Characterization of residual soil in tailings dam foundations: A combined analysis of in-situ tests and geophysical surveys with emphasis on method correspondence

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ABSTRACT

New legislation in Brazil requires that all upstream tailings dams must be closed. However, understanding the characteristics of the foundation and deposits of these dams can be complex, and conventional investigations may not be sufficient. Geophysical methods can complement conventional approaches and provide a better understanding of geotechnical structures. The objective of this investigation is to comprehensively characterize residual soil from a tailings dam foundation by integrating data from direct and indirect approaches. Data from CPTu, SCPTu, SPT, geophysical profiles, and MASW were analyzed. The results were compared with empirical correlations for other soil types, and it was found that the equations are not effective to represent the materials. The study discusses the advantages and limitations of using these empirical equations.

Keywords: Geophysical, MASW, residual soil.

1. Introduction

Brazil has recently introduced new legislation that prohibits using the upstream construction method for building or increasing the height of dams. This is stated in ANM Resolution No. 13/2019 and Law No. 14,066/2020. It is mandatory to de-characterize (closure) all dams within a specified deadline, as mentioned in § 1 of Art 2-A of Law No. 14,066. Any extension to this deadline can only be granted in cases of technical unfeasibility, as per §3 of Art 2-A of Law No. 12,334/2010.

However, tailings storage facilities are some of the most challenging structures to operate in the mining industry, and some of these structures are susceptible to liquefaction and piping failure modes, as reported by Olivier et al. (2018) and Silva et al. (2022) the behaviour of these tailings has brought the importance of small-strain stiffness to the geotechnical forefront.

To de-characterize works, it is crucial to have a reliable and suitable selection of geotechnical design parameters. Therefore, it is essential to understand the soil foundation behaviour to choose the most appropriate, effective, and sustainable solution for closure design of the structure.

Non-invasive subsurface imaging techniques utilizing seismic wave propagation have become increasingly popular in recent decades. They are a more cost-effective alternative to conventional invasive site characterization methods and can cover large areas. These imaging techniques primarily focus on capturing two crucial soil parameters of particular interest in geotechnical engineering: the small-strain shear wave velocity (V_S) and damping ratio (D_S). V_S is directly related to the small-strain shear modulus (G_0 or Gmax), which represents the stiffness of the soil. D_S quantifies the soil's internal energy dissipation at low strains (Abbas et al., 2024).

This study characterized the residual soil in a tailings dam foundation using data from in-situ tests, and indirect geophysical methods. The data from CPTu, SCPTu, and SPT surveys were compared with investigations using geophysical profiles and Multichannel Analysis of Surface Waves (MASW). The utilization of data from multiple sources can significantly contribute to a better understanding of these geotechnical materials.

2. Material description and methods for characterization

2.1. Residual soil

The mine is situated in the eastern of the Iron Quadrangle, specifically south of Serra do Caraça, Minas Gerais, Brazil. The elevation of this area is higher than the surrounding regions, with level differences of over a thousand meters. The study area is characterized by the presence of various types of rock, mainly those found in the Minas Supergroup, including Itabirites (phyllitic and dolomitic) and high-grade compact Hematites from the Itabira Group, phyllites and mudstones from the Batatal Formation, and quartzites, quartz-sericite schist, and quartz-mica schist from the Moeda Formation, both belonging to the Caraça Group. The Cenozoic era covers the rocks present in the region, such as laterites and alluvium. Numerous mafic dikes intercept the rocks above. The primary structures in the area are represented by thrust faults, which run east/southeast to west/northwest, and strike-slip faults, which alter the dip of the layers regionally, presenting traces of three primary deformational cycles: Trans-Amazonian, Brasilian, and South Atlantian. The ridges of the mountain range align with the rock layers. To the north, the Santa Rita syncline is observed in a SW-NE direction.

