THE EVALUATION OF THE MORFODYNAMICS BOUNDARY CONDITION, SOIL INVESTIGATION AND LOADS ASSESSMENTS FOR THE REALIZATION OF IMPROVED QUAY WALLS STRUCTURE IN DURRES PORT

A THESIS SUBMITTED TO
THE FACULTY OF ARCHITECTURE AND ENGINEERING
OF
EPOKA UNIVERSITY

BY

Alket Bedini

IN PARTIAL FULFILLMENT OF THE REQUIREMENTS
FOR
THE DEGREE OF MASTER OF SCIENCE
IN
CIVIL ENGINEERING

FEBRUARY, 2022

Approval sheet of the Thesis

This is to certify that we have read this thesis entitled **"The Evaluation of the Morfodynamics Boundary Condition, Soil Investigation and Loads Assessments for the Realization of Improved Quay Walls Structure in Durres Port"** and that in our opinion it is fully adequate, in scope and quality, as a thesis for the degree of Master of Science.

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I hereby declare that all information in this document has been obtained and presented in accordance with academic rules and ethical conduct. I also declare that, as required by these rules and conduct, I have fully cited and referenced all material and results that are not original to this work.

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ABSTRACT

THE EVALUATION OF THE MORFODYNAMICS BOUNDARY CONDITION, SOIL INVESTIGATION AND LOADS ASSESSMENTS FOR THE REALIZATION OF IMPROVED QUAY WALLS STRUCTURE IN DURRES PORT

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Ports, like any other industry, is facing the need to respond to the challenge of globalization which, in the port maritime sector, is mainly manifested in the growing of ships dimensions the so-called “Naval gigantism”. In order not to be excluded from the maritime traffic market, ports must handle large volumes of traffic with an incresed efficiency, in order to reach international markets through multimodal corridors.

The direct surveys done on site were integrated and elaborated together with existing available data. A series of numerical mathematical simulation models were executed in order to fully understanding of the geological, bearing capacity, navigation and meteorological aspects that are interested in the project. These mathematical simulation models led to what is the final design for the new improved quays detailed design.

Being the structure closer to the entrance, quays are exposed to long waves from the South, which the driving factor for excessive horizontal motions for ships moored in harbor basins exposed to swell. Long waves are the driving factor for excessive horizontal motions for ships moored in harbour basins or coastal waters exposed to swell. This work shows how the efficiency of a port exposed to swell waves can be improved by adapting on pile structures.

After review of the existing documentation, site visits and discussions with stakeholders, three options for dealing with the quays were identified. The first option is the “business as usual” scenario, in which no additional work is carried out and only regular maintenance of the infrastructure and equipment takes place. The second option consists in a rehabilitation of the existing quay wall, construction of two arrays of concrete piles supporting two concrete beams to serve as foundations for the crane rails, demolition of the existing deck and construction of a new concrete deck and apron, and deepening of the sea level. The third option consists of the construction of a retaining wall at the existing quay line and of a new deck supported by an array of drilled piles, bringing the new quay line farword.

The process the slope stability analyses were carried out for the settlement analysis and liquefaction for the sheet pile construction with material of properties similar to the backfill material. The overall stability checks carried out the temporary slip fill and for the retaining structure and the pile depth.

Incremental revenues are calculated on the basis of incremental product, expressed in tons of general cargo estimated to be handled annually Under option three, maximum annual production is calculated at approximately 1,800,000 tn. overcomes its inferior financial and economic performance compared to the other proposed options (investment costs 70% higher for similar financial and economic benefits) thanks to, inter alia, its ability to handle larger vessels.

Keywords: *Port, Port Authority, Quay, Apron, Yard, Detailed Design, Reconstruction.*

ABSTRAKT

VLERËSIMI I KUSHTEVE MORFODINAMIKE, STUDIMET GJEOLOGJIKE DHE VLERËSIMET E NGARKESËS MBI STRUKTURË PËR REALIZIMIN E KALATAVE ME KAPACITET MBAJTËS TË PËRMIRËSUAR NË PORTIN E DURRESIT.

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Portet, si çdo industri tjetër, përballen me nevojën për t'iu përgjigjur sfidës së globalizimit e cila, në sektorin detar port, manifestohet kryesisht në rritjen e dimensioneve të anijeve të ashtuquajturën "Gjigantizmi detar". Në mënyrë që të mos përjashtohen nga tregu i trafikut kryesor detar, portet duhet të trajtojnë vëllime të mëdha me efikasitetin më të madh, duke arritur tregjet ndërkombëtare përmes korridoreve multimodale.

Studimet dhe investigimet e bëra në terren u integruan dhe u përpunuan së bashku me të dhënat ekzistuese të disponueshme. U realizuan një sërë modelesh numerike matematikore për të kuptuar plotësisht aspektet gjeologjike, të kapacitetit mbajtë të structures së kalatave, të lundrimit dhe meteorologjike që preken në projekt. Këto modele simulimi matematikor çuan në atë që është dizajni përfundimtar për projektin e detajuar të kalatave të reja të përmirësuara.

Duke qenë struktura më e afër me hyrjen e portit, kalatat janë të ekspozuara ndaj valëve të gjata nga Jugu, të cilat janë faktor shtytës për lëvizjet e tepërta horizontale për anijet e ankoruara në kalatat portuale të ekspozuara edhe ndaj fryrjes së erës. Valët e gjata janë faktori shtytës për lëvizjet e tepërta horizontale për anijet e bregëzuara.

Nevoja për të bërë ndryshime thelbësore në infrastrukturën e vjetëruar prodhoi risi në rikonceptime të reja të infrastrukturave portuale të afta për të ndërvepruar, në mënyrën më të përshtatshme dhe më të shpejtë të mundshme, me rrjeten logjistike intermodale për të mbështetur aktivitetet kryesore social-ekonomike bashkëkohore.

Pas shqyrtimit të dokumentacionit ekzistues, vizitave në terren dhe diskutimeve me palët e interesuara, u identifikuan tre opsione për trajtimin e kalatave. Opsioni i parë është skenari “business as usual”, në të cilin nuk kryhen punë shtesë dhe bëhet vetëm mirëmbajtja e rregullt e infrastrukturës dhe pajisjeve. Opsioni i dytë konsiston në rehabilitimin e murit ekzistues të kalatës, ndërtimin e dy grupeve shtyllash betoni që mbështesin dy trarë betoni për të shërbyer si themele për shinat e vinçit, prishja e kuvertës ekzistuese dhe ndërtimi i një kuverte dhe platforme të re betoni dhe thellimi. të nivelit të detit. Opsioni i tretë konsiston në ndërtimin e një muri mbajtës në vijën ekzistuese të kalatës dhe të një kuverte të re të mbështetur nga një sërë shtyllash të shpuara, duke sjellë vijën e re të kalatës.

Të ardhurat në rritje llogariten mbi bazën e produktit në rritje, të shprehur në ton ngarkesë të përgjithshme që vlerësohet të trajtohet çdo vit Sipas opsionit tre, prodhimi maksimal vjetor llogaritet në afërsisht 1,800,000 tn. kapërcen performancën e tij financiare dhe ekonomike inferiore në krahasim me opsionet e tjera të propozuara (kostot e investimeve 70% më të larta për përfitime të ngjashme financiare dhe ekonomike) falë, ndër të tjera, aftësisë së tij për të trajtuar anije më të mëdha.

Fjalët kyçe: *Port, Autoritet Portual, Kalatë, Shesh përpunimi, Projekti i detajuar, Rindërtim.*

ACKNOWLEDGEMENTS

I would like to express mi sincere gratitude and give my warmest thanks to my supervisor **Assoc. Prof. Dr. Huseyin Bilgin** who made this work possible. His patience, guidance, advice and immense knowledge carried me through all the stages of writing my thesis. The completion of this work would not have been possible without him.

I would also like to thank my committee members for letting my defense be an enjoyable moment, and for your brilliant comments and suggestions, thanks to you.

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Abbreviations

Abbreviation	Meaning
CBA	Cost-Benefit Analysis
CEF	Connecting Europe Facility
DD	Detail Design
EBRD	European Bank for Reconstruction and Development
EC	European Commission
EIA	Environmental Impact Assessment
ENPV	Economic Net Present Value
ERR	Economic Rate of Return
ESIA	Environmental and Social Impact Assessment
FNPV	Financial Net Present Value
FRR	Financial Rate of Return
FRR/C	Financial Rate of Return on Investment
FRR/K	Financial Rate of Return on National Capital
FS	Feasibility Study
IR	Inception Report
IRR	Internal Rate of Return
NKE	Non-Key Expert
NPV	Net Present Value
O&M	Operating & Maintenance
PGA	Peak Ground Acceleration
PFS	Pre-feasibility study
QE	Quantitative Easing
SNKE	Senior Non Key Expert
SPV	Special Purpose Vehicle
ToR	Terms of Reference

CHAPTER 1

INTRODUCTION

1.1 General

The term "quay" refers to those structures built for the berthing of ships or boats to the mainland, and which allow a safe docking for the embarkation and disembarkation of people or the loading and unloading of goods.

Historically the construction solutions are of multiple forms, from massive gravity support works up to the most modern techniques that involve the installation of sheet piles reinforced by various types of restraint systems, capable of facilitating the subsequent construction of the reinforced concrete quay. Given the high compressibility of the ground levels making up the seabed and the simultaneous need to have a sufficient useful height with respect to the average sea level, the quays are often complex works from a geotechnical point of view. They are typically stressed by horizontal actions exerted by ships and boats on the mooring bollards and fender profiles. In the case of docks for commercial use, a huge operating load is added on the quay level relating to the runways of the cranes and the storage of containers.

The complexity in constructions does not only derive from the geotechnical aspect, but also from a mechanical prospective and load bearing capacity point of view. The applied load during the exercise lifetime are signification. In this context, harbor cranes are massive superstructures which can apply significant loads up to 10 ton/m2, but also live load of trucks and that of the heavy auxiliary machineries that may be used during during loading and unloading process of the ship.

Being one of the most versatile structure of a port facility, quays play an extremely important role in the port operations.Regarding this the Durrës Port Authority has prepared and launched a progressive plan of interventions capable of allow the static consolidation and functional adaptation of the quays in order to be able to guarantee its functionality, also in anticipation of the planned in-depth interventions of the backdrops, in line with the functional technical adaptation of the works.

Quays 1 and 2 are part of the Durrës's Port West Terminal. With a total length of 400m are mainly used as quays for loading and unloading cereals. Build respectively in 1954 and 1972, both quays were reconstructed in 1994.

In 2012, Durrës Port Authority commissioned a study to assess the potential privatization potential of the West Terminal as well as to prepare conceptual and detailed projects for the reconstruction of quays 1 and 2. In 2017, a grant was provided from the WBIF (West Balkans Investment Framework) to conduct, among other things, a pre-feasibility study for the construction of quays 1 and 2. This grant is implemented by the IPF7 consortium. The result of the pre-feasibility study showed the necessity to adapt the actual port infrastructure to the maritime transport market request.

The above mentioned quays are on pile structures with a projected bearing capacity of 4 ton/m^2 and the yards connected to them have a bearing capacity of 6 ton/m^2. Due to the absence of a well-defined maintenance plan these capacities may not be the real actual capacities. The underwater structure is interested by a high degree of corrosion as result of the concrete decay. Large areas of the yards are presents depressions. The berthing line has also structural issues with iron bars exposed to the atmospheric agents and due to the lack of fenders ship strokes has created numerous sagging. In order to have optimal structure and operational conditions the total reconstruction seems to the only way to improve the quay capacity and make it more suitable for the today market requests.

1.2 Thesis Objective

In this thesis, are included some of the most important aspects of the detailed design of a complex structure such as a port quay.

The intervention object of the design has as its main objective the achievement, in the port quay, of the static requirements necessary for carrying out the activities, within the canons of safe functionality and operation, this in relation to provisions of current legislation as well as the operating procedures dictated by modern technologies and work systems. The intervention therefore assumes a predominantly structural connotation of works aimed at the renewal and adaptation of the existing on the site conditions, which concerned (water supply lines, collection system and disposal of rainwater).

A process that passes through the set of problems connected with the preliminary site inspection, the prefeasibility study, the meteorological surveys and both the entry and exit maneuvers of the ships from a port facility and with the mooring phases along the docks constitute the maritime aspects of functionality and operation of a port, all joined together in the final produced that is the detailed design.

1.3 Scope of works

As in any project, for a better understanding of the framework in which the structure will operate a series of survey must be help before undergoing the procedure of the detailed design. In maritime construction an important role is placed by the meteorological condition, tides, wave motion but most importantly the depth of the piles in which the strength and durability of the structure is guaranteed. In collaboration with the competent state bodies were carried out the meteorological and navigational conditions survey.

The scope of the work is to show step by step all the different phases, issues and peculiarities that a project in marine environment undergoes.

1.4 Organization of the thesis

This thesis is divided in 8 chapters. The organization is done as follows:

In Chapter 1, the problem statement, thesis objective and scope of works is presented. Chapter 2, includes the literature reviw of variuous books, publications and scientific articles related to the topic of maritime constructions. Chapter 3 consissists of a general describtion of the port of Durres and the significant aspect of the estale limits are described. The whole Chapter 4 is dedicated to the describtion of the four terminals that compose the facility infrastructure. It is completed with the mian information regarding the way of the terminal operation and the specific informations. The description of the actual structure, the underwater inspection showed the very poor condition in which the structure actualy operates. The foto and their describtion in this chapter give a perfect idea of why the need for intervention are also included in this chapter. The values of the fundamental environmental conditions that will influence the project calculationas, wind, tides, wave motion and bathymetric survey are described, without leaving aside the navigational aspect and the bathymetric survey. Chapter 5 is dedicated to the most important survey that mostly has an impact on the project, that of the soil investigation. To have a better understanding of the geomorphological conditions two distend surveys done in different occations are used. The main focus on Chapter 6 are the reliminary assesments of the project. The initial parameters that will help to bulid the base for the continuation of the project are specified. A first step in the identification of the best possible alternative in reconstructing the quays is evaluated. The evaluation goes along with the traffic data analysis, the environmental assessment and the cost benefit analysis. Chapter 7 is the chapter dedicated entirely to the detailed design process and the numerical calculations. In Chapter 8, are pointed out all the conclusion reached during the realization of the detailed design.

CHAPTER 2

LITERATURE REVIEW

The transport of goods by sea is the lifeblood of the world economy, which makes it the “de facto” proponent of globalization and its backbone. It is also at the heart of cross-border transport networks that support supply chains and enable international trade.

According to data from the "Maritime Traffic Review 20191" about 80% of world trade (in volume and 70% in value) is transported by ships, and according to the World Shipping Council, an association made up of 26 shipping companies, maritime trade moves a quantity of products worth 4 trillion dollars every year.

The modern technique of maritime constructions is based on the knowledge of the environment in which the building is inserted, of the interactions between the environment and the work to be carried out and of the functions that the latter must perform.

The knowledge of the environment includes the regime of winds, tides, currents and the state of the sea in the area, the physical-chemical characteristics of sea water, the geomorphological and geotechnical characteristics of the seabed. Among the main interactions determined by the insertion of the work in the environment are the mutual effects between wave motion, structure and seabed in front of the artifact, effects that are expressed in the reflection and diffraction phenomena of the incident wave and, sometimes, in the phenomena of erosion at the foot of the breakwaters. To identify these interactions, it is necessary to know the physical-mathematical theories of wave motion and of the reliefs in nature and in model of the effects that the waves exert on the works and on the seabed. As with any civil engineering work, the maritime work also arises in the context of spatial planning problems, both when it contributes to the expansion and defense of a port (tourist, industrial, etc.), and when it is placed to protect an eroding coast or constitutes the last branch of a hydraulic network that discharges into the sea (E. Musso,1996).

Eduardo Benassai in his work “On the refraction of the wave motion in the bay of Naples” points out that for a seaport it will be a question of making the protected basin to carry out the task of guarding against wave agitation and at the same time to fulfill, in the most favorable conditions, the function of node of various transport systems; for the defense works of a coast it will be a question of reducing the dangers of invasion of human in along the coast by the sea; for drainage works, at the mouth of watercourses or downstream of cooling hydraulic circuits serving thermoelectric power plants, it will be a question of making circulation efficient even in particularly unfavorable sea conditions, respecting the ecological balance as much as possible of the neighborhood.

The litrurature on logistic in maritime transport is one of the topic to which this work was based. Maritime transport has played and continues to play a leading role in process of integration of the world economy. Consequently, the port, as an infrastructure, affects the economy through the ability to generate a significant mass of economies from which all the subjects present in the catchment area benefit: operation infrastructure guarantees the offer of a wide range of goods and services with positive consequences for the entire contiguous system, in the logic of competitiveness. Therefore, the ports constitute a significant and strategic asset for the economic area in which they are located. There standardization of loads, integrated logistics, intermodality require connections with the hinterland and with the sea that represent a further one possibility for ports to attract traffic and influence choices, economically advantageous, in distant areas.

Over the past 10 years or so, numerous excellent paper, book and manuals on "Port and Port Engineering" and "Coastal and Oceanic Engineering" have been published in various regions all around the globe. Some of these publications are held as reference to these works. The authors of the above-mentioned works offer a complete and detailed discussion on intervention regarding the marine environment and its effects on port design, port hydraulics, port operation, coastal drift and sedimentation, coastal geomorphology, technology, port and maritime economy, and the design and construction of a floating port and other structures. In some cases, proportionally to the other aspect of port logistics and operations, the geotechnical and structural aspects of the port construction receive very little attention. This occurs, may be, because the set of engineering marine structures is in itself very broad; is a blend that incorporates the range of engineering disciplines, eg. civil, structural, material resistance, geotechnical, corrosion, hydraulic, naval architecture and others whose knowledge is required to produce a solid and economical design of a modern port or sea terminals (PIANC, 1987).

Despite the various aspects regarding the construction of a civil infrastructure such as ports, the very fundamental base literature on which to hinge the development of this works / projects remains that which deals with the type of structure. Currently, a variety of high performance sheet pile sections are available from different sheet pile manufacturers. In addition to the above-mentioned piles, often in marine applications straight-rib steel sheet piles with low relative sectional modulus are often used for the construction of cellular type bulkheads. Such a structure of piles was first produced and put in place in the United States between 1908 and 1909. Now days the usage of steel tube type sheet piles is becoming widely used for deep water construction due to their capacity improved resistance where higher strength roofing is required.

The beginning of this century saw the staring of usage of concrete sheet piles as essential elements in the construction of quays and harbors. Usually, they are considered to be components interconnected to the repair and maintenance of a wall made of sheet pile. Even thought in the last 50 years' different types of designs have been proposed and developed, the straight piling bar web equipped with a groove and tongue, it is the most used. Precasted reinforced concrete sheet piles, since the 1950s, have almost completely replaced normal reinforced concrete ones. Especially in a seawater environment, the prestressing of concrete sheet pile reinforcement presents the most advantage, since in this way the cracking of the concrete in the tension zone is largely eliminated and the risk of corrosion of the reinforcement is reduced. The same is true for ordinary concrete poles, which are widely used in marine applications. The advantage of pile-dwelling structures is that they allow virtually free passage of waves, which makes them particularly attractive for deep-water offshore terminal construction (PIANC, 1987).

CHAPTER 3

PORT REVIEW

3.1 Introduction

The Port of Durrës is located in south-western part of Durrës City at the north end of the Bay of Durrës, an extensive body of water between Kala e Turrës and Cape Durrës, in the west of the Albanian coast line. Its geographical coordinates are Latitude 41°19'N and Longitude 19°27'E. The geographical location makes the port the largest port of Albania and one of the largest among ports in the Adriatic and Ionian Sea. The port is situated in the south-west part of the city of Durrës and 38 km East of the country's capital, Tirana.

The port is formed by two breakwaters which protect an inner basin of 67 hectares with a depth ranging from 7.5 to 11.5 meters and 11 berths with about 2200 meters of quay. The total land area is about 79 hectares.

