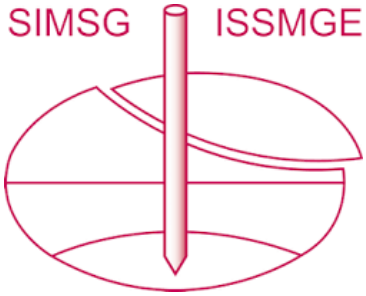
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Numerical Modeling of Piles Constructed Through Different Techniques in Sandy Soils

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Abstract

A successful numerical modelling of load-displacement curve of a pile, soil column and shallow foundation cannot be done without the parameters obtained from a proper soil characterization. However, prediction events revealed that, even with sophisticated and good number of tests might not be enough for a successful prediction. In this context, this paper presents a novel methodology for soil characterization through the numerical modelling of pressuremeter test in FLAC3D. To present the performance of the new methodology, three different piles were selected from the pile prediction event held in Bolivia in March 2017. All piles have same length of 9.5 m and were constructed in sandy and silty soils with different techniques and diameters. Results of SPT and CPT were used for stratification and obtaining initial parameters of the model. However, two pressuremeter tests were used to find the final set of parameters, which were used for numerical modelling of piles. It was shown that construction stages and interface information are very important for obtaining the load-displacement curves. Results reveal that, a calibrated numerical model through pressuremeter test, a proper constitutive model and modelling the stages and interface enable a promising and effective approach for modelling the geotechnical structures.

1 Information

Pressuremeter is known as an unique test to obtain load-displacement curve of soils in any depth and as one of the most useful site tests to determine and/or evaluate deformation characteristics of soils. In this context, the analysis and decoding of pressuremeter test curve has been a challenge for engineers and researchers for many years (Hughes *et al.* 1977, Carter *et al.* 1986, Manassero 1989, Fahey and Carter 1993, Whittle 1999, Bolton and Whittle 1999, Monnet 2012). Some of the researchers (Yin & Hicher, 2008, Frydman 2011, Likitlersuang et al. 2013, Balachandran et al. 2016) considered pressuremeter test results as beneficial for determining, controlling and/or calibrating the soil parameters.

Majority of studies in literature, which aims to simulate pressuremeter test, usually preferred to implement Mohr-Coulomb constitutive model in axisymmetric condition. The general lack of these studies is that not taking into account all the expansion and contraction phases of a pressuremeter test. Oztoprak and Bolton (2011, 2013) indicated that the degradation of shear modulus causes non-linearity before plastic yielding, and this needs to be taken into account in deformation-based design of geotechnical structures. With this respect, Oztoprak and Bolton (2011) proposed a new hyperbolic model, which was used to replace the conventional linear elastic part prior to peak strength in the FLAC3D Strain Hardening/Softening Mohr Coulomb (SHS-MC) soil model. The performance of the model was checked by the pressuremeter test data in Thanet sands. Oztoprak et al (2018) modified the Oztoprak and Bolton (2011) method to model the pressuremeter test and introduced gap, disturbance and creep parameters to obtain the complete field curve of a pressuremeter test.

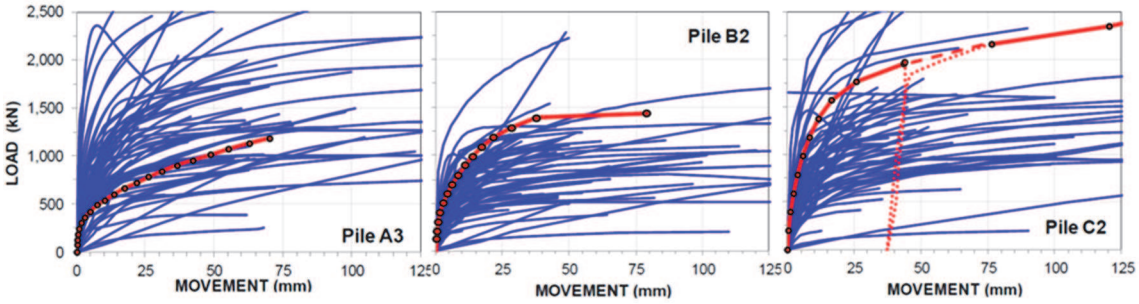


Fig. 1: All predictions (blue lines), test results (red lines) (Fellenius 2017)

In April 2017, ISSMGE TC212 in Bolivian Experimental Site organized a pile prediction event for Testing (B.E.S.T), Santa Cruz, Bolivia. The contenders were asked to send their predictions about behaviour of four types of piles which constructed by different methods. Fig. 1 compile all predicted pile-head load-movement curves of piles A3, B2 and C2 (Fellenius 2017).

