Assessment of small strain dynamic soil properties of railway site Agartala, India, by bender element tests

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Abstract
Small strain dynamic properties of soil are the primary input parameter in seismic ground response analysis studies. This study examines the small strain shear modulus ($G_{\text{max}}$) and damping ratio ($\xi$) using bender element (BE) tests on subsoil samples along with evaluation of liquefaction potential (sandy soil) using well-accepted cyclic resistance ratio relationship collected from a railway construction site at Agartala, North Eastern (NE) state of India. The objective of this study is to develop a database and empirical relationship on dynamic properties of Agartala soil which may help to carry out future site-specific seismic hazard studies of Agartala city and other regions with similar soil properties. Experimental results indicate that shear wave velocity ($V_s$), $G_{\text{max}}$, and $\xi$ vary within a wide range such as 60.61 to 234.64 m/s, 5.3 to 104.6 MPa, and 8.9 to 22.1% respectively depending upon the type of soil and a closed form empirical equations are proposed to calculate $G_{\text{max}}$ for different types of soil. Besides, a validation study also highlights well agreement of results on $V_s$ obtained through experimental measurement and field in situ-based relationship. Finally, higher liquefaction susceptibility of the study area is reported based on calculated liquefaction potential index ($LPI$).

Keywords Shear wave velocity · Shear modulus · Bender element test · Damping ratio · Liquefaction

Introduction
Determination of small strain dynamic parameters of soil such as shear wave velocity ($V_s$), maximum shear modulus ($G_{\text{max}}$), and damping ratio ($\xi$) plays a vital role in carrying out several earthquake geotechnical engineering applications, such as ground response analysis, soil-structure interaction analysis, liquefaction potential evaluation, and earthquake resistant design of different foundations as well as geotechnical structures. Bender element (BE) test is found to be well recognized to obtain accurate values of $V_s$, $G_{\text{max}}$, and $\xi$ from element testing (Lings and Greening 2001; Lee and Santamarina 2005; Jung et al. 2009; Kumar and Madhusudhan 2010; Jaya et al. 2012; Gu et al. 2013, 2015; Cai et al. 2015; Ogino et al. 2015; Ruan et al. 2018; Irfan et al. 2021; Wang et al. 2021; Liu and Wei 2022; Alshameri 2022). $V_s$ measurement by BE test was originally introduced by Shirley and Hampton (1978) and further utilized widely by many researchers. BE testing is widely accepted due to its various advantages, such as, (i) low cost experiment and easy to perform in the laboratory, (ii) strains produced in the piezo-electric transducer are in the range of $10^{-6}$ which is well within the elastic limit of all varieties of soil and hence this test is non-destructive, (iii) methods adopted for data interpretation are simpler, and (iv) BE can be incorporated with other apparatus such as oedometer, and triaxial cell (Biot 1956; Dobry and Vucetic 1987; Fam and Santamarina 1995; Brignoli et al. 1996; Brocanelli and Rinaldi 1998; Valle and Stokoe 2012). Several researchers highlighted on field based geophysical experiments which may be applied to obtain $V_s$ profile of the ground (Zhang et al. 2004; Xu et al. 2006; Sil and Sitharam 2013; Kirar et al. 2016; Sil and...
However, determination of $V_s$ from field experiment is an indirect method based on several assumptions. Several correlations based on field $V_s$ measurement and standard penetration test (SPT) also presented in many literatures using which an estimation $V_s$ can also be determined (Jafari et al. 1997; Chien and Oh 2000; Kiku et al. 2001; Hasancebi and Ulusay 2006; Dikmen 2009; Anbazhagan et al. 2012; Sun et al. 2013; Sil and Sitharam 2013; Sil and Haloi 2017; Aataee et al. 2019; Gamal and Elhussein 2021). Hence, it clearly indicates that BE results are more reliable since it is a direct assessment technique and several dynamic parameters, such as $V_s$, $G_{\text{max}}$, and $\xi$ can be calculated from a single experiment. In addition, advanced tests such as resonant column (RC) and torsional shear (TS) test were also performed by many researchers to validate the results of BE test (Karl et al. 2008; Kumar and Madhusudhan 2010; Gu et al. 2013; Yang and Gu 2013; Cai et al. 2010; Dammala and Krishna 2019a, 2019b; Liu et al. 2021). It was reported that BE results were found to be in good agreement with RC and TS tests.

On the other hand, a number of studies also highlighted the importance of confining pressure and range of input frequency which influences $V_s$ measurement during BE test (Jung et al. 2009; Karl et al. 2008; Kumar and Madhusudhan 2010; Jaya et al. 2012; Gu et al. 2013; Cai et al. 2015; Prasanth et al. 2018; Dammala and Krishna 2019a, 2019b; Wang et al. 2021). However, a considerable research on dynamic characterization of various types of soil based on BE experiment was reported (Brignoli et al. 1996; Brocanelli and Rinaldi 1998; Lings and Greening 2001; Reddy et al. 2010; Murillo et al. 2011; Valle and Stokoe 2012; Li et al. 2012; Celik and Canakei 2014; Payan et al. 2016a, 2016b; Gu et al. 2021; Thota et al. 2021; Wang et al. 2021; Alshameri 2022; Liu and Wei 2022). Several studies presented empirical correlations for different types of soil to calculate $V_s$, $G_{\text{max}}$, and $\xi$ based on BE test results (Andrus and Stokoe 2000; Arroyo et al. 2003; Kumar and Madhusudhan 2010; Bai 2011; Jaya et al. 2012; Gu et al. 2015; Wang et al. 2021). Few studies highlighted dynamic characterization of a special soil type (peat) using BE test (Kramer 2000; Kishida et al. 2009; Karl et al. 2008; Rahman et al. 2015; Sarkar and Sadrekarimi 2020, 2022; Wahab et al. 2020). Furthermore, several works on BE testing were also performed in India on mainly alluvium sandy soil (Jaya et al. 2007; Kumar and Madhusudhan 2010; Uma Maheshwari et al. 2010; Jaya et al. 2012; Rahman et al. 2015; Dammala et al. 2017a, 2017b; Dammala and Krishna 2019a, 2019b; Kirar and Maheshwari 2021, Prasanth et al. 2018, Rao and Choudhury 2021, Ram and Mohanty 2021, Thomas and Rangaswamy 2022).

In view of the above context, it is evident that site-specific assessment of dynamic characteristics is important and rational for performing seismic design of various civil infrastructures. Present study is an attempt in this direction to perform dynamic characterization of soil and evaluate dynamic parameters in the form of $V_s$, $G_{\text{max}}$, and $\xi$ which plays a crucial role in assessing the stiffness and response of soil in the event of an earthquake loading. Present study deals with dynamic characterization of four soil types in the form of soft clay (CH), silty sand (SM), peat (Pt), and reddish sandy clayey silt (CI-MI). Low strain, non-destructive test (NDT) based BE tests are conducted on soil samples collected in disturbed (DS) and undisturbed (UDS) form to evaluate dynamic parameters ($V_s$, $G_{\text{max}}$, and $\xi$). Laboratory test results of $V_s$ were further validated with $V_s$ based on in situ standard penetration test (SPT) results. Site-specific empirical equations are also proposed showing variation of $G_{\text{max}}$ with confining pressure and Poisson’s ratio ($\mu$). In addition, liquefaction potential of the site was also evaluated by probabilistic (Cetin et al. 2004) and $V_s$ (Youd et al. 2001) based approaches as the silty sand encountered has close resemblance to the sand that had undergone liquefaction during the recent Dhalai 2017 EQ.

