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Comparative Analysis for Numerical Modelling and Design of Anchored Retaining Structures

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Abstract - An in-depth knowledge of the main principles governing the calculation methods implemented in the most widespread software is a key element for suitable geotechnical and structural design. This paper presents the main results of some comparative numerical analyses carried out for the design of flexible back-anchored retaining structures. The adopted codes are based on the concept of an elastic beam lying on Winkler springs and the finite element approach. The main results obtained through the different methods are shown in terms of stress, internal forces, and strains acting on the structures and the surrounding soil. Similarities and differences in results are interpreted and then compared with some outcome measurements performed on the structure during its construction phases, according to the observational method. Some basic suggestions are also provided for designers to choose the most suitable approach, taking into account complexity, type of problem, time consumption, and costs.

***Keywords*:** numerical model, FEM, Winkler, observational method, retaining structures.

1. Introduction

In recent years, the increased power of numerical codes led to imposing them as key tools for many geotechnical problems ([1]-[2]). Under some hypotheses, they allow for the prediction of the stress-strain response of geotechnical structures system considering multiple factors, such as the tensional history due to construction phases or the soil-structure interaction which would be simplified by adopting analytical calculation procedures (i.e. “Limit Equilibrium Methods”) or empirical/semi-empirical approaches ([3]-[4]-[5]-[6]). Nowadays, the most established numerical methods in geotechnical practice can be grouped into two main categories. The first one is the simplest and comprises all the models based on the assumption of soil that can be parameterized as a Winkler spring support (known as “Subgrade Reaction Method”). The second approach considers the soil as a continuous/discontinuous medium (i.e. “Finite Element Methods”, “Discrete Element Methods”, or “Boundary Element Methods”). The Subgrade Reaction Method (SRM) is mainly based on the use of subgrade constant parameter to define the spring stiffness whose estimation does not only depend on the strength/deformability properties of the soil but often requires the use of back-analysis procedures or empirical methods ([7]-[8]). On the other hand, the second category is more complex and allows consider the soil as a deformable medium, and the geotechnical problem is governed by differential equations based on the momentum and mass conservation principle. Based on the hypothesis of “small displacement”, the Finite Element Method (FEM) is certainly the most used numerical approach, because of its capability to give satisfactory solutions in terms of time-consuming and costs with an approximation level that is largely satisfactory in many engineering problems. The choice of the most suitable approach depends on many factors, i.e. the type of geotechnical problem to deal with, the required approximation level of the solution, and always should be based on a knowledge of the basic principles governing the different approaches. This paper gives two comparative analyses for professional practice to investigate similarities and differences when using SR Methods or FE approach. The first analysis deals with the flexible structure to support a railway line and consists of back active-anchored micro-piles wall. The second analysis consists of a closed structure for the construction of a metro station in an urban environment (top-down excavation method) and deals with back-anchored micro-piles contrasted by two orders of struts. Results obtained through the two approaches are shown, similarities and differences are interpreted, and some suggestions are given.

2. Case study n.1

2.1. Numerical modeling and analysis description

Two analyses were carried out regarding a real case, consisting of a temporary retaining structure to support an existing railway line (Fig.1). The excavation, approx. 6.2 m depth, is supported by a spaced micro-pile wall consisting of steel tubular sections, filled with concrete. The structure is reinforced by orders of pre-stressed anchors, while some shotcrete is applied step-by-step to the front face of the micro-pile wall to prevent any localized soil detachments (Tab.1).

Table 1: Back anchored micro-pile wall: details of the structural elements.