As shown in Figure 1, the residual soil varies in color, ranging from gray (A and C), yellow (B), and reddish-brown (D); the tones vary according to the proximity to the mother rock and the degree of alteration of the material, where the grayish soil would be the least altered. The yellow soil would be the most oxidized. It has a silty sandy from the Particle Size Distribution (PSD) with few passages of clay, and the residual material is mainly compact, varying from moderately compact at lower depths to very compact at higher depths.

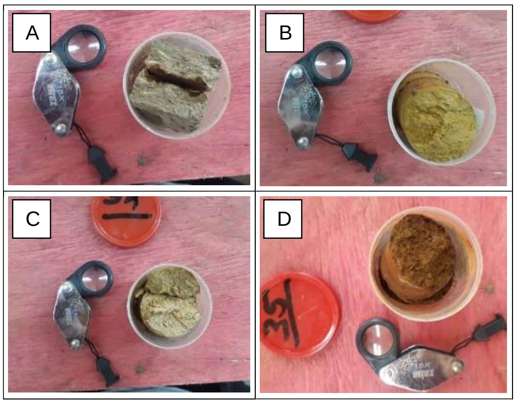

Figure 1. Residual soil ranges from gray (A and C), yellow (B), and reddish-brown (D);

2.2. Field investigation

The residual soil was thoroughly investigated using SPT and SCPTu methods at two survey points (SM-16 and SM-607). Additionally, a MASW section was indirectly employed to investigate the site (MASW-12). Piezocone penetration tests and multichannel analysis of surface waves (MASW) were conducted at the three locations. The Standard Penetration Test (SPT, Figure 2) were carried out according to Brazilian standards NBR 6484 (2020) and NBR 6502 (1995).

At the study site, the water table ranged from approximately 3 to 8 meters deep. Figure summarizes the Nspt index properties of the residual soil layer, up to around 50 meters deep. Figure 3 shows the different layers of soil in the area. The top layer is made up of Soil 1(CL), which is Laterite Colluvium. It has a reddish-brown color and contains clayey silt with granules and fragments of laterite. Some parts of it are fine clay or silty sand. Below Soil 1, there are layers of residual soil with low resistance. Soil 2 (SR-IM) is Soft to Very Soft Residual Soil of Manganiferous or Dolomite Itabirite. It is a soft dark gray-brown sand with yellow portions, low clay content, and reacts strongly to hydrogen peroxide. Soil 3 (SR-m-IM) is Residual or Saprolitic Soil of Manganiferous or Dolomitic Itabirite. It is a dark gray-brown clay sand that also reacts strongly to hydrogen peroxide. Once you go deeper than 50 meters, you will reach the most stable and strong soil, which is Soil 4 (SP-IM). This is Manganiferous or Dolomite Itabirite Saprolite, which has a clayey silt texture, is slightly sandy (fine sand), and has a gray-brown color. It has a stiff consistency and reacts strongly to hydrogen peroxide.

