The rotation hinge water seal and rubber in the grooves of the gate is installed in order to keep the water head of the reservoir. There is water leakage in the joint of the two sides of the seal and the side seal when the water head is and below 100m. And the rest part can seal the water. The compressive deformation under different water head is shown in Table 4.

Table 4. The compressive deformation of the rotation hinge seal with different water head

Water head（m）	70	80	85	90	100
Left compressive deformation（mm）	2.0	2.0	2.0	2.5	2.5
Right compressive deformation（mm）	2.3	2.6	2.6	3.1	3.1

The correlation of the water head and the preset compressive deformation of different part of the seal can be illustrated clearly in the Fig. 5.

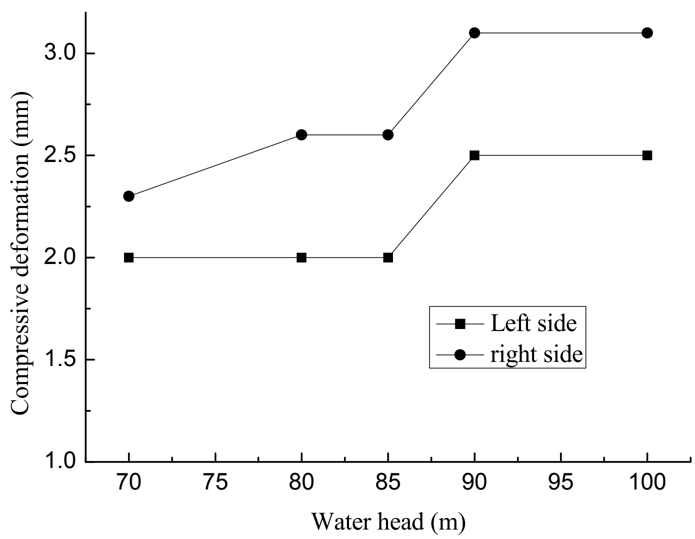

FIG. 5. Correlation of the water head and the preset compressive deformation of the rotation hinge seal

CONCLUSIONS

Some conclusions can be obtained from the experiments in this research:

If the geometry dimension of the specimen is controlled qualified, the sealing can be done well under the water head of 85m when the compressive deformation of the top, bottom and side water seal is 4mm, 10mm and 3-5mm, respectively.

The shape, the overall connection type joint and the structure of the water seal is much better for sealing after the modification in this experiment.

Not only the material properties of the water seal should be satisfied the requirements, but also the geometry dimension must be strictly controlled. In this experiment, because of the deviation of the dimension the water leakage occurred. Then the accuracy is affected.

It is recommended that the preset compressive deformation of the side water seal should be 3-5mm after the installation of the gate and grooves of the gate. So the sealing can satisfy the design index well.

The bolts must be tightened with uniform force because the subplate of the top water seal is of rubber plate with thickness of 2mm. It can avoid the rough surface after the installation of the top seal. The length of the right and left side of the subplate of the top water seal should be extended with 3mm and above in order to keep a certain preset compressive deformation in the joint of the subplate of the top water seal with the corner of the side seal. So the water leakage in this part can be restricted.

ACKNOWLEDGEMENTS

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Harbour Geotechnics: the Case of the Portuguese Small Harbours

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ABSTRACT: Almost all aspects of geotechnical engineering are present in harbour works, namely foundation engineering, stability problems, above and under water, dredging, and the knowledge of sediments features, both geotechnical, key to define the type of dredger to use, and gradation and composition, since the chemical and organic contamination of those sediments define the final destination of the dredged material. But cliffs are also important, as often they overtop the harbour facilities and/or their accesses. One must also emphasized the presence of soft soils, the necessity of their improvement, related to structure and engineered fill foundations. Underwater embankments and their particular construction processes and quality control are also central. This paper presents the main geotechnical aspects of the Portuguese small harbours, detailing three major concerns: dredging, cliff stability and foundation of structures and embankments on soft soils.

INTRODUCTION

Although Portugal is a small country in area, it has a shoreline more than 1,000 km long and, along it, a set of harbours infrastructures, differing in sizes and locations, essential for the country sustainable development. Nevertheless, it´s the small harbours, usually with several hundred of years of use and infrastructures with less of fifty years that require more maintenance and construction renovation each year.

