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Zagreb Pier Container Terminal Project in Rijeka – Croatia: Earthquake engineering and geotechnical aspects

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ABSTRACT: This paper aims to describe the earthquake and geotechnical aspects of design of the Zagreb Pier Container Terminal Project in Rijeka. This new facility will increase the competitiveness of Rijeka as a port City, realizing a new quay with a total length of 680 m (to be achieved in two phases). The new structure is conceived as a semi-open quay, realized with precast caissons laying on a submerged embankment. The characteristics of the foundation soil are particularly severe, on site, due to the presence of a thick layer of soft cohesive soils, overlaying a rocky substratum. A soil improvement was therefore carried out by executing stone columns and jet-grouting. As part of design a Probabilistic Seismic Hazard Study (PSHA) was carried out, to overcome possible limitations of code spectra and 3D non-linear dynamic analyses in the time domain were developed to precisely asses seismic actions.

1 FOREWORD

1.1 *Short historic overview*

The port of Rijeka (in Italian Fiume) has an ancient history: it is cited by the sources since the 13th century but acquires a key historical role in the 18th century, when it became the largest seaport of the Kingdom of Hungary (1779). After the annexation to the Kingdom of Italy, Rijeka becomes a peripheral port and loses most of its traffic. From 2000 however, the maritime trades of the world routes that from Asia, through the Suez Canal and the Mediterranean, reach Europe and North America have drastically increased. The access to this market is therefore of strategic importance for the growth of an entire area.

This scenario explains the origin of the Rijeka Gateway Project: a development plan aiming to extend the port infrastructures with the construction of the new Zagreb Pier Container Terminal. Thanks to the significant depth of the water (up to about -40m) and to the well-protected position characterizing the port area, the new plan foresees the realization of a new large quay, with a total length of 680 m (to be achieved in two phases), which allows to extend the handling capacity of container ships of the latest generation. For this purpose, the Port Authority of Rijeka has launched an international tender, according to the FIDIC procedures (International Federation of Consulting Engineer), for the design and construction of the new infrastructure. Given the environmental constraints in the site area (linked to the high depth of the seabed and to the poor characteristics of the foundation soil), the winning solution foresees an innovative use of the reinforced concrete cellular caisson technology.

2 PROJECT

2.1 *Project description*

The construction site is located within the area of the existing Port of Rijeka, representing its western extension. The maximum water depth on the quay front is about - 20m below low water level and the new infrastructure will permit to host ships with a Length Overall from 100 m (displacement tonnage 10.000 t), up to 366 m (displacement 225.000 t).

The new Zagreb Pier Container Terminal is conceived as a semi-open quay, with an open deck in pre-stressed precast beams with U-section, supported on cellular caissons in precast reinforced concrete, typically launched at about 23.3m of interaxle spacing. The caissons will be placed on a submerged embankment with a maximum height up to 20m, leaning on the foundation ground previously improved with the use of stone columns (with the function of accelerating the subsidence and increasing the mechanical characteristics of the soil) and jet-grouting (for the transfer of load to the deep rocky layers).

2.2 *Operational phases*

The development of the project involves three distinct operational phases (Figure 1):

- Phase 1 concerns the construction of 400m of new quay, with the relative operational area for the loading and unloading of containers, and for their handling with cranes. The work related to phase 1 began in August 2014 (Figure 2) and will be completed in the first months of 2019.

Figure 1. Phase 1 and Phase 2: design plan.

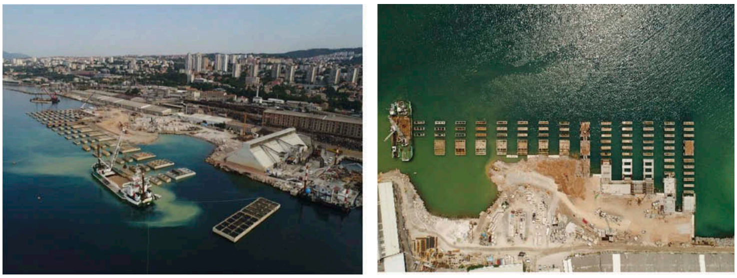

Figure 2. Phase 1: construction site.

- Phase 1A concerns the design of the filling area to complete the infrastructure, up to the existing coastline. As part of the contract, the preliminary design of phase 1A was completed, while, in accordance with the Port Authority's plans, the construction works will be entrusted to a concession dealer, to be selected following an international public tender.
- Phase 2, finally, relates to the design of further 280m of quay, with the relative filling area towards the existing coast. As part of the contract, the main design of phase 2 was completed, while, in this case too, the construction works will be entrusted after the identification of a concession dealer.