In general, it can be noted that presently available area is not used optimally, with presence of scrap and old warehouses which are out of use and occupying valuable port area and with the presence of wrecks occupying parts of the basin. Cleaning up the port area and re-distribution of the terrains is a quick-win, allowing improvement of the operational efficiency of the port on the short term and without any major investments.

The Port of Durrës is the most significant seaport in Albania and it is one of the most important ports for cargo traffic in the Western Balkans and the SEETO region. According to INSTAT, during 2020 it handled roughly 91,6% of the country's seaborne trade in tonnage terms and 52,9 % of all the export and import trade (by value) of the country.

Figure 1 - General view of Port of Durrës

The port consists of four specialized terminals, a container terminal, a dry bulk terminal, a general cargo terminal, and a ferry terminal. All terminals' operations have been privatized under concession contracts, except for the general cargo terminal (the GCT or the West Terminal) which remains owned and operated by the Durrës Port Authority (DPA).

3.2 Port Estate Limits

The port of Durres is located 38 km away from Tirana, north of Durres Bay. It processes passenger ferries, Ro-Ro ships, container and general cargo ships, as well as bulk cargo ships, while oil and gas operations have shifted to Porto Romano.

The port of Durrës represents almost 90% of Albania's maritime trade, a growing trend also due to the increasing containerization of goods transported by sea. The port has a total area of 79 ha (793,000 m2), where only the basin has an area of 67 ha (674,000 m2). The entrance canal has a length of 6.7 km and a width of up to 115 m. The basic technical characteristics of the Port of Durrës are:

11 piers with a depth ranging from 7.5 m, in piers 1 and 2, up to 11.5 m in pier 11. The length of piers for processing is 2,275 km and the processing capacity is about 5 million tons / year. The port is currently organized in four terminals:

- Ferry Terminal
- Container Terminal
- Dry Bulk Terminal / East Terminal
- General Catgo Terminal / West Terminal

The port currently operates in part as a "Land lord" where the operations of two of the terminals are performed by the private sector (bulk and ferries), while the other two terminals (containers and general cargo) are operated by the Durrës Port Authority. Over the last decade, the Port of Durres has implemented only 30% of the initial Master Plan approved in 2009, and has been significantly constrained by the surrounding urban pressure. In the last decade, the only investment in port infrastructure that the Durrës Port Authority has been able to realize with self-financing has been the deepening (still in process) of the basin and the access channel.

Figure 2 - Durres Port estate limits

CHAPTER 4

PORT INFRASTRUCTURE

4.1 Terminals

The Port took its actual configuration in the ’50 where important investment in terms of human recourses and economic terms were made. Till then no significative changes are made to the port infrastructure as a whole. As previously mentioned the port facility is divided into four terminals each operating in specific activity.

Figure 3 - Port Terminals

4.2 General Cargo Terminal / West Terminal

Figure 4 – General Cargo Terminal layout

The western terminal is located on the left side of the entrance to the port basin and is bordered by the area designated for the navy and law enforcement agencies such as police and customs on the one hand, and the container terminal and the administrative and logistics area on the other. The initial quays (quay no. 1-4) were built in 1949-1951 and reconstructed in 1994. Later, specifically in 1996, the terminal was expanded with quay 5. The terminal is operated by the Durrës Port Authority and processes various loads of general and cereals. The terminal is equipped with 5 grain silos.

4.2.2 General Cargo Terminal Specifications

Terminal processing capacity: if we consider a terminal capacity of 1.5 million tons, the yield of the site (ie processing capacity per hectare of the terminal site) is estimated at 162,000 tons / ha. This is high efficiency for a general cargo terminal. International comparative figures show an average square yield between 40,000 tons / ha and

100,000 tons / ha. For the design of a new general cargo terminal it is recommended to work with an average square yield of 70,000 tons / ha.

Length of the quays, number of landing site and terminal square: currently, the western terminal includes 3 different quays due to the angles and corners in the planimetry of the quay wall no.2 (293m), quay no.4 (173.8 m) and quay no.5 (235.9 m). This means that terminal cranes can only be used below the optimum at 1 quay, leading to potential operational inefficiencies and underutilization of equipment. Also, this means that the length of the terminal can only be used below the optimum, as a result of the empty space in front of and behind the anchored vessel.

The terminal provides 3 to 4 berths, depending on the LOA of the ships in the docking:

- quay no.2 has a length of 293 m, which should be enough to process 2 vessels of total cargo up to 100 m LOA. However, the depth near the quay wall, which allows only draft vessels up to 6.6 m, is a significant limitation. Even after the deepening works in the basin and the port entrance canal, this restriction will continue to exist until the completion of the quay wall renovation project.

- quay No. 4 has a length of 174 m, which should enable the accommodation of 1 vessel with LOA up to about 130 m. However, even here the limited navigation draft constitutes a considerable limitation.

- quay no.5 has a length of 236 m, which should enable the accommodation of 1 boat with LOA up to 180 m. After deepening the basin and the inlet channel, this landing site will allow draft vessels up to 9 m.

Table 1 – General Cargo Terminal volumes rapresentation

Volume of goods processed per hour in 2018:	669.454 ton	Volume of goods processed per hour in 2019:	773.319 ton
nr. of ships in 2018:	251	nr. of ships in 2018:	277
Avarage cargo volume in 2018:	2.667 ton	Avarage cargo volume in 2019:	2.788 ton
Ratio import/export	96% / 4%		

Table 2 - Ships's particular that mostly bearth General Cargo Terminal

LOA	nr. of bearths	LOA avarage	GT avarage	DWT avarage
60m – 80m	59	75m	1.400 ton	1,800 ton
80m – 100m	137	86m	2,100 ton	3,600 ton
100m – 120m	50	108m	3,450 ton	5,100 ton
120m – 140m	24	135m	5,400 ton	6,500 ton
140m – 160m	4	148m	9,850 ton	14,600 ton
+ 160m	3	184m	27,000 ton	42,300 ton

The WT plays a very important role in the port revenues. If we compare the revenues of the recent years a progressive decrese of volume will be imideately noted. This mainly due to the poor condition of the terminal. It I now clear that the DPA in order to achive a growth volume need to intervene in improving the superstructures conditions. This procces may not be as easy as it may seem but it is necessary to maintaine competiviness.

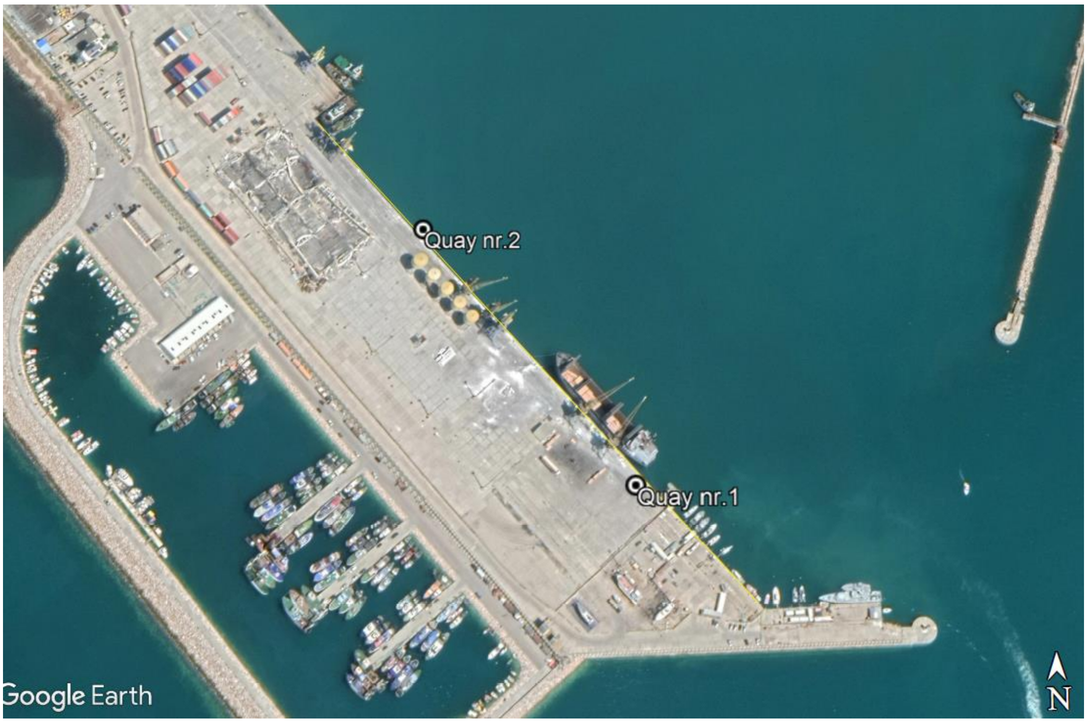


Figure 5 - Position of quay 1 and 2 in the General Cargo Terminal

4.3 Entrance Channel.

The navigable entrance channel, is oriented in such way to bring it as close as possible to the dominant winds. Another important factor that identifies an access channel is its depth. The depth of the access channel with respect to the mean sea level is determined by the sum of the following contributions

- low tide level;
- fully laden draft of the project vessel;
- increase in navigation of the draft of the stern called "squat";
- wave motion;
- security free;
- dredging tolerance.

Taking in consideration all the above and the container vessels taken as reference vessel the channel can guarantee the passage of vessels having a draft of 9m. Intense investments are done by the Durrës Port Authority to maintain this level of depth. This due to the low module of sand in the Durrës bay which can be easily transported especially in storm condition.

The minimum width of the access channel was determined considering a value of speed of 6 knots and it is delimited by 3 fares pairs of lights, the turning light and the two entrance lights.

4.4 Port Basin

Ports of commerce are called those ports, which, in addition to ensuring the access and parking of ships, are equipped with stretches of water and basins where the ships themselves can easily and quickly proceed with commercial operations.

These basins therefore offer landing stages and adequate means and facilities for the loading, unloading, storage and transit of goods, for the embarkation and disembarkation of passengers and mail, for the repair and supply of ships. With a complex of works and preparations that is present today, for the importance and variety of the constituent elements.

The basin of Durres Port has a compressive area of nearly 67ha. It can be reached by passing through an entrance having width of 202m. The turning basin of 520m guarantees the sufficient space for ships to turn while maneuvering even during adverse weather conditions.

Figure 6 - Port basin and the red and green lights that delimit the port entrance

4.5 Bathymetric survey

Bathymetry is a discipline of oceanography and geodesy that deals with the measurement of depths and the cartographic representation of the seabed. The bathymetric surveys are therefore carried out for the knowledge of the morphological trend of the seabed and lakes. They are necessary for the design and monitoring of maritime and river works, in dredging and nourishment and in studies for coastal erosion. Surveys are generally carried out from a vessel equipped with a precision echo sounder, Single beam or Multi beam. The location is provided by a GPS receiver. A bathymetric software allows to follow the planned survey routes and to acquire the data coming from the different instruments.

For updating the bathymetric data of the Albanian coast, the Albanian Hydrographic Service "Shërbimit Hidrografik Shqiptar (SHHSH)" is the state body in charge. The measurements were carried out periodically respecting al the standards of the International Hydrographic Organization (IHO).

The coastal bathymetric survey is conducted by the means of a single-beam echo sounder following the seabed profile lines, generally placed orthogonally to the coast, supplemented by further routes perpendicular to the previous ones. The surveys are generally carried out in a completely calm sea and in the absence of wind. The sounding sections are covered with a boat that moves at a low and constant speed along the predetermined routes. Generally, small boats with low draft are used for this type of relief as they must get as close as possible to the shore or breakwaters.

For the Durrës port the area interested has a surface of 1.896.174 m^2 by the survey is included between the following coordinates:

Survey reports are mainly compiled for the Albanian Navy Force. Their distribution is carried out in accordance with existing cooperation agreements with third parties and is sent to institutions such as the Institute of Geography and Military Infrastructure (IGJIU) and the General Directorate of Maritime Administration (DPAD) as well as the Durrës Port Authority (APD).

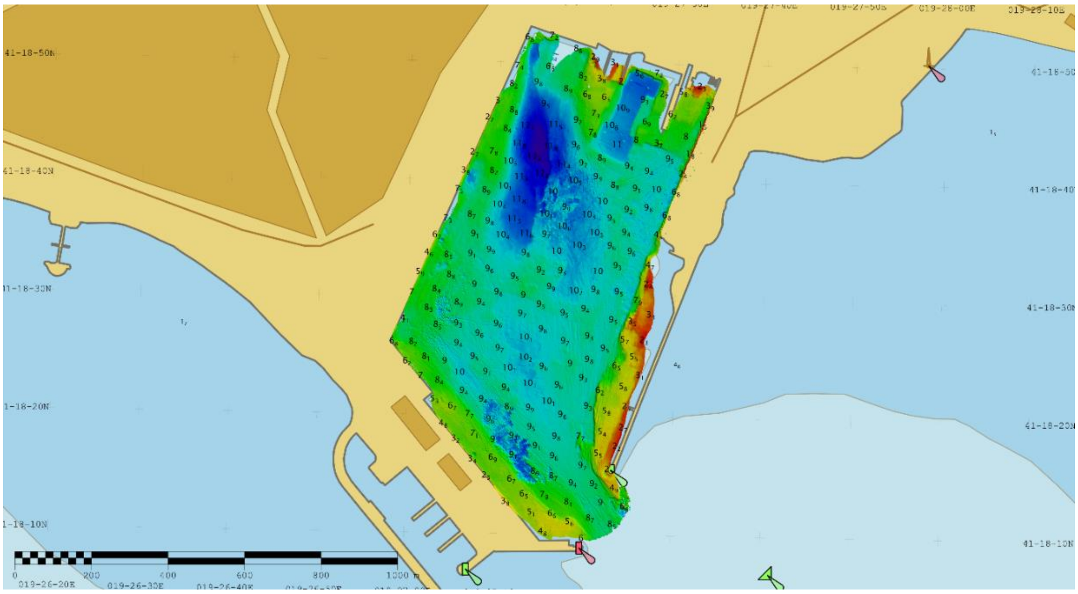


Figure 7 - Basin bathymetric survey

4.6 Wind records

The knowledge on the regime of winds, tides and sea currents are more specifically part of other disciplines (meteorology and oceanography) and therefore are not dealt with in full here.

Wind direction data is presented in the below table. A meteorological station is located in the immediate area of the Durres port. It must be noted that this location is protected from winds in the direction of east and northeast and this results in low frequencies of winds from these directions. As can be concluded from this table, the dominant wind direction is North-Wind. However, the strongest winds come from southeast and southern direction.

Table 3 - Wind direction and frequency

Item	Wind Direction and Frequency								
Wind direction	**Calm**	**N**	**NE**	**E**	**SE**	**S**	**SW**	**W**	**NW**
Frequency (%)	6.2	27.3	4.3	3.8	21.9	7.6	6.6	13.3	8.5

In the Table 8 the maximum wind speed for different directions is presented and are being derived from a Hydro Meteorological Institute study of 1993. These wind data were used in previous design works to dimension the mooring loads on the quays.

Table 4 - Maximum wind speed

TR [years]	**Safety [%]**	**Maximum Wind Speed [m/s]**							
Wind direction		**N**	**NE**	**E**	**SE**	**S**	**SW**	**W**	**NW**
50	2	28	21	24	35	41	41	31	21
20	5	25	19	22	32	36	35	28	21
10	10	23	17	20	28	32	32	26	20
5	20	21	16	19	26	29	28	24	18

4.7 Wave Motion

Of great importance for maritime construction is the knowledge on the wave motion produced from the wind.

The wave motion is characterized by a sequence of more or less large regular direct oscillations in the same direction of the wind, without transport of fluid mass. The distance travelled from the wind above the body of water before reaching the mainland is called fetch. The bigger the fetch the greater the possibility of developing large waves. (R. Bertoni, 2018).

Surface waves initially appear as small surface ripples free due to the effect of the wind and are called capillaries. They have wavelengths of some centimeter and width of the order of a millimeter. In this first phase there is an interaction between the action of the wind and the intrinsic forces of sea water (viscosity and surface tension). Self the wind energy is insufficient to break the balance with these forces the surface becomes calm again, otherwise, as soon as the wind exceeds certain values of intensity and duration, they begin to to form waves of greater amplitude and period that can maintain and propagate even if the wind ceases. If friction is ignored, gravity is the only force acting on this type of waves. Therefore, these waves are also called gravitational. The effect of the wave motion is maximum in surface and acts less and less in depth: in deep water conditions, in fact, this it does not act on the seabed. (G.Trossarelli, 2014)

To identify the state of the sea in a given area, it is necessary to define some characteristic quantities of the wave motion. The length between two successive crests is called the wavelength. The ratio between the wavelength and the period is given the name of propagation celerity. The ratio between the height H and the width L, otherwise known as steepness, is still of considerable interest for an evaluation of the origin of the waves. The statistical distribution of the waves of a given set or train of waves is usually identified through the definition of a significant wave, that is, that wave that has the average values of the heights and periods of the highest third for height and period of the waves present in a given wave train.

It has been found, through the analysis of numerous wave recordings, that the height of the significant wave coincides with a good approximation with the height deducible from purely visual observations of the sea state. A wave disturbance study has been carried out, by means of numerical simulation, in order to investigate the effect of the reconstruction works of Quays 1 and 2 on the wave disturbance inside the port basin.

The basic conclusion arising from the wave model regarding the current situation of the port (without any extension of the Quays 1&2) has as follows:

There is some wave penetration into the port basin for waves coming from S direction. The maximum wave height does not exceed the 0.75m at quays 1 & 2.

The basic conclusions that can be drawn from the investigation of the wave disturbance after the reconstruction of Quays 1&2 and the dredging works, have as follows:

The wave heights in the port entrance are increased after the construction of the new quays. The wave disturbance in the port basin is affected by the construction of the quays but the wave height even for the south incoming waves remain very low inside the port.

Table 5 - Maximum wave heights at the port entrance and at quays 1 & 2 before and after the construction of Quays 1&2.

	Before the construction of the Quays 1&2		**After the construction of Quays 1&2**	
Direction of incoming waves	**South**	**West**	**South**	**West**
Port Entrance	2.25m	1.25m	2.75m	1.5m
Quays 1&2	<0.75m	<0.5m	<0.75m	<0.5m

Harbor oscillations (seiching), a standing wave related to the natural period of the harbour basin; Long waves (also called infra-gravity waves, period between 30 and 300 s) induce resonance of the moored ship. Long waves, excessive ship motions and breaking lines are still a problem today. Over the last decades several methodologies have been derived to compute the behaviour of moored ships in complex wave fields These methodologies typically consist of 3 steps:

1. Assessment of the wave field at the mooring location, taking into account all wave processes (refraction, diffraction, shoaling, reflection, breaking, etc.);

2. Computation of the wave forces on the restrained ship using the outcome of step 1;

3. Computation of the response of the moored ship taking into account the nonlinearities of the mooring system.

During the measurements the surge, sway, yaw and roll motions, the motions of the moored vessel were measured with two GPS receivers located on the wings of the bridge. During most of the time, the GPS receivers operated in RTK (Real-Time Kinematic) mode, which relies on a single reference station to provide real-time corrections, providing up to centimetre-level accuracy. The case study has shown that the resonance response of the moored ships has a significant reduction in the quay structure has on pine instead of sheet piles or gravity blocks.

4.8 Tides

The fluctuation of water levels along the Albanian coast is due to tide phenomena and hydro-meteorological events (especially winds). The tidal period duration is 12 hours and is irregular. The tide processes along the Albanian coast are very week and the daily fluctuation in amplitude of the water level averages from 20-30cm.