Structural element		Micro-piles	Anchors
Type	(-)	Tubulars	Active anchors (2 strands, 200mm^2)
Length, L	(m)	10.0	7.0 (free length), 6.0 (bulb length)
Spacing, l	(m)	0.40 (horizontal)	1.60 (horizontal), 1.50 (vertical)
Diameter, Φ	(mm)	273 ($\Phi_{borehole}$), 193.7 ($\Phi_{ext.}$)	160 ($\Phi_{borehole}$)
Thickness, s	(mm)	25	-
Material	(-)	Steel S355	Steel Y1860
Pre-tensioning, P	(kN)	-	120
Inclination, α	(°)	-	20

The comparative 2D analyses were performed by using two codes, respectively *ParatiePlus* (Vers. 22) and *Plaxis2D* (Vers. 21.01.00.479). The former is based on the Winkler-spring approach where the soil is assumed to be made by springs whose stiffness is evaluated through geometric considerations and an elastic-plastic constitutive law ([4]). On the other hand, the latter code implements the Finite Element Method, so small displacements are assumed together with the presence of interfaces at soil-structure contact. In both cases, modeling was performed under plane deformation conditions, while the excavation and the support application were modeled step-by-step. Appropriate boundary conditions were also applied: *Plaxis2D* model was constrained by hinges on the lower edge and rollers on the vertical edges. The soil was discretized by 3,090 triangular meshes that were densified in the most relevant clusters.

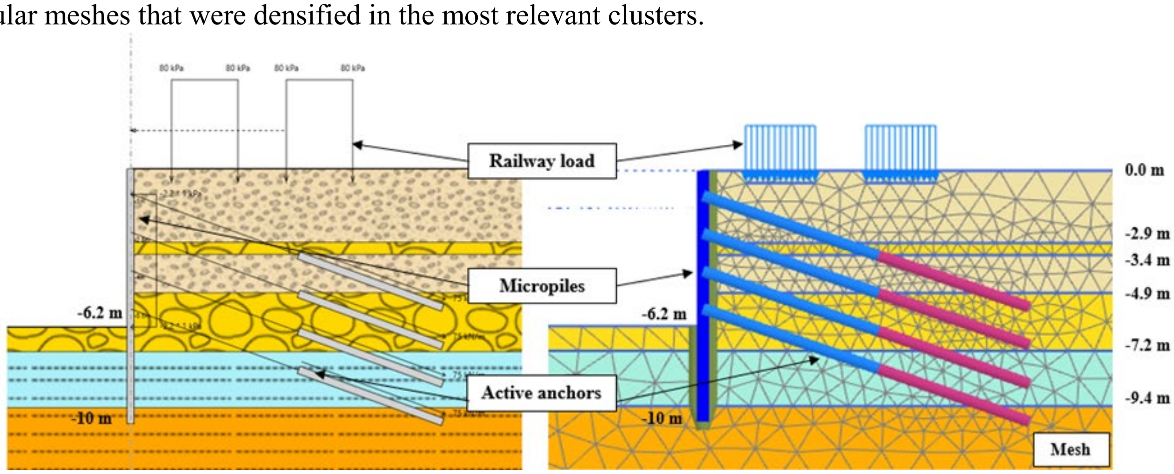

Fig. 1: Geometry and loads at the last step: excavation: left) P*aratiePlus*; right) *Plaxis2D*.

Perfectly plastic Mohr-Coulomb failure criterium was adopted for *ParatiePlus* model (Tab.2), while a Hardening-Soil constitutive law was used in *Plaxis2D* model (Tab.3). No water table was detected, hence, dry conditions were assumed, while the presence of two railway tracks was simulated by applying 80 kPa overload, applied 0.50 m below the ground level.

Table 2: Geotechnical properties, Mohr Coulomb failure criterium (*ParatiePlus* model).

Material	γ (kN/m³)	c' (kPa)	φ' (°)	E (MPa)	v (-)	Constitutive Law
Embankment	17.0	0.0	29.6	17.0	0.25	ELPLA Mohr-Coulomb
Debris 1	19.0	0.0	32.0	22.0	0.25	ELPLA Mohr-Coulomb
Debris 2	19.0	3.6	29.2	17.0	0.25	ELPLA Mohr-Coulomb
Fluvial soil	19.0	0.0	35.0	25.0	0.25	ELPLA Mohr-Coulomb

Table 3: Geotechnical properties Hardening-Soil failure criterium (*Plaxis2D* model).