Table 1. Operational phases of the project.

	Pier Length	Pier Area
Phase	m	sqm
Phase 1	400	56.000,00
Phase 1A	680	175.000,00
Phase 2	280	37.000,00

2.3 *Geotechnical conditions*

As previously mentioned, the environmental conditions in which the Zagreb Pier is placed have proved to be decidedly challenging, with the presence of a thickness of about 25-30m of soft cohesive soils and cohesionless soils, set on a rocky substratum (Figure 3).

Layer N0 is a stone deposit consisting of debris with a heterogeneous composition, mixed with sandy silty clay and waste materials. Layer N1 is a very soft silty sandy clay. Layer N2 is a clayey silty fine to medium carbonaceous sand (loose to medium dense), while layer N3 is

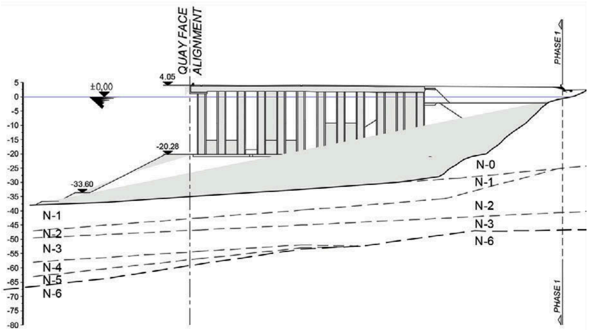

Figure 3. Typical geotechnical profile

Table 2. General classification of layers N0 to N6 by material type

			Cone resistance from CTP
Layer	Material Type	Stratigraphic affiliation (Age)	MPa
N0	Madeground – backfill	Recent	–
N1	Clay	Quaternary period (Q)	0 – 1,0
N2	Sand	Quaternary period (Q)	1,0 – 2,0
N3	Clay	Quaternary period (Q)	1,0 – 3,0
N4	Sand	Quaternary period (Q)	>20
N5	Weathered rock	Lower Cretaceous period	–
N6	Soil limestone	Lower Cretaceous period	–

again a very soft to soft silty clay. Layer N4 is a clayey sand (loose to medium dense). Layer N5 is a weathered rock and N6 is the base rock.

2.4 *Design of the geotechnical works*

The project involves the construction of a large submerged embankment (with a maximum height of 20m) with a surface stone armour, representing the support element for the structures in elevation of the quay. The foundation soil was reinforced by means of:

- Jet Grouting (diameter of 1400mm and 1700mm), arranged in a grid of about 7m x 4m;
- Stone Columns (diameter of 80cm), in a quincunx scheme, with an inter-axis of about 3,5 m.

The geotechnical design involved an in-depth analysis of the behaviour of the soil, with reference to the subdivision into phases of the construction activity and the need for an adequate organization of preloads, to contain the development of settlements over time (in dock areas the contractual limits impose a maximum vertical displacement equal to only 50mm).

3 SEISMIC SITE CHARACTERIZATION

3.1 *Probabilistic Seismic Hazard Assessment (PSHA)*

The goal of this study was to carry out a complete, state-of-the art, PSHA to be used for the design of the caissons and platform of the Terminal Pier, for the design return periods of 95 years and 475 years. In terms of seismo-tectonic setting, the site is placed in the structural domain of the Dinarides, at the north-eastern corner of the Adria - Europe collision zone. Adria is a crustal microplate, i. e. a block of continental lithosphere, that includes the relatively stable Adriatic basin (Po Valley, Adriatic Sea and Apulia), surrounded on the eastern, northern, and western margins by the Dinarides and Albanides, the Alps and the Apennines, respectively.

For the definition of the design seismic action, Croatia issued in 2011 a National Application Document [HZN, 2011] to Eurocode 8 [Comité Européen de Normalisation, 2004] in which maps of a_{gR}, reference peak ground acceleration on type A (rock) ground, are provided. While elastic response spectra can be obtained as a design basis from the combination of a_{gR} and the applicable code spectral shapes, the option of determining site specific probabilistic spectra was preferred for an important project such as the Terminal Pier. Experience has shown that the bandwidth of code spectra may be too conservative for some medium-stiff soil profiles (as in the present case), while the level spectral plateau may be insufficient in the short period range.

