Container Terminal on Soft Soil

Terminal de conteneurs sur un sol mou

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ABSTRACT: This paper analyzes soil - structure interaction and effectiveness of soil improvement through the back- analyses based on measurements conducted on the example of a new container terminal in the port of Ploce. The terminal was built on part of the Neretva River delta, which is dominated by Quaternary sediments represented by delta deposits over 100 m thick, accumulated over carbonates. Coastal construction is based on bored vertical reinforced concrete piles and hammered steel battered piles. Storage areas (fibre-reinforced concrete slabs) are located behind the coastal structure, and they are founded on soil improved by vibrated stone columns. Construction of container terminal was divided into four main phases. Each phase of the project is analyzed through interactive collaboration of construction designers and geotechnical designers. The applied measurement system consisted of geodetic and geotechnical measurements - geodetic points, vertical inclinometers-deformeters in the vertical piles, horizontal inclinometers, vertical deformeters, tiltmeters and load testing of piles. Works were success fully completed and verified by test loads.

RÉSUMÉ : Cet article analyse l'interaction sol – structure et l'efficacité de l'amélioration des sols en utilisant une analyse en retour, basée sur des mesures effectuées sur le cas du nouveau terminal à conteneurs du port de Ploče. Ce terminal a été construit sur une partie du delta du fleuve Neretva dont le sous-sol est constitué par des sédiments quaternaires formés des dépôts deltaïques de 100 mètres d'épaisseur, surmontant une base calcaire. La construction repose sur des pieux forés en béton armé et des pieux en acier inclinés. La zone de stockage des conteneurs du terminal (dallage de béton renforcé de fibres), située en retrait des constructions côtières, est construite sur les colonnes ballastées. La construction du terminal à conteneurs a été divisée en quatre grandes phases. Chaque phase du projet est analysée grâce à la collaboration entre les ingénieurs de génie civil et les ingénieurs en géotechnique. Le système de mesures utilisé comportait des mesures géodésiques et géotechniques : points de contrôle géodésiques, tassomètres et inclinomètres dans les pieux verticaux, inclinomètres horizontaux, tassomètres verticaux, capteurs de rotation et tests de charge des pieux. Ces travaux ont été réalisés avec succès et puis contrôlés lors de la mise en oeuvre des tests de charge.

KEYWORDS: Container terminal, soft soil, bored vertical piles, vibrated stone columns, field mesaurements, back analyses.

1 INTRODUCTION

The container terminal is located in the Port of Ploče, which is situated in a part of the large Neretva River Delta. Quaternary deposits represented by delta sediments, accumulated over limestone paleorelief dominate in this area. The thickness of Quaternary deposits exceeds 100 m. Deposits have different grain size distribution. One contrasting environment in the vertical geological profile is represented by gravels, and partly sands.

The coast for containers is a surface structure at the level +3.0 m a.s.l., of width 27.4 m and berth length of 280.0 m. The coastal structure is divided in three segments by length. The total length of coastal structure is 88.6 + 102.8 + 109.5 = 300.9 m. The coast is 15.1 m in depth, for vessels of bearing capacity up to 60,000 DWT and draught of 13.5 m. The coastal structure is founded on drilled reinforced-concrete vertical piles (Benoto) of nominal diameter 1,500 mm, and on hammered steel battered piles of diameter 812.8 mm. Vertical piles end in a layer of gravel at the depth of -44 m a.s.l. They are made in single sided formwork from -20 to -44 m or double sided steel formwork down to the depth –20.0 m. Axial distance of piles in longitudinal direction is 7.1 m and in the transversal 8.0 m (four pile rows along coast width).

Storage and traffic surfaces are located behind the coastal structure. Pavement structure consists of fibre-reinforced concrete slabs which are founded on soil improved by stone columns. Stone columns are of 110 cm diameter on a grid 2x2 m to 2.8x2.8 m, of depth down to -15 m a.s.l., i.e. -20 m a.s.l.

