Nat Hazards (2017) 88:1741–1768
DOI 10.1007/s11069-017-2944-x

ORIGINAL PAPER

Selection of representative shear modulus reduction and damping curves for rock, gravel and sand sites from the KiK-Net downhole array

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Received: 25 July 2015 / Accepted: 31 May 2017 / Published online: 22 June 2017

Abstract Representative computation of ground response parameters requires accurate information about nonlinear dynamic behavior of the soil column, commonly incorporated in site response analysis through shear modulus reduction and damping curves which are functions of the strain level. Most site response studies are carried out by considering a set of existing shear modulus and damping curves, without knowing its suitability for the in situ soil type. In this study, an attempt has been made to identify suitable shear modulus and damping curves for soil grouped into different classes viz. sand, gravel and rock. Soil profiles of sites having surface and bedrock motion recordings are selected from the KiK-Net downhole array database and equivalent linear and total stress nonlinear site response analysis has been carried out by varying the shear modulus and damping curves for different sites. Estimated surface response spectra for each set of shear modulus and damping curves are compared with the observed response spectra at each site, and a detailed analysis is made to find out which set of curves gives a best match with the recorded data. Based on this study, representative property curves for rock, gravel and sand are suggested, which could be used for further site response studies in the region. This study shows that only a set of shear modulus and damping curves ensure a compatible spectrum with the recorded data from the KiK-Net downhole array sites, among the many available shear modulus and damping curves in the literature.

Electronic supplementary material The online version of this article (doi:10.1007/s11069-017-2944-x) contains supplementary material, which is available to authorized users.

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Keywords Soil dynamics · Shear modulus and damping curves · Site response · DEEPSOIL · KiK-Net

1 Introduction

The modification of seismic waves as it propagates through soil stratum is a well-observed and documented issue since over a century (Milne 1898). Local site effects play a crucial role in focusing the energy density of the ground motions to a range of frequencies, that often proves destructive to many structures. Apart from the ground shaking hazard alone, these effects also trigger hazards due to the ground shaking, including the cyclic mobility and flow liquefaction hazards. (Kramer 1996). Evident with the destruction caused in the 1987 Mexico City earthquake, where long-period structures founded on soft lacustrine soil suffered intense damages (Seed et al. 1987). Site effects were studied extensively for many earthquakes such as the Loma Prieta earthquake (Seed and Sun 1989), 1995 Kobe earthquake, 2001 Bhuj Earthquake, 2010 Canterbury earthquake and the recent Nepal earthquake (Moss et al. 2015; Mugnier et al. 2011). The estimation of the in situ dynamic properties of soil deposits is of prime importance in studying the response of structures founded on such sites.

The incorporation of site effects, in studying the earthquake response of structures, has been done through amplification factors in existing literature (Choi and Stewart 2005; Boore and Atkinson 2008) integrated into various ground motion prediction equations (Boore et al. 2014 (NGA WEST GMPEs Project) and in various code provisions (BSSC 2001; CEN 2004) as functions of the 30 m average shear wave velocity (V_s^{30}). For several important projects, site-specific data could also be used to perform detailed site response analysis for the site, where the wave propagation equation is solved for a site condition and a ground motion. Among the methods, for large-scale projects, site response analysis is often used to estimate the ground motion characteristics at the surface of the strata, after the seismic waves pass through the deposit, as it can physically model aspects of the wave propagation problem unlike the other empirical studies, including effective stress models, pore water pressure generation and dissipation models. In actual practice, only the one-dimensional (1D) wave propagation equation is usually solved as evident with the existence of various site response software viz. Shake91, DEEPSOIL, EERA, STRATA. Even though site response study and amplification estimation essential most of modern codes, site response studies are started in India after Bhuj earthquake 2001 and then several researchers carried out ground response analysis for many cities in India. Govindaraju et al. (2004) and Shukla and Choudhury (2012) for Gujarat; Mahajan et al. (2007) for Dehradun; Boominathan et al. (2008) for Chennai; Mohanty et al. (2007) and Kamatchi et al. (2008) for Delhi; Anbazhagan and Sitharam (2008) for Bengaluru, Govindaraju and Bhattacharya (2008) for Kolkata region, Raghu Kanth et al. (2009) for Guwahati, and Kumar et al. (2012) for Lucknow, Uttar Pradesh. Mahajan et al. (2007) preformed 50 MASW surveys at different locations across the Dehradun and performed site response analyses yielding amplifications of peak ground acceleration (PGA) ranging from 1 to 4 in different parts of the city. Anbazhagan and Sitharam (2008) performed the site response analysis in Bengaluru region and performed one-dimensional site response analyses obtaining amplification factors varying from 1 to 4 in the PGA. Boominathan et al. (2008) performed the ground response analyses for Chennai city and reported amplification factors between 4 and 6. Govindaraju and Bhattacharya (2008) performed the site response analysis based on the equivalent linear approach for the city of Kolkata. More than 100 boreholes collected from various agencies to understand the subsoil lithology variation for the city of Kolkata. Ground response analysis for these soil columns by Govindaraju and Bhattacharya (2008) showed amplification factors were in between 4.4 and 4.8. Kamatchi et al. (2008) performed site-specific analysis of Delhi region and reported the amplifications of the sites between 2.02 and 3.3. Hanumantha Rao and Ramana (2009) carried out response studies for sites in Delhi, using synthetic motions and the corresponding acceleration response spectrum at four different site classes (Site class A, B, C and D) as per IBC (2009).

Phanikanth et al. (2011) carried out response studies of the typical sites in Mumbai considering 3 sites varying up to rock depth resulting in amplification factors varying from 1.07 to 3.35. Similar studies were performed by Shukla and Choudhury (2012) carried out response studies of the four ports (Kandla, Mundra, Hazira and Dahej) sites in Gujarat using three synthetic time histories. Kumar et al. (2012) carried out site response studies for different locations in Lucknow city. Response parameters at ground surface were estimated by nonlinear site response analysis considering multiple input ground motions given at very dense soil layer having V_s of 760 ± 60 m/s at 29 locations, and the authors found that amplification factors are in the range of 3.5–5.54 in the northern parts of the city. However, the successful application of such site response study in specific cases is essentially dependent on the incorporation of the representative soil properties in the response analyses (Seed et al. 1986; Bradley 2011; Thompson et al. 2012) and the calibration of the program used in the analysis.

In this respect, downhole arrays provide a valuable comparison tool in assessing the capability of such programs and the assumptions used in implementing them. Downhole arrays contain a pair of seismograms with which they are usually coupled and triggered in the event of earthquakes. Various studies have been undertaken at vertical downhole array sites to obtain the in situ soil conditions through system identification (Elgamal et al. 1995; Pecker 1995; Glaser 1995, 2006; Elgamal et al. 1996; Assimaki and Steidl 2007; Assimaki et al. 2006), in evaluating the dynamic behavior of the soil (Zeghal et al. 1995; Zeghal and Elgamal 1994; Zorapapel and Vucetic 1994; Zeghal et al. 1996; Assimaki et al. 2008a, b, 2011; Glaser and Baise 2000; Chang et al. 1996) and for studying the effectiveness of various computational models of site response (Kwok et al. 2008; Seed and Idriss 1970; Idriss and Seed 1970; Johnson and Silva 1981; Shearer and Orcutt 1987). Elgamal et al. (2001) provides detailed insight into the various downhole arrays operating over the world and the various practices of system identification for obtaining measures of site amplification for such deposits. Bradley (2011) proposed a framework to assess the validity of the computational models used to compute seismic site response underlining the common sources of uncertainty in estimating the response, including uncertainties associated with the site characterization, constitutive model parameter estimation, constitutive model selection and computational model methodology. This was underlined in Thompson et al. (2012), where extensive linear site response analysis was performed using low-amplitude ground motion on the sites in the KiK-Net (Kiban-Kyoshin Network) array, delineating 16 profiles, where the assumptions of one-dimensional SH (horizontally polarized shear wave) propagation assumptions are valid and the deposit could be modeled without assuming three-dimensional effects. These profiles were further used by Kaklamanos et al. (2015) in comparing equivalent linear and nonlinear total stress site response models where a heavy dependence was observed in the assumed modulus reduction and damping curves. Kaklamanos et al. (2013) identified the critical parameters affecting the bias and variability of the site response analysis using strong motion data.

Since the behavior of soil during dynamic loading is a nonlinear function of numerous exogenous variables including stress history, soil strata, etc., which are not ascertained before the response, the most common method to study the soil response is assuming that the tip of the stress–strain path the soil undertakes follows an established modulus reduction curve and the associated increase in hysteretic damping with shear strain to follow another specified curve. Therefore, the representative dynamic shear modulus degradation and damping ratio behavior with strain are required for accurate computation of ground response parameters, which are incorporated through the equivalent shear modulus and damping curves in site response analysis. The main objective of the study is to identify suitable shear modulus reduction and damping curves for sand, rock and gravel soil sites, through parametric investigations considering pairs of bedrock and surface recorded ground motion data with the respective soil profiles from the Kiban-Kyoshin Network. Soil profiles of sites having pairs of surface and bedrock motion recording are selected from the Kiban-Kyoshin Network (KiK-Net, http://www.kyoshin.bosai.go.jp/), and detailed equivalent linear and nonlinear site response analysis has been performed with the rock recorded earthquake ground motion data as input, which removes the possibility of mixing site effects with the other source and path effects, and the response spectrum is evaluated for each site by parametrically varying the modulus reduction and damping curves for the sites.

The evaluated surface response for each set of shear modulus and damping curves is compared with the recorded data at the surface. Comparison is mainly concentrated to identify the best match in the residual between the observed and calculated response spectra to quantify the misfit. Based on the results of the analysis with real ground motion data and available borelog details, appropriate curves for a particular soil class are suggested. Shear modulus and damping curves suggested for different soil columns in this study may be useful for estimating representative ground response parameters at sites where no site-specific shear modulus and damping curves are available in the Japanese sites.