One of the major geotechnical problem in Portuguese harbours, representing a large expenditure each year, is the accretion of sediments inside the harbours' basins and along their channel accesses. Knowing the rate of deposition and gradation, it is possible to plan maintenance dredging to a five years period. But it is also necessary to consider the type of contamination of those sediments, in order to establish their disposal. As contaminated sediments cannot be used for beach deposition or for construction, they must be deposed in an environmental safe area, onshore or not, but clean sediments are too valuable to be disposed off.

In this context, the Portuguese institution ensuring harbour management, the *Instituto Portuário e dos Transportes Marítimos* - IPTM, IP, implemented best practices and environmental techniques to improve efficiency and effectiveness, and to mitigate impacts, either upstream, in the case of estuary harbours, or downstream, selecting and monitoring disposal sites, particularly in the marine environment. Hydrodynamic studies, complemented with the analysis of several years topographic surveys, allows to establish the average rate of deposition in each harbour. The procedures adopted are based on the OSPAR Convention, aiming an integrated environmental management of dredging in harbour areas. It was developed an integrated approach using GIS that establish a basis for environmental and geotechnical quality of sediments mapping for dredging operations and management of dredged materials, implementing a Model for Environmental Dredging of Zones (MEDZ). A insight on this procedure is presented hereafter.

Several of the small harbours present cliff stability problems, both in their road access, as in the interface with the water plan. So the study, monitoring and risk analysis of those cliffs are also a major concern. In a case detailed in this paper, the cliffs are shaped of alternating weak and hard rock layers, with differential resistance to erosion.

Also of great importance is the presence of soft soils, in the foundations of both harbour structures and engineered fills. In this scope, the case of Tavira fishing harbour, south of Portugal, is also discussed. It was necessary to consider the design of both stone columns for the stabilization of the breakwaters and the acceleration of consolidation by preload associated with PVD in half of the embankment area.

Finally, some conclusions are presented.

DREDGING

The Portimão harbour, located at the Estuary of Arade River, close to the city of the same name, presents some of the best conditions in the south coast of Portugal and is a very demanded area for tourist, commercial and the fishing activities. Two jetties protect the harbour entrance, followed by a navigation channel 180m wide (Fig. 1). The harbour administration area goes from the river mouth and extends until some 13 km upstream. A present limitation for this harbour is the channel depth which bottom level is presently at -8m. Considering the needs of the dredging to maintain the operating conditions of this port (300,000m^3 every three years) intended to establish bottom levels compatible with the seagoing cruise ships that will use the harbour, this case presents a model of planning and management of dredging using the geoprocessing automatic GIS environment, that has been developed and applied to ensure a sustainable exploration of this and others IPTM IP's harbour facilities.

Based on georeferrenced detailed topo-hydrographic and physicochemical analyzes of the sediment, identifies the spatial distribution of zones of erosion and accretion, as well as those who have different types of contamination, allowing the division of the same dredging.

The analysis system developed was based on a conceptual model of morphodynamic and multi-temporal sediment in port areas, automating the production of maps (Silva et al., 2010). It includes converting between different data formats, the generation of surfaces of different particle sizes by interpolation methods of several particle size fractions, the zoning of the area under analysis and the calculation of areas and volumes eroded and deposited sediment for each typology (four particle classes: pebble, sand, silt and clay).

FIG. 1. View of the River Arade estuary and the Portimão harbour.

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This system was implemented by applying Model Builder software ArcGIS ® (ESRI) and consists of a set of tools that operate sequentially in the calculation of the various components of the model.

Thus, five modulus were implemented, designated M1 to M5, each with specific processes to obtain automatic digital maps with the desired information, allowing:

- M1 –conversion of CAD entities in vector entities (GIS shapefile), Fig. 2;
- M2 –development of bathymetry digital elevation models (DEM), tin and raster, Fig. 2;
- M3 - interpolation of particle sizes, deterministic method IDW;
- M4 – preliminary zoning of sediments of harbour areas;
- M5 –calculation of areas and volumes of erosion and accretion / dredging;
- M6 - Modulus for calculation of areas and volumes of dredged material.

Studies relating the flow field pattern and tides (FEM) with size distributions in estuarine sediments (MEZD), indicating their source, allow to divide the Portimão harbour area for the purpose of dredging in three main areas, namely:

a) the bar and the navigation channel downstream of the commercial wharf, in which the material to be dredged can be used for artificial feeding of beaches near the bar;

b) The area of the commercial wharf and turning basin (S. Francisco dock), fishing port and downstream end of the shoal, whose dredged sediments should be dropped to six miles from the bar;
c) The area of the shoal, whose sediment features may allow their eventual commercialization.

