Determination of subgrade reaction coefficient through DMT results for continuous beam foundation design

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ABSTRACT: This article describes the determination of soil-structure interaction parameters for the design of continuous beams foundations, aimed to support mobile cranes in a port terminal complex located on the Paraguay river frontside, relying on DMT results. A geotechnical campaign was undertaken at the site, where a hydraulic fill with a maximum height of 6 m lies over silty clayey soils. The followed procedure consisted in defining the soil Elastic Modulus from the obtained Oedometer Modulus, and afterwards introducing such value into a finite element model for back analysis calculations. The prediction of deformations allowed the definition of subgrade reaction coefficients. The DMT results have proven to be appropriate for the design of continuous foundation beams. In contrast to other penetrations tests, the DMT introduces less disturbance of in situ soil conditions at penetration and applies low strain levels for parameter determinations.

Keywords: dilatometer test; beam foundation; hydraulic fill; subgrade reaction coefficient

1. Introduction

The Central Department of Paraguay is one of the most populated and fast-growing areas of the country. Five of its nineteen districts are located on the Paraguay River frontside. Sudden urban and industrial expansion has encouraged the execution of several hydraulic sand fills over the highly compressible floodplain of the river as land reclamation technique. Previous geotechnical campaigns, such as the ones performed for the Asuncion's Bay Waterfront [1,2], have brought some insides on the applicability of in-situ testing techniques for the characterization of the fine silty sand from the Paraguay's riverbed, used as main hydraulic fill material. However, little information has been published for the public knowledge until now.

This paper describes the determination of elastic deformation and soil-structure interaction parameters for the design of continuous beam foundations, relying on DMT (Marchetti Dilatometer Test) results. They were aimed to support mobile cranes in a port terminal complex located in the district of Villeta, where a hydraulic fill with a maximum height of 6 m lied over silty clayey soils. The followed procedure consisted in defining the soil Elastic Modulus E_Y from the Oedometer Modulus M_{DMT}, and afterwards introducing such value into a linear elastic plane strain FEM (Finite Element Method) model for back analysis calculations. The prediction of deformations allowed the definition of subgrade reaction coefficients K_V.

Although the Winkler model [3], which introduces the concept of the subgrade reaction coefficient K_V, does not accurately represent the behavior of foundation structures, it is still widely used and preferred over more complex models. In this work, coefficients corresponding to the settlement of 30 cm square plates K_{30} are also determined, directly through correlations from in situ results. These are then transformed applying Terzaghi's considerations [4] for long beams over soil subgrades.

DPSH (Dynamic Probe Super Heavy) and CPT (Cone Penetration Test) tests were also performed on the site. A reasonable agreement among results was obtained, but parameters derived from the DMT were selected as principal input values. In contrast to other penetration tests, the DMT introduces less disturbance of in situ soil conditions at penetration and applies low strain levels for parameter determinations. Former studies, as listed in [5], have shown a sounding correspondence between DMT predicted and measured settlements.

2. Project description

The terminal port complex is located on the left margin of the Paraguay River in the district of Villeta, Central Department. A hydraulic fill has been performed on the site covering and estimated surface area of 186.000 m^2, as seen in Fig. 1. The land access is through the Villeta – Alberdi Route at km 13.

Figure 1. Aerial view of Terminal Port Complex before constructions.

Two reinforced concrete beams of 300 m length, with expansion joints every 40 m, were to be installed to support large mobile cranes. The foundation plane was at a depth D of 2 m from ground surface. The beams had a height H of 2,5 m and a width W of 2 m. Mobile cranes exerted two moving point loads of 100 kN each, separated a distance of 6 m. The disposition of each element can be observed in Fig. 2.