For all above values reference is made to the information provided by "Geoscience Institute and Energy, Water and Environment (IGJEUM)

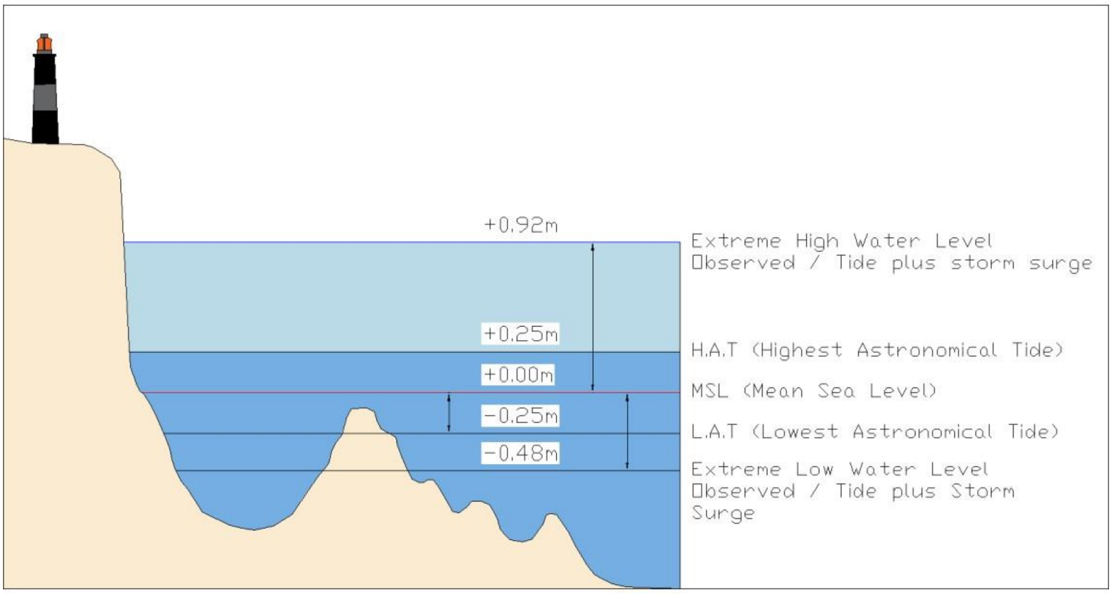


Figure 8 - Tides in Durres Port according to IGJEUM mesurements, letter prot 90/1 dt.08.04.2019

According to same source, the Lowest Astronomical Tide (LAT) at Durres is -0.25m below MSL.

4.9 Quays actual conditions and the need for intervention

The idea of improving the quays conditions and the yard connected to them has very early roots. The last Masterplan compiled for the Durres Port was done in 2008. One of the Masterplan impportand intervention in the facility infrastrukture was the repairing of the quays and yard in the West Terminal. Following up the Masterplan indication the Durres Port Authority comissioned the preliminary evaluation of that time existing construction conditions. The company in charge for the study was an Italian company called Selhorn s.p.a. In their evaluation of the most of the port facility infrastructures thay also pointed out the necessity of improving the conditions of the quays one and two.

At thi point the Albanian government in collabaration with the DPA were able to guarantee a grand from the Europian Union. Grand that was part of the financing pacage for the Western Ballkan Investment Framework.

The grand will guarantee for the entire detailed design and partialy the construction proces.

The Western Balkans Investment Framework (WBIF) is a financing facility launched in December 2009 by the European Commission, together with the Council of Europe Development Bank (CEB), the European Bank for Reconstruction and Development (EBRD), the European Investment Bank (EIB), Bilateral Donors, and Western Balkans countries with the purpose to deliver funding for strategic investment projects in beneficiary countries. Eligible sectors include infrastructure development in the environment, energy, transport, social and digital sectors as well as private sector development.

The consortium chosen for the detailed design is the IPF7. A very usefull colaboration started between the consortium and the DPA representative part.

The quay 1 and 2 are build respectively in 1954 and 1972 according to the designed showen in the below figures. In 1994 the yard was totally reconstructed. This intervetion did not touch and part of the structure under the yard pavement.

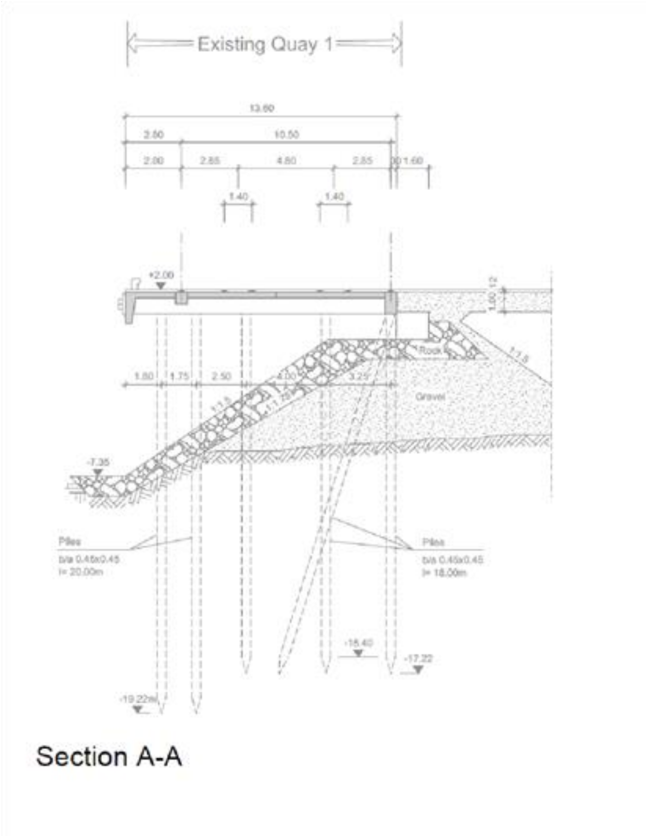
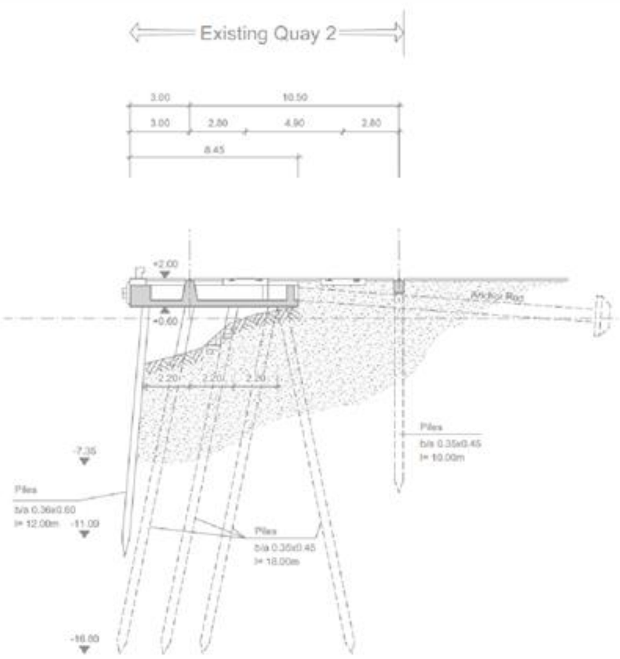


Figure 9 - Quay nr.1 actual design

In oreder to have a better understanding of the situation more than one inspection were done to the quays piles, wall, berthing line ect. From the first steps of the investigation the situation seemed to be critical. The underwater struction showed a very heavy corrosion. The absence of maintenance has led the concrete to disintegrate due to the wave motion action. Without the protection of concrete, a chain of non optinal events can start. The most common and import is the corrosion of the steel bars which guarantee the strenght of the entire structure.



Section B-B

Figure 10 Quay nr.2 actual design

On 19 and 20 October 2020, an underwater inspection took place. The inspection started from the corner between Quaywalls 2 & 3 and ended at the corner between Quaywalls 0 & 1. The condition of Quaywalls 1 & 2 can be depicted in photos below

Figure 11 - Cavity at the joint between Quays 2 and 3. This damage may lead to filling material loss as mentioned before

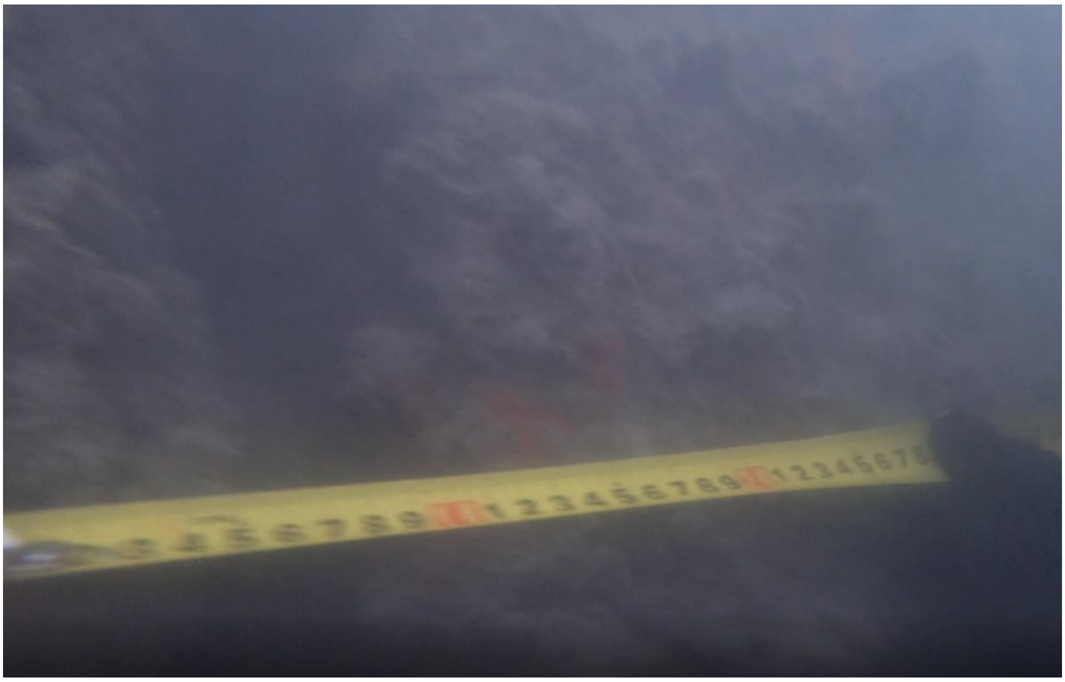


Figure 12 - Void between retaining piles of Quaywall 2

Figure 13 - Lightly damaged apron of Quaywall 2.

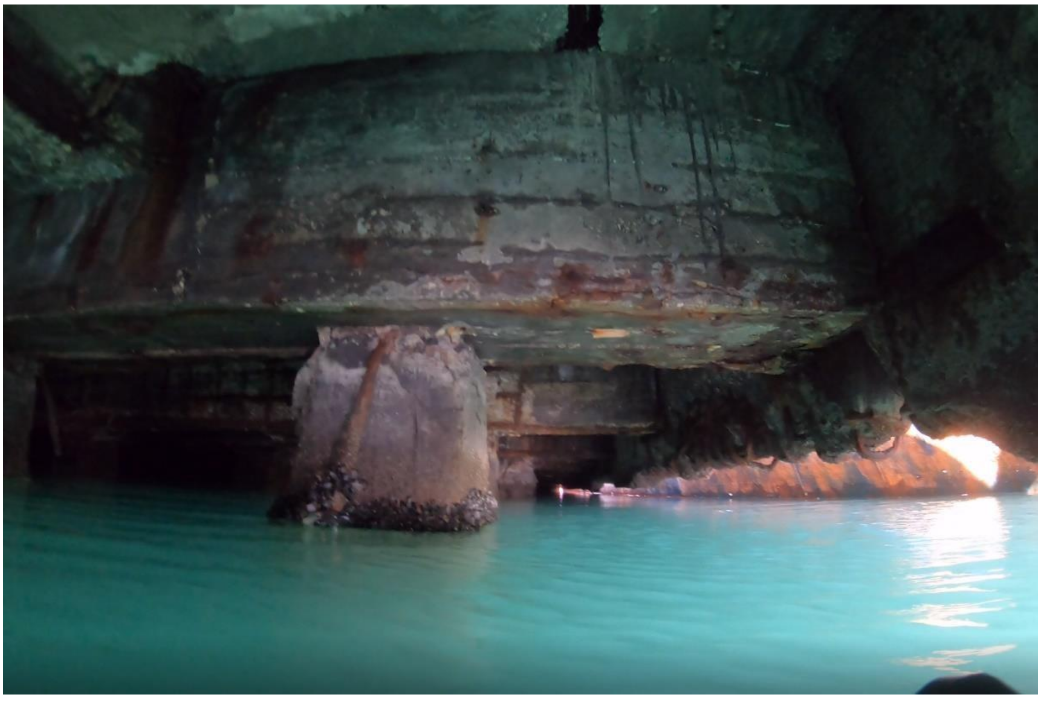


Figure 14 - Corrosion of RC structure

Figure 15 - Exposed heavily corroded reinforced bars at the apron's bottom of Quaywall 1.

Figure 16 - Exposed heavily corroded reinforced bars at the apron's bottom and poor connection joint with the vertical pile - Quaywall 1.

Figure 17 - Exposed heavily corroded reinforced bars at the apron's bottom - Quaywall 1.

It must be noted that the underwater inspection took place in low visibility conditions due to the quality of the water. Despite that, the retaining wall of the Quaywall 2, seems to be in good conditions with minor voids between the pile elements and minor damages and deteriorations of the apron.

On the other hand, Quaywall 1 deck's inspection showed that it is heavily deteriorated. More specific, at the bottom side of the deck, the reinforcement bars are visible and heavily corroded in extensive manner. Moreover, the connections between pile and deck in lot of locations seem to be poor and malfunctioning.

CHAPTER 5

SOIL INVESTIGATION

The soil is a collection of rock fragments (the granules) of dimensions that can vary in very wide intervals. Rocks, which are the result of magmatic, sedimentary or metamorphic activities, after being crushed by mechanical agents, thermal or chemical, they can remain in place and thus give rise to residual soil or, once shattered and therefore made more easily mobilized, they can be reworked and transported by wind or hydraulic carriers, hence the name of land transported. In any case, as a whole, the ground can be schematized from the geotechnical point of view as an aggregate of irregularly shaped particles of various sizes, separated by empty spaces (interstices) or occupied in whole or in part by water. Underwater soils retain their negative qualities longer, especially those composed of very fine granules (silts and clays), in fact, in this case, the natural compaction of its own weight is counteracted by the under-thrust of the water and the cementation by the solvent property that water exerts against many cementing substances.

The soil investigation process was done by the consortium in collaboration of the DPA rapresentatives, and was a integration of the survey done in 2012 and that done in 2019.

5.1 Drilling and Sampling procedures to be applied

Depending on the type of work and/or the extent of the intervention, as well as the complexity of the geological system and the soil-structure interaction, specific and complete geological and geotechnical investigations on site and in the laboratory must be planned and carried out. Comprehensively allow the definition of the geological and geotechnical model.

The preliminary geological report is drawn up for the purpose of a geological framework and the hydrogeological and geomorphological characteristics, in addition to the feasibility of the work and what are the useful data for the design of the work, of the area in which the design of the new quays. The following report accompanies the project, for which indication will be given regarding the investigations to be carried out, preparatory to the drafting of the final project.

In order to have a precise procedure through all the soil investigation progress a pre-determined framework made of a set of steps were followed.

5.2 Drilling and Sampling

Drilling is performed with the aid of hydraulic rotary drilling rig(s).

The drilling rig(s) will be equipped with the necessary tools that will improve sampling quality, such as single and double tube barrels 101mm and 86mm external diameter, split inner tube core barrels 101mm, temporary casing 131mm and 114mm external diameter, undisturbed samplers, SPT samplers and drop hammers.

In addition to these, a piston sampler is required to be used in very soft soil layers.

For the offshore boreholes, a suitable jack-up platform or a barge to be used with sufficient dimensions to accommodate drilling equipment, casing pipes and temporary storage for the core boxes and also to allow drilling operations to be carried out effectively. In case a barge is used, it should be equipped with four heavy anchors, at least two winches and sufficient lengths of cables to secure stabilization during drilling operations.

a. For drilling in soil formations a single tube core barrel, with carbide bits and 101mm in external diameter was utilized. The use of drilling fluid was avoided were possible, in order to preserve the samples' natural water content and avoid washing down of fine grained material. In case of the use of drilling fluid, core runs were limited to a length of 50cm to 100cm with the last 20cm of each run being drilled without the use of drilling fluid. For drilling in rock formations double tube core barrels was used with diamond drilling bits and 101mm in external diameter, with or without split inner tube where applicable.

b. Drilling in poor soil or rock conditions, such as fragmented rock, temporary casing was used in order to protect the borehole walls from collapsing. Temporary borehole casing was installed during drilling and after sampling. Casing tube external diameters will be 131mm and 114mm and their depth depended on soil or rock conditions. After reaching its final depth, casing tubes will be removed.

c. In soil formations standard penetration tests (SPT) was executed at 2m intervals and undisturbed soil samples was obtained at 3m to 5m intervals with the aid of Shelby tubes 75mm in diameter with inner PVC lining, were deemed necessary. If very soft soil layers are encountered, a piston sampler waso be used at 3m to 5m intervals.

d. In soft rock or rock formations, continuous sampling was executed and RQD (Rock Quality Designation), TCR (Total Core Recovery) and discontinuities description was recorded, so that the minimum TCR will be not less than 80%.

For each borehole, a description of layers was performed by an experienced Geologist or Geotechnical Engineer, based on the retrieved samples.

The drilling fluid used was the sea water.

Standard penetration tests were performed at 2m intervals in soil formations, in order to obtain the in-situ density of cohesionless soils and/or consistency of cohesive soils.

5.3 Storage and Handling of Samples

After the end of each core run, samples were carefully extracted from the core barrel and placed in an appropriate wooden core box, which will comprise from 4 or 5 compartments each 1.0m long. The box had a wooden cover with a latch and rope handles on its side.

Cohesive soil samples were cleaned to remove free moisture and contaminated soil. They were then paraffined in order to preserve natural water content and were placed inside two polythene bags, each one being sealed separately after removing excess air. Non cohesive semi disturbed samples were also placed inside polythene bags by applying the same handling procedure as for cohesive samples without paraffining them. Undisturbed samples were marked and then paraffined and wrapped with plastic membrane and placed into two polyethylene bags. All samples were properly labelled.

Soft rock and rock cores were carefully extracted from the samplers and were placed into the boxes properly aligned and oriented. Rock cores extracted from a split inner tube core barrel were placed in a PVC pipe, 75mm in diameter, sawn in halves. Beginning and end of each core run were marked with wood partitions. When drilling in soil, the depth was marked at one meter intervals at the sides of the core box.

The top cover of the core boxes contains information regarding the job title, client, borehole name and number of the box, depth, and sampling dates. Same information was noted in a label nailed to the inside part of the cover.

After completion of each borehole, all core boxes were transferred to the testing laboratory.

Representative samples were selected by the Geotechnical Consultant for laboratory testing which that were forwarded to the testing laboratory.

Photographing of the samples were carried out on site with a high resolution camera. Each photograph refered to a box where the title of the project, the number of the borehole and the beginning and end of each row of samples will be shown. During photographing, care was taken so that there are no shadows that may falsify the quality of the photograph of the samples. Additionally, a special color scale reference will be provided in each box. The photographs took on site before the selection and removal of samples for the performance of laboratory tests. Checking of the photograph quality were carried out in situ before the selection – removal of samples for the laboratory tests.

Selective samples were collected accordingly for the execution of laboratory tests. For a better understanding of the geological conditions of the soil nearby the quays a combination with the recent geological study an earlier geological survey of 2012 was used.