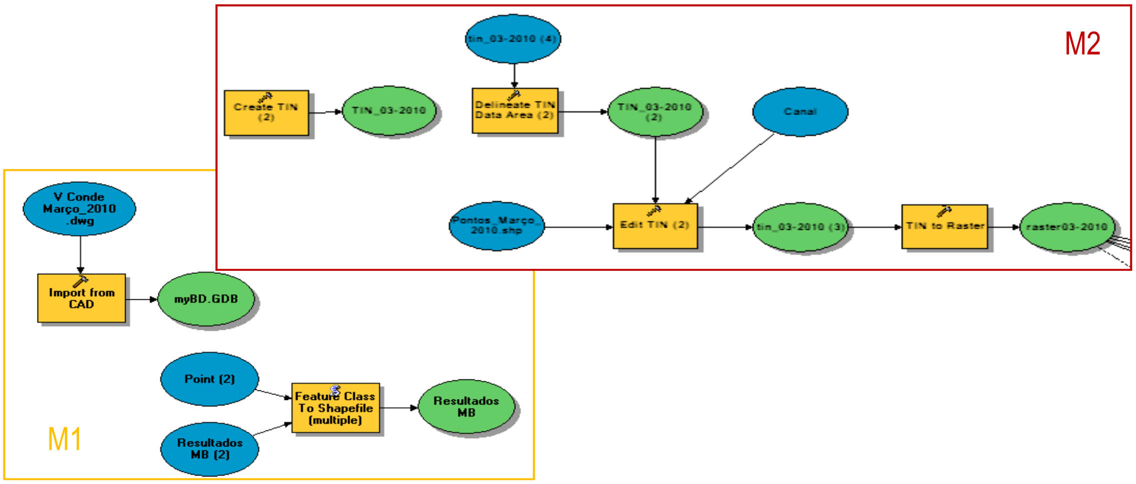


FIG. 2. Modulus M1 and M2 of MEZD.

At present, the sedimentation area upstream the commercial quay is traveling downstream, with a natural increase of the sedimentation rate at the manoeuver area. It is important for the harbour operation to access how the new infrastructures planned will change this situation.

CLIFFS

Rocky stretches of coast are common along Portuguese shoreline, imposing some major geotechnical issues to harbour infrastructures, namely by the high cliffs they give rise to. One of this case, an emblematic fishing and touristic harbour, locates in Ericeira, near the Portuguese centre coastline, where there were identified 8 zones with stability problems (Fig. 2). These cliffs are quite problematic due to (Fig. 3):

- configuration and huge height of the sometimes vertical cliffs, raising in some points above 25 m;
- the geological nature and structure of materials that form them - alternating layers of more or less fractured hard limestones (Fig. 3), sandstones and claystone/marls, these two more easily erodible and destructible, either by direct action of the waves, and, above all, by the action of wind and rain;
- and constant concern of the inhabitants whose houses, now resting on the verge of the cliff, run more or less the risk of destruction due to collapse of the cliff, since under some shallow foundations a few small coves have been identified.

The technical intervention has been several, encompassing a close monitoring of all 8 zones under the jurisdiction of IPTM, IP and, lately, included essentially preventive and remediation works, usually embracing solutions of stabilization by shotcrete, mesh and bolting of problematic blocs that couldn´t be dismantled from the cliff.

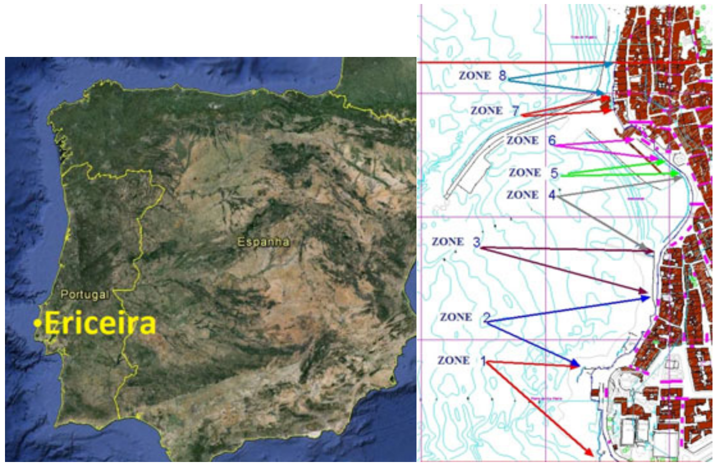


FIG. 2. Ericeira´s 8 cliff zones with stability problems.

