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The potential of 3D numerical modelling for the interpretation of load tests on a sheet pile quay wall

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ABSTRACT: Load tests are essential to verify the actual behaviour of every large infrastructure and ensure the safety of the activities and workers. Paradoxically, there are situations in which the evaluation of the load test result is more challenging than the design of the work. This is the case of some strengthening and renovation of old geotechnical works in which an entirely new structure is added close to the old one, which is left in place. The design of the renovation can assume that the new structure is able to bear the entire working load on its own, so that the existence of the old structures is totally ignored. Differently, the evaluation of the load test result needs to consider the real arrangement of all the structures existing in the significative volume of soil involved in the test. In this paper, the availability of monitoring data relating to the activities carried out to renovate a large sheet pile quay wall in a major Italian port, allowed to explore the potential of 3D numerical analysis to assess the behaviour of the work taking into account the coexistence of old and new structures.

Keywords: 3D numerical modelling; Quay; Sheet pile wall; Load test, Renovation

1 INTRODUCTION

Load testing is typically used to demonstrate that existing or repaired structures can safely resist design loads. The load testing procedure is well established for bridges in order to evaluate the load carrying capacity by measuring stress and strain under the actual loading. In geotechnics, load testing on piles and ground anchors to verify their real performance are usual for every construction site.

In maritime infrastructures, load testing is traditionally restricted to the bitts to verify their response to mooring actions. Less frequent are the load tests of the entire quay walls, even though some examples can be found in the literature (den Adel et al., 2019; Feremans and Vanhooydonck, 2019; Alesiani et al., 2023).

However, the evolution of maritime worldwide commerce toward larger cargo ships requires highly performing port facilities, so that the importance of in-situ load tests to verify the response of quay walls to heavy operative surcharge is progressively increasing. As a consequence, many important works are now tested by applying the design load on the apron at the end of their construction. Moreover, in many cases the expansion of port facilities into new areas is not possible so that the current trend is to adequate old quay walls to more demanding requirements, including deeper seabed, larger surcharge and mooring loads (Ruggeri et al., 2019; Ruggeri et al., 2021a). In these cases the renovation project often consists of the construction of a new structure, close to the old one, able to bear the entire working load on its own. This solution makes easy the design of the new structure but, at the same time, the evaluation of the safety of the new work at the end of its construction becomes an hard task. This is because the interaction between the old and the new structure makes hard to reliably forecast the structural response of the entire work and, in any case, the presence of the old structure cannot be ignored.

In this paper, numerical modelling has been adopted to evaluate the response to a load test of an anchored sheet pile quay wall, where an important renovation work has been carried out by adding a new structure near to the old one. The interaction between the two structures required a 3D numerical model and advanced constitutive laws for the soils. The two-stage progression of the work suggested the carrying out of two different load tests, each time applying the design load on the apron close behind the quay wall and measuring displacements of the top beam, horizontal deformation of the diaphragm and stress on the ground anchor. The first load testing is carried out after the renovation works but with the current seabed, while the second test is planned after the excavation of the seabed down to the final design configuration.

Monitoring data after the first load case are included here and compared with the prediction from the numerical model. Also, class A prediction obtained by the numerical model is provided for the second load stage.

2 CASE STUDY DESCRIPTION

The case study of interest is represented by an old quay wall, located in the Italian port of Ravenna, that has been recently renovated by adding a totally new structure, very close to the existing one, and able to bear the entire working load by its own.

The old quay wall, built in the 1970s, is a cast in-situ diaphragm supported at the top by a series of ground anchors. The diaphragm is made of concrete rectangular panels 2.50 × 0.80 m, with the sides shaped to fit together, deepened up to 18 m under the sea level. All the panels are joined by a large top beam 2.50 × 1.80 m (B × H) forming 50 m long segments, hosting the heads of the anchors every 2.5 m and the 500 kN bitts every 25 m. The ground anchors, inclined 21° from the horizontal, have a free length of 15 m, a foundation of 10 m and are reinforced by 4 "7 wire strands" of 15.2 mm diameter. The structural system was designed for an apron at 1.5 m, a seabed at -9.40 m on m.s.l. and for a surcharge of 40 kPa.