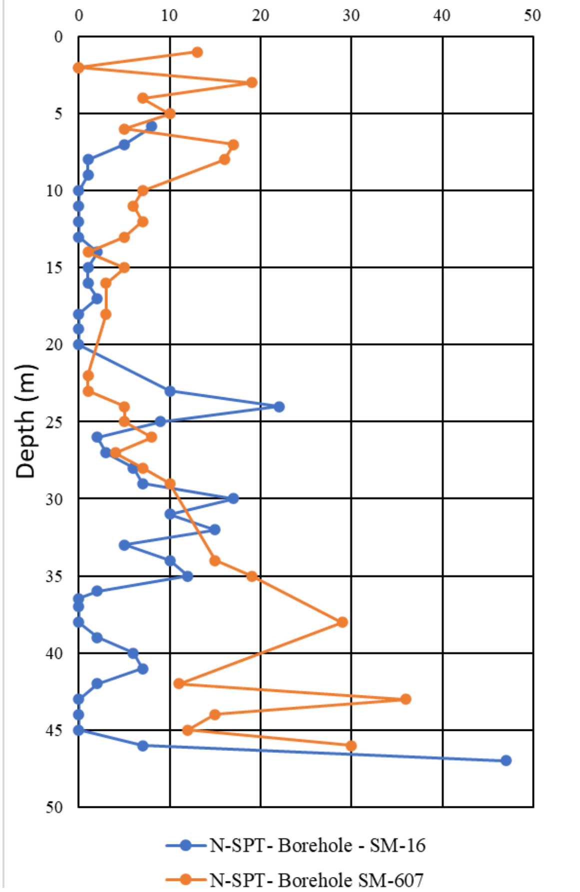

Figure 2- Standard Penetration Tests (SPTs), borehole SM-16 and SM-607

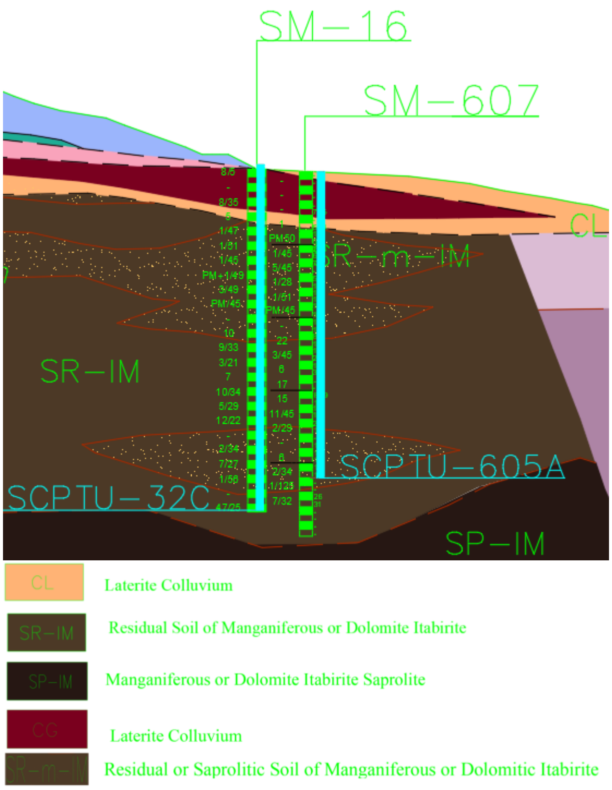

Figure 3- Investigation profiles, SCPTu32C / SCPTu-605A and boreholes SM-16 and SM-607 with N-SPT.

The Cone Penetration Test (CPT) is a valuable technique for evaluating soil properties. It involves measuring both tip and side resistances on a probe that is inserted into the soil (Lunne et al., 1997). The Seismic-CPT (SCPT) test is a modified version of the CPT that allows for the measurement of shear wave velocities in a downhole testing setup (Campanella et al., 1986). Seismic energy is produced near the insertion point of the cone by using a horizontal impact on an embedded anvil to create horizontally polarized waves. The travel times of the shear wave energy, either direct or interval, are measured at one or more locations above the cone tip. After testing at one depth, the cone is pushed further into the soil, and the test is repeated. This method is advantageous because the seismic data can be combined with the cone resistance values to gain a better understanding of soil type, strength, stiffness, and layering. Stokoe & Santamarina (2000) suggest that this is an excellent example of using multiple techniques to investigate sites.

The Cone Penetration Test (shown in Figure 4) has revealed that downstream of the structure, even at significant depths, there are low-strength silty sand to sandy silt and clean sand to silty sand materials. To measure Soil Vs values directly at 40 different points, the Seismic Cone Penetration test with pore pressure measurements (SCPTu) was also conducted. This test measures the velocity of shear waves propagating through the soil, and soil layers were measured approximately every 1 meter. It was observed that the stiffness of the residual soil layer is approximately 140 m/s.

