Field Site Characterization of a Coastal Aeolian Sandy Subgrade for a Proposed Tank Farm Site in Peru

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ABSTRACT

Analysis of shallow foundations, pavements, and slabs-on-grade requires adequate site characterization of the near-surface soils. Key properties required for the above are the soil stiffness and its nonlinearity. In this paper, we present the results of field site characterization at a coastal aeolian sand site located in Villa El Salvador, Peru, where large storage tanks are being proposed to be supported on concrete slabs-on-grade and shallow ring foundations. The site characterization included standard penetration tests (SPT), dynamic cone penetrometer (DCP) tests, lightweight deflectometer (LWD) testing, and geophysical tests such as multichannel analysis of surface waves (MASW), seismic refraction, and electrical resistivity. The paper focuses on estimating elastic soil stiffness (E_s) values on the main bearing layer of the tanks. The observed large variability of E_s estimates was associated with inherent soil variability at the site, the in-situ test method, and the use of empirical correlations between E_s and different in-situ tests. Good elastic soil stiffness estimates for the bearing layer (Upper SP sand) were required to make reliable estimates of absolute and differential settlements for the tanks. Due to soil nonlinearity, the elastic stiffness values from the different methods decreased with increasing levels of shear strains associated with each test. Accordingly, the highest elastic stiffness values were from the MASW, followed by the LWD, and the lowest from SPT-based correlations. The soil stiffness estimates using SPT-based correlations yielded the highest variability due to the high uncertainty and low confidence of the empirical correlations between E_s and SPT field values.

Keywords: Aeolian sand, SPT, DCP, LWD, MASW, elastic soil stiffness.

1. Introduction

This paper describes the site characterization efforts performed for a proposed petroleum storage farm tank at a coastal site located about 25 kilometres south of Lima, Peru. The hazardous nature of petroleum products in large cylindrical storage tanks makes settlement and differential settlement a major design consideration, as these can cause tank damage and lead to possible leaks (Bell and Iwakiri 1980, Chen et al. 1987). Thus, the foundation analyses and design for this project required meeting stringent absolute and differential settlement requirements. A detailed site investigation was performed to obtain a reliable estimation of tank settlements, as described in this paper. The paper is divided into 6 sections following this introduction as follows: i) project description and site geology, ii) geotechnical site characterization and representative site profile, iii) SPT-based soil stiffness values, iv) soil stiffness values from alternative test methods, v) comparison and discussion, and vi) conclusions.

2. Project description and site geology

2.1. Description and design requirements

The proposed tank farm is in the coastal community of Villa El Salvador, which is located approximately 25 kilometres south of Lima, Peru. A general location map is shown in Fig. 1.

Figure 1. General location map of the project site.

The site is approximately 200 by 220 m, without vegetation cover, and not developed. The topography is predominantly flat, sloping from east to west, towards Conchan Beach, with slope angles ranging from 0 to 15 degrees, as shown in Fig. 2.

Figure 2. Photo of the proposed tank farm site looking West.

The proposed tank farm will include 12 tanks with diameters ranging from 4.0 to 30.0 m and heights from 8.0 to 23.0 m. The cylindrical storage tanks will hold large volumes of petroleum; thus, spills and leakages of this potentially hazardous content were a major concern.

Based on tank dimensions and content, the estimated applied pressures for tanks filled to full capacity and for the two foundation systems considered of reinforced concrete slab-on-grade and shallow ring foundations ranged approximately between 70 and 220 kPa. The specified design requirements were bearing capacity, a minimum global factor of safety for foundation bearing capacity for the tank at full capacity of 3.0, an allowable absolute settlement of 50 mm, and a maximum edge-to-centre distortion of 1/500.

2.2. Site geology

The site is between the shoreline of the Pacific Ocean and to the east of the “Lomo de Corvina” hill. According to the Peruvian Geology maps, and Palacios et al. (1992), at Lomo Corvina the surficial deposits are predominantly aeolian sands that rest over old Lima alluvial deposits. These wind-deposited sands form the large Lomo Corvina sand dune aged Pleistocene-Holocene. The sands in this dune present a massive porous structure, with a skewed stratification and total thicknesses exceeding 15 meters. Nuñez and Villacorta (2011) found that the Lomo de Corvina dune is considered a fossil dune where the aeolian sands have been permeated with calcium carbonate and salt, resulting in a cemented soil skeleton structure. The Lomo de Corvina fossil dune has zones with dense to very dense cemented sands with high shear strength as corroborated by the presence of several steep scarps and slopes. A 3D view of the site showing some steep scarps and excavations along the Lomo Corvina can be seen in Fig. 3.