Material	γ (kN/m³)	c' (kPa)	φ' (°)	E_{50}^{ref} (MPa)	E_{oed}^{ref} (MPa)	$E_{ur}^{ref}=1.6 \cdot E_{oed}^{ref}$ (MPa)	Constitutive law
Embankment	17.0	0.0	29.6	17.0	17.0	27.2	Hardening-Soil
Debris 1	19.0	0.0	32.0	22.0	22.0	35.2	Hardening-Soil
Debris 2	19.0	3.6	29.2	17.0	17.0	27.2	Hardening-Soil
Fluvial soil	19.0	0.0	35.0	25.0	25.0	40.0	Hardening-Soil

2.2. Results and preliminary considerations

The horizontal displacements at the final stage are shown in Figure 2a for both analyses. *ParatiePlus* model provides backward displacements at the top of the wall approx. 4 mm, with maximum values of about 5 mm (solid line) that can be observed forward. The FE model provides top forward displacement approx. 6 mm and maximum values 9 mm (dotted line), while displacements at the wall toe are negligible for both models. At the ground level, FE analysis also provides vertical displacements comparable with those obtained by *ParatiePlus* model (Fig. 2b). The only exception is given by the area close to the top of the wall, where the Winkler model estimates some uplift, probably due to the pre-tensioning force applied to the first order of anchors. The maximum subsidence, in the order of 5-6 mm, is well comparable for both models.

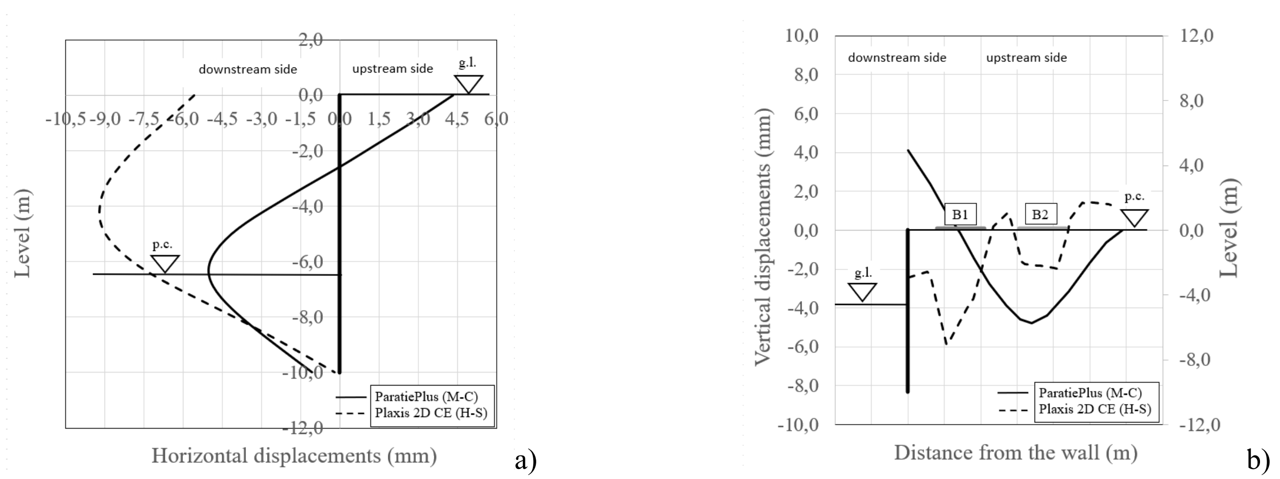

Fig. 2: Comparison of horizontal displacements (a) and vertical displacements at ground level (b).