A probabilistic approach was thus pursued, using the results of recent international projects specifically devoted to the analysis of seismic hazard in Europe [BSHAP, 2011]. Two different representations of earthquake sources were used, i.e. an area source and a fault model plus background seismicity. Ground motions that could be generated at the project site by the different sources were calculated by means of four empirical attenuation relations, regarded as appropriate for Active Shallow Crustal Regions, such as the region at study. In the end, the hazard at Rijeka was found to be governed by an area source directly underlying the site, with seismic activity predominantly associated with the External Dinarides Thrust Fault Belt (including the Sneznik, Vinodol and Velebit thrust fault systems), and – for return periods up to few tens of years – by the local, lower level of seismic activity (with magnitude typically < 5.0).

3.2 *Site amplification*

The ground shaking severity was expressed in terms of response spectral accelerations covering the whole range of significant response modes of the structures. Uniform hazard spectra (UHS) were provided for the two return periods of interest. With reference to the Croatian norms and the Eurocode 8 provisions, the seismic hazard exceedance curves and uniform hazard spectra have been developed both for category C ground profile and for a type A (rock) profile.

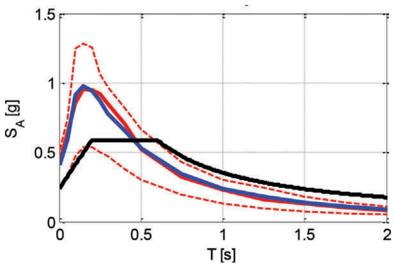

Figure 4. UHS (percentiles, mean and median results, red and blue curves) for return period of 475 years for ground type C. The black curve is the Eurocode 8 (type I) spectrum for ground C.

The difference in shape and short period levels between the presently calculated UHS and the Eurocode Type 1 spectra adopted in the Croatian norms (Figure 4) is not surprising, since the Eurocode 8 spectra were obtained by enveloping spectra from a small number of European strong motion records available some 15 years ago (Rey et al., 2002), while nowadays GMPE (ground motion prediction equations), constrained by hundreds of observations, are used (Bindi et al., 1899–1920).

4 SEISMIC RESPONSE ANALYSIS

4.1 *Dynamic analysis for the elevation structures*

Non-linear dynamic analyses in the time domain were performed to account for the rocking behaviour of the pier caissons at the foundation level and to include hydrodynamic effects. Regarding the latter, the caissons are subjected to hydrodynamic loads, being partially submerged bodies. The Rijeka caissons, more specifically, behave as "bluff-bodies" with a point of fluid flow separation located in correspondence to the vertical edges. A bluff-body, such as a building or the considered rectangular cross-section caissons, tends to block the fluid flow as a result of its shape: indeed, the fluid flow is separated over a substantial part of its surface. In contrast, a streamlined body has a shape in which effort is made to align such a shape to the anticipated streamlines in the flow. As a result, bluff-bodies are characterized by a dominant pressure drag while streamlined bodies are characterized by a dominant frictional component. In the present case, the friction drag is therefore neglected in the analyses. In the case of an earthquake ground movement it is possible to assume that the viscous effect and the velocity gradient of the fluid are limited to the base of the structure. In the analyses, the hydrodynamic effects were considered by including hydrodynamic added masses, related to inertial effects due to pressure gradients, generated by fluid accelerations [Goyal et al., 1989].

Regarding the soil-structure interaction and the rocking attitude at the caissons' base, the non-linear behaviours in terms of base shear vs lateral displacement and of base moment vs rotation were obtained from non-linear static analyses, on planar finite element models (Figure 5) with the software Abaqus (2011). The soil and the jet grouting columns were modelled with plane strain 8 nodes quadrilateral elements considering the Mohr-Coulomb failure

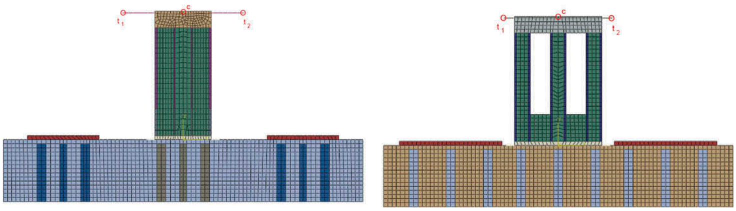

Figure 5. Finite element model of the caisson along its principal directions.

Table 3. Soil and jet grouting material parameters.