In this study, the new approach of Oztoprak et al. (2018) were used for soil characterization through pressuremeter modelling. Following the designation of in-situ stress conditions, index, stiffness and strength parameters, three piles were simulated in FLAC3D including the construction stages.

2 Methodology and Numerical Analysis

2.1 Non-linear Elastic Perfectly Plastic Soil Model

A novel methodology is presented for obtaining the load displacement curves of piles. In this context, soil characterization was carried out through the numerical modelling of pressuremeter test in FLAC3D in which a new hyperbolic model was developed to replace the conventional linear elastic model prior to investigation of

the peak strength of the Strain Hardening Softening Mohr-Coulomb (SHS-MC) soil model to improve the performance in the small strain range (Fig. 2). To incorporate the new model in FLAC3D, FISH function was written to represent the reduction of shear modulus during straining, up to failure. Stress dependent shear modulus calculations were carried out through another FISH function.

For the varying shear modulus of new model Oztoprak and Bolton (2011, 2013) equations were implemented. Shear modulus degradation G/G_0 given in Eqs. (1), (2), (3) and (4). Authors were also proposed eq. (5) for calculating the maximum shear modulus.

$$\left(\frac{G}{G_o}\right) = \frac{1}{1+\left(\frac{\gamma-\gamma_e}{\gamma_r}\right)^a} \tag{1}$$

where γ_r, γ_e and a are as defined in eqs. (2) to (4):

$$\gamma_r\,(\%) = 0.01 \cdot U_c^{-0.3} \cdot \left(\frac{p'}{p_a}\right) + 0.08 \cdot e \cdot I_D \tag{2}$$

$$\gamma_e\,(\%) = 0.0002 + 0.012 \cdot \gamma_r\,(\%) \tag{3}$$

$$a = U_c^{\,-0.075} \tag{4}$$

$$G_o = \frac{5760 \cdot p_a}{(1+e)^3} \cdot \left(\frac{p'}{p_a}\right)^{0.5} \tag{5}$$

Here; γ_e = Elastic threshold strain, γ_r = Reference strain (shear strain required to reduce G/G_o to 0.5), a = Curvature parameter, U_c = Uniformity coefficient, e = void ratio, I_D = Relative density, p' = Mean effective stress, p_a = Atmospheric pressure.

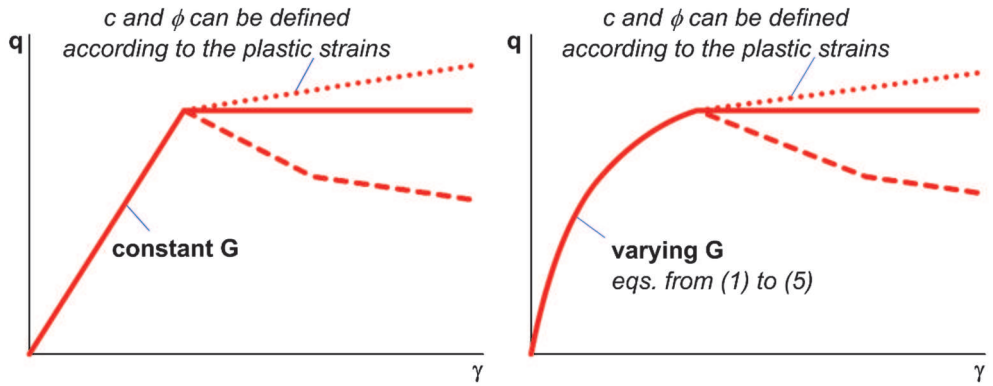


Fig. 2: Stress-strain curves of; (a) embedded strain softening/hardening Mohr-Coulomb model, (b) developed non-linear elastic, strain softening/hardening Mohr-Coulomb model

