QUAY WALL AND BREAKWATER DESIGN AND CONSTRUCTION OF THE NEW PORT OF PATRAS

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ABSTRACT

Design and construction of marine structures in weak foundation soils in a seismic environment constitutes a major challenge. Several ground improvement measures were employed in the design of the quay wall and breakwater of New Port of Patras to cope with extremely adverse geotechnical and earthquake conditions. Dictated by new findings during the construction phase of the project, significant modifications of the soil improvement designs were made, which led to enhancement of the static and seismic stability of the temporary and permanent structures, as well as to reduction in the overall construction time.

INTRODUCTION

The city of Patras, on the shores of Peloponnese in southwestern Greece, constitutes the country's third largest city and the main gateway to Italy. Major infrastructure works in the region include a New Port south of the existing port facilities to satisfy continuously increasing transportation and commercial demands. The New Port is constructed in an area characterized by unfavorable site conditions due to the presence of very soft foundation soils, an active fault which crosses the project site, and large craters throughout the seabed. Design engineers, a Joint Venture of Greek ADK SA and Triton Inc. (1994), proposed an extensive soil improvement program in order to safely found the marine structures in this adverse environment.

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Construction of the first phase of the New Port, a €40 M worth contract, was awarded to the Joint Venture (JV) of British Christiani & Nielsen and Bachy - Soletanche and Greek Technical Company of General Construction (now EMPEDOS S.A.). Currently under construction, the first phase of the New Port includes 430 m of quay wall, and 880 m of breakwater fronts. Water depth ranges between 10 – 15 m, and 30 – 40 m, in the quay wall and breakwater areas, respectively. The Quay wall consists of 22 precast concrete caissons retaining granular backfill. The Breakwater is a composite structure consisting of about 20 m to 30 m of rock fill embankment on top of which 45 caissons are placed. Layout of the first phase of New Port of Patras is shown on Figure 1.

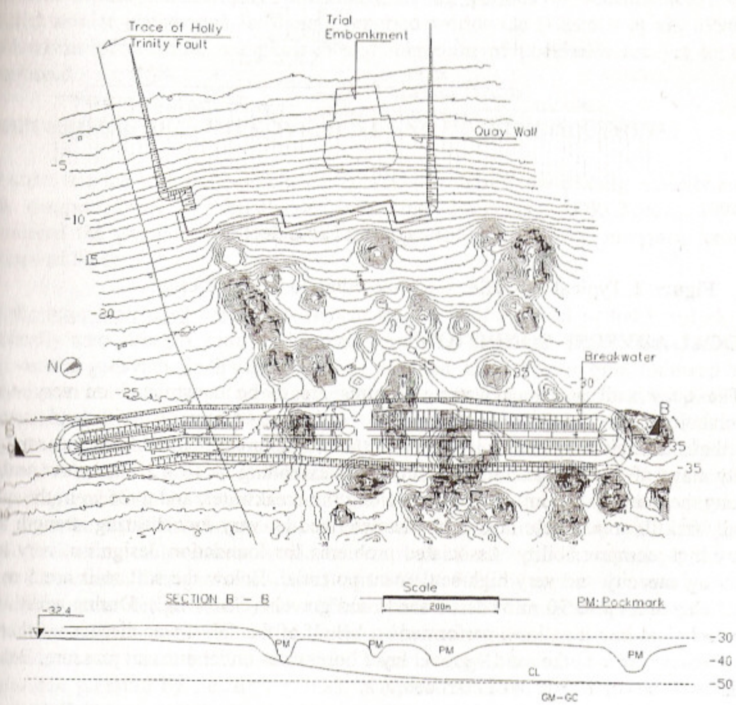
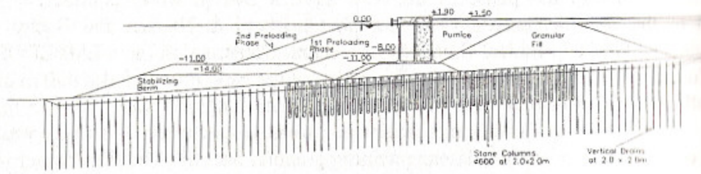
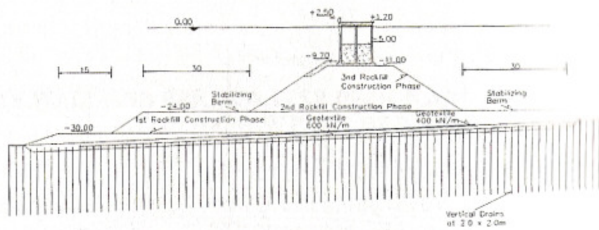


