Indian Geotech J (October 2022) 52(5):1079–1096
https://doi.org/10.1007/s40098-022-00609-z

ORIGINAL PAPER

Effective Use of SPT: Hammer Energy Measurement and Integrated Subsurface Investigation

Panjamani Anbazhagan[1] · Kumar Ayush[1] · M. E. Yadhunandan[1] · Kumar Siriwanth[1] · Kailasam Suryanarayana[2] · Gade Sahodar[2]

Received: 21 November 2021 / Accepted: 24 February 2022 / Published online: 28 March 2022

Abstract Standard Penetration Test is a widely used geotechnical exploration test worldwide. SPT is easy to perform and is cost-effective; hence, it has become prevalent. Several factors affect the test results, i.e., the number of blows (N-value) for penetration of the sampler into the soil at any given depth. Among those factors, hammer energy (E_H) is the most important. Even though it's important, there have been limited attempts to correlate E_H with different subsurface properties, which are later estimated using N-values. Several empirical relations have been developed between N and subsurface properties, both static and dynamic, predominantly in developed countries, which can be used only to a region and a particular E_H value. However, the influence of considering proper in-situ hammer energy in these correlations is not clearly understood yet and thus, it is still not practised in many developing countries. This study highlights the importance of hammer energy in N-value corrections and studies the effect of hammer energy on soil properties like low strain shear modulus and SBC values and integrates with the subsurface imaging methods to determine spatial variation of these parameters. The influence of different SPT corrections is studied along with the effect of including energy measurements in analyzing the correlation between the SPT N and soil properties such as, SPT and low strain shear modulus. To address the highly localized interpretation of SPT restricted to a borehole, and to understand the spatial distribution of these design parameters across the study area, a 2D subsurface profile has been generated using geophysical tests, which was later integrated with the borehole data.

Keywords Subsurface · Investigation · SPT N-values · Shear modulus · SBC · Spatial variability

Introduction

Standard Penetration Test (SPT) is the most widely used field test in soil investigation work. This test classifies subsoil layers in terms of penetration resistance (N-value), i.e., the number of blows needed to cause the sampler to penetrate the last 30 cm of total 45 cm penetration, using the driving weight of 63.5 kg falling over an anvil from a drop height of 75 cm. It is of great use in the cases where obtaining an undisturbed sample is difficult, like gravelly, sandy, silty, sandy clay soils, or weak rock formations. It is often used to approximate the in-situ density and angle of shearing resistance of cohesion-less soils and the strength of cohesive soils. The samples obtained help in the identification of different subsurface layers; N-values can be used for geotechnical design purposes, as several other dynamics and static properties of subsoil layers are well correlated with SPT results [1–3]. Recently, SPT has gained significance in the empirical determination of a sand layer's susceptibility to liquefaction. Because of the importance of SPT as a primary in-situ test in geotechnical engineering, it is important to study the factors which affect its results. Energy Transferred by hammer blows to the sampler is observed to be the most important factor affecting N-values [1, 3] and is considered in designs as a hammer energy correction factor. Changes in hammer energy are observed to significantly affect the properties obtained from correlations. An increase in hammer energy will lead to a linear reduction in N-values and vice versa. Thus, if the hammer energy for

✉ Panjamani Anbazhagan
anbazhagan@iisc.ac.in

1 Indian Institute of Science, Bangalore, India

2 Central Power Research Institute, Bangalore, India

N-value is unknown, correction factor based on standard recommendations or previous measurement will not be appropriate, and may lead to underestimation or overestimation of subsurface properties, e.g., shear modulus, density, bearing capacity, etc. [3–5].

In India, SPT N-values have been used in the estimation of almost all design parameters for geo-structural design through exiting correlations irrespective of their applicability to the region. These correlations are developed using data acquired at specific hammer energy and in a specific region. Thus, caution should be exercised in their application in other regions. As discussed above, hammer energy is the most important factor affecting SPT; it is imperative to study its influence while deriving different subsurface properties using N-value correlations. Moreover, it is also observed that variations in hammer energy also influence the termination depth of the Borehole, which is generally called a rebound layer and often considered as the resting layer for foundations. Thus, ambiguity in hammer energy may also lead to the selection of a weaker soil layer for foundation construction. Hence, in this study, the influence of hammer energy has been studied on two important soil properties: low strain shear modulus ($G_{\max}$) and Soil Bearing Capacity (SBC) and their variation for different hammer energy and spatial distribution are presented within the site. Boreholes were drilled at several locations in a site; the SPT N value with hammer energy was measured, and soil/rock samples were collected. Apart from these tests, in-situ shear wave velocities are measured using Downhole and Crosshole seismic borehole tests and seismic surface wave tests of Multichannel Analysis of Surface Wave (MASW) method. The subsurface imaging method of Ground Penetrating Radar and 2D MASW were also carried out to map the spatial distribution of these design parameters. These data and results are used to explain the effects of hammer energy on rebound/termination of borehole depth associated with N-values, shear modulus & Young's modulus and safe bearing capacity. Soil samples are used to determine the index and engineering properties of soil and rock layers from the Borehole and integrated geophysical results to assess the spatial distribution of design parameters in the site. Detailed testing data, analysis and results are presented in subsequent sections, and suggestions have been presented to overcome some recent crack and settlement issues in the residual terrain of Bangalore due to unusual rain.

Corrections Applied to Recorded SPT N-Values

SPT N-values recorded in the field are affected by equipment-related or operational variables such as the operation of hammer dropping, length and verticality of guide rods, hammer–anvil dimensions and weights, sampler type, and hammer blow rate. Apart from these, unavoidable site factors are groundwater table, fine contents in soil and depth of soil layer being tested. These variables are accounted in the analyses using several correction factors. Some of these variables as summarized by Aggour and Radding [6] are presented in Table 1.

Common corrections are overburden pressure correction (C_N), hammer energy correction (E_{h}), borehole diameter correction (C_2), sampler liner correction (C_3), rod length correction (C_4) and dilatancy correction (N“). Another correction for percent fines contents is applied when liquefaction analysis is being carried out. The overburden pressure affects the penetration resistance in cohesionless soils. The soil of the same density will give a larger N-value at a higher depth due to overburden pressure. Hammer energy accounts for the efficiency of the blows given by the hammer. The energy delivered to the sampler is not the same as the theoretically calculated energy. Thus, recorded N-values need to be corrected for energy to be used in correlations and design problems. Dilatancy correction accounts for the presence of a water table in silty sand. Under undrained conditions, dense fine or silty sand tends to dilate. Hence, the N-values may be recorded abnormally higher below the water table in such cases. For dry boreholes, the corrected standard blow count $(N_1)_{60}$ is widely calculated using the relation [4]

$$(N_1)_{60} = E_h C_2 C_3 C_4 C_N N \tag{1}$$

Bowles [4] suggested correction as unity for the case of a small borehole, no sample liner and drill rod longer than 10 m. Thus, the measured N-value needs to be corrected only for hammer energy and the overburden pressure. There is no clear idea regarding how much each correction changes the final value and which one is highly influential. This analysis was extended by Anbazhagan et al. [3, 5] and concluded that the hammer energy and overburden pressure are the most important SPT corrections for the conditions as mentioned by Bowles [4]. Even in the twenty-first century, Indian code IS 2131 [7] uses outdated correction factors for overburden pressure and fines contents. Seismic design code IS 1893 [8] recommends assumed hammer energy correction factor for liquefaction without accounting important correction factor for Hammer Energy. Authors found that these correction factors are not developed for major soil types found in India but are still widely used in practice for all geo-structural designs.

Importance of Hammer Energy Correction

Several early studies showed the dependence of blow count on hammer energy. Schmertmann and Palacios [9] showed experimentally that the measured blow count was inversely

Table 1 Factors affecting SPT N-values (after [6])

Factor/Procedure	Consequence	Effect on N-value
Inadequate cleaning of the borehole	Sludge may be trapped in the sampler	Increase
Not seating the sampler spoon on undisturbed material	Test conducted on disturbed soil layer	Incorrect N-values
Driving of the sampler spoon above the bottom of the casing		Increase in sands and Decrease in cohesive soils
Insufficient hydrostatic head in boring	Sand at the bottom of the Borehole may become loose	Decrease
Attitude of operators (mood, time of day, fatigue)	Blow counts for the same soil using the same rig can vary	Increase or Decrease
Overdriven sampler	Sampler may undergo wear and tear	Increase
Sampler plugged by gravel	Overestimation of resistance of loose sand	Increase
Sampling below the groundwater table	Hydrostatic pressure causes the sand to rise and plug casing	Increase
Over washing ahead of casing	Sand is loosened by over washing	Decrease
Drilling method (cased holes vs. mud stabilized holes)		Different N-values
Free fall of the drive weight is not attained	Fall of the drive weight is restricted	Increase
Not using correct weight	Test is no more standard	Increase or Decrease
Weight does not strike the drive cap concentrically	Impact energy is reduced	Increase
Not using a guide rod	Fall of hammer is not controlled	Incorrect N-values
Not using a good tip on the sampling spoon	Damaged tip reduces opening of sampler	Increase
Use of drill rods heavier than standard	More energy is absorbed by the rods	Increase
Not recording blow counts and penetration accurately	Test becomes unreliable	Incorrect N-values
Using drill holes that are too large	Higher overburden reduction	Decrease
Using too large a pump	Soil at the base is loosened	Decrease

proportional to the energy delivered to the drill rods for blow counts less than 50. Since different SPT equipment (based on hammer and rods type and configuration) were observed to produce variable energy inputs in the test, Seed et al. [10] suggested that measured blow counts (N-value) be corrected to reference energy of respective application, e.g., 60% for liquefaction. This analysis was based on the liquefaction data in the regions with liquefaction history, which resulted from a famous safety hammer with rope and cathead release. Hence, hammer energy standardisation at the 60% level as per regional practices was proposed.

The energy correction factors for equipment in different countries were first reported by Skempton [11] and later updated by Anbazhagan et al. [1, 3]. It can be observed that the method of release has a large effect on the efficiency of the hammer. Thus, the energy correction factor is highly dependent on local practice and should not be generalised. Further, selection of energy correction based on equipment type only is also not appropriate, as hammer energy variation is site-specific [12] as well. Anbazhagan et al. [2] showed that energy values vary significantly within the same depth and soil. Thus, a generalised approach to account for energy variations is debatable because it does not consider differences in the specifications of the SPT equipment, which are known to affect the transferred energy significantly.