The developed model can be defined as a modified Mohr-Coulomb model when the shear strength parameters (c and/or ϕ) keep constant. However, it is possible to obtain a strain softening/hardening soil model by adjusting shear strength parameters with plastic strains. Similar to hardening soil model of Plaxis (Schanz et al 2000), stiffness parameters can be linked to reference pressure as given in eqs. (6) and (7).

$$G_o^{ref} = f \cdot \frac{5760 \cdot p_a}{(1+e)^3} \cdot \left(\frac{p^{ref}}{p_a}\right)^{0.5} \tag{6}$$

$$G_o = G_o^{ref} \cdot \left(\frac{c \cdot \cos\phi - p' \cdot \sin\phi}{c \cdot \cos\phi + p^{ref} \cdot \sin\phi}\right)^m \tag{7}$$

Here; G_o^{ref} = Initial reference shear modulus value, p^{ref} is the reference stress, p' is the corresponding mean effective stress at the achieved loading state and m is stiffness exponent. According to PLAXIS (2014) and FLAC3D (2016) it is between 0 and 1. f is the fitting coefficient.

For the numerical modeling of a pressuremeter test, evolution of G and G_o should be defined. As demonstrated in Fig. 3(a), initial shear modulus (G_o) values are automatically updated through the equations (1) and (7) at points from 1 to 7 according to each zone's mean effective stresses. The initial part of the curve of a pressuremeter test may be affected by the compliance of the instrument to the cavity (g) and disturbance due to drilling (d). The gap (g) between the membrane and cavity wall and the thickness of the disturbed soil zone (d) were included in the procedure to capture the initial curve. In modeling of a pressuremeter test, number of loops has crucial importance, since the location, inclination, and size of loops are affected by the stiffness parameters.