Figure 1. Plan View of Quay Wall and Breakwater Structures, Cross Section along Center line of Breakwater

Typical cross sections of the two structures, including ground improvement measures proposed by the designers, are depicted in Figure 2 (ADK - Triton, 1994).



a. Quay Wall Cross Section



b. Breakwater Cross Section

Figure 2. Typical Cross Sections According to Tender Design

LOCAL ADVERSE CONDITIONS

The quay wall and breakwater structures are to be constructed on very weak foundation soils and crater-stigmatized seabed, and are to withstand severe earthquake excitation. The soil profile in the project area includes very soft sandy silty clays of low to medium plasticity (CL) extending to 10 - 25 m below seabed along the quay wall front, to 5 – 15 m along the breakwater, and to 35 m in the quay wall backfill area. These layers are characterized by very low shearing strength and very high compressibility. Associated problems for foundation design are very low bearing capacity and very high settlement potential. Below the soft soils are 5 m of stiff clay and up to 70 m of dense sands and gravels (GM – GC). During additional geotechnical investigations, performed on behalf of the JV, it was discovered that in the breakwater area the sand – gravel layer bore water under artesian pressure. Below this layer, lies up to 200 m of marl bedrock.

According to the Greek Earthquake Design Code, the city of Patras falls in Zone III with a corresponding peak ground acceleration of 0.24g. This value has a 10% possibility of exceedance in 50 years. A site specific seismic hazard analysis was performed by the University of Patras (1994) to define the design level peak rock acceleration. Ultimately, in the design, a value of 0.27g was used, associated with an event that has a return period of about 500 years. In addition to high seismicity, the

active fault of Holly Trinity crosses the project area (Fig. 1). Creeping displacements are manifested along the entire length of the fault. West of the trace of Holly Trinity fault the seabed has a unique relief due to the presence of numerous "craters" (pockholes - pockmarks), which, according to geophysical and bathymetric surveys, reach depths ranging between 0.5 m and 15 m, and diameters ranging between 25 m and 180 m. The side walls of the pockholes stand relatively steep at 20° – 23°. In the breakwater area craters often extend through the top clayey layers and reach the underlying dense sand – gravel layer. Craters have been created by release of gases (hydrogen sulfide and methane), which are trapped at the interface of the coarse sand – gravel layer and the overlying fine layer. Gases migrate upwards through the soft sediments steadily or abruptly particularly during periods of seismic activities. Limited similar phenomena have been reported worldwide (Hasiotis et al., 1996). Mechanisms of creation, maintenance and migration of pockholes are yet to be determined.

PROPOSED GROUND IMPROVEMENT AND DESIGN SOLUTIONS

In order to cope with unfavorable site conditions, mainly low bearing capacity and high compressibility of foundation soils, the designers (ADK–Triton, 1994) considered the ground improvement and design solutions shown on typical cross sections of Figure 2.

In the quay wall area, soil improvement started with removal of the top 2 m, a practically zero strength zone, and replacement with sand and gravel. Next, 19-m-long vertical geosynthetic drains were to be installed in a 2m x 2m grid, followed by construction in two phases of a preloading embankment to accelerate manifestation of anticipated consolidation settlement. In the first construction phase, preloading embankment would be raised to level -14.0 m in the stabilizing berm area, and to level -11.0 m in the quay wall and land areas. After a waiting period of 8 months, preloading embankment would be raised, in a second construction phase, to level -11.0 m in the stabilizing berm area, and to level ±0.0 m in the quay wall and land areas. Preloading embankment would be removed after a second waiting period of about 12 months. Designers estimated that 80% - 90% of expected consolidation settlement under design loads, and sufficient increase of strength characteristics of foundation soils would take place during the two waiting periods. For design purposes, it was assumed that undrained shear strength increases with increasing overburden pressure by $\Delta c_u/\Delta\sigma'_v = 0.23$, a typical ratio for normally consolidated clays (ADK–Triton, 1994). Ground improvement scheme would be completed with installation of 10-m-long stone columns to further increase bearing capacity of the foundation soils. Stone columns of nominal diameter 0.60 m were to be installed by vibro-replacement in a 80-m-wide zone (30 m in front, and 50 m behind the quay wall front) in a 2m x 2m grid (Drettas et al., 1997).