For the same soil conditions, an SPT hammer with lower energy efficiency would result in a higher SPT N number than a higher energy efficiency SPT hammer. Thus, the N value should be standardised to a site-specific energy level using correction factors to reduce the variability of the SPT N-values due to the considerable variation in the energy delivered. The standardization will enable uniformity among the in-situ soil properties correlated to the measured N-values. It has been observed that because of changes in region and equipment, the correlations are often defined for different energy levels. If the energy measurement is not carried out during SPT, variation in hammer energy will cause incorrect design parameters estimated from those correlations.

It is now established that hammer energy delivered to penetrate split spoon sampler is different from energy measured below Anvil, i.e., above ground very close to hammer. But current practice is to measure hammer energy below Anvil is prevails, as most correlations were developed based on these types of data. ASTM D4633 [13] suggests performing energy measurement below Anvil, at least in the three depths of SPT N and in the five depths preferred for quality data, while using the SPT system in a nearly routine manner as practical. ASTM D4633 [13] suggests preferably to make as many measurements as possible and averaging the energy results. BS EN ISO 22476-3 2005 + A1 2011 [14] suggests that at least five measurements of the hammer energy should be taken to estimate the average energy value. The reason behind these recommendations is not clearly specified in these codes. Generally, these countries use standard SPT equipment and the same hammer dropping mechanism throughout the country, and well trained and certified operators conduct the test. Such a scenario is not practiced in the rest of the world, especially in developing countries. Considering this, one must verify the minimum energy measurements required for a borehole based on variations in SPT. It may not be sufficient to adopt ASTM and BS codal recommendations completely for the number of readings. Even though ASTM recommends, it may not be suitable for the equipment used in developing countries. It may be advised to continue the energy measurement for all the blows as it has significant variation, which was also observed in this study. Besides, it provides both digital record of N value and energy measurement. Energy measurement is not a time-consuming process and is required for all SPT measurements.

As evident from the above discussions, hammer energy has not been given much attention previously. N-values are being related to several soil properties, which often leads to errors due to ambiguity in the energy value. The following sections discuss the influence of hammer energy on borehole termination criteria and estimated soil properties like Low strain shear modulus and Soil bearing capacity.

Effect of Hammer Energy on Prediction of Denseness and Rebound

State of soil, i.e., loose, or very dense/weathered rock is interpreted based on SPT N-values in most of the soil investigations. This state also affects borehole termination depth. Generally, boreholes are terminated when a very dense/weathered rock layer is reached which is defined by SPT N-values more than 50 or 100, depending upon the code provisions or tender documents. It is important to understand how hammer energy influences the assessment of the state of soil. Table 2 presents the most widely used soil states for sands as a function of SPT N-values given by Terzaghi and Peck [15]. They also presented a similar table for clays, relating relative density or state of soil and unconfined compressive strength with N-value. It is important to note that most of the SPT equipment in USA were observed to deliver 60% energy, hence N_{60} was taken as reference for all these classifications. If these recommendations are followed without knowing applied energy to measure SPT N-values, the interpretation of denseness of soil changes completely. For example, if 30% energy is applied to measure SPT N values, we will interpret loose to medium soil as dense soil (see Table 1).

It has come to authors' observation that often either the measured N-values are directly used as N_{60} in Table 2, or a standard energy value is assumed based on previous experience or code recommendations [7]. If the energy transfer ratio (ETR = ratio of actual delivered hammer energy to the theoretical maximum potential energy) values are much lower, e.g., 20–30%, then there is a danger of overestimating the denseness of the soil state. As discussed previously, considering an ETR of 30%, if a rebound is considered at N-value of 50, the actual corrected N value is 25, which corresponds to Medium relative density as per Table 2. Thus, a lower ETR value will give a higher N-value and lead to early termination of the Borehole much before the desired termination depth. This misjudgment of soil state will have severe consequences if the selected rebound layer cannot bear the load of the structures built later.

It has been observed that in India, some geotechnical practitioners use 50 or 100 as uncorrected N-values in rebound soil layers and over 50 or 100 along with penetration values in weathered rock layers (as specified in the tender documents). The Indian code IS 2131 [7] recommends a maximum cumulative blow count of 50 for the last two increments to terminate the borehole. This recommendation is different from the most widely used borehole termination criteria given in ASTM D4633-16 [13], which recommends rebound as a layer where the blow count exceeds 50 for any one of the increments or 100 in total. A common practice in India is to mention it as “Rebound” instead of providing complete blow count and penetration depth, which leads to assuming rebound N-values as 50 or 100 and will lead to error. Photographs of samples in Fig. 1 show samples collected by boring using single barrel drilling in rebound layers at 6.5–8 m and 8–9.5 m run. It is visible that the samples cannot be considered hard or dense. Thus, the accuracy of visual inspection of soil samples obtained during drilling also depends upon the method of drilling used. If a multi-barrel core is used, better quality samples could be obtained.

Hence giving termination criteria of borehole based on N-values without mentioning energy applied to respective N-values, as in IS 2131 [7], may lead to unreliable soil investigation. So, restricting the overall blows to 50, as mentioned in IS 2131 [7] is inappropriate.

Table 2 State of sandy soils according to N_{60} values (Terzaghi and Peck [15])

Relative density/in-situ state	N_{60}	Measured N-values for energy transfer ratio (ETR, %)			
		30	40	50	70
Very loose	0–4	0–8	0–6	0–4.8	0–3.43
Loose	4–10	8–20	6–15	4.8–12	3.43–8.57
Medium	10–30	20–60	15–45	12–36	8.57–25.71
Dense	30–50	60–100	45–75	36–60	25.71–42.86
Very Dense	> 50	> 100	> 75	> 60	> 42.86

Fig. 1 Photographs of samples obtained at rebound layers in this study

Correlations with Soil Properties

N-value is the most used parameter to estimate soil properties required for foundation design, site response, and liquefaction hazard estimation. The N-values are correlated with different soil parameters, i.e., unit weight (γ), angle of internal friction (φ), relative density (D_r), and undrained compressive strength (Su). Many empirical relationships have been developed between N-value and soil properties measured in lab or field. Some common soil parameters which are determined using correlations with N-values are friction angle (φ), cohesion (c), elastic modulus (E), Shear wave velocity (V_S), Low strain shear modulus (G_{max}), Poisson's ratio (μ), Density (γ), Relative density (D_r), Bearing capacity & Settlement.

It is well known that hammer energy plays a significant role in the estimation of any soil properties estimated using N-value correlations [3]. Without knowing hammer energy and soil type in correlation development, it is not advisable to use any such correlation. Because use non-region-specific N-based correlation in absence energy ratio may lead to wrong soil properties estimation as explained below.

Low Strain Shear Modulus for Different Hammer Energy

It may not always be feasible to determine shear wave velocity (V_S) and low strain shear modulus (G_{max}) in-situ using seismic methods because of space constraints and high noise levels (often in urban areas), which can contaminate the recordings. Therefore, it was found necessary to determine these dynamic properties through indirect methods such as correlating with SPT N-values. Hence, several exercises have been carried out to evaluate soil's low strain dynamic properties (V_S and G_{max}) for different soils and testing conditions by identifying empirical relationships between these properties.

Schmertmann [16] stated that shear modulus depends on the dynamic stress–strain properties of the soil and the level of strain in the travelling shear waves. Penetration of sampler during SPT involves dynamic soil shear behavior at the failure reference level of shear strain and modulus. Hence a correlation between N-values and maximum shear modulus at very low strain can be expected [4]. Over time, the practice of SPT has been improved through standardization and hammer energy measurement, which is used to determine ETR for correction of N-values to a standard ETR of 60% in the USA. Anbazhagan et al. [1, 3] reviewed all SPT N versus G_{max} correlations, highlighted popularly used correlations' limitations, and gave updated correlations. Further, hammer energy measurement was emphasized to use correlations wisely. It was found essential to select a correlation that should consider local site SPT testing practice and prevalent energy ratio imparted by the SPT hammer.

After the literature review, the authors observed that most of the current correlations were obtained based on studies in Japan, with hammer energy different from the other regions, thus limiting their applications. The correlation was developed by eliminating the assumed and extrapolated data and can be directly used in all regions where hammer energy is the same. For regions with different hammer energy, modification factors for the equation were presented. Anbazhagan et al. [3] reevaluated the modification factors for the correlation to be used for different ETR values and validated them using field acquired Crosshole and downhole data. The updated equation and modification factors to be used for different ETR values is discussed below

$$G_{max} = 16.40 N_{78}^{0.65} (\text{MPa}) \quad (2)$$

N_{78} is the SPT blow count corrected for 78% energy ratio. It is to be noted that the data used for the development of correlation was adopted from Japan with ETR 78%. Hence, N_{78} is used in the equation. Figure 2 shows the G_{max} estimated for different ETR values as per Eq. 2.

To adjust the equation for changes in ETR values, the modification can be done as follows:

$$G_{max} = 16.40 N_{78}^{0.65} = 16.40 \times \left(CF_{(78 \text{ to } ERm)} \times N_{ERm}\right)^{0.65}$$

Correction factor for N-value

$$CF_{(78 \text{ to } ERm)} = (ER_m/78) \quad (3)$$

The general form of the equation for measured energy ratio ETR_m

$$G_{max} = a_m N_{ERm}^{0.65}$$

Modified coefficient a_m for ETR_m

$$a_m = 16.40 \times (ER_m/78)^{0.65} \quad (4)$$

For example, if ETR_m is 60%, then the equation will be $G_{max} = 13.83 N_{60}^{0.65}$. It is to be noted that N-values have a linear relation with ETR, while G_{max} does not. Hence, care should be exercised in using the corrections for energy for the evaluation of G_{max}. So one as to be careful about using SPT N without knowing energy applied to estimating shear modulus and Young's modulus, which is an important input for any geotechnical design calculation and analysis.