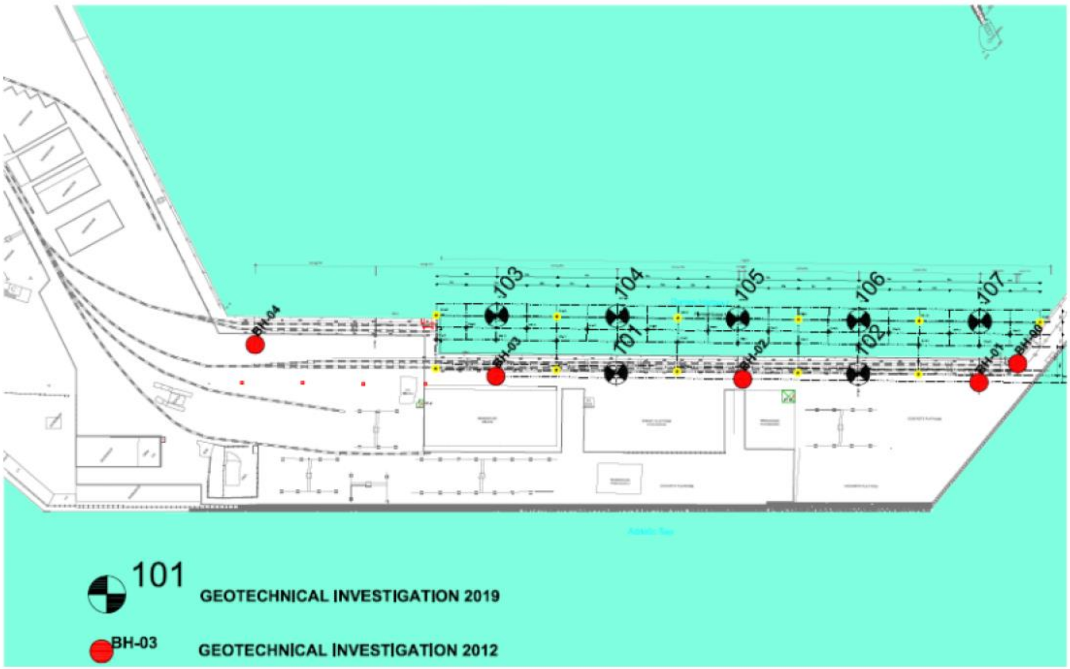


Figure 18 - Borewhole position of the goelogical investigation done in 2012 and 2019

Considering all the available geotechnical data and the conducted, the Geotechnical Profiles of Areas A & B showed in Table 10 & Table 11, have been drafted. That means, that the geotechnical stratigraphy changes from the northern to the western side, along the quay wall.

Table 6 - Geotechnical parameters for Design Area A

Soil Layer	**Thickness (m)**	**γt (kN/m³)**	**Cu (kPa)**	**c' (kPa)**	**ϕ' (deg)**	**Eoed (MPa)**
FILL	4,5					
I	6,5	18	12	4	22,5	2
II	9	18,5		1	29	9
III	7	19,5	50	8	25	8
IV	25	19,5	120	11	25	20
V	10	20	220	14	27	35

Table 7 - Geotechnical parameters for Design Area B

Soil Layer	Thickness (m)	γt (kN/m³)	Cu (kPa)	c' (kPa)	ϕ' (deg)	Eoed (MPa)
FILL	9,5					
I	6,5	18	25	6	23	3
II	6	18,5		1	29	9
III	10,8	19,5	50	8	25	8
IV	20,5	19,5	115	12,5	25	18
V	5	20	210	12,5	27	34

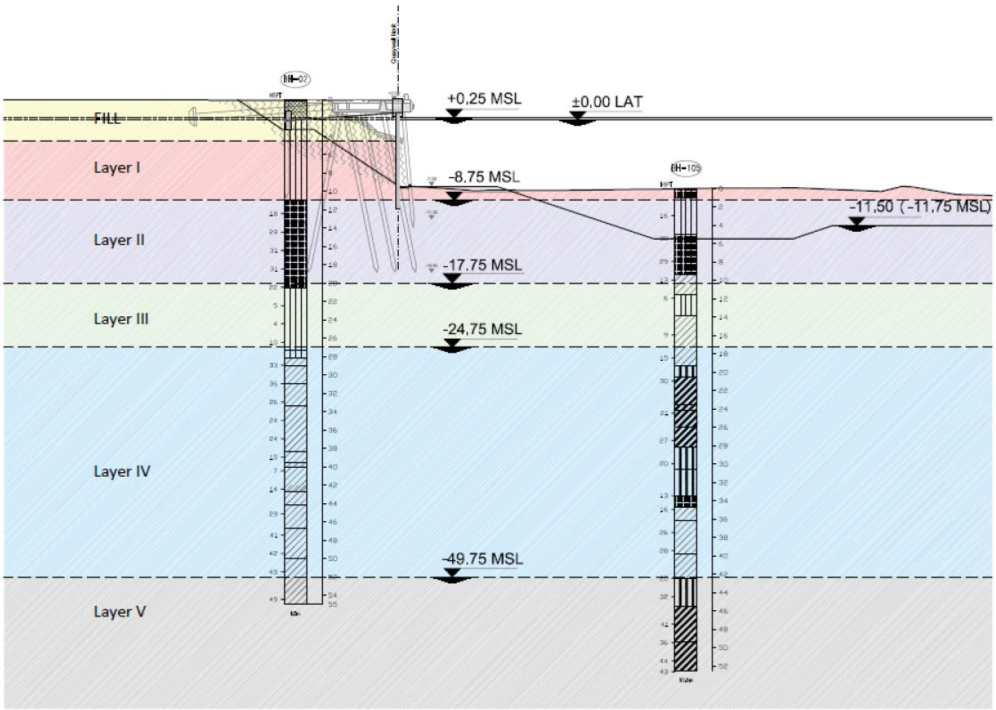


Figure 19 - Geotechnical profile of the Area A which includes works sektion quay nr.1.

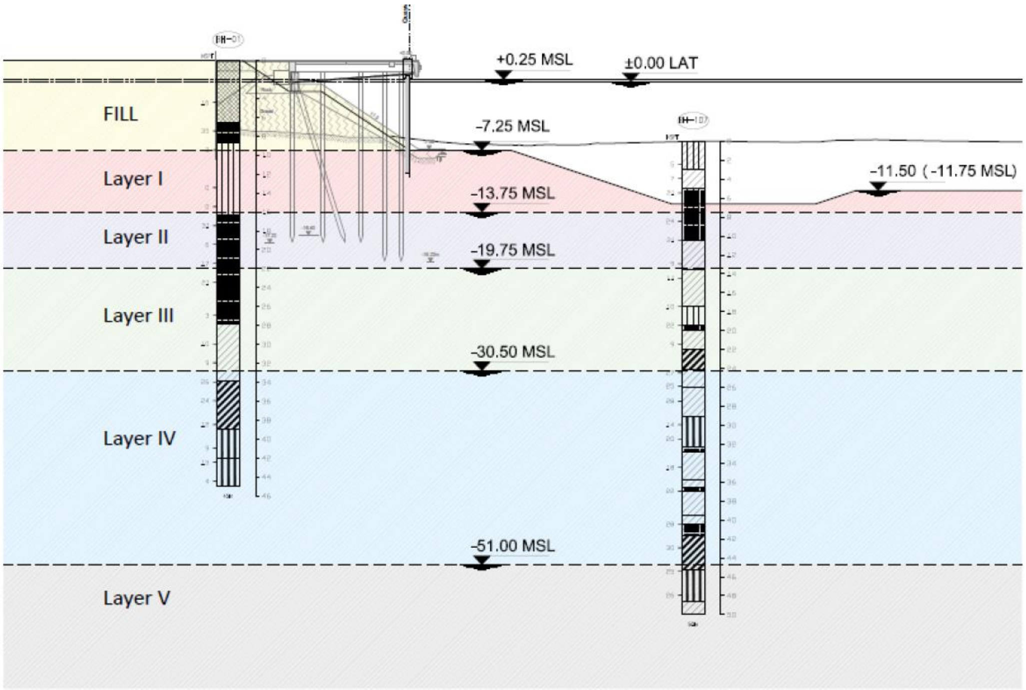


Figure 20 - geotechnical profile of the area B which includes work sections quay nr.2.

CHAPTER 6

PROJECT PRE ASSESMENT

Maritime transport has played and continues to play a leading role in process of integration of the world economy. The productive evolution that has taken place in the maritime transport sector has made it possible and economically advantageous displacement of huge quantities of finished and semi-finished goods, raw materials and products agricultural over ocean distances. To this end, ports act as catalysts for economic activities connected to them and contribute to determining successes or troubles of businesses, generating effects of income creation and redistribution. The performance of a port has a significant impact on the location choices of companies, as it activates a mechanism for the distribution of goods and services fast and efficient, reduces trading costs, opens up new market opportunities and increases the efficiency of the economic system. Consequently, the port, as an infrastructure, affects the economy through the ability to generate a significant mass of economies from which all the subjects present in the catchment area benefit: operation infrastructure guarantees the offer of a wide range of goods and services with positive consequences for the entire contiguous system, in the logic of competitiveness. Also their demand for goods and services necessary for the operation of the infrastructure determines further positive repercussions for the economic fabric. Therefore, the ports constitute a significant and strategic asset for the economic area in which they are located. There standardization of loads, integrated logistics, intermodality require connections with the hinterland and with the sea that represent a further one possibility for ports to attract traffic and influence choices, economically advantageous, in distant areas.

6.1 Identification of possible alternatives

Several alternatives were examined in the context of “Preparation of Conceptual Design, Final Design and Bill of Quantity for Reconstruction of Quays 1&2”, regarding the method of supporting the deck and the formation of the front wall to achieve the proposed depth of -11,50 m. The alternatives were either front sheet piles with grouted anchor combined with structural fill, or completely open deck construction with vertical piles and formation of the sea bed with dredging, or with vertical piles and rear sheet piles with grouted anchor and protection of the sea bed slope with quarry run and geotextile filter, or front sheet piles combined with vertical piles and rear sheet piles with grouted anchor and structural filling in between.

Finally, the selected alternative adopted in the final design stage was a combination between the above which consisted of a deck supported on vertical piles with rear sheet piles as retaining wall without grouted anchor and the formation of the sea bed was proposed with structural fill protected with geotextile filter and quarry run. The new quay shall consist of a jointless concrete deck with a cast in-situ beam girder, transversal and longitudinal beams and precast slabs with concrete topping on concrete piles with a diameter of 1,5 m. The offset between harbour depth at CD-11,5 m and terminal area at CD+2,0 m is formed by a slope with an inclination of 1 in 3 covered with revetment beneath the deck and a steel sheet pile wall at the landside edge of the deck.

After review of the existing documentation, site visits and discussions with stakeholders, the Consultant identified three options for dealing with the quays. In addition, one of these options is further divided in two alternatives, depending on a phasing or not of construction works.

• The first option is the "business as usual" scenario, in which no additional work is carried out and only regular maintenance of the infrastructure and equipment takes place at an estimated annual cost of €400,000. Under this option, maximum annual production is calculated at approximately 960,000 tons and the required storage area of 18,840 m² is fully available within the existing footprint. This will be referred to as "Option A".

• The second option is proposed by the consultant and consists in a rehabilitation of the existing quay wall, construction of two arrays of concrete piles supporting two concrete beams to serve as foundations for the crane rails, demolition of the existing deck and construction of a new concrete deck and apron, and deepening of the sea level. Under this option, the existing quay wall line is not changed. The estimated construction cost is €27.1 million. Under this option, maximum annual production is calculated at approximately 1,800,000 tn. When compared to the traffic forecast, this capacity would be fully used by year 15 or year 20 at most. The main bottleneck is the cranes which are old and have limited capacity (5t/cycle). Replacing these with six cranes of larger capacity (10t/cycle) at a total estimated cost of €3 million would increase maximum annual production to approximately 2,700,000 tn. This would allow to satisfy the highest demand envisaged under the traffic forecast over the forecasting horizon for total project costs of approximately €30 million. The required storage area of 52,000 m² is fully available within the existing footprint. Annual maintenance costs are estimated at €520,000. Duration of construction is 18 months. This will be referred to as "Option B".

• The third option is the one prepared in the 2012 project, for which the consultant has devised two variants. It consists of the construction of a retaining wall at the existing quay line and of a new deck supported by an array of drilled piles, bringing the new quay line in the continuation of quay 4. Under this option, maximum annual production is calculated at approximately 1,800,000 tons, equivalent to that of option B without cranes replacement. Construction costs are estimated at €46.3 million This option will be referred to as option C1. A variant of that option considers staging the work with Quay 2 upgraded first and Quay 1 following 6 years later. This variant will be referred to as option C2. Under option C2, maximum annual production is calculated at approximately 1,200,000 tn.

As for option B above, the terminal would not be able to handle the expected long term traffic growth without replacement of the existing cranes with six new larger ones. After replacement, maximum annual production for both C1 and C2 would increase to approximately 2,700,000 tons, equivalent to Option B. The required storage area would be 52,000 m² under option C1 (equivalent to option B) and 34,000m² under option C2. In both cases, ample space is available within the existing footprint to meet these requirements. Annual maintenance costs are estimated at €520,000. Duration of construction is 24 months for Option C1, and 2 X 12 for Option C2.

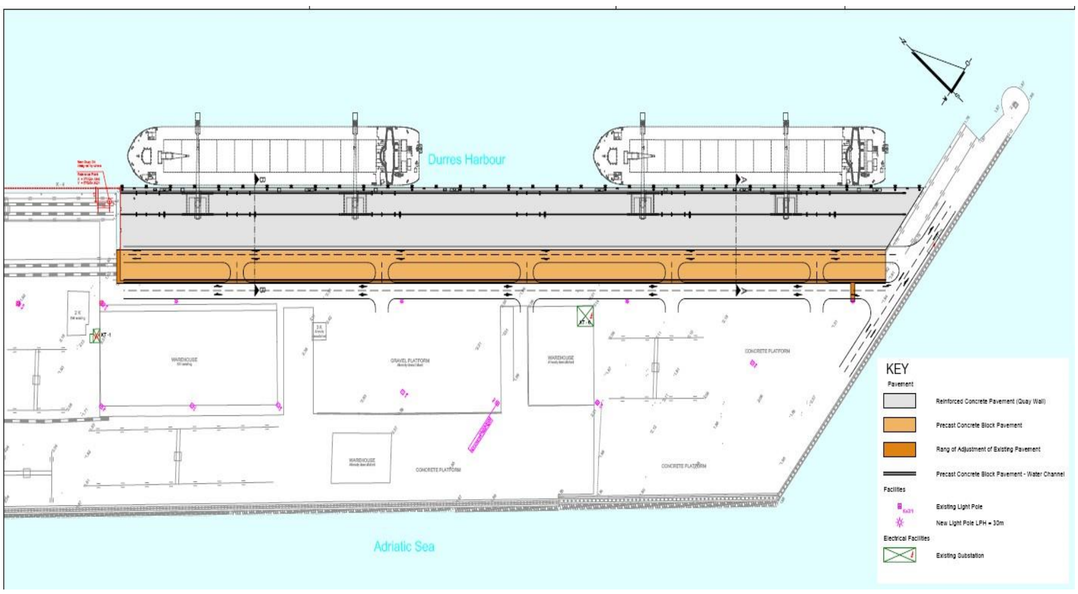


Figure 21 - Layout as proposed by "Conceptual Design, Final Design and Bill of Quantity for Reconstruction of Quays 1&2".

The proposed works, according to "Preparation of Conceptual Design, Final Design and Bill of Quantity for Reconstruction of Quays 1&2" include the following:

- Expansion of the Quays in order to bring the waterfront line to the same line as that of the redesigned Quay 4 (eliminating the turn at Quay 3)
- Provision of a bearing capacity of the quays' aprons of 4 tons/m² for the new berths
- Achieving optimum conditions for the 3-phase power supply to the operating ships
- Protection of the waterfront side of the berths using fenders with energy absorption and reaction force for the ships approach up to 30.000dwt
- Safeguarding the cranes on the quay during different storms, safeguarding the ships and other operational equipment against fire.

6.2 Simplified Cost-Benefit Analysis

As suits a pre-feasibility study, a simplified CBA was conducted. It examines all options defined above and compares them to the do-nothing case. The exercise followed EU and EC/MOVE guidelines. In particular, projections were developed over a 30-year horizont. The results of the financial analysis and of the economic analysis are as follows.

		option		
		B	**C1**	**C2**
Financial Net Present Value of the investment	FNP(C)	**-0,6**	**-18,6**	**-13,0**
Financial Internal Rate of Return of investment	FRR(C)	**3,9%**	**1,2%**	**1,7%**
Financial Net Present Value of capital	FNP(K)	**2,1**	**-16,5**	**-9,1**
Financial Internal Rate of Return of capital	FRR(K)	**4,8%**	**0,1%**	**1,2%**
Benefit /Cost ratio of the capital (B/C k)	(B/C k)	**0,3**	**0,2**	**0,3**

Table 8 - Financial rate of options B & C.

Note: NPVs at 4% discount rate

Table 9 - The simplified summary of key economic indicators analyse.

		option		
		B	**C1**	**C2**
Socioeconomic Net Present Value in mEUR	ENPV	**43,9**	**28,4**	**33,6**
Economic Internal Rate of Return	ERR	**13,0%**	**8,8%**	**10,5%**
Benefit / Cost ratio	B/C	**1,9**	**1,4**	**1,5**

No

te: Social Discount Rate (SDR) at 5%

All options generate acceptable economic returns over the period. ERRs range from high single digit to low double digit, comfortably above the SDR, resulting in positive ENPVs. From a socio-economic point of view, option B is the most favorable, followed by options C2 and then C1, as with the results of the financial analysis as there are only small variations in externalities between options.

For a better understanding a further more analyse was done, the multi criteria analysis. This to evaluate all options and select a preferred option for further analysis in the next phase of the assignment. With the beneficiary criteria to be considered and their respective weightings. Criteria categories used include: Technical feasibility and constructability; Economic feasibility; Regional impact; Transport demand and forecasts; Environmental and social impacts and opportunities; and Physical risks associated with the current infrastructure condition. Investment costs are not evaluated independently but under both Economic feasibility and Transport demand and forecasts.

The table below summarizes the outcome of the multi-criteria analysis. Option C1 ranks marginally ahead of option B while options C2 and A rank significantly lower.

	option			
	A	**B**	**C1**	**C2**
Gross weighted score	2.55	3.24	3.34	2.85
Normalized score	0.76	0.97	1.00	0.85

Table 10 - Thesummarizes outcome of the multi-criteria analysis. Option C1 ranks marginally ahead of option B while options C2 and A rank significantly lower.

C1 overcomes its inferior financial and economic performance compared to B (investment costs 70% higher for similar financial and economic benefits) thanks to, inter alia, its ability to handle larger vessels and a greater comfort of operation resulting in a somewhat better regional competitive position as well as a faster implementation, due to existing detailed designs. This results in a higher indebtedness under option C1 and higher national budget support requirements to assist with loan amortization as operations, are insufficient to cover loan.

6.3 Seismicity

It is interesting to underline how the strategies to ensure the robustness of a particular construction with regard to an exceptional action and the design criteria of the same construction with regard to the seismic action have in some cases compatible objectives, in other cases instead antithetical. In fact, if an increase in the resistance and displacement capacity of the vertical resisting elements constitutes an advantage in both situations, the design strategies can be very different in the case of horizontal planes, especially if of significant extension. If the classic design criteria for seismic actions require a connection at the level of horizontals, which must ensure adequate stiffness and strength to allow the redistribution of the actions on the vertical load-bearing elements, in the case of exeptional events. For example, the coverage in areas not structurally connected to each other, may be the best strategy to adopt, if not the only one, to prevent localized damage from involving the entire structure. For different reasons, it is also useful to remember that structures whose shape has been optimized with respect to specific types of project actions, may present robustness problems in the case of exceptional actions not considered in the design phase, as such structures may not be able to admit the establishment of alternative routes of loads designed to avoid collapse. The theme of design, at least for significant constructions, should therefore always be set on a multi-risk approach, in which to consider all the potential critical situations, exceptional or otherwise, that they may encounter during their useful life.

According to the carried out seismic assessment (REPORT ON GEOLOGICAL AND GEOTECHNICAL CONDITOINS AT QUAY 1 & 2 IN DURRES PORT by A.L.T.E.A &GEOSTUDIO 2000 Sh.p.k), the PGA value for a return period of 475 years and with probability of exceedance 10% in 50 years, multiplied with the subsoil factor (S), gives a value of axS=0.381g.

This seismic report summarises the evaluation of seismic hazard potential for the project and estimates, among others, the peak ground acceleration (PGA) for a return period TNCR = 475 years and with a probability of exceedance 10 % in 50 years. Furthermore, two estimations are depicted, based on the available data.

Table 11 - Assessment of environmental and social constraints.