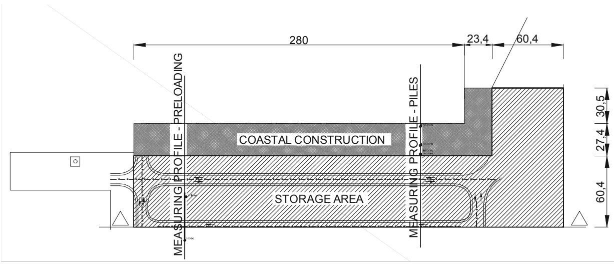

Figure 1 Plan view of the container terminal.

As a part of site investigations, a total of 27 CPT probes were made and 7 boreholes on an area of 15 ha. The lengths of CPT probes range from 22 m to 43 m, while the lengths of boreholes were from 42 m to 58 m. Out of in-situ testing in boreholes, vane tests and standard penetration tests were performed. Laboratory tests included the following: testing of grain size distribution, direct shear test, consolidated undrainedtriaxial test, etc.

2 GEOTECHNICAL SOIL PROFILE

Through implementation of geotechnical investigations and laboratory tests, the following geotechnical soil profile in the area of the container terminal was established:

- Surface layer is represented by siltysand and low plasticity silt, of thickness 8 m.
- Under the surface layer, clays of low to high plasticity dominate. This layer is 25 m thick.
- Low plasticity silt to sand, poorly graded.
- Well-compacted gravel of layer thickness 10m.
- Gray to gray-green clay of stiff consistency.

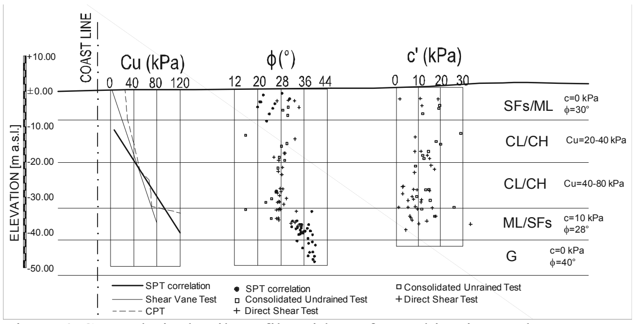

Figure 2 Geotechnical soil profile with performed in-situ exploratory works and laboratory tests.

3 MEASURMENTS AND CONSTRUCTION PHASES

Terminal construction was divided into four main phases: execution of vibrated stone columns and preloading in the storage area, execution of piles from the coast on previously prepared terrain, excavation and underwater embankments, and construction of the coastal structure.

There are two zones of execution in stone columns: dense (triangular grid 2x2m) and sparse improvement (square grid 2.8x2.8m). Dense soil improvement was carried out down to -8m a.s.l. Sparse improvement was performed down to -15m a.s.l. i.e. -20m a.s.l. in such a way that every other column of dense improvement is extended down to the required depth. The role of stone columns is to provide global stability, and settlement reduction and acceleration.

Because of terminal construction dynamics, pre-loading of storage areas was, due to limitations of material for preloading, performed independent of phases, on the condition that they are executed before the execution of coastal structure. Preloading was executed in three segments (fields).

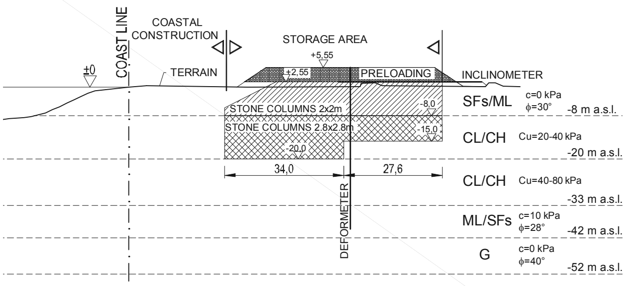

Figure 3 Construction phase I: improvement of foundation soil with stone columns and pre-loading in the area of storage and traffic surfaces.