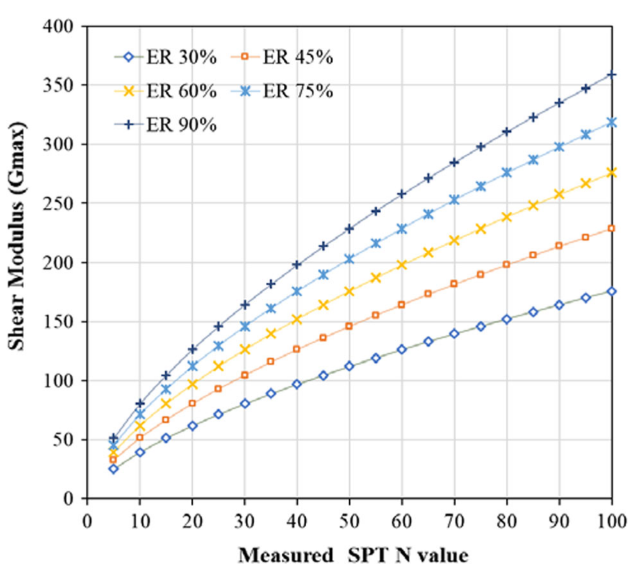


Fig. 2 G_{max} values estimated for measured N at different ETR values (after Anbazhagan et al. [3])

Hammer Energy and SBC

As discussed previously, correlations between SPT N and any property of soil depend on various factors. The important factors which govern correlation for bearing capacity are the type of foundation (isolated, continuous, etc.), dimensions of foundation (depth, width, etc.), type of soil (cohesive, non-cohesive, etc.), and allowable settlement.

Based on the early studies by Burland et al. [17], Terzaghi et al. [18] devised methods for estimating settlements and bearing pressures of footings on sand from corrected N-values. Parry [19] presented the Allowable bearing capacity of cohesionless soils for 55% ETR. One of the most widely used relationships was given by Terzhaghi and Peck [15], which is also over-conservative [4]. Meyerhof [20, 21] produced similar equations and curves for allowable bearing capacity for a 25 mm settlement, which again was conservative [4]. These equations have been used extensively for decades and are based primarily on N-values from the early 1960s. Thus, ETR is likely to be in the range of 50–55% and not the more popular 60%. Bowles [4] adjusted Meyerhof's equations for an approximate 50% increase in Allowable bearing capacity. Thus, it is necessary to understand how the change in ETR affects bearing capacity.

The most widely used Net ultimate bearing capacity from the shear criteria formula in General shear failure is shown in Eq. 5 [22]. Net SBC from settlement criteria is given in IS 8009 Part 1 [23]. Building load (dead load and live load) is assumed as per the IS 875–1987 (Part 1 to part 4) [24–27]. Immediate settlements were calculated to be within the safety limit for the assumed foundation contact pressure of 1000 kPa.

$$q_d = CN_C s_C d_c i_c + q\left(N_q - 1\right) s_q d_q i_q + (1/2) B_\gamma N_\gamma s_\gamma d_\gamma i_\gamma W' \quad (5)$$

where q_d = Net ultimate bearing capacity (kN/m^2); N_c, N_q, N_γ = bearing capacity factors; B = Width of the footing (m); C = Cohesion of soil (kN/m^2); γ = Unit weight of soil (kN/m^3); q = Effective surcharge pressure at the base level of foundation (kN/m^2); s_c, s_q, s_γ = Shape factors; d_c, d_q, d_γ = depth factors; i_c, i_q, i_γ = Inclination factors; W' = Correction for water table.

As per the Indian code of practice, N-values are corrected by applying overburden correction and excluding dilatancy correction as the water table was below 13 m. It has previously been proposed by Anbazhagan and Ingle [28] that the overburden correction in IS 2131 [7] is outdated and needs to be updated based on detailed study or universally adopted corrections. In this study, N-values were corrected by applying overburden and SPT hammer energy correction. These N-values are further used to calculate friction angle (φ) and subsequently the bearing capacity factors as per IS 6403 [22]. φ is used to estimate the Bearing capacity factors from linear interpolation. The same shape, depth, inclination factor, and water table corrections are applied to calculate the Net Ultimate and Net SBC. A factor of safety of 3 is assumed against bearing capacity failure [29]. Figure 3 shows the variation of Net SBC for shear and settlement criteria for different N-values with and without energy ratio correction. All other factors were considered the same and only the corrected N-value was varied, which is reflected in changes in φ. A significant difference can be seen in Net SBC in both criteria due to applying energy corrections to the measured N-values, which is not considered in the current SBC calculation of codal practice in most Asian countries.

The importance of the influence of measured energy values over the estimation of SBC values is now established. It can be discussed with the help of measured ETR values in this study.

In Situ Tests

Field investigation with borehole tests and subsurface scanning tests was carried out at a residual soil deposit at Central Power Research Institute (CPRI) in Bangalore to understand how the hammer energy varies with different SPT setups (equipment) and its effects on static and dynamic properties estimation. Six boreholes were drilled with SPT N value and hammer energy measurements. Disturbed and undisturbed soil samples and core rock samples were collected for classification tests. Figure 4 shows the location of the site and boreholes layout. Six boreholes were drilled with N-value measurement using two types of drilling equipment (hydraulic and rotary drill rig). Hammer energy at each blow was measured using SPT HEMA by connecting sensors below the Anvil and above the Split spoon sampler. SPT HEMA results are cross validated on this site with SPT Analyzer energy measuring instrument [5]. Since Borehole is point-based information that may not be sufficient to know subsurface layers spatial variation, MASW and GPR surveys were carried out to map subsurface layers. The location of MASW and GPR survey lines are shown in Fig. 4.

Further, the downhole and cross-hole seismic survey were carried out to determine average V_S values, shear modulus and young modulus at the site and compare with N-values values. Borehole drilled for SPT was cased as per ASTM D4428/D4428M—14 [30] in the site, and compressional (P-) and Shear (S-) wave velocities are measured, which are further used to determine Poisson's ratio

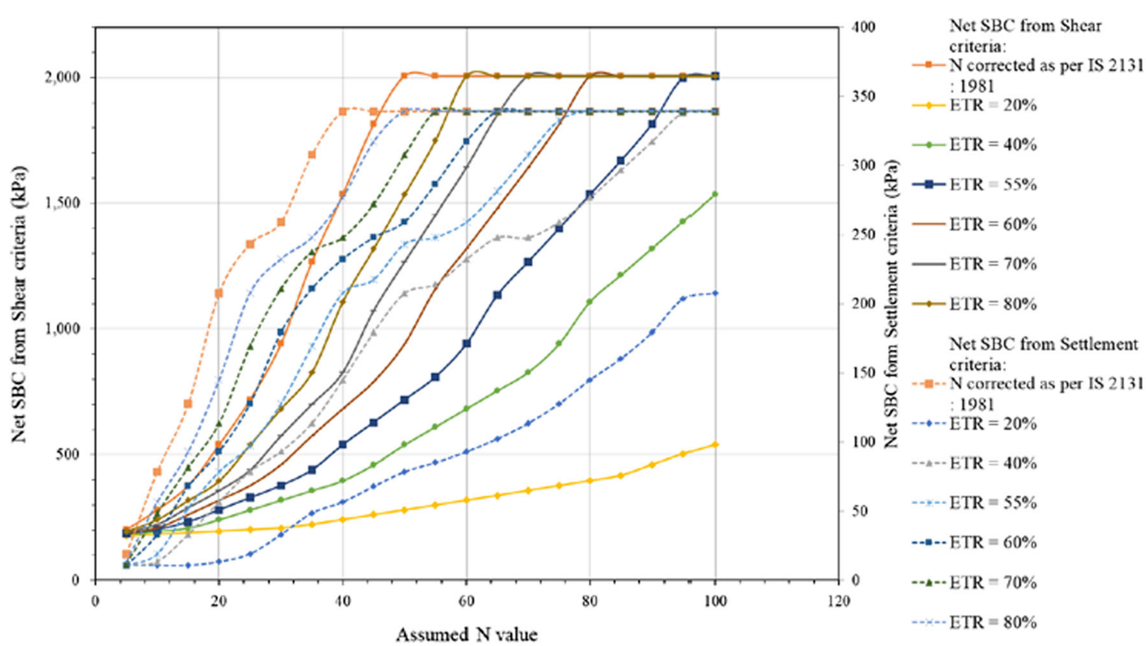


Fig. 3 Typical Net ultimate bearing for different N value corrected for different ETR (%) by assuming all other aspects constant capacity from (**a**) Shear criteria, (**b**) Settlement criteria (25 mm settlement) (after Anbazhagan et al. [5])

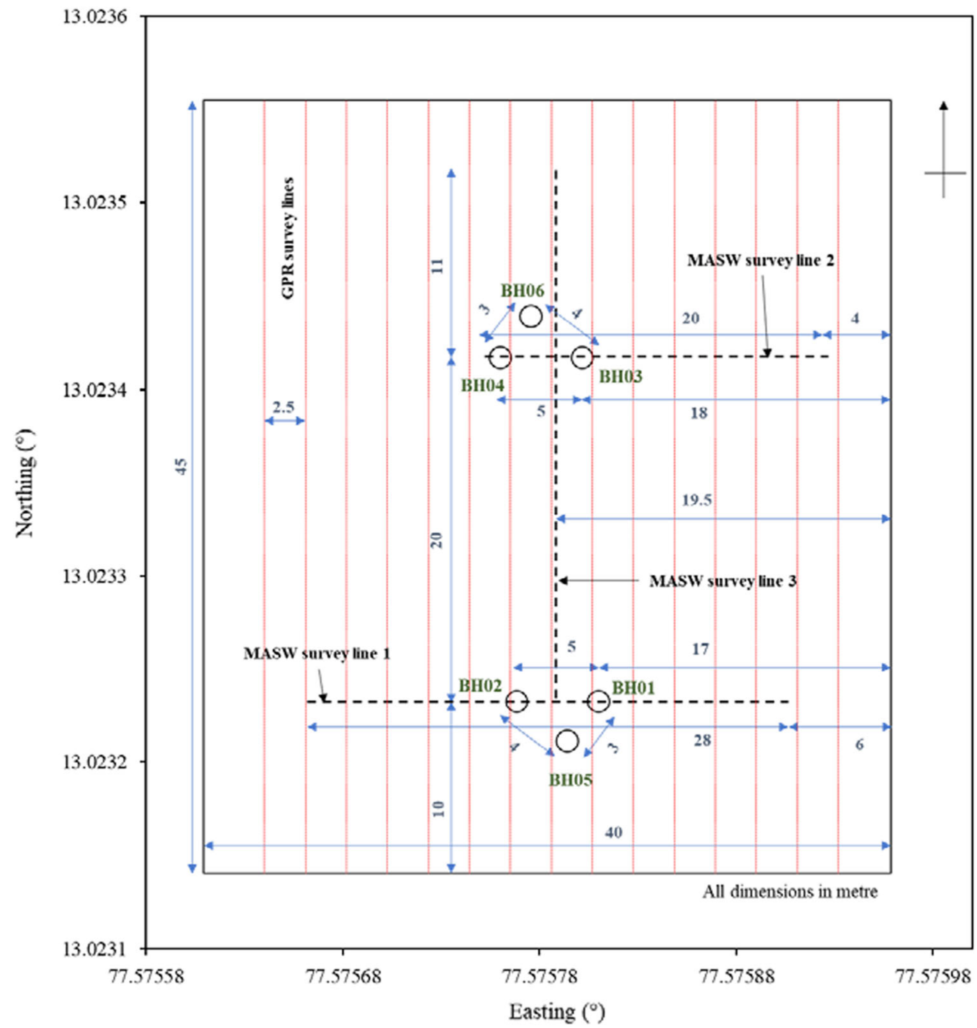


Fig. 4 Borehole locations in the study site

and modulus of subsurface layers. Details of tests and typical results are presented in respective figures in later sections. N-values and 1D subsurface profiles determined from borelogs are presented in Fig. 5.