No partial factors were introduced in both analyses, therefore stresses are provided with their characteristic values. Despite some differences, bending moment M_k and shear force T_k are generally comparable in magnitude (Fig.3a,b) and this has a certain relevance for design purposes since the maximum values are used - after appropriate factoring - for the design of the structural elements. A comparison of the maximum axial force P_k acting on the anchors is given in the following Table, regarding the final stage. Anchors are stressed by similar forces for both models and once again, this aspect is relevant for design purposes since, before appropriate factoring, these values will be used for internal and external capacity checks of the anchors.

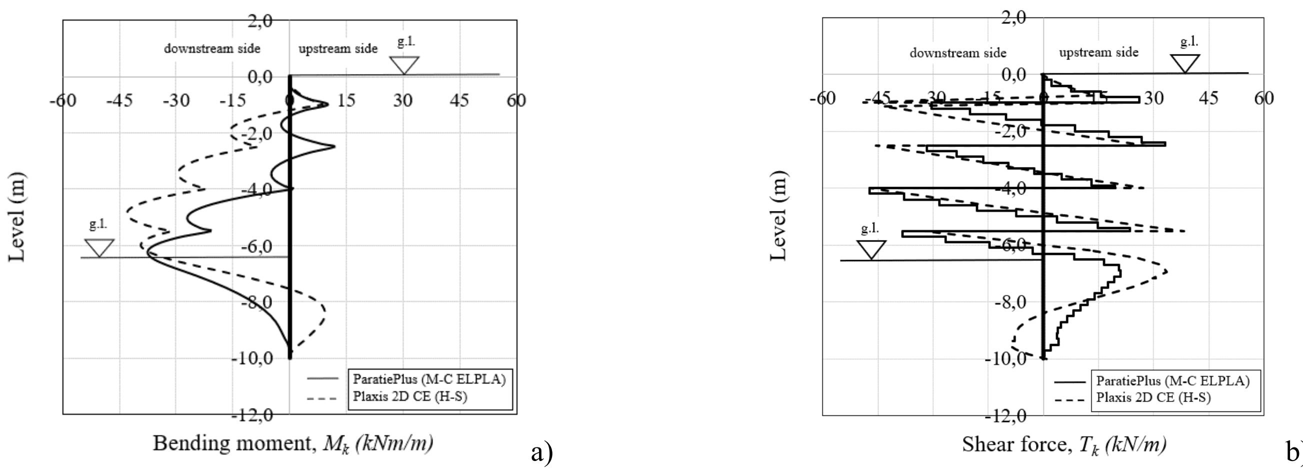

Fig. 3: Comparison of bending moment (a) and shear force along the micro-pile wall (b).

Table 4. Axial actions on the anchors (characteristic values on individual anchors).

Order	Level (m)	*ParatiePlus* - P_k (kN)	*Plaxis2D* - P_k (kN)
1°	-1.0	114.3	126.0
2°	-2.5	121.0	128.5
3°	-4.0	124.6	132.9
4°	-5.5	120.0	120.0

Due to the more accurate numerical approach and the considered constitutive law, results given by FE model in terms of deformations - especially far from the wall, e.g. under the railway tracks - are considered to be more reliable than those obtained by the Winkler-spring approach, where the soil-structure interaction is modeled through elastoplastic springs at the wall-soil contact. Despite this, the latter method still offers an acceptable approximation for the most common design purposes, since stresses along the structure find good agreement with both approaches.

3. Case study n.2

3.1. Numerical model and analysis description

The current case study consists of a closed retaining structure in an urban context, to allow the construction of a metro station. The rectangular excavation for the station box (20.0 m deep, approx. 67.0 m x 19.0 m) is supported by spaced micro-piles and shotcrete. It is supported through two orders of hollow struts and back-anchored by active anchors (Table 5). A shotcrete layer is applied at the wall face during the lowering of the excavation to prevent any local soil failures. Some photos of the site during construction works are still available on the official website of “Canada line” with interesting details about the project (Fig. 4).

Fig. 4: Construction works of The Canada Line: Yaletown – Roundhouse [9].