In order to further improve stability of the quay wall structure, the design included a light quay wall consisting of concrete precast semi-filled cellular 12.8-m-high caissons, backfilled with light weight soils (pumice), thus reducing both vertical loads and horizontal earth pressures.

In the breakwater area, the ground improvement design included removal of top 2 m, replacement with sand and gravel, and installation of 12-m-long vertical geosynthetic drains to allow rapid consolidation and thus strength increase in the clayey soils. Next, two layers of geotextiles, covering the entire footprint of the breakwater, would be placed to provide increased resistance against slope failure (Platis et al., 1997). Construction of the rubble mound embankment was scheduled to take place in three stages with intermediate waiting periods to avoid temporary stability problems. Stabilizing berms were to be constructed on both sides of the main rock fill embankment to improve stability of the composite breakwater structure. The first phase of rock fill construction would include placement of the first berm to a crest elevation of -30 m. After a waiting period of about 1 – 2 months, second phase construction would take place and the second berm would be completed to elevation -24 m. After a second waiting period of 9 months, the third final construction phase would take place raising the rock fill to elevation -11 m. Finally, light weight 12.8-m-high caissons would be placed on top of the rock fill embankment after a third waiting period of 9 months to complete the composite breakwater structure.

Prior to any preloading embankment construction in the quay wall area, the design called for construction of a very well instrumented trial embankment to verify rate and magnitude of expected settlement. Geosynthetic drains were to be installed at different spacing on each of the four quadrants of trial embankment to study the effect of spacing on time rate of settlement (one quadrant without drains and three quadrants with square grids of drains spaced at 1.5 m, 2.0 m, and 2.5 m).

FINDINGS DURING CONSTRUCTION

Severity of local adverse geotechnical and earthquake conditions was recognized in the preliminary feasibility study. However, the extreme morphology of the seabed (exact location, number, dimensions of craters) was only revealed during construction based on a bathometric survey assigned to Akti Engineering by the contracting JV. The survey revealed an extremely anomalous relief particularly in the breakwater footprint area, west of the trace of Holly Trinity fault. The craters are depicted in Figure 1, which also shows the plan view of the seabed and one cross section along the center line of the breakwater.

Additional geotechnical investigations performed on behalf of the JV revealed the presence of several thin sand layers within the top clayey soils. These layers facilitated drainage in the horizontal direction during consolidation process, thus reducing importance of vertical drain spacing. Indeed, evaluation of trial embankment monitoring results indicated practically no effect of vertical drain spacing on time rate of settlement. Measured settlement values at the trial embankment site, in the order of 0.8 – 1.0 m, confirmed consolidation characteristics assumed in the design.

Tender drawings indicated that construction of stone columns in the quay wall area was to take place either as a land operation prior to removal of preloading embankment, or as a marine operation after removal of preloading embankment. Trial stone column construction at the trial embankment site, led to a number of important conclusions. Firstly, land based construction of stone columns through the trial embankment was practically impossible due to nature and degree of compaction of imported fill materials. This fact dictated that marine construction of stone columns was the only feasible operation. Secondly, based on records of stone consumption versus depth, it was calculated that diameter of 10-m-long stone columns averaged 1.6 m at the bottom 1 – 2 m, and 0.9 m at the remaining stone column length. Bulging near the tip of stone columns was alleviated when 15-m-long trial stone columns were constructed. These findings indicated that, given the larger produced diameter due to stone compaction requirements, grid spacing of stone columns could be increased to achieve the required replacement ratio. Marine stone column construction was performed using the bottom feed method. During trial marine operations it was also concluded that, in order to avoid bulging near the top of stone columns, a layer of about 3 m of fill should be placed on the seabed prior to construction of stone columns to generate the necessary overburden pressure.

BREAKWATER ALTERNATIVE DESIGN

Proposed ground improvement operations in the crater field were extremely difficult to implement. JV produced an alternative design for the breakwater (1999), which was approved by the governing authorities and actually implemented. The proposed new typical section is depicted in Figure 3.