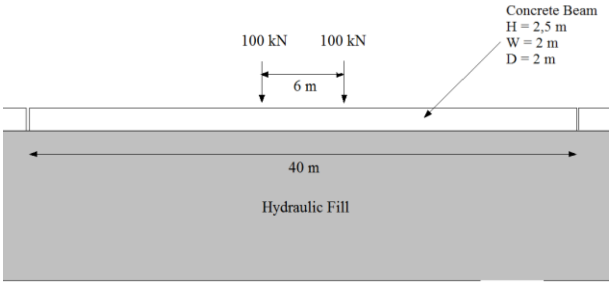


Figure 2. Schematic drawing of beams over the hydraulic fill with mobile loads exerted by cranes.

2.1. Geotechnical campaigns

The geotechnical investigation and design works were carried out in between November 2015 and June 2016. A first campaign was conducted to specifically verify the quality of the hydraulic fill. For this objective 6 CPT tests and 15 DPSH, with maximum depths of 6,2 m and 4,2 m respectively, were performed. The second campaign was related to the request of a K_V value for the design of foundation beams. In this stage, 7 DMT tests with a maximum depth of 10,6 m were executed.

The SPT (Standard Penetration Test) is still the most common in situ test practiced in the country. Only since 2010, DPSH, CPT (CPTu) and DMT tests became available and proved to be highly convenient for hydraulic fill applications. In this work, DPSH results (DP) were correlated to N_{SPT} values through an expression derived by the author [2] for fine silty sands from the Paraguay Riverbed:

$$N_{SPT} = 0{,}8\ (DP_{20} + DP_{40} + DP_{60}) \qquad (1)$$

where DP_{20}, DP_{40} and DP_{60} are the number of strikes for a penetration of 20 cm, 40 cm and 60 cm respectively.

2.2. Soil characteristics

Soil was classified according to the Normalized Soil Behavior Type [6] and Marchetti 1980 [7] for CPT and DMT results, respectively. The hydraulic fill was well identified in the thickness of 6 m, where clean sand to silty sand appeared. The sand is dense to very dense with mean relative density of 60% to 80%. Beneath the hydraulic fill, medium dense clayey silt is present. The stratigraphy has no considerable variations in the horizontal direction. The water table position varies according to the river level. At the time, it was detected at an average depth of 4,2 m from ground level.

3. FEM input parameters

The plane strain linear elastic FEM model was developed through SIGMA/W software. The model is shown in Fig. 3. and Fig. 4. The mesh has 1.624 triangular and squared elements and the beam is considered as a flexural element, formulated according to the Bernoulli beam theory. Only one 40 m long beam is considered for analysis, considering that expansion joints allow individual behavior of segments. Point loads are placed at the center of the beam.

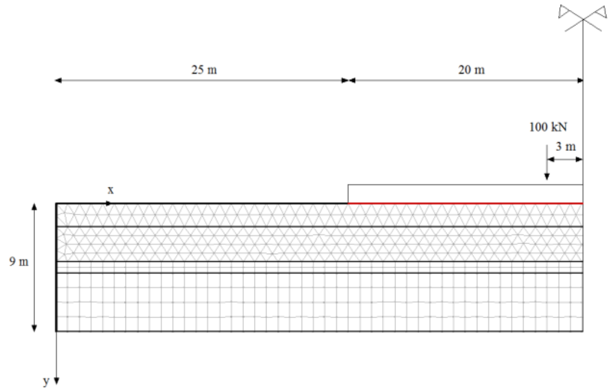


Figure 3. 2D linear elastic FEM model with triangular and squared elements.

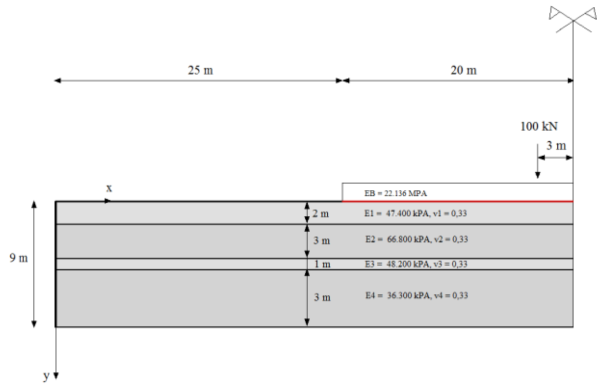


Figure 4. Chosen stratification for 2D linear elastic FEM model.