Figure 3. 3D image (looking NE) of the project site. The extent of the site is indicated with a red polygon. North side of the site along the toe of Lomo Corvina (Google Earth image).

3. Geotechnical site characterization

The field site investigation program for this project involved: 8 test trenches, 21 geotechnical boreholes using wash boring, 14 geotechnical boring using diamond drilling (due to the presence of a very dense sand layer at 3 to 4 m depths). The geotechnical borehole component included primarily Standard Penetration Testing (SPT) testing using a tripod and a donut hammer with a measured energy efficiency of 59%. The geotechnical drilling, SPT testing, and sampling were complemented with geophysical testing that included 5 seismic refraction surveys, 5 multichannel analyses of surface waves (MASW), and 2 electrical resistivity surveys. Finally, to obtain additional soil stiffness information for the foundation design lightweight deflectometer (LWD) tests were performed at the locations of the tank foundations. At the locations of the LWD tests a Peruvian adaptation of the dynamic cone penetrometer (DCP) (INDECOPI 2001) was performed to detect the depth of the upper SP sand layer that will be described in Subsection 3.2.

The general stratigraphy found at the tank farm site is described below. However, a more detailed description and discussion of the area of Tank TK-12 will follow and will be the focus of this paper.

3.1. General geotechnical site conditions

Based on the results of the field site characterization program, in general the geotechnical conditions at the project site consisted of a fill, over a medium dense to dense sand, that was underlain by a dense to very dense saturated sand that extended to the final depth investigated of about 20 m. A schematic generalized geotechnical profile for the project is shown in Fig. 4.

The fill layer, shown in Fig. 4, is an uncontrolled anthropogenic fill composed of sands, gravels, construction debris, and waste such as bottles and plastic. Its average thickness is 1 m. The foundation recommendations require the total removal of this fill layer as part of the site preparation.

Below the fill layer, a layer of fine sand was encountered that had variable thicknesses between 2 and 5 m, with an average thickness of 4 m. The fine sand is dark greyish brown with some non-plastic fines and a unified soil classification system of SP. The sand was dry to slightly moist, with the moisture content increasing with depth. This layer is referred to as the Upper SP layer. A total of 174 SPT tests were collected in the upper SP layer from 29 borings. The SPT values in this layer are summarized in Fig. 5. This plot shows the field SPT values (N_{59}) in the plot to the left, and the values corrected for hammer energy, rod length, and overburden correction ($N_{1,59}$) are shown to the right. As shown in these plots the SPT values had a large variability with field SPT blow counts ranging from 9 to 50, with an overall average value of 38.1, and a standard deviation of 13.2. Each plot in Fig. 5 shows a thick red line representing the average in the upper SP layer for five sublayers with a thickness of 1 m. The plots also show two dashed red lines representing the average values per depth plus and minus one standard deviation.