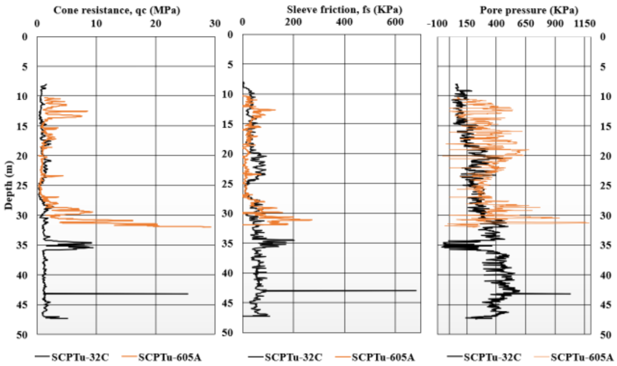

Figure 4- Cone resistance, sleeve friction and pore pressure- CPT data (SCPTu-32C and SCPTu-605A)

The MASW is a seismic technique developed by (Park et al., 1999). The primary objective of this method is to determine the variation in the propagation speed of S-waves by recording ground or surface waves. The MASW procedure involves analysing surface waves to determine the variation in the speed of S-waves with depth. Creating a dispersion image from surface waves can provide a model of S-wave speed variation, which can be achieved through either 1D or 2D MASW procedures. S-waves can be used to calculate the small-stiffness shear modulus and are directly related to the overall material's stiffness. Therefore, it is a crucial parameter for foundation engineering. There are two procedures for data acquisition: active and passive. In active MASW, a sledgehammer is typically used as the energy source, and the maximum depth of investigation is usually 30 meters.

On the other hand, passive MASW uses ambient noise, both artificial and natural, as the energy source, and the depth range can reach a few hundred meters. Active MASW can be 1D or 2D, while passive MASW can only be 1D. A step-by-step guide to acquiring and processing 1D MASW data is available in (Park et al., 2007).

The passive mode MASW method was used to investigate sixteen points, named MASW_1 to MASW_16. Data was acquired using a GEODE seismograph and 4.5 Hz geophones in a linear arrangement. The collected data was then processed using ParkSeis software, which Choon Park developed. The results of the 1D modeling at each point are presented in Table 1 and Figures 5-6, which show the Vs values of the layers. At this study stage, we will solely evaluate data from point MASW_12.

These results, illustrated in Table 1, determine the rock velocity transition. The 1D model suggests that the shift from slightly decomposed to healthy rock, with a Vs value of 2047 m/s, would occur at a depth of 60 meters (Layer 6 of Model 1D). The Vs 30 value calculated is 298 m/s, corresponding to Class C of the European Code.

Table 1- Modelo 1D do Ponto MASW-12

Layer	Depth(m)	Thck(m)	Vs (m/s)
1	7,364	7,364	296,85
2	16,568	9,204	300,8
3	27,808	11,24	276,89
4	42,456	14,648	474,23
5	60,434	17,978	591,98
6	82,906	22,472	2047,69
7	110,996	28,09	2056,15
8	146,109	35,113	2056,15

3. Discussion

In addition to the data from the 2 SCPTu (SCPTu-32C and SCPTu-605A, Figure), using CPT data, two empirical correlations were used to obtain the velocity of shear waves.

Below are the relationships used:

- Mayne and Rix (1995), compiled data from 31 different clay sites (is valid for intact and fissured clays):

$$Vs = 1.75\, qc^{0.627} \quad (1)$$

where qc (cone tip resistance) is expressed in kPa and the shear wave velocity (Vs) in m/s.

- Hegazy and Mayne (1995), expressed the shear wave velocity in terms of the cone tip resistance (qt in Kpa) and friction ration ($Rf = (100\, f_s/q_t)$ in percent):
- This expression is valid for a variety of different soil types (clays, silts, sands, and mixed soils):

$$Vs = (10.1 \log q_c - 11.4)^{1,67}(R_f)^{0.3} \quad (2)$$

- This expression is valid for clays:

$$Vs = 3.18\, q_c^{\ 0.549} f_s^{\ 0.025} \quad (3)$$

It should be noted that corrected cone resistance values q_t were used, not q_c.

