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S-79

PROJEKTOVANJE I IZGRADNJA LUKE NAUTIČKOG TURIZMA U OKVIRU KOMPLEKSA PORTONOVI

Rezime:

U Herceg Novom je u toku izgradnja luksuznog turističkog kompleksa Portonovi u Kumboru, koji će se, između ostalog, sastojati i od Luke nautičkog turizma. Luka obuhvata marinski basen i formiraju je dva gata fiksne dužine 200 m, mjereno od postojeće obalne linije. Projektovanu konstrukciju gata čine montažni elemenati koji se oslanjaju na bušene šipove. Prilikom projektovanja uzeti su u obzir geotehnički uslovi, seizmičnost lokacije, trajnost u morskoj sredini, itd.

Ključne reči: radovi u marini, luka nautičkog turizma, likvefakcija

DESIGN APROACH AND CONSTRUCTION OF A NEW PORT OF NAUTICAL TOURISM IN PORTONOVI

Summary:

The municipality of Herceg Novi projected a wide development known as Portonovi in Kumbor, which is a world class resort with a Port for nautical tourism. The port is formed through the construction of two main fixed jetties which extend 200 m in length from the existing coast line and encapsulate the marina basin. The new jetties are designed as pre-cast decks supported by concrete bored pile; the design takes into account problems related to geotechnical conditions, seismicity, durability in the sea water.

Key words: Marina works, port of nautical tourism, liquefaction

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1 INTRODUCTION

As part of the restoration of degraded areas of Montenegro and their integration into the urban system, the municipality of Herceg Novi projected a wide development known as Portonovi in Kumbor, which is a world class resort located along the coast in Kotor bay with a Port for nautical tourism.

Portonovi Marina has been designed in order to accommodate boats from 8m up to 110 m in length, and to be fully functioning in all weather conditions. The port is formed through the construction of two main fixed jetties which extend 200 m in length from the existing coast line and encapsulate the marina basin.

Both jetties have a width ranging from 12 to 20 m and will act as a breakwater structures. They are conceived with a sufficient width to allow vehicular access and to accommodate the following specific facilities: support buildings for port operation management, electrical and mechanical plants, refueling.

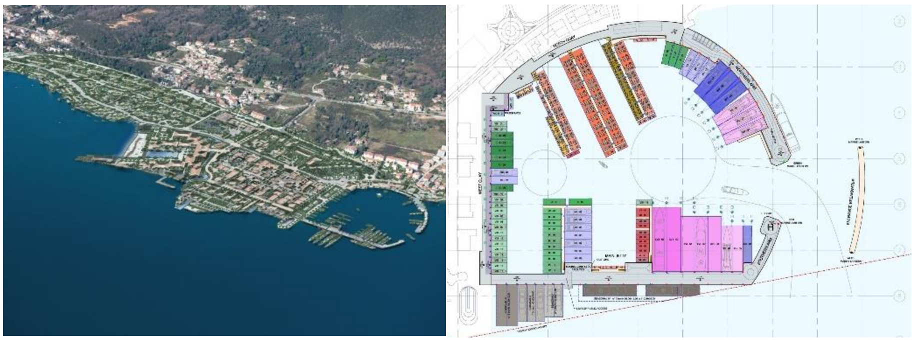

Figure 1. Portonovi resort: Masterplan and Berth arrangement

2 STRUCTURAL LAY-OUT OF JETTIES AND QUAYS OF THE NEW PORT

The new jetties and the quays are designed as opened concrete bored pile-supported decks, taking into account all problems related to geotechnical conditions, seismic activity in the area, durability of structures in the sea water.

Decks are constituted by a grid of precast beams completed by a cast in place concrete slab 30 cm thick.

Concrete structures are suspended on cast in situ concrete bored piles, executed with the aid of a drag bit and a permanent hollow-section steel casing; diameter of concrete piles varies from 600 mm to 800 mm in diameter to support the berth structure. Piles length varies roughly from 15 to 35 m.

Typical transversal dimensions of jetties and quays vary between 12 and 20 m; the rows of piles are spaced 6-8 m and are composed by 3 up to 6 piles each row.

Jetties are provided with transversal joints every 80/100 m; typical sections for jetties and Quays are shown in the following figures.