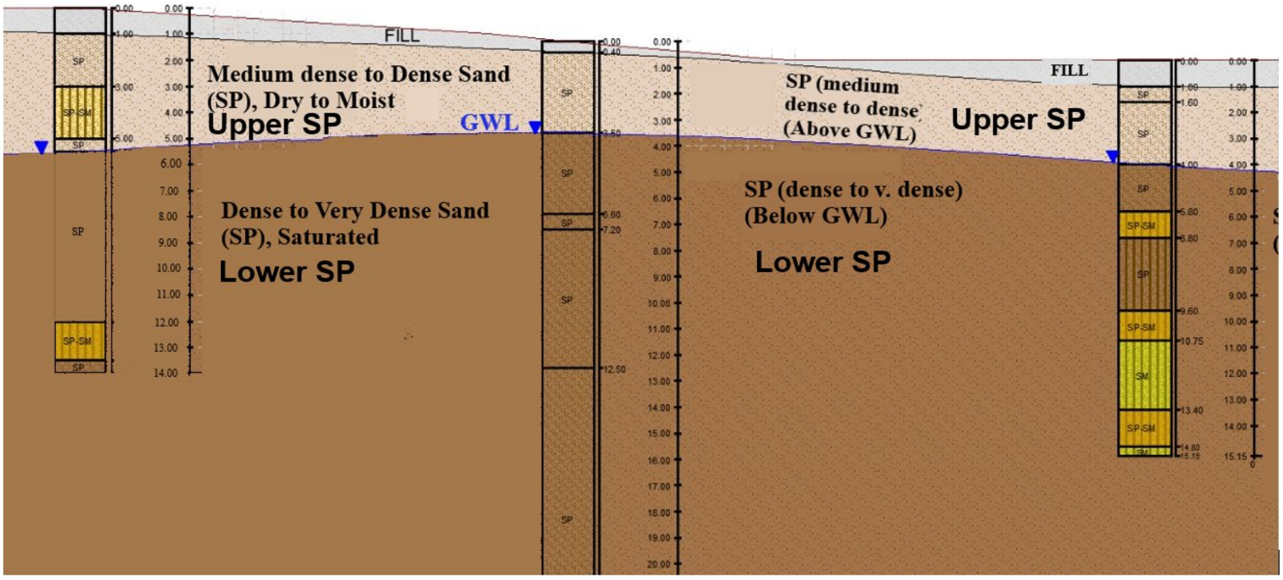

Figure 4. Generalized geotechnical profile at the tank farm site.

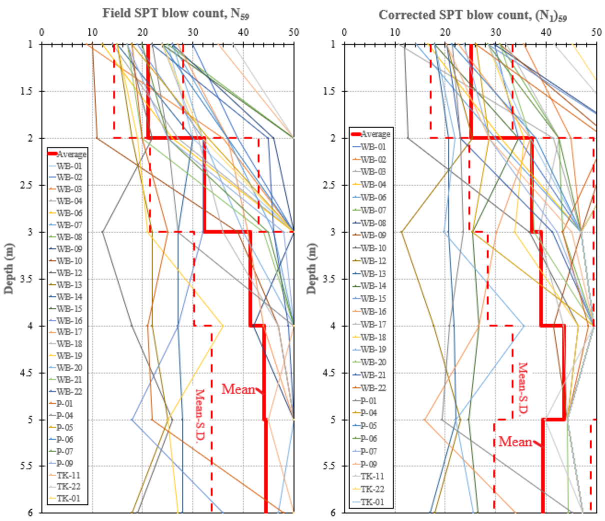

Figure 5. Summary of SPT test results in the Upper SP layer.

A summary of the SPT data for the five sublayers considered for the upper SP layer is presented in Table 1. Based on the SPT data summarized in Table 1, and correlations with relative density in McGregor and Duncan (1998), the first two sublayers of the Upper SP have predominantly a relative density of medium dense, and the bottom three sublayers are predominantly dense, but with some low SPT blow counts falling in the category of medium dense. This is common with the variability of the relative density of coastal sand deposits.

Table 1. SPT Data for sublayers of the Upper SP layer

Depth (m)	Field SPT - N_{59}					Corrected SPT - $N_{1,59}$				
	n	Min	Max	Average	St. Dev.	n	Min	Max	Average	St. Dev.
1.0-2.0	29	9	38	21.2	6.9	29	11	45	25.2	8.2
2.0-3.0	29	11	50	32.2	10.7	29	13	58	37.1	12.3
3.0-4.0	29	12	50	41.4	11.2	29	11	47	38.9	10.6
4.0-5.0	29	18	50	44.1	10.4	29	18	49	43.6	10.3
5.0-6.0	29	18	50	44.4	10.8	29	16	44	39.3	9.5

The upper SP layer was underlain by the lower SP layer, which extended to the final depth investigated. This layer was below the groundwater table and consisted of fine sand with SPT blow counts exceeding 50 blows per foot, corresponding to a very dense relative density.

The average shear wave velocity in the upper 30 meters (Vs_{30m}) obtained from MASW testing within the tank farm footprint was 488.8 m/s with a standard deviation of 49.5 m/s. Based on this average (Vs_{30m}) the International Building Code (ICC 2015) seismic site class for the site was found to be Class C.