No	*Source*	*PGA Value*
1	*New Seismic Hazard Assessment*	*0.24 - 0.30g*
2	*Report for Quay 1 & 2*	*0.381g*

In the current design, the second estimation is used which is considered as a more conservative approach.

6.4 Design Vessel

For the future operations, according to the master plan, which was used as reference for the present design, the port, is developed for the maximum possible vessel with a capacity of about **30,000 DWT for dry bulk** or equivalent for General Cargo, based on the configuration of the basic infrastructure of the port.

These are the maximum vessels which can be accommodated in the existing Harbour basin, in accordance with international safety standards, and corresponding to a reasonable investment. Based on aforementioned information, the design vessels for the new Quays 1 and 2 are given in the Table 12.

Table 12 - Design for large vessels

Parameter	Unit	Bulk Carrier	General Cargo
Overall length	m	176	174
Length between perps	m	167	164
Beam	m	26.1	25.5
Max. draught	m	10.3	10.5
Carrying capacity	dwt	30000	26923
Displacement	t	36700	35185
Freeboard	m	3.95	4.04

CHAPTER 7

THE PROJECT

The proposed works, according to **“Preparation of Conceptual Design, Final Design and Bill of Quantity for Reconstruction of Quays 1&2”** include the following:

- Expansion of the Quays in order to bring the waterfront line to the same line as that of the redesigned Quay 4 (eliminating the turn at Quay 3)
- Provision of a bearing capacity of the quays’ aprons of 4 tons/m² for the new berths
- Achieving optimum conditions for the 3-phase power supply to the operating ships
- Protection of the waterfront side of the berths using fenders with energy absorption and reaction force for the ships approach up to 30.000dwt

Safeguarding the cranes on the quay during different storms, safeguarding the ships and other operational equipment against fire.

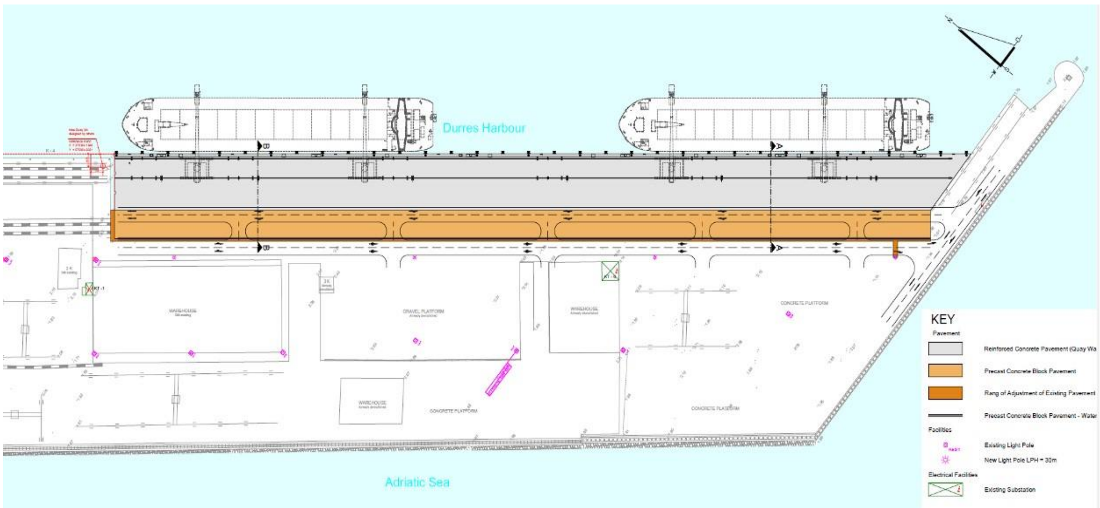


Figure 22 - Proposed general layout.

Several alternatives were examined in the context of **“Preparation of Conceptual Design, Final Design and Bill of Quantity for Reconstruction of Quays 1&2”**,

regarding the method of supporting the deck and the formation of the front wall to achieve the proposed **depth of -11,50 m**. The alternatives were either front sheet piles with grouted anchor combined with structural fill, or completely open deck construction with vertical piles and formation of the sea bed with dredging, or with vertical piles and rear sheet piles with grouted anchor and protection of the sea bed slope with quarry run and geotextile filter, or front sheet piles combined with vertical piles and rear sheet piles with grouted anchor and structural filling in between.

Finally, the selected alternative adopted in the final design stage was a combination between the above which consisted of a deck supported on vertical piles with rear sheet piles as retaining wall without grouted anchor and the formation of the sea bed was proposed with structural fill protected with geotextile filter and quarry run. The new quay shall consist of a jointless concrete deck with a cast in-situ beam girder, transversal and longitudinal beams and precast slabs with concrete topping on concrete piles with a diameter of 1,5 m. The offset between harbour depth at CD-11,5 m and terminal area at CD+2,0 m is formed by a slope with an inclination of 1 in 3 covered with revetment beneath the deck and a steel sheet pile wall at the landside edge of the deck.

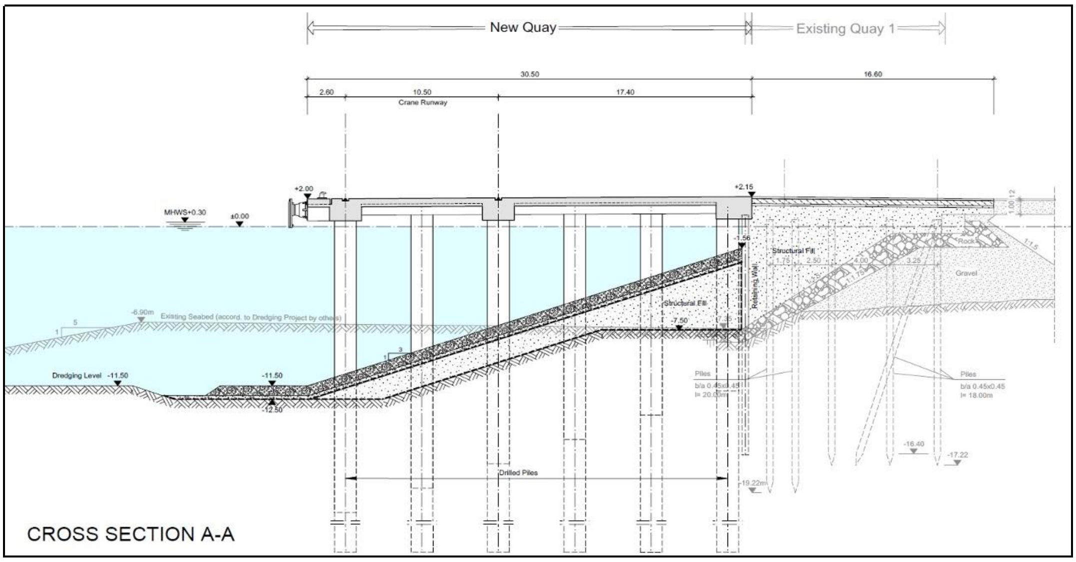


Figure 23 - New quay nr.1 detailed design.

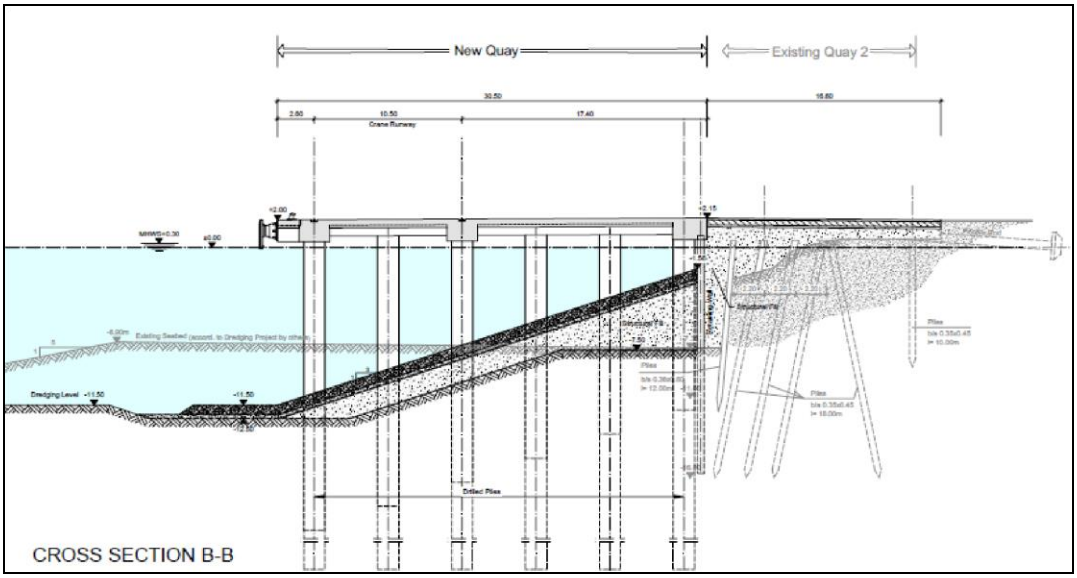


Figure 24 - New quay nr.2 detailed design

The proposed options are described as follows:

<u>Option A.</u> Maintain the existing quay wall as it is (“Do Nothing” or Business-as – Usual scenario).

In this first Option, no works are foreseen to expand the existing infrastructure, whereas only regular maintenance of the infrastructure and equipment takes place.

<u>Option B.</u> Maintain the existing quay wall line, rehabilitate the existing wall and deepend the waterfront.

This second Option is proposed by the Consultant and consists in a rehabilitation of the existing quay wall, construction of two arrays of concrete piles supporting two concrete beams to serve as foundations for the crane rails, demolition of the existing deck and construction of a new concrete deck and apron, and deepening of the sea level. Under this option, the existing quay wall line is not changed.

The quaywall is deepened to the desired nominal depth (-11,5m) by constructing a continuous secant pilled wall with intersected piles It is noted here that the proposed quay wall line of option A and B.

Option C. The selected option according to the 2012 Detailed Design: Construction of a retaining wall at the existing quay, arrays of drilled piles and a revetment structure (slope and scour protection).

In this third Option the current project explores the possibility of constructing the envisaged expansion works in two distinct phases, thus the Consultant has devised two variants. The purpose of this investigation is to elucidate the relation between forecasted traffic demand and the required port infrastructure capacity, in order to follow a feasible and sustainable path for expanding the port. Additionally, the construction methodology, materials for the back fill etc. for the selected cross-section will be sought to be optimized, although this will not be considered as a distinct option. Therefore, the following phases will be considered:

Phase 1: Construction of New Quay 2 (approximately 250 m)

Phase 2: Construction of New Quay 1 (approximately 250 m)

Hence, two options are considered:

Option C1: Construction of the expansion works for Quays 1 & 2 in **one phase**

Option C2: Construction of the expansion works for Quays 1 & 2 in **two phases,** staging the work with Quay 2 upgraded first and Quay 1 following 6 years later.

The above **four (4) options (A, B, C1, and C2)** were compared, on the one hand, from an environmental and social point of view and on the other hand, they were assessed through a simplified cost-benefit analysis, and a Multi Criteria Analysis, in order to determine the preferred option.

7.1 Description of Proposed Port Infrastructure Interventions

The existing quay wall will continue to accommodate vessels as it is. No works are foreseen to rehabilitate or expand the current port infrastructure of Quays 1 & 2. It is noted here, that if this option is to be selected, a thorough inspection, underwater and overwater, will be undertaken in order to record in detail all existing damages (scouring, spalls, cracks etc.) and a rehabilitation plan will be carried out at a later stage of this Project.

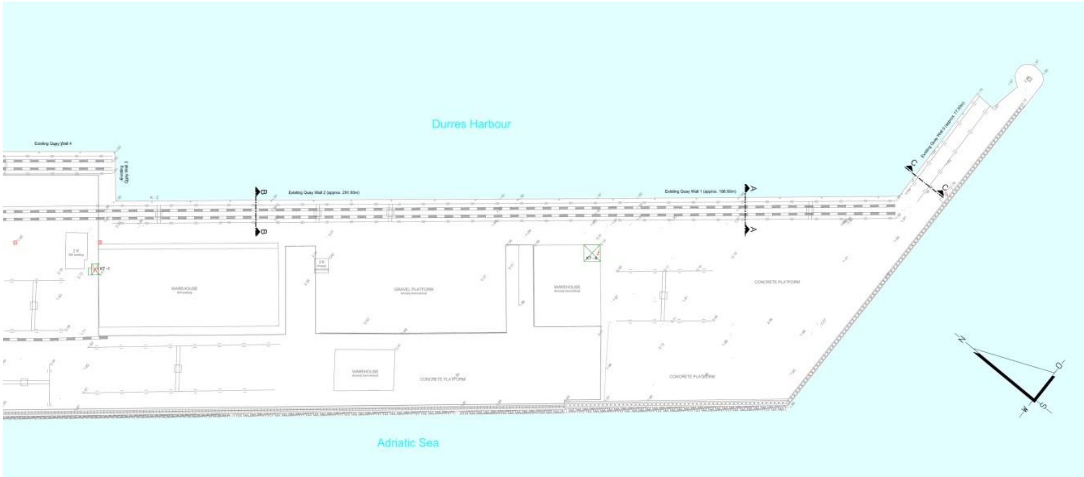


Figure 25 - Layout of existing Quays 1 & 2 (Source: 2012 Detailed Design).

Quay 1 or Berrth 1 is located at the port side upon entrance of the vessel into the port.

Starting at the corner (with berth 0), the quay is used by the Albanian customs and police. The next section of the quay is used by the navy and the remainder is available for cargo handling. The cargo handled mainly comprises import of dry bulk such as coal and cokes. The quay was constructed in 1972 and in 1994 major overhaul of the yards behind the berths 1-4 and 7 and 8 was carried out. During these works the level of the deck has been increased by adding a layer of concrete on top of the, by then, existing deck. The Consultant’s Team, which executed the Master Plan, expressed their concerns regarding the quay level rising. In particular, they mentioned the following:

“We express here concerns which apply to all such quay level raising programmes. The raising of berths and apron areas increase the loads applied to the structures due to the increased deadweight of the structure. This could affect the bearing capacity of the berths and back up areas. This is because the extra weight will reduce the ability of those structures to carry live loads due to cargo handling and cargo storage. We assume that these factors have been taken into account and that the PDA has been advised of any working restrictions that should be applied.”

The structure of the quay is a concrete deck on reinforced piles. Key characteristics of the quay are presented below:

Length: 178,5 m

De

sign load: 4 tons/m²

Design depth: 7,35 m Dredged depth: 7,35 m

Quay 1 is an asset of General Cargo Terminal / West Terminal and Military area.

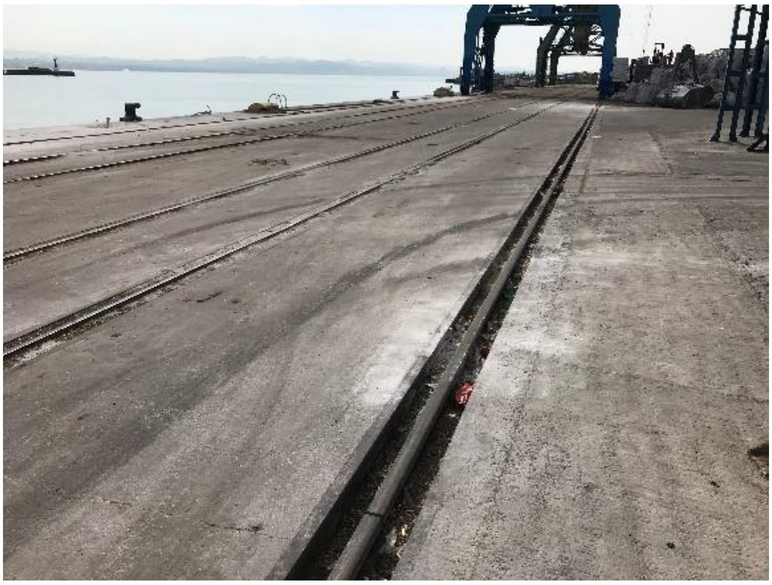


Figure 26 - Quay top deck view

In general, the top of the deck appears to be in serviceable condition despite some cracks. The water front face of the coping beam however is damaged at many places and corroded rebars are exposed. The rear of the deck was reported to have settled and the rear crane rail has been raised several centimetres to provide a level track.

Since its construction, the structure of the quay is not maintained beneath the slabs and beams so the degree of corrosion of the steel bar is very high. A fully protective layer of concrete does not exist. An underwater investigation is necessary.

Backyard behind the quay consists of concrete slabs which are recently reconstructed and are in good condition. However, minor cracks have begun to appear, and immediate maintenance works are needed.

Also, the electrical duct behind the quay is broken at several places. The concrete covers were replaced in 2011. Potable water network is already installed in this duct.

There are no fenders in the water front beam. For fendering, used truck tires are hung over bollards. Durres Port Authority has replaced the old 30 tons bollard, and, in their position, the new 75 tons bollards are recently installed at a distance of approximately 30 m.

There are three Ganz cranes on Quays no.1 with a capacity of 5 tons each. They are very old but still in working condition.

Figure 27 - Crane damaged power plug, on apron of quay 2

In terms of utilities the quay has a cable duct with a power cable. There are connection points with sockets for the electrical cranes and at the police berth a permanent switchboard has been installed. Manholes and covers are in bad condition and some of them are already damaged.

There are two rail tracks between the crane rails; however, these are not connected to the railway network outside of the port. The lines end near the car exit of the West Terminal, adjacent to the Gate No.2 of the Durres Port.

Quay 2 or Berth 2 is located north of berth 1 in line with berth 1. The quay is used for general cargo handling and for grain, the latter being directly loaded onto trucks.

The quay was constructed in the period 1949-1951 and fell under the 1994 reconstruction works funded by the Kuwait Fund. Part of the quay structure is a suspended deck on piles. The deck and piles both consist of reinforced concrete. The other part of the quay, near Berth 3, consists of a concrete sheet pile retaining wall combined with a deck on pile structure. Key characteristics of the quay are presented below:

Length: 292,2 m
Design load: 4 tons/m²

Design depth: 7,35 m Dredged depth: 7,35 m

Quay No.2 is an asset of General Cargo Terminal / West Terminal.

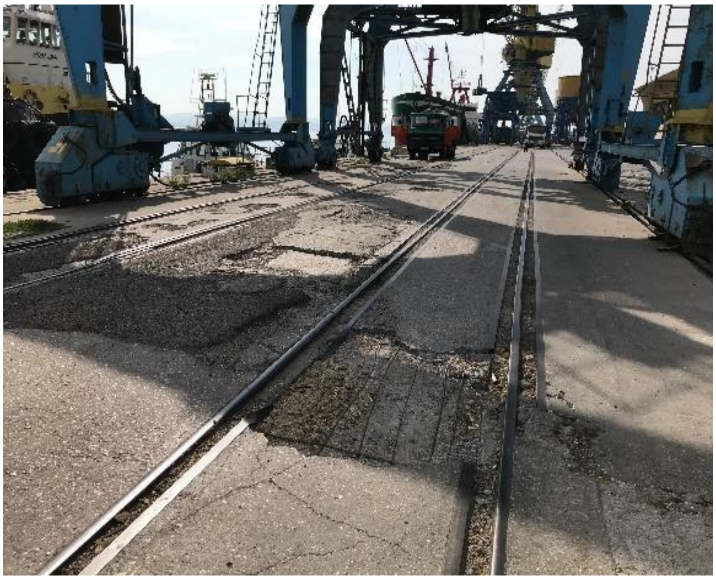


Figure 28 - Quay nr.2 top view

The top of the deck seems in very bad condition, damaged areas and heavy cracks can be observed everywhere. Several rebars are no longer covered by concrete.

The front of the coping beam on top of the sheet piles shows severe signs of disintegration. From approximately 0.3 m below the deck level to the underside of the coping beam concrete coverage of the rebars has disappeared.

Despite being the oldest quay in the port, the sheet piles appear to be in remarkably good condition however it should be mentioned that only a short stretch below the coping beam is visible from the surface. These piles appear to have been well connected to the coping beam and there are only a few signs of

corrosion which, at present, do not appear to be serious.