The renovation aims to improve the structure for major port requirements including a surcharge of 60 kPa, a seabed at -13,0 m on m.s.l. and a mooring load of 1,000 kN.

A preliminary assessment for the upgrade of the existing structure has highlighted two serious problems: the lack of diaphragm embedment and the insufficient resistance of the ground anchors. Moreover, there were important concerns about the reliability of strands after decades of working life in an aggressive environment without efficient protection systems. For these reasons, the designer decided to rely on a totally new structure abandoning the existing one.

Practical reasons related to constructive issues shaped the adopted solution. It consists of an alignment of bored piles, 1.10 m diameter and 27 m long, quincunxes arranged, so that two piles fit the space between existing ground anchors leaving about 0.15 m each side to preserve the old anchors. A new, massive, concrete top beam joins the piles partially encapsulating the old top beam. Also, a new series of ground anchors, spaced 2.5 m, inclined 18° from the horizontal, with a free length of 15 m and a foundation of 20 m are built from the new top beam and prestressed to 400 kN. Some more ground anchors strengthened the top beam near the bitts and all anchors are reinforced by 8 "7 wire strands" of 15.7 mm diameter.

Therefore, the new structure is similar to the old one but it is largely more robust and adopt state-of-the-art technological solutions to face the aggressive environment (e.g. double corrosion protection for ground anchors, high strength concrete for top beam).

To mitigate the risk of liquefaction under seismic shaking of the shallow sandy deposit and improve the pavement performance, an array of full displacement piles (FDP), 0.50 m diameter, 10 m long and 2.50 × 2.50 m spaced were also built behind the quay wall.

In Figure 1(a) a vertical cross-section of the quay wall is represented: the old structures are drawn in black, while the new ones are in red. It is possible to note the proximity of the two wall, less than 2.5 m axle-to-axle. Also, considering that a jet grouting treatment is adopted to seal the gap between adjacent piles and that the old and new structure share the top beam, their interaction is unavoidable.

2.1 Site characterization

The area of interest belongs to the coastal part of the Po Plain (northern Italy) and the soils in the volume of interest for the work are of recent origin and generally characterized by poor geotechnical properties.