2 Site and ground motion selection from the KiK-Net database

The soil profiles analyzed are obtained from the Kiban-Kyoshin network (KiK-Net, K-Net, http://www.kyoshin.bosai.go.jp/). The KiK-Net and K-Net arrays were commissioned in 1996, operated by the National Research Institute for Earth Science and Disaster Prevention (NIED) in the aftermath of the 1995 Kobe earthquake. The array consists of more than 1000 observation stations of which 700 have downhole and surface high-quality seismographs, which form the KiK-Net array. At these respective sites, surface-source downhole receiver logging is performed to obtain the shear wave velocities at the respective sites, along with the information regarding the major type of soil at each depth and its period of its origin. Pairs of acceleration time histories for the both horizontal components at the site are collected from the KiK-Net database for each of the profiles selected. The processing of ground motions is done using the methodology developed in Dawood et al. (2016) using a high-pass fourth-order acausal butterworth filter implemented as per Boore and Bommer (2005) through the Boore Fortran Programs (TSSP). The corner frequencies are selected from the flatfile, through the procedure developed from Dawood et al. (2016) obtained from the corresponding NEES flatfile for all the KiK-Net sites.

2.1 Selection of shear wave velocity profiles

Thompson et al. (2012) extensively studied the KiK-Net downhole arrays and used 100 sites with 4862 ground motions recorded from 1573 earthquakes, filtering them to an accelerations less than 0.1 g at the surface, to identify 16 locations where the conditions of one-dimensional horizontally polarized shear wave propagation (1D SH) and 1D site response analysis hold, terming these sites, for low intraevent variability and good fit to the 1D SH assumptions (LG). Apart from these 16 identified profiles, Thompson et al. (2012) also identified 53 sites, which had low interevent variability, but a poor fit to the 1D SH wave propagation assumption (LP), for low intraevent variability and poor fit to the 1D SH wave propagation assumption, stating that these profiles required nonlinear modeling, and care had to be taken to identify the source of heterogeneity. From the 100 profiles mentioned in

Thompson et al. (2012), profiles containing predominantly sand or gravel lying over rock are selected in indentifying the best shear modulus reduction and damping curves from a family of commonly used curves, which could be used for further site response studies in the region.

For predominant rock profiles, categorized as LG, 15 and 10 pairs of acceleration time histories are used, respectively, for the IWTH27 and IWTH08 profiles. For the rock LP sites, 5 pairs of low-amplitude ground motion are used, respectively, for IWTH05 and FKSH18 for calibration and 10 pairs of records are used each, for site response analysis. In the case of predominant gravel profiles, FKSH11, TCGH12, TKCH08, 10 records are selected each for the site response analysis, and in case of sand, 10 pairs of ground motions are selected each for KSRH05, FKSH08, TCGH15, respectively. Summary of all sites considered and ground motions selected is given in Table 1.

In this study, it has been assumed that the source of misfit to the 1D SH wave propagation assumption is the soil heterogeneity and shear wave velocity structure. Though the shear wave velocity structure for the KiK-Net sites is obtained by downhole logging and peak picking to obtain travel times, which are not subjected to a large degree of uncertainty, in this study, we assume that the shear wave velocity structure for such sites is inaccurate. Thompson et al. (2012) note this and compared the results of V_s profiles estimated through spectral analysis of surface waves and from the KiK-Net database and observed differences in the shear wave velocity structure at the vicinity of the profiles.

To address this issue, detailed linear site response analysis has been carried out at the LP sites, through Monte Carlo simulations varying the shear wave velocity structure of the site, with low-amplitude ground motions (PGA $\sim$ 0.005 g), recorded in the downhole sensor. On obtaining the ground motion response spectrum at the top of the deposit for each of the simulated shear wave velocity sites, the Pearson's correlations coefficient (R^2) is used to compare the geometric mean of the calculated and observed response spectra at the surface of the deposit. The profiles are ranked according to their correlation coefficients and the velocity profile which is most similar to the seed profile as obtained from the KiK-Net database is studied further. The variation of small strain damping with depth is a major factor in linear site response analysis, and various authors (Park and Hashash 2005) have provided different methodologies to obtain estimates of small strain damping profile of the site. To obtain estimates of the small strain damping, the profile obtained from the Monte Carlo simulations is studied further and the small strain damping has been varied in each layer of the deposit. The seed value of the small strain damping is provided as the average

Table 1 Description of the soil profiles used in the study

Station	Predominant soil type	Z_r	Z_s	n_{eval}	n_{calib}	V_s^{30} (m/s)	Site class as per NEHRP2000	Class as per Thompson et al. (2012)	Maximum PGA at Z_r (g)	Range PGA at surface (g)
IWTH05	Rock	103.3	–	20	10	429	C	LP	0.17	0.09–0.81
FKSH18	Rock	103.0	–	20	10	307	D	LP	0.04	0.05–0.35
IWTH08	Rock	103.0	–	20	–	305	D	LG	0.04	0.03–0.37
IWTH27	Rock	103.0	–	30	–	670	C	LG	0.14	0.05–0.76
FKSH11	Gravel + Rock	118.2	35.0	20	–	240	D	LG	0.12	0.02–0.27
TCGH12	Gravel + Rock	123.0	106.0	20	–	344	D	LG	0.05	0.04–0.16
TKCH08	Gravel + Rock	103.0	78.0	20	–	353	D	LG	0.10	0.05–0.43
KSRH05	Sand + Rock	333.10	137.0	10	10	389	C	LP	0.11	0.02–0.29
FKSH08	Sand + Rock	108.00	50.50	10	10	563	C	LP	0.04	0.04–0.13
TCGH15	Sand + Rock	303.3	20.7	10	10	423	C	LP	0.02	0.04–0.12

Z_r, depth of downhole sensor (m); Z_s, thickness of soil (m); n_{calib} indicates the number of ground motions used to perform linear site response analysis for LP Profiles; n_{eval} indicates the number of ground motions used to perform nonlinear site response analysis; V_s^{30} is the 30 m average shear wave velocity small strain damping value for the respective profile as provided by Kaklamanos et al. (2013).

The shear wave velocity profiles are identified from the mentioned 16 LG sites and calibrated 53 LP sites for identifying the corresponding shear modulus reduction and damping curves which fit the model the best, in comparison with the surface recorded time histories. Rock profiles are identified in these sites amounting to predominantly rock sites among the 16 sites identified as having low interevent variability and a good fit to the 1D SH wave propagation assumptions (LG) and 3 sites among the 53 identified as having low interevent variability between ground motion records and a poor fit to 1D SH wave propagation assumptions (LP) as studied by Thompson et al. (2012). For the gravel soil class, 3 profiles are selected from the 16 LG identified profiles, and for the sand soil class, 4 profiles are selected from the 53 LP sites. Summary of the profiles selected with the class is provided in Table 1.

The obtained shear wave velocity profiles are used as an input for site response analysis, performed using DEEPSOIL (Hashash et al. 2015). Typical shear wave velocity profiles of the sand, rock and gravel soil class are shown in Fig. 1. Along with the V_s values, density values are required for calculating the low-strain shear modulus. Kaklamanos et al. (2013) used the Boore (2007) correlation of density with P wave velocity as in situ measurements were not available in the KiK-Net database.

A widely used correlation between in situ density and shear wave velocity is the relation developed by Gardner et al. (1974). This relationship is only valid for sites having a velocity of more than 1524 ms^{-1}. Anbazhagan et al. (2016) compared the developed correlation with studies by Gardner et al. (1974), Boore (2007) and correlations by Inazaki (2006) and found good agreement between all the relations. In this study, the density of each layer is estimated as follows, using the shear wave velocity and density correlation suggested by Anbazhagan et al. (2016).

$$\rho = 0.52 V_s^{0.2} \tag{1}$$

where V_s is the shear wave velocity in m/s and density is in g/cc.

2.2 Site response analysis

In this study, DEEPSOIL (Hashash et al. 2015) has been used to perform both the nonlinear and equivalent linear total stress site response analysis for the final identified shear wave velocity profiles. For calibration of the LP shear wave velocity profiles, STRATA has been

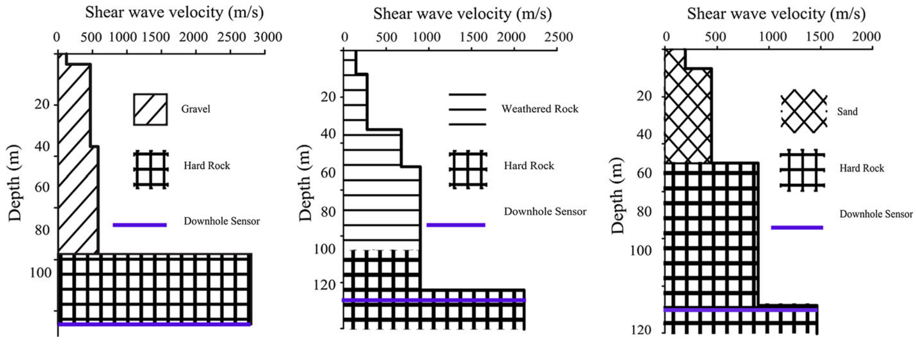

Fig. 1 Typical shear wave velocities of soil columns of rock (IWTH08), gravel (TKCH08) and sand (FKSH08), from the KiK-Net database

used to perform the Monte Carlo trials and linear site response analysis using low-amplitude ground motions. DEEPSOIL discretizes the entire 1D soil column into lumped multidegree of freedom elements with individual model parameters. The nonlinear behavior of soil is captured through the pressure-dependent hyperbolic model for the backbone curve, developed by Konder and Zelasko (1963), modified by Matasovic (1993). The unloading and reloading formulations are based on the extended Masing rules (Hashash et al. 2015), and the within boundary condition is used at the depth of the downhole sensor.