Nevertheless, some retention walls were built in the past and one of them, a century and a half old retaining wall (1853), supporting the south access road to Ericeira fishing harbour (Fig. 3), was founded on a hard limestone layer that collapsed. The foundations presented alternating layers of limestone, 0.5 m to 1.0 m thick, and marls, 0.1 to 0.50 m thick. The base of the cliff was protected by rocky debris. The effect of sea breaking against the base of this cliff was enough to erode the marly layers in a depth of 4 to 5 m. As a consequence, the lower layer of limestone, left in balance, did not had enough strength to support the retaining wall weight, and collapsed.

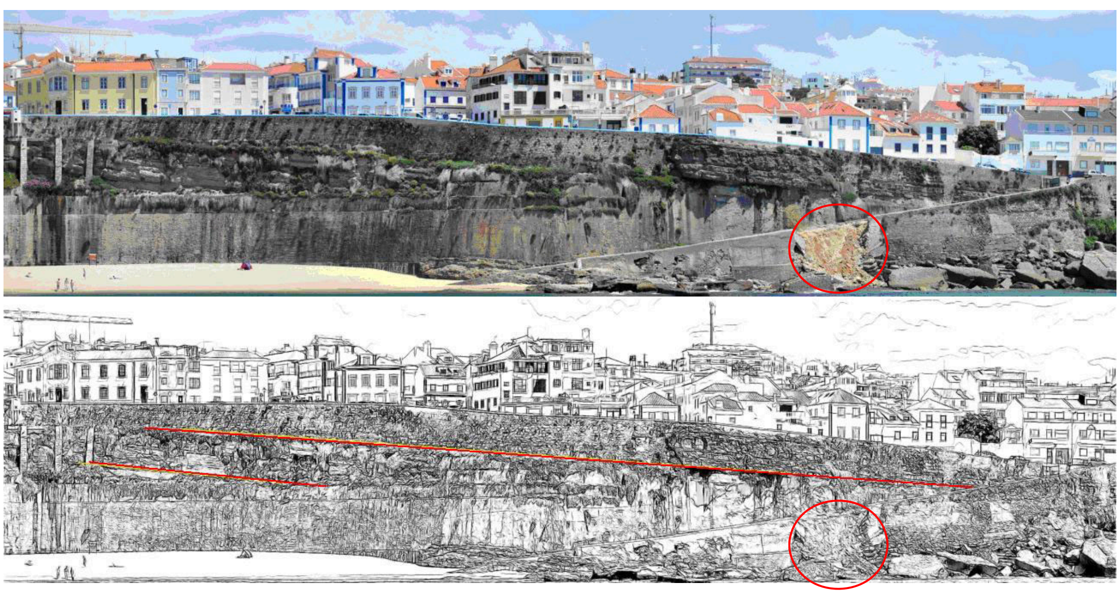


FIG. 3. Aspect of the cliffs at Zone 3 (Fig. 2) located above and under the access road to the harbour beach (centenary retaining wall rupture highlighted by a red circle); hard carbonate layers underlined with red lines on the sketch.

The preventive measures presently implemented included the monitoring of the cliff located over the harbour facilities or above their accesses, either by visual inspections, either by precision measures, since this cliffs were three years ago submitted to laser scanning and there is a referenced situation to compare with the natural ongoing evolution processes.

SOFT SOILS

Most of the Portuguese small harbours are located in estuaries, and so soft soils are often present. These soils in an harbour can represent two different types of problems: first, in the navigation channels and manouvering and anchoring bassins, they represent usually material that needs special care in its disposal, when there is necessity of dredging, either maintenance or first establishment; second, they present a low strength and high deformability layer, inadequate for foundation of most of the harbour's structures. In this section, it is the second effect that will be analyzed.

The new harbour planned for Tavira, south coast of Portugal, presents a soft to very soft clay layer. According to the project layout (Fig. 4), most of the harbour structures present no limitation to the presence of this layer. Anyway, the need to implement the breakwaters as vertical quays, due to environmental limitations, with a small accepatble differential settlement and relatively high load on the foundations, imposes soil reinforcement under these structures.

