

Dynamic Properties of Soils at Large Strains in Roorkee Region Using Field and Laboratory Tests

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Abstract The evaluation of the dynamic properties of natural soil deposits is important to study the site response, to predict the ground motion and to proceed to seismic zonation. In this paper, dynamic behavior of soils in Roorkee (India) region has been investigated. The study focuses primarily on four sites: Solani Riverbed site, Bhagwanpur site, Bahadradab site, and Haridwar site. The soil samples were collected from different depths at these four sites using SPT. Soil samples collected were tested in the laboratory for index properties and subjected to bender element tests to measure the maximum shear wave velocity. Dynamic behavior of soil samples collected from all sites was investigated by a series of cyclic triaxial tests. Strain controlled undrained cyclic triaxial tests were carried out as per ASTM D3999 for determination of dynamic soil properties (i.e., shear modulus and damping ratio). The effects of shear strain (in the range 0.03–2%) on shear modulus and damping ratio have been investigated at different depths. The results show that the variation in shear modulus and damping ratio with shear strain is greater for the samples collected from greater depths as compared to the samples collected from shallower depths. Variation in shear modulus and damping ratio with shear strain were compared with the published literature and found to be in good agreement.

Keywords Dynamic properties · Large strains · Field tests · Laboratory tests · Cyclic triaxial

Introduction

Small cities in country are growing at a very fast rate and safety of these during earthquakes is very important. Roorkee is a small city (population about two-hundred thousand and situated about 185 km North of New Delhi) and lies in seismic zone IV IS: 1893 [1]. The behavior of soil during strong shaking is nonlinear, particularly at large strains. In many cases, the maximum value of shear strain is about 1% and equivalent linear model can be used with reasonable accuracy. Dynamic properties of soils are important for ground response analysis, seismic slope stability analysis, and liquefaction assessment, Kramer [2]; Towhata [3]. Further, to evaluate seismic response of a soil deposit, it is necessary to evaluate the dynamic properties of the soil layers. The cyclic triaxial is widely used in the laboratory for investigation of soil characteristics at intermediate and large shear strains. Seed and Idriss [4], Kokusho [5], and many other researchers have reported the usefulness of the cyclic triaxial apparatus for evaluation of dynamic soil properties. Hardin and Drnevich [6] presented variation of shear modulus and damping ratio with shear strain for different soils. The similar trends have been observed for sands [7–12] and clays [13, 14].

This paper presents the effect of shear strain (in the range 0.03–2%) on dynamic properties of soil samples collected from four different sites in Roorkee region: Solani Riverbed site, Bhagwanpur site, Bahadradab site and Haridwar site. A number of strain controlled undrained cyclic triaxial and bender element tests were conducted. Shear modulus and damping ratio at different strain levels have been determined and compared with that given in the literature. The effect of depth (from which sample is collected) on the dynamic behavior is rarely reported in the literature, the same has been examined in this paper. The

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results in this paper are based on laboratory tests where the samples are collected using field tests.

Site Selection and Location

The SPT (Standard Penetration Tests) were conducted at following 4 sites in Roorkee region:

1. Solani Riverbed site: about 3 km from IIT Roorkee, SPT conducted in Solani river bed and the distance of SPT bore hole was about 50 m from Solani via Aquaduct.
2. Bhagwanpur site: situated around 15 km from Roorkee on National Highway (NH) No. 73 and the SPT conducted in the ground of Govt. Inter College.
3. Bahadradab site: about 18 km from Roorkee on NH 58 and SPT conducted inside the ground of Arya Inter College at about 100 m right side from NH 58.
4. Haridwar site situated around 32 km from Roorkee on NH 58 and was situated inside the ground of Pannalal Bhalla Municipal College.

The SPT samples from different depths were collected from all these four sites (Table 1) and these samples are subjected to cyclic triaxial and bender element tests in the laboratory.

The locations of all four sites are shown in Fig. 1 and others details of the sites e.g. longitude, latitude, depth of water table and number of samples collected are given in Table 1, total 22 samples were collected from the field. All the samples (mostly of sand) were subjected to cyclic triaxial tests. Water table is not available (NA) for Haridwar site.

Geotechnical Investigations

The geotechnical investigations were carried out using Standard Penetration Tests (SPT) according to IS 2131 [15]. The laboratory tests were conducted on all the samples collected to evaluate their Index Properties i.e. Grain Size Distribution, IS: 2720-Part 4 [16]; Specific Gravity, IS: 2720-Part 3 [17]; Maximum and Minimum Void Ratio,

Coefficient of Uniformity, Coefficient of Curvature, Dry Unit Weight and Relative Density [18].

For Solani Riverbed site, 4 samples were collected using SPT from depths 0.75, 1.5, 2.25, and 3.5 m. All these samples were examined to determine their index properties. Grain size distribution (GSD) curves for different depths are given in Fig. 2a. It can be observed that about 80% of the particle size falls in medium sand range and less than 5% is passing through 0.075 mm sieve. Therefore, as per Unified Soil Classification System, ASTM D2487 [19], the soil can be classified as poorly graded sand i.e. SP type. Further, the effect of depth is not significant on grain size distribution curves.

The above exercise is repeated for other 3 sites too and grain size distributions for Bhagwanpur site, Bahadradab site and Haridwar site are presented in Figs. 2b, 2c and 2d, respectively. GSD are shown only for cohesionless soil samples, it can be observed that for a particular sieve size, there are differences in percentage of finers for different sites. However in general, for all sites, the soil can be classified as SP soil. Effect of depth on GSD is difficult to generalize. Other properties like specific gravity (G), dry unit weight (γ_d) and relative density (RD) etc. for all the samples collected from the 4 sites are given in Table 2.

The stiffness of the soil column is estimated using SPT. Variations of penetration resistance (N) with depth for all four sites are shown in Fig. 3. It can be observed that the measured uncorrected SPT blow counts, for all sites, fall in a narrow range of 2–13 indicating loose sand in the investigated depth. The SPT values are relatively greater for Bahadradab and Haridwar sites, while low for Solani Riverbed site.

Laboratory Tests and Formulation

For determination of dynamic soil properties, two types of laboratory tests namely bender element and cyclic triaxial were conducted on all the samples collected from the sites. While the former evaluate the maximum shear wave velocity which in turn provide maximum shear modulus (G_{max}) and later provides variation of dynamic properties with shear strain.

Table 1 Details of sites

Site	Longitude	Latitude	Water table depth (m)	Bore hole depth (m)	Number of samples
Solani Riverbed	E077° 53.965'	N29° 52.907'	3.5	3.5	4
Bhagwanpur	E077° 48.820'	N29° 56.203'	2.5	8.0	7
Bahadradab	E078° 02.307'	N29° 55.087'	6.0	9.0	6
Haridwar	E078° 09.019'	N29° 59.420'	NA	6.0	5