At the beginning of quay no.2 (starting from the quay no.3) some piles must be damaged as it has been found to be a loss of filling material. An underwater investigation is necessary, especially in this section.

Figure 29 - Quay nr.2 pavement failure

The pavement behind the quay consists of concrete slab, which is damaged between the crane rails and the warehouse 4&5. This damage is likely caused by hard landings of the grabs used to unload vessels and lack of maintenance.

Also, the electrical duct behind the quay is broken at several places. The concrete covers are partly replaced in 2011. Potable water network is already installed in this duct.

There are no fenders in the water front beam. For fendering, used truck tires are hung over bollards. Durres Port Authority has replaced the old 30 tons bollard, and, in their position, the new 75 tons bollards are recently installed at a distance of approximately 30 m.

In terms of utilities the quay has a cable duct with a power cable. There are connection points with sockets for the electrical cranes and at the police berth a permanent switchboard has been installed. Manholes and covers are in bad conditions and some of them are already damaged.

There are two rail tracks between the crane rails; however, these are not connected to the railway network outside of the port. The lines end near the car exit of the West Terminal, adjacent to the Gate No.2 of the Durres Port.

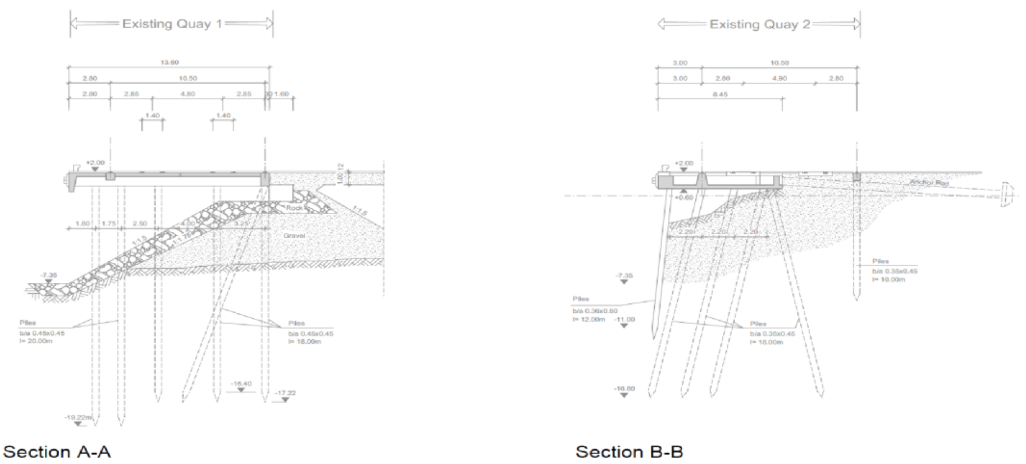


Figure 30 - Quay 1 nad quay 2 existing structure layout

7.2 General Layout and typical cross-section

The reconstruction of the existing quay wall of Quays 1 & 2 will be carried out with the builtup of an extension, so their berthing line is brought to the same straight line of the redesigned Quay 4. The extension will have a total length of the berthing line of 501m.

However, due to transition slopes with the neighbouring seabed levels, the net quay wall length with a nominal depth of at least -11.5m (LAT) is 458.5m.

The width of the extension will be 30.50m, and is constructed as a deck supported on vertical piles. The new quay shall consist of a joint-divided concrete deck, with a cast in-situ beam mesh girder in transversal and longitudinal direction and slabs with concrete topping on concrete piles with a diameter of 1,5 m.

The apron of the quay wall will be equipped with the appropriate fenders and bollards for the complete range of the design vessels foreseen to berth on the quay wall and with rails for cranes of 10.50m gauge.

The transition zone with the existing quaywall has a length of approximately 18.6m will be constructed by following: a) removal of the existing deck (pavements & piles) b) construction of retaining sheet pile wall with one row of anchorage applied at +0.60 MSL and backfill with coarse grained material.

Construction of heavy-duty pavements, utilities, drainage system on the additional yard area, required for the full operation of the berth and the storage yard area.

The offset between harbour depth at -11,5m (LAT) and terminal area at +2,25m (LAT) is formed by an armoured slope with an inclination of 1 in 3 covered with revetment beneath the deck, for protection against erosion.

Dredging operations will cover an area of 50.0m width from the new quaywall, in order a berthing zone of -11,5m (LAT) to be constructed. Beyond that zone, a wider area of approximately 75.0m width, will be dredged with depth of -10.25m, to create an area, needed for the navigation channel and manoeuvring area, so the safe approach and berthing of the maximum design vessel will be secured.

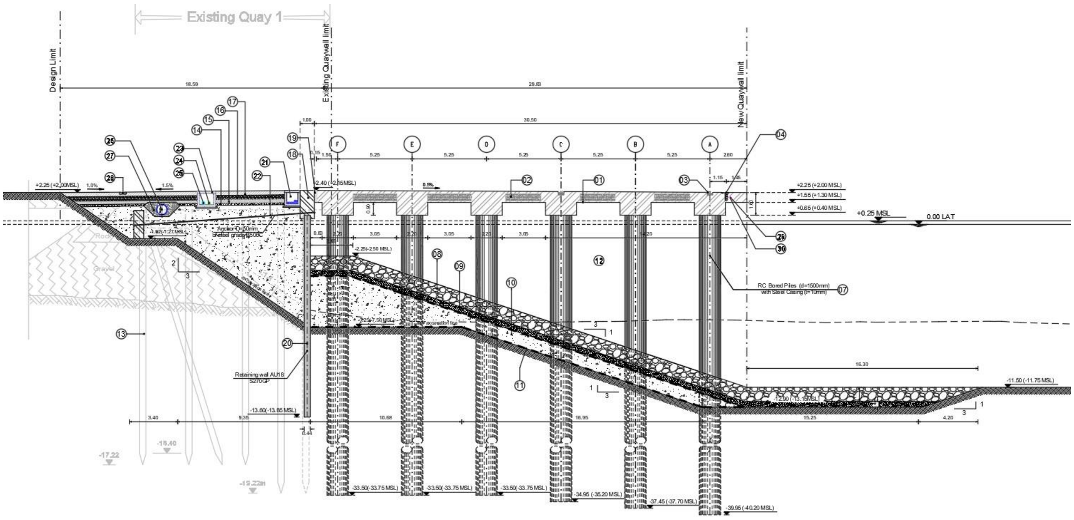


Figure 31 - Typical cross section of quaywall 1

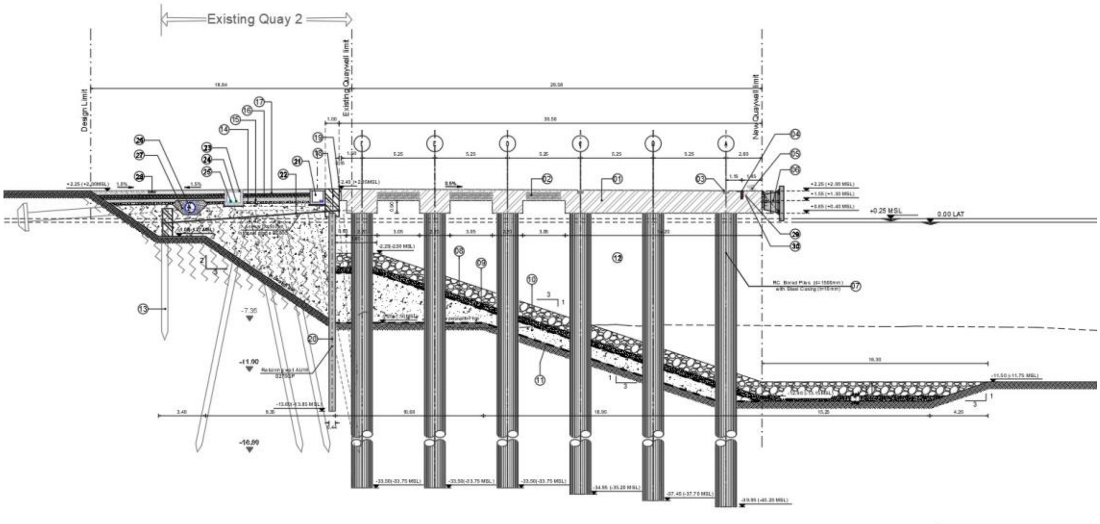


Figure 32 - Typical cross section of quaywall 2

7.3 Loads

The first design difficulty is to study the distribution of vertical and horizontal actions between the different structural components. Furthermore, it is necessary to verify that the deformation framework is compatible with the operating requirements of the neighboring structures and the quayside activities. Numerical modeling is the only calculation tool currently available capable of adequately quantifying the distribution of loads between the structural components and the response in terms of system displacements, both in the short and long term. It is possible to estimate the level of work of the individual components, and therefore the mobilization of the system's resistance reserves, during the various construction phases.

The numerical analysis also constitutes a tool for assessing the adequacy of any variants and alternative design solutions for the optimization of the project, which, considering the longitudinal development of these works, which is usually large, implies significant economic repercussions on the total construction costs.

7.3.2 Dead Loads

Gantry crane loads

7.3.3 Live Loads

Imposed Loads from bulk materials on deck

The imposed loads due to bulk material on deck are described below:

- 40kN/m² uniform distributed life load between crane rails that corresponds to normal bulk cargos and conventional general cargos and containers.
- 60kN/m² uniform distributed life load at backside area of new quay.

Fender Loads

The imposed loads due to fender actions are presented herein:

- E_{CV} = 754,00 kNm (constant velocity berthing energy)
- R_{CV} = 1.248,40kN (reaction force created by the fender onto the structure)

Mooring Loads

The imposed loads due to bulk material on deck are described below:

- 800 kN × 1.2 = 960 kN per mooring position.

7.3.4 Seismic Loads

Using EN 1998 and the Project specifications:

- Peak ground acceleration (for the specific project): 0.381g.
- Ground type: not applicable, since the ground effect is included in the peak ground acceleration.
- Importance factor: included in the peak ground acceleration.

Behavior factor: q=2.0 (see Introduction for substantiation).

7.4 Anchor System Dimensioning

The verification of an anchor should ensure that the design value, Ra; d, of the resistance force Ra should satisfy the following limit condition:

$$Ed \leq Rd$$

Where Ed the design value of the anchor system due to external loads. The design value of the resistance anchor value should be the less of the passive resistance caused by the “dead-man” type anchorage (Ra; d) and the tensile strength of the steel tendon (Ri;d).

The value of the passive resistance of the “dead-man” type (Ra; d) has been calculated based on the shear strength of the backfill material (assumed φ’=35°), the depth and the geometry of the rear beam as well as the height of the retaining wall.

Firstly, the anchoring system should be effective, when the potential active failure zone is located outside the developed passive zone. In figure 3-4 below a graphical verification of this check, has been drafted.

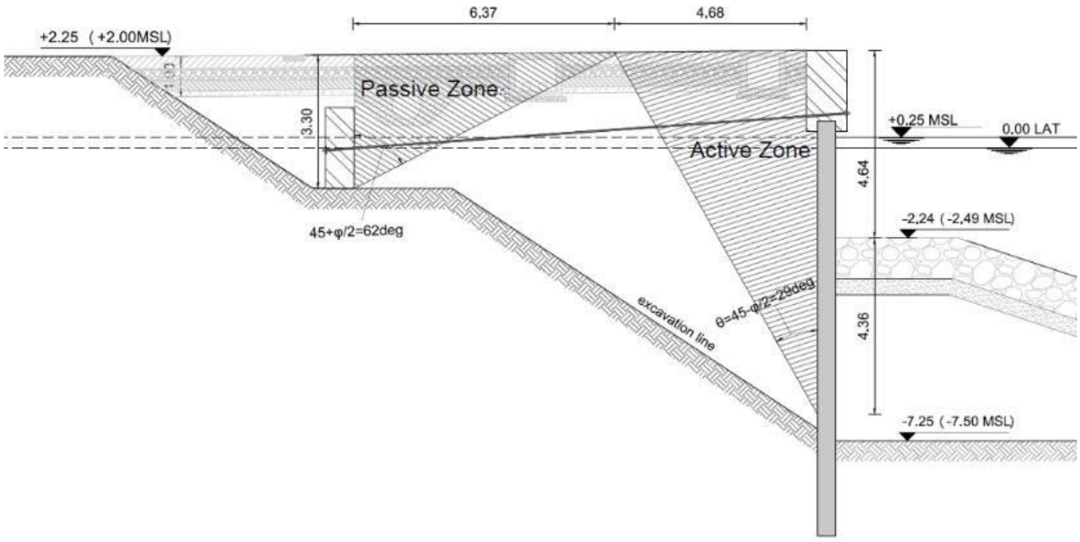


Figure 33 - Geometry of the "Deadman" type anchoring

Secondly, the resistance which is calculated from the rear beam passive resistance or the tie bar tensile strength, should be higher than the loads due to external actions. The passive resistance of the rear beam is calculated for a depth of -3.3m from the top surface layer. Any loads on the pavements which are favorable actions are neglected.

$$R_{a;d} = R_{a;k} / \gamma_a$$

$$R_{a;k} = 346.5 kN/m$$

And for $\gamma_a = 1.4$ the design passive resistance force is equal to:

$$R_{a;d} = 247.5 kN/m$$

The design tensile resistance of anchor bar ($R_{t;d}$) is calculated through the following equation:

Table 13 - Design tensile resistance

Maximum tensile force	**Spacing of anchors**	**Tensile force per anchor**	**Anchorage characteristics**	**Design tensile resistance of tie bar***	**Design passive resistance of rear beam**
(kN/m)	(m)	(kN)	(-)	(kN/m)	(kN/m)
137.35	4.0	549.40	Nom. Diameter 50mm.Nom. cross-section 1963mm^2. Steel grade of 500C F_d = 853kN per anchor	**213.25**	**247.5**

*no corrosion allowance has been considered.

7.5 Settlements Assessment

The loading of the area around the existing quay walls, through the construction of permanent (backfilling of the sheet pile wall & revetment) and temporary fills (for the construction of the sheet pile wall), will induced settlements. It is estimated that the maximum applied load around the location of the new sheet pile wall is 77kPa.

Since the stratigraphy in Areas A & B is multi-layered, the settlements induced by the granular soil will be elastic and will be developed in short terms, whereas the settlements induced by the cohesive layers will be due to consolidation and will take place in long terms.

The calculations are carried out for Areas A and B. Firstly the heigh of the temporary fill is considered and afterwards when the construction of the sheet pile will be finished, a portion of this fill will be removed. In terms of soil mechanics, the first stage loading will be in “virgin” state and after the removal any new load will be in “reloading” state.

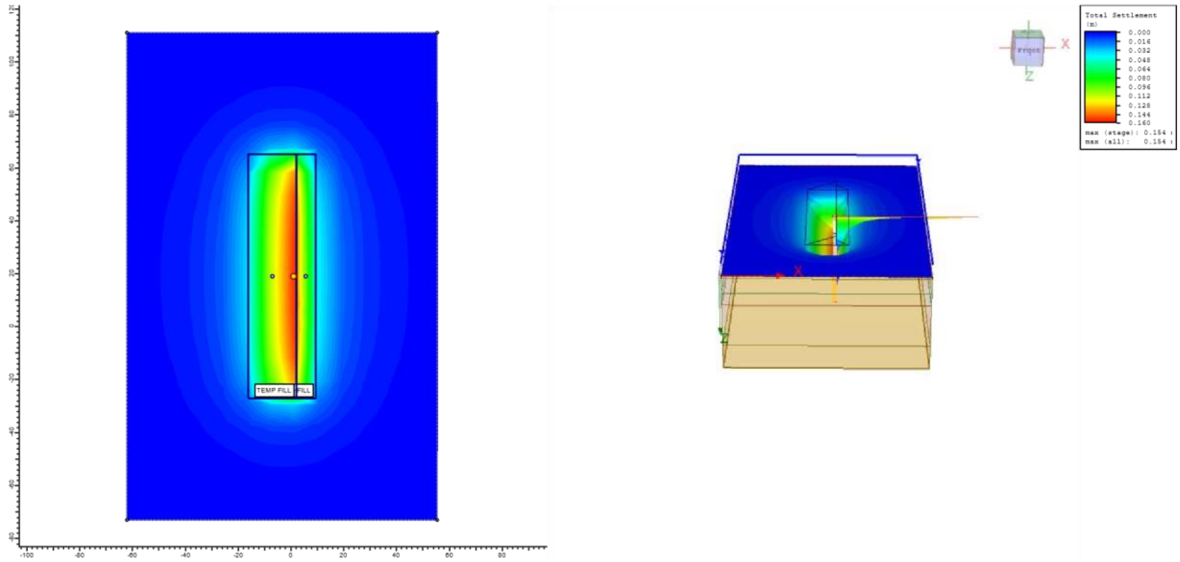


Figure 34 - Settlement calculation graphical results – Area A

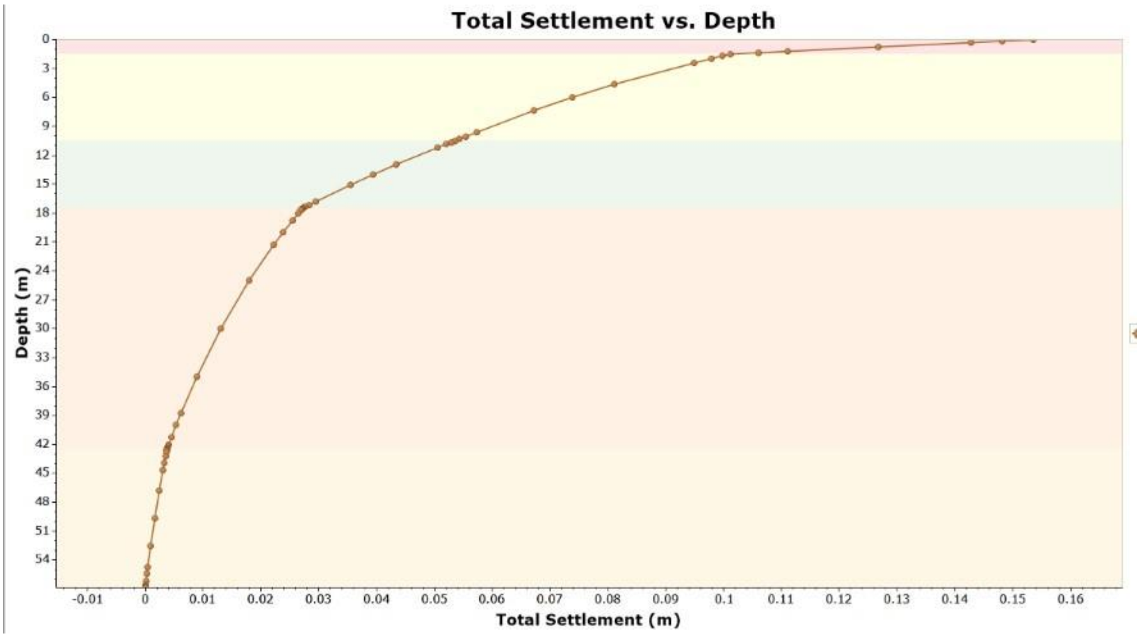


Figure 35 - Vertical profile at the location with the maximum settlement – Area A.