SPT N-Values and Energy

In all the six boreholes, N-values were measured at every 1 m interval, and soil samples were collected as per IS 2131 [7]. All boreholes were drilled with a diameter of 150 mm and prepared as per IS 1892 [31]. Beyond the hard strata/rebound layer (SPT $N > 50$ or 100), coring was conducted with an NX core barrel. The percentage Total Core Recovery (TCR) and Rock Quality Designation (RQD) were recorded for the extracted rock cores. All boreholes are advanced until specified termination depth or meeting the core recovery/RQD requirements (85%). The rebound layers of all six boreholes were observed to be dense layers of silty sandy gravel or weathered rock layers. The measured N-values in different boreholes along with subsoil layers are given in Fig. 5. All N-values are measured with hammer energy, and more details of typical reading and plots are presented in Anbazhagan et al. [5]. Most of the time, the energy measurement is taken for specific equipment in one Borehole, and it is assumed as constant for the rest of the boreholes in the project, which is also not correct as the energy ratio for different equipment varies in a project [28].

The energy transfer ratio (ETR) variation in the top sensor of all six boreholes at each N-value measurement is shown in Fig. 6. Further, all energy values in each borehole are used to calculate the average energy ratio of a borehole. Boreholes drilled by the same equipment are used to estimate an average equipment energy ratio. Based on these field observations, four different average ETR are considered, (1) an average of all blows (N-values) of a depth (*average each depth energy in a borehole: ADE*), (2) an average of all depth of a borehole (*average borehole energy: ABE*), (3) an average of all boreholes of equipment (*average equipment energy: AEE*), and (4) an average of

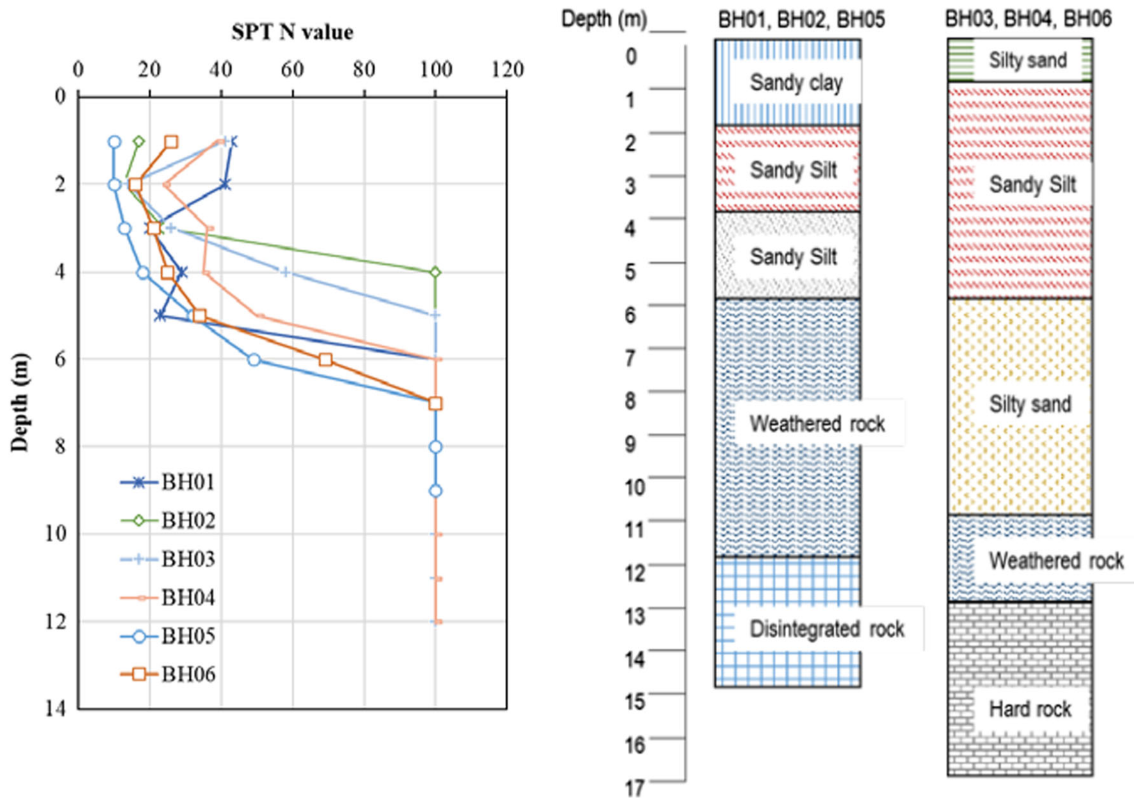


Fig. 5 Measured SPT values in the four boreholes, along with the subsurface profile

all boreholes with different equipment of a site (*average site energy: ASE*). Figure 6 also shows the average energy ratio with variation in each borehole, equipment, and entire site. A significant difference in ETR values can be observed within a borehole, e.g., ~ 50% in the case of BH05 and BH06; similar differences can be observed between the two boreholes of the same equipment. When different types of equipment are used in one site, a high energy difference of 30–40% between the measured values can be reported, as shown in this study. This variation clearly shows the importance of measuring energy at each blow and considering average energy for N-values in each

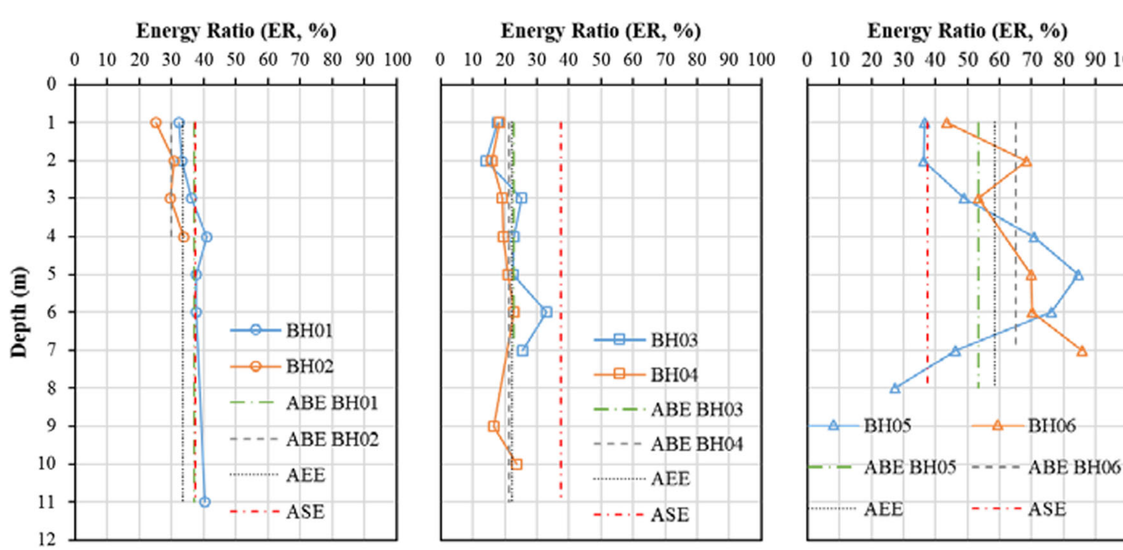


Fig. 6 The Energy Transfer Ratio (ETR) variation in the Top sensor of all six boreholes for each SPT N value measurement (**a**) BH01-02, (**b**) BH03-04, (**c**) BH05-06

borehole rather than measuring at only one depth and recommending it for other depths or other boreholes. Adopting a few energy measurements as per ASTM [13] may not be appropriate in developing countries like India as all SPT equipment is not similar and very limited automatic systems are engaged. Also, most SPT tests are carried out by unqualified and uncertified technicians. Hence, it must be made necessary to measure hammer energy for every blow at a depth and average them for the N value because most of the SPT equipment being used are locally fabricated. Here, it can be highlighted that energy measurement at selected depth or numbers per ASTM D4633 [13] and EN [14] may not be appropriate for countries using untrained operators and different types of SPT setup.

Subsurface Profile by GPR

GPR is an electromagnetic (EM) method that detects interfaces between subsurface materials by detecting changes in dielectric constant (κ). A typical GPR system consists of a transmitter, a receiver, and a profile recorder to process the received signal and later to display the recorded data in a graphical format. An antenna houses both the transmitter and the receiver. The transmitter radiates high frequency electromagnetic signals into the earth, which are reflected to the receiver by interfaces between materials with different dielectric constants. The intensity of the reflected wave signal depends on the contrast in κ at the interface, travel medium's conductivity, and signal frequency. Such reflections can be caused by 1) contrast in sediment composition, (2) presence of bedrock or high-density soil/rock, voids, and water content, or (3) human-induced discontinuities, such as soil backfill, buried debris, tanks, pipelines, and utilities. The profiling recorder produces a continuous cross-section of the subsurface interface reflections, referred to as reflectors.