Material	Elastic modulus - E (MPa)	Poisson coefficient - ν	Angle internal friction - φ' (°)	Dilatancy ψ'	Cohesion - c' (kPa)
Soil	40	0.15	35	5	0.1
Jet grouting	1000	0	35	5	200

criterion. Only the rubble mound was modelled in the analyses. The thickness assigned to the jet grouting elements was selected to obtain an equivalent cross-section area; in correspondence to the jet grouting columns, superimposed to the soil elements, the soil thickness is reduced to maintain the total thickness. The soil and the jet grouting material parameters are shown in Table 3.

A minimum cohesion of 0.1 kPa in the soil was assigned for convergence issues. The soil Mohr-Coulomb parameters were selected according to effective stresses obtained from reducing the soil density by the water density during the gravity load analysis step. The foundation-structure interface had a normal behaviour modelled with the "hard contact" formulation and a tangential behaviour according to the "Penalty" formulation with an angle of friction equal to the soil angle of internal friction (35°), being the caisson foundation roughened to increase shear friction capacity.

The RC caissons were modelled with plane stress elastic quadrilateral elements with elastic modulus E = 35000 MPa and Poisson coefficient $\nu = 0.2$. The foundation slab wings and deck were modelled with beam elements: the former with an elasto-plastic material in accordance to a moment-rotation analysis, the latter with the same elastic material used for the caisson. Regarding boundary conditions, the rubble mound lateral edges were restrained in the horizontal direction while the bottom edge was restrained to vertical and horizontal movements. Only half ot the deck slab was modelled. In order to maintain the compatibility between adjacent caissons, the slab tips (t_1 and t_2) vertical displacements were set equal to the caisson top central point (c) displacement.

In the present case study, it was not possible, in computational terms, to include the nonlinear behavior at the caisson base in a detailed manner (Figure 5) in a global 3D non-linear dynamic model, therefore a lumped nonlinear link was introduced at the caisson base to simulate the soil-structure interaction in the non-linear response history analyses. The software Midas Gen (2012) was used. At this regard, a nonlinear spring according to the Bouc-Wen (Wen, 1976) model was introduced to simulate the foundation macro-element hysteresis. This model is typically used to describe non-linear hysteretic systems in structural engineering. It could be referred to as a "semi-physical" model because it is not obtained from a detailed analysis of the physical behavior of the system, but it combines physical understanding of the system along with a functional suitable to derive the hysteretic shape. The model is essentially a first-order nonlinear differential equations that relates the input displacement history to a nonlinear restoring force.

The parameters governing the hysteresis of such springs were selected after a best fit procedure applied to the cyclic results of the non-linear static analyses. An example of the results of the fitting procedure is shown in Figure 6.

The two construction stages corresponding to Phase 1 and Phase 2 (see paragraph 2.2) were considered. In Phase 1, the first 18 caisson rows were included (Figure 7), while in Phase 2 the final Pier configuration was considered. Both permanent and live loads were included. Two different scenarios were analysed: in the first one, a portion of the live load due to the containers was considered as a permanent load and the remaining part as a live load, with a participation coefficient $\psi_2 = 0.8$; in the second scenario, no live loads were included.

The results of the non-linear dynamic analyses were represented in terms of the mean value of the maximum results obtained from the ground motions, in accordance to Eurocode 8. The results were expressed in terms of: maximum displacements reached in the deck and at the foundation level; internal actions in the beam connecting the caissons; reactions on the soil; deck slab and the caisson internal actions (Figure 8).

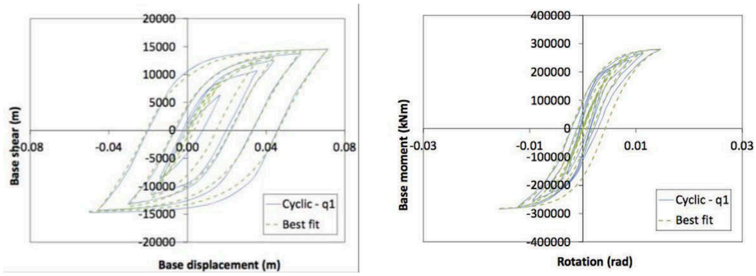

Figure 6. Example of foundation macro-element comparison: Caisson type T1.

