BREAKWATERS ON VERY SOFT SOILS. DESIGN, CONSTRUCTION AND GEOTECHNICAL MONITORING FOR FIUMICINO NEW SEAPORT

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INTRODUCTION

Rubble mound breakwaters are characterized by their considerable mass, as they generally consist of a quarry-run core, rock filter layer(s) and external armour layer made of rocks or concrete blocks.

The weight (load) of the breakwaters is fully borne by the foundation ground. Since these structures are made at sea, foundation ground aligns with the seabed, which can be either natural or requires preliminary dredging. Consequently, the seabed and all subsoil layers experience a significant increase in tension. Depending on the nature and mechanical properties of subsoil, this could potentially lead to relevant geotechnical issues, ranging from lack of slope stability to large settlements.

If thick layers of soft soil (e.g., normally consolidated clay) are present, critical settlement behaviour might be expected. Therefore, specific design solutions must be implemented. Due to the challenge of reducing the total amount of settlements, significant efforts are directed toward accelerating and concentrating settlements as much as possible during the construction phase.

Design and (still ongoing) construction of the 630 m long main breakwater of Fiumicino (Rome, Italy) new seaport face all these issues, as it lays above a thick layer of soft clay. It is important to underline that the new port is situated on the Northern side of Tiber River delta. Therefore, geological and geotechnical conditions are the results of thousands of years of sediment transport and deposit by the river that flows across the city of Rome just a few kilometres upstream. The coastline in the Tiber delta has been steadily advancing for more than 20 centuries, covering over 3.5 km, until the 60-70s, when the amount of solid sediment transported by the river significantly decreased with consequent coastal erosion and protection works.

SITE CONDITIONS

Two important geognostic campaigns were performed in the area in 2012 and 2020 during the development of preliminary and detailed design. The first campaign directly investigated the breakwater footprint, while the second focused more on the reclamation areas along the seashore. Older data (1998, 2005, 2008) were also available.

The 2012 campaign consisted of 15 boreholes and several Cone Penetration Tests (CPTs). Additionally, in 2020, 4 boreholes, 2 CPTs and 4 dilatometer tests were performed. Fig.1 shows the locations where these tests were conducted. Please note that the port's general layout from the preliminary design (purple) is significantly wider than the detailed design (light blue), whose construction in now ongoing (2023-2026). That is just the first step (fishing vessels dock) before the completion of the whole project.

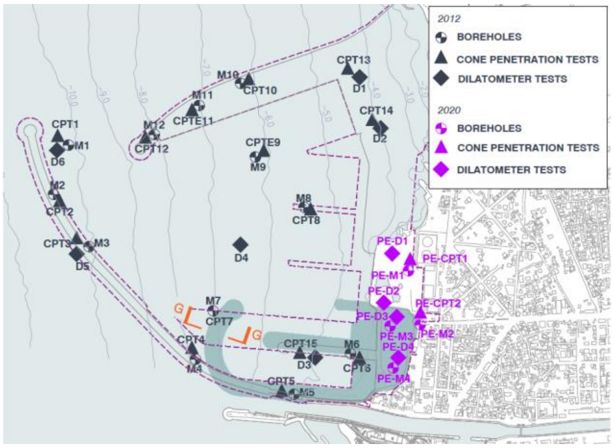

Figure 1 – Layout of Fiumicino (Rome, Italy) new seaport

Results showed that three soil layers can be defined.

- Upper layer made of **sand** and **silty sand** (A). The thickness of this layer significantly decreases from onshore to offshore. Nearshore a thickness of around 10 m of sand is found. At the breakwater head it reduces up to 4 m. Other boreholes show that, moving offshore, the sandy layer tends to completely fade.
- Below the sand, a thick layer (30-35 m) of **silty clay** (B).
- A stiff bottom layer of **granular material** (C) is shown from -35 m onward, below the sea bottom. Overall thickness of this layer is unknown.

As expected, A and C exhibit good to excellent mechanical properties (bearing capacity), with the exception of the top of sandy layer. Unfortunately, the design had to face the 30-35 m thick, soft/weak clayey layer.

In more details, B is characterized by a unit weight between 16.0 and 18.5 kN/m^3 (average 17.5 kN/m^3). The void ratio naturally decreases with depth, with an average value of 1.0. As to the Atterberg limits, plastic index varies between 10% and 40%, while the water content equals the liquid limit and occasionally exceeds it.

As to mechanical properties, silty clay shows low shear strength, with soil cohesion (c') not exceeding 5 kPa and an average angle of internal friction (φ') at 23°. The values for undrained cohesion range from 15 to 30 kPa, highlighting that silty-clayey layer can be considered as **normally consolidated**.

Oedometer tests show very low consistency index, between 0.3 and 0.5, with a decrease in value with depth. Moreover, average swell index (Cs) is about 0.007, while oedometric modulus (Eed) is ≤ 5 MPa. The coefficient of consolidation (Cv) varies between $1.6*10^{-8}$ and $8.1*10^{-8}$ m^2/s, though the more recent (2020) campaign highlights notably higher values (Cv between $1.7*10^{-8}$ and $3.2*10^{-7}$ m^2/s).