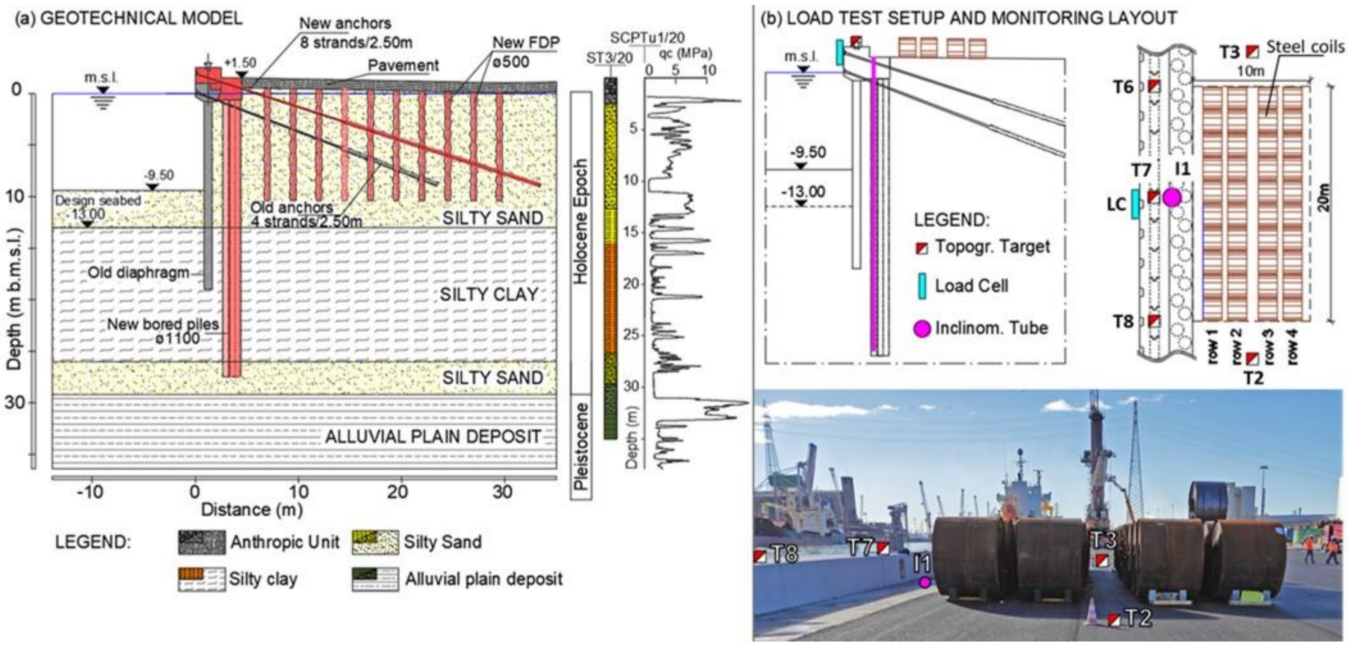

Figure 1. (a) Geotechnical model of the sheet pile quay wall; (b) Load test setup and monitoring layout

From a geological perspective, the first 25-30 m are Holocenic deposits of marine origin laying on Pleistocene soils of alluvial deposition. From a geotechnical point of view, the stratigraphic sequence can be grouped in 4 Units: from the ground surface we find a medium dense Silty Sand layer, 14 m thick, followed by a very soft Silty Clay deposit up to 26 m of depth. A thin layer of Sand indicates the passage to the Pleistocene soils of alluvial plain, constituted by a sequence of fine and coarse-grained layers of generally good mechanical properties. The first layer of the deep Pleistocene deposit is typically fine-grained, so we decided to model such Unit as fine grained. In Figure 1(a) the outlined stratigraphic model is represented with the structures of interest. The availability of several laboratory and in situ testing (including CPTs, DMTs, triaxial and oedometer tests, geophysical and resonant column tests) allowed the evaluation of all the ground properties (Ruggeri et al., 2021b). A CPTu profile of the tip resistance in Figure 1(a) gives an idea of the strength of the 4 Units.

2.2 *First load testing setup and monitoring*

The first static load test aims to reproduce the effect of heavy operative loads on the quay wall apron with the current seabed depth. To simulate this condition, an area of 10 m × 20 m was loaded with an array of 40 steel coils, arranged on 4 raws, to achieve the characteristic design value of 60 kPa. The loaded area has a width similar to the seabed depth and a lenght twice that value, as shown in Figure 1(b).

The monitoring system, depicted in Figure 1(b), consists of a strain-gage load cell on a ground anchor head (LC) to record the value of the load on the anchor, an inclinometer tube (I1), 29.5 m long, placed between new and old wall to evaluate its deformation, and some topographic targets on the top beam and on the apron to measure the horizontal displacement of the beam and settlement of the pavement.

As shown in Figure 2, inclinometer, topographic and load cell readings were carried out before the loading (“before test”), soon after the full load application (“load test day”), the day after the full load was in place (“following day”) and after the removal of the load (“after unloading”). Unfortunately, the topographic measurements resulted not enough accurate to catch the small displacements exhibited by the structures after loading and were considered unreliable.

Inclinometer readings indicate a maximum displacement of 1.0 mm soon after the application of the loading, that increases to 1.8 mm after 24 hours, due to a slight release of the top anchor restrain. The observed phenomenon could be favored by a tide variation of about 1 m, very large for the site. After unloading, the recovery of the displacements is negligible, confirming that retaining wall displacements are largely non-recoverable.