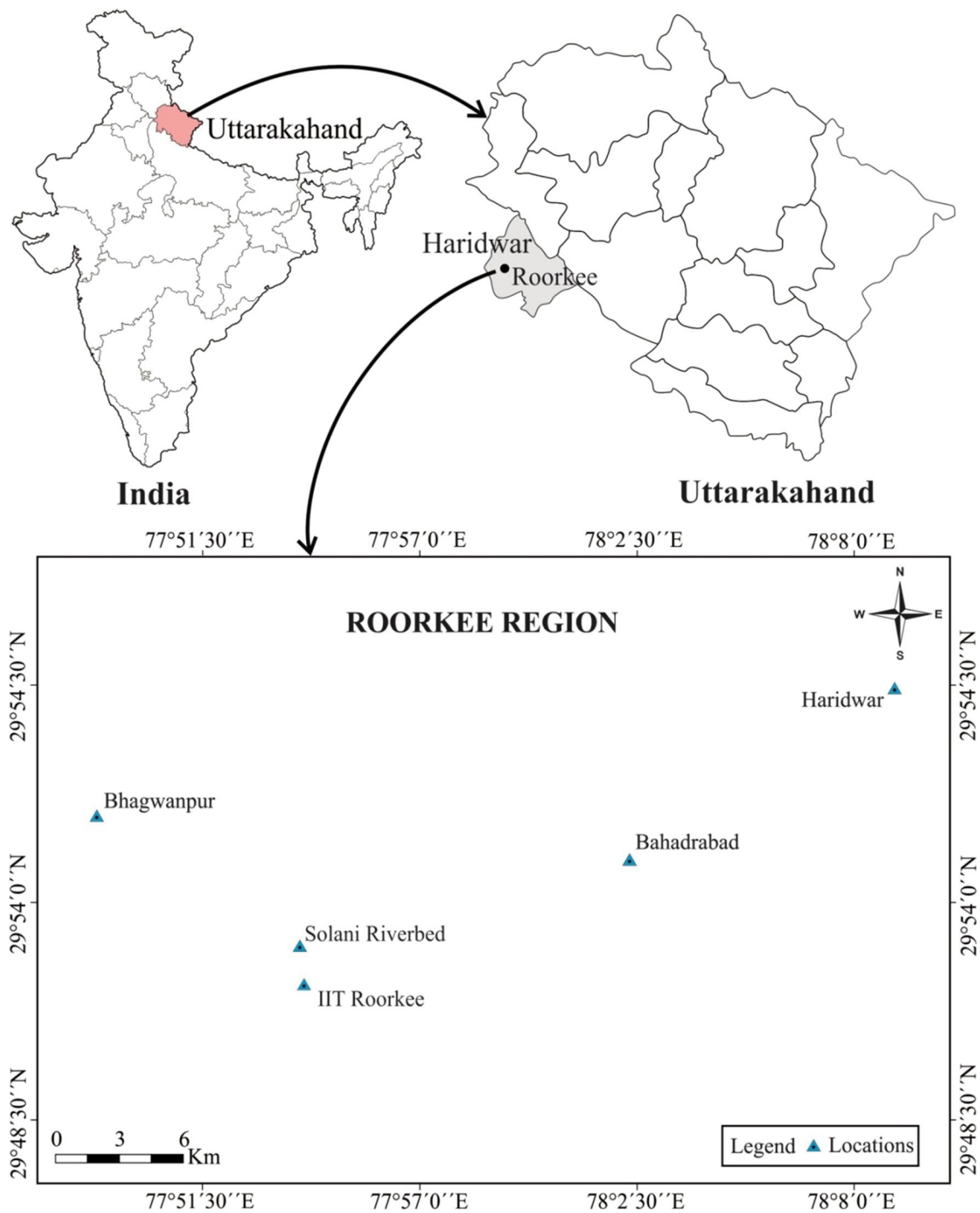


Fig. 1 Location of sites

Bender Element Test

Bender element test has been used to measure the shear wave velocity on laboratory specimens [20, 21]. Bender elements are constructed by bonding two piezoelectric materials together in such a way that a voltage applied to their faces causes one to expand while other contracts

causing the entire element to bend [2]. Figure 4 shows a set of bender elements vacuum top cap and base pedestal used in tests. Subsequently in this paper, Bender Element tests are referred as BE.

The time difference (t_0) between the two voltage pulses is measured with an oscilloscope as shown in Fig. 5 and shear wave velocity (V_s) is calculated as

Fig. 2 **a** Grain size distribution for Solani Riverbed site, **b** grain size distribution for Bhagwanpur site, **c** grain size distribution for Bahadrabad site and **d** Grain size distribution for Haridwar site

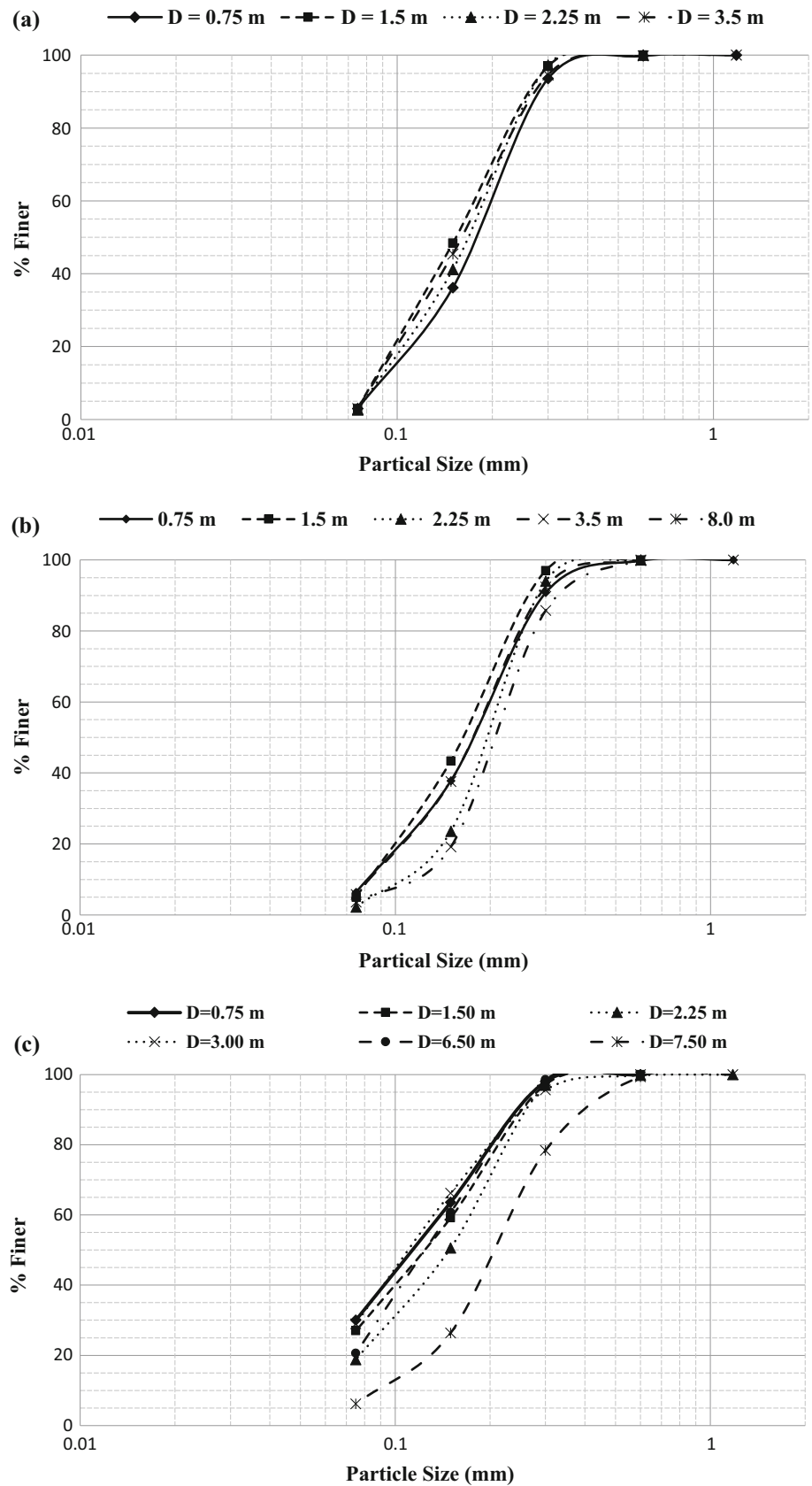


Fig. 2 continued

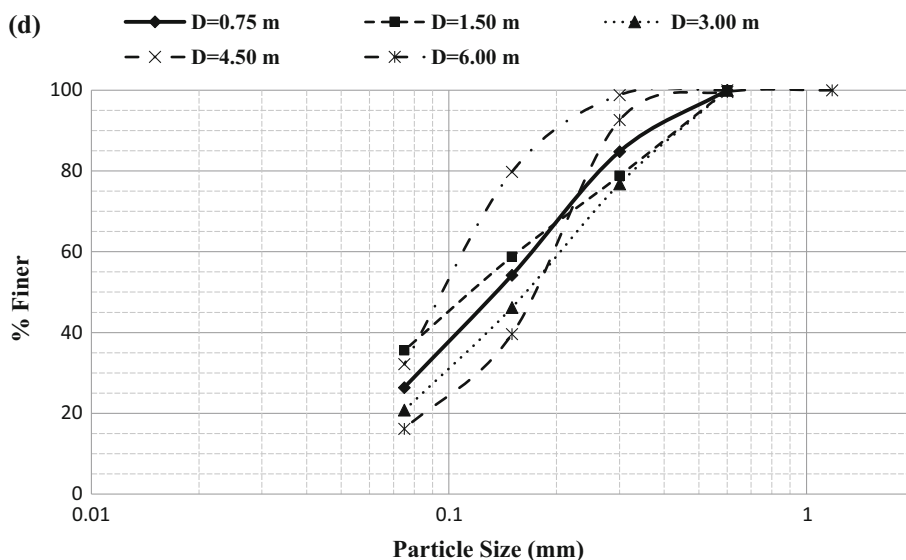


Table 2 Index properties of the samples collected from all four sites

Site name	S. no.	Depth (m)	Soil type	Specific gravity	γ_d (kN/m ³)	e	RD (%)	N value	σ'_3 or σ'_v (kPa)
Solani Riverbed	1	0.75	SP	2.64	15.85	0.66	34	7	12
	2	1.5	SP	2.65	15.60	0.69	28	4	27
	3	2.25	SP	2.63	16.29	0.61	52	12	42
	4	3.5	SP	2.66	16.63	0.60	55	12	58
Bhagwanpur	5	0.75	SP	2.65	15.80	0.67	26	2	15
	6	1.5	SP	2.64	16.63	0.58	39	5	27
	7	2.25	SP	2.65	16.10	0.64	44	8	41
	8	3.5	SP	2.63	17.33	0.51	57	10	57
	9	4.5	CL	2.67	14.57	0.83	PI = 41%	8	67
	10	6.0	CL	2.68	14.73	0.81	PI = 21%	5	82
	11	8.0	SP	2.64	17.76	0.48	60	12	106
Bahadrabad	12	0.75	SP	2.66	16.57	0.61	36	8	13
	13	1.5	SP	2.67	16.71	0.59	44	9	27
	14	2.25	SP	2.67	16.85	0.58	52	10	40
	15	3.00	SP	2.68	17.15	0.56	63	13	55
	16	6.50	SP	2.64	16.37	0.61	51	8	127
	17	7.50	SP	2.65	16.53	0.60	54	8	144
Haridwar	18	0.75	SP	2.66	15.94	0.67	38	7	13
	19	1.5	SP	2.70	16.45	0.64	46	8	27
	20	3.00	SP	2.68	17.18	0.56	60	13	58
	21	4.50	SP	2.67	16.44	0.62	50	10	88
	22	6.00	SP	2.68	16.68	0.61	55	12	119