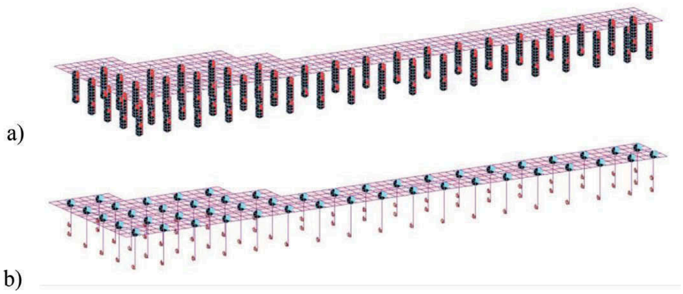

Figure 7. FEM model of Phase 1: a) hydrodynamic masses; b) non-linear springs and deck seismic mass.

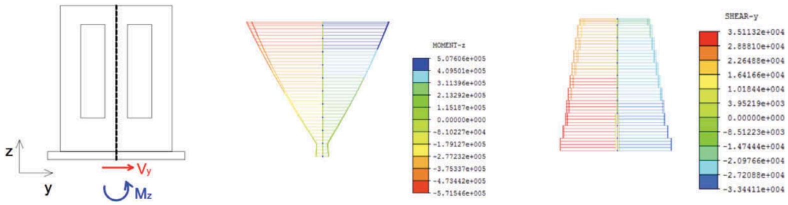

Figure 8. Example of caisson internal actions (Moment in kNm; Shear in kN).

4.2 *Dynamic analysis for the geotechnical works*

Non-linear dynamic analysis in the time domain were performed to estimate the permanent displacements of the global pier structure due to the design earthquake. Analysis were performed developing a simplified 2D FEM model of the pier structure and of the foundation soil in a critical cross-section (alignment 12-12) perpendicular to the pier longitudinal axis.

The FEM model (Figure 9) consists of five caissons, the upper deck, jet grouting columns, seven existing soil/rock layers and the rubble mound (submerged embankment). Caissons connected by the upper deck were modelled as geometrically simplified linear elastic structures, while for modelling the soil layers an elastic-plastic material model was used. Bottom bedrock was assumed as linearly elastic. Acceleration time histories (defined in the contest of the PSHA study) have been applied at the bedrock boundary of the model. The time integration of equations of motion provided a complete information related to stresses and displacements time histories. The analysis allowed to asses the efficiency of the soil improvement in reducing permanent displacements (in terms of sliding towards sea-side and vertical components) due to the design earthquake (with a return period of 475 years). The foreseen permanent vertical settlements are reduced, thanks to the soil improvement, to a value less than 40 mm.

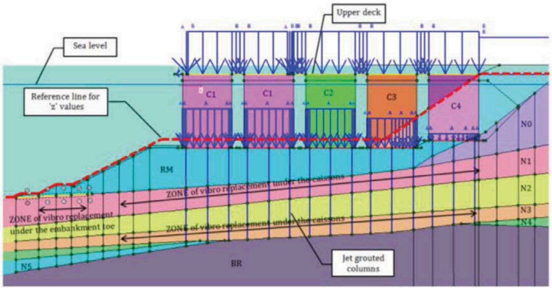

Figure 9. Non-linear dynamic analysis in the time domain: simplified 2D FEM model.

5 CONCLUSIONS

The characteristics of the foundation soil in Rijeka port area are particularly severe, due to the presence of a thick layer of soft cohesive soils, overlaying a rocky substratum. In the project of the new Zagreb Container Terminal Pier a soil improvement was therefore carried out by executing stone columns (for reducing soil settlements), and jet-grouting (to transfer loads to the deeper soil layers). As a part of the project, for the determination of the design earthquake, a Probabilistic Seismic Hazard Study (PSHA) was developed to overcome possible limitations of code spectra. This study was based on adequate earthquake catalogue data and methods of analysis, introducing two different representations of sources, i.e. an area source model and a fault model plus background seismicity.

Both 2D non-linear static and 3D non-linear dynamic analyses in the time domain were developed to account for the rocking of the pier caissons at the foundation level and to include hydrodynamic effects. A specific 2D non-linear dynamic analysis in the time domain was also performed, to estimate the permanent displacements of the global pier structure, comprising the rubble mound (submerged embankment), the different soil layers and the bedrock.

For a construction site characterized by poor soil conditions like Rijeka harbor, these studies and analysis have allowed to prove how an innovative use of the cellular caisson RC technology, in a large pier infrastructure, can be successful both from the technical standpoint and in terms of cost implications, compared to more classic solutions.

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