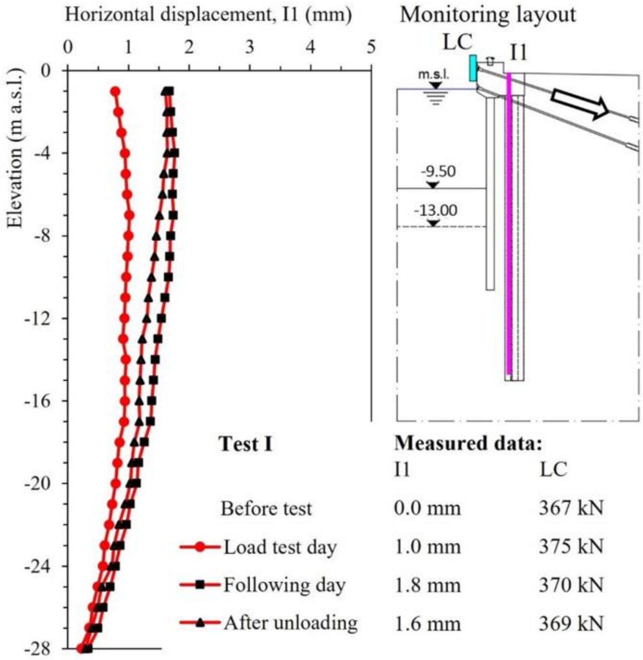

Figure 2. Measured horiz. displacements and load cell values

The load cell detects a slight increase of the tension force on the new anchor of about 8 kN soon after the application of the surcharge. However, the day after the loading the force decreases of 5 kN even if the top wall displacement increases of 1 mm, with a behaviour not easily explainable. After unloading the apron, the load cell does not detect any significant variation, so keeping the reached load.

3 FINITE ELEMENT MODEL

Due to geometry of the structure and load test configuration, a 3D model was chosen to simulate the case study.

All the numerical analyses were performed using the three-dimensional finite element code PLAXIS 3D (Brinkgreve et al., 2013) which allows the simulation of construction phases, excavation work and load testing by activating and deactivating soil clusters and structural elements.

The low level of deformations detected by monitoring during the test suggested to adopt the Hardening Soil with Small Strain Stiffness (HS-Small) model for the two main geotechnical units.

3.1 *Model description*

The numerical model includes soil Units, pavement, new and existing retaining walls, top beam, ground anchors and displacement piles. The model width, equal to 50 m, includes an entire quay wall sector between two joints of the top beam, while length and height were chosen large enough to avoid significant boundary effects, 100 m and 40 m respectively.

The two concrete walls and top beam were modelled as elastic volume elements with the parameters shown in Table 1. The new wall, made of adjacent concrete piles, was modelled as an equivalent rectangular section. Ground anchors free length was modelled as node-to-node element whereas the foundation as embedded beam (Table 2). The displacement piles were also modelled as embedded beam elements (Table 3).

Table 1. Input parameters for elastic volume elements

Parameter	Unit	Existent wall	New wall	Beam
γ	kN/m^3	25	25	25
E	MN/m^2	21000	34000	30000
ν	$-$	0	0	0.15

Table 2. Input parameters for anchor elements

Parameter	Unit	Anchor 4×0.6"	Anchor 6×0.6"	Anchor 8×0.6"S
Anchor free length				
EA	kN	112000	168000	240000
Preload	kN	492	715	400
Anchor foundation				
EA	kN	$9.4 \cdot 10^5$	$9.4 \cdot 10^5$	$14 \cdot 10^5$
T_{skin}	kN/m	80	80	80

Table 3. Input parameters for displacement piles

Parameter	Unit	Value
Diameter	m	0.5
E	MN/m^2	30000
T_{skin}	kN/m	80

Considering the rough soil-structure interface between cast-in-place diaphragms, piles and soil, a full friction angle between concrete and soil can be assumed and consequently no interface elements were adopted.

Uniformly distributed surface loads applied on the pavement were used both to account for the load history of the old quay wall and to model the loading test on the new quay, at the end of the renovation works.

The 3D model geometry with the 10-node tetrahedral finite elements mesh is illustrated in Figure 3. Standard boundary conditions were adopted, with the bottom of the model fixed in all directions and the vertical sides constrained in the horizontal out-of-plane direction only.

The numerical analyses were performed as elastic-plastic drained analyses.