The main aim of the present experimental study is to generate database as well closed form empirical expressions on small strain dynamic properties, such as $V_s$, $G_{\text{max}}$, $\xi$ along with liquefaction potential of typical Agartala soil. Variation of $G_{\text{max}}$ with confining pressure and Poisson’s ratio ($\mu$) for varying soil types is also discussed in the present study which would enable in better estimation of $G_{\text{max}}$. The outcome of this study will help to perform seismic site-specific hazard studies on Agartala with similar soil properties available across the state in a reasonably accurate manner and vulnerability studies of different lifeline structures located in this region.

**Seismicity of the study area and site characterization of target site**

As per seismological zonation, the whole north eastern India including Agartala, the capital of Tripura (23.75° – 23.90° N and 91.25° – 91.35° E) is categorized under highest seismic zone V [IS:1893 (Part 1)-2016]. The seismo-tectonic features of North Eastern area is complicated because of interaction between the active north-south convergence along the Himalaya, the east-west convergence and folding within the Indo-Burma ranges which attribute a deformation known as subduction (Kayal 2008). Two third boundary of Tripura including Agartala and some parts of Assam, Mizoram are sharing international border with Bangladesh. In past, several moderate ($M_s\approx5$ to 7) to few mega range ($M_s\geq7$) earthquakes had occurred in this region mainly due to the presence of numerous active faults (Oldham 1898) and this region was recognized as Indo-Bangla potential seismic zone (Sil and Sitharam 2013; Saha et al. 2020; Deb Nath et al. 2022; Bashir et al. 2022). Based on the past earthquakes and geomorphological survey, this zone was subdivided...
into five parts along with notable earthquakes that have occurred along these faults are presented in Table 1. As per the tectonic framework, it is observed that the study area Tripura lies within Tripura fold belt zone which is in the proximity of the Bengal basin and Indo-Burmese arcs in the west and eastern side respectively. Tripura fold belt zone is moderately active and interpreted as plate boundary activity which caused some moderate to damaging earthquakes (for instance, 1984 Cachar earthquake, $M_w > 5.8$, 1988 Manipur earthquake, $M_w > 5.8$). Appendix-I (Fig. A1) presents seismo-tectonic map of the study area. The upcoming Agartala (India) – Akhaura (Bangladesh) railway connectivity project site within Indian territory at Agartala (23°46′40″ N to 23°55′0″ N and 91°14′0″ E to 91°20′50″ E) is selected as study area for the present work. The total length of the project is 15 km, out of which 5.0 km railway line along with one station yard (at Nischintapur) is under construction within Indian territory and the site is located at southern part of Agartala municipal area (AMC) as presented in Fig. 1. This international railway project funded by Government of India will be instrumental for development of socio-economic condition of both the countries and especially development of whole North Eastern region of our country. The construction of proposed railway embankment and viaduct involved geotechnical investigation of nearly 50 boreholes upto a maximum depth of 18 to 25 meter below ground level for the total stretch of 3.64 km in India side. A total of 50 boreholes were conducted in the total stretch of 3.64 km which were approximately placed at a distance of 70 m from one another. Out of these 50 BHs, a total of 12 BHs were diligently selected which would represent the spatial variation of samples of the target region. Similar methodology was adopted by Yildiz et al. (2022) where 8 boreholes were selected out of 721 boreholes representing characterization of entire area spread over a total land of 213 km².

In addition, both undisturbed and disturbed soil samples were collected from twelve boreholes during site exploration program at site which are reported in present study. It was found that subsoil profile observed at twelve boreholes will reasonably represent the spatial variation of the whole stretch. Another unique feature of the site is low lying soft marshy ground with deposits of decomposed organic matters (or) peatland deposits. A variation in subsoil stratification is observed and accordingly the stratification profile along with field standard penetration test (SPT) results are presented in four categories as shown in Fig. 2a to d respectively.

From the subsoil stratification observed in Fig. 2a to d, it is realized that about fifty percent of the total length of the project comprises of highly compressible peat (Pt) soil to very soft clay (CH) which are mixed with decomposed organic substances with high water content having significant thickness which approximately ranges from 5.0 to 10.0 m below existing ground level (EGL). The organic samples had undergone anaerobic decomposition, and were amorphous in nature. Organic matter (OM) of the soil ranged from 30 to 36% and was considered peat based upon the

Table 1 Maximum level of earthquakes in different tectonic zone at North East (NE) India and Bangladesh

| SI no. | Tectonic block                        | Maximum magnitude of earthquake | Earthquake                  | Reference                      |
|--------|---------------------------------------|---------------------------------|------------------------------|--------------------------------|
| 1      | Bogra fault zone/Bengal basin          | 7.30                            | 1918 Srimangal Earthquake    | Barua et al. 2012; Saha et al. 2020 |
|        |                                       | 6.20                            | 1935 Pabna Earthquake        | Baro and Kumar 2017            |
|        |                                       | 7.10                            | 1941 Andaman Earthquake      | Sil and Sitharam 2013          |
|        |                                       | 6.80                            | 2011 Sikkim Earthquake       |                                |
| 2      | Tripura fold belt zone                 | 5.80                            | 1984 Cachar Earthquake       | Barua et al. 2012             |
|        |                                       | 5.80                            | 1988 Manipur Earthquake      | Kayal et al. 2012             |
|        |                                       | 5.70                            | 2017 Dhalai Tripura          | Saha et al. 2020              |
| 3      | Assam fault zone                       | 6.50                            | 1941 Tezpur Earthquake       | Ben-Menahem et al. 1974       |
|        |                                       | 5.10                            | 2009 Assam Earthquake        | Barua et al. 2012             |
| 4      | Shilong plateau                        | 7.50                            | 1869 Cachar Earthquake       | Bilham and England 2001       |
|        |                                       | 8.10                            | 1897 Great Indian Earthquake | Kayal et al. 2012             |
|        |                                       | 7.10                            | 1923 Meghalaya Earthquake    | Baro and Kumar 2017           |
|        |                                       | 7.10                            | 1930 Dhubri Earthquake       | Bashir et al. 2022            |
|        |                                       | 7.20                            | 1943 Hojai (Assam) Earthquake|                              |
|        |                                       | 7.30                            | 1947 Assam Earthquake        |                              |
|        |                                       | 7.00                            | 1957 Arunachal Pradesh Earthquake|                                |
|        |                                       | 8.50                            | 1950 Great Assam Earthquake |                              |
|        |                                       | 7.30                            | 1988 Manipur earthquake      |                              |
| 5      | Manipur and eastern boundary thrust    | 6.80                            | 1939 Manipur EQ              | Yadav et al. 2010             |
|        |                                       | 7.70                            | 1954 Arunachal Pradesh Earthquake| Kumar et al. 2018           |
|        |                                       | 7.20                            | 1957 Manipur EQ              |                              |
|        |                                       | 6.40                            | 1963 Manipur EQ              |                              |
|        |                                       | 6.70                            | 2016 Manipur EQ              |                              |
previous studies which have indicated that (OM≥28%) can be used as a limiting criterion to distinguish between organic soil and peat (e.g. Nie et al. 2012, Chen et al. 2018, Paul et al. 2018, Ozcan et al. 2020, Debnath et al. 2021). This highly compressible poor layer is found to be followed by moderate to good bearing layers of silty sand/clayey silty sand (SM/SC) to poorly graded clean sand/gravelly sand (SP/GP) upto termination depth. This may be confirmed from stratification presented for BH01 to BH06. However, BH07 to BH09 also confirms presence of clay layer (CI-CH) with slight traces of organic matter having lesser thickness at shallow depth. On the other hand, BH10 to BH12 indicates that the top virgin layer consists of medium/stiff clayey silt/silty clay layer having moderate thickness and further followed by silty/clayey sand (SM-SC) and poorly graded clean sand (SP) respectively upto termination depth. However, it may be noted that the horizontal extent of bottom sand layer is found to be continued longitudinally irrespective of boreholes. The subsoil profile encountered in the boreholes indicates relatively younger alluvial/fluvial sedimentation formation of Holocene age which is also experienced in most of the parts of Agartala basin and can be considered typical Agartala soil. Total nine undisturbed (UDS) and three disturbed (DS) samples are collected from different boreholes. Nine UDS samples are collected from shallow depth highly compressible soft to medium cohesive/very soft peat and clayey layers available in BH01-BH06 and medium to stiff soil layers of sandy clayey silt (CI-MI) are present in BH10-BH12. Similar nature of alluvium nature of subsoil deposits is available in various parts of India. Tables 2 and 3 present details of physical and static engineering parameters of UDS and DS samples respectively. It is to be noted that consolidated undrained triaxial tests were carried out on both UDS and reconstituted DS.
samples prepared maintaining identical in situ density and at varying confining pressure (50 kN/m², 100 kN/m², and 150 kN/m²). However, the shear strength parameters are reported in terms of total stress. Details regarding the test results of triaxial testing are stated in Table 4. Graphs derived from the test results to obtain shear strength parameters are stated in Fig. 3. Sample graphs for all soil types showing variation of deviator stress with respect to axial strain are stated in Appendix-I (Fig. A2). Falling and constant head permeability test is performed on reconstituted soil samples to obtain hydraulic conductivity of soil (i.e. coefficient of permeability) of all types. It is found that peat and sand poses least and highest permeability respectively. All static tests are performed following relevant Indian standards. A comparison of loose sandy nature soil available from the target site is drawn in the form of grain size graph with liquefiable soils available across various parts of India indicating that the sandy soil of Indo-Bangladesh region is highly prone to liquefaction.