The shear modulus reduction and damping curves were used to fit the modified hyperbolic model using the MRDF-UIUC procedure developed by Phillips and Hashash (2009). Many authors including Hashash et al. (2010) and Stewart and Kwok (2009) have proposed modifications to the hyperbolic relationship to obtain reasonable estimates of shear strength post-fitting the model. Groholski et al. (2015) proposes a generalized quadratic/hyperbolic strength controlled model to address this issue, and the module has been implemented in DEEPSOIL. In this study, the formulation by Hashash et al. (2010) is used to obtain estimates of shear strength. Since, in downhole arrays, the static shear strength is not known beforehand, correlations between the shear wave velocity and undrained shear strength suggested by Dickenson (1994) are used. Frequency-independent Rayleigh damping is used to model the small strain damping as suggested by Phillips and Hashash (2009). Dependence of overburden pressure on the behavior of the modulus reduction curve and small strain damping is modeled through two coefficients in DEEPSOIL.

Pairs of input ground motion has been given at recorded depths of the downhole sensor. As information on water table is unavailable, pore water effects are not considered and total stress analysis is carried out. Geometric mean of the pair of response spectra obtained at the surface from the analyses was compared with that obtained from recorded motion. The residuals of spectral acceleration for the ith ground motion were calculated between 0.02 and 0.6 s as,

$$\delta(T)_i = \ln(S_a(T))^i_{\text{recorded}} - \ln(S_a(T))^i_{\text{predicted}} \tag{2}$$

The bias, i.e., mean (μ_i) and standard deviation (σ_i) of the residuals, was used to quantify the fits between multiple shear modulus reduction and damping curves within a soil class. A residuals plot is used for quantitative estimate of the applicability of the model, as the model with residual having zero mean and less standard deviation is more compatible. The set of shear modulus reduction and damping curves which provided the least values of mean and standard deviations across multiple ground motions were selected as the best-fit selection for that particular soil class.

3 Summary of shear modulus reduction and damping curves

Over the years, several researchers have presented the variation of shear modulus and damping ratio values with shear strain for different materials. Of the several available modulus reduction and damping curves available for different soil types from existing literature, a set of curves are popularly used in the site response analysis. Widely used shear modulus and damping curves were developed by Seed and Idriss (1970), EPRI (1993), Vucetic and Dobry (1991), Darendeli (2001), Ishibashi and Zhang (1993), Seed et al. (1986), Sun et al. (1988) and Rollins et al. (1998) for representing the dynamic behavior of the soil column.

For shallow profiles, certain shear modulus reduction and damping curves have been widely used in the literature as they required simple input parameters to be used in the site response analysis. In case of deeper profiles, Hardin and Drnevich (1972), Kokusho (1980) and several other researchers recognized the effect of confining pressure on dynamic soil properties as the most significant for granular profiles. Hashash and Park (2001) used the measured results of Laird and Stokoe (1993) and Seed and Idriss (1970) and from the computational model of Ishibashi and Zhang (1993) to study the dependence of overburden pressure on the dynamic soil properties and observed that the Laird and Stokoe measured curves were the most appropriate for calibration of their nonlinear pressure-dependent hyperbolic soil model used in DEEPSOIL. Hashash and Park (2001) also studied the influence of confining pressure on the 1D site response analysis noting the differences in the response of the pressure-independent and pressure-dependent nonlinear hyperbolic soil models for soil columns of 100, 500 and 1000 m depth, and it was reported that the differences in the pressure-independent and pressure-dependent models were similar for the 100 m soil column and the pressure-dependent model shows larger amplitudes for shorter periods. Though it was reported that the differences in the models were significant even for the 100 m soil column by Hashash and Park (2001), we have adopted the pressure-independent model as the former curves reported in the literature viz. Seed and Idriss (1970) do not have specific information on the effect of confining pressure on the modulus reduction and damping properties. Zhang et al. (2005) highlighted that strain (γ), mean effective confining stress (σ'_m), soil type and plasticity index (PI) are the most important factors that affect the ratio of shear modulus ($G/G_{\max}$). Other factors which affects the $G/G_{\max}$ curve but appear to be less important include the frequency of loading, number of loading cycles, over-consolidation ratio, void ratio, degree of saturation and grain characteristics (Darendeli 2001; Zhang et al. 2005). The most important factors that affect damping ratio (ξ) are strain (γ), σ'_m, soil type and PI, frequency of loading, and number of loading cycles. With the increase of σ'_m, ξ tends to decrease for all strain amplitudes.

In the absence of the site-specific shear modulus reduction and damping ratio variation with shear strain, site response studies are carried out by considering a well-known set of curves for the same broad soil class. Over the years, several researchers have used these curves for site response studies. Hanumantharao and Ramana (2008) carried out studies to find dynamic properties of Delhi soil through cyclic triaxial tests. Kamatchi et al. (2008) used Vucetic and Dobry (1991) curves for site-specific analysis of the Delhi region. The Seed and Idriss (1970) average curve for sand was used by Anbazhagan et al. (2010) for the microzonation of Bangalore. During the seismic hazard assessment of Chennai city, Boominathan et al. (2008) used Sun et al. (1988), Seed and Idriss (1970) and Idriss (1990) curves for clay and sand, respectively, and the Schnabel (1973) curve of the rock and carried out ground response analysis on 38 representative sites. Shukla and Choudhury (2012) studied site-specific responses of port sites in Gujarat and used Sun et al. (1988) and Vucetic and Dobry (1991) curves for clay, Seed and Idriss (1970) average, Seed et al. (1986) and Idriss (1990) upper curves for silty sand and the Roblee and Chiou (2004) curve for silty sandy gravel. Govindaraju and Bhattacharya (2012) used the Vucetic and Dobry (1991) curves for response studies in Kolkata region. Kumar et al. (2012) carried out response studies of deep sites in the Indo-Gangetic basin considering the curves from Sun et al. (1988) for clay and Seed and Idriss (1970) for sand. Karastathis et al. (2010) used EPRI (1993) for evaluation of nonlinear site response in Greece. Hartzell et al. (2004) also used EPRI (1993) sand and rock curves for sand and Vucetic and Dobry (1991) curves for clay while studying the nonlinear effects in Seattle area.

In actual site conditions, soils exist as a mixture of gravels, sands, silts and clays. It is hence imperative to understand the performance of such soils under dynamic loading. Various researchers have attempted to study the performance of such soils during dynamic loading, including Hanumantharao and Ramana (2008), Okur and Ansal (2007), Lee and Sheu (2007), Wang and Kuwano (1999) and Yamada et al. (2008a). Hanumantharao and Ramana (2008) performed tests using various mixtures of Yamuna sand and non-plastic fines. Different proportions of fines (15, 30 and 50%) were mixed with clean Yamuna sands to obtain the soil mixtures available in different parts of the Delhi to study the response of soil without plastic fines, and it was observed that the modulus reduction and damping curves obtained were similar to the Seed and Idriss (1970) lower bound curve. Okur and Ansal (2007) studied samples from Turkey post the 1999 Kocaeli Earthquake which contained ML, MH, CL and CH soil types and confirmed that PI was the governing factor in both the modulus reduction and damping curves as explained by Vucetic and Dobry (1991) and developed semi-empirical correlations to study the dynamic properties of such soils.

The dataset used by Rollins et al. (1998) consisted of both poorly graded clean gravels and gravelly sands, with few well-graded gravels, viz. GP, SP and SW of the USCS classification system. The mean values reported in the gravel curve are 10–30% lower than the Seed and Idriss (1970) curves with a slight overlap. Due to the similarity in the curves, additional tests were performed to study the behavior of both gravelly sands and sandy gravels, which are commonly observed in field conditions. Evans and Zhou (1995) summarizes the specimen procedure, and it is seen that on increasing gravel content, there is only a slight increase in the normalized shear modulus, which pushes the 60% gravel curve further into the sand range.

For cohesive soils containing larger amounts of sand content, Yamada et al. (2008a) performed a series of cylindrical torsional simple shear tests to study the performance of soils with a wide range of grain sizes and plasticities. Marine clays were mixed with silica sands at various proportions to study the dynamic response of soils. With increase in fines content, measured through the equivalent plasticity index (I_P^*), the behavior of sand-clay mixtures changes from that of clays to that of sands as fines decreases. Given the value of equivalent plasticity index, where $I_P^* = I_P * R$, where I_P is the plasticity index and R refers to a reduction factor as defined by Yamada et al. (2008b), a table is developed from which the standard modulus reduction and damping curves could be obtained for various types of soil. Wang and Kuwano (1999) studied the response of 4 types of clay–sand mixtures using low-plastic soils. Marine clay from the Tokyo bay was mixed with amounts of Toyoura sand to obtain various proportions of clay–sand mixtures.

Several generic shear modulus reduction and damping curves also exist in the literature for a wider range of soil classes including Darendeli (2001), Roblee and Chiou (2004), Andrus et al. (2003), Menq (2003), Kishida et al. (2009), Yamada et al. (2008a), Zhang et al. (2005). These curves often require various parameters which are usually not obtained in downhole arrays, without in situ soil testing and though extensively used in various studies and have not been considered in this study. As clay behavior is significantly dependent on PI, there has been large scatter in the dynamic properties of clay (Anderson and Richart 1976) and the clayey soil class is also not considered presently for study.