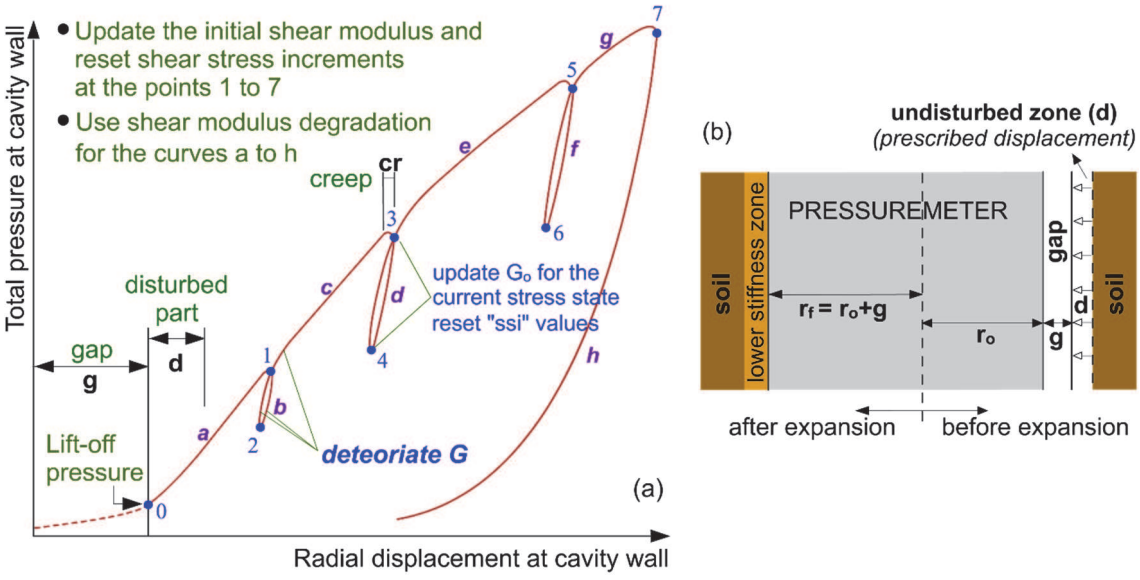


Fig. 3: Modelling steps of pressuremeter tests and definition of parameters

2.2 Exploration of Testing Site and Selected Tests

A pile prediction event was organized for the 3rd Bolivian International Conference on Deep Foundations, Santa Cruz de la Sierra, Bolivia April 27 - 29, 2017. Site investigations were performed at the Bolivian Experimental Site for Testing Piles (B.E.S.T.) employing boreholes (SPT), cone penetrometer tests (SCPTU), pressuremeter tests (PMT), and dilatometer tests (DMT). Fellenius et al. (2017) and Fellenius and Terceros (2017) provided all the information including site explorations, laboratory test results and pile construction details. Primary information can be seen in Table 1. Two pressuremeter tests exist within the length of piles (Fig. 4). The static tests of three single piles were selected for this paper. All piles have same length of 9.5 m and were constructed in sandy and silty soils with different techniques. One pile (Pile A3) was drilled with slurry, one (Pile B2) was constructed with a continuous flight auger, and one (C2) were constructed by full displacement equipment. During pouring the concrete, pressure grouting were implemented instead of gravity grouting. This information was provided after testing. Soil profile and SPT/CPT results for A3 and B2 piles are shown in Fig.4.

Tab. 1: Primary information on the test piles (Fellenius and Terceros, 2017)

Pile	Type	Diameter	Toe Augment	Test Methods Applied	Remarks
A3	Bored with bentonite slurry	620 mm	---	Head-down Test (HD)	Pressure grouted
B2	Continuous Flight Auger (CFA)	450 mm	---	Head-down Test (HD)	Pressure grouted
C2	Full Displacement Pile (FDP)	450 mm/ 350 mm	---	Head-down Test (HD)	Pressure grouted

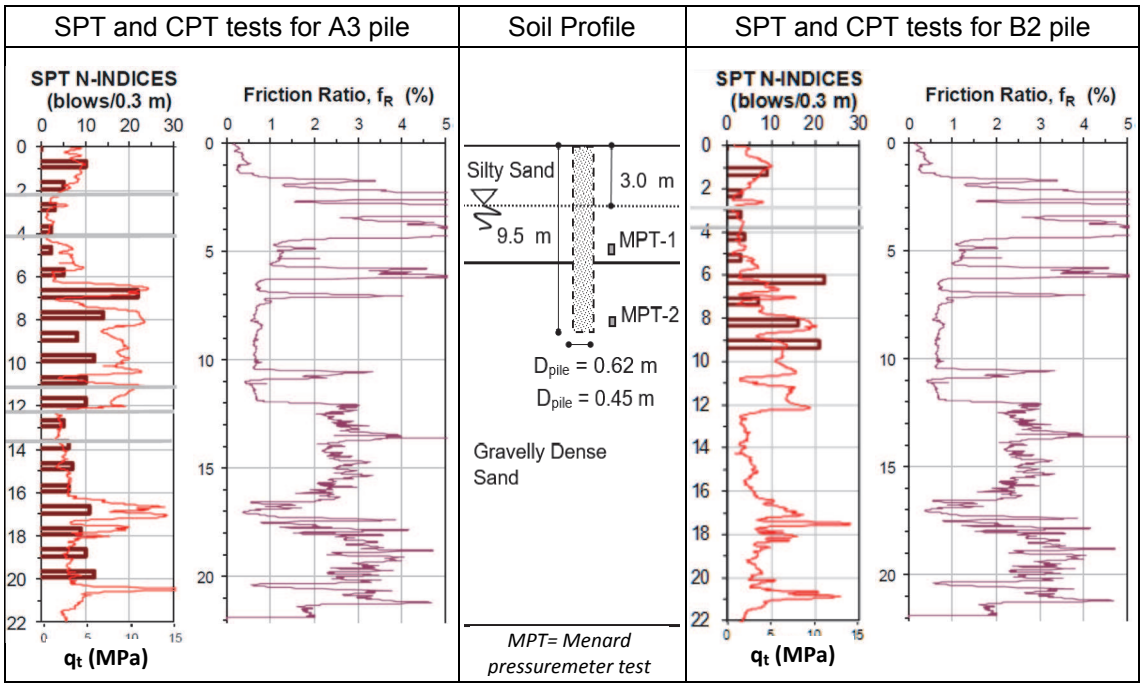


Fig. 4: Soil profile and SPT/CPT results and MPT locations for A3 and B2 piles

2.3 Back Analysis of Pressuremeter Test

To carry out modelling of pressuremeter test, the modified MC model was incorporated in FLAC3D. To enable this, a FISH function was written to represent the reduction of shear modulus during straining, up to failure. Stress dependent shear modulus calculations were carried out through another FISH function.