3.2 Constitutive model and soil input parameters

The Hardening Soil with Small Strain Stiffness (HS-Small) is an advanced elastic-plastic material model for soft and hard soils (Benz et al, 2009) which has been shown to correctly simulate several soil-structure interaction problems involving retaining structures (Tschuchnigg & Schweiger, 2013).

The HS-Small model formulation requires as input, in addition to the Mohr-Coulomb strength parameters φ' and c', three different stiffness parameters at a reference pressure (p^{ref}), namely E_{50}^{ref}, E_{ur}^{ref} and E_{oed}^{ref}.

To consider the small-strain soil stiffness and its non-linear dependency on the strain amplitude reached during construction and loading stages, the HS-Small model includes the stress-strain curve $G(\gamma)$ for small strains by a simple hyperbolic law:

$$\frac{G}{G_0} = \frac{1}{1+0.385\left|\frac{\gamma}{\gamma_{0.7}}\right|} \tag{1}$$

where the two additional parameters that allow to consider the variation of stiffness with strain are the initial shear modulus G_0^{ref} and the shear strain level $\gamma_{0.7}$ at which the secant shear modulus is reduced to about 70% of its initial value. The calibration procedure, not reported here for brevity issues, were validated through back analysis of a former load test on a quay wall at the same site of interest (Segato et al., 2010). The assumed HS-Small input parameters are summarized in Table 4. A linear elastic perfectly plastic Mohr-Coulomb (MC) model was used for the pavement. The 1 m total thickness was modelled by considering two main layers: a cement-treated foundation and a bonded layer of bituminous conglomerate. The MC model parameters assumed for the pavement are summarized in Table 5.

Table 4. Input parameters for soils: HS-Small model

Parameter	Unit	Silty Sand	Silty Clay
γ	kN/m^3	18	19
G_0^{ref}	kN/m^2	55000	60000
$\gamma_{0.7}$	%	0.02	0.02
ν_{ur}	$-$	0.2	0.25
E_{50}^{ref}	kN/m^2	12000	6200
E_{oed}^{ref}	kN/m^2	12000	3100
E_{ur}^{ref}	kN/m^2	36000	24000
p^{ref}	kN/m^2	50	100
m	$-$	0.5	1.0
c'	kN/m^2	0	0
φ'	°	36	31
ψ	°	0	0
K_0^{nc}	$-$	0.412	0.485

Table 5. Input parameters for pavement: MC model

Parameter	Unit	Bonded layer	Foundation layer
γ	kN/m^3	21	19
E	MN/m^2	2000	700
ν	$-$	0.2	0.2
c'	kN/m^2	577	200
φ'	°	30	30
σ_t	kN/m^2	0	0