ADOPTED DESIGN SOLUTIONS

Due to the weak mechanical properties of the subsoil, several design solutions for geotechnical reinforcement have been applied, as indicated below:

a) **Soil replacement**: the upper 1.20 m of low-density sands of A layer was excavated and fully replaced by quarry run.

b) **Soil improvement**, by installing geo-composite prefabricated vertical drains, 95 mm wide, 4 mm thick and 17.70 m long, placed according to a specific geometrical pattern with a spacing of 3.5 m. Those elements are made of a polypropylene core covered by a geotextile polypropylene filter. The installation of geo-composite prefabricated vertical drains significantly accelerates the consolidation process of the clayey layer and related settlements.

   Gravel piles could also be an alternative, but their realization was rejected for both technical and economic reasons. Low values of soil shear strength may cause horizontal cuts along piles, significantly reducing their effectiveness. Additionally, the construction of gravel piles is generally more costly than the adoption of geo-composite prefabricated vertical drains.

c) **Basal reinforcement**, obtained by posing a monoaxial geogrid at the bottom of the excavation (see point a)) for the stability of breakwater. Geogrid is made of polyester stripes with high tensile strength, up to 1300 kN/m (see fig.2).

Three construction steps were defined for the breakwater:

1) Building core, filter and armour layers up to +0.3 m above water level.
2) Building core, filter and armour layers up to +4.8 m above water level.
3) Completion of the breakwater crest, up to +7.9 m above water level.

Among these three steps, construction is expected to pause for 90 days, to facilitate the gradual dissipation of water pressure and consolidation process of the silty clay. Steps were designed with the aim to make sure that safety factors for undrained conditions were respected during breakwater construction.

Numerical modelling was widely implemented for breakwater's geotechnical design. For both settlements and slip surfaces analyses PLAXIS 2D model was used.

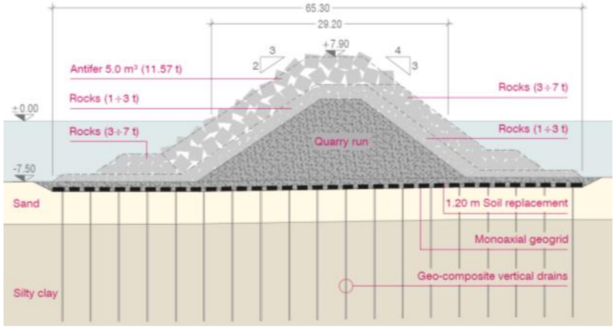

Figure 2 – Typical breakwater cross-section

RESULTS

For each cross-section and construction phase, vertical displacements (settlements) were estimated. At G-G' cross-section (fig.2-3) the following values were obtained.

End of step 1	End of step 2	End of step 3	End of consolidation
0.26 m	0.51 m	0.56 m	1.36 m

End of step 3 represents undrained conditions as it precisely coincides with end of breakwater construction. Once the consolidation process ends (about 33 years later), an additional settlement of 0.80 m is expected. Hence, **1.36 m vertical displacement is expected to occur**.

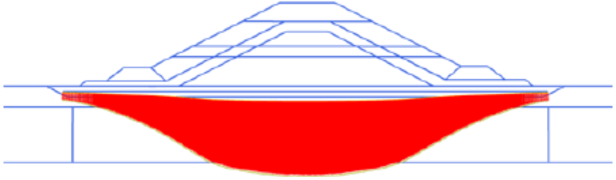

Figure 3 – Vertical displacement field (from PLAXIS 2D)

GEOTECHNICAL MONITORING

When works were contracted, the Client (Port Authority) and the Contractor agreed on developing an extensive geotechnical monitoring campaign, under the guidance of Acquatecno, the engineering company in charge of design and supervision of works.

A 60 m long detached portion of breakwater is built (in steps) where subsoil conditions are most critical. The goal of the experimental campaign is to collect measured field data to compare with estimates from numerical modelling. The data come from a complex system of geotechnical monitoring instruments, such as electrical piezometers, inclinometers, settlement gauges, etc, based on an accurate monitoring plan.

CONCLUSIONS

From numerical modelling estimations, by splitting construction into steps, more than 40% of total vertical displacements (0.56 m) occur before the end of breakwater construction. The required final design crest height (+7 m) is obtained by enlarging the cross-section, at the end of works, by only +0.8 m vertically. It is significant to underline that, if geotechnical reinforcement (a, b, c) were not adopted, settlement development and stability safety factor would have been remarkably more critical, making the construction of the breakwater simply unfeasible. Ongoing "field" geotechnical monitoring campaign may confirm/validate predictions or highlight some bias between measured and estimated values of vertical displacements. Results (to be shown at Conference) can improve settlement predictions, according to a back analysis procedure, and lead to relevant design optimization and construction phasing.

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