The variation of E_Y with depth was defined from the DMT parameter profile shown in Fig. 5. M_{DMT} was first calculated as indicated by [7]:

$$M_{DMT} = R_M \times E_d \qquad (2)$$

in which E_d is the Dilatometer Modulus defined as:

$$E_d = 34{,}7 \times (p_1 - p_0) \qquad (3)$$

where p_0 is the internal membrane pressure and p_1 is the pressure needed to obtain a membrane expansion of 1,1 mm. R_M values are listed in [7] as a function of the Material Index I_d and the Lateral Stress Index K_d:

$$I_d = (p_1 - p_0)/(p_0 - u_0) \qquad (4)$$

$$K_d = (p_0 - u_0)/(\sigma_v') \qquad (5)$$

u_0 is the in situ at rest pore pressure and σ_v' the vertical effective stress. E_Y was then derived through the standard elastic relationship introducing Poisson's Ratio v:

$$M_{DMT} = E_Y \times (1 - v)/((1 + v) \times (1 - 2v)) \qquad (6)$$

A constant value of $v = 0{,}33$ was selected, typical for medium to dense silty sands.

Mean values of E_Y derived from CPT and DPSH tests are shown in Fig. 6. The total cone resistance q_t was related to E_Y as suggested in [6] for uncemented silica sands. N_{SPT} values, obtained through Eq. (1), were related to E_Y using a relationship recommended by [8] for sands:

$$E_Y = 8.000 \times {N_{SPT}}^{0{,}8}\ [kPA] \qquad (7)$$

In general, a good correspondence among results is observed.

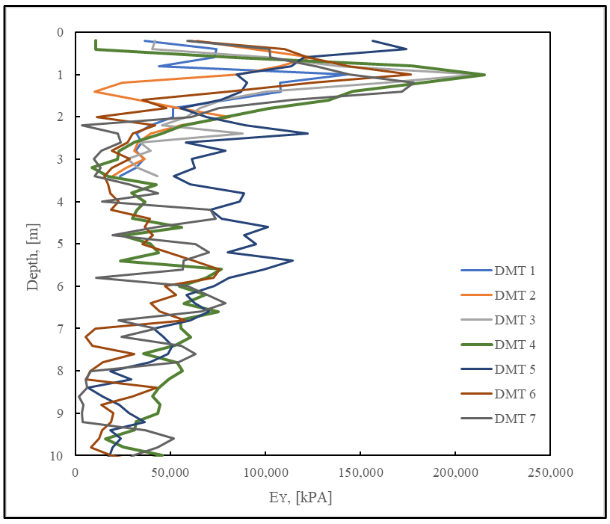


Figure 5. Young Modulus derived from DMT results.

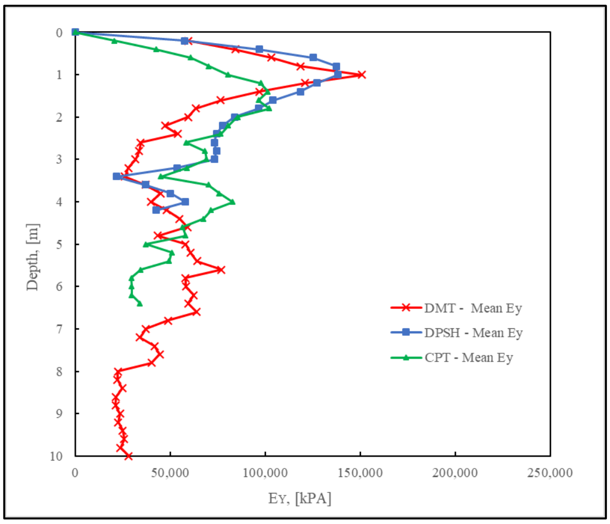


Figure 6. Average Young Modulus derived from DMT, CPT and DPSH results.