Very limited guidelines are available for benchmarking the selection of a set of shear modulus and damping for the site response study where prior investigations into the dynamic soil properties are not done. This will raise serious issues in representing the soil behavior while carrying out response analyses. It would also be very helpful if there were a set of modulus reduction and the damping curves for different soil types, which could represent the soil behavior fairly accurately in the absence of parameters other than effective confining pressure (sand) and plasticity index (clay). Over the years, researchers in the Geotechnical field have presented a variation of shear modulus and damping ratio with shear strain for several materials. Available shear modulus and damping curves are collected from the literature for soil classes studied viz. sand, gravel and rock. Parametric studies were carried out considering sites with rock and surface recorded earthquake data to identify suitable curves. Available modulus and damping curves for different soil class are summarized here.

3.1 Selection of shear modulus reduction and damping curves for gravel and rock

Study of shear modulus reduction and damping ratio with strain of different soils was started much earlier, but for gravel and rock samples, studies were started after 1984. Results of the first study of rock samples were published by Seed et al. (1984, 1986) from large diameter (=300 mm) cyclic triaxial shear tests on four rock-fill darn materials (Rollins et al. 1998). Rollins et al. (1998) summarized the shear modulus and damping of gravel from Iida et al. (1984), Seed et al. (1986), Shamoto et al. (1986), Hatanaka et al. (1988), Hynes (1988), Shibuya et al. (1990), Goto et al. (1992, 1994), Yasuda and Matsumoto (1993, 1994), Kokusho and Tanaka (1994), Konno et al. (1994), Souto et al. (1994), Hatanaka and Uchida (1995) and presented best-fit curves for shear modulus reduction and damping relationships with strain and further highlighted the factors affecting shear modulus and damping ratio. The shear moduli and damping characteristics of the gravel were determined from the hysteretic stress–strain relationships determined by cyclic undrained triaxial tests (Rollins et al. 1998).

3.1.1 Selection of curves for rock sites

Very limited modulus reduction and damping curves exist for rock as noted by Stewart et al. (2014). EPRI (1993) provide confinement-dependent curves for studying the nonlinear behavior of rocks during dynamic loading. Other modulus reduction curves include the pressure-independent one provided by Schnabel (1973) and the pressure-dependent Choi (2008) class of curves. Equivalent linear and nonlinear site response analysis for predominantly rock profiles is studied with the three aforementioned curves. Figure 2 shows the modulus reduction and damping curves used in the current analysis for rock profiles. In the analyses, 10 pairs of ground motions are used for the site response analyses of LP rock sites IWTH05, FKSH18 and the LG rock site IWTH08, whereas 15 pairs are considered from the IWTH27 LG rock site. Five pairs of low-amplitude ground motions (PGA $\sim$ 0.005 g) are used to calibrate the shear wave velocity structure of the LP rock profiles IWTH05 and FKSH18.

Plots of typical response spectra obtained from the analysis and observed response spectrum at the surface are shown in Fig. 3a for LP profile FKSH18 and Fig. 3b for LG profile IWTH27. The residuals of response spectra are shown in Fig. 4, indicating that among the 3 curves used for rock, the pressure-dependent EPRI (1993) curves provide the least bias. The mean and standard deviation of all the profiles and ground motions considered in the analysis are provided as an electronic supplementary material (refer Table ET1). The Choi (2008) curves significantly underpredict the response at the studied

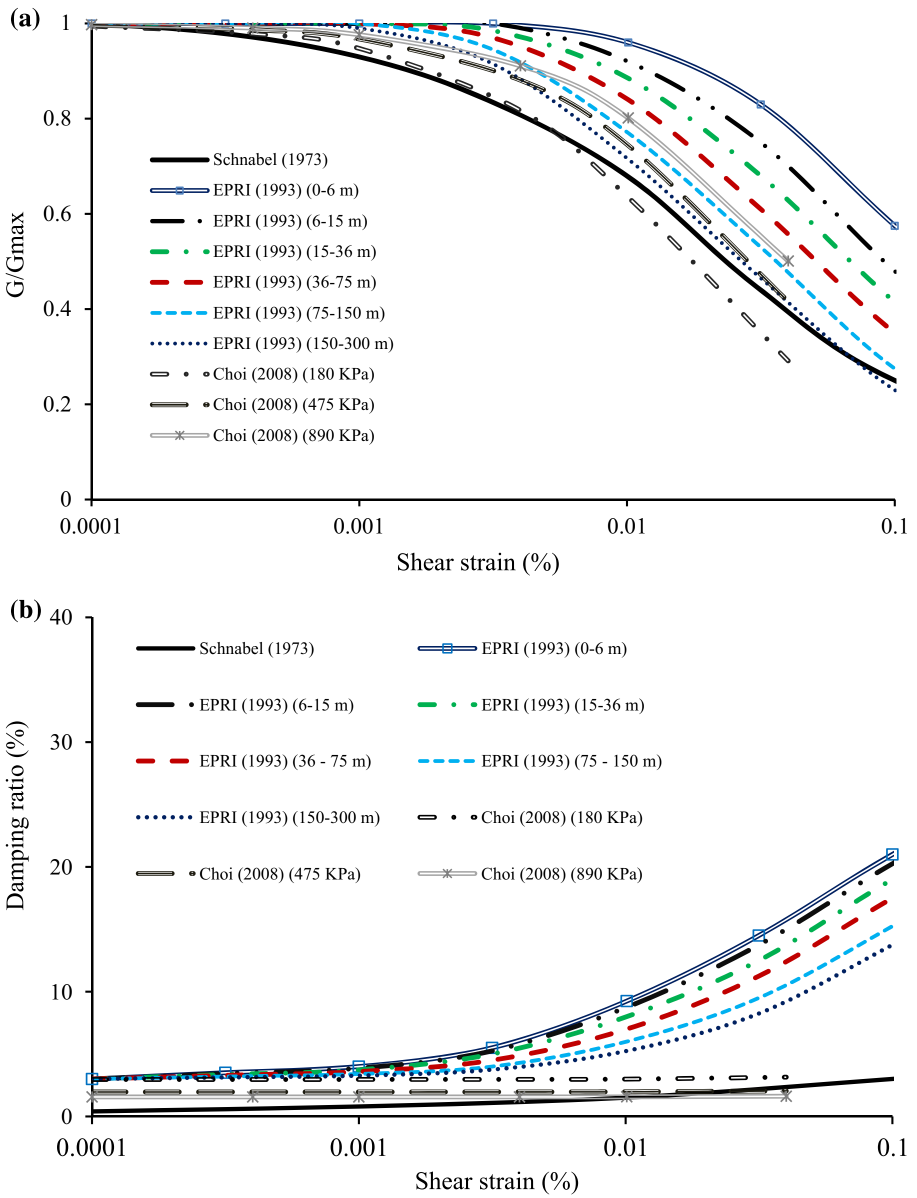

Fig. 2 Variation of shear modulus (**a**) and damping (**b**) with shear strain for rock

period range, and the Schnabel et al. (1972) curve provides higher residuals than the EPRI (1993) curves. The bias in the profiles for all sites is shown in Fig. 5. The box plots show the relative differences between the three different modulus reduction and damping curves used in the analyses and the difference between equivalent linear and nonlinear site response methodologies. Between the three curves used in the analysis viz. Schnabel et al. (1972) curve, the Choi (2008) curve and the EPRI (1993) curves, the EPRI (1993) curve

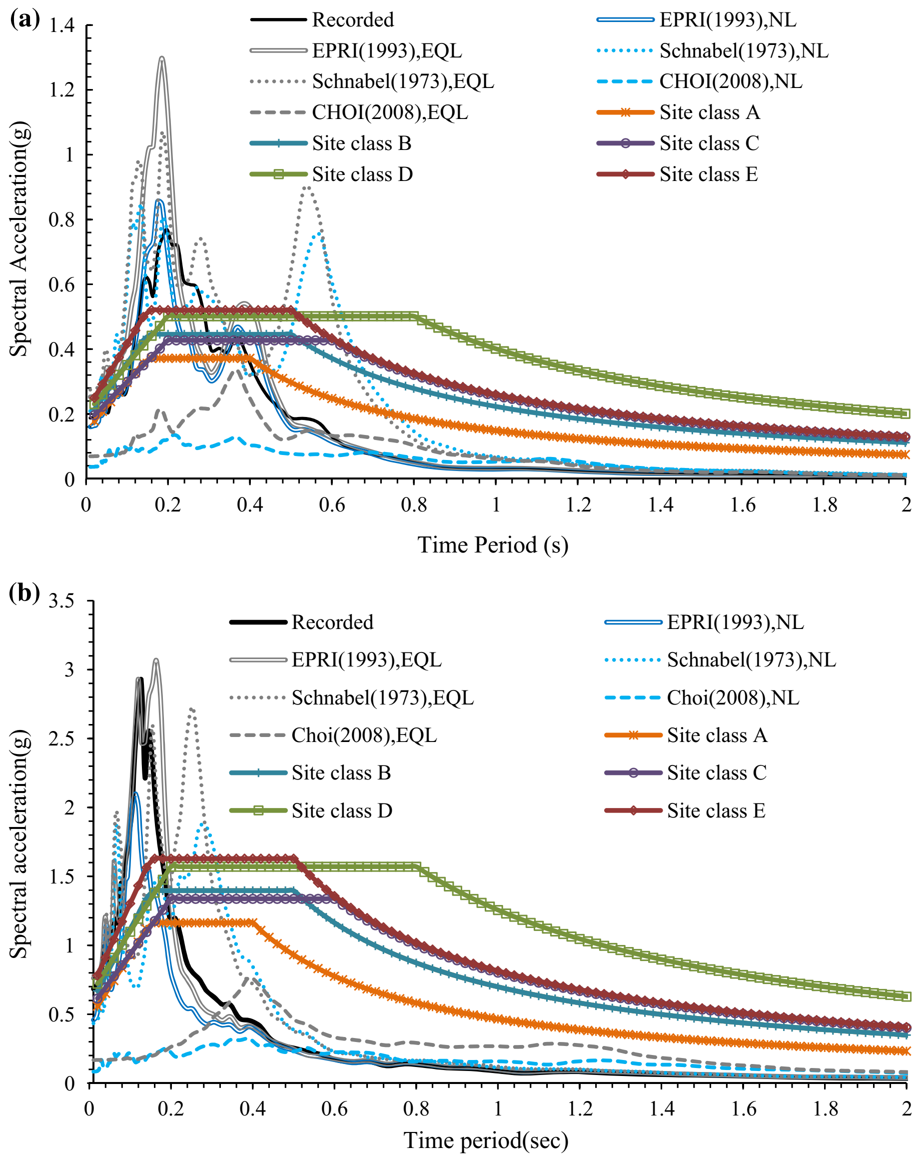

Fig. 3 Variation in response spectra for different modulus reduction and damping curves at **a** LP rock site FKSH18 **b** LG rock site IWTH27

best predicts the output response spectrum and could be utilized to carry out site response studies for other rock profiles in the region. Though in this case the nonlinear analysis provided a smaller bias in the mean of the residual, the equivalent linear analysis has lesser scatter about the mean for different ground motion recordings. This could be indicative of transient resonances occurring in the nonlinear analysis, and care must be taken during analysis to obtain consistent estimates. It is noticed that difference exists between the observed and calculated response spectra at the surface in the period range between 0.06