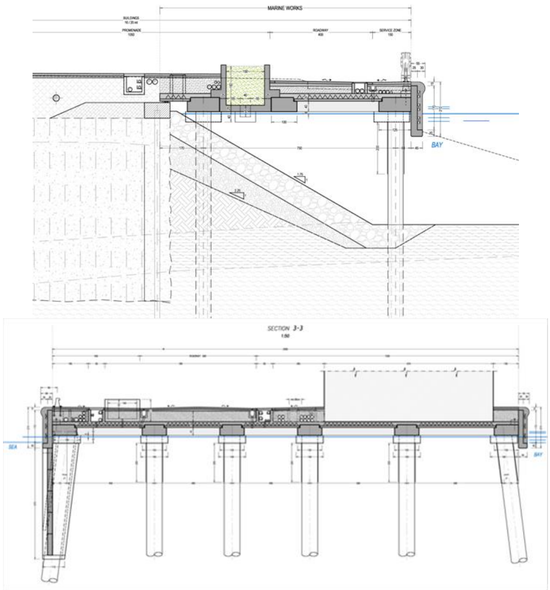

Figure 2. Quays and Jetties: typical section

3 SITE WIDE GEOTECHNICAL AND GEOLOGICAL CHARACTERISTICS

3.1 STRATIGRAHY

The ground conditions at Portonovi site consist of a bedrock overlain by sequences of Quaternary, or recent soil deposits having different origins. Site-wide ground investigation has been performed both into land and marine areas.

The bedrock consists of Flysch, a “weak rock”, which comprises inter-bedded layers of mudstones and sandstones. The Flysch bedrock has weathered over time to produce a residual clayey soil (HWF) which occurs as a “blanket” over the Flysch bedrock.

The uniaxial compressive strength of Flysch is within the range 3.6 to 10.8 MPa, The most frequent values of the uniaxial compressive strengths are in the range 4.5÷6.0 MPa. The calculated RQD values obtained from marine investigation in the first 15 m of the geotechnical formation vary mainly in the ranges from 0 to 40% with most frequent value (48%) in range 20-40%.

Quaternary Deposits, often referred to as “Terra Rossa”, are eluvial soils derived by the weathering of the limestone present higher up the mountainside above the site. Marine Deposits are alluvial material laid down over parts of the site by the effect of fluctuation of the sea. Man-made soils are present in more superficial stratum because of the reclamation efforts. Not cohesive strata have a friction angle varying in the range 28°-35°; the maximum value c’ is equal to 20-30 kPa in the cohesive strata.

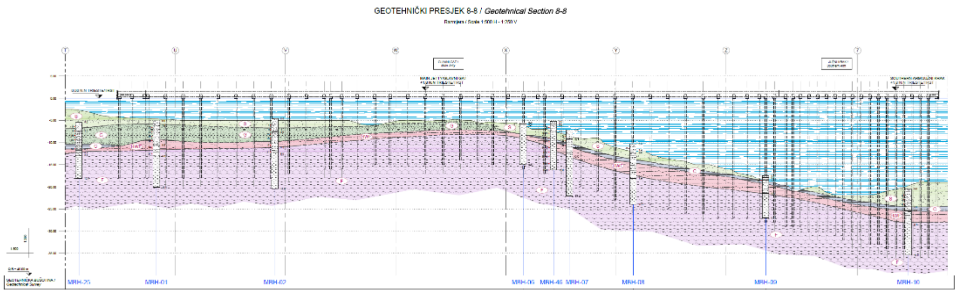

Figure 3. Geotechnical section along the Main Jetty

3.2 SEISMICITY AND LIQUEFACTION PROBLEMS

The site is located in the part of Montenegro with the highest seismicity. The most recent major earthquake took place in 1979 in Budva located approximately 56 km from the site and measured a magnitude of 7.0 on the Richter scale. Three further earthquakes with magnitudes between 4.0 and 7.0 have been recorded in 1979, 1984 and 1992 within 100 km radius from the site. In the Marina area the expected peak ground accelerations is 0.28g. In the zones where marine deposits have a thickness greater than 4÷5 m this value was amplified to take in account a local amplification of a seismic event.

Due to the high seismicity and the presence of fine loose materials under the sea water level, liquefaction analyses have been performed in order to verify the effective risk in the Marina area and to quantify the extent of the liquefiable layers.

The factor of safety (FS) against liquefaction triggering has been calculated from SPT, CPT and Shear Wave velocity (VST) results, following the recommendations of Idriss and Boulanger (2008); in particular, the calculation of the stress reduction factor rd has been performed using the NCEER method. The obtained results demonstrate that potentially liquefiable soils are present mainly in the upper sand layers. The maximum depth of liquefaction obtained from the in-situ tests examined is variable over the whole area; in the design analyses all the cohesionless strata above the HWF and above cohesive units were assumed liquefiable.