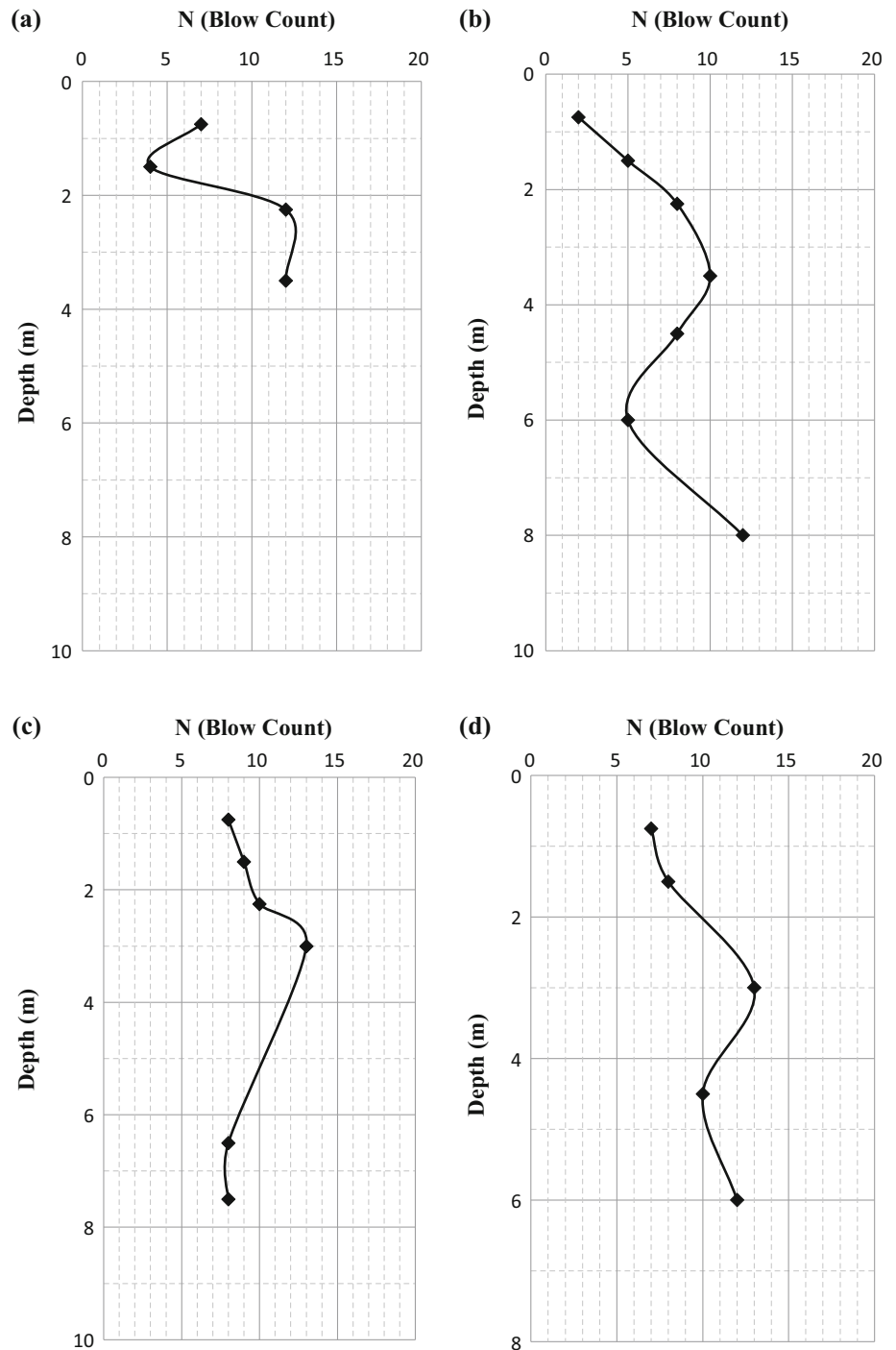
$$V_s = \frac{L_{tt}}{t_o} \tag{1}$$

Where, L_{tt} = tip to tip distance between transmitter and receiver. The measured shear wave velocity (V_s) was used to calculate maximum shear modulus (G_{max}) as follows

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

Where, ρ = mass density of the specimen. Time histories of the transmitted signal and the received signal for a typical test are shown in Fig. 5. In order to interpret the travel time of the shear waves, the method of visual picking has been used, where the first major deflection of

Fig. 3 Variation of penetration resistance (N) with depth for all sites. **a** Solani Riverbed site, **b** Bhagwanpur site, **c** Bahadradbad site and **d** Haridwar site



the received signal is taken as the shear wave arrival time [22].

Cyclic Triaxial Test

Cyclic triaxial test is the most commonly used test for the evaluation of cyclic strength and strain-dependent dynamic properties of soils at high strain levels. In this study, the

deviator stress is applied cyclically on specimens under stress-controlled condition.

A computerized automated triaxial testing system with five sensors, including a load cell to monitor the axial load, an LVDT to measure the vertical displacements and three transducers to detect the cell pressure, pore water pressure and volume change was used in this study. The loading system (Fig. 6) consists of a load frame and hydraulic