Two pressuremeter tests (5.0 m and 9.0 m) exist within the length of each piles (Fig. 4). Before starting the designation of parameters to capture the pressuremeter curves, some parameters should be fixed with some equations and tools. The lateral earth pressure coefficient, K_o is estimated by using 'lift-off' pressure ($p_{h,o}$) (Fig. 3). For the initial attempts for estimating ϕ, SPT N - ϕ relation of Hatanaka and Uchida (1996) was used: $\phi=(15.4 \cdot N_{1,60})^{0.5}+20$. Dilation angle ($\psi$) values were found through the equation of Bolton (1986): $\phi_p=\phi_{cv}+0.8\,\psi$. The constant volume friction angle was selected as $\phi_{cv}=32°$. Uc are defined from granulometry curve and ID can be estimated from SPT or CPT tests. These parameters can slightly be changed during capturing the pressuremeter test curves (Fig. 5). Table 2 presents the obtained parameters from the back-analysis of corresponding pressuremeter tests.

Tab. 2: Obtained parameters with back analysis of pressuremeter

Pile	Test depth (m)	γ (kN/m³)	K_0 (-)	e_0 (-)	U_C (-)	I_D (%)	f (-)	m (-)	p'ref (kPa)	$G_{0,ref}$ (MPa)	v (-)	c (kPa)	ϕ (°)	ψ (°)
A3	5.00	18.0	0.55	0.98	60	38	0.30	0.10	100	27	0.35	1.0	34.0	2.0
	9.00	18.5	0.40	0.80	30	60	0.43	0.12	100	40	0.35	1.0	44.0	13.5
B2	5.00	18.0	0.50	0.98	50	38	0.35	0.10	100	26	0.35	1.0	33.0	1.0
	9.00	18.5	0.30	0.90	30	50	0.35	0.20	100	29	0.35	1.0	39.0	9.0
C2	5.00	18.0	0.50	0.98	50	38	0.35	0.10	100	26	0.35	1.0	33.0	1.0
	9.00	18.5	0.35	0.82	20	60	0.38	0.10	100	36	0.35	1.0	40.0	9.5

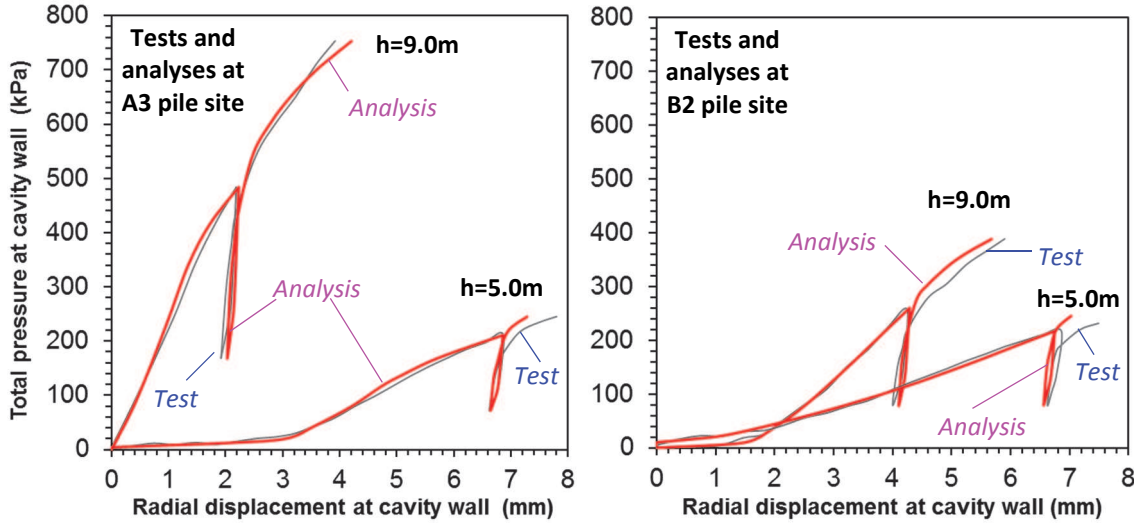


Fig. 5: Back-analyses of pressuremeter load displacement curves at A3 and B2 pile sites