Each construction phase was monitored through geotechnical and geodetic measurements. Geotechnical measurements in the area of storage and traffic surfaces consisted of measuring vertical displacements by horizontal inclinometers, and settlement measurements by layers using vertical deformeter (see Figure 3). A net of geodetic points was placed on the top of pre-load.

Pre-loading was performed in three fields, in such a way that the material was transferred from one field to another. Settlement of the first field was measured by horizontal inclinometer and vertical deformeter, and with a net of geodetic points 12x12 m. Measurements showed the settlement of 70 cm in the period of 105 days, and they are shown on Figure 4. The difference in settlement between the inclinometer and deformeter is caused by the fact that the deformeter's reference point is at 44 m and underneath it there are compressible layers which could not be followed by the deformeter.

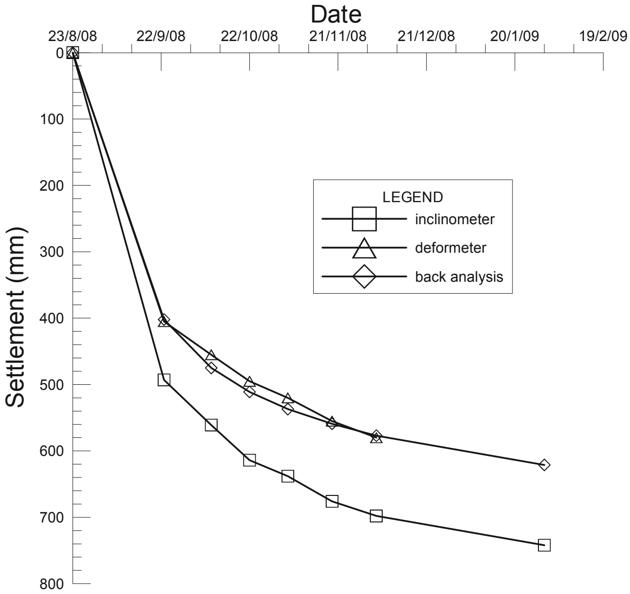

Figure 4.Inclinometer and deformeter settlement measurements together with the settlement curve obtained by back analyses in the first field of preloading.

The requirement for duration of preloading of 90 days was obtained by analysing the consolidation curve. For the other two fields, the same preloading duration requirement was set.

Settlement measurements in the second and third field were performed on geodetic points. Measurements in the second field have shown settlement of 30 cm, and in the third of 50 cm. The reason for smaller measured settlement is longer time of placing pre-loading, and the fact that the reference measurements were performed only after preloading was completed.

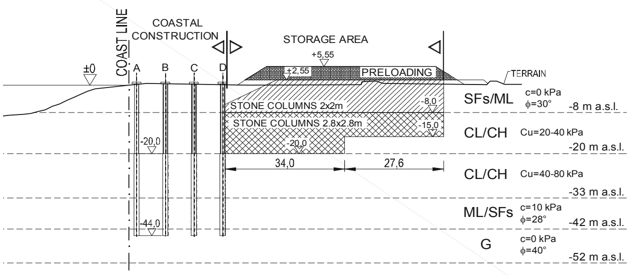

Figure 5 Construction phase II: execution of piles from the coast on previously prepared terrain.

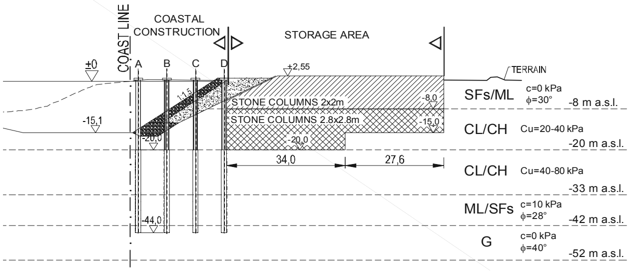

Figure 6 Construction phase III: undersea excavation and execution of undersea embankment.