4 FOUNDATION PILES

4.1 DISEGN CRITERIA

Structural numerical models were developed in order to design all the structures and the piles; the models analyze the behavior of Jetties and Quays under design loads by means of linear elastic analysis. Refer to § 6 for structural analysis details.

The geotechnical verifications included:

- evaluation of horizontal pressures on piles due to liquefaction occurrence;
- evaluation of negative skin friction on piles due to settlements after liquefaction event;
- calibration of the parameters for the linear elastic constrains to be included in the numerical models in order to consider the interaction between piles under horizontal loading and the surrounding soil;

- development of the axial bearing capacity curves (compression and tension) for piles with diameter Ø600 mm and Ø800mm;

The depth of potential liquefiable soils is estimated for each section of Jetties and Quays; each pile in the structural model is loaded with an equivalent force equal to 30% of lithostatic pressure along the pile (as suggested in JRA, 1996-2002). The liquefaction phenomena is not combined with maximum seismic peak acceleration according to main literature references – see figure below (cfr. Subhamoy Bhattachary, Domenico Lombardi – 2012). All verifications have been carried out according to Eurocodes and to API RP WSD 2007 recommendations.

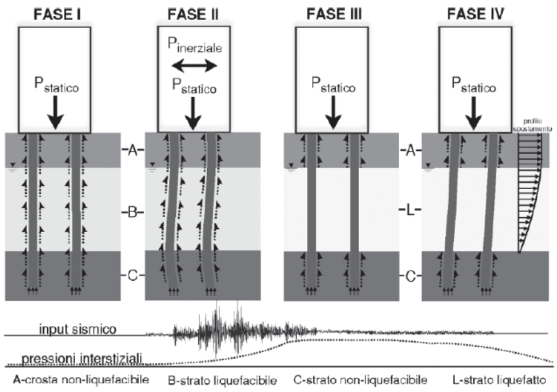

Figure 4. Different stage of loading on pile foundation during an earthquake- qualitative trend of the excess pore pressure due to liquefaction

Deep foundations of decks extend through potentially liquefiable sand layers near the seabed surface and are socketed in more competent layers (Flysch bedrock). When liquefaction occurs, the skin friction in the liquefied layer would be expected to decrease to zero and, as the liquefiable layer settles, negative skin friction could develop around the pile in this layer. Vertical ground settlement will occur as excess pore-water pressures induced by liquefaction dissipate, resulting in down-drag loads on and loss of vertical support for deep foundations.

The down-drag load (or skin friction) within a liquefied soil increases over time as excess pore water pressures dissipate (effective stresses increase) during the sand reconsolidation process after liquefaction. The down-drag load has been calculated assuming the conservative hypothesis that ru=0, corresponding to the end of the reconsolidation process. Down-drag loads are a consequence of seismic shaking, then they should not be applied in conjunction with the design seismic loads, because they will not occur at the same time. The sum of the expected axial pile load and down-drag load from the liquefied layer has to be less than the pile bearing capacity.

4.2 CONSTRUCTION APPROACH AND EXPERIENCES

Concrete bored piles were preferred to driven piles, due to the presence of the aforesaid flysch bedrock under sea level, which comprises inter-bedded layers of mudstones and sandstones. As highlighted in § 3.1, the uniaxial compressive strength varies within the range 0.6 to 12.5 MPa and piles are socked into the Flysch at least 5-6 m, as defined by calculations. It was not possible to further limit the depth of embedding of the piles tip, because the alteration of superficial Flysch strata cannot be predicted and piles work under relevant compressive forces. Bored piles were executed by sea using pontoons equipped with crawl, hammer, drilling tools or by earth, where possible along West and North Quay. The figures below shows the piling works by sea and by earth.

The use of vertical piles in lieu of battered piles was considered the optimal solution, both for design and construction purposes, taking into account that:

1. all standards and codes (i.e. Eurocodes, Italian NTC 08. § 7.2.5), and specific recommendations for piers and wharves such as Polb 2009 (§§ 4.2 e 5.5.4) and Pola Seismic code 2010 (§ 1.4.1 pt. C) state that battered piles should avoided - if possible - in order to increase safety of constructions in seismic zone;
2. vertical piles offer also the possibility to simplify construction sequences with not negligible time and cost savings.

There were no relevant problems during execution with a daily production up to 3 piles within a day by rig installed on site.


Figure 5. Piling works from pontoons and along the future quays

Structural numerical models were developed in order to design all the structures and the piles; the models analyze the behavior of Jetties and Quays under design loads by means of linear elastic analysis. Refer to § 6 for structural analysis details.