The soil, modeled with an elastic-perfectly plastic Mohr-Coulomb constitutive law, consists of thickened silty sand and lenses of loose sand (Table 6). The water table lies 9.5 m below the ground level, while an overburden of 12 kPa is applied at ground level. Again, comparative numerical analyses were conducted using the *ParatiePlus* (Fig.5a) and *Plaxis2D* (Fig.5b) codes. The FE model consists of 2300 triangular elements along the entire model densified in the most relevant clusters.

Table 5. Structural elements: micropiles, struts, anchors, and foundation slab.

Structural element	Micropiles	Struts	Anchors	Foundation slab
Type	Steel pipe	Steel pipe	Steel hollow bars	Concrete plate
Length, L (m)	27.0	18.8	6 - 7 (free length) 11.0 (anchored length)	18.8
Spacing, i (m)	0.55 – 0.65 (horizontal)	5 (horizontal) 3.7 (vertical)	1.3-2.0 (horizontal) 2.0 (vertical)	-
Diameter, d_{ext} (mm)	177.8	457.0	70 (perforation) 1'' (hollow bar anchors)	-
Thickness, s (mm)	10.0	12.0	-	200
Material	Steel (f_{yk} = 350 MPa)	Steel (f_{yk} = 350 MPa)	CTS/TITAN IBO, Steel (f_{yk} = 454 MPa)	Concrete C20/25 Steel (f_{yk} = 500 MPa)
Pre-tensioning force, P (kN)	-	-	140 - 250	-
Inclination, α (°)	-	-	15 - 35	-
Installation depth, p (m)	-	8.5 (1° order) 12.2 (2° order)	14.0 (1° order) 17.0 (2° order)	20

Table 6. Geotechnical properties of the soil: weight per unit volume (γ), cohesion (c'), friction angle (φ'), elastic modulus (E), Poisson ratio (ν), hydraulic conductivity (k), overconsolidation ratio (OCR) and constitutive law.

Material	γ (kN/m^3)	c' (kPa)	φ' (°)	E (MPa)	ν (-)	k (m/s)	OCR (-)	Constitutive law
Type A (silty sand)	21.5	33.0	32	150	0.25	$6 \cdot 10^{-8}$	1.0	M-C ELPLA
Type B (loose sand)	21.5	31.0	35	50	0.25	$1 \cdot 10^{-5}$	1.0	M-C ELPLA

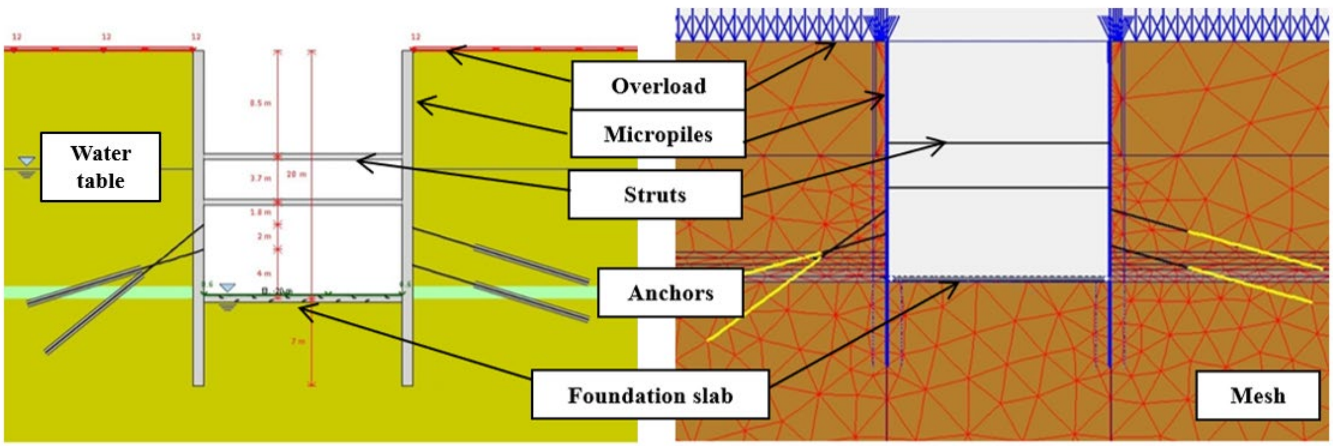

Fig. 5: Geometry and loads at the last step: a) *ParatiePlus* and b) *Plaxis2D* [10].