2.4 Modelling the Construction Details of Piles

Construction techniques and corresponding interface formation were tried to be considered in FLAC3D modelling. According to the Fellenius and Terceros (2017), concreting of all piles were enabled through pressure grouting. This process avoided the interface necessity for the bottom of all piles. However, bentonite would cause a cake-formation at the interface of the A3 pile. Therefore, thin elements were provided during the meshing (Fig. 6). The same is valid for the full-displacement C2 pile at which driving would form a thin reconstructed, totally disturbed interface. In addition to this, a cylindrical strength-reduced zone, which was influenced from driving process, was defined with a diameter of 1.5 times of pile diameter. Strength-reduction of this zone was defined by using the work of Castro and Karstunen (2010). Auger-cast B2 pile would be affected less from the construction process. Therefore, interface zone was not defined for this pile. For the strength reduction of the surrounding zone of piles, a multiplier was defined. As seen in Fig. 6, C2 pile was formed with two diameters. After opening a 350 mm diameter hole, top 8.35 m of 9.5 m was formed by defining prescribed displacements from 350 mm to 450 mm.

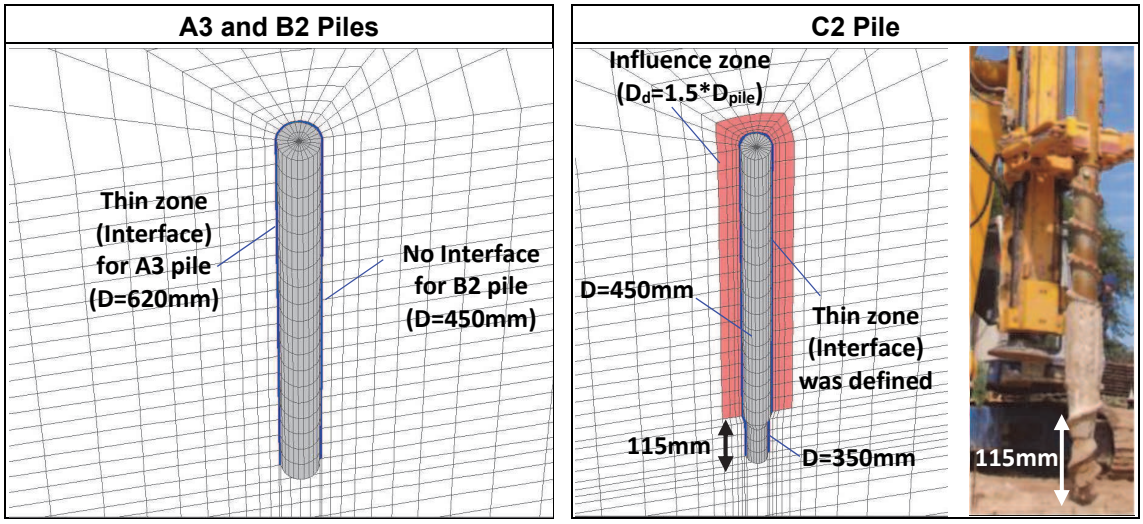


Fig. 6: 3D view of FLAC grids and information about piles

3 Results and Discussion

Three different analyses were carried out to discuss the interface and parameter effect (Table 3). Analysis-1 of each pile corresponds to obtained parameters from the modelling of two pressuremeter tests. Load displacement curve of A3 pile was captured by defining a thin zone and 0.75 multiplier for the interface.

On the opposite, no interface zone and multiplier was required for B2 pile. Even without interface, lower load values were obtained for medium strains. However, 90% of predictors were also underestimated the service load of this pile (Fig. 1).

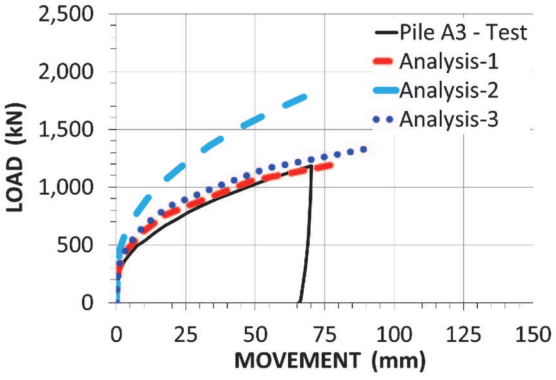
When the ϕ and ψ are increased 1°, it seems possible to capture the pre-yielding part of curve.