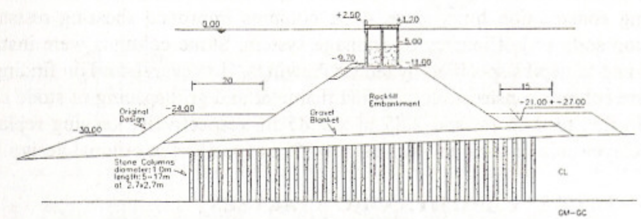


Figure 3. Typical Breakwater Cross Section According to Alternative Design

Excavation of the top 2 m, installation of vertical drains and placement of two layers of geotextiles in water depths exceeding 40 m in pockhole areas, which would require unprecedented equipment, technology and manpower, were eliminated. Instead, ground improvement included placement of a gravel blanket about 3 m thick, followed by reinforcement of foundation soils via approximately 14,000 stone columns in a 100-m-wide zone along the entire breakwater footprint. Stone columns of 1.0 m diameter, constructed by vibro-replacement in a 2.7 m x 2.7 m square grid, penetrated top clayey soils (5 m – 17 m) and reached the underlying sand-gravel layer.

Furthermore, based on elaborate slope stability analyses, stabilizing berms on both sides of the main rock fill embankment were reduced as shown on Figure 3. In the pockhole areas stone columns were constructed till level -2 m from top of the craters.

Stone columns have multiple functions. They provide immediate reinforcement of foundation soils by replacing approximately 11% of clayey ground. This replacement ratio guarantees that rate of construction of rock fill embankment is independent of strength increase of clayey layers during consolidation process. Any temporary stability issues are therefore automatically overcome. Waiting periods during construction of rock fill embankment, which totaled 20 months according to the original design, are completely eliminated. Stone columns also constitute a very effective drainage system, which facilitates consolidation of clayey layers. It was estimated that expected consolidation settlement due to weight of rock fill, in the order of 75 cm, would take place during placement of the embankment. Finally, stone columns provide a dissipation path for unimpeded upward migration of gases, thus eliminating potential for abrupt creation of new pockholes, which could jeopardize stability of entire breakwater structure.

QUAY WALL DESIGN ENHANCEMENTS

A change in the sequence of operations was implemented during construction of the quay wall in order to increase temporary stability during placement of preloading embankment and to reduce overall construction time. In the modified program, stone columns were installed after placement of the first phase of preloading embankment. This modification led to significant improvement of temporary stability during construction of second phase of preloading embankment, and to reduction of remaining construction time, since stone columns improved shearing resistance of foundation soils and efficiency of drainage system. Stone columns were installed in the area and to depths specified by tender drawings. However, based on findings from trial stone column construction, nominal diameter and grid spacing of stone columns were adjusted to 0.86 m, and 2.85 m x 2.85 m, respectively, keeping replacement ratio of clayey soils in the quay wall area to 7% as specified in original design.

SLOPE STABILITY AND DYNAMIC ANALYSES

Two dimensional limit equilibrium methods and software (Wright, 1986) were used to assess static and pseudo-static safety against slope instability of the proposed breakwater alternative design and the quay wall modified construction sequence. Potential failure surfaces and corresponding safety factors for a typical section of the breakwater are depicted in Figure 4. These calculations are based on an average friction angle of 23.5° for the 100-m-wide zone representing the matrix of clayey soils and stone columns.

Expected permanent seismic deformations of the quay wall and the breakwater were estimated based on results of dynamic analyses (Yegian, 1998). The dynamic

response of the proposed port structures was investigated using the finite element method (Hudson et al., 1994) and accelerograms from Greek and international earthquakes. Figure 5 depicts the FE grid, critical sliding surfaces, and peak values of average accelerations, K_{amax}, resulting from dynamic analyses using four accelerograms.

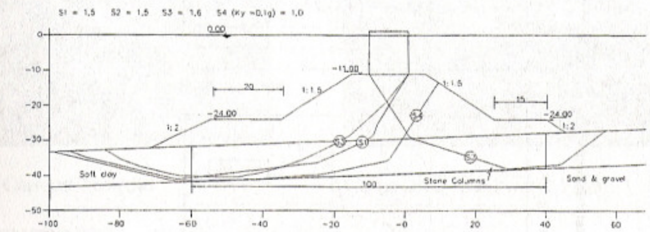


Figure 4. Slope Stability Analysis Results of the Breakwater Structure

Earthqauke	Ka_{max}, Average Horizontal Peak Acceleration of Critical Sliding Mass		
	Sliding Mass, M1	Sliding Mass, M2	Sliding Mass, M3
Aigion	0.26g	0.15g	0.39g
Kalamata	0.33g	0.19g	0.33g
Pacoima	0.25g	0.13g	0.27g
Arleta	0.41g	0.19g	0.40g