**Bender element test and sample preparation**

**Test apparatus**

BE tests have grown popularity in the recent years due to the NDT mode of testing, and wide range of frequencies which can be applied on the soil in the form of sine waves. Waves being low strain (<10⁻⁶%) are well within the elasticity of most of soils, which have gained popularity, since its first usage Shirley and Hampton (1978). Piezoelectric transducers present in the BE apparatus consist of two elements, namely transmitter and receiver, which are placed at top and bottom. At the top, S-wave transmitter is kept and the receiver is kept at bottom, which are shown in a schematic diagram in Appendix-I (Fig. A3). The elements are 2 mm in width and 10 mm in height. Sinusoidal waveforms having varying excitation frequencies are generated with a maximum voltage of 20V which is controlled by the power generator. Input frequency of the apparatus can extend up to a maximum value...
| Borehole no. | Identification of specimen | Description of soil |
|-------------|---------------------------|---------------------|
| BH01        | U0101                     | Blackish gray very soft peat |
|             |                           | 0, 22, 44 (OM-34)    |
| BH02        | U0201                     | Blackish gray very soft peat |
|             |                           | 0, 26, 44 (OM-30)    |
| BH03        | U0301                     | Blackish gray very soft peat |
|             |                           | 0, 24, 40 (OM-36)    |
| BH04        | U0401                     | Light gray very soft clayey silt |
|             |                           | 7, 38, 55           |
| BH05        | U0501                     | Light gray very soft clayey silt |
|             |                           | 6, 40, 54           |
| BH06        | U0601                     | Dark gray very soft clayey silt mixed with decomposed organic matters |
|             |                           | 4, 38, 50 (OM-8)    |
| BH10        | U1001                     | Brownish medium to stiff clayey silt |
|             |                           | 19, 39, 42          |
| BH11        | U1101                     | Brownish medium to stiff clayey silt |
|             |                           | 30, 42, 28          |
| BH12        | U1201                     | Brownish medium to stiff clayey silt |
|             |                           | 34, 46, 20          |
of 10 kHz. For the present study, sine waves were generated within a wide range of input excitation frequencies of 0.50–10 kHz, due to limitations of the apparatus. Source and receiver signals are recorded in the digital oscilloscope. The soil specimens in the triaxial cell are kept under isotropic confining pressure of 50, 100, 200, and 400 kPa before the test for a considerable period of time, to undergo adequate amount of consolidation. Such high range of cell pressure was taken into consideration following several studies published elsewhere (e.g. Leong et al. 2009; Jaya et al. 2012; Gu et al. 2013; Cai et al. 2015; Gu et al. 2015; Subramanian et al. 2015; Kirar and Maheshwari 2021; Xia et al. 2019; Debnath et al. 2022). These selected ranges of cell pressure were considered herein since these ranges simulate the in situ pressure condition with reasonable accuracy.

**Sample collection and preparation**

Undisturbed soil (UDS) samples are collected from site by sampling tube having 10 cm diameter and 40 cm length having inside clearance ranging from 1 to 3% and area ratio < 10% conforming to IS:11594-1985. Three sets of UDS samples were collected for Pt, CH, and CI-MI soil types from BH 01 to BH 03, BH 04 to BH 06, and BH 10 to BH 12 respectively from a depth varying from 1.50 to 4.50 m below EGL. Collected sampling tubes were waxed on both the open ends, covered with plastic packets, and taped immediately in order to minimize the moisture loss, as per IS:11594-1985. Samplers were kept vertically in upright position in a room having 85% humidity with a constant room temperature of 20°C in order to minimize moisture loss from UDS samples and similar procedure of storing samples in vertical manner, at high humid conditions and low temperature can be referred from Wehling et al. (2003). SM samples were obtained in disturbed form BH 07, 08, and 09 by split spoon sampler. UDS specimens of CH, Pt, and CI-MI were prepared from UDS samplers as shown in Appendix-I, Fig. A4 (a) maintaining a height (H) to diameter (D) ratio of 2.00. Reconstituted samples were prepared for SM soils at relative density of 40% (in situ density). The in situ density was evaluated by correlating with standard penetration test (SPT) values obtained at various depth (Fig. 2) as proposed by Mittal and Shukla (2012). Further maximum and minimum densities were also evaluated which are presented in Table 3. The maximum and minimum densities of sand are calculated by determining the weight of sand required to be filled in the split mould (having volume = 196.34 cm³) in loose state (i.e. without tamping energy) and dense state (i.e. with tamping energy) respectively. By dividing the weight of sample required to fill up the split mould with the known volume of mould, density is determined. Furthermore, the sand samples are prepared by tamping the sand in three layers with \( H/D = 2 \) under in situ condition which is presented in Appendix-I, Fig. A4 (b).
Methodology of shear wave velocity evaluation by bender element test

Shear wave velocity measurement

BEs are mounted at the top and bottom of a triaxial apparatus as shown in Appendix-I, Fig. A3 (a) and the shear waves are being transmitted through the piezoceramic BE and the signals are recorded in the digital oscilloscope. Sample graph obtained for soft clay (CH) is presented in Fig. 4 and results obtained for other soil samples (Pt, SM, and CI-MI) are presented in Appendix-I, Fig. A5 (a) to (c). Determination of travel time to evaluate $V_s$ can be carried out by three different methods: (a) the first time of arrival, (b) the first peak to peak, and (c) the cross-correlation method. First peak to peak method was adopted for the present study due to its simplicity in travel time calculation which was also adopted by several other researchers (Leong et al. 2009; Youn et al. 2008; Cai et al. 2015; Ruan