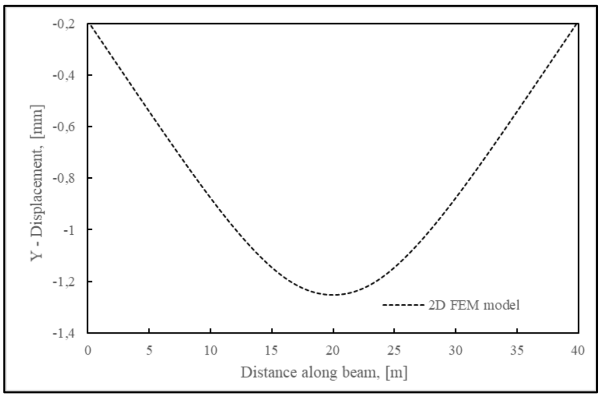
4. FEM results

The results from the 2-D FEM model are shown in Fig. 7. As already stated by [9], K_V distribution is not constant along the beam length. The rate of variation is small in almost the entire central part of the beam. On the edges K_V is 1,62 times greater than the central value of 5.960 kN/m^3.

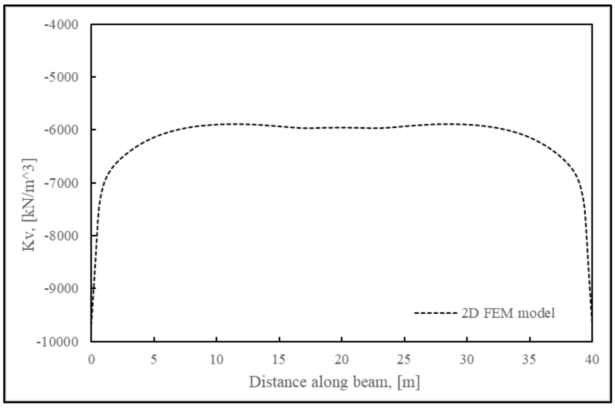
5. K_v definition

Foundation calculation methods relying on Winkler's model often include only one constant value of K_V. Adopting a constant value equal to the lowest K_V obtained in the FEM model does not necessarily result in a conservative structural design. Introducing higher K_V values on the edges results in larger structural distorsion, and hence, the possibility of larger structural efforts. Furthermore, when a constant distributed load is applied to Winkler's model with a constant K_V, no change is observed in structural efforts because of constant settlements underneath the beam.

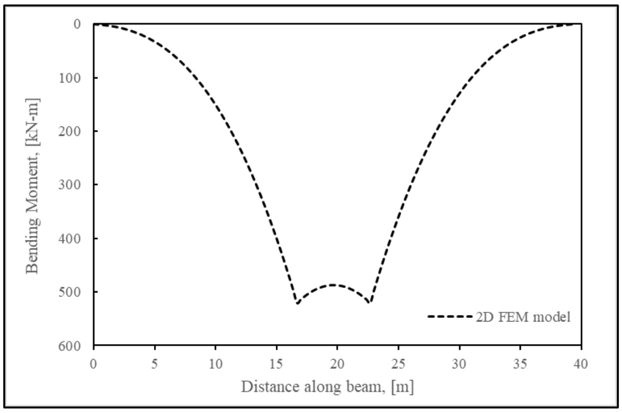
A Winkler´s model of the same beam and load configuration, considering a constant K_V, was afterwards solved. The code used for this objective was developed by [10]. A subgrade reaction coefficient of 3.200 kN/m^3 yielded comparable bending moment and shear force values as seen in Fig. 8.