3.2. In-situ testing at Tank TK-12

The main consideration in the tank foundation design was the settlement due to elastic compression of the upper SP fine sand layer. For the sake of brevity, this paper describes the geotechnical conditions at the location of one of the storage tanks (Tank TK-12).

Given the importance of the oil storage tanks and the consequences of possible oil spillage or leakage, an expanded site investigation was performed within the tank footprints. The locations of the different site investigation strategies performed at Tank TK-12 are shown in Fig. 6.

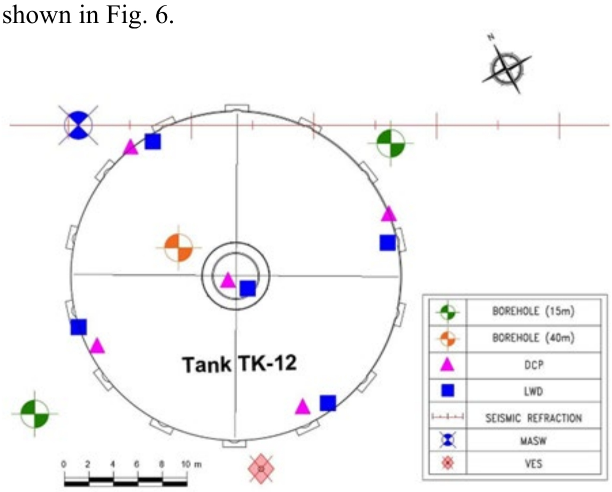

Figure 6. Site investigation performed at Tank TK-12.

Fig. 6 shows that the site investigation in the area of Tank TK-12 involved three boreholes with SPT tests, one MASW, one seismic refraction, five DCP soundings, five LWD, and one vertical electrical sounding (VES). The dynamic cone penetration (DCP) tests followed the Peruvian Technical Standard NTP 339.159-2001 (INDECOPI, 2001). The DCP test results at this tank location are summarized in Fig. 7 in terms of blows per 0.1 m. Based on the SPT test results in the 3 boreholes and the 5 DCP tests, the thickness of the upper SP layer at the location of Tank TK-12 was estimated to be 1 meter that extended from a depth of about 1 m to a depth of 2 m.

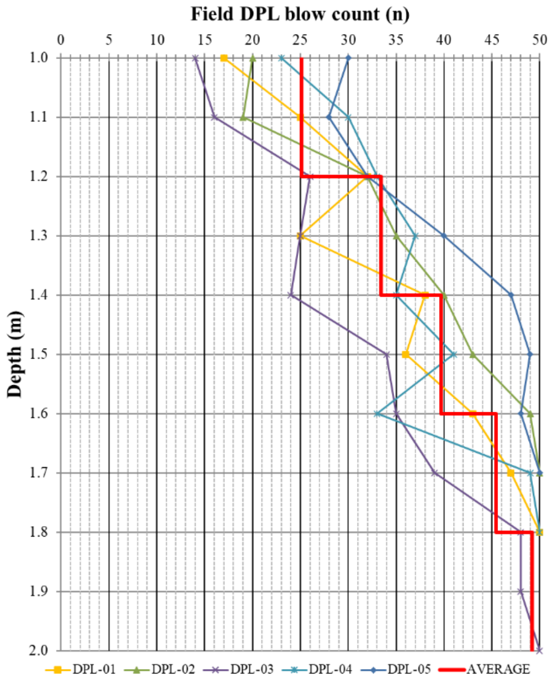

Figure 7. DCP test results at Tank TK-12

The field SPT N_{59} values measured in the area of Tank TK-12 for the Upper SP layer ranged between 17 and 47, with an average and standard deviation of 34.1 and 10.6, respectively.

4. SPT-based soil stiffness estimates for the Upper SP layer

Due to the complexity of sampling sands without disturbance, the estimation of the soil stiffness (E_s) is often done based on correlations with in-situ tests such as the SPT. There have been many correlations between the SPT and the secant soil stiffness (E_s). Mitchell and Gardner (1975) warned about the challenge to reliably estimate the elastic soil stiffness using SPT-based correlations. This is illustrated in Fig. 8, which shows the large variability and range of E_s values for any SPT value. For example, for the overall average SPT blow count N_{60} of 38.1 computed for the upper SP layer, Fig. 9 predicts a wide range of E_s values ranging from about 12.8 to 84.5 MPa (120 to 790 tsf). This very large range of secant elastic stiffness values (E_s) for the upper SP sand layer is unsuitable for calculating tank settlements for the project.