Table 4. Details of triaxial test results

| Soil type               | Identification of sample | Principal stress, $\sigma_1$ (kN/m$^2$) | Cell pressure, $\sigma_3$ (kN/m$^2$) | Deviator stress, $\sigma_d$ (kN/m$^2$) | TX test results [$c_y$ (kN/m$^2$) / $\phi$ (degs)] |
|-------------------------|--------------------------|------------------------------------------|-------------------------------------|---------------------------------------|---------------------------------------------------|
| Peat (Pt)               | BH 01 (U0101)            | 62.70                                    | 50                                  | 12.70                                 | 6.37/0                                            |
|                         |                          | 112.82                                   | 100                                 | 12.82                                 |                                                   |
|                         |                          | 163.06                                   | 150                                 | 13.06                                 |                                                   |
|                         | BH 02 (U0201)            | 65.20                                    | 50                                  | 15.20                                 | 7.35/0                                            |
|                         |                          | 115.52                                   | 100                                 | 15.52                                 |                                                   |
|                         |                          | 165.68                                   | 150                                 | 15.68                                 |                                                   |
|                         | BH 03 (U0301)            | 63.37                                    | 50                                  | 13.37                                 | 6.86/0                                            |
|                         |                          | 113.49                                   | 100                                 | 13.49                                 |                                                   |
|                         |                          | 163.94                                   | 150                                 | 13.94                                 |                                                   |
| Soft clay (CH)          | BH 04 (U0401)            | 79.52                                    | 50                                  | 29.52                                 | 11.76/2°                                          |
|                         |                          | 134.03                                   | 100                                 | 34.03                                 |                                                   |
|                         |                          | 187.04                                   | 150                                 | 37.04                                 |                                                   |
|                         | BH 05 (U0501)            | 76.48                                    | 50                                  | 26.48                                 | 10.78/2°                                          |
|                         |                          | 131.40                                   | 100                                 | 31.40                                 |                                                   |
|                         |                          | 185.38                                   | 150                                 | 35.38                                 |                                                   |
|                         | BH 06 (U0601)            | 77.93                                    | 50                                  | 27.93                                 | 12.75/1°                                          |
|                         |                          | 129.05                                   | 100                                 | 29.05                                 |                                                   |
|                         |                          | 183.16                                   | 150                                 | 33.16                                 |                                                   |
| Silty sand (SM)         | BH 07 (DS07/4.5)         | 129.89                                   | 50                                  | 79.89                                 | 0/26°                                             |
|                         |                          | 260.39                                   | 100                                 | 160.39                                |                                                   |
|                         |                          | 374.81                                   | 150                                 | 224.81                                |                                                   |
|                         | BH 08 (DS08/6.0)         | 125.17                                   | 50                                  | 75.17                                 | 0/25°                                             |
|                         |                          | 253.87                                   | 100                                 | 153.87                                |                                                   |
|                         |                          | 367.97                                   | 150                                 | 217.97                                |                                                   |
|                         | BH 09 (DS09/6.0)         | 140.93                                   | 50                                  | 90.43                                 | 0/28°                                             |
|                         |                          | 275.40                                   | 100                                 | 175.40                                |                                                   |
|                         |                          | 415.65                                   | 150                                 | 265.65                                |                                                   |
| Reddish sandy clayey    | BH 10 (U1001)            | 126.10                                   | 50                                  | 76.10                                 | 32.36/3°                                          |
| silt (CI-MI)            |                          | 180.76                                   | 100                                 | 80.76                                 |                                                   |
|                         |                          | 234.23                                   | 150                                 | 84.23                                 |                                                   |
|                         | BH 11 (U1101)            | 123.55                                   | 50                                  | 73.55                                 | 33.34/2°                                          |
|                         |                          | 177.03                                   | 100                                 | 77.03                                 |                                                   |
|                         |                          | 230.47                                   | 150                                 | 80.47                                 |                                                   |
|                         | BH 12 (U1201)            | 137.56                                   | 50                                  | 87.56                                 | 35.30/5°                                          |
|                         |                          | 195.10                                   | 100                                 | 95.10                                 |                                                   |
|                         |                          | 257.33                                   | 150                                 | 107.33                                |                                                   |
et al. 2018; Thomas and Rangaswamy 2022). In the first peak to peak method, arrival time refers to the time ($t_{pp}$) between peak of source signal and received signal ignoring initial weak portion of the signal which is attributed due to near field effects (if any). The length travelled by the signal is the height of the soil sample which is being referred to as length of travel time ($L_{tt}$). The corresponding time required to travel that length recorded in the digital oscilloscope helps in evaluating the $V_s$ of the soil which can be conducted at varying confining pressure and input frequency ($f_{in}$). Four different confining pressure of 50, 100, 200, and 400 kPa are considered in present study. The $V_s$ produced by BE is always accompanied with compression, and reflected shear waves which are basically signals of opposite polarity and are referred to as near field effects which appears before any signal, as shown in Appendix-I, Fig. A6. The presence of near-field effect affects the shape of the received signal and it creates difficulty in finding accurately the arrival of shear waves in the BE test (e.g. Brignoli et al. 1996, Jovicic et al. 1996, Cai et al. 2009, Leong et al. 2009, Chan et al. 2010b, Kumar and Madhusudhan 2010, Gu et al. 2015, Wang et al. 2021). Hence, simultaneous increase of $f_{in}$ and by varying the frequencies at higher intensities leads to a better and reliable prediction of travel time of shear waves. Present study considers range of frequency from 0.5 to 10 kHz to minimize near field effects. The shear wave velocity ($V_s$) of soil is calculated for a frequency range of 0.5 to 10 kHz. A convergence study is performed to find out the $V_s$ which remains constant with an increase in frequency and confirms reducing distortion caused by near field effects. A non-dimensional parameter, i.e. ratio of wavelength path ($L_{tt}$) to that of wavelength ($\lambda$) of a particular signal is used for the convergence study. Figure 5 presents a sample result of measured $V_s$ with respect to $L_{tt}/\lambda$ for clayey soil whereas sample results for other three soil types are presented in Appendix-I, Fig. A7 (a) to (c). Each soil type is tested under four different confining pressures which are also presented in the same figures. It is observed from Fig. 5 and Appendix-I, Fig. A7 (a) to (c) that the converging trend of $V_s$ is observed in four different types of soil at different values of $L_{tt}/\lambda$ which lies within 1.40 to 4.40. Furthermore, Fig. 6 also indicates range of $V_s$ encompassing confining pressure of 50, 100, 200, and 400 kPa in order of 74.32–108.65, 145.45–168.44, 60.61–76.21, and 148.15–234.64 m/s for CI-CH, CI-MI, Pt, and SM type soil respectively. Experimental $V_s$ of all twelve samples are presented with respect to confining pressure in Fig. 6. An increasing trend of $V_s$ with increase in confining pressure is observed irrespective of type of soil samples; similar trend was observed by Leong et al. (2009), Kumar and Madhusudhan (2010), Jaya et al. (2012), Gu et al. (2015), and Dammala and Krishna (2019a, 2019b). The increase in $V_s$ can be mainly attributed to the change in the orientation of the soil particles and resulting in a more denser/stiffer matrix resulting in higher $V_s$ values.