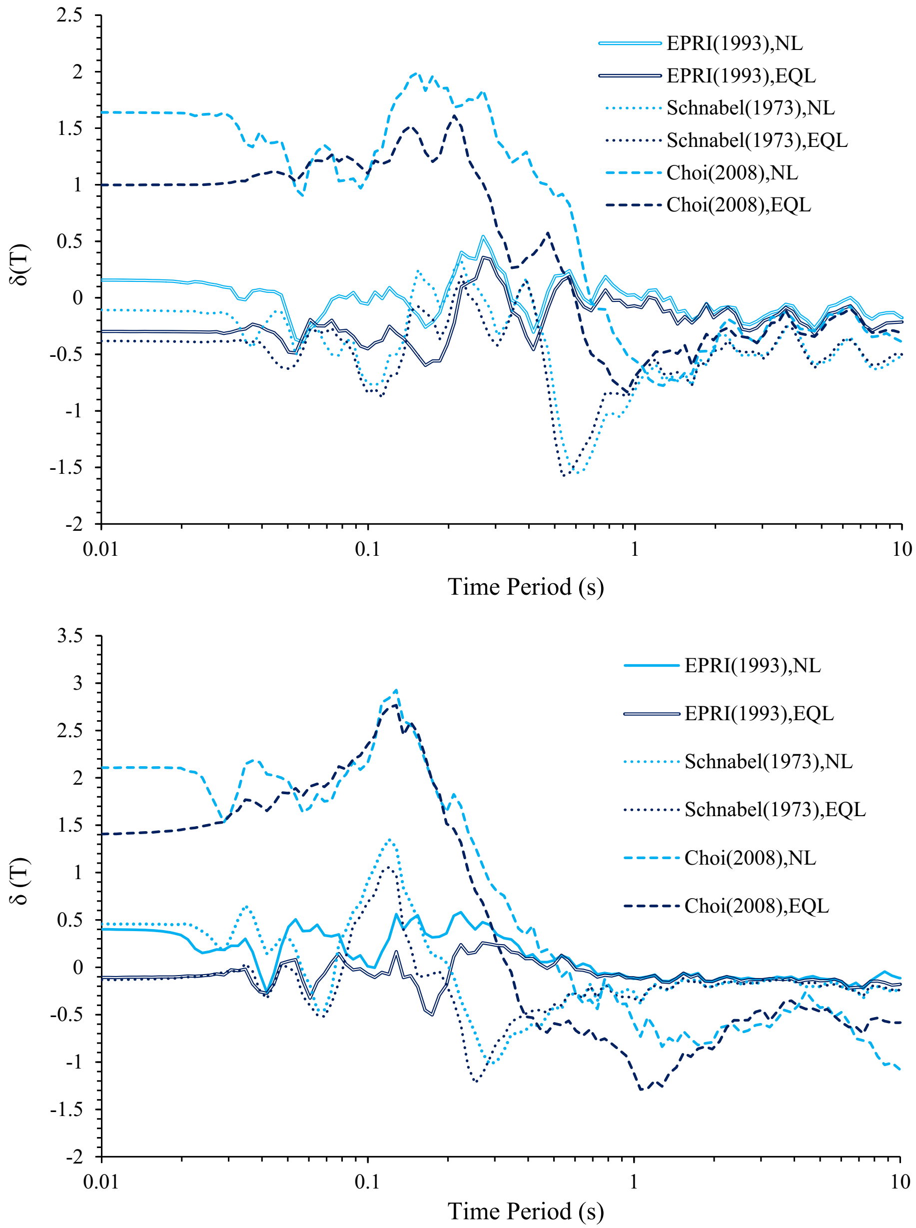

Fig. 4 Variation in the spectral residual with time period for different modulus reduction and damping curves **a** LP rock site FKSH18 **b** LG rick site IWTH27

and 0.13 s for few sites in both equivalent linear and nonlinear analysis. It is to be noted that between 0.06 s and 0.13 s for rock sites FKSH19 and IWTH27, the residual is significantly different from zero for all the curves used. In this study, the EPRI (1993) curves for rock are used for performing further site response analysis in the region for the sand and gravel soil class.

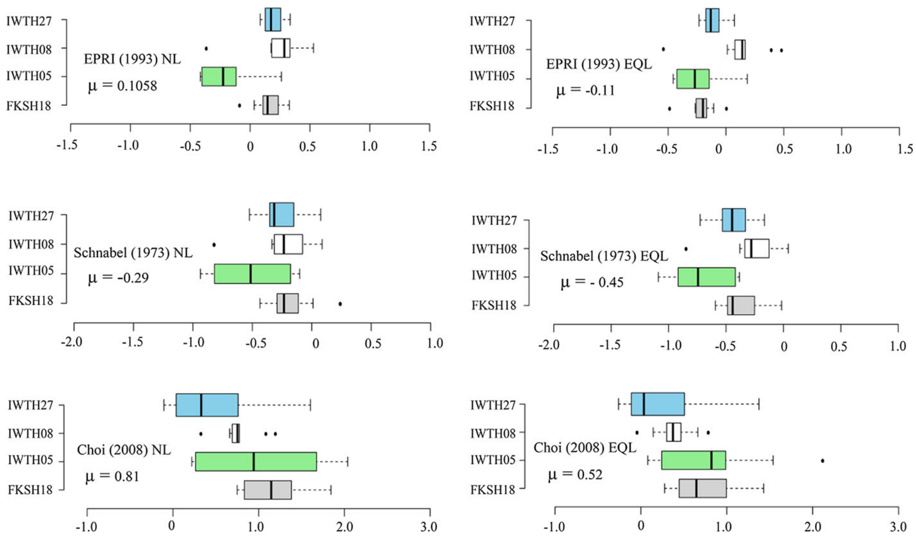

Fig. 5 Variation of the bias in the residual of the response spectra for rock profiles considered in the analyses

3.1.2 Selection of curves for gravel sites

Damping ratios for sands and gravels are similar and gravel damping ratios are slightly affected by density and not significantly depending on the grain size of the particles. Based on data from 15 investigators, Rollins et al. (1998) concluded that the best-fit hyperbolic curve can be used to define the mean normalized shear modulus versus cyclic shear strain curve for gravels. Rollins et al. (1998) analyzed 980 data points from previous studies and found that the shear modulus reduction curve of gravel is almost independent of sample disturbance, fines content (range 0–9%) and relative density but depends on the confining pressure. Rollins et al. (1998) summarize the tests indicating that the mean obtained curve for gravels was closer to the Seed and Idriss (1970) sand curve than the Seed et al. (1986) gravel curve. Figure 6 shows the shear modulus and damping for gravel recommended by Rollins et al. (1998) (mean, mean + 1 SD, mean − SD), Seed et al. (1986) and Roblee and Chiou (2004). In this study, the Seed et al. (1986) and Rollins et al. (1998) curves are pressure independent, while the Roblee and Chiou (2004) curve is dependent on the overburden pressure. Three gravel profiles have been considered in the analyses, FKSH11, TCKH08 and TCGH12 of which all were LG as per Thompson et al. (2012). Ten pairs of recorded ground motion have been used for each site, where the thickness of the gravel profile varied between 35, 106 and 78 m for FKSH11, TCGH12 and TCKH08, respectively. A typical variation of the observed and calculated surface response spectra and the spectral residual with time period is shown in Figs. 7 and 8 for gravel sites. Figure 9 shows the distribution in the average spectral residual for all three sites considered in the analysis. The mean and standard deviation of all the profiles and ground motions considered in the analysis are provided as an electronic supplementary material (refer Table ET2). Since the behavior of granular soil is heavily dependent on pore water pressure, for stronger ground motions, the 1D SH fails to predict the surface response spectrum. In the model FKSH11, for a ground motion of peak acceleration 0.35 g fails to model the system accurately with the predominant period of the observed response proving stiffer than the predominant