5 QUAYS DESIGN

5.1 DISEGN CRITERIA

The Quays were designed as opened quays and not linked to the ground behind them; a longitudinal joint separates quays from ground retaining structures. For this reason, piles were designed in order to withstand forces caused by self – weight and live loads applied on the deck and not for static and seismic ground pressures.

Ground behind quays was preliminary improved with specific deep soil mixing tecniques (cfr. § 5.2) in order to create a continuous diaphragm wall.

The MIP wall works as a retaining structure to allow dredging necessary for the port, but at the same time it is conceived also as a liquefaction countermeasure against lateral spreading.

In fact, the cells of the containment wall isolate and enclose the potential liquefiable soil during an earthquake event in such a way that the soil contained within the grid cells will not liquefy. As a result, the wall constitutes a stable block capable to withstand the lateral forces due to the spreading of liquefied soil outside the structure itself.

The open quays solution was proposed, instead of gravity block walls initially designed, for several reasons:

1. the solution on piles is suitable in a port for nautical tourism in order to mitigate the effects of wave reflection inside the basin and to guarantee the optimum conditions for vessel berthing;
2. along the north quay the bedrock depth is continuously increasing in the east direction up to 15-16 m. The marine deposits above this bedrock are subjected to liquefaction. A design solution with a block wall founded on rocky strata should be not feasible or in any case of high impact on construction time and costs. The following figure shows that for values of ag/g higher than 0.25 to 0.35 the minimum width B of the block wall is greater than the height and the work becomes inconvenient from the economical point of view;
3. if the concrete wall was founded on the marine sediments and not on bedrock, it would require a ground improvement under foundation extended also towards sea in order to avoid liquefaction and slope instability with related significant costs and time for execution;
4. due to the presence of a longitudinal joint along the quays, movements or liquefaction phenomena will not affect quays behavior during an earthquake;
5. it is possible to use a single structural solution for both jetties and for quays with optimization of times and execution costs;
6. the proposed solution allows to separate marine works and ground improvement works behind quays.

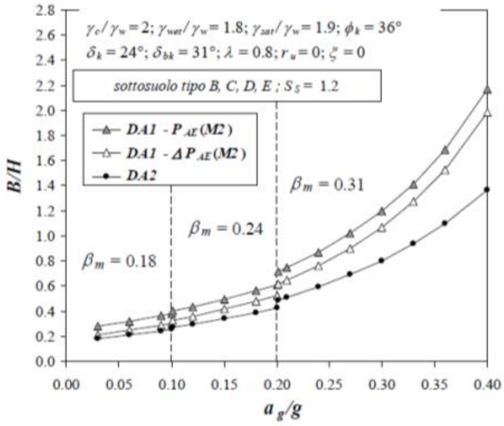

Figure 6. B/H ratio for walls vs ag/g values

5.2 CONSTRUCTION APPROACH AND EXPERIENCES

The ground improvement along the quays was executed by means of deep mixing. The heavy-duty drilling rigs used for the site were equipped with three parallel continuous flight augers each with a diameter of 0,55 m. To execute the panels, the triple counter-rotating auger unit drills the ground whilst binder slurry is simultaneously injected. Once final depth is reached, the soil-cement mixture is homogenized by alternating rotation of the individual augers and concurrent upward and downward movement of the entire auger assembly, depending on soil-cement properties (Figure 7).

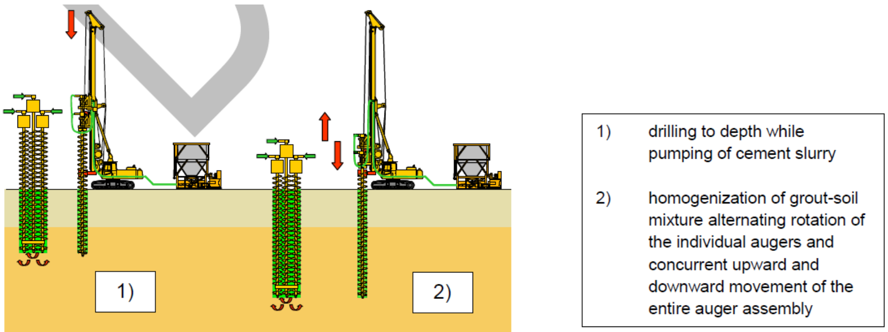

Figure 7. Production sequence for MIP

The result will be a continuous body of cemented soil, with dimensions defined by the geometry of the auger unit: a length of 1,70m (as measured from the outside of the outer augers) and a width of 0,55 m corresponding to the auger's diameter. When the panels form the continuous wall along the Quays the elements are performed fresh to fresh Before hardening of the panels, steel beams were installed in order to guarantee the necessary bending stiffness to the ground improved panels (Figure 8). All these steel beams were finished with a continuous reinforced concrete capping beam (Figure 9). The ground improvement works were completed on site with great efficiency, proving the advantages of the design choice compared to other conventional techniques (gravity wall, concrete diaphragm wall and so on).