**Results and discussion**

**Shear wave velocity of soil ($V_s$)**

Shear wave velocity ($V_s$) of soil is calculated for a frequency range of 0.5 to 10 kHz. A convergence study is performed to find out the $V_s$ which remains constant with an increase in frequency and confirms reducing distortion caused by near field effects. A non-dimensional parameter, i.e. ratio of wavelength path ($L_{tt}$) to that of wavelength ($\lambda$) of a particular signal is used for the convergence study. Figure 5 presents a sample result of measured $V_s$ with respect to $L_{tt}/\lambda$ for clayey soil whereas sample results for other three soil types are presented in Appendix-I, Fig. A7 (a) to (c). Each soil type is tested under four different confining pressures which are also presented in the same figures. It is observed from Fig. 5 and Appendix-I, Fig. A7 (a) to (c) that the converging trend of $V_s$ is observed in four different types of soil at different values of $L_{tt}/\lambda$ which lies within 1.40 to 4.40. Furthermore, Fig. 6 also indicates range of $V_s$ encompassing confining pressure of 50, 100, 200, and 400 kPa in order of 74.32–108.65, 145.45–168.44, 60.61–76.21, and 148.15–234.64 m/s for CI-CH, CI-MI, Pt, and SM type soil respectively. Experimental $V_s$ of all twelve samples are presented with respect to confining pressure in Fig. 6. An increasing trend of $V_s$ with increase in confining pressure is observed irrespective of type of soil samples; similar trend was observed by Leong et al. (2009), Kumar and Madhusudhan (2010), Jaya et al. (2012), Gu et al. (2015), and Dammala and Krishna (2019a, 2019b). The increase in $V_s$ can be mainly attributed to the change in the orientation of the soil particles and resulting in a more denser/stiffer matrix resulting in higher $V_s$ values.
Small strain shear modulus ($G_{\text{max}}$)

Small strain shear modulus ($G_{\text{max}}$) was recognized as a key parameter in earthquake geotechnical engineering design practice. Besides this, $G_{\text{max}}$ plays an important role in characterization of soil deformability from the viewpoint of propagation of waves. The well-accepted expression of $G_{\text{max}}$ derived from the elastic wave propagation analysis is presented herein which is available in numerous published literatures (e.g. Leong et al. 2009; Bai 2011; Jaya et al. 2012; Gu et al. 2013; Dammala and Krishna 2019a, 2019b).

\[ G_{\text{max}} = \rho V_s^2 \]  
where $\rho$ = density of soil specimen.

Two main parameters that mainly influence $G_{\text{max}}$ are confining pressure and initial void ratio ($e$) as suggested elsewhere (Chien and Oh 2002; Mayoral et al. 2008; Bai et al. 2010; Dammala et al. 2018). Confining pressure plays a vital role in determining the stiffness of the material. Variation of $G_{\text{max}}$ with confining pressure for four types of soil are shown in Fig. 7. $G_{\text{max}}$ values varied from 10.4–22.4, 41.8–57.1, 5.3–8.7, and 38.6–104.6 MPa in case of CH, CI-MI, Pt, and SM respectively. The increase in $G_{\text{max}}$ values is mainly attributed due to their structural composition and variation in their engineering parameters such as void ratio, density, shear strength parameters, and varying structural composition which can be referred from Tables 2 and 3. Such variation in $G_{\text{max}}$ value on varying soil types can also be referred from published literatures available elsewhere (Stokoe et al. 1986, Boulanger et al. 1998, Kramer 2000, Wehling et al. 2003, Bai 2011, Wang et al. 2021). An average percentage of increment in $G_{\text{max}}$ with confining pressure for different type of soil is presented in Table 5. It is observed that the effect of confining pressure is maximum and minimum in order of 58.35% and 15.81% respectively in case of SM and CI-MI type of soil respectively.

Proposed empirical relationship to determine $G_{\text{max}}$

Site-specific evaluation of $G_{\text{max}}$ with the help of empirical equation has been proposed by various researchers (Chien and Oh 2002; Jaya et al. 2012; Dammala and Krishna 2019a, 2019b; Wang et al. 2021) which are also used in site response studies. Equations proposed by various researchers are different. However, these can be broadly classified into two forms, as presented in Equations (2) and (3) can be referred from Seed and Idriss (1970) and Seed et al. (1986).

\[ G_{\text{max}} = A \times F(e) \times p^{(1-m)} \times \sigma_c^m \]  
\[ G_{\text{max}} = 1000 \times (K_2)_{\text{max}} \times \sigma_c^m \]

From different forms of equations for evaluating site-specific $G_{\text{max}}$, equation proposed by Hardin (1978) as stated in Equation (4) has been adopted for the present study which is an extension of Equation (2) by adopting $F(e)$ as $I/((0.3+0.7e^2)$, as the equation proposed is one of the most widely adopted equation to evaluate $G_{\text{max}}$ for various earthquake applications and has been adopted by various researchers due its dimensional consistency and $F(e)$
value can be adopted for soil having wide range of void ratio and \((K_2)_{\text{max}}\) is maximum soil modulus coefficient. The value of \(F(e)\) in Equation (4) is referred from Saxena and Reddy (1989).

\[
G_{\text{max}} = \frac{A \times (P_a)^{1-m} (\sigma'_c)^m}{(0.3 + 0.7e^2)}
\]

(4)

where \(G_{\text{max}}\) is maximum shear modulus (MPa), \(A\) is a constant term depended on soil, \(P_a\) is standard atmospheric pressure, \(\sigma'_c\) confining pressure acting on soil specimen, and \(e\) is considered initial void ratio in present study. \(m\) is stress depending factor.

In the present study, BE tests were performed on four types of soil samples at varying confining pressure. Non-linear regression curve is used to best fit the results and the equation of the curve is arranged in the form of Equation 4 in order to obtain the constant parameters for different soil types. Figure 8 shows the regression equation of different types of soil and the various constant parameters such as \(A\), \(m\) and coefficient of correlation \((R^2)\) are also stated. The values of \(A\), \(m\), and \(R^2\) can be evaluated from Fig. 8 for SC, CI-MI, Pt, and SM taking initial void ratio into consideration. Constant term parameters obtained from the present study are compared with well-accepted expressions reported in literatures as presented in Table 6. It is observed that the variation of fitting parameters is significant with the existing values which indicate the importance of development of site-specific parameters. Hence, the predicted expression of \(G_{\text{max}}\) will be more useful for typical soil deposits available in Tripura.

**Estimation of Poisson’s ratio (\(\mu\)) and correlation with \(G_{\text{max}}\)**

Poisson’s ratio (\(\mu\)) was computed by performing consolidated undrained triaxial (CUTX) test on four types of soil samples. Saturated specimens of 38 mm diameter and 76 mm height were consolidated at varying confining pressure of 50, 100, 150, 200, and 400 kPa in the triaxial cell for 24 h and change in volume (\(\Delta V\)) of the sample was recorded from the volume change gauge of the triaxial apparatus. A sample graph to determine \(\Delta V\) of Pt is shown in Appendix-I, Fig. A8. Post-consolidation length of sample was obtained from Equation (5) and from Equation (6) change in length (\(\Delta L\)) is obtained, and subsequently, \(\mu\) was evaluated for the soil sample by using Equation (7). The equations presented here in can be referred from Mittal and Shukla 2012.

\[
L = L_0 \left(1 - \frac{\Delta V}{3V_0}\right)
\]

(5)

where \(L=\) post-consolidation length, \(L_0=\) initial length, \(\Delta V=\) change in volume (from Appendix-I, Fig. A8), \(V_0=\) initial volume of soil sample.

\[
\Delta L = L - L_0
\]