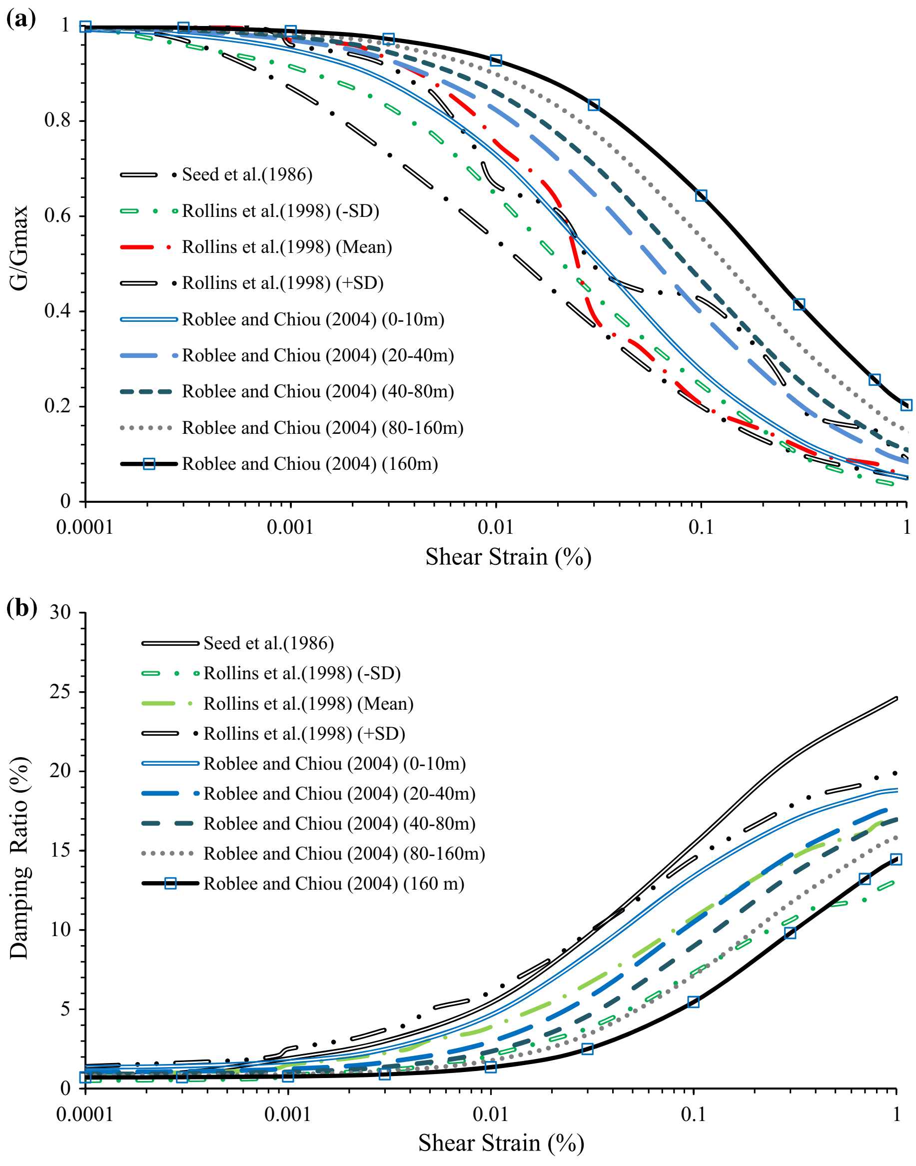

Fig. 6 Variation of shear modulus (**a**) and damping (**b**) with shear strain for gravel

period of the calculated motion and could be dilative nature of the response of the gravel deposit, which is heavily dependent on the rise in pore pressure. Since a total stress analysis is carried out and no information is available about the depth of the water table, it is assumed that pore pressure effects do not contribute to response of the gravel deposit.

The analysis shows that the Rollins et al. (1998) (−SD) curve can be used as best to describe the nonlinear soil behavior at the site. The Roblee and Chiou (2004) curve also provides reliable estimates of the ground response in the studied range of periods, but is marginally bettered by the Rollins et al. (1998) (−SD) curve. Since, the Roblee and Chiou

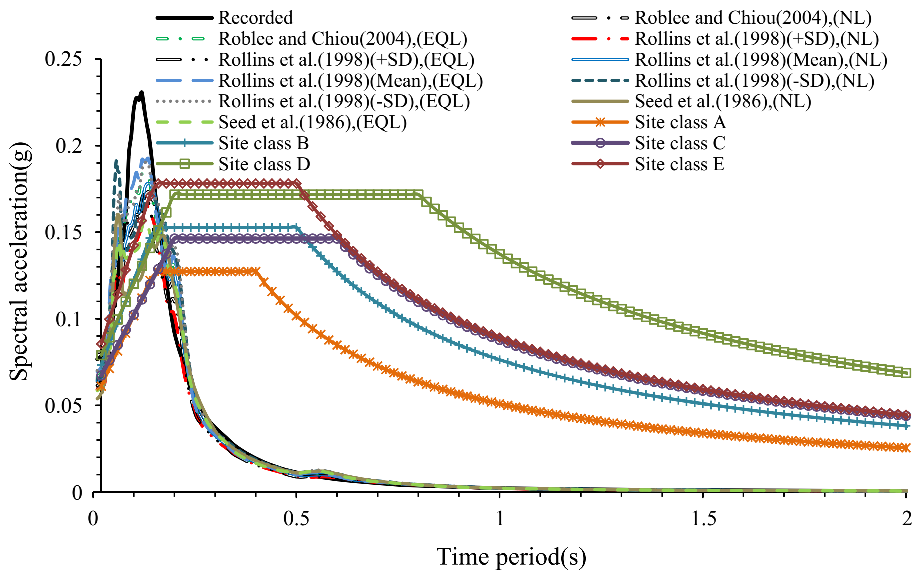

Fig. 7 Variation in response spectra for different modulus reduction and damping curves at LG gravel site TKCH08

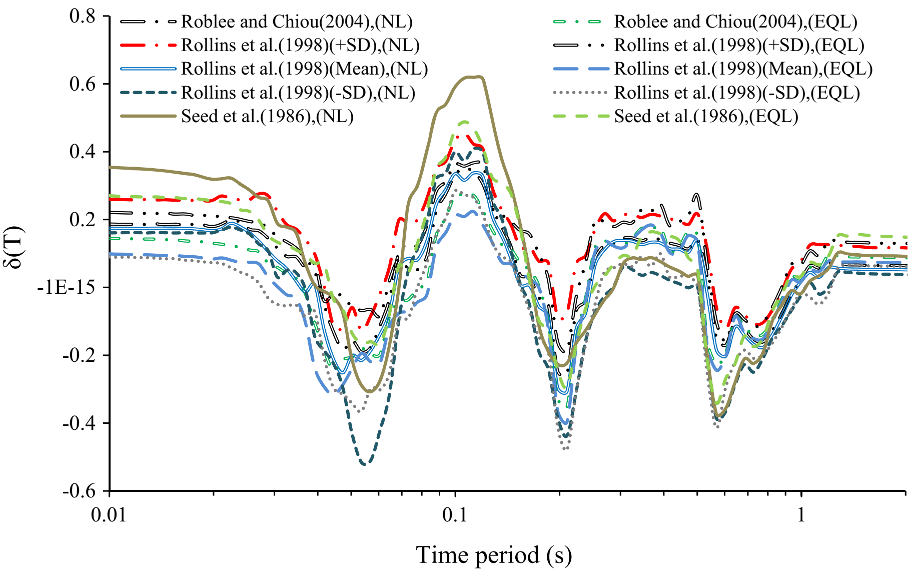

Fig. 8 Variation in residual with time period for different modulus reduction and damping curves at LG gravel site TKCH08

curve (2004) is pressure dependent in nature, for deeper soil profiles, since overburden pressure significantly affects the nonlinear soil behavior, it could prove to be the better fit. The trend toward the lower standard deviation site for the Rollins et al. (1998) (−SD) curve also indicates a decrease in the gravel fraction at the site, and future site response

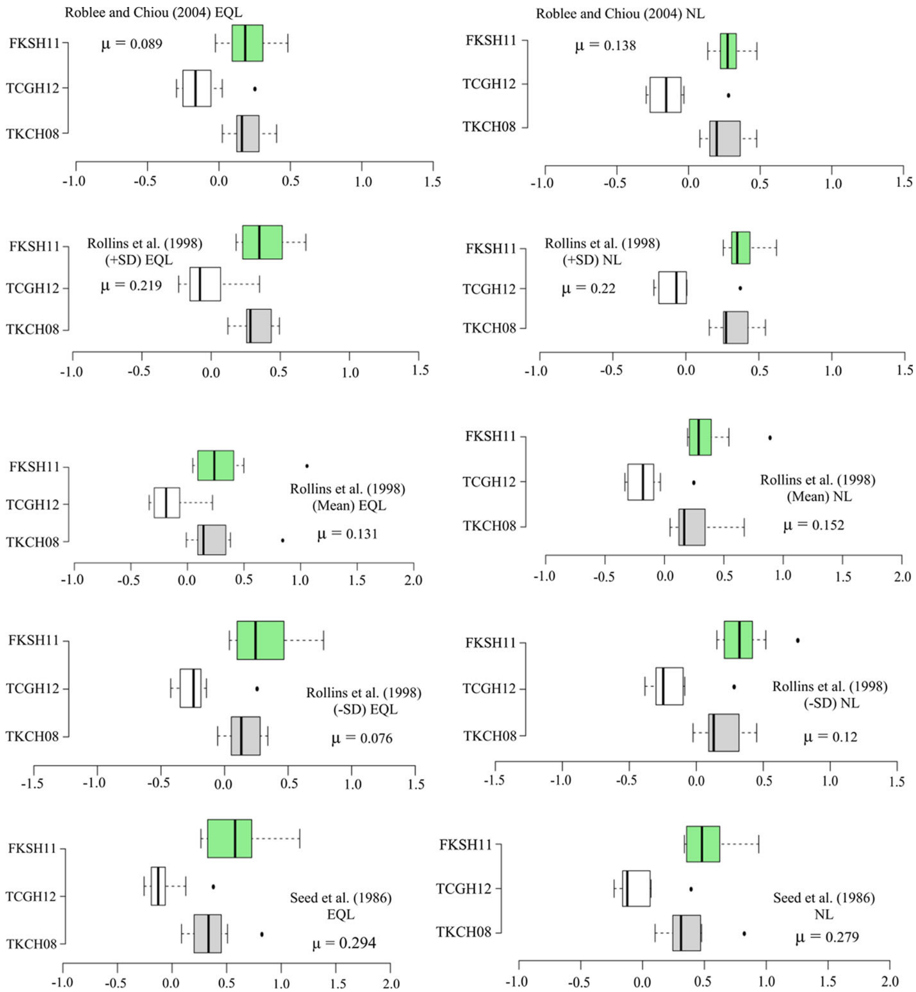

Fig. 9 Variation of the bias in the residual of the response spectra for gravel profiles considered in the analyses

studies in the region could adopt the Rollins et al. (1998) (−SD) curve. For gravel sites, the residual plot indicates that for all the curves used there is a significant difference in the calculated and observed spectral acceleration at the soil surface between the period ranges of 0.06–0.11 s and 0.16–0.17 s for TKCH08.