(a)



(b)



(c)

Figure 7. 2D FEM results: a – vertical displacement; b – subgrade reaction coefficient; c – bending moment.

5.1. K_{30} from in situ tests

Sometimes subgrade reaction coefficients are provided from square plate load tests. These can be modified, through Terzaghi's considerations [4], for their application to long beams over soil subgrades:

$$K_{V\ plate} = K_{W1} \times 0{,}67 \times ((W + W_1)/2W)^2 \quad (8)$$

where W_1 is the dimension of the square plate, typically equal to 0,3 m.

Values of K_{30} have been recovered from the empirical expression obtained by [11] for sandy soils:

$$K_{30} = 1.800 \times N_{60}\,[kN/m^3] \quad (9)$$

relating DPSH values to N_{60} with Eq. (1). and DMT results with Eq. (10) [7]:

$$N_{60} = E_d\,[MPA]/2{,}5 \quad (10)$$

As seen in Fig. 9, at a depth of 2 m $K_{V\,plate}$ from DPSH tests is equal to 7.710 kN/m^3 and from DMT tests equal to 5.269 kN/m^3. These values are comparable to those obtained in the FEM model. However, they are considerably higher than that defined in Winkler's model with constant K_V, selected to obtain similar structural efforts to those of the computational simulation.

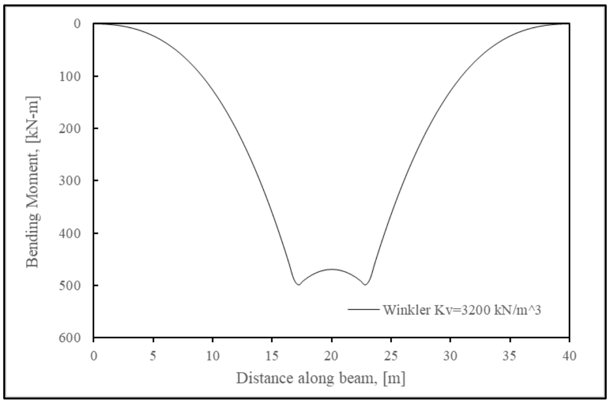


Figure 8. Winkler's model results with a constant Kv of 3.200 kN/m3:

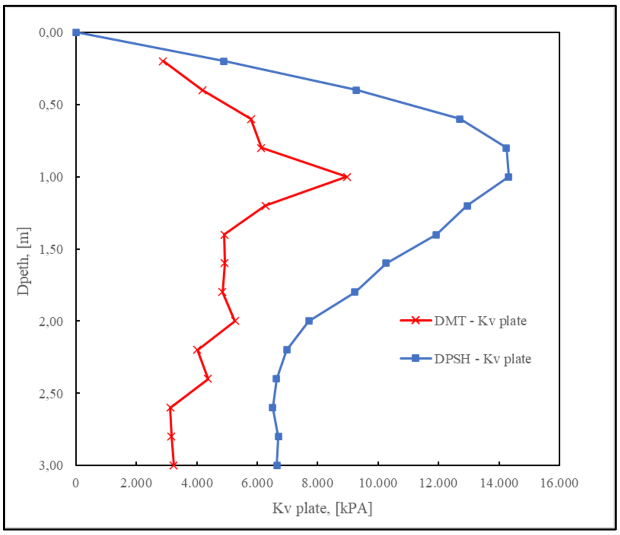


Figure 9. Kv values associated to square plate load tests obtained through Eq. (9) and modified for beams on soil subgrades.

6. Conclusions

DMT results have been used to define parameters for a 2D Linear Elastic FEM. The model had the objective of defining suitable values of K_V for the design of a continuous beam lying over a silty sand hydraulic fill. DPSH and CPT results, obtained in a previous campaign, yielded comparable results.

A non-constant K_V along the beam is recommended, with higher values on both ends of the beam. If models relying on Winkler's concept are to be used with a constant K_V, attention must be drawn to the limitations of such consideration.

7. References

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