(6)

\[
\mu = \frac{1}{2} \left[1 - \frac{1}{\pi d^2} \left(\frac{\Delta V}{\Delta L}\right)\right]
\]

(7)

where \(\Delta L=\) change in length, \(d=\) diameter of specimen. Poisson’s ratio for all soil types is obtained at similar confining pressure which was used for obtaining \(V_s\) and

**Table 5. Variation of maximum shear modulus \((G_{\text{max}})\) in percentage with change of confining pressure (CP)**

| SI no. | Identification of sample | % change in \(G_{\text{max}}\) due to variation in CP |
|--------|--------------------------|---------------------------------------------|
| 1      | Soft clay (CH)            | 46.19                                       |
| 2      | Clayey silt (CI-MI)       | 15.81                                       |
| 3      | Peat (Pt)                 | 25.74                                       |
| 4      | Sandy soil (SM-SC/SM)     | 58.35                                       |
$G_{\text{max}}$ in the present study. To describe the effect of confining pressure on $\mu$, the normalized Poisson’s ratio ($\mu/F(e)$) is presented with respect to confining pressure values. $F(e)$ is considered equivalent to $1/(0.3+0.7e^2)$ as discussed in previous section. Finally, a curve is best fitted by a power law relationship as presented in Fig. 9. From the observation of results, it can be stated that $\mu$ is not constant for a material but is dependent on $e$ and confining pressure. Furthermore, from the results noted in previous section, it is observed that both $G_{\text{max}}$ and $\mu$ are dependent on common parameters, i.e., confining pressure and $F(e)$. Hence, an empirical correlation is established between $G_{\text{max}}$ and $\mu$. In fact, development of such correlations is meaningful as simultaneous measurement of both the parameters is difficult and expansive too. However, similar correlation was proposed by Gu et al. (2013) for sandy soil. Figure 9 presents correlations between $\mu$ and $G_{\text{max}}$ for CI-MI, CH, Pt, and SM soil samples collected from study area. Similarly, a power law best fitted curve is proposed to represent the empirical relationship between $\mu$ and $G_{\text{max}}$. Empirical correlations established between $\mu$ and $G_{\text{max}}$ in the form of equations are stated below. The proposed equation of 8, 9, 10, and 11 is adopted from the published literature of Gu et al. (2013).

$$\mu = 23.56 \left( G_{\text{max}} \right)^{-0.702}, \text{silty clay soil (CI – MI)} \quad (8)$$

$$\mu = 1.14 \left( G_{\text{max}} \right)^{-0.301}, \text{soft clay (CH)} \quad (9)$$

$$\mu = 2.32 \left( G_{\text{max}} \right)^{-0.633}, \text{peat (Pt)} \quad (10)$$

$$\mu = 1.72 \left( G_{\text{max}} \right)^{-0.311}, \text{silty sand(SM)} \quad (11)$$

**Comparison of field and laboratory determination of $V_s$**

Attempt was also made to calculate $V_s$ based on in situ standard penetration test (SPT) results. Depth-wise SPT results for each borehole are available in Fig. 2. Correlations proposed by Dikmen (2009) are used for evaluating in situ $V_s$ for the present study. The correlations proposed were based on extensive study which compares in situ $V_s$ obtained by MASW and data set of SPT borings (0.5 to 30.45m) at 264 different locations. The empirical expressions proposed by Dikmen (2009) for different types of soil are presented herein in Equations 12, 13, and 14.

$$V_s = 58N_{60}^{0.39}, \text{for all soil types} \quad (12)$$

$$V_s = 44N_{60}^{0.48}, \text{for cohesive soils} \quad (13)$$

$$V_s = 73N_{60}^{0.33}, \text{for cohesionless soils} \quad (14)$$

Table 7 presents comparison of $V_s$ obtained from correlation with $N_{60}$ value and BE test carried out in present study on UDS samples at similar confining pressure of 100 kPa. It is observed that $V_s$ obtained from empirical correlation exhibits well agreement with experimental results.

**Determination of damping ratio of soil ($\xi$)**

Three most widely adopted methods used to determine soil $\xi$ from BE test are Rayleigh-Ritz analysis, analysis based on multiple reflections, and self-correcting method (Karl et al. 2008). However, multiple reflections-based analysis is utilized in present study to estimate $\xi$. This method has advantage of measurement of same wave on sender and receiver.
cap of BEs. Multiple reflection in BE appear as repetitions of first arrival with decayed peaks which gets shifted with time.

Time histories recorded by BE are denoted by \( d_i(t) \) and \( d_j(t) \) where the subscript ‘i’ and ‘j’ denotes order of first and subsequent arrival of shear waves, represented as a function of time at applied input frequencies of 3, 4, and 5 kHz respectively for the present study, as presented in Appendix-I, Fig. A9. Similar methodology was adopted by Karl et al. (2008). Arrival of shear waves is separated by rectangular windows with the peak showing maximum amplification of that particular window. Obtained time histories divided into rectangular windows as per the following equation suggested elsewhere (Karl et al. 2008).

\[
\xi = \frac{SV_s}{2\pi}
\]

where \( V_s \) = shear wave velocity of soil, \( S \) = slope of linear trend line.

**Correction for boundaries**

In a BE cell due to presence of piezoceramic BEs which are present at top and bottom plate of triaxial cell supporting a soil specimen, impedance changes are bound to happen between soil and piezoceramic BEs. Such changes of impedances at an interface causing reflection and transmission of waves are to be minimized by boundary propagation factor and the equation presented herein can be referred from Karl et al. (2008).

\[
c = \frac{1 - K}{1 + K}
\]

where \( c \) = boundary constant factor, \( K = \rho V_s \), \( \rho \) = density of soil.

Santamaria et al. (2001) reported a frequency dependent boundary constant factor \( c(\omega) \) for a frequency range below 1 kHz, whereas in case of frequencies higher than 1 kHz, boundary constant \( c \) is used. In order to minimize the effect due to boundary conditions, a constant boundary condition factor \( c \) is used which can be evaluated from Equation 17 for reducing the boundary condition effects on \( \xi \).

**Example**

Time histories are obtained for four soil types where arrival of peaks upto 3\(^{rd}\) order adopting similar methodogy as adopted by Karl et al. (2008). Varying input frequencies ranging from 0.5 to 10 kHz are used for the present study. However, \( \xi \) evaluation is mainly limited to three frequencies 3, 4, and 5 kHz which would be enough for obtaining slope. Time histories divided into rectangular windows as presented in Appendix-I, Fig. A9 having window length of 0.5×10\(^{-4}\), 40×10\(^{-5}\), 25×10\(^{-4}\), and 20×10\(^{-4}\) s for Pt, CI-MI, SM-SC, and CH soil respectively. Peaks obtained are represented in the form of time histories which are \( d_f(t) \), \( d_2(t) \), and \( d_3(t) \). Time histories obtained were converted in frequency domain by Fourier transform. Accordingly, \( \alpha_t \) is calculated. Graphs are plotted between \( \alpha_t \) and input frequency \( f_i \), a linear relationship is obtained as shown in Fig. 10. \( \xi \) is calculated from the slope \( (S) \) of the graph as per the following equation suggested elsewhere (Karl et al. 2008).

\[
\xi = \frac{SV_s}{2\pi}
\]
(c) is applied in case of all soil types which is obtained from Equation 17 which ranged from 0.97 to 0.99 for all four varieties of soil. Finally, Table 8 presents calculated ξ for four varieties of soil after boundary effect correction. Values of S for different soil varieties are evaluated from the graph plotted between αs and frequency (Hz). The calculated ξ values are found to be having wide range for four different type of soil which varied from 8.90 to 22.10%. The Pt-classified type soil exhibited the highest ξ of 22.10% whereas CH type is found to exhibit the lowest value, i.e. 8.90%. However, an average ξ of 19.90, 9.30, 15.10, and 12.70% was obtained in case of Pt, CH, SM-SC, and CI-MI respectively.