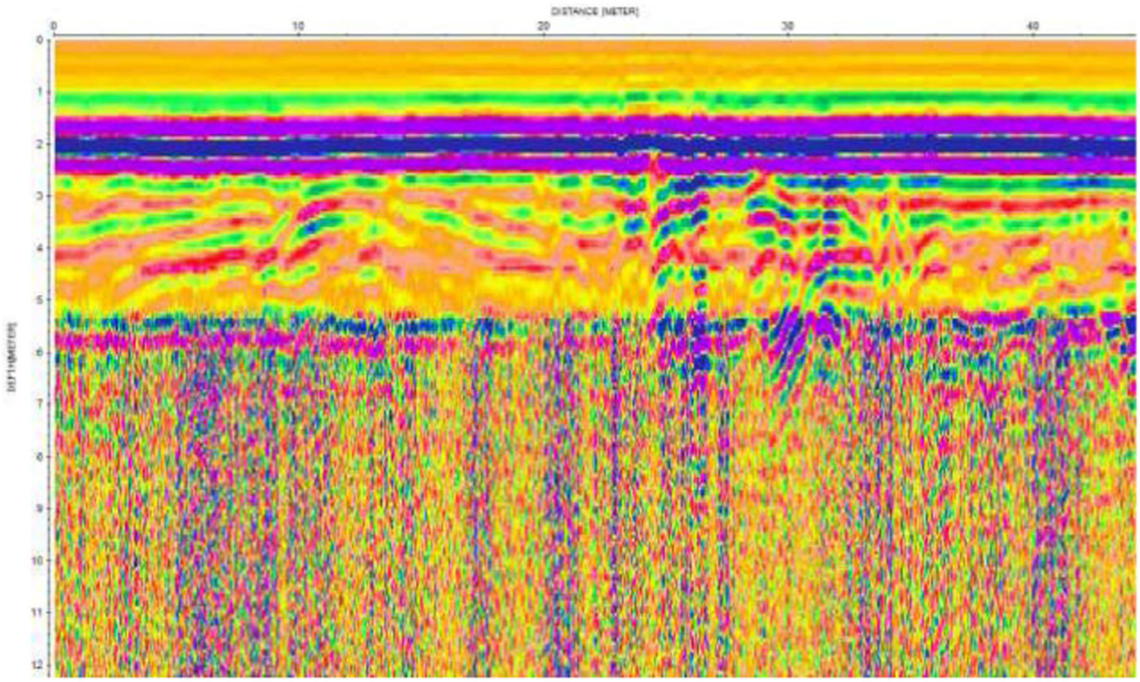
In this study, MALÅ GX antennas of 80 MHz and 250 MHz were used for deeper and shallow depth subsurface profiling, respectively. Depth of investigation using GPR is limited by signal attenuation by the subsurface materials. Materials with high electrical conductivity (e.g., clays and brackish groundwater) cause more attenuation than materials with low conductivity (e.g., unsaturated sand or rock). The maximum depth of investigation generally increases with decreasing frequency; however, the ability to identify smaller features reduces as frequency decreases. The largest antennas typically radiate the lower frequencies necessary to detect the deepest targets. The smallest antennas radiate the highest frequencies with the highest resolution required to detect small, shallow targets. Reflected electromagnetic waves are displaced in the monitor and also stored in the system as raw GPR radargram and give a rough idea about the subsurface and depth of data available. Data obtained from the field survey are processed using REFLEX and PRISM software. Figure 7a, b shows a typically processed radargram from 80 and 250 MHz GPR antenna corresponding to the survey line close to BH01. These radargram and waveform are further used to delineate and understand subsurface layers' heterogeneity. Three subsurface layers were identified based on radargram, which was also verified later by borelogs. The top layer of sandy clay soil was uniform throughout the site and found in all the borelogs. Other layers consist of sandy silt and silty sand, which are also found in the whole area and vary in thickness with distance.

Subsurface Profile by MASW

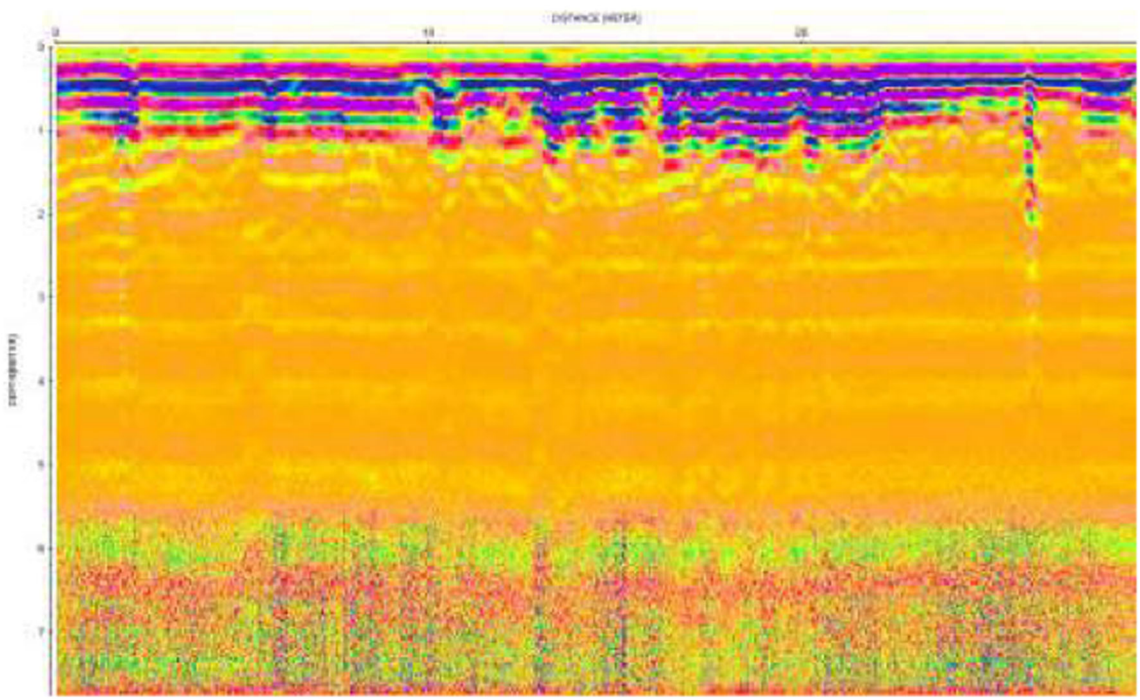
MASW, an in-situ non-destructive method, uses frequency-dependent properties of horizontally travelling surface waves (Rayleigh) to visualize and characterize sub-surface lithology [32]. In this study, an MASW setup consisting of 24 geophones of 4.5 Hz Natural frequency and a 24-channel geode seismograph has been used to obtain a 2D V_S profile of three survey lines (Fig. 4). Survey line 1 passes through BH01 and BH02, whereas line 2 passes through BH03 and BH04. An active source created the seismic waves at the site by impounding a 15-pound sledgehammer on a 300 mm × 300 mm mild steel plate of 1-inch thickness. A stack count of 10 shots were taken for recording the data set. The geophone spacing was kept at 1 m, and the shot location was kept at 5 m from the nearest geophone to obtain the best achievable signal-to-noise ratio. The survey was repeated in the forward direction to perform a 2D MASW survey. The wave data, thus recorded by vertical geophones, were then processed by ParkSEIS software to obtain a 2D V_S profile across the length in three simple steps: (1) Application of Fourier transformation to convert the wave signal from time to frequency domain. (2) Dispersion-curve analysis and (3) Inversion. A typical 2D V_S profile along line 1 is presented in Fig. 8. The profile is coherent with the borelog, where a sharp impedance is observed at 10–12 m depth. Shallow profiles have low V_S, which consist of sandy sand silty soils. 5–10 m of depth consists of transition layers (weathered rocks) which is evident from V_S values of 300–600 m/s. $V_S > 1500$ m/s is found at a depth of 20 m which signifies a very hard rock layer. Overall, the bedrock depth seems almost constant, varying between 10 and 12 m.

Seismic Borehole Test

The Downhole (DH) and Crosshole (CH) seismic test consists of the generation of P-waves and S-waves at/near the surface, which travels down to a borehole receiver/



(a) Typical radargram using 80 MHz antenna



(b) Typical radargram using 250 MHz antenna

Fig. 7 (**a**) Typical radargram using 80 MHz antenna. (**b**) Typical radargram using 250 MHz antenna

array of receivers installed in the Borehole at a given depth. Seismic wave velocities can be obtained from retrieving the travel time of waves at different depths. The downhole tests were performed at the six boreholes independently, and four cross-hole tests were conducted. These results can be used to determine the dynamic properties like shear modulus and Poisson’s ratio and cross-validate MASW results.

In the downhole test, subsurface seismic velocity is determined by placing a source at the ground surface near

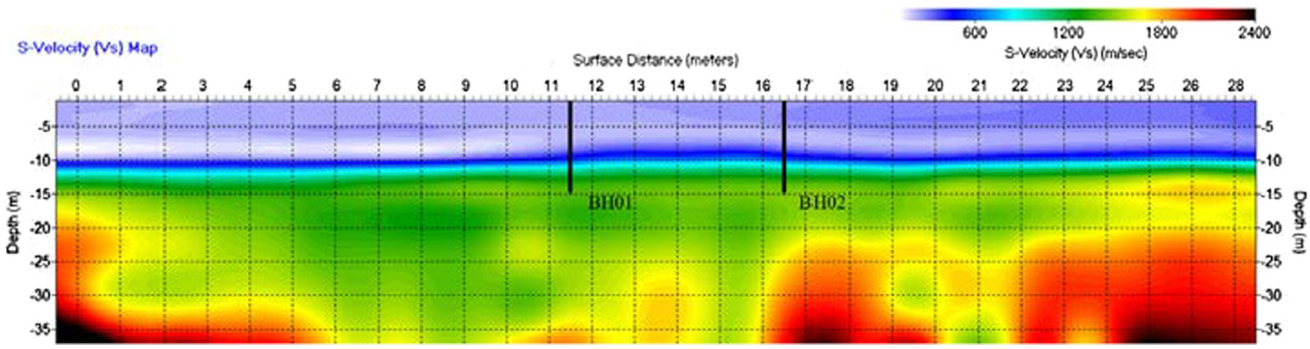


Fig. 8 2D V_S profile along the survey line1, passing through Boreholes BH01 and BH02

the borehole and measuring travel times at multiple intervals in the borehole, usually with a 3-component geophone. A wooded beam with metal capped ends is used for the generation of shear waves at the surface. In this study, BGK5 receiver from Geotomographie GmBH is used for recording. A crosshole survey measures the velocity of seismic waves between the boreholes. This method involves lowering a seismic source in one borehole and a 3(or more) component geophone borehole receiver in the adjacent borehole(s). The source and the receiver are kept at the same depth to record the impulse from the source. In this study, BIS SH source from Geotomographie GmBH is used for impulse generation. Based on S-wave and P-wave velocities (V_S, V_P), other dynamic properties such as low strain shear modulus (G), Young's modulus (E), Bulk modulus and Poisson's ratio can be obtained. Typical V_S profile and Poisson's ratio profile obtained from tests is given in Fig. 9.