3.2. Results and comments

Effective stresses, bending moment, and shear force along the wall are shown for both approaches (Fig.5a,b,c). Again, some differences can be observed in terms of bending moment and shear stress of the Winkler approach is used rather than the FE method. The effective stress distribution along the wall makes it possible to guess the reasons for the differences observed in terms of stress-strain response by applying FE method or Winkler approach. *ParatiePlus* analysis gives horizontal effective stresses approximately linearly increasing with depth (solid line). On the other hand, *Plaxis2D* model provides a horizontal effective stress distribution made by peak values, i.e. high-stress zones close to the struts, anchors, and the bottom of the excavation. The reasons for these differences can be found in the different approaches used by the codes to evaluate the stress state of the soil. The Winkler approach implemented by *ParatiePlus* code does not allow for interruptions in the distribution of stresses on the wall, since the springs representing the ground are always connected to the wall itself and therefore always provide non-zero effective stresses. The soil thrusts evaluated by the Winkler approach will raise the active or passive thrust as extreme values if failure conditions are reached, but they will never return any null thrust value, starting from the critical depth of active rupture. On the other hand, the soil thrust evaluated by FE analysis may be null. This is due to the retaining wall's high flexibility and the presence of the wall-soil interface, which may allow detachments or overlaps, so the wall could not be in contact with the adjacent soil at some points, ceasing to receive its load contributions.

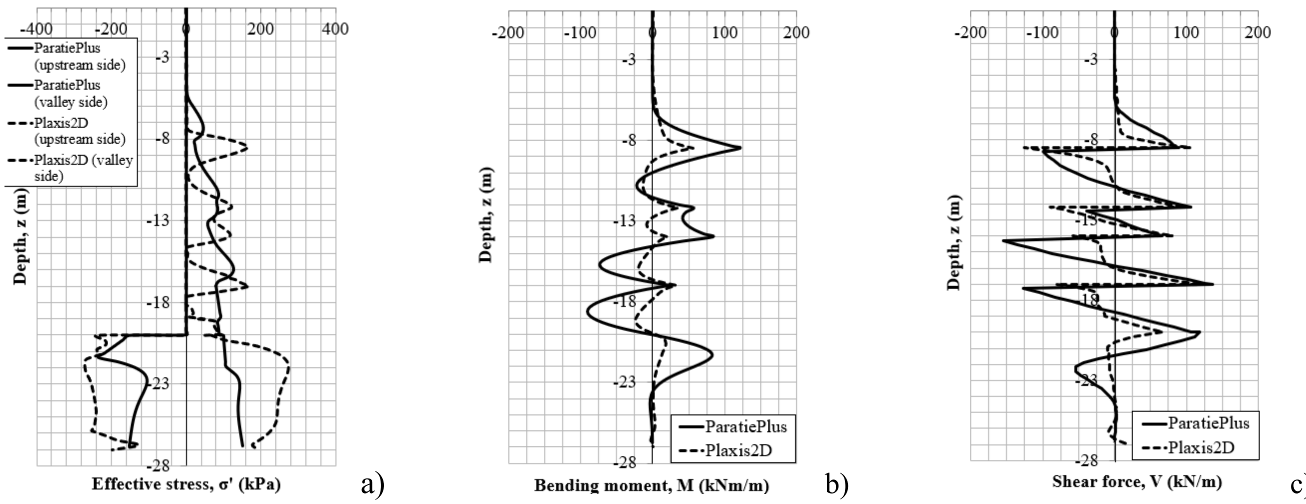

Fig. 5: Comparison of the main results obtained from *ParatiePlus* and *Plaxis2D* model: a) effective stress; b) bending moment; c) shear force.