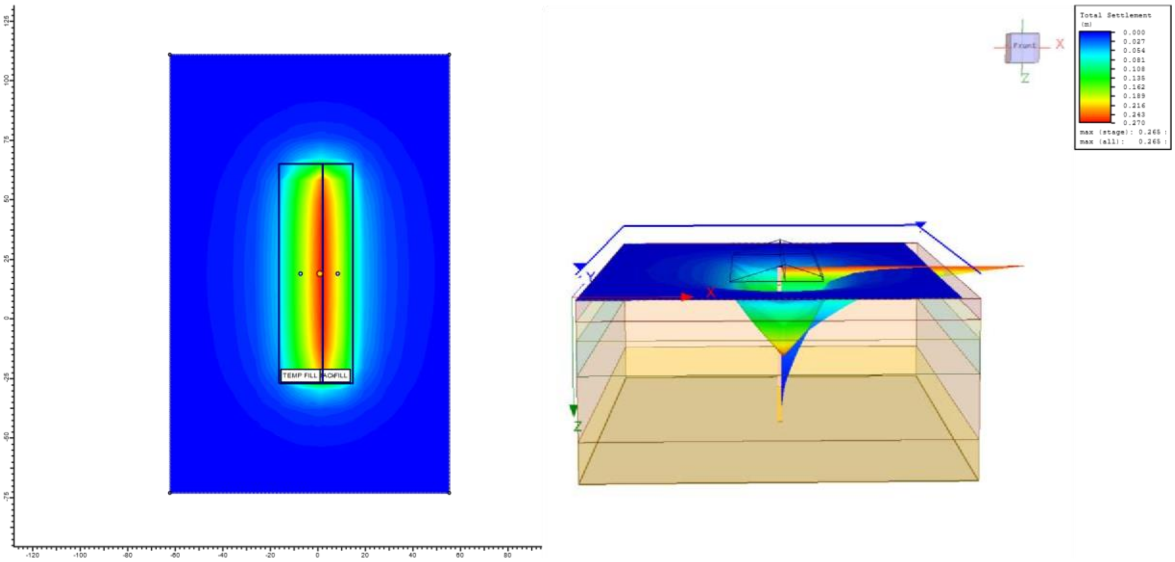


Figure 36 - Settlement calculation graphical results – Area B

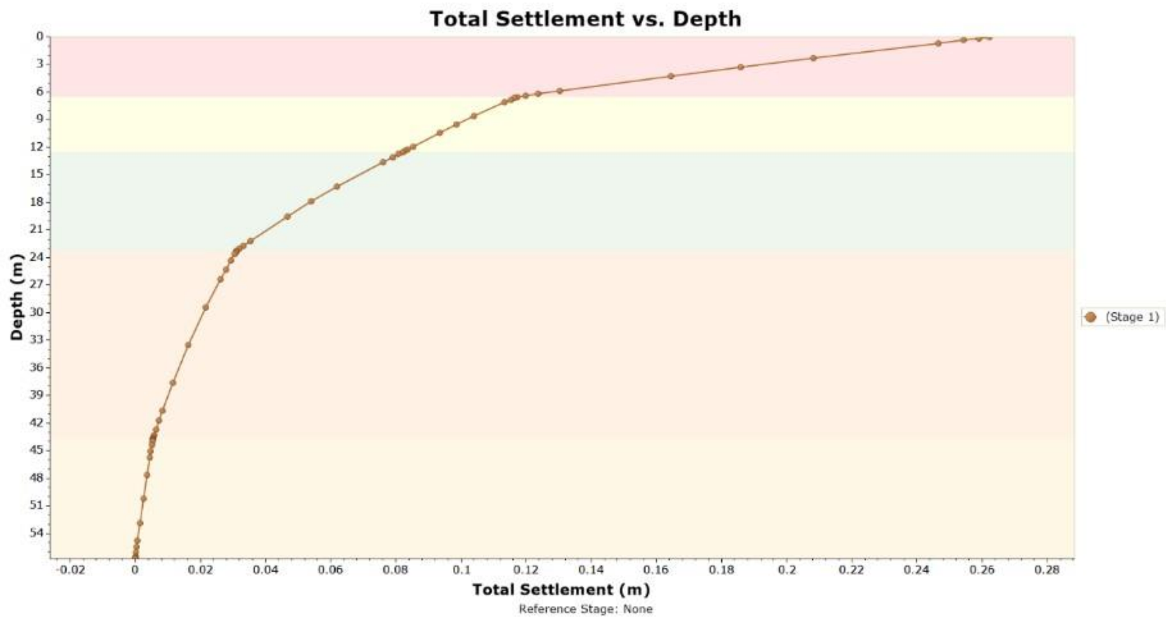


Figure 37 - Vertical profile at the location with the maximum settlement -Area B.

The above methods determine the total settlements through the Eeod elasticity modulus. With this method the one-dimensional settlement is calculated, since the parameters are determined from the results of the oedometer test. These settlements will commence on application of the loading but a proportion of them can be expected to occur after construction and this may affect sensitive parts of the structure.

For this purpose, an analysis of the time evolution of the settlements, has been carried out. The results, which are influenced by many uncertainties (drainage boundaries, uniformity of clayey layers etc.). With this estimation, it is considered feasible, that within 6 months after the installation of the temporary fill, the 70% of the total settlements will have occurred.

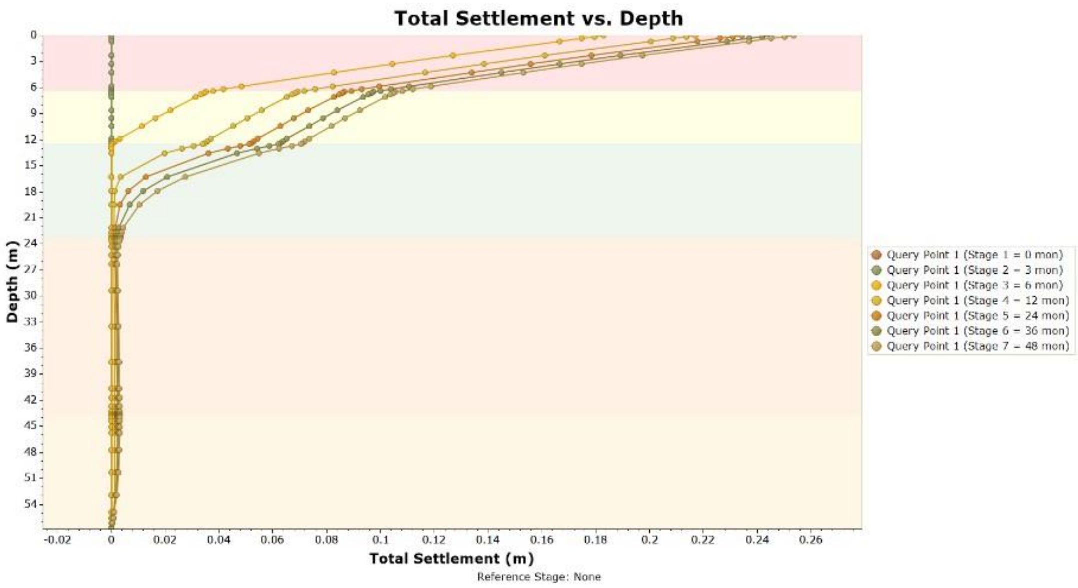


Figure 38 - Settlements evolution through time – Area B.

Table 14 - Estimation of the chronical evolution of the settlements

	Total settlement (m)	**Settlements after 6 months / remaining (%) (m)**	**Settlements after 12 months / remaining (m)**
Area A	0.154	0.113 / 0.04 (26%)	0.137 / 0.017 (11%)
Area B	0.245	0.180 / 0.065 (26%)	0.217 / 0.028 (11%)

From the results, it is estimated, that a large percentage of the total settlements (~70%) will be completed after 6 months of the temporary fill construction to the level of +0.40m. The remaining settlements will be completed by the end of construction (e.g.

3 years). In any case a monitoring system should be installed during construction, to record, in monthly basis, the settlements and hence have a more precise estimation about the settlements evolution.

7.6 Liquefaction Assessment

7.6.2 Empirical solutions

In the following figure the Seed et al method1 has been carried out from the available data (liquid limit, plasticity index and water content) of the boreholes BH101 to BH107. The implementation of this method allows the estimation layers that consist of fine-grained materials (fines >50%) and my loose part of their strength due to cyclic loading. In general, materials prone to liquefaction are considered the non-cohesive ones and thus the clayey bedrock were excluded from this assessment.

Most of the examined samples are selected from the upper layers zone, which consist mainly of silt & sand mixtures with varying proportions of clay and gravels. It is depicted that a small proportion of the samples is located withing the “blue-very likely to liquefy” area and the most are within the “red area” where the possibilities are conditional (see Figure 39).

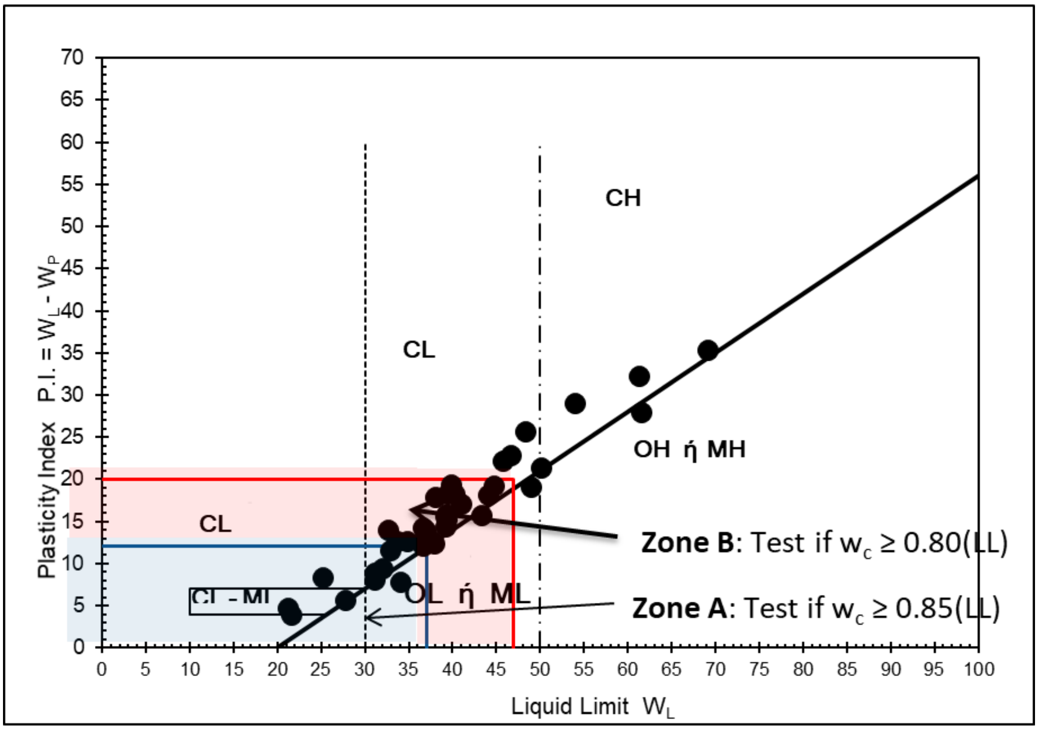


Figure 39 - Seed et al method Criterion 1

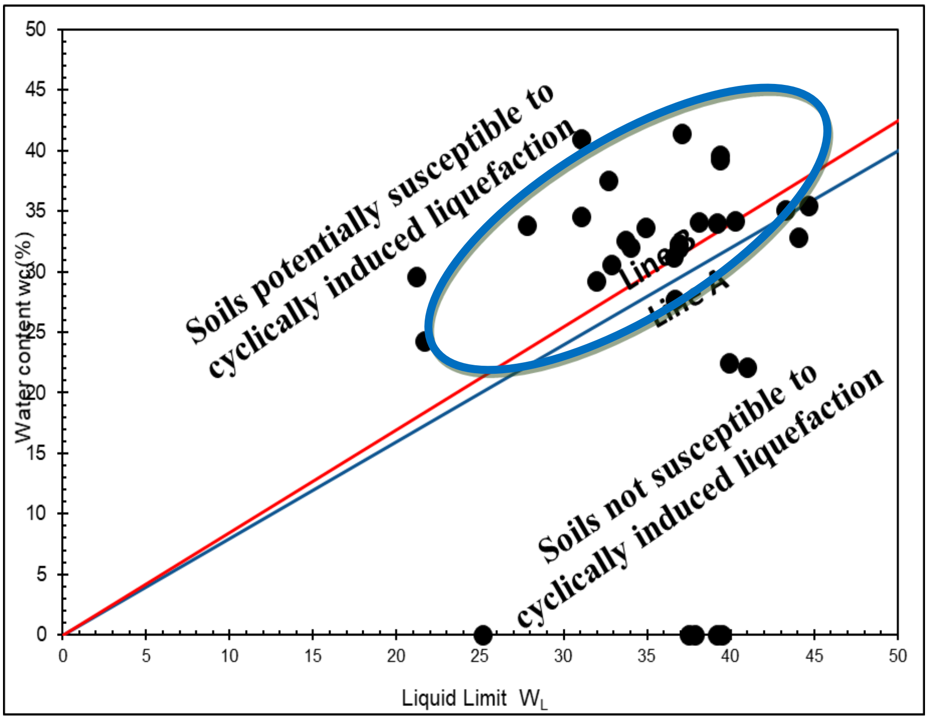


Figure 40 - Seed et al method Criterion 2. Soils potentially susceptible to cyclically induced liquefaction are marked with a blue lined circle.

7.6.3 Analytical solutions

The following assumptions have been made based on the available data (see Ref. 5). More specific the following assumptions have been applied:

- The peak seismic acceleration at the ground surface is taken as 0.38g.
- A maximum expected earthquake of Mw=6.4 is considered.
- The safety factor of liquefaction potential based on calculations should be over unity.
- The layers examined consist of coarse-grained materials with fine content less than 50%.

In the Figure 53 and Figure 54, the results of the liquefaction per borehole are depicted. The first column shows the calculation of the coefficients CSR and CRR, the second column the fluctuation of the F.S. regarding the depth and the third the probability of liquefaction is depicted.

From the borehole findings and their evaluation, it can be concluded that the upper sandysilty layer, which is found in every borehole, do not have uniform properties in terms of density and composition. Therefore, its behavior, during a severe earthquake scenario regarding liquefaction, differs and most probably liquefaction will occur locally.

For design purposes, a very conservative assumption is made, that the upper 8m along the Areas A & B will be subjected to liquefaction. These means, that the contribution of the R.C. piles at that certain layers will not be accounted to the total bearing capacity in the earthquake scenario.

7.7 Bearing Capacity of a pile group

The bearing capacity of a group of piles is not critical, and it will not be calculated due to the fact that the piles' spacing is over 3 times their diameter (Burland).

7.8 Overall Stability Assessment

The overall stability checks will be carried out for the temporary slip fill and for the retaining structure. The overall check including the deck on pile structure and the revetment, it is omitted. The reason is that due to the deep length of the piles (L>30m) it is considered that the check is not critical.

Slope stability analyses were carried out for the temporary fill which will be constructed for the sheet pile construction with material of properties similar to the backfill material. The slope of the temporary fill will be of 2:1 (hor; vert).

Furthermore, overall stability analyses were made for the permanent conditions (operational & seismic) of the sheet pile wall for the Areas A & B. Detailed information on the software input and output data can be found at Appendix D.

In the Figure 58 to Figure 60, the results of the stability checks are depicted.

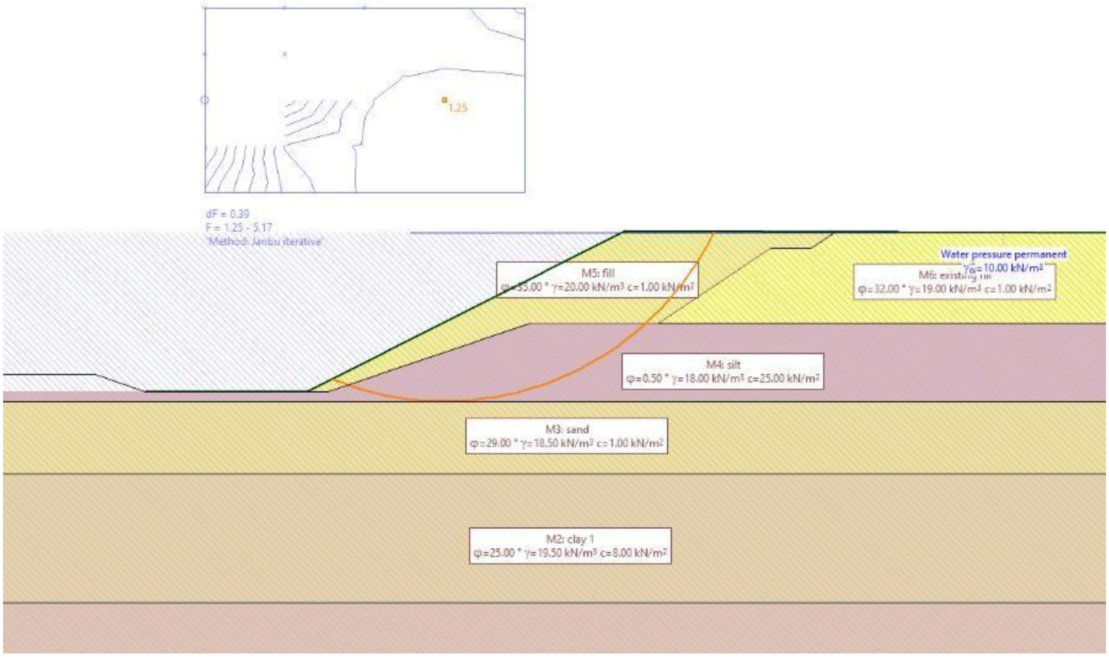


Figure 41 - Overall slope stability of the temporary fill in undrained conditions – GF.

Loading condition	Allowable F.S.	Existing F.S.
FS3	1.20	GF = 1.25

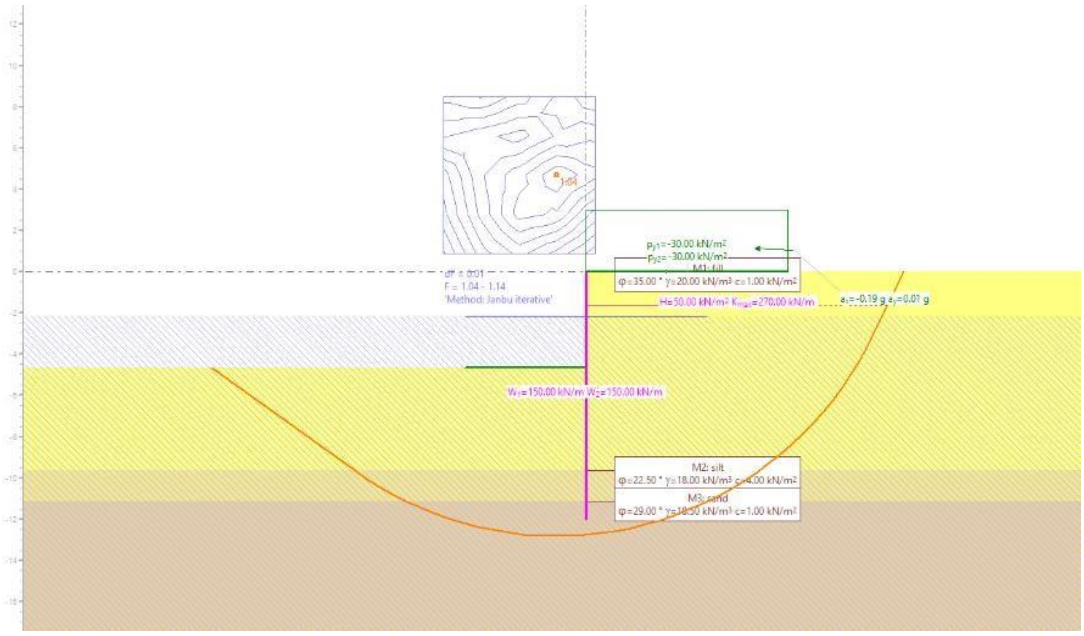


Figure 42 - Overall slope stability results of sheet pile wall.

Table 15 - Slope stability results for area A and B

Area	Loading condition	Allowable F.S.	Achieved F.S.
A	FS1	1.00	**1.61**
	FS2	1.00	**1.04**
B	FS1	1.00	**1.72**
	FS2	1.00	**1.01**

7.9 Deck on Pile

The quay has a total length of 501m and a width of 30.50 m. It will be constructed adjacently to the existing quay structures; from which it will be separated by expansion joints of 0.15m thickness. The quality of the concrete will be C 35/45 and the reinforcement bars of B500C.