3.1.3 Shear modulus and damping curves for sand

Many experimental investigations for sandy soils have been carried out and formulated for site response studies by Seed and Idriss (1970), Hardin and Drnevich (1972), Seed et al. (1986), Iwasaki et al. (1978), Kokusho (1980). Seed and Idriss (1984) reviewed a number of studies and found that in general the shear modulus values of sands are strongly influenced by effective confining pressure, strain amplitude and void ratio but not significantly by variation of grain size characteristics. Seed et al. (1984) plotted the ratio of modulus at shear strain γ to modulus at shear strain 10^{-4} % from the Hardin and Drnevich (1972) relationship to obtain a band of results and noted that the results fall within a narrow range. Many other investigators found results in the same general range (Iwasaki et al. 1978; Kokusho 1980; Prakash and Puri 1981).

The damping ratio is slightly affected by grain size characteristics, degree of saturation, void ratio, lateral earth pressure coefficient, angle of internal friction and number of stress cycles, which was highlighted by Seed et al. (1986) based on studies shown by Hardin and Drnevich (1972), Seed and Idriss (1970) and others. Electric Power Research Institute (EPRI 1993) presented a set of depth-dependent generic modulus reduction and damping curves for cohesionless soils accommodating the effect of confining pressure. Developed in the aftermath of the 1989 Loma Prieta earthquake, data from three earthquakes were considered in formulating these curves viz. the 1979 Coyote Lake Earthquake, the 1989 Loma Prieta earthquake and the 1984 Morgan Hill Earthquake. Revisions were made post the Northridge 1994 earthquake when it was realized that the initially developed EPRI curves underpredicted the response. Hence, from the peninsular range, data were complied and the EPRI standard curves were revised. These curves were based on laboratory testing of undisturbed samples obtained at different depths from three reference Gilroy sites. The laboratory tests conducted for modulus reduction curves were resonant column and torsional shear (RCTS) and pulse setting tests.

In the current analysis for the sand soil class, the modulus reduction and damping curves are studied using the Seed and Idriss (1970) mean, mean + SD and mean-SD curves which are pressure independent in nature and the pressure-dependent EPRI (1993) curves and the Roblee and Chiou (2004) suggested curves for cohesionless soils. The modulus reduction and damping curves used in the analysis are shown in Fig. 10. Three profiles were considered for the analyses viz. KSRH05, FKSH08 and TCGH15, which were classified as LP, according to Thompson et al. (2012). For the calibration of profiles to obtain the shear wave velocity structure which best fits the observed response spectra at the surface, 5 pairs of low-amplitude ground motions (PGA $\sim$ 0.005 g) were used to perform linear site response analysis. On obtaining the shear wave velocity structure, 5 pairs of strong motion data were used to perform the parametric studies to compare the observed and calculated response spectra at the surface. The depth of the sand layer varied as 137 m, 50.5 m and 20.7 for KSRH05, FKSH08 and TCGH15, respectively. Figure 11 shows the comparison between observed and the calculated response spectrum at the surface, and the variation of the average spectral residual with time period is shown in Fig. 12. The distribution in the average spectral residual for all three sites considered in the analysis is shown in Fig. 13. The mean and standard deviation of all the profiles and ground motions considered in the analysis are provided as an electronic supplementary material (refer Table ET3).The analyses show that all the curves used for sand show similar estimates of the output ground motion response spectrum with the Seed and Idriss (1970) upper limit curve marginally producing the best fit. Similar to the gravel profiles, since a total stress analysis is performed, the pore pressure rise is not modeled in the analysis. The output response spectra show significant deviation from the recorded response spectrum between the 0.06 and 0.1 s and 0.13–0.14 s period ranges in both the nonlinear and equivalent linear analysis and could be due to the model having lower low-strain damping estimates at larger depths. Hence, for further analyses in the region, the Seed and Idriss (1970) upper curves could be used to obtain reliable fits. For deeper deposits, as the EPRI (1993) curves are pressure dependent in nature and is only marginally bettered by the Seed and Idriss (1970) upper limit curve, it could be used to model the nonlinear behavior of sands at deeper depths.

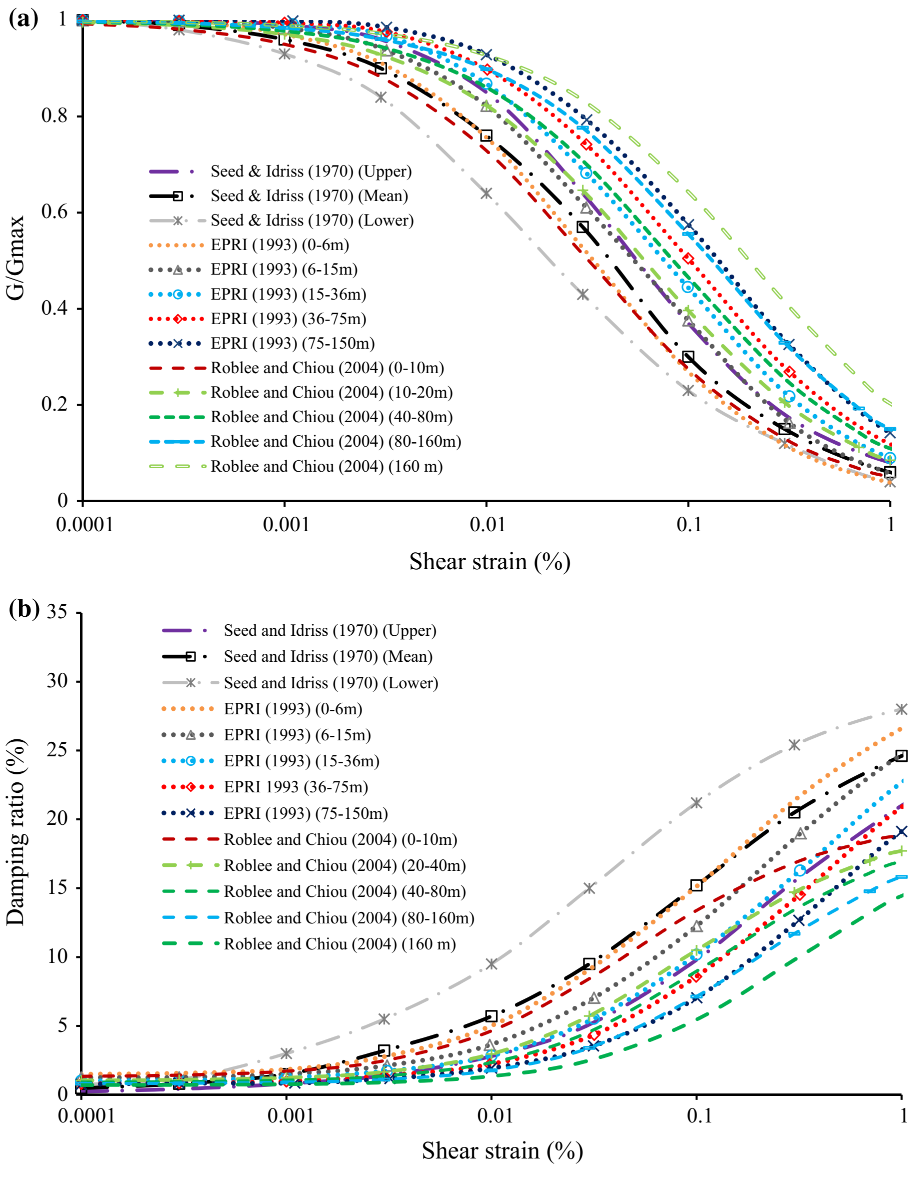

Fig. 10 Variation of shear modulus (**a**) and damping (**b**) with shear strain for sand

3.2 Limitations and assumptions of the current analysis

This study aims to identify and select a set of shear modulus reduction and damping curves for soils broadly divided into rock, gravel and sand sites part of the KiK-Net downhole array Network. Since grain size distribution at the respective sites is not known beforehand, a qualitative estimate is used obtained from the KiK-Net database to estimate the soil class. The difference in the computed and observed response spectra at the surface is used