The results of applying the empirical concepts and Vs values measured by SCPTu and MASW-12, are reported in Fig. 5 (SCPTu-32C) and Fig. 6 (SCPTu-605A).

To facilitate a better understanding of the data and due to the number of points sampled through the standard CPT data obtained from the SCPTu-32C, the Vs values were averaged at each meter for the three empirical relationships (Figure 7).

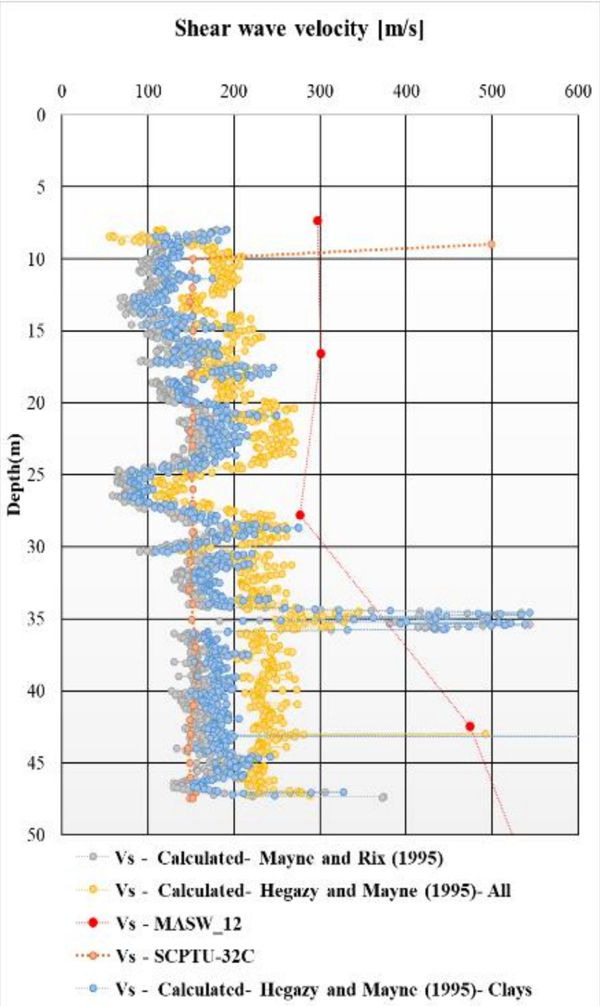

Figure 5- Shear wave velocity : empirical relationships, SCPTu-32C and MASW-12.