Geotechnical measurements on the coastal structure consisted of measurements in vertical inclinometers-deformeters inside the pile. Measuring equipment was installed in one profile, so that one vertical inclinometer-deformeter was placed in tensile zones of each of piles A and C. In pile D, two vertical inclinometers-deformeters were placed –one in the compressive zone and the other in the tensile zone. The tops of inclinometers-deformeters were at the same time geodetic points, whose displacements were geodetically followed. Inclinometer measurements showed displacements up to 40 cm at the top of the pile before superstructure was placed. Measurements in the phase of test load have shown minimum displacements, which are within the limits of elastic deformation of concrete.

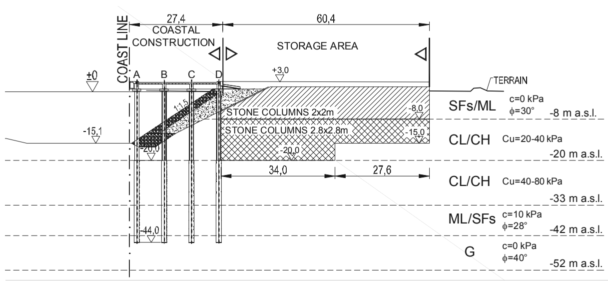

Figure 7 Construction phase IV: construction of coastal structure over piles.

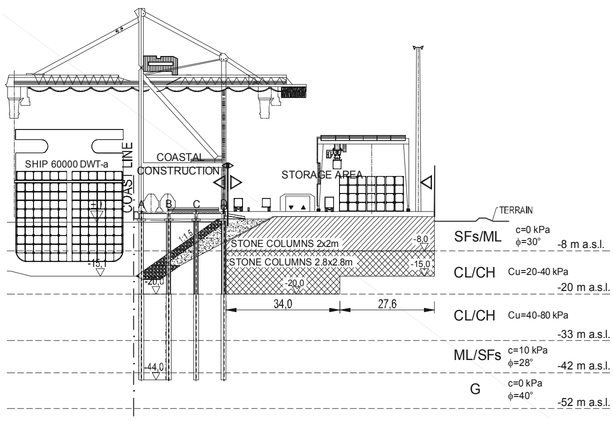

Figure 8 Use of container terminal.

4 BACK ANALYSES

Design and monitoring during construction by means of back analyses based on data obtained by measurements were carried out by the same design team. The design team was made up of designers of the coastal structure and geotechnical engineers. Soil-structure interaction was taken into account through cooperation of design teams in the design process and construction.

Based on measured data on soil settlement and pile displacement back analyses were performed. The objective of back analyses was to establish “actual” soil parameters and internal forces in the structure.

Two models were made for the implementation of back analyses. Inclinometer and deformeter measurements on storage areas through back analyses were processed in Model 1. Material parameters used in Model 2 were obtained on Model 1 based on actual displacements. In Model 2, analysis of pile displacements during terminal construction (up to phase III) was performed, as well as the comparison with measured displacements. Internal forces in piles were calculated on the model calibrated in this manner.

Since the measurements after execution of superstructure on piles and during test loading showed minimum displacements within the limits of elastic deformation of concrete, these construction phases will not be discussed in this paper.

Model 1 was made in Settle 3D software. The soil was set as a linear material through the coefficient of compressibility and vertical consolidation coefficient, and the parameters obtained by back analyses are shown in Table 1. A soil profile was made by means of deformeter measurements, which approximately describes the actual condition in the soil. Figure 9 shows the settlement curve from the vertical deformeter and the curve obtained through back analyses. The curves show soil settlement by layers after 90-day preloading period.