However, the damping ratio values obtained from BE tests are at high frequency range, contrary to the low frequency range commonly encountered in civil engineering problems. Thus, in order to rationally represent the values derived from BE testing, Karl et al. (2008) performed tests at relatively higher frequency (in kHz range) compared to resonant column and torsional pendulum test which are performed at lower frequency range. Despite of this limitation of generation of frequency for all ranges, the bender element method is most widely used due to its simplified operation, less expansive, and wide availability. Hence, compromising a balance between rigour of experiment, non-availability of high-end expensive instrumental facilities and time, the bender element method was utilized to assess the material damping ratio of site-specific soil layers in present study. Furthermore, the multiple reflection technique of bender element was selected in present study in order to determine the material damping of soil considering the fact that average material damping ratio calculated following bender element resonant method (which was suggested as prepared method out of three different BE methods adopted by Karl et al. 2008) was found to exhibit a marginal difference (in order of maximum 6.3%) for silty samples with respect to counterpart results obtained from multiple reflection technique of BE experiment.

On the other hand, understanding the importance of frequency dependent material damping proposed by Karl et al. (2008) including other researchers (Stoll 1979, 1985, Vucetic and Dobry 1987, Shibuya et al. 1995, Rix and Meng 2005), the authors would like to state that a percentage reduction (may be in order of 60 to 70%) on damping ratio of soil obtained from BE experiments may be applied to rationally predict the damping ratio of the same material at a lower frequency range. The percentage reduction of 60 to 70% has been arrived after careful examination of average results of damping ratio obtained from lower frequency of resonant

| Identification of sample | Depth of sample below G.L (m) | SPT value | \( V_s \) obtained from correlation, proposed by Dikmen (2009) with SPT value (m/s) | \( V_s \) obtained from BE test (m/s) |
|--------------------------|-------------------------------|-----------|--------------------------------|---------------------------------|
| CH (U0401)               | 3.00                          | 03        | 75.00                          | 80.00                           |
| CH (U0501)               | 3.00                          | 04        | 85.59                          | 84.00                           |
| CH (U0601)               | 3.00                          | 04        | 85.59                          | 87.00                           |
| Pt (U0101)               | 3.00                          | 01        | 58.00                          | 64.40                           |
| Pt (U0201)               | 4.50                          | 02        | 76.00                          | 66.00                           |
| Pt (U0301)               | 4.50                          | 12        | 165.74                         | 168.00                          |
| SM-SC (DS07/4.5)         | 3.00                          | 12        | 161.05                         | 166.00                          |
| SM-SC (DS08/6.0)         | 4.50                          | 11        | 161.05                         | 166.00                          |
| SM (DS09/6.0)            | 6.00                          | 14        | 174.40                         | 174.00                          |
| CI-MI (U1001)            | 3.00                          | 12        | 152.86                         | 153.00                          |
| CI-MI (U1101)            | 4.50                          | 12        | 152.86                         | 150.00                          |
| CI-MI (U1201)            | 1.50                          | 13        | 157.71                         | 156.00                          |

Fig. 10. Sample graphs between attenuation coefficients with respect to frequency for four types of soil
column and torsional pendulum tests when compared with bender element resonant method of mitigation technique.

**Assessment of liquefaction potential of target site**

The liquefaction susceptibility assessment of target site is performed considering subsoil deposit encountered at BH07, BH08, and BH09 are based on two widely adopted approaches. To perform the analysis, sand samples were collected from moderate depth of 3.2 m below existing ground level and liquefaction susceptibility is evaluated in case of BH 07, BH 08, and BH 09. Similar natured alluvial sand had resulted in lateral spreads, sand boiling, and liquefaction during the recent 2017 Dhalai EQ ($M_w$ 5.7). Extensive cases of liquefaction induced failures are presented in Appendix-I, Fig. A10. To determine liquefaction susceptibility, the first approach is adopted from Cetin et al. (2004) which was probabilistic SPT-based correlations whereas the second approach is $V_s$-based evaluation of liquefaction potential as suggested by Youd et al. (2001). PGA of 0.36g and $M_w$ of 7.5 were considered in the present analysis. Input of earthquake parameters was assumed as per IS: 1893-(Part 1)-2016, considering the target site is located in severe seismic zone (zone-V) of the country. On the other hand, a preliminary analysis based on grain size distribution of sandy soil sample from the study area and 2017 Dhalai EQ.

| Identification of sample | Slope ($S$)×10^{-3} | Boundary constant ($c$) | Damping ratio ($\xi$) (%) | Average damping ratio ($\bar{\xi}$) (%) |
|--------------------------|----------------------|-------------------------|---------------------------|-----------------------------------------|
| Pt (U0101)               | 5.50                 | 0.99                    | 22.10                     | 19.92                                   |
| Pt (U0201)               | 5.80                 | 0.99                    | 19.57                     |                                         |
| Pt (U0301)               | 5.60                 | 0.99                    | 18.09                     |                                         |
| CH (U0401)               | 19.90                | 0.98                    | 8.90                      | 9.33                                    |
| CH (U0501)               | 21.20                | 0.98                    | 9.60                      |                                         |
| CH (U0601)               | 19.60                | 0.98                    | 9.50                      |                                         |
| SM-SC (DS07/4.5)         | 4.25                 | 0.99                    | 14.90                     | 15.10                                   |
| SM-SC (DS08/6.0)         | 4.10                 | 0.99                    | 14.40                     |                                         |
| SM (DS09/6.0)            | 4.38                 | 0.99                    | 16.10                     |                                         |
| CI-MI (U1001)            | 4.70                 | 0.99                    | 12.10                     | 12.70                                   |
| CI-MI (U1101)            | 4.88                 | 0.99                    | 12.80                     |                                         |
| CI-MI (U1201)            | 4.96                 | 0.99                    | 13.20                     |                                         |

![Fig. 11. Physical properties of Agartala sand (Indo-Bangladesh region) lying under highly liquefiable prone zone.](image-url)
liquefaction case study area is performed and presented in Fig. 11. Further to be noted that Fig. 11 also presents grain size distribution of soil samples from various parts of India as reported by different researchers. It is found that both soil sites of Tripura pose similar grain size distribution and comply highly susceptible for liquefaction and also have a closer agreement with soils from other parts of India. First, the depth-wise factor of safety \((FS)\) with respect to liquefaction considering cyclic resistance ratio (CRR) and cyclic stress ratio (CSR) for BH07 to BH09 are calculated following probabilistic SPT- and \(Vs\)-based approach. Details of calculations for both the approaches are not presented herein for the sake of brevity and can be referred from Cetin et al. (2004) and Youd et al. (2001) respectively. Depth-wise \(Vs\) are calculated using validated empirical equations based on SPT values. This empirical relationship was validated with the BE test results in the present study as presented in the previous section. Figure 12 presents the depth-wise factor of safety \((FS)\) with respect to liquefaction for BH07 considering both the approaches. Results indicate higher liquefaction susceptibility \((FS < 1.0)\) at moderate depth due to presence of silty fine sand layer. Both \(Vs\) and probabilistic SPT-based approach exhibit well agreement in results. In addition, the liquefaction severity of the study area is also assessed by calculating the liquefaction potential index \((LPI)\) based on depth-wise \(FS\) with respect to liquefaction as suggested by Sonmez (2003). This may provide better insight into the liquefaction potential of the study area. Table 9 presents the correlation between \(FS\) with respect to liquefaction and \(LPI\) value. Finally, the \(LPI\) for BH07, BH08, and BH09 are evaluated and presented in Table 10. It is observed that the site is highly prone to liquefaction as per suggestive ranges of \(LPI\).