The estimated secant soil moduli for the five sublayers of the Upper SP sand layer were also estimated using the following correlation of E_s for clean to slightly silty fine to medium sands with respect to the corrected SPT $(N_1)_{60}$ reported by the Sabatini et al. (2002):

$$E_s(MPa) = 0.7 \cdot (N_1)_{60} \qquad (1)$$

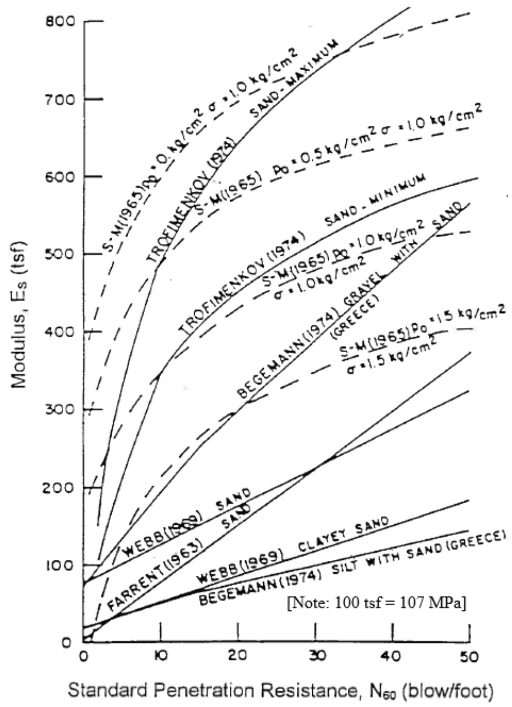

Figure 8. Correlations of secant soil stiffness (E_s) with the SPT N_{60} (McGregor & Duncan 1998; Mitchell & Gardner 1975).

Fig. 9 shows the plot of estimated secant soil stiffness (E_s) estimated using Eq. (1). The error bars show the large variation of E_s estimates using predictions based on Equation (1). The range of E_s values using this correlation is from 17.6 to 30.5 MPa. This range is not as large as the range predicted using the correlations in Fig. 8, which predict E_s values ranging from 12.8 to 84.5 MPa.

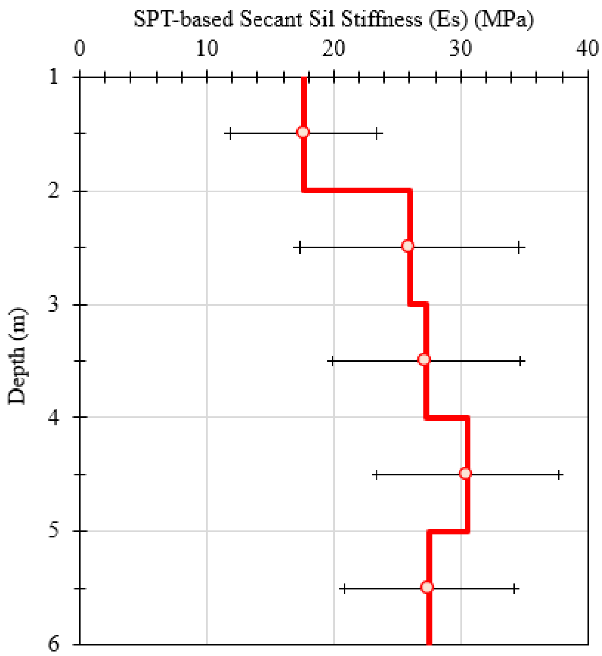

Figure 9. Estimated profile of secant soil moduli (E_s) for the Upper SP sand layer based on Equation (1).

Possible factors contributing to the very wide range of values of secant E_s values when using SPT-based correlations could be differences of SPT hammer energy, different sand characteristics, correlations developed for different ranges of depths (i.e., stress levels), consideration of different ranges of stresses (or strains) used by the different authors to obtain the reported secant soil moduli (E_s), differences in the stress history of the different sand deposits tested, etc.