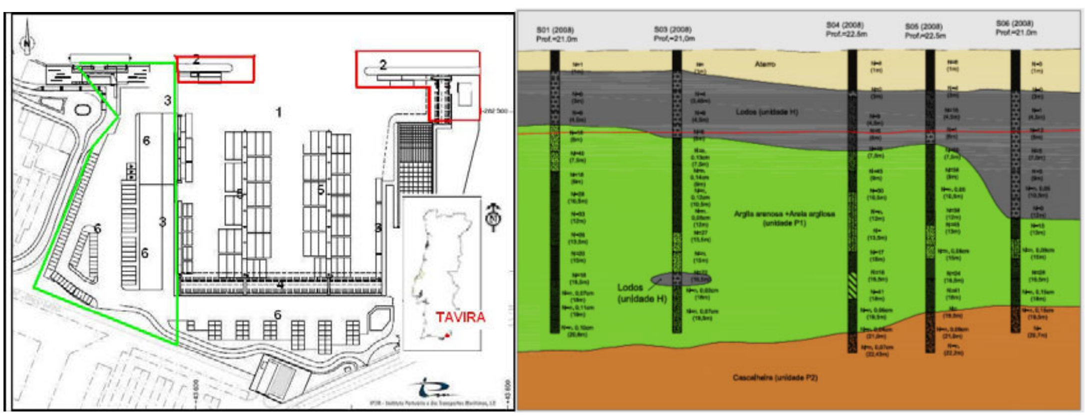
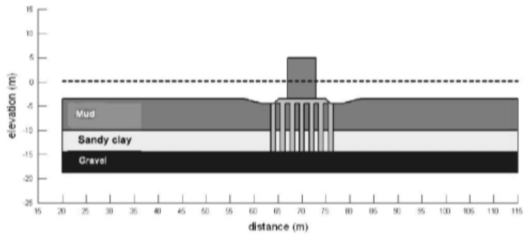


FIG. 4. Layout of the Tavira fishing harbour, with a geotechnical profile along the breakwaters; in grey, soft to very soft clays, in green clays, clayey sands and sandy clays, and brown are gravels, with N_{SPT} greater than 60.

In the same harbour there is also a different situation, in the area of a building foundation, parking lot and circulation areas. Here, only the expected settlements are a concern, and so a soil improvement technique is used. Fig. 4 presents the harbour layout and has marked the two intervention areas – in red the reinforcement of the quay walls and in green the building and parking lot area soil improvement.

Soil reinforcement under the breakwaters

A FEM analysis (Figs. 5 and 6) assessed a total amount of 13 cm for the maximum settlement after soft soil reinforcement by stone columns, against 54 cm without reinforcement, and since part of it should occur during construction, it is an acceptable value for the rigid vertical quays. This value is quite similar to the one obtained with Priebe method (15 cm, see Table 2). This stress deformation analysis emphasizes the role played by the stone columns, transferring the surcharge to the deep, frictional, more resistant geotechnical unit (GU4), Fig. 5. Santos-Ferreira and Santos (2011), have demonstrated that the structure is safe, both under static and seismic conditions



Geotechnical unit	soils	layer depth (m)	N_{SPT}	γ_s (kN/m³)	φ (°)	c_u (kPa)	Young modulus E (MPa)	Poisson ν
GU1	earthfill	-0.50	3	17.3	25	2	20.0	0.33
GU2	mud, sandy mud and muddy sand	-10.0	5	16.0	20	1	1	0.40
GU3	sand, sandy clay and clayey sand	-14.5	25	19.8	30	5	15	0.30
GU4	gravel	-18.8	60	20.5	37	0	170	0.29

FIG. 5. Geotechnical profile and solution in stone columns for the breakwaters foundation reinforcement and the geotechnical parameters used in FEM.