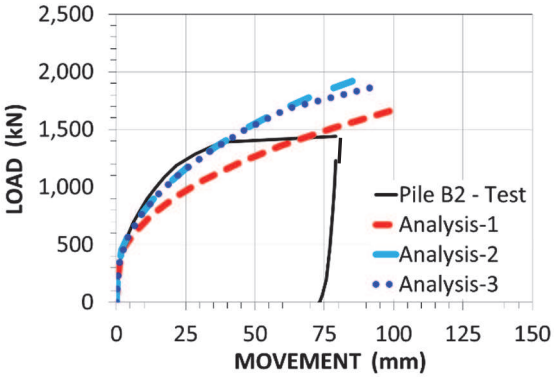
For C2 pile best prediction were obtained by using 0.85 multiplier for influence and 0.4-0.45 for interface.

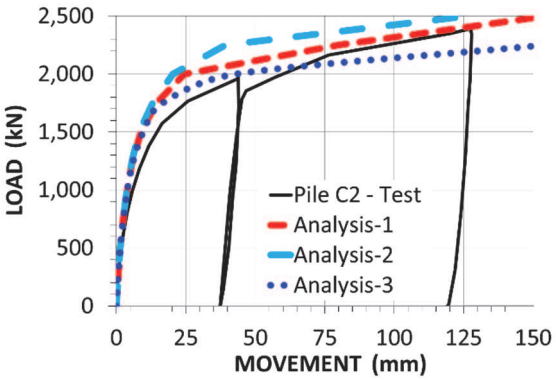
According to Eurocode, displacement value to achieve the capacity of pile is %10 of pile diameter. In this work, this value was chosen 60 mm for all piles. In addition to this, 15 mm displacement was accepted to define the serviceability load of piles. Obtained load values can be seen in Fig. 7. in order to compare the performance of analyses. Results are close to obtained values from tests.

Tab. 3: Obtained parameters with back analysis of pressuremeter

Pile	Type	Analysis	Influenced zone	Interface thickness	Interface value	Explanations
A3	Bored (With bentonite slurry)	Analysis-1	No	Thin	0.75	-
		Analysis-2	No	Thin	1.00	-
		Analysis-3	No	Thin	0.80	-
B2	Continuous Flight Auger (CFA)	Analysis-1	No	Normal	1.0	-
		Analysis-2	No	Normal	1.0	ϕ and ψ are increased 2°
		Analysis-3	No	Thin	1.0	Pile diameter is 50 cm.
C2	Full Displacement Pile (FDP)	Analysis-1	Yes (0.85)	Thin	0.45	-
		Analysis-2	Yes (0.85)	Thin	0.50	-
		Analysis-3	Yes (0.85)	Thin	0.40	-







Pile	Capacity (kN) for %10*Dmm disp.			
	Test	Analysis-1	Analysis-2	Analysis-3
A3	***1,100***	*1,100*	1,700	1,150
B2	***1,400***	*1,300*	1,550	1,550
C2	***2,000***	2,150	2,250	*2,050*

Pile	Service Loads (kN) for 15mm disp.			
	Test	Analysis-1	Analysis-2	Analysis-3
A3	***620***	*700*	1,000	720
B2	***1,000***	*800*	920	920
C2	***1,500***	2,000	2,080	*1,850*

Fig. 7: Load-displacement curves and ultimate/serviceability loads of tests and analyses for three piles

4 Conclusion

A new approach was proposed for numerical modelling of piles and columns by pre-calibrating (verifying) a pressuremeter test through a modified strain softening/hardening Mohr Coulomb model in finite difference code, FLAC3D. Adopted procedure considers stiffness evolution, which leads to a better soil characterization. All the load-displacement curves were captured reasonably. However, some disturbance and interface adjustment are needed to be done for a successful prediction. By defining proper interface and/or disturbance due to driving proposed approach seems to work successful. For a bored pile constructed with steel sleeve or bentonite slurry, thin interface should be defined with multiplier value between 0.7-0.8. On the opposite, it is not required to define an interface for pressure grouted CFA piles. C2 pile were needed to be modelled through prescribed displacement. A multiplier of 0.8-0.85 for the influence zone and 0.4-0.5 multiplier for interface seems to be defined for numerical modelling.

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