5. Soil stiffness values for the Upper SP from LWD and MASW testing

As shown in the preceding section, the secant soil stiffness values (E_s) estimates using SPT-based correlations yielded a very large range of values that decreased the confidence and reliability of the tank settlement estimates. This section summarizes soil stiffness values obtained using the LWD and MASW tests. As discussed in Section 6, it is important to note that these test methods involve different volumes of soil and induce different levels of shear strain. Thus, the soil stiffness (E_s) must be used for the appropriate strain level and depth range.

5.1. LWD Test

A total of five light weight deflectometer (LWD), tests were performed in the area of tank TK-12. The LWD is a hand-portable falling weight dynamic plate load test that originated in Germany and was first developed in 1981 (White et al. 2013). The LWD device used for this project was a Danish Dynatest 3031 LWD device with a 30 cm base plate, as shown in Fig. 10. The test consisted of applying impact load pulses of increasing amplitude following a procedure in general accordance with ASTM Standard E2583 (ASTM 2007). At each test location, six tests were performed for each of the three drop masses of 10, 15, and 20 kg. For a given drop mass, the test consists of releasing the mass until it impacts a buffer system that helps lengthen the duration of the final vertical load pulse transferred to the base plate with durations ranging between 15 to 30 ms. The LWD system records load and plate displacement at the centre using a load cell and a geophone, respectively. An Elastic stiffness (E_{LWD}) can be back calculated using the applied load and vertical deflection measurements based on the Boussinesq's solution shown below for a rigid circular plate resting on an linear elastic half-space:

$$E_{LWD} = \frac{f \cdot P_{max} \cdot (1-\nu^2)}{\pi \cdot r \cdot \delta} \qquad (2)$$

where, P_{max} is the maximum load of the applied load pulse, δ is the deflection of the centre of the plate, r is the radius of the plate, ν is the Poisson's ratio of the soil tested, and f is a shape factor that is equal to 2 for a uniform pressure distribution. All variables in Eq. (2) must be entered in consistent units.

The computed LWD elastic moduli are often approximated to equivalent secant static soil stiffness values, but the user must remember that its computation involves several simplifying assumptions. More details on the back analysis procedures used to derive the LWD soil stiffness (E_{LWD}) can be found in Fleming et al. (2007) and Mooney and Miller (2009).

a) LWD Dynatest 3031 b) Photo of field LWD test

Figure 10. LWD testing at Tank TK-12.

The back-calculated LWD soil stiffness values from LWD test measurements at 5 locations (Calicata No. 1 through 5) within the Tank TK-12 are summarized in Fig. 11. This figure shows for each test location the average E_{LWD} from all tests and error bars corresponding to one standard deviation. The horizontal dashed lines correspond to the overall average and the plus and minus one standard deviation. The overall average for all LWD tests at the 5 test locations was 99.6 MPa and the standard deviation for all the data was computed as 27.7 MPa.

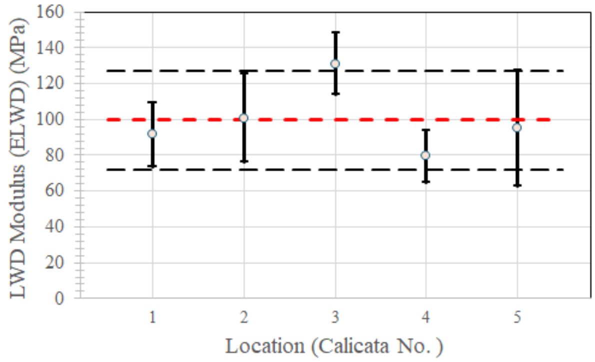

Figure 11. Summary of LWD Moduli (E_{LWD})at Tank TK-12.

5.2. Stiffness estimates from MASW testing

Shear wave velocity profiles from MASW tests for the Upper SP layer are summarized in Fig. 12. As shown in this figure, the average shear wave velocity for the Upper SP layer was 275 m/s. Based on the density of this layer, the maximum shear modulus (G_{max}) is estimated to be 139.9 MPa. For a Poisson's ratio of 0.35, the maximum Young's modulus (E_{max}) is estimated to be about 378 MPa.