Table 2. Stress increase and settlements prediction for the foundation of the east quay by Priebe method

Geotechnical unit	Reinforced soil φ_{sol} (°)	Reinforced soil c_{sol} (kPa)	Stress increase (kPa)	Settlement without reinforcement (cm)	Settlement with reinforcement (cm)
UG2	34,6	0,3	47,6	62,9	13,9
UG3	36,7	1,8	29,8	1,7	0,5
UG4	--	--	21,6	0,5	0,5
Total settlement				**65,2**	**14,8**
Global security coefficient					2,62

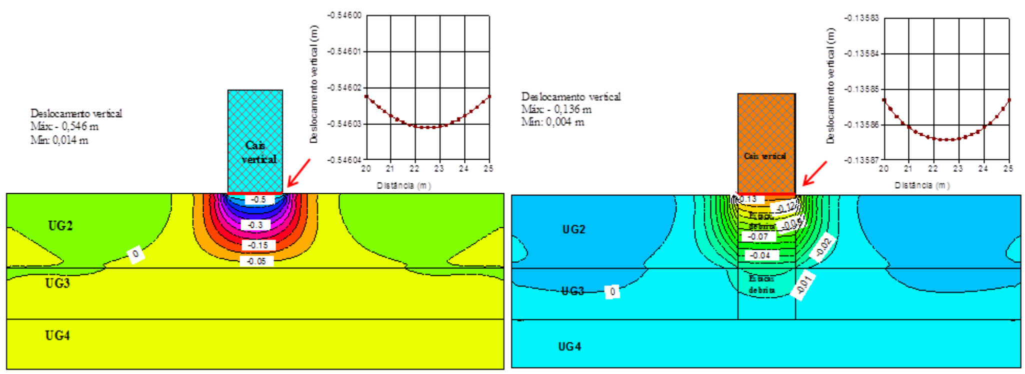


FIG. 6. Settlements by FEM analysis at the base of the quay, without and with soil reinforcement, respectively maximum of 54 cm and 13 cm.

Soil improvement in the building and parking lot area

Considering the type of building, in reinforced concrete with a slab foundation, the total allowable settlement was defined as 15 cm. As the soft clay layer will have a maximum thickness of 10 m (Fig. 5), the expected settlement would be largely superior to the allowable value. So, an improvement by preload and PVD of the soft soil was decided, in order to force most of the settlement to occur before the building construction. The grid of PVD used is triangular with 1.8m side. Although a 3D analysis of the PVD settlement was retained for the stress field in the soil, the settlements were calculated considering a 1D traditional analysis. The results are summarized in the graph on the left of Fig. 7, showing the settlements and heaving after the precharge removal, and the settlement after the building construction: from day 0 to day 365, under a 2.5 m sand preload, considered to stay in place for one year; at day 366 there is the removal of the preload, till day 450; construction of the building is assumed instantly at day 451. As shown in this chart, the settlement after the construction of the building does not exceed 12 cm, value that is adequate to this type of construction. The right side of Fig. 7 displays the equal settlements preview for 300 days after the building construction, in plan and under the center point of the building.

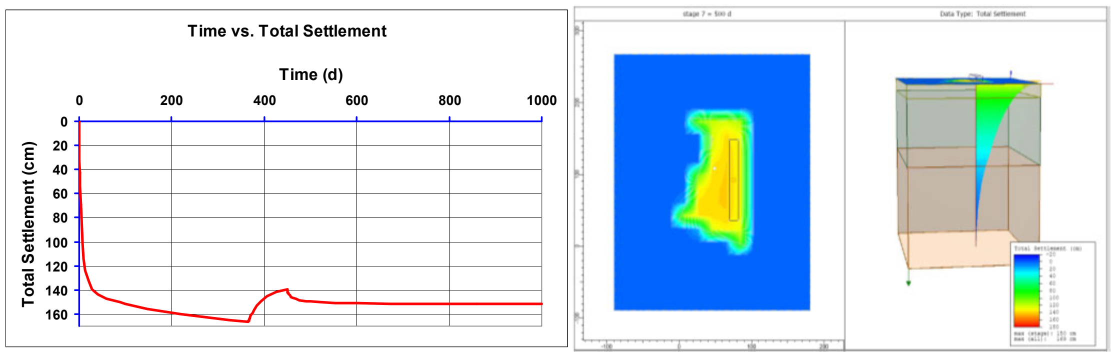


FIG. 7. Settlements evolution with a PVD improvement before and after the building construction.

CONCLUSIONS

As a final remark, small harbours usually need the use of most of the geotechnical expertise necessary for larger harbours, namely in their design, construction and maintenance. This was underlined by the three case studies presented where tools like GIS and FEM were used to model and predict the behaviour and the necessity of soil improvement and reinforcement works in scope of foundations and slope stabilization.

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