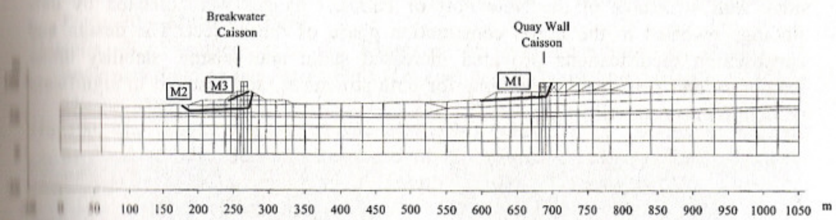


Figure 5. Finite Element Mesh, Critical Sliding Surfaces, and Peak Average Accelerations

Average horizontal acceleration, K_a, of a critical surface is calculated as a function of time by adding shear forces along the sliding surface and then dividing the sum by the sliding mass. Permanent seismic deformations are calculated when the average seismic acceleration K_a exceeds the critical acceleration K_y, which is calculated via pseudo-static slope stability analysis, as the acceleration producing a safety factor equal to one. As an example, Figure 6a depicts calculated time history of average acceleration K_a for sliding mass M1 (quay wall) using the Pacoima rock record, downscaled so that maximum acceleration is 0.27g to coincide with site specific maximum bedrock acceleration. Figure 6b depicts time history of permanent seismic deformations calculated using Newmark's method with critical acceleration K_y = 0.02g.

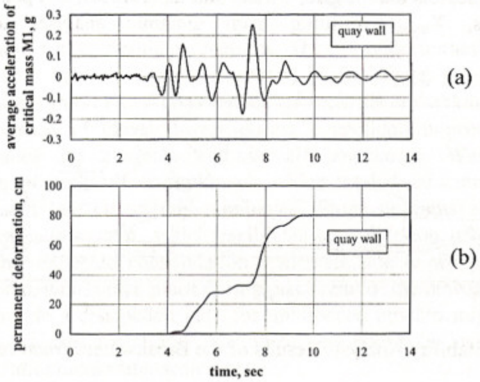
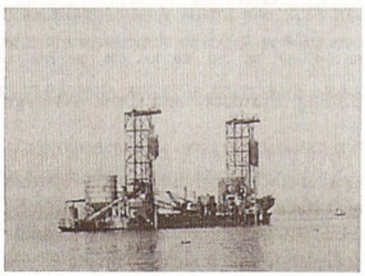
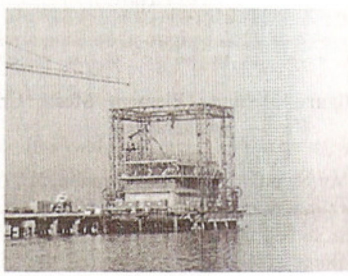


Figure 6. (a) Time History of Average Acceleration (b) and Permanent Seismic Deformations for Sliding Mass M1 (Quay Wall) using Pacoima Rock Record

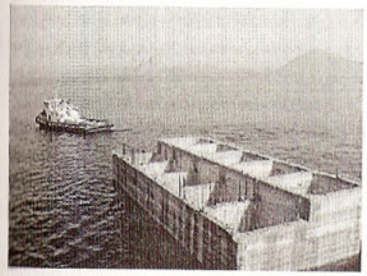
CONCLUSIONS

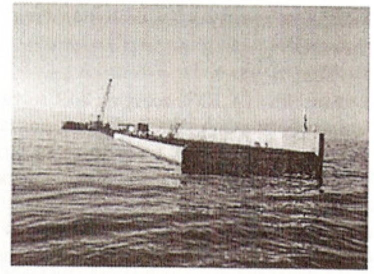
Significant design changes, mainly in the ground improvement measures and sequence of operations, were implemented during construction of the breakwater and quay wall structures of the New Port of Patras. Changes were dictated by new findings revealed in the initial construction phase of the project. The design and construction modifications provided increased static and seismic stability under temporary and permanent conditions for both structures, and resulted in significant reduction in construction time.





Photos : Stone Column Installation — Caisson Construction by Slipforming Method from a Jetty Founded on Piles





Photos : Caisson Tow out — Completed Breakwater Front

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