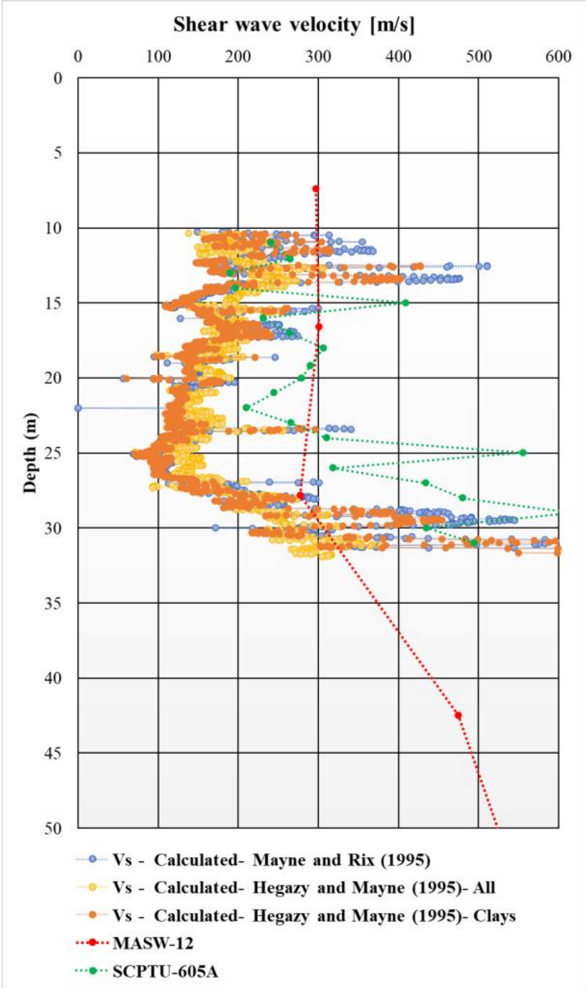

Figure 6- Shear wave velocity : empirical relationships, SCPTu-605A and MASW-12.

Through the 5 and 6, it can be observed that:

- The values obtained directly by SCPTu-32C and MASW-12 differ approximately 100% for similar depths;
- For empirically calculated Vs, Fig. 5 and 7 show that the formula obtained by Mayne and Rix (1995) has the best fit to the SCPTu measurements.
- The empirical relationship of Hegazy and Mayne (1995) for Clay deviates a little further from the SCPTu. However, their results are closer to MASW-12 (Figure 7);
- The Vs of MASW-12 and SCPTu-605A near to 17 and 18 meters have very similar orders of magnitude (Figure 6);
- The data from the three empirical relationships are close to the data measured in the field by MASW-12 at approximately 28 m, while the data from SCPTu-605A are distant (Figure 6).

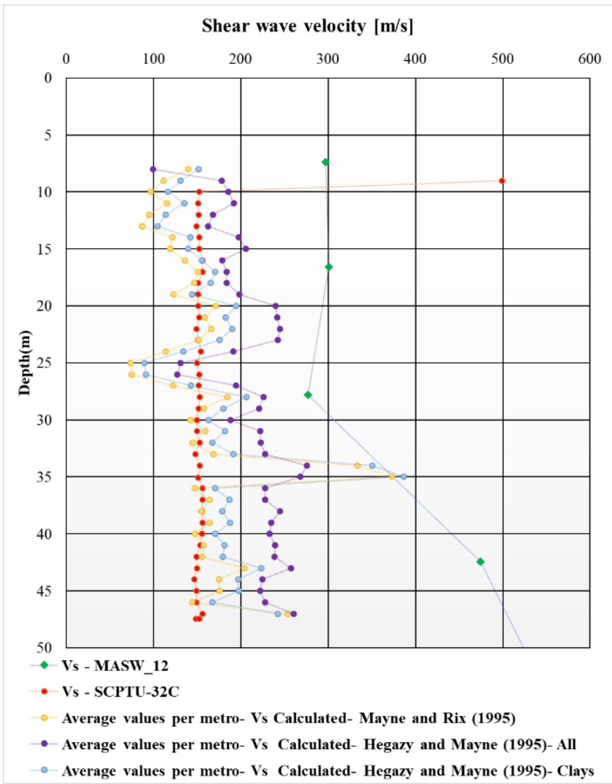

Figure 7- Average of Vs values at each meter for the three empirical relationships, SCPTu-32C and MASW-12.

For the SM-607 borehole, it is clear that the Vs values obtained by SCPTu-605A have similar trends with the N-SPT (Figure 2) values (at 15 m, 26 m, 29 m are some examples of increase).

For hole SM-16 at 35 meters, the N-SPT, qc, fs, show increases in the measured values. Consequently, empirical correlations also show this increase, however SCPTu-32C does not.

4. Conclusions

The study analysed the strength and stiffness of residual soil from foundation of a tailings dam, located at Minas Gerais, Brazil using direct and indirect methods. It was found that there was a not good correlation between the resistance measured in penetration tests and the soil stiffness measured directly through geophysical methods (MASW).

The empirical correlation proposed by Mayne and Rix (1995) between the tip resistance of the CPTu test and the shear wave propagation speed Vs was found to be suitable for this study. However, on average, the correlation overestimated the value by 10%.