Fig. 4 Set of bender elements vacuum top cap and base pedestal used in tests

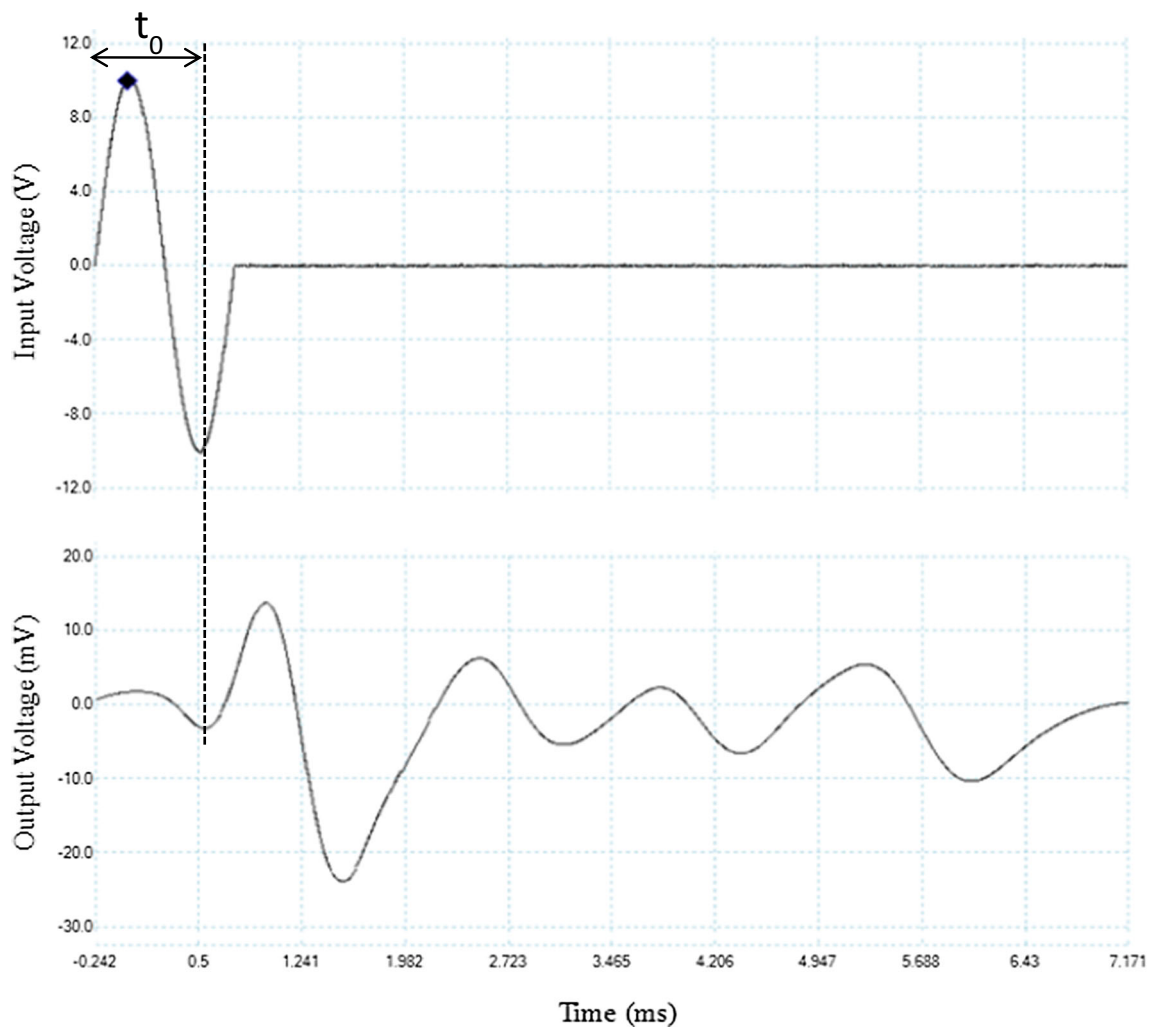
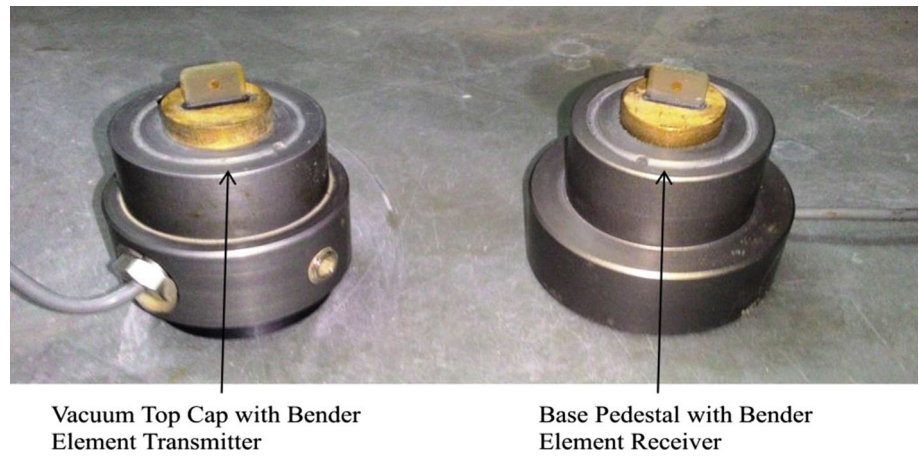


Fig. 5 Time histories of the transmitted signal and the received signal

actuator capable of performing strain-controlled as well as stress-controlled tests with a frequency range of 0.01–10 Hz. From cyclic triaxial test results, hysteresis loops similar to that shown in Fig. 7 are obtained by

plotting the maximum deviator stress (σ_{dmax}) versus maximum axial strain (ϵ_{max}). The slope of the secant line connecting the extreme points on the hysteresis loop is the Young’s modulus (E) [3].

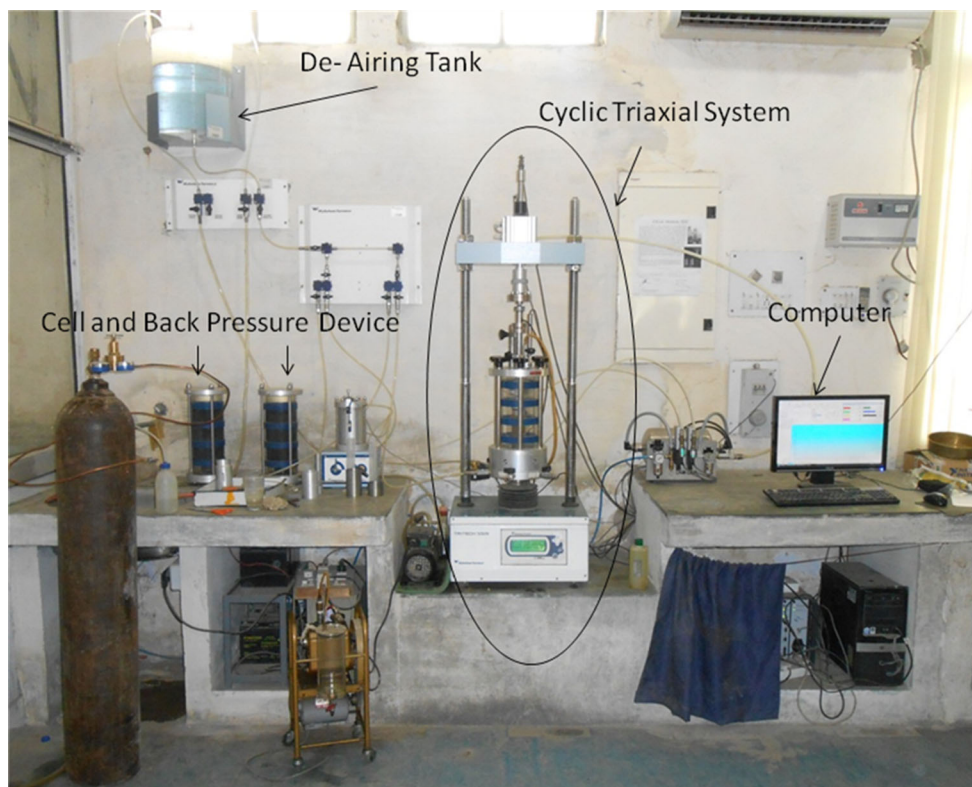


Fig. 6 Cyclic triaxial system used for testing of soil samples

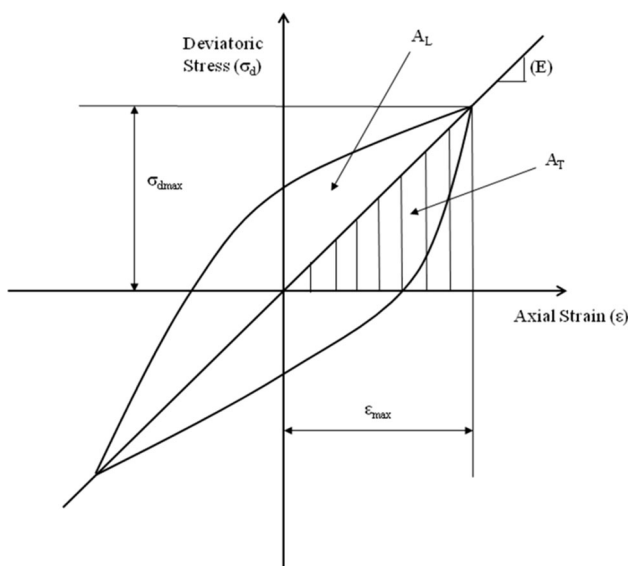


Fig. 7 Hysteretic stress–strain relationship

$$E = \frac{\sigma_{dmax}}{\epsilon_{max}} \tag{3}$$

The shear strain (γ) and shear modulus (G) are evaluated using following relations

$$\gamma = (1 + \mu)\epsilon_{max} \tag{4}$$

$$G = \frac{E}{2(1 + \mu)} \tag{5}$$

where, μ is the Poisson’s ratio that may be taken as 0.5 for saturated undrained specimen [3]. The damping ratio D , is a measure of dissipated energy versus elastic strain energy and is computed by

$$D = \frac{1}{4\pi} \frac{A_L}{A_T} \tag{6}$$

Where, A_L = Area enclosed by the hysteresis loop and A_T = Area of the shaded triangle (Fig. 7). The maximum shear modulus (G_{max}) is also calculated by the following empirical equation [6]

$$G_{max} = 1230 \times \frac{(2.973 - e)^2}{(1 + e)} (OCR)^k \sqrt{\sigma'_m} \tag{7}$$

where e is the void ratio; OCR is the overconsolidation ratio; σ'_m is the mean principal effective stress. In Eq. 7, both G_{max} and σ'_m are in pounds per square inch (psi), and k is a constant which depends upon the PI of clay, its value may be taken as zero for sand. For abbreviation, citation ‘Hardin and Drnevich [6]’ is referred as ‘H&D’ in rest of the paper. In Eq. 7, k is assumed zero for sand and to use in SI units, this equation is revised as follows