For shallow depths, MASW, CH and DH show matching V_S values. Some mismatches can be observed at the weathered rocks and hard rock layers interface. These variations can be attributed to the method of calculation of V_S in different methods. For example, MASW V_S profile is the average value of entire spread length, and Downhole V_S is calculated for travel path from source (surface) to receiver (in the borehole at a particular depth); and Crosshole measures V_S values at a depth horizontally. Both crosshole and downhole methods resulted in the same Poisson's Ratio values at some depths, indicating that the entire depth consisted of the same subsurface material. At some depths, variations can be noticed, possibly due to different compositions of the same material present in different sides of the borehole.

Soil Properties from SPT N, Corrected N and In Situ Tests

G_{max} values have been estimated using the correlation previously discussed in Sect. 4. As per Eq. 2, $G_{max} = 16.40N_{78}^{0.65}$. Based on the delivered hammer energy, measured N-values are normalized to 78% reference ETR. Then, G_{max} is estimated using uncorrected and corrected N-values and further compared with G_{max} calculated from V_S profile. Similarly, for Young's modulus (E), Bowles's [4] correlation for normally consolidated sands has been used.

$$E = 500(N_{55} + 15)/1000 \text{ (MPa)} \quad (6)$$

The mentioned correlations from Eqs. 2 and 6 are used to estimate G_{max} and E for the recorded N-values using correction for ADE values. These values are compared with (1) cases when no energy correction is applied, (2) when the correction is applied as per IS 1893 [8], and (3) moduli determined from V_S measured in-situ (Fig. 10).

Figure 10a shows that uncorrected and 80% corrected N give the same G_{max} because uncorrected N is assumed to be at 78% ETR in the correlation. The ADE corrected N-values show lesser G_{max} compared to the other two. The values measured from the field tests of CH and DH agree with modulus from ADE-based correlation, although the difference is high in the weathered rock layer. They do not follow the trend shown by the G_{max} values derived from correlation, which depend only on the N value. In Fig. 10b, E obtained from CH and DH results is much higher when compared to the correlation results. For shallow depths, modulus from CH and DH is roughly 8–10 times than those from correlation. However, after soil layers, there is a sharp jump in modulus from CH and DH, which correlation-values fail to capture. In rebound layers, this might be because of low energy values, which lead to a rebound, and thus N-values are restricted. The same can be commented about Fig. 10a as well in the rebound region. Energy-based modulus correlation developed for the region is well comparable with in-situ values measured modulus values.

SBC from SPT N, Corrected N and In Situ Tests

Similar to the previous section, the influence of ETR on SBC is studied. Based on Eq. 5, corrected N-values $(N_1)_{70}$ is used for SBC estimation and compared with the

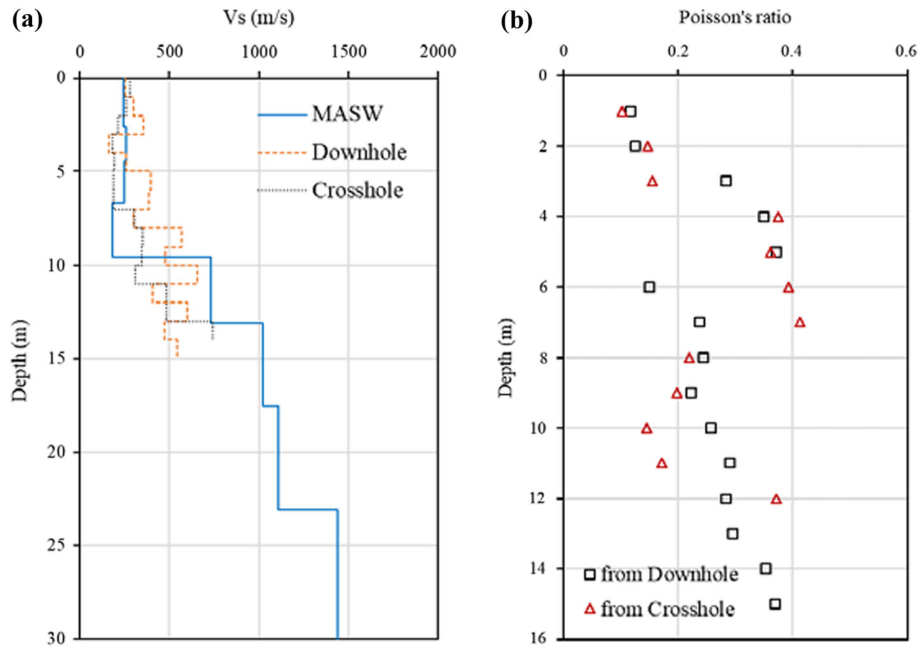


Fig. 9 (**a**) Typical Vs plots and (**b**) Typical Poisson's Ratio values

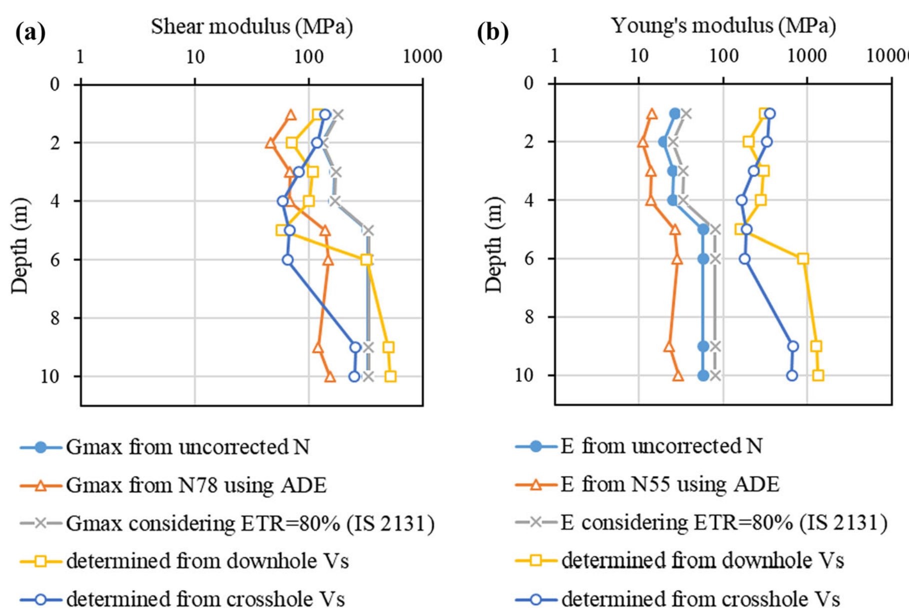


Fig. 10 Estimation of (**a**) G_{max} and (**b**) E using standard correlations and in-situ V_S profiles

correction method suggested in IS 2131. Calculations are carried out for depths of 1.5 m and 3 m for square footings of 1.5 m, 3 m, and 6 m width. Average N-values above foundation depth are considered to estimate the unit weight of soil adopting the correlations from Anbazhagan et al. [2]. Average corrected N-values up to 2B below the foundation depth are considered for estimating bearing capacity factors as per IS 6403 [22]. A correction factor of 1.3 for square shape is accounted, and no water table factor is considered as the depth of water was at 13 m. Similarly, no depth correction is needed as the soil is virgin ground, and no slope factor is considered as the ground is flat. These values are used to estimate q_d as per Eq. 5 and further Net SBC with a factor of safety 3 at each case and plotted in Figs. 11 and 12. Both these figures highlight the difference between the SBC values when calculated considering the in-situ ETR measurement and neglecting the same.

Figures 11 and 12 show a significant difference between SBC from IS 2131 [7] method and $(N_1)_{70}$ method (N-values corrected for 70% reference ETR) based on ADE. Overestimation of SBC, when calculated using, IS 2131 [7] method, is clearly evident. In Fig. 11, the effect of ETR on SBC is predominantly noticed in shallow layers. It is not so prominent in deeper layers such as rock or rebound layers. This is possible because of increased N-values as delivered hammer energy is low and N-values remain uncorrected. When the Net SBC values from shear criteria are compared in between the different dimensions of the foundation, i.e., 1.5 m, 3 m, and 6 m, no significant difference can be found except a slight increase in the Net SBC value when the dimension of the foundation increases from 1.5 to 6 m. At 3 m depth, the net SBC values are significantly higher than those at 1.5 m depth. This is because when all the soil parameters are kept constant, the SBC mainly depends on both the depth and depth factor of the foundation, and the angle of shearing resistance saturates at a higher N-value (> 75). Although with the increase in depth, the difference between SBC values for the three footing dimensions becomes marginal.

No significant difference is observed in Net SBC by settlement criteria (Fig. 12) except a slight decrease in the Net SBC value when dimension increased from 1.5 to 6 m. Saturation of SBC value can be observed because of the restriction of maximum allowable settlement, Depth factor, and Settlement from N-value. Net SBC from shear criteria is around 10–15 times greater than the settlement criteria. This is because of the restriction in settlement of the foundation under the applied load on soil. The lesser SBC out of settlement and shear criteria need to be considered for the design of the foundation, which will always be a safer design. It can be noted here that SPT N versus angle of internal friction given in IS 6403 [22] is valid for sand studied by Terzhaghi and Peck [15]. When the foundation is designed for silty sand with clay (Bangalore region), additional in situ properties based on SBC calculation need to be followed [5].

Integrated Subsurface Investigation

Borehole explorations only furnish the subsurface profiles in vertical directions, generally spaced apart by a significant distance. Because of this fragmentary nature of profiles, interpolation of soil layers between boreholes is often necessary [15]. Hence, the degree of reliability of the computation will depend entirely on the difference between real and interpolated profiles. Recently, a new Integrated Subsurface Investigation (ISI) method of subsurface exploration has been developed [33] to get detailed geotechnical data required for the any project. Multiple in-situ tests should be used to fulfill the requirement of the project. Chandran and Anbazhagan [34] and Anbazhagan et al. [35] applied ISI for different projects and showed how ISI effectively addresses various industry problems in complicated geological deposits and site conditions. In ISI method, we select different in-situ tests among a pool of most reliable in-situ tests in geotechnical engineering.