The MASW method was able to predict the contact between the soil and the rocky top, but the Vs values estimated for the soil layer were overestimated by more than 100%. This indicates that the method needs improvements to determine soil stiffness accurately.

Park (2007) demonstrates the comprehensive acquisition and processing of MASW data. It is evident that in these surveys, the obtained data may be subject to lateral influences. Conversely, in the SCPTu method, information is gathered in a highly localized manner, along a vertical profile, thus experiencing significantly less lateral influence. Furthermore, depending on the distance between the source and the sensor, the data are highly discretized, enabling measurements at regular intervals, potentially as fine as every meter.

MASW data are acquired through an inversion process applied to field survey data. The field data are collected in both the spatial and temporal domains, as well as in the spatial and frequency domains. Subsequently, a dispersion curve depicting the relationship between frequency and phase speed is derived. This dispersion curve is then subject to adjustment by the processor and serves as the initial basis for the inversion process. It is important to note that this process entails uncertainties, which may not pose a significant issue when discerning contrasts between different materials and conditions. However, when aiming for precise results (e.g., on a meter-by-meter basis), similar to those obtained directly in the field, discrepancies may arise.

Additional investigations employing a more comprehensive section mesh are anticipated to elucidate the presence of thin or soft soil layers. It is imperative to emphasize that these findings represent preliminary data within a comprehensive research program aimed at enhancing the comprehension of dam foundation behavior in Brazil.

Acknowledgements

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References

Associação Brasileira de Normas Técnicas – ABNT. (2020). NBR 6484. Solo – Sondagem de simples reconhecimento com SPT – Método de ensaio. Brazil

Associação Brasileira de Normas Técnicas – ABNT. (1995). NBR 6502. Rochas e Solos. Brazil

Campanella, R. G., Robertson, P. K., and Gillespie, D. G., 1986, "Seismic Cone Penetration Test," Use of In Situ Tests in Geotechnical Engineering, ASCE GSP 6, ASCE, New York, pp. 116-130

Hegazy, A.Y., Mayne W.P. "Statistical correlations between VS and cone penetration data for different soil types", In: Proceedings of International Symposium on Cone Penetration Testing, CPT '95, Linkoping, Sweden, Vol. 2, pp. 173–178, 1995

Mayne, W.P., Rix, J.G. (1995). Correlations between shear wave velocity and cone tip resistance in natural clay. Soils and Foundations, 35(2), pp. 107–110. https://doi.org/10.3208/sandf1972.35.2_107

Lunne, T. A., Robertson, P. K. J., & Powell, J. J. M. (1997). Cone-penetration testing in geotechnical practice. Soil Mechanics and Foundation Engineering, 46, 237. https://api.semanticscholar.org/CorpusID:107190834

Olivier, G., De Wit, T., Brenguier, F., Bezuidenhout, L., & Kunjwa, T. (2018). Ambient noise Love wave tomography at a gold mine tailings storage facility. Geotechnique Letters, 8(3), 178–182. https://doi.org/10.1680/jgele.18.00016

Park, C. B., Miller, R. D., & Xia, J. (1999). Multichannel analysis of surface waves. GEOPHYSICS, 64(3), 800–808. https://doi.org/10.1190/1.1444590

Park, C. B., Miller, R. D., Xia, J., & Ivanov, J. (2007). Multichannel analysis of surface waves (MASW)—

active and passive methods. The Leading Edge, 26(1), 60–64. https://doi.org/10.1190/1.2431832

Silva, J. P. de S., Cacciari, P. P., Ribeiro, L. F., & Jefferies, M. (2022). Influence of compaction on small-strain shear modulus of iron ore tailings. Proceedings of the Institution of Civil Engineers: Geotechnical Engineering, 175(2), 247–260. https://doi.org/10.1680/jgeen.21.00036

Stokoe, K. H., & Santamarina, J. C. (2000). Seismic-Wave-Based Testing In Geotechnical Engineering. All Days, ISRM-IS-2000-038.