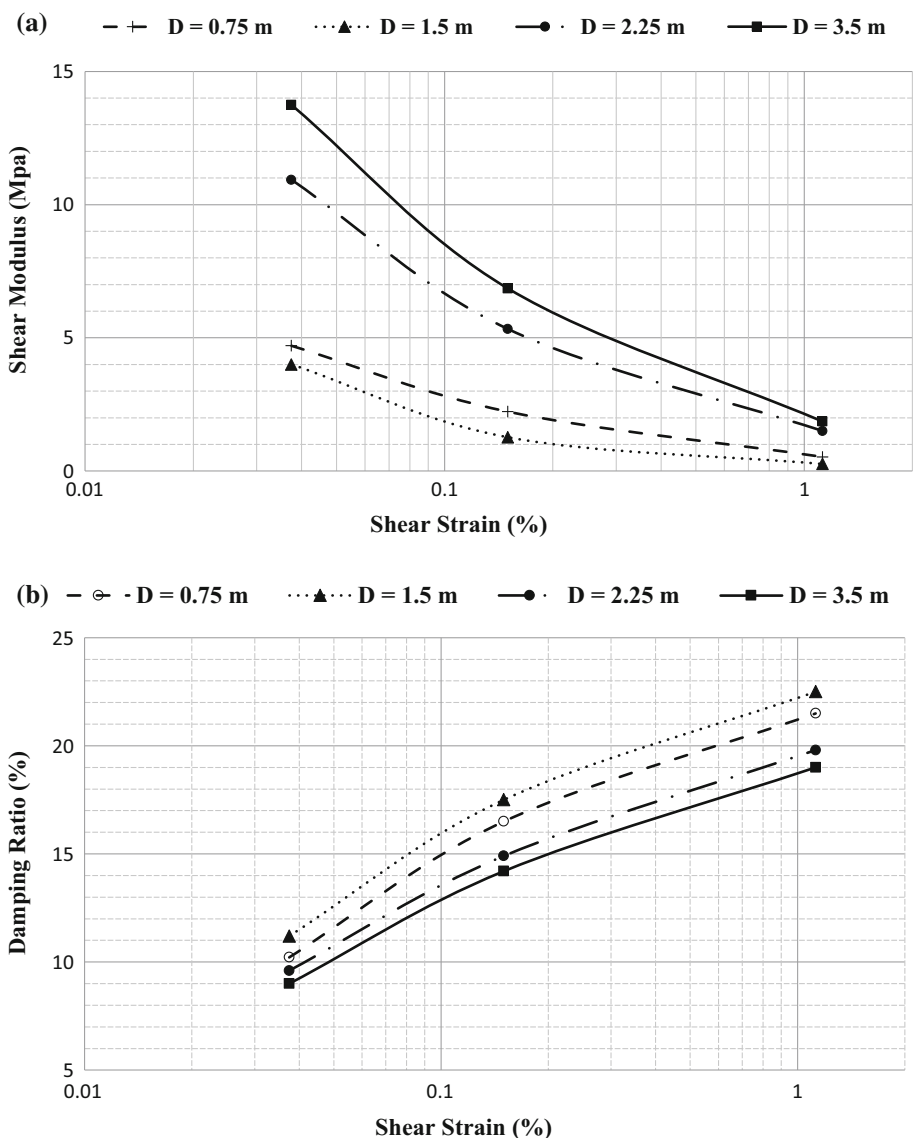
Table 3 Value of G_{max} with depth for Solani Riverbed site

Depth (m)	G_{max} (MPa)	
	Hardin and Drnevich [6]	Bender element (BE)
0.75	38.570	20.713
1.50	52.775	25.171
2.25	74.219	31.412
3.50	86.934	40.669

$$G_{max} = 3230 \times \frac{(2.973 - e)^2}{(1 + e)} \sqrt{\sigma'_m} \quad (8)$$

Where, now both G_{max} and σ'_m are in kPa. If data of bender element tests or empirical relation is not available, the value of G_{max} can be found using V_s (Eq. 2) where V_s can be estimated using correlation such as Kirar et al. [23].

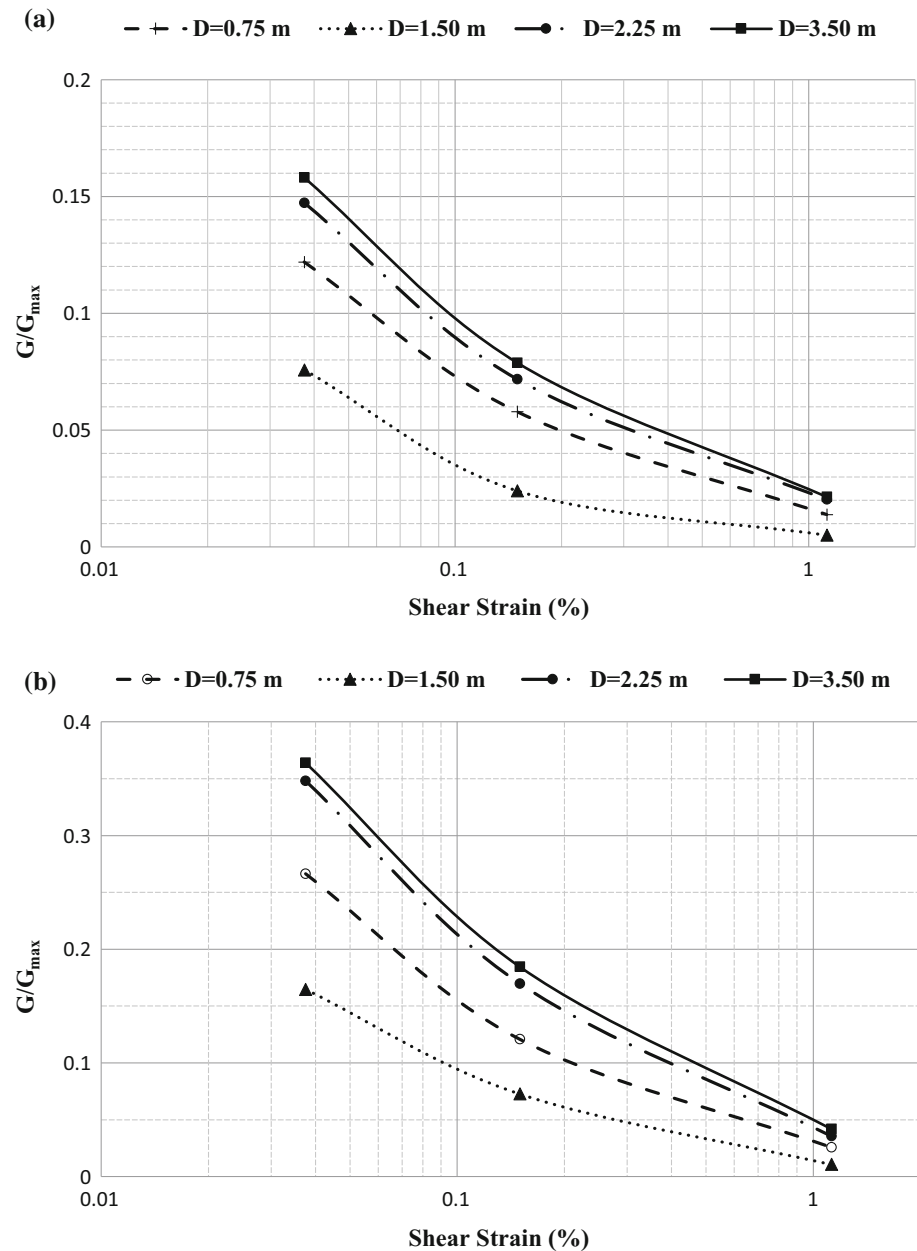
Fig. 8 a Shear modulus versus shear strain for different depths (Solani Riverbed site) and **b** Damping ratio versus shear strain for different depths (Solani Riverbed site)



Effects of Shear Strain on Dynamic Soil Properties

Cyclic triaxial tests were carried out on each sample prepared with 50 mm diameter and 100 mm height, at the in situ relative density and tested at effective confining pressure equivalent to overburden pressure. The water sedimentation method has been used for sample preparation [24]. Procedure of preparation of a saturated sample of sand at a particular relative density is discussed in detail by Maheshwari et al. [12]. Each of the 22 samples was tested at three different axial strains equal to 0.025, 0.1, and 0.75% (corresponding shear strains are 0.0375, 0.15, and 1.125%, respectively). Since for each point i.e. at a particular value of shear strain, a separate test need to be done in cyclic triaxial equipment, therefore, only three points were considered to reduce total number of tests. All the tests were undrained strain controlled at a loading

Fig. 9 **a** Modulus reduction curves (H&D) for different depths (Solani Riverbed site) and **b** Modulus reduction curves (BE) for different depths (Solani Riverbed site)



frequency of 1 Hz with sinusoidal wave. Thus total 66 specimens were tested on the cyclic triaxial system. For each test, a hysteresis loop similar to shown in Fig. 7 is obtained. As per ASTM D3999 [25], each sample is tested for 40 cycles, however, in most of the cases a regular hysteresis loop was obtained just in first 3 cycles.

For each site, values of maximum shear modulus G_{max} at all depths using both BE and H&D methods are presented in tables. Further, for each site, following three graphs are presented

1. Shear Modulus Curves: variation of G with shear strain at all depths.

2. Damping Ratio Curves: variation of D with shear strain at all depths.
3. Modulus Reduction Curves: variation of G/G_{max} with shear strain at all depths using BE.

