasaki et al. 1978). The damping behaviour is also influenced by plasticity characteristics as observed by Kokushu et al. (1982). The damping ratio decreases with the increase in plasticity index for the same cyclic shear strain amplitude. However, damping behaviour of the low plastic soils depends upon effective confining pressure. The damping behaviour of gravels is quite similar to that of sands (Seed et al., 1986). Puri et. al. (2020) developed the site specific backbone curves for Yamuna Sands and have performed ground response analysis. Results from their study show that at all the sites, the curves proposed by Seed and Idriss (1970) and Darendeli (2001) underestimate the PGA at all depths. An increase in the calculated amplification factor over the proposed ones ranging from about 12 to 75% has been observed for the sites. The variation can be attributed to overprediction of strains at these sites by standard curves. The impact of using region-specific curves has been further investigated by plotting the response spectra for all

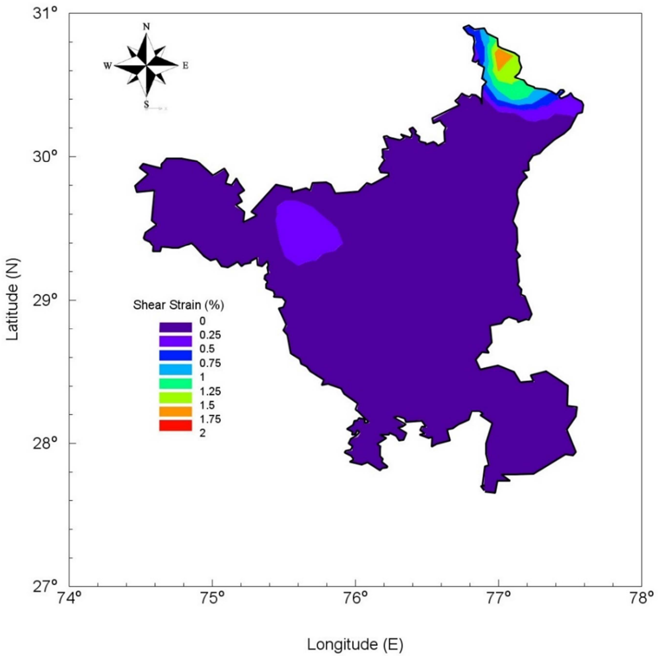

Fig. 14. Maximum shear strain (%) distribution in Haryana

the sites. It has been observed that the spectral acceleration also follows the same trend as that for PGA. For all the sites, Seed and Idriss (1970) and Darendeli (2001) curves, underestimate the response almost at all periods.

Shear modulus (G_{max}) plays a fundamental role in characterization of the stiffness of soil in ground response analysis. Suitable correlations can be selected based on soil type, age of deposits, methodology adopted for SPT test data used to develop correlations. Investigators need to be cautious while selecting a correlation from the literature as any wrong assumption while selecting a correlation could lead to spurious results. The existing correlations available in the literature have been reviewed and assessed in depth by Anbazhagan et al. (2010, 2012, 2015, 2016). A number of correlations suitable for Indian site conditions have also been suggested.

The location of bedrock needs to be defined as it sets a boundary condition for the equation of motion. The boundary condition indicates that the behaviour of strata below the bedrock does not affect the result of ground response analysis. This also means that an earthquake motion at the engineering bedrock can be assumed as an incident wave, and it cannot be affected by the behaviour of strata below. The wave can reflect back into the bedrock and the boundary condition can also consider radiation damping. An engineering bedrock following such a boundary condition is called as an elastic bedrock. As per the recommendations of Pacific Earthquake Engineering Research Center (PEER), a rock outcropping motion should be applied without any modification, for an elastic base, for time-domain analyses, or a within motion should be used without modification in conjunction with a rigid bedrock (Stewart and Kwok, 2008).

There are two types of earthquake time histories available for dynamic analyses; recorded and simulated earthquake time histories. But it is not advisable to use time histories other than the recorded ones in non-linear ground response analysis. This is so because in spectrum compatible time histories, the velocity and displacement component often get distorted, and the energy content is greatly exaggerated, which leads to overprediction of response. Moreover, there is no representative natural earthquake time history recorded in Haryana, which can justify the expected dynamic loading, and can be used as a good seed record without expecting the unrealistic changes in the velocity, displacement and energy components. Since, the earthquake hazard in Haryana is mostly associated with Himalayan Thrust System due to the proximity to the area, recorded earthquake time history of a Himalayan earthquake has been used in the present study.

It has been observed that for India, the earthquake acceleration time histories recorded only at outcrop are available. The acceleration time history of 1995 Chamba earthquake has not been used in the study considering that the earthquake was of low magnitude, and records are available for soil outcrop. The 2001 Bhuj earthquake was recorded in a building at a soil site and therefore, it has not been considered. There is only one record available for 2011 Sikkim earthquake, and hence it cannot represent dynamic loading expected at all the sites selected for ground response analysis. The number of recordings available for 1999 Chamoli earthquake are insufficient to represent the expected earthquake motions in Haryana. Therefore, the recorded acceleration time histories of the M_s 7.0 1991 Uttarkashi earthquake, having a focal depth of 10 km, with PGA (g) ranging from 0.05g to 0.31g, have been used as input motion for the analysis.

CONCLUSION

The amplification of strong ground motions due to local site conditions is an important aspect that needs to be incorporated in the design of earthquake resistant structures. Considering the tendency of soil to behave nonlinearly at high strains, nonlinear approach has been adopted to carry out seismic ground response analysis for various sites in the state of Haryana, India. Geotechnical site characterization based on NEHRP provisions indicates that most of the sites fall under site class D representing medium soils. Appropriate correlations for estimating maximum shear modulus, and curves for shear modulus degradation and damping ratio have been selected after extensive review of literature. An elastic bedrock has been assumed to be present at refusal condition of the SPT split-spoon sampler. The rock outcrop motion of the M_s 7.0 1991 Uttarkashi earthquake has been taken as the input in the analysis as the recorded parameters are compatible with those calculated for the study area. The average amplification factors of 1.81 for $PGA_{rock} \leq 0.1g$ and 1.30 for $PGA_{rock} > 0.1g$ have been calculated for site class D. The calculated amplification factors for PGA and Sa are comparable with those reported in NEHRP provisions. These factors have been used to modify the seismic hazard maps developed for rock sites at 10% probability of exceedance in 50 years. For the north and north-eastern parts of the state with higher expected earthquake hazard, development of high shear strains ranging from 0.01% to 2% have been observed, and the possibility of earthquake induced settlements and liquefaction are indicated.

Acknowledgement: Authors would like to thank Mr. Bob Simons, Employee, CoHort Software for providing CoPlot and CoStat softwares for scientific graphing and mathematical analysis. Sincere thanks are due to Officials of Public Works Department (PWD) Haryana, Northern Railways (NR), Haryana Urban Development Authority (HUDA) and Delhi Metro Rail Corporation (DMRC) for providing all the assistance for collection of geotechnical data for the state of Haryana. Also, many thanks are due to Dr. Ghumman & Gupta Geotech Consultants (Chandigarh), Jindal Consortium (Ambala), Mananda Test House (Dera Bassi, Mohali) and Magma Infrastructures Private Limited (Delhi) for providing several geotechnical reports of the state.

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(Received: 30 November 2020; Revised form accepted: 3 May 2021)