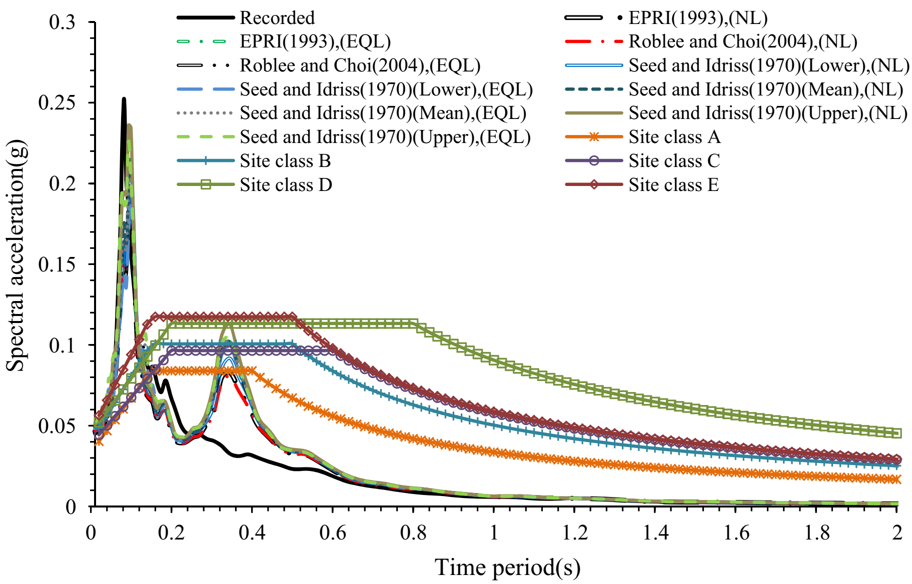

Fig. 11 Variation in response spectra for different modulus reduction and damping curves at sand site FKSH08

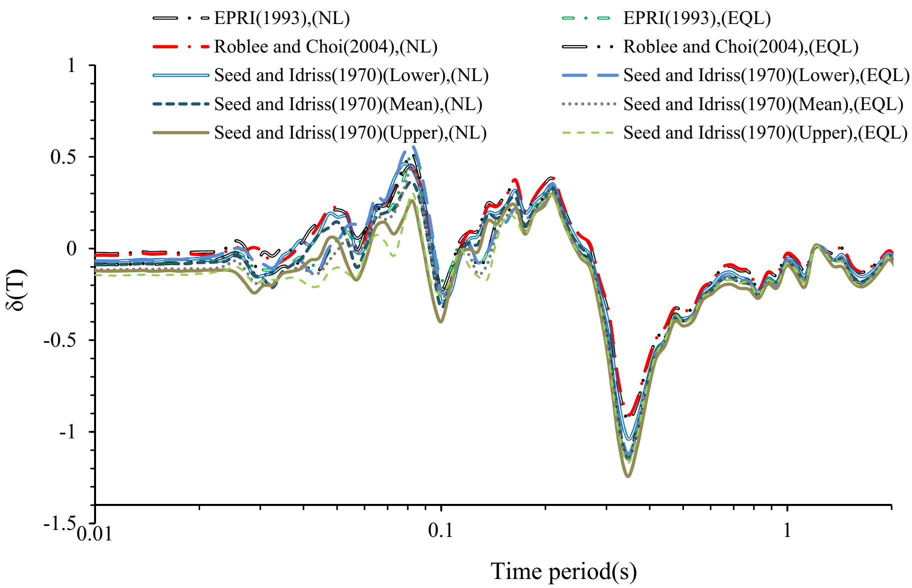

Fig. 12 Variation in residual with time period for different modulus reduction and damping curves at sand site FKSH08

as a measure to estimate the suitability of the set of shear modulus reduction and damping curves for the soil class. Sites are selected based on Thompson et al. (2012), which have low intraevent variability, classified into LP (poor fit to 1D wave propagation assumptions)

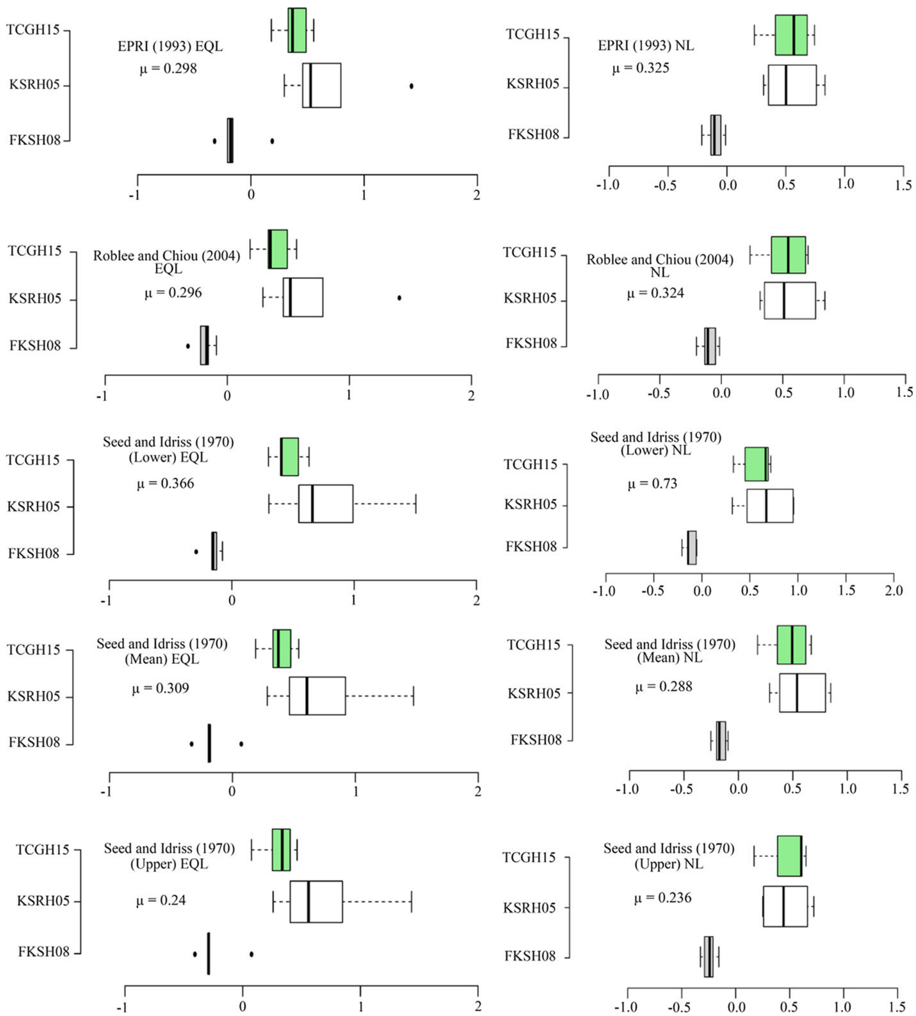

Fig. 13 Variation of the bias in the residual of the response spectra for sand profiles considered in the analyses

and LG (good fit to the 1D wave propagation assumptions). For sites labeled as LP, non-vertical incidence is not assumed as a source of error and the error is attributed to differences in the shear wave velocity and small strain damping ratio of the profile. Initial estimates of the in situ small strain damping ratio are obtained from Kaklamanos et al. (2013), and a Monte Carlo simulation is performed varying the shear wave velocity profile structure of the site to perform a linear 1D site response analysis. Once the shear wave velocity profile of the site is estimated, the sites are further used to perform the site response analysis using the proposed set of shear modulus reduction and damping curves.

Several generic shear modulus reduction and damping curves exist in the literature, which require additional parameters which are difficult to obtain for a soil deposit without sampling, and hence among many such curves existing in the literature, few are selected and compared to estimate which set of curves would produce the best fit. Assumptions of

1D horizontally polarized shear wave propagation assuming vertical incidence are used in the analysis, and ground motions are filtered using Boore and Bommer (2005), with the corner frequencies selected from Dawood et al. (2016). Limitations of the analysis involves the use of results only for Japanese sites, data are entirely from the Japanese seismotectonic regime and when being used for other site, care must to be taken. It is also noted that these curves should not be used as a replacement for available existing data at sites and can only be used as an initial estimate to predict the surface response spectra, through a site response analysis in the absence of additional information about the soil deposit.

4 Conclusions

In this study, suitable shear modulus reduction and damping ratio curves for different soil types are identified by the parametric site response studies. Three sand column profiles, three gravel profiles and four rock profiles are taken from the KiK-Net downhole array database along with the recorded ground motion from the uphole and downhole sensors for the corresponding sites. The sites which showed low interevent variability were selected in accordance with earlier studies. For sites which were not a good fit to the one-dimensional wave propagation assumptions, Monte Carlo simulations were carried out, varying the shear wave velocity structure of the site to perform linear site response analysis with low-amplitude ground motion. With the profiles obtained, nonlinear total stress site response analysis was carried out using strong motion data, giving the rock recorded ground motion as input to the obtained shear wave velocity structure. The modulus reduction and damping curves were taken from the literature and varied in the deposit for different soil classes grouped as gravel, rock and sand. Response spectrum at the surface is arrived from each trial and compared with the observed surface record, and the fits are compared using the average spectral residual between 0.02 and 0.6 s. Shear modulus and damping curves given by Seed and Idriss (1970) upper limit can be used to predict reliable response parameters in sandy soil sites. Shear modulus and damping curve of Rollins et al. (1998) (−SD) give the most accurate match for sites with dominant gravel layers. Shear modulus reduction and damping curves for rock are very limited, and the curves given by EPRI (1993) are found to be more appropriate than the other two curves in the literature. These findings may be further improved when a large number of profiles are analyzed. The recommended shear modulus reduction and damping curves for close matching of several profiles is considered as the suitable curve of the soil class and is recommended for site response analysis of similar soil column where such curves are unavailable sites. Equivalent linear analysis provided lower scatter in the range of the estimates means of the residuals than nonlinear analysis, which could be due to transient resonances and thus care must be taken in the development of profiles for nonlinear analysis. Certain profiles showed changes in the predominant period of the soil column, indicating that pore pressure rise and effective stress behavior significantly affects the performance of gravels and sands.

Acknowledgements The authors wish to thank the Kyoshin Net Strong Motion Network of Japan for providing an excellent earthquake database and valuable feedback for conducting this research. The authors extend their sincere appreciation to the Deanship of Scientific Research at King Saud University for its funding for this Prolific Research Group (PRG-1436-06).

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