Further for Solani site, the variation of G/G_{max} with shear strain γ using H&D is also presented. In case of modulus reduction and damping ratio curves, comparison has been performed with that reported in literature.

Solani Riverbed Site

Four samples were collected from different depths. Table 3 lists the values of G_{max} obtained using H&D and BE

Fig. 10 **a** Modulus reduction curves: literature and Solani Riverbed site and **b** Damping ratio curves: literature and Solani Riverbed Site

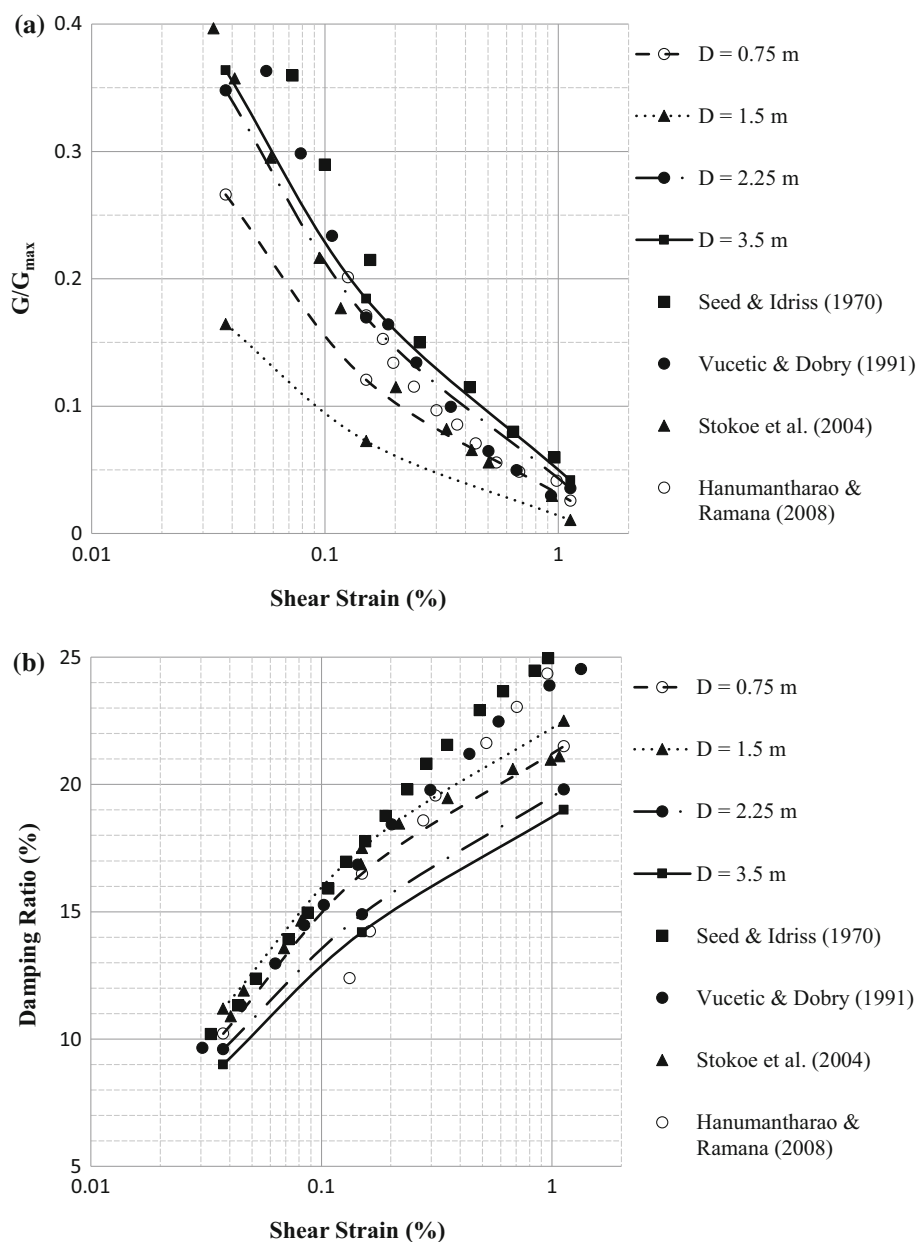


Table 4 Details of relative density and confining pressure

Literature	RD (%)	σ'_3 (kPa)
Seed and Idriss [4]	30–90	48–144
Vucetic and Dobry [27]	–	100
Stokoe et al. [28]	–	25
Hanumantharao and Ramana [29]	55	146

Table 5 Value of G_{max} with depth (Bhagwanpur site)

Depth (m)	G_{max} (MPa)	
	Hardin and Drnevich [6]	Bender element (BE)
0.75	42.519	27.080
1.50	64.152	33.098
2.25	82.498	42.490
3.50	102.375	49.141
4.50	78.381	37.516
6.00	93.601	49.306
8.00	143.241	60.112

methods for this site. It can be observed that value of G_{max} is increasing with depth which is normally expected. However, the values obtained using H&D method are

Fig. 11 **a** Shear modulus versus shear strain for different depths (Bhagwanpur Site), **b** Modulus reduction curves: literature and Bhagwanpur site and **c** Damping ratio curves: literature and Bhagwanpur site

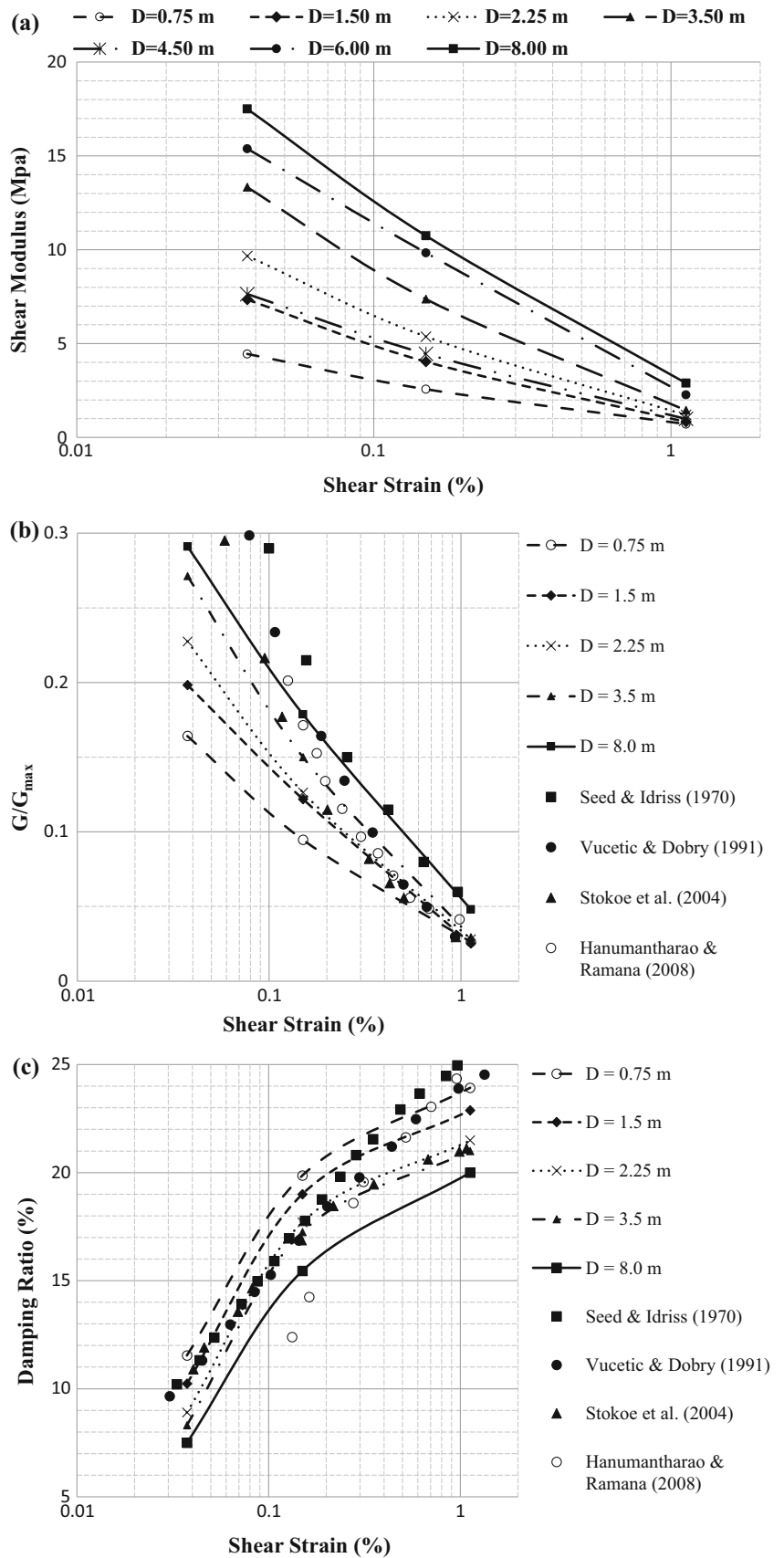


Table 6 Value of G_{\max} with depth (Bahadradab site)

Depth (m)	G_{\max} (MPa)	
	Hardin and Drnevich [6]	Bender element (BE)
0.75	42.636	36.061
1.50	63.910	41.354
2.25	77.761	44.418
3.00	94.900	75.774
6.50	133.536	82.178
7.50	143.590	96.695

almost double of that found using BE method. The difference could be attributed to the fact that former is an empirical method while later is an experimental one.