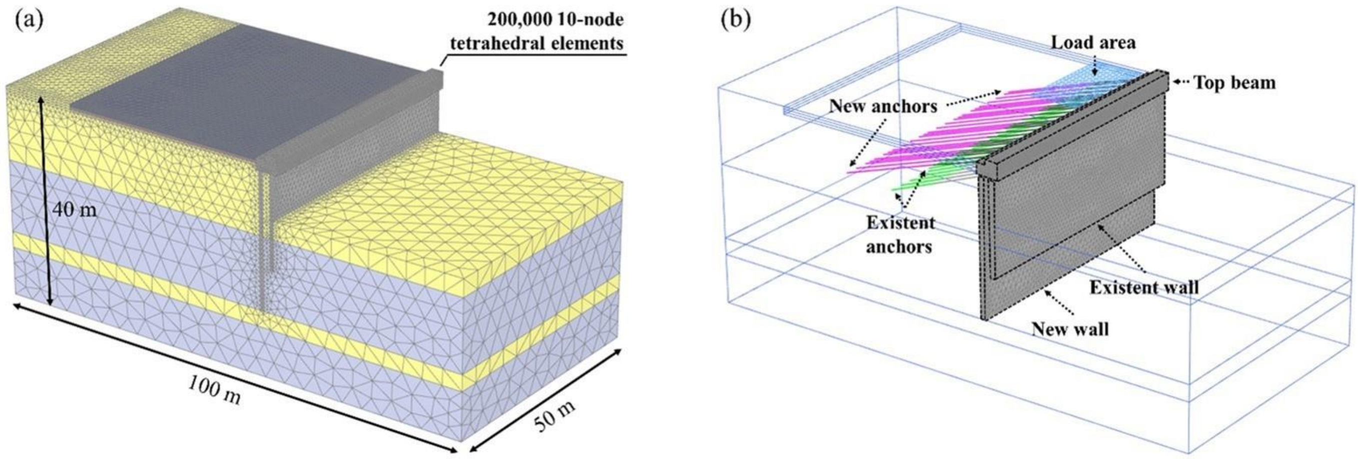

Figure 3. Model geometry. (a) Finite element mesh; (b) Structural elements

3.3 Construction and load test sequence

The most relevant construction phases, from installation and operational use of the existent work to the quay wall renovation, are reproduced in the numerical analysis to correctly simulate the load history.

After that, three more phases were considered to reproduce: 1) the load test carried out in the current configuration (i.e. Load Test 1); 2) to perform a class A prediction of the seabed deepening effect (i.e. Excavation to -13.0 m.s.l.); 3) the final load test (i.e. Load Test 2).

4 RESULTS

4.1 Test 1 - Comparison with monitoring data

Figure 4 shows the comparison between measured and predicted horizontal displacements due to load Test 1.

The values of inclinometer I1 are those recorded the day of the load test. The small displacements of the wall, demonstrating a rather stiff response of the composite system formed by the new and the existing structures, are well captured by the model, both in shape and size, proving the numerical model soundness.

The model gives a 407.5 kN on the node-to-node element of the new anchor, that means an increment of 7.5 kN due to the surcharge application. This is in very good agreement with the recorded value, equal to 8 kN. Of course, the model cannot explain the drop of load observed 24 hours after the application of surcharge.

4.2 Excavation and Test 2 - Class A prediction

Figure 4 shows the class A prediction of the horizontal displacement induced by the future seabed excavation to -13,0 m on the m.s.l. and the planned Test 2 load case.

Whilst Test 1 induced a very small effect on the structures, the deepening of the seabed and the loading of Test 2 are expected to produce a more relevant response of the work. In particular, the maximum deflection of the wall is expected reaching 5 mm after deepening (i.e. +4 mm from Test 1) and 9 mm after Test 2 loading (i.e. +3 mm from deepening).

It is worth noting that the increment of deformation due to the excavation is expected to be comparable to that induced by the surcharge of the quay wall. Correspondently, the anchor load is expected equal to 416 kN (i.e. +9 kN from current seabed) after excavation and to 425 kN (i.e. +9 kN from deepening) after loading Test 2.

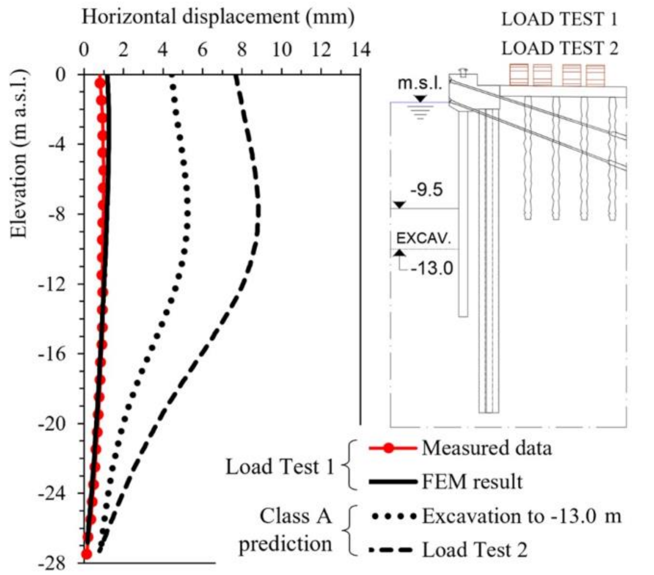

Figure 4. Measured vs predicted displacement of Test 1. Predicted displacement of Excavation to -13.0 m and Test 2

4.3 Effect of FDP and existing structures

Focusing on load Test 1, two more analyses have been performed to check the relevance of detailed modelling on the numerical results. In particular, the presence of the full displacement piles strengthening the Silty Sand and the existence of old structures have been considered.