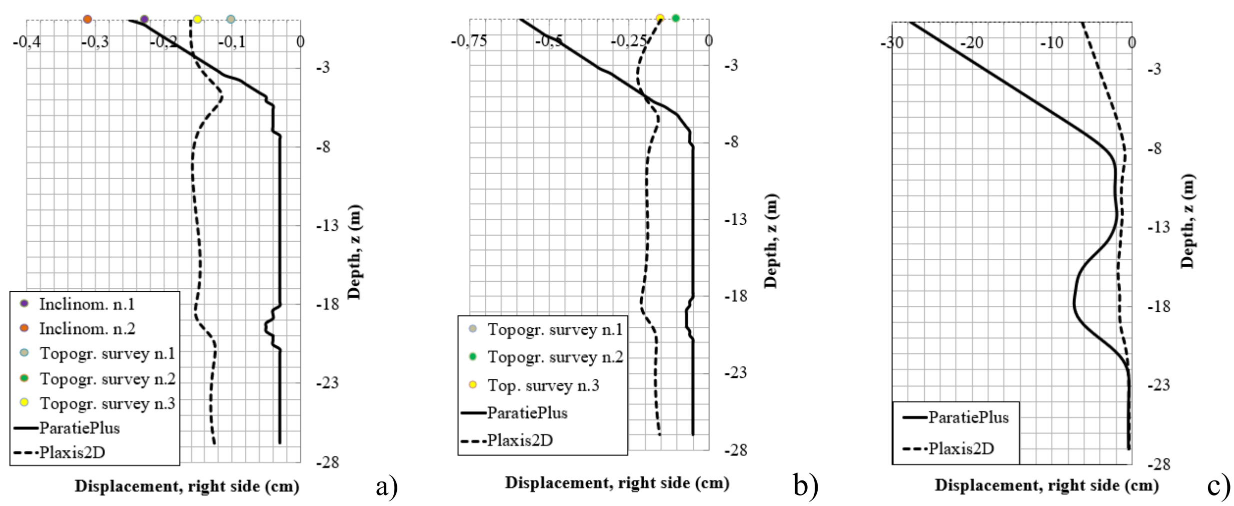

Fig. 6: Predicted and measured displacement along the wall during the excavation phase: a) depth: 4.5 m; b) depth: 6.0 m; c) depth: 20.0 m.

Furthermore, the FE code evaluates the soil thrust by considering all the actions involved in the whole construction of the structure. So, the principal stresses at some points of the model can be rotated until reaching 90°, mainly because of the filtration, presence of local loads, or application of the pre-tensioning force on the anchors, resulting in stress peaks in the effective stress distribution. Generally, these effects cannot be considered by the Winkler models: the only way to consider the horizontal thrust inclination - which is theoretically just attributed to the wall/soil friction - is the adoption of limit equilibrium coefficients that already consider the stresses rotational effect. Displacements along the wall during some excavation phases are finally given for both numerical models, together with measurements recorded on the structures at ground level (Fig.6) consisting of inclinometers and topographic surveys with target points. The differences increase when the load conditions of the wall become more severe, especially in the cantilevered part at the top. The Winkler approach gives higher displacement than those provided by the FE approach, while the latter finds a good agreement with the measurement results.

4. Conclusion

Two case histories are presented and the main results obtained by applying different methods - respectively FEM Winkler approaches - are compared in terms of stress, displacement, and internal forces acting on two retaining Outcomes are interpreted based on working principles adopted by the different approaches. Despite FEM codes more realistic results, also simpler Winkler approach codes are suitable for satisfactorily dealing with medium-problems. A detailed knowledge of the main principles governing the calculation methods remains therefore an essential tool for designers' purposes.

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