Carrying out selected in-situ tests systematically, processing the data and finally generating an integrated subsurface profile by combining all test results. Final subsurface profiles in 2D or maps to show the variation of thickness and stiffness of subsurface materials. The number of in-situ tests is decided based on the depth of information required and the space available. This study uses borehole, GPR and seismic data and estimated design parameters to

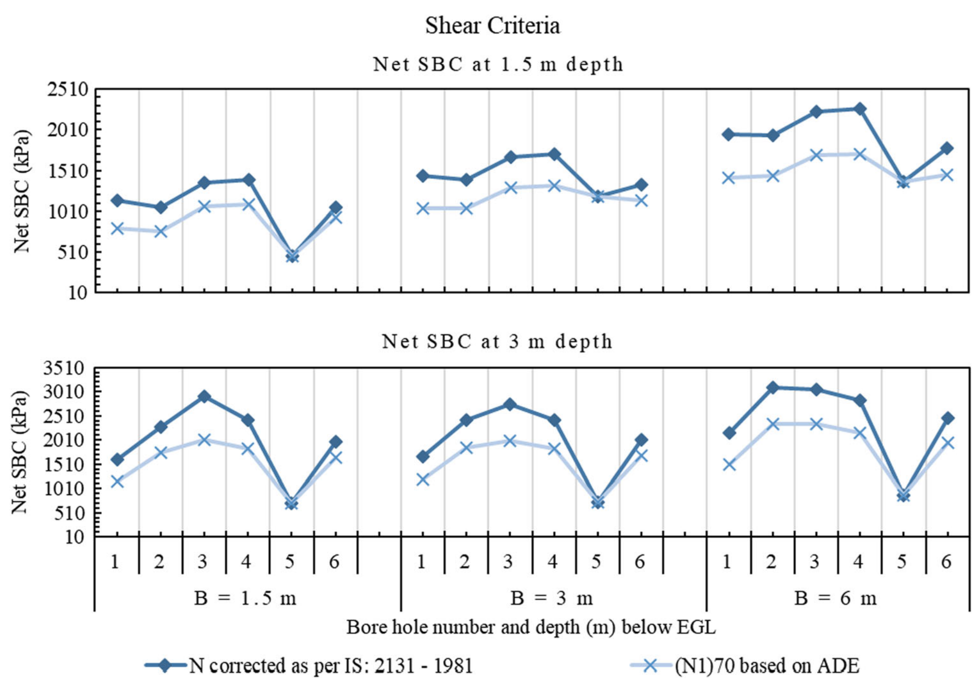


Fig. 11 Net SBC by shear criteria at 1.5 m and 3 m foundation depth for 1.5 m, 3 m and 6 m foundation width

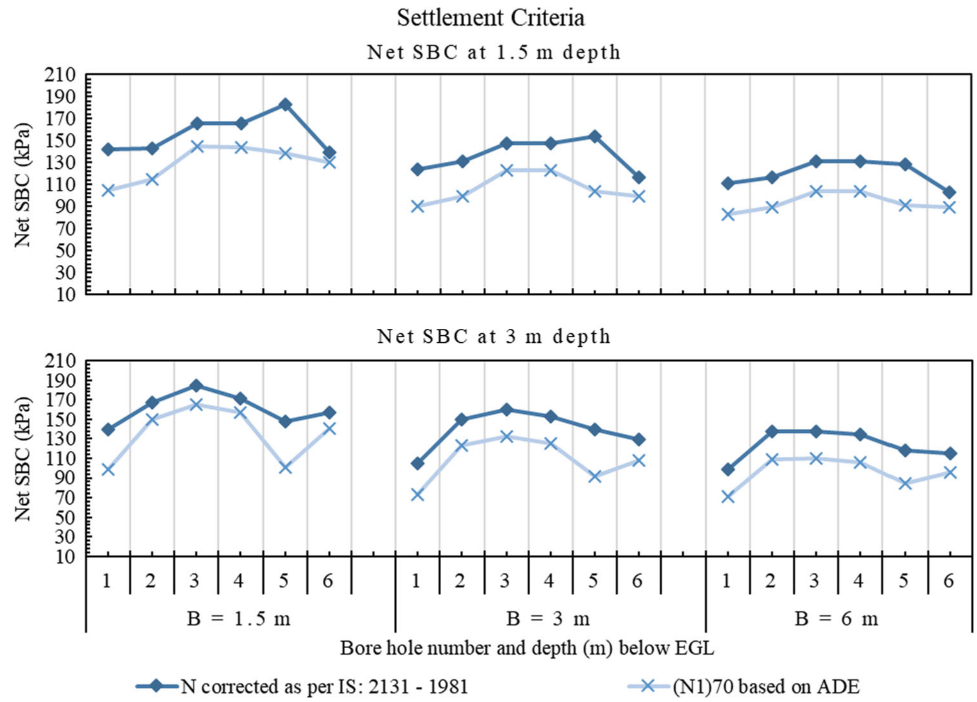


Fig. 12 Net SBC by settlement criteria at 1.5 m and 3 m foundation depth for 1.5 m, 3 m and 6 m foundation width

generate integrated subsurface profiles at different alignments to show variation.

Integrated profiles of selected locations have been generated by combining GPR 250 MHz and 80 MHz radargram data, borehole data and V_S measured from MASW survey. The thickness of the shallow subsurface layers was obtained from 250 MHz radargram and other subsurface layers obtained from 80 MHz radargram. The material type was obtained from nearby borehole data, and average shear wave velocity was obtained for subsoil layers. 2D profiles at a selected construction site location were developed by integrating GPR radargram and MASW 2D Vs profile. Integrated subsurface profiles were created for locations shown in Fig. 4 and presented as Fig. 13. It can be observed that the top subsurface layer up to a depth of 4 m is uniform spatially, beyond that is the dense silty sand layer, weathered rock layer and hard rock stratum found with different stiffness as shown in Fig. 8. The very dense silty sand layer above the weathered rock is not uniform across the site and its thickness varies. The spatial distribution of V_S above the weathered rock layer helps identify material with similar stiffness, which is confirmed from borelogs data. Very hard rock with V_S more than 1500 m/s is found within 35 m depth and its depth varies in the site.

Figure 13 also shows the layer's dynamic properties identified by the ISI method. The respective layers show an increasing trend of V_S with depth in general. However, in the shallow layers till 6 m, V_S does not show much change and is almost constant. A similar trend is observed in the other properties. The borelog did not extend up to the very hard rock layer, but through the MASW survey, these values were obtained for higher depths. Poisson's ratio was obtained by combining MASW, CH and DH survey results and used to estimate Young's modulus and bulk modulus.

Recently, due to heavy rains in Bangalore's residual terrain [Study area is located), many buildings, cracks and failures have been reported. Some of the buildings have collapsed on their own or settled extensively. To prevent these accidents from occurring again, government officials identified over 500 structures for demolition [36]. Video recording of a few collapsed buildings helped understand some aspects like the initial formation of cracks, followed by tilt and collapse after several hours. The sequence of events of building collapse needs to be identified and understood. Many experts have inferred that these failures are due to soil foundation rather than structural defects. Residual soil is generally found with different thicknesses and mixed materials of silty sand with little clay. Integrated profiles reveal the spatial variation of these materials to account for variation in SBC and settlement, which is missed by boreholes. For any big project or building construction, the ground is levelled (layout making) by filling down the area, but soil testing is done by the point-based method of boring and SPT N count, which remain uncorrected for hammer energy. Spatial distribution of subsoil due to filling are not accounted to suggest SBC for the design.

Moreover, as highlighted in the SBC section, N-values are predominantly used to determine φ as per IS 6403 [22]. Then, bearing capacity factors are calculated as per Peck

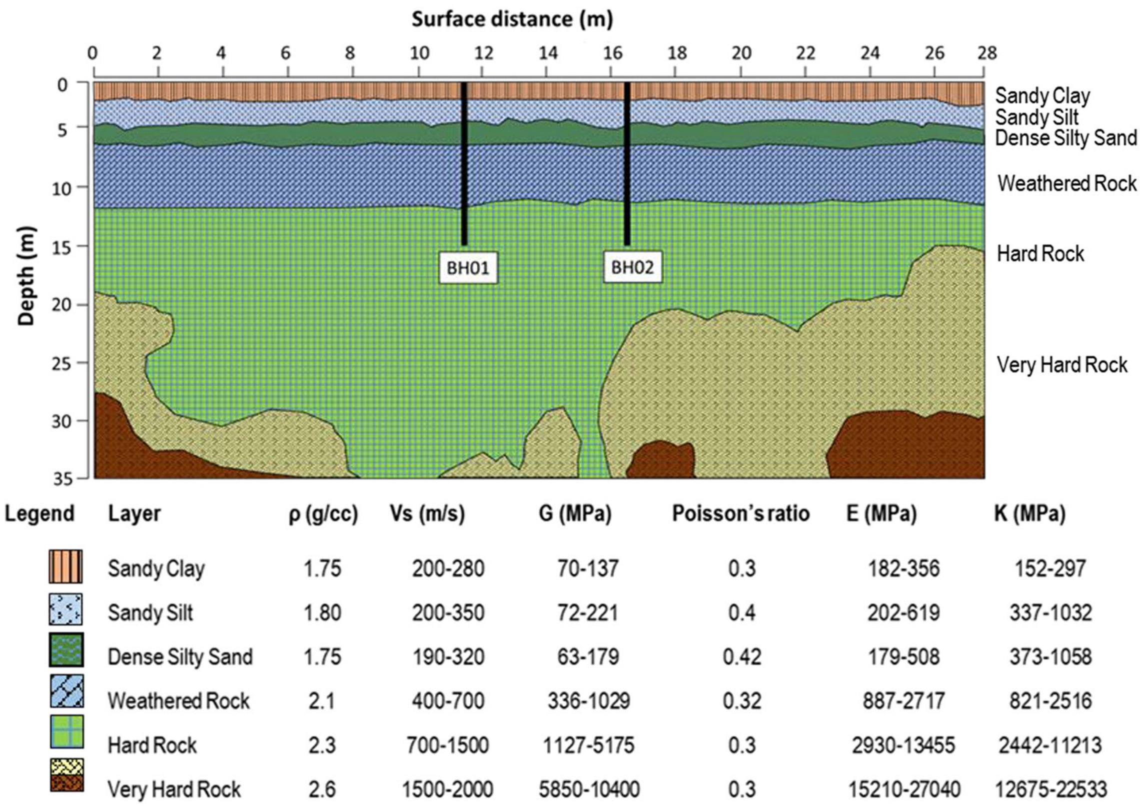


Fig. 13 Integrated Subsurface Profile along Line 1 in Fig. 4

et al. [37], which was developed for sandy soil. This chart [37] is strictly for non-plastic soil, where most of the residual region like Bangalore has silty sand with clay, which may have different φ and settlement than those estimated from charts given in IS codes. Authors opine the following practices lead to error in the determination of soil properties: (1) current practice of not measuring hammer energy during SPT (2) estimating unit weight or state of soil using uncorrected N-value (3) predicting soil properties such as φ and modulus using uncorrected N-values in correlations and charts, (4) Not accounting spatial variation of subsurface layers of a site in design, as currently geo-structural design are based on point-based SPT tests with uncorrected N values. These errors cumulatively lead to an estimation of SBC for shear and settlement criteria. Hence, it is highly suggested to measure hammer energy and to use corrected N-values in region-specific correlations along with its spatial variation of subsurface layers through ISI to estimate SBC. Unfortunately, there is no correlation between energy-corrected N-values with any soil properties in India. So one can go for an alternate way of measuring in-situ properties through seismic testing and use the same to validate code-based results to avoid future geohazards. Integrated profiling can help to understand soil thickness and stiffness variation and account for them to estimate suitable SBC for shear and settlement criteria.