Figures 8a and 8b show the variation of shear modulus and damping ratio with shear strain for Solani Riverbed site. It can be observed that as the shear strain increases, the shear modulus decreases while damping ratio increases. Further, in general, the shear modulus increases while damping ratio decreases with the depth for the same shear strain.

Figures 9a and 9b show the variation of shear modulus ratio (G/G_{\max}) with shear strain using H&D and BE methods, respectively. Effect of G_{\max} can be clearly observed on modulus reduction curves. For this range of shear strain (0.35% to 1.1%), for sands, the ratio G/G_{\max} using BE (0.36 to 0.01) appears to be in realistic range [2]. Also it can be observed that at high shear strain (i.e. 1.125%), the value of shear modulus ratio is not much different for the samples collected from different depths. The trend of the results is similar to that observed by Maheshwari et al. [26].

Comparison with Literature

It is important to compare the results obtained from the present work with those available in literature. In Figs. 10a, b, the modulus reduction curves (using BE method) and damping curves for Solani site, are compared with the curves recommended by four other researchers i.e. Seed and Idriss [4], Vucetic and Dobry [27], Stokoe et al. [28], and Hanumantharao and Ramana [29]. Though, several other data for Indian sites are available in the literature, for example Sitharam et al. [9], Govindaraju [10], Hanumantharao and Ramana [11, 29] and Maheshwari et al. [12]. However, the result of present study is compared with Hanumantharao and Ramana [29] and other foreign researchers. This is done for better visibility of results. Thus total four prominent data sets are used for comparison. In this comparison, only curves for $PI = 0$ i.e. sandy soil has been selected for other researchers as the samples

tested on triaxial were mostly of sand (Table 2). Details of relative density and confining pressure used by these researchers in the analyses are shown in Table 4. It can be observed that there is no variation of depth, therefore relative density and confining pressure is mostly constant while in Table 2, both of these parameters are varying with depth. Results presented in Figs. 10a, b are further analyzed based on these data.

From Fig. 10a, it can be observed that the modulus ratio obtained in the present analysis for the samples at greater depths (>1.5 m) are in very good agreement with those presented by other researchers. This is attributed to the fact that the value of confining pressure used at depths 0.75 and 1.5 m are smaller than those used by other researchers. Further, there is effect of relative density too. Also it can be observed that for modulus ratio curves, the agreement is very good at high strain but not so good at low strain, with Seed and Idriss [4] and Vucetic and Dobry [27]. Results of the samples from greater depths are in very good agreement with Stokoe et al. [28] and Hanumantharao and Ramana [29] for the whole strain range considered.

Similarly, Fig. 10b compares damping ratio curves for Solani site and it can be observed that the agreements in results are very good at low strain but not so good at high strain. This is contrary to what observed for modulus reduction curves. Further, the agreement is good for the curves collected from all the four depths.

In general for modulus reduction curves, deviation is at low strain and for damping ratio it is at high strain with those presented by Seed and Idriss [4] and Vucetic and Dobry [27]. This variation is justified in view of difference in relative density and confining pressure (Table 2 and Table 4).

Bhagwanpur Site

In case of Bhagwanpur site, seven samples were collected from different depths. Table 5 lists the values of G_{\max} obtained using H&D and BE methods. Again in general, value of G_{\max} increases with depth except at a depth of 3.5 m where the value of G_{\max} is relatively greater than nearby values using both H&D and BE methods. This could be attributed to the fact that the SPT value is also high at 3.5 m (Fig. 3b). Thus the results of SPT, H&D and BE are correlating well. Also, for this site at all depths, values of G_{\max} are greater using H&D method than that using BE method.

Figures 11a–c show the variations of shear modulus, shear modulus ratio (G/G_{\max}) and damping ratio with shear strains respectively. In Fig. 11 (b) and Fig. 11(c) where the results of Bhagwanpur site are compared with other researchers, the data of clay samples (depths 4.5 and 6 m) are not included because data of other researchers are of

Fig. 12 **a** Shear modulus versus shear strain for different depths (Bahdrabad site), **b** Modulus reduction curves: literature and Bahdrabad site, **c** damping ratio curves: literature and Bahdrabad site

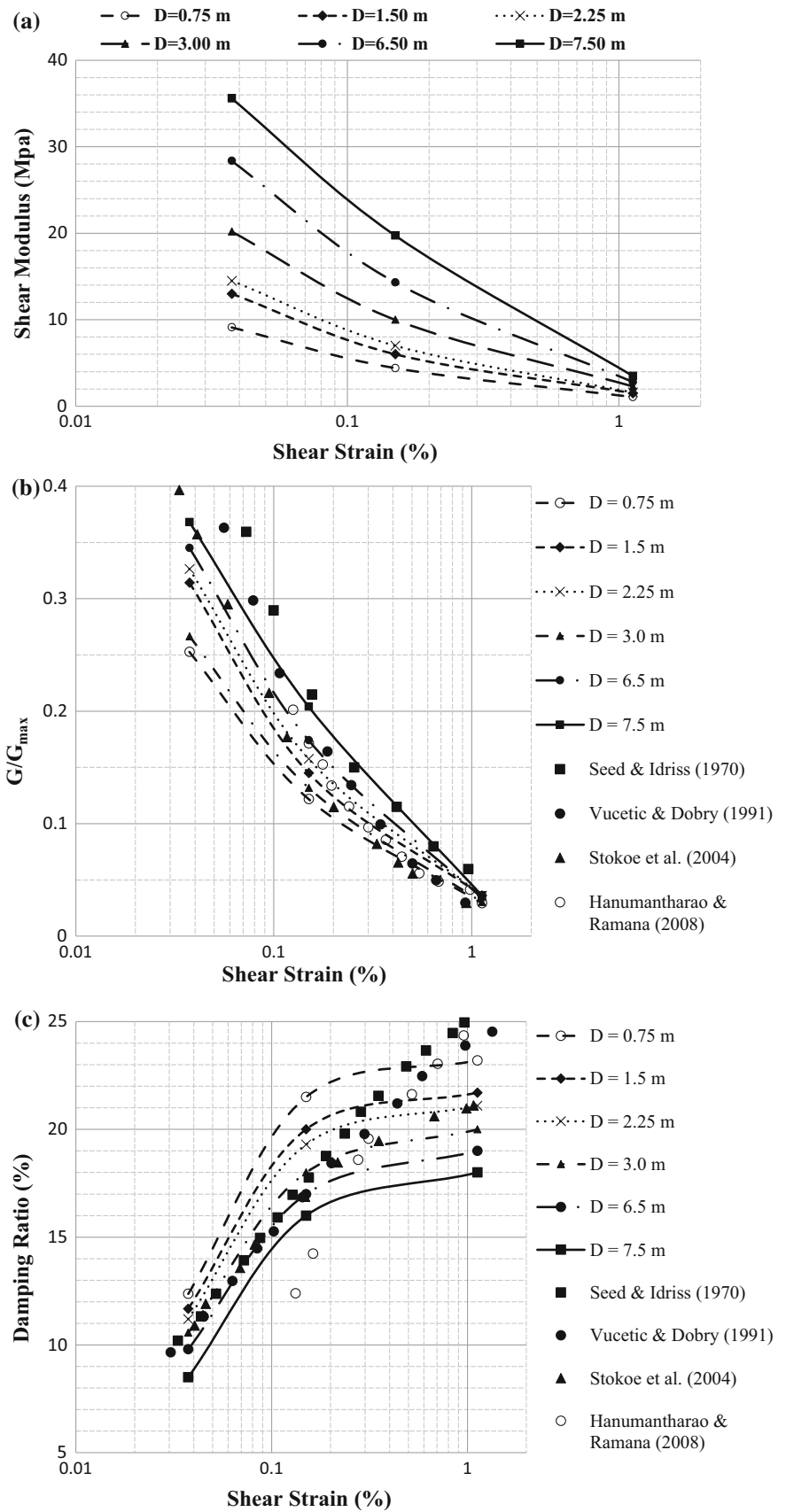


Table 7 Value of G_{\max} with depth (Haridwar site)

Depth (m)	G_{\max} (MPa)	
	Hardin and Drnevich [6]	Bender element (BE)
0.75	40.894	26.997
1.50	59.458	34.040
3.00	97.035	42.869
4.50	108.833	65.136
6.00	129.879	76.990

sand samples. The trends of the results are similar to that observed for Solani site. In general, the shear modulus and shear modulus ratio increases while damping ratio decreases with the depth for the same shear strain. Figures 11b, c also compare the modulus reduction curves and damping curves for Solani site with those presented by other researchers. It can be observed that the agreement is very good, particularly for modulus reduction at high strain and for damping ratio curves at low strain. This is in line with that observed for Solani site.