### Summary and conclusions

Present study primarily focuses on dynamic properties characterization of subsoil deposit of proposed India Bangladesh railway construction site at Agartala by performing BE tests on four different category of soil samples in the laboratory along with proposal of site-specific empirical correlations. Furthermore, determination of physical and engineering characteristics of subsoil deposits upto 20.0 m depth were conducted in the first phase of the study. In second

#### Table 9. Liquefaction criteria based on factor of safety \((FS)\) and liquefaction potential index \((LPI)\), as per Sonmez et al. (2003)

| Liquefaction criteria | Factor of safety \((FS)\) | Liquefaction potential index \((LPI)\) |
|-----------------------|---------------------------|-------------------------------------|
| Non-liquefiable       | \(FS \geq 1.2\)          | \(LPI = 0\)                         |
| Very low              | \(1.2 > FS \geq 1.0\)    | \(2 \geq LPI > 0\)                 |
| Low                   | \(1.0 > FS \geq 0.95\)   | \(5 \geq LPI > 2\)                 |
| High                  | \(0.95 > FS \geq 0.85\)  | \(15 \geq LPI > 5\)                |
| Very high             | \(0.85 > FS\)            | \(LPI > 15\)                        |
phase, the dynamic properties such as \( V_s \), \( G_{\text{max}} \), and \( \xi \) are obtained from BE tests, and finally, an attempt was made towards liquefaction assessment of the target site. Based on this, the study offers following salient conclusions.

- It is found that major portion of the site consists of marshy land with very soft peat and clay where the percentage of decomposed organic matters varies along depth and horizontally. The maximum percentage of decomposed matter found in order of 36%. Furthermore, silt mixed with fine sand layer is also found in shallow to moderate depth of boreholes which are highly prone to liquefaction and special attention should be given towards liquefaction while carrying out any construction in this region.

- From BE tests, it is found that the near field effects gets subdued at \( L_T / u_1 \) of 2.80, 2.00, 4.40, and 1.45 for soil types of CH, CI-MI, Pt, and SM-SC respectively. The range of average \( V_s \) is calculated to be 74.32–108.65, 145.45–168.44, 60.61–76.21, and 148.15–234.64 m/sec for CI-CH, CI-MI, Pt, and SM-SC types of soil respectively. It is observed that \( V_s \) tends to increase with increase in confining pressure. An empirical prediction model of \( V_s \) from present study is presented for different soil types which may be adopted for similar natured soil available in other regions for site-specific studies.

- Set of empirical correlation equations of \( G_{\text{max}} \) are proposed as a function of void ratio \( (e) \) and confining pressure for four different soil types. Furthermore, another set of empirical relationship of \( G_{\text{max}} \) as a function of \( \mu \) which was obtained from CUTX tests for different soil types are also presented.

- Damping ratio \( (\xi) \) for four different soil types are calculated based on BE test using peak to peak method. The mean range of \( \xi \) are observed in order of 19.90, 9.30, 15.10, and 12.70% in case of Pt, CH, SM, and CI-MI respectively. The values obtained would be substantial in carrying out site-specific studies for similar natured soil.

- Liquefaction assessment of the target site was performed based on preliminary assessment by grain size distribution and empirical relationships proposed in past researches which confirms the presence of liquefaction susceptibility material. Furthermore, the \( LPI \) were calculated to be ranges between 18.32–23.74 (probabilistic approach) and 19.62–26.80 (\( V_s \) approach) indicating study area is highly prone to liquefaction.

Summarily, the outcome of this study would help to estimate dynamic properties of typical subsoil of similar nature soil available in Tripura. Suggested correlations will be useful in seismic hazard/earthquake geotechnical engineering related studies of Tripura and India. However, present study also highlights dynamic characteristics of typical soft peat clayey type soil which is abundantly available in northeastern region of India and various other parts of India. Liquefaction potential of study area also assessed which may guide engineers to design the future project in similar nature deposit and retrofit the important existing structures.

**Supplementary Information** The online version contains supplementary material available at https://doi.org/10.1007/s12517-022-10749-4.

### Table 10. Determination of \( LPI \) considering PGA of 0.40g and \( M_w 7.5 \) for target site by probabilistic and shear wave velocity approach

| Bore hole | Depth (m) | Average SPT | Unit weight (KN/m³) | Soil type | \( r_p \) | FC (%) | Factor of safety (FS) | \( LPI \) of bore hole | Limiting criteria (Sonmez et al. 2003) |
|-----------|-----------|-------------|---------------------|-----------|---------|--------|------------------------|-------------------------|-------------------------------------|
| BH07      | 0–3.2     | 10          | 18.80               | Clay-like | 1.11    | 0.99   | 70                     | 2.60 2.62               | 23.74 26.80 >15.00 (highly liquefiable) |
|           | 3.2–11.2  | 11          | 18.14               | Sand-like | 0.97    | 0.95   | 30                     | 0.33 0.25                |                                     |
|           | 11.2–14.6 | 57          | 20.12               | Clay-like | 0.83    | 0.82   | 75                     | 3.30 2.59                |                                     |
|           | 14.6–24.0 | 96          | 20.80               | Sand-like | 0.69    | 0.71   | 05                     | 2.20 2.52                |                                     |
| BH08      | 0–2.7     | 04          | 14.00               | Clay-like | 1.11    | 0.99   | 78                     | 1.80 1.88               | 21.13 22.56                |
|           | 2.7–5.7   | 12          | 17.95               | Sand-like | 1.03    | 0.98   | 26                     | 0.41 0.38                |                                     |
|           | 5.7–8.2   | 35          | 18.80               | Sand-like | 0.95    | 0.96   | 22                     | 0.92 0.96                |                                     |
|           | 8.2–25.0  | 87          | 21.00               | Sand-like | 0.79    | 0.81   | 08                     | 1.88 1.95                |                                     |
| BH09      | 0–4.2     | 04          | 14.50               | Clay-like | 1.11    | 0.99   | 80                     | 1.69 1.74               | 18.32 19.62                |
|           | 4.2–8.8   | 12          | 19.02               | Sand-like | 1.01    | 0.95   | 16                     | 0.65 0.54                |                                     |
|           | 8.8–24.0  | 35          | 21.00               | Sand-like | 0.77    | 0.70   | 06                     | 2.21 2.16                |                                     |

*Note: SPT, standard penetration test; \( r_p \), shear stress reduction coefficient factor; FC, fine content; FS, factor of safety; PA, probabilistic approach; SA, shear wave velocity approach; LPI, liquefaction potential index*
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Code availability All codes used during the study appear in the article.

Author contribution Rajat Debnath—visualizing and conceptualizing the problem, conducting experiments, analyzing data, writing and formatting the manuscript. Rajib Saha—visualizing and conceptualizing the problem, review and editing of manuscript, and supervision of work. Sumanta Haldar—visualizing and conceptualizing the problem, review and editing of manuscript, and supervision of work.

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Declarations

Competing interests The authors declare no competing interests.

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