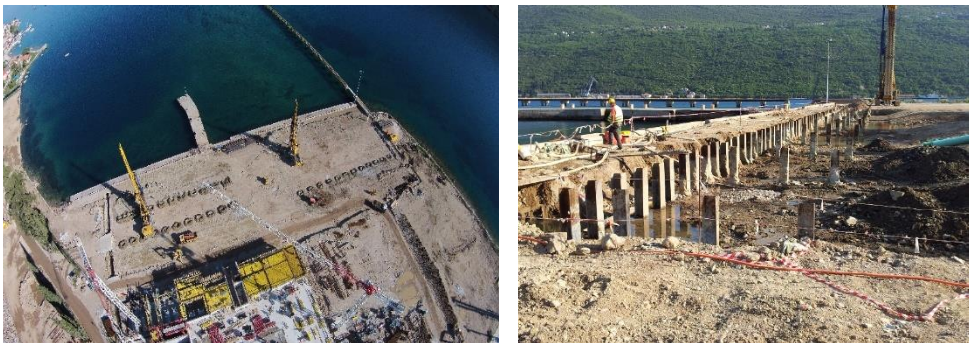

Figure 8. Ground improvement works along the shore line with deep soil mixing.

Figure 9. Capping beam of the future West Quay Wall

6 DESIGN OF DECK STRUCTURES

6.1 DISEGN CRITERIA

6.1.1 Structrual analysis

In order to verify deck structures and piles, 3D finite elements models were developed for each part between two structural joints (structural analysis were performed with the aid of the software Sap 2000): piles and deck beams are modelled by means of beam elements while deck slab is modelled using plate orthotropic elements to take into account the mono-directional behavior of concrete slab on planks. Spring elements, which stiffness was evaluated using geotechnical data, are assigned at pile tips socketed into the flysch (F) strata.

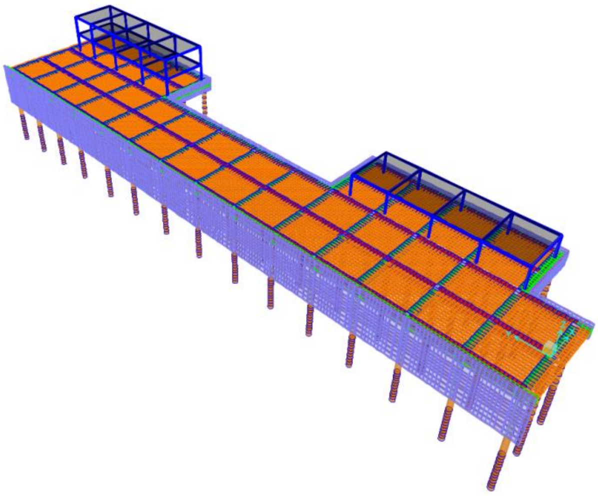

Figure 10. Example of Sap Model for structural analysis

6.1.2 Corosion protection strategy

Corrosion of structures in the marine atmosphere and in the splash zone was achieved using the following strategy:

1. **Bored concrete piles protection:** re-bars in the concrete piles are protected by permanent casing and by means of a proper cover; the head of the casing in the splash zone (the most critical one) will be coated with an epoxy layer;
2. **Concrete elements protection:** the corrosion of reinforcements is prevented by:
   - ✓ using a concrete C40/50 with an exposure class XS3, providing an adequate concrete non less than 6 cm.
   - ✓ requiring a strict crack width limitation
   - ✓ using galvanized reinforcements for structural elements in contact with the wave splash (splash zone) and up to a thickness of 100mm from the face exposed to seawater.

7 CONCLUSIONS AND ACNOWLEDGES

Technical solutions designed for the new port of nautical tourism in Portonovi were described: solutions were specifically developed in order to face the challenging conditions related to geotechnical aspects, liquefaction, seismicity, durability, project requirements. Experiences on site demonstrated their validity and feasibility in terms of construction site organization time and cost savings.

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LITERATURE

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