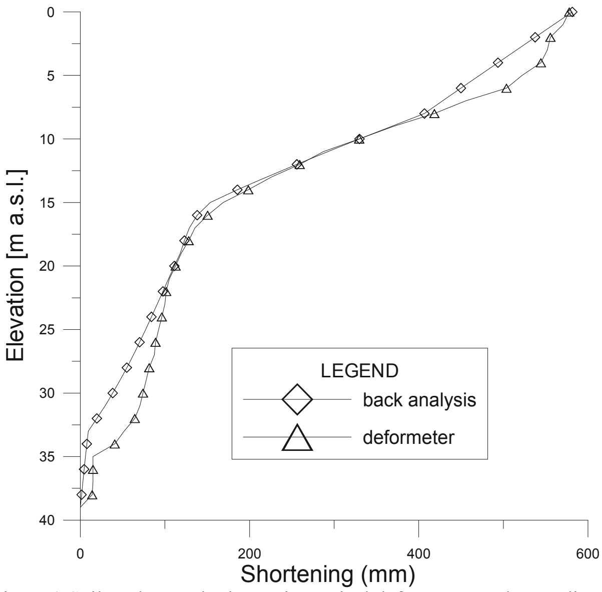

Figure 9 Soil settlement by layers in vertical deformeter, and according to back analyses after 90 days of preloading.

Table 1.Modules of compressibility and vertical consolidation coefficients for back analysis in Model 1.

Parameter / Soil	*Elev. [m]*	*Ms [MPa]*	*Cv [m²/s]*	*Ch / Cv*
SFs / ML	0 -8	4.5	$1.15*10^{-5}$	4
CL / CH	8 -15	2.5	$3.5*10^{-7}$	2
CL / CH	20 – 33	5	$8.1*10^{-6}$	-
ML / SFs	33 - 39	30	$1.5*10^{-5}$	-

Figure 10 shows the ratios of modules of compressibility assumed in the design and those obtained by back analysis. For design values, the improvement of compressibility parameters due to soil improvement by stone columns was not taken into consideration. The scope of ratios ranges from 0.625 to 1.5.

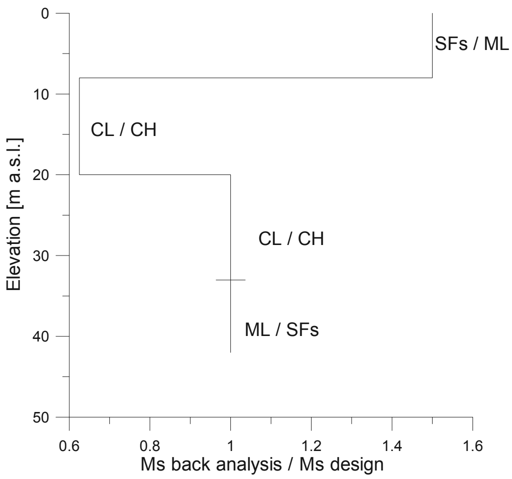

Figure 10 Ratios of modules of compressibility according to back analyses and design values.

Thus in parts where vibrated stone columns are placed in a triangular grid 2x2m, the value of soil improvement factors of 1.5 was obtained, which falls within the design assumptions. It is interesting to consider the ratio that was lower than 1, which was in the layer where stone columns were placed in a square grid 2.8x2.8m. Therefore, instead of obtaining an increase in module of compressibility due to soil improvement by stone columns, we got a decrease with respect to the design values. From there it follows that the compressibility of the CL/CH layer was overestimated in the design. Other layers according to the obtained measurement results were correctly determined in the design in terms of the modules of compressibility.

Based on the calculated parameters of compressibility from Model 1, a numerical model of finite elements was made in Plaxis 2D-Model 2. The soil was described as an isotropic elastoplastic material with linear elasticity properties until failure and by Mohr-Coulomb strength law for stresses at failure. A comparison of horizontal pile displacements before superstructure execution was carried out through the model obtained by back analysis. Also, the comparison of bending moment diagrams obtained by back analyses and on the basis of measured displacements was also carried out.