In Figure 5 the measured displacement of the wall is compared with the prediction of the numerical model described before (continuous line) and with the prediction of the numerical model without FDP (dashed line). It is evident the difference in the responses of the two models especially in the shape of the expected deformation of the wall.

In Figure 6 the relevance of the old structures in the predicted displacement is evident. The expected shape of the wall would have been different if the presence of old structures had been neglected.

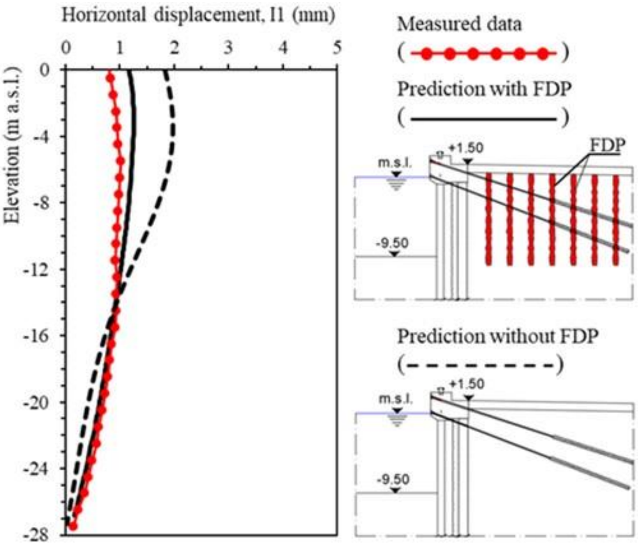

Figure 5. Predicted displacement with and without FDP vs measured displacement

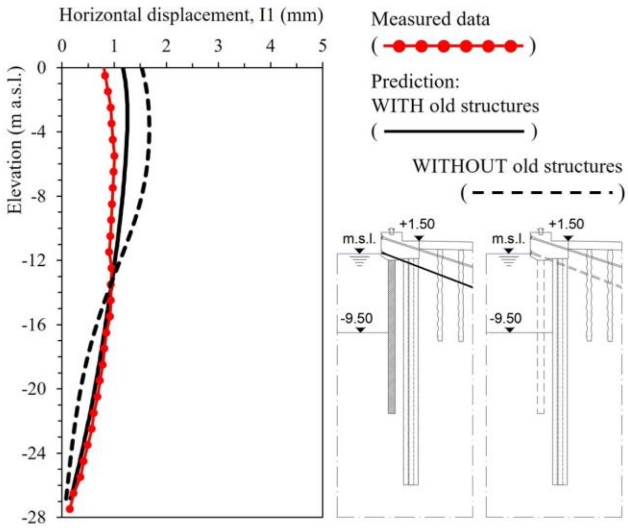

Figure 6. Predicted displacement with and without old structures vs measured displacement

5 CONCLUSIONS

The paper has shown the potential of a 3D numerical modelling to accurately predict the effects induced on a renovated quay wall by a static load test carried out to reproduce the operative surcharge on the apron.

The paper pointed out the need of advanced and detailed numerical modelling when the mobilized level of deformation of the soil is very low, as happens in the considered case study. In particular, the old structures and the improvement of Silty Sand unit are relevant on the response of the composite system to the applied load and therefore their presence cannot be ignored in the numerical model.

This outcome appears of general relevance for the serviceability analyses. On the other hand, while ultimate limit state analyses benefit of a simplified modelling, in which the number of structures is limited to the essential ones, the prediction of displacements needs a detailed modelling of all the components of the construction in the significative volume. In this sense the 3D modelling is found particularly appropriate for the considered case study.

The good matching for load Test 1 between numerical results and measurements makes confident about the class A prediction for the response of the quay wall to the future seabed excavation to -13,0 m on the m.s.l. and the planned Test 2 load case.

6 ACKNOWLEDGEMENTS

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