Bahadradab Site

A total six samples from different depths were collected from the Bahadradab site using SPT. Values of G_{\max} using H&D and BE methods are listed in Table 6 where values are greater using former method. However, using both methods, values are continuously increasing with the depth which is as expected. Figures 12a–c show the variations of shear modulus, shear modulus ratio (G/G_{\max}) and damping ratio with shear strains respectively. The trends of results are similar to that observed for Solani and Bhagwanpur sites i.e. the shear modulus and shear modulus ratio (G/G_{\max}) increases and damping ratio decreases with the depth. Further comparison with the other researchers is also good (Figs. 12b, c).

Haridwar Site

Cyclic triaxial test were conducted on samples collected from five different depths at Haridwar site. Stones were found at 6 m depth, therefore more samples could not be collected. The values of G_{\max} with depths are listed in Table 7 using both BE and H&D methods.

Figures 13a–c show the variations of shear modulus, shear modulus ratio (G/G_{\max}) and damping ratio with shear strains respectively. It can be observed that as the shear strain increases, the shear modulus and shear modulus ratio (G/G_{\max}) decreases and damping ratio increases. It can also be observed that, the shear modulus and shear modulus ratio (G/G_{\max}) increases and damping ratio decreases with

the depth. These trends are in-line with those observed for other sites. Comparison with the other researchers are also good.

Range of Shear Modulus Ratio and Damping Ratio

Table 8, provides the range of shear modulus ratio (G/G_{\max}) and damping ratio for all 4 sites.

From Table 8, it can be observed that values of G/G_{\max} using BE method is almost two times of that obtained using H&D method. Overall for all sites, values of G/G_{\max} using BE method are in the range of 0.36–0.01 which is very much in the realistic range for the level of shear strain applied to the specimens. Also, for all sites the damping ratio falls in the range of 7.5–24 for the level of shear strain used.

Summary and Conclusions

The dynamic soil properties are very important for Geotechnical Earthquake Engineering problems. This paper evaluates the dynamic soil properties at large strain in the Roorkee region. This was never reported in literature so far, even though this has very much practical significance. Results obtained from the present analysis are compared with those presented in literature by four researchers. Trend of results is similar and also in close agreement except at few points. The deviation is justified in view of the difference of relative density and confining pressure. This is the first study where the effects of depth on modulus reduction and damping ratio curves are examined.

This paper summarized the results of laboratory experiments that evaluated the nonlinear dynamic properties of soils at four different sites around Roorkee region. The relative density (RD) of the samples varied from 26 to 63% and in situ vertical effective consolidation stresses (σ'_v) ranged from 12 to 144 kPa. From this study, following conclusions can be drawn:

- Shear modulus and shear modulus ratio (G/G_{\max}) decreases while damping ratio increases with shear strain. For the considered strain range (0.03 to 2%), value of ratio (G/G_{\max}) decreases up to 0.01 while damping ratio increases up to 24%.
- In general, both shear modulus and shear modulus ratio increases with depth. However, damping ratio decreases with the depth. Variation with the depth is as high as four times. However, the effect of depth on shear modulus is quite significant at low strain but decreases as the level of strain increases.

Fig. 13 **a** Shear modulus versus shear strain for different depths (Haridwar site), **b** Modulus reduction curves: literature and Haridwar site and **c** damping ratio curves: literature and Haridwar site

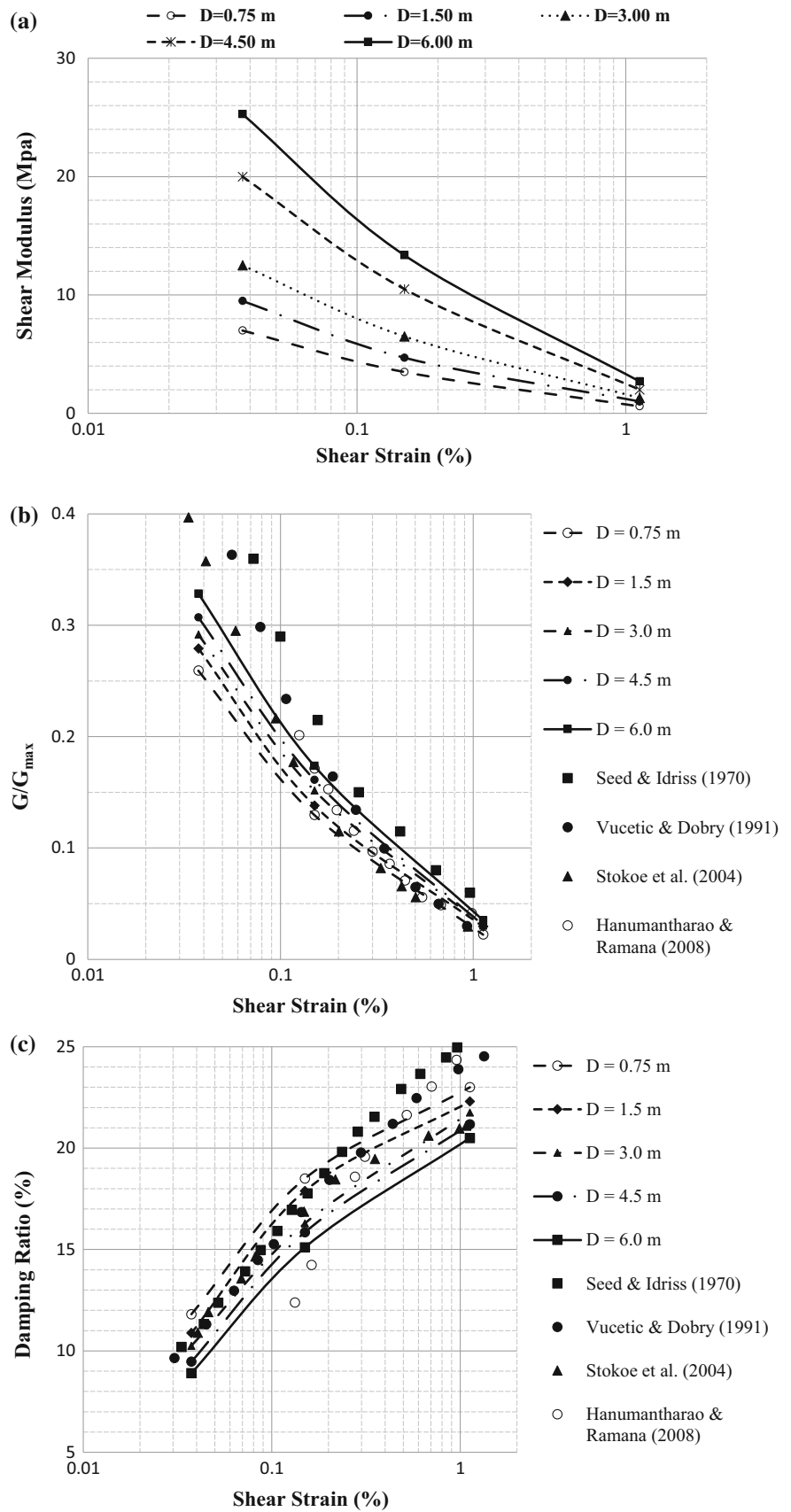


Table 8 Range of shear modulus ratio and damping ratio

Site	Ratio (G/G_{\max})		Damping ratio D (%)
	H&D	BE	
Solani via Aquaduct	0.16–0.005	0.36–0.01	9–22.5
Bhagwanpur	0.16–0.01	0.32–0.03	7.5–24
Bahadrabad	0.25–0.01	0.36–0.03	8.5–23
Haridwar	0.20–0.01	0.33–0.03	9–23

- The results show that the variation in shear modulus and in damping ratios with shear strain is large for the samples collected at higher depth. Where the modulus ratio decreases from 0.36 to 0.01 and damping ratio increases from 7.5 to 24.
- The results obtained from the present study are in very good agreement with those shown by other researchers.

The conclusions and results presented are of very much practical use. The curves presented for modulus reduction ratio and damping ratio can be directly used for analysis and design of foundation in the field for the considered region. One of the limitations of present study is small shear strain range i.e. only high strain range is considered. Though, present study may provide valuable data to local geo-technical community but it can be extended to similar type of sites. Further, present study has many new things compared to literature e.g. effect of depth is considered.

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