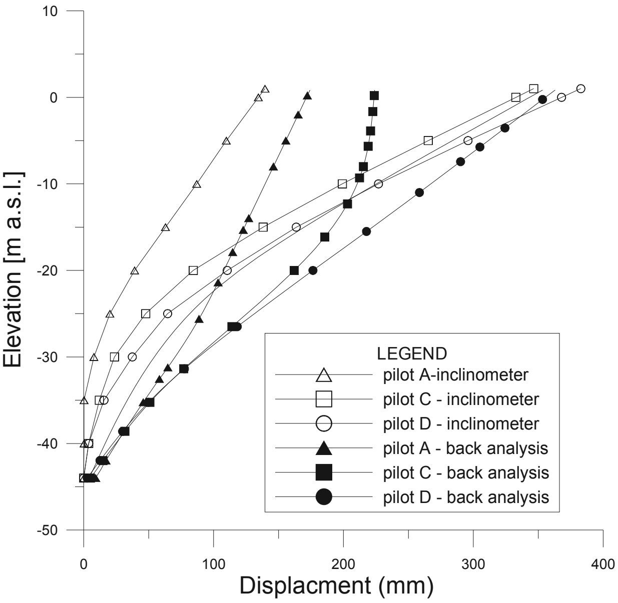

Figure 11 Horizontal displacements of piles A, B and C obtained by measurements in horizontal inclinometer and by back analysis through Model 2.

Figure 11 shows horizontal pile displacements during construction, before construction of coastal structure. Differences in horizontal displacements at the top of the piles obtained by measurements and through model 2 are within tolerance limits. The shapes of displacement curves do not coincide, and therefore the distribution of internal forces is also different. From Figure 11 it follows that the layers from -20 m to -44 m are less compressible than in data obtained based on back analyses through models 1 and 2.

Figure 12 shows bending moments obtained on the basis of measurements and back analyses through models 1 and 2 for pile D (last pile landwards).Diagrams show the temporary phase of bending moment before the construction of coastal structure. Maximum bending moments appear in the case obtained on the basis of measurements of horizontal displacements in inclinometer.

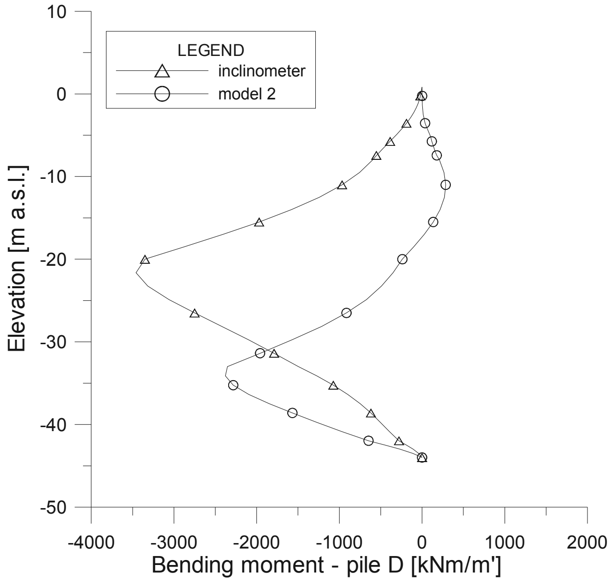

Figure 12 Bending moments in pile D, on the basis of displacements in vertical inclinometer and for back analyses model 2.

5 CONCLUSION

The paper aims to describe the design process and control of execution of a demanding structure –coastal structure, which takes into account soil-structure interaction.

The procedure was carried out iteratively through collaboration of design teams of structural engineers and geotechnical engineers.

The geotechnical model was made based on delivered loads and design assumptions of soil parameters. Through the geotechnical model and finite element method, the coefficients of soil reaction were determined through soil pressure and displacements. Ground reaction coefficients were delivered to the designers of the structure. By means of such procedure in several steps, through collaboration of design teams, soil-structure interaction assumed in the design was obtained.

Verification of efficiency of planned works was performed through geotechnical measurements described in the paper.

Based on geotechnical measurements, back analysis of soil parameters (Model 1) and the condition of internal forces and displacements of the structure (Model 2) was performed.

In this paper, we wanted to point out that it is necessary to perform back analyses during and after execution of demanding structures on the basis of performed measurements and through collaboration of structural and geotechnical engineers.

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