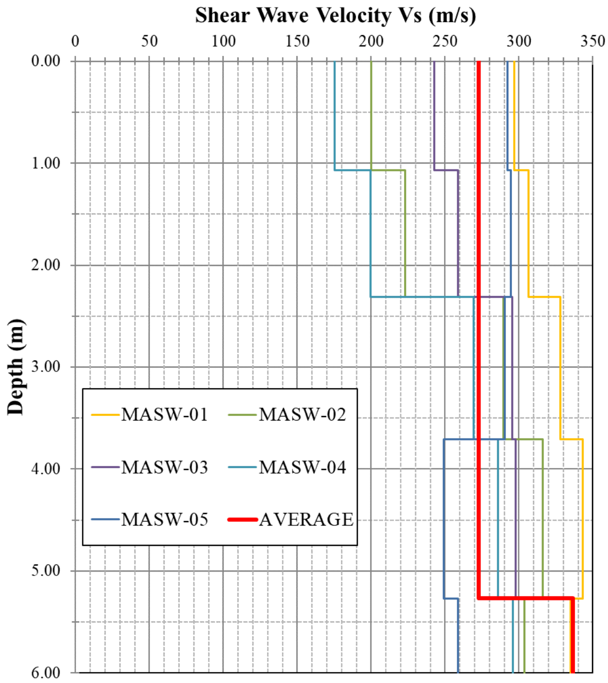

Figure 12. Shear wave velocity profiles in the project based on MASW testing (MASW-03 is located at Tank TK-12).

6. Summary of soil stiffness values

The soil stiffness of the Upper SP layer was evaluated based on field tests such as SPT, MASW, and the LDW, recognizing that each test measures the soil stiffness based on different levels of strain and volume of soil, the soil stiffness values are not directly comparable. In contrast, they should be compared using a soil stiffness degradation plot. Such a plot, summarizing all the soil stiffness values measured, or estimated for the Upper SP layer is presented in Fig. 13.

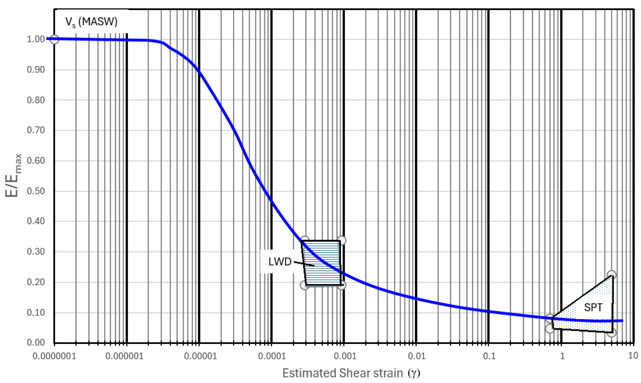

Figure 13. Normalized soil stiffness from different tests

In this plot, the MASW soil stiffness is the highest as it corresponds to a low strain stiffness (E_{max}), this value is followed by the stiffness values obtained from the LWD tests where the shear strain values are estimated based on the plate dimensions and dynamic deflection data. The lowest soils stiffness values correspond to the SPT-based stiffness values. The shear strain values for the SPT tests were estimated based on the range of penetration per blow for the measured field SPT values, and a thickness of a shear zone around the split spoon sampler. The results for each test are presented using a polygon that shows the estimated variability. As discussed before, the SPT-based E_s values have the greatest variability and level of uncertainty.

7. Conclusions

As described in this paper, an oil tank farm project at a coastal Aeolian sand site, due to the risk associated to oil spillage, the project required improving the reliability of settlement estimates for the tanks. This settlement was primarily associated with elastic compression of an upper SP layer. The paper presents the results of an expanded site investigation and in-situ testing program that included MASW, SPT, DCP, and LWD. This detailed program helped achieve a better characterization of the soil stiffness of the upper SP layer. The results from the different tests complemented each other and allowed also to estimate the soil stiffness degradation curve, i.e., E_s versus shear strain, for the Upper SP layer that was considered the main contributor to the tank settlements.

This case history highlights the importance to complement different site characterization in-situ testing techniques to assess engineering properties of complex natural deposits like the Aeolian sands near Lomo de Corvina, Peru.

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