Conclusions

In this study, the influence of hammer energy has been discussed on soil properties such as density, low strain shear modulus and soil bearing capacity. A site investigation study was conducted to observe the influence of hammer energy measurement on these properties. SPT was conducted in six boreholes at different depths and hammer energy was measured in all the tests. Energy variation was observed to affect the estimation of G_{max} and SBC significantly. Modification factors to use G_{max} correlation for any ETR value were discussed, and its difference from regular N-value correction was highlighted. Several rebound criteria were studied to understand the role of hammer energy measurement in establishing refusal/rebound strata. The rebound depth in SPT was found to be dependent on the energy delivered from the hammer to the sampler. Lower ETR values resulted in shallow rebound depth as the N-value becomes higher, which could lead to severe overestimation of soil stiffness and shear strength. The use of uncorrected N-values resulted in overestimated SBC values as per IS 2131, and values calculated from energy corrected N-values were more reliable. Further, to understand spatial variation of soil properties, integrated 2D subsurface profiles were generated using GPR, MASW and borelogs. Shallow subsurface layers were identified using GPR radargram and deeper layers using MASW 2D V_S profiles. These profiles were also validated with the borelog profiles. Crosshole and Downhole tests were conducted to measure the Poisson's ratio of different depths. The V_S values obtained from the three seismic methods agree with each other in shallow depths. Clear visualization of the soil and rock layers was presented, which affirmed that dependence on only boreholes to delineate different subsurface layers and their spatial variation is not ideal. Non-destructive surface methods should accompany borehole studies to obtain more reliable profiles. It can be noted here that soil parameters (φ & C) required for geotechnical design are estimated from exiting correlation developed elsewhere; most of these correlations are not directly applicable to Indian soil. The geotechnical community can take up the systematic study to arrive at regional soil properties of φ & C and relate them with the same soil's energy corrected SPT N values. So that our SBC estimation and design are representative and appropriate for the region.

Acknowledgements The authors thank the Dam Safety (Rehabilitation) Directorate, Central Water Commission for funding the project entitled "Capacity Buildings in Dam Safety" under Dam Rehabilitation and Improvement Project". The authors also thank SERB, DST for funding the project "Development of correction factors for standard penetration test N-values in India through energy measurement and field experiments – Step toward a reliable Liquefaction Potential Assessment" Ref: SERB/F/198/2017-18 dated 11/05/2017." Author thanks M/s. SECON Private Limited, Bangalore for funding project "Effect of Shear Wave Velocity Calibration on Amplification of Shallow and Deep Soil Sites."

References

1. Anbazhagan P, Parihar A, Rashmi H (2012) Review of correlations between SPT N and shear modulus: a new correlation applicable to any region. Soil Dyn Earthq Eng 36:52–69. https://doi.org/10.1016/j.soildyn.2012.01.005
2. Anbazhagan P, Uday A, Moustafa SS, Al-Arifi NS (2016) Correlation of densities with shear wave velocities and N-values. J Geophys Eng 13:320–341. https://doi.org/10.1088/1742-2132/13/3/320
3. Anbazhagan P, Kumar A, Ingle SG, Jha SK, Lenin KR (2021) Shear modulus from SPT N-value with different energy values. Soil Dyn Earthq Eng 150:106925. https://doi.org/10.1016/j.soildyn.2021.106925
4. Bowles JE (1996) Foundation analysis and design. MacGraw Hill, New York
5. Anbazhagan P, Yadhunandan ME, Kumar A (2022) Effects of hammer energy on borehole termination and SBC calculation through site-specific Hammer energy measurement using SPT HEMA. Indian Geotech J. https://doi.org/10.1007/s40098-021-00582-z
6. Aggour MS, Radding WR (2001) Standard penetration test (SPT) correction. Research Report No. MD02–007B48, The Bridge Engineering Software and Technology (Best) Center, Department of Civil and Environmental Engineering, University of Maryland, College Park, MD 20742
7. IS 2131 (1981) Indian standard method for standard penetration test for soils (first revision) reaffirmed 2002. Bureau of Indian Standards, New Delhi
8. IS 1893–Part 1 (2016) Indian Standard Criteria for Earthquake Resistant Design of Structures Part 1 General Provisions and Buildings. Bureau of Indian Standards, N4[ew Delhi
9. Schmertmann JH, Palacios A (1979) Energy dynamics of SPT. J Geotech Eng Div 105:909–926. https://doi.org/10.1061/ajgeb6.0000839
10. Seed HB, Idriss IM, Arango I (1983) Evaluation of liquefaction potential using field performance data. J Geotech Eng 109(3):458–482
11. Skempton AW (1987) Discussion: Standard penetration test procedures and the effects in sands of overburden pressure, relative density, particle size, ageing and overconsolidation. Géotechnique 37:411–412. https://doi.org/10.1680/geot.1987.37.3.411
12. Kovacs WD, Salamone LA, Yokel FY (1981) Energy measurement in the standard penetration test. U.S. Department of Commerce and National Bureau of Standards. doi: https://doi.org/10.6028/nbs.bss.135
13. ASTM D4633 (2016) Standard test method for energy measurement for dynamic penetrometers. ASTM International, West Conshohocken, PA. doi:https://doi.org/10.1520/d4633-16
14. BS EN ISO 22476–3:2005+A1:2011 (2006) Geotechnical investigation and testing, Field testing, Standard penetration test. British Standards Institution (BSI), London
15. Terzaghi K, Peck RB (1967) Soil mechanics in engineering practice. Wiley, New York
16. Schmertmann JH (1978) Use the SPT to measure dynamic soil properties?—Yes, But..!. Dynamic Geotechnical Testing, ASTM STP 654. American Society for Testing and Materials, pp 341–355
17. Burland JB, Burbidge MC, Wilson EJ (1985) Settlement of foundations on sand and gravel. Proc Inst Civ Eng 78:1325–1381. https://doi.org/10.1680/iicep.1985.1058
18. Terzaghi K, Peck RB, Mesri G (1996) Soil mechanics in engineering practice. Wiley, New York
19. Parry RHG (1977) Estimating bearing capacity in sand from SPT values. J Geotech Eng Div 103:1014–1019. https://doi.org/10.1061/ajgeb6.0000484
20. Meyerhof GG (1957) Discussion of penetration tests and bearing capacity of cohesionless soils. J Soil Mech Found Div. https://doi.org/10.1061/jsfeaq.0000034
21. Meyerhof GG (1974) Ultimate bearing capacity of footings on sand layer overlying clay. Can Geotech J 11:223–229. https://doi.org/10.1139/t74-018
22. IS 6403 (1981) Indian standard code of practice for determination of breaking capacity of shallow foundations (first revision) reaffirmed 2002. Bureau of Indian Standards, New Delhi
23. IS 8009 (Part 1) (1976) Indian standard code of practice for calculation of settlements of foundations, shallow foundations subjected to symmetrical static vertical loads, reaffirmed 2003. Bureau of Indian Standards, New Delhi
24. IS 875 (Part 1) (1987) Indian standard code of practice for design loads (other than earthquake) for buildings and structures, dead loads – unit weights of building materials and stored materials (second revision) reaffirmed, 2003. Bureau of Indian Standards, New Delhi
25. IS 875 (Part 2) (1987) Indian standard code of practice for design loads (other than earthquake) for buildings and structures, imposed loads (second revision) reaffirmed 2008. Bureau of Indian Standards, New Delhi
26. IS 875 (Part 3) (1987) Indian standard code of practice for design loads (other than earthquake) for buildings and structures, wind

loads (second revision) reaffirmed 2003. Bureau of Indian Standards, New Delhi
27. IS 875 (Part 4) (1987) Indian standard code of practice for design loads (other than earthquake) for buildings and structures, snow loads (second revision) reaffirmed 2003. Bureau of Indian Standards, New Delhi
28. Anbazhagan P, Ingale SG (2021) Status quo of Standard penetration test in India: a review of field practices and suggestions to incorporate in Is 2131. Indian Geotech J 51:421–434. https://doi.org/10.1007/s40098-020-00458-8
29. Cheney RS, Chassie RG (1993) Soils and foundations workshop manual, 2nd edn. FHWA HI-88–009
30. ASTM D4428/ D4428M (2014) Standard Test Methods for Crosshole Seismic Testing. ASTM International, West Conshohocken, PA.
31. IS 1892 (1979) Indian standard code of practice for subsurface investigation for foundations (first revision) reaffirmed 2002. Bureau of Indian Standards, New Delhi
32. Park CB, Miller RD, Xia J (1999) Multichannel analysis of surface waves. Geophysics 64(3):800–808
33. Anbazhagan P (2017) Subsurface investigations- integrated and modern approach. In: Dey A, Sreedeep S, Krishna A (eds) Geotechnics for natural and engineered sustainable technologies. Developments in geotechnical engineering. Springer, Singapore. https://doi.org/10.1007/978-981-10-7721-0_13
34. Chandran D, Anbazhagan P (2017) Subsurface profiling using integrated geophysical methods for 2-D site response analysis at Bangalore City-India: a new approach. J Geophys Eng 14:1300–1314. https://doi.org/10.1088/1742-2140/aa7bc4
35. Anbazhagan P, Divyesh R, Prabhakaran A, Vidyaranya B (2018) Identification of Karstic features in lateritic soil by an integrated geophysical approach. Pure Appl Geophys 175(12):4515–4536. https://doi.org/10.1007/s00024-018-1908-8
36. Ramani CV (2021) BBMP identifies over 500 dilapidated structures in preliminary report. The Hindu, 20 October 2021. The Hindu (Online), https://www.thehindu.com/news/cities/bangalore/bbmp-identifies-over-500-dilapidated-structures-in-preliminary-report/article37083189.ece
37. Peck RB, Hanson WE, Thoruburn TH (1953) Foundation engineering, 2nd edn. Wiley, Canada