The structure of the quay deck consists of reinforced concrete beams and slabs supported by bored reinforced concrete piles covered with steel casing. The deck will be supported with 6 longitudinal beams of 2.2m width and varying height from 1.6m to 1.7m. The three seaside longitudinal beams will be connected also in the traversal direction with beams of 2.2m width and varying height of 1.6m minimum.

The division in independent modules has resulted in the construction of eight (8) typical segments coded as A to H and an end segment (E) corresponding to the triangular end part of the quay. Each module has a length, in the longitudinal direction, of approximately 58.41m with 0.09m width expansion joint.

The slabs will have a minimum thickness of 0.7m. Since the concrete weight constitutes a significant portion of the seismic mass, it has been reduced, where possible, by using voids inside the slabs. The voids will be formed through the use of certified standard void fillers with a fire resistance capacity comparable to that of a solid slab.

The concrete piles are connected to the deck only in beam locations. They are constructed in 5.25mx4.50m mesh by C35/45 concrete and reinforcement bars of B500C, enclosed into a steel casing of t=10mm thickness. Their layout is in six rows with varying length starting from the seaside: Row A 40.60m, Row B 38.10m, Row C 35.60m and Rows D, E & F 34.20m. Along the deck two crane rail channels, of 0.4m width, will be constructed to serve the cranes of 10.5m gauge. For this purpose, a cable channel as well as all necessary utilities crossings and pits, such as electrical and water supply, IT etc. have been foreseen to be operational accommodated into the deck body.

The top surface of the deck will be inclined to the sea for drainage with a 0.5% slope.

7.10 Sheet Pile Wall & Passive Anchorage System (Deadman)

Between the transition zone with the existing quaywall and the 30.50m wide deck on piles, a sheet pile wall will be constructed to retain a heigh of 4.65m which is formed between the upper level of the pier pavement and the crest of the revetment.

The sheet pile will form a continuous wall composed by AU 18 steel section of grade S270GP. It will be 477.0m long and its total height (including cap beam and embedment length) will be 16m.

At the top end, the sheet pile will be connected with a cap beam of dimensions 2.0mx1.0m (height x width) made of C35/45 concrete and the reinforcement bars of B500C. The cap beams will be interrupted in the longitudinal direction, with a 0.05m width joint every 28m approximately.

The retaining wall will be supported with a single row of passive anchorage system applied at +0.60 MSL. This "*deadman*" type system, comprises from anchors Ø50mm, of steel B500C with spacing 4.0m, that efficiently connect the front cap beam with a rear beam of RC C35/45, located 12.73m beneath, with dimensions 2.0mx0.7m (height x width). The seaside displacement of the sheet pile will mobilize the passive resistance of the rear beam.

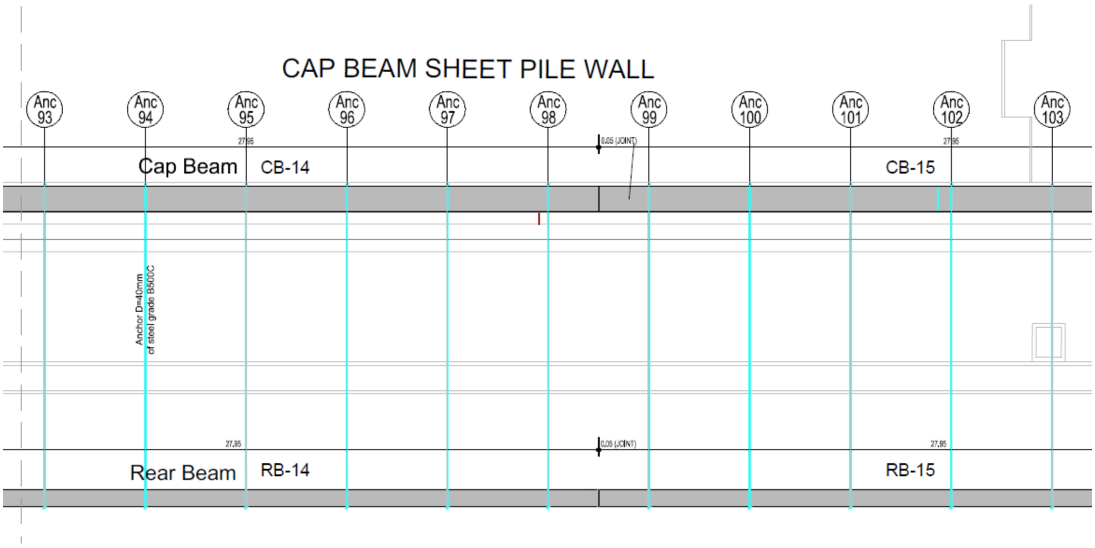


Figure 43 - Saple of quay 1 cap beam sheet pile wall

7.11 Dredging Works

Dredging operations will cover an area of 50.0m width from the new quaywall, in order a berthing zone of -11,5m (LAT) to be constructed. Beyond that zone, a wider area of approximately 75.0m width, will be dredged with depth of -10.25m, to create an area, needed for the navigation channel and manoeuvring area.

More specific the transition zone, where the existing pier stands, it is being excavated with slope 3:2 (hor: vert) and all existing piles are being trimmed accordingly. Special care should be made to the location, where the sheet pile will be constructed, so all existing piles or very coarse materials that reduce the drivability of the sheet sections, to be completely removed.

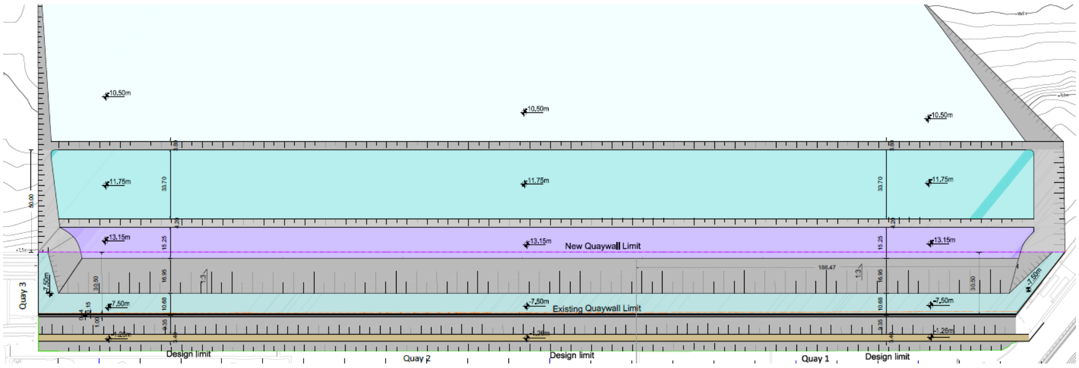


Figure 44 - Dredging plan

7.12 Backfilling Works

At the back of the sheet pile a relief prism, made of coarse-grained backfilling material, will be placed, up to a level of approximately +1.00 (Mean Sea Level - MSL). This way on the one hand a reduction in the active earth pressures on the quay wall is achieved, on the other hand smaller pressures are achieved at the bearing. From this point and up to the free surface of the quay wall (+2.15m maximum from LLW) subgrade layers of aggravates are laid and condensed according to the specifications.

7.13 Pavement Design

In order to achieve an acceptable and more effective result both from the technical and economic point of view, the consultant based on experience, has relied on hypotheses and calculation parameters of some of the most popular contemporary calculation methods for flexible road packages such as:

- AASHTO 1986-93 design procedure;
- CNR Design Guide.

- ARDM Albanian Road Design Manual as well as manuals and contemporary studies of various authors presented in international engineering forums such as "MDSHA Evaluation of Mechanistic-Empirical Design ProcedureVolume 2, CBR-Index soil properties Samar A.Taha - Academia.edu_files", etc.

All these calculation methods conclude in almost the same more or less constructive results for the function and the load that our path will have. However, in accordance with the tradition and practice of calculations of the model of rigid pavement in our country reflected in the approved standard of design, we have chosen the modeling of the road package based on calculations according to the method AASHTO '93.

Rigid pavements which consist of a properly prepared sub-grade, a sub-base or base layer, and a Portland cement concrete slab, the thickness of which is determined from the existing geotechnical and environmental conditions, and the anticipated loading it will experience during its service life, are selected. The material used as well as the thickness of each layer are depicted in the below figure.

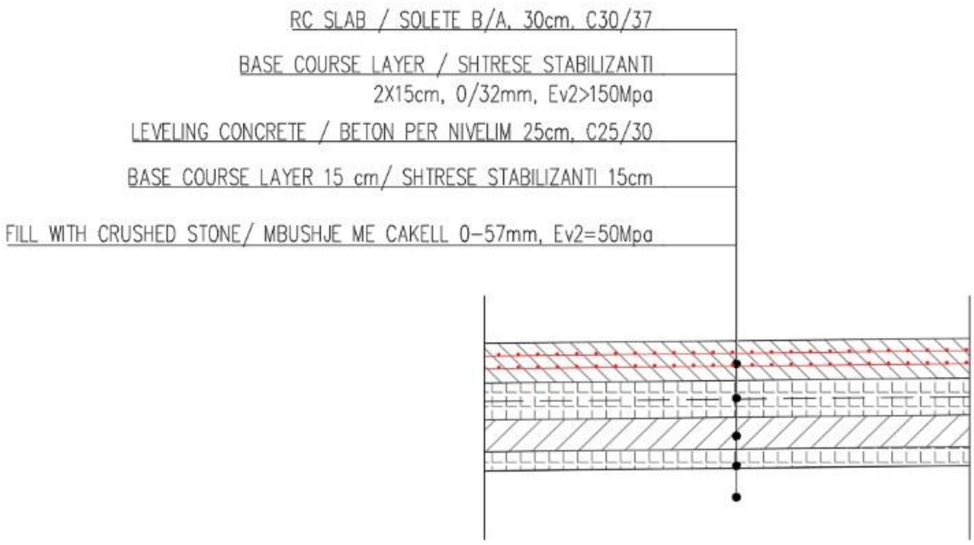


Figure 45 - The purposed pavement layers of quays 1 and 2

The basis for the design of any pavement structure is its ability to carry the intended loading over its design period. In rigid pavements, this would be the necessary slab thickness required to carry the lifetime loading. This thickness is a function of the following parameters:

- Traffic Loading Volumes over the design period; such as the volumes of the base year and terminal year;
- Traffic Loading Composition during the design period; such as the percent traffic mix composition of Multi-Unit, Single-Unit and Personal Vehicles as percentages (totalling to 100);
- The Modulus of Rupture fr of the concrete (flexural strength), is a measure of the flexural strength of the concrete as determined by breaking concrete beam test specimens. A fr of 200 KN/cm2 at 28 days must be used with the current specification for concrete pavement design.

The Effective Modulus of Subgrade Reaction, keff, allows pavement designers to take into account

7.14 Bollards

The capacity of the bollards placed on the superstructure of the quay wall is in accordance with the Recommendations of the Committee for Waterfront Structures, Harbours and Waterways - EAU 2009, the aforementioned recommendations and for a Bulk Carrier Vessel as a design ship, which has a displacement of 36.700tn, bollards of 80tn are proposed. Taking into consideration dynamic effects of the actions a bollard of **100tn** has been selected. The bollards are placed every 13.50m.

7.15 Fenders

According to the calculations (see Calculation Report) a fender every 13.50m will be placed. An indicative SUPER CONE FENDER SCN 1100 F2.7 type

(Trelleborg or equivalent), made of a natural and synthetic rubber blend is proposed, with the following properties:

- Maximum Energy, **E_{CV}= 754,00 kNm**
- Maximum Load on quaywall, **R_{CV}= 1.248,40 kN**

CHAPTER 8

CONCLUSIONS

The need to make substantial changes to obsolete infrastructure, produced innovations, additions and new designs of capable port infrastructures to interact, in the most convenient and rapid way possible, with the logistics network intermodal to support the main socio-economic activities contemporary.This work is based on the collaboration between the Durrës Port Authority and the international consultant, which arises from the consideration and aims to the implementation, by the Authority, of a more efficient multipurpose terminal, in order to keep the port in step with the rapid evolution inherent in the role and tasks of seaports around the world.

The literature landscape is already very vast in itself. Where was consider the interdisciplinary cut of this work, it is evident that it is itself, the result of a preliminary approach and subsequent interaction with previous experiences substantiated in the theme on one specific structure.

The scope of this work was to show the necessity of the Durres Port Authority to adapt his facility infrastructure to the demand of the market due to the exponential growth in the past decades of sea transport, trend that seems to continue. The principle in port activity is simple, more quay and more yards are directly proportional to the growth in revenues.

The work done in collaboration with the international consultant is of high level. Although the lack of information in the preliminary phases as the survey regarding winds direction and intensity, and wave motion and the lower number of similar investments in the national territory, the detailed design fulfills all the criteria to be classified as high rate project. The new design brings the versatility of multipurpose usage of the quays, which can handle grain cranes and mobile harbor cranes. With the new depth and the improved berthing line the port will be able to accommodate bigger vessels and handle higher volumes of freights.

The case have shown how the efficiency of a port exposed to swell can be improved. The wave penetration into the basin was reduced, yielding a diminished motion response. The response of the moored ship was reduced because of a reduced wave excitation and/or increased damping. A reduced vessel response implies an increased productivity and uptime, meaning increased revenues.

A multi-criteria analysis to evaluate all options and select a preferred option for further analysis in the next phase of the assignment. It was agreed that with the beneficiary criteria to be considered and their respective weightings. Criteria categories used include: technical feasibility and constructability; economic feasibility; regional impact; transport demand and forecasts; Environmental and social impacts and opportunities; and Physical risks associated with the current infrastructure condition. Investment costs are not evaluated independently but under both Economic feasibility and Transport demand and forecasts.

C1 overcomes its inferior financial and economic performance compared to B (investment costs 70% higher for similar financial and economic benefits) thanks to, inter alia, its ability to handle larger vessels and a greater comfort of operation resulting in a somewhat better regional competitive position as well as a faster implementation, due to existing detailed designs. This results in a higher indebtedness under option C1 and higher national budget support requirements to assist with loan amortization as operations, are insufficient to cover loan.

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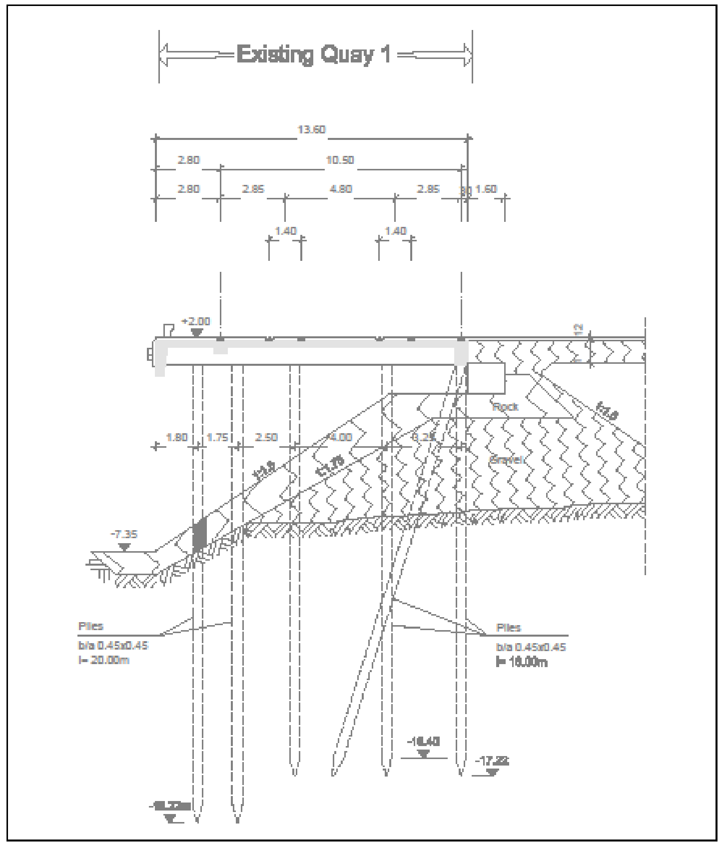
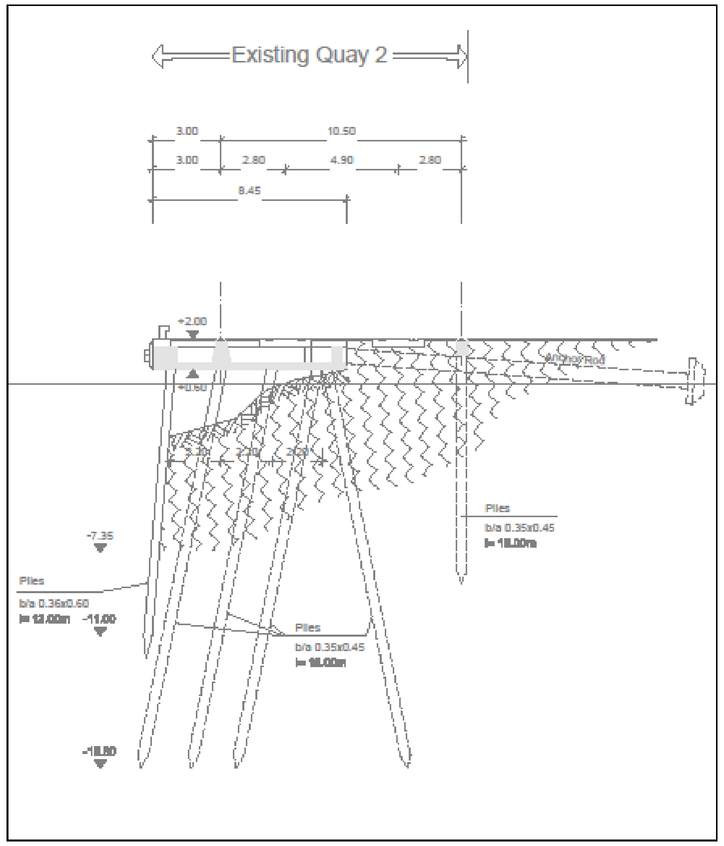
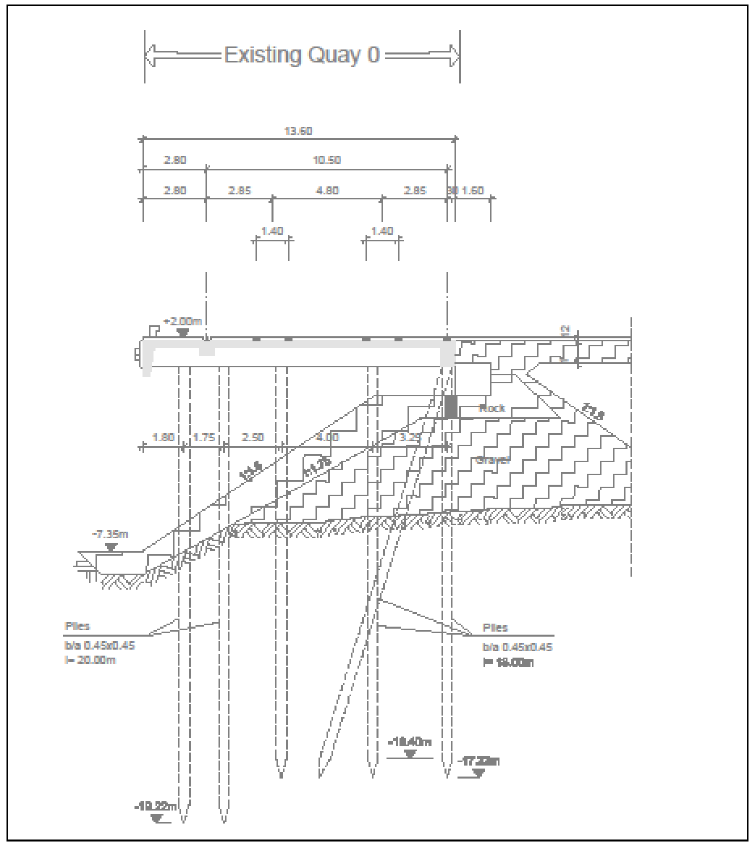
APPENDIX I - DRAWINGS



Section A-A



Section B-B



Section C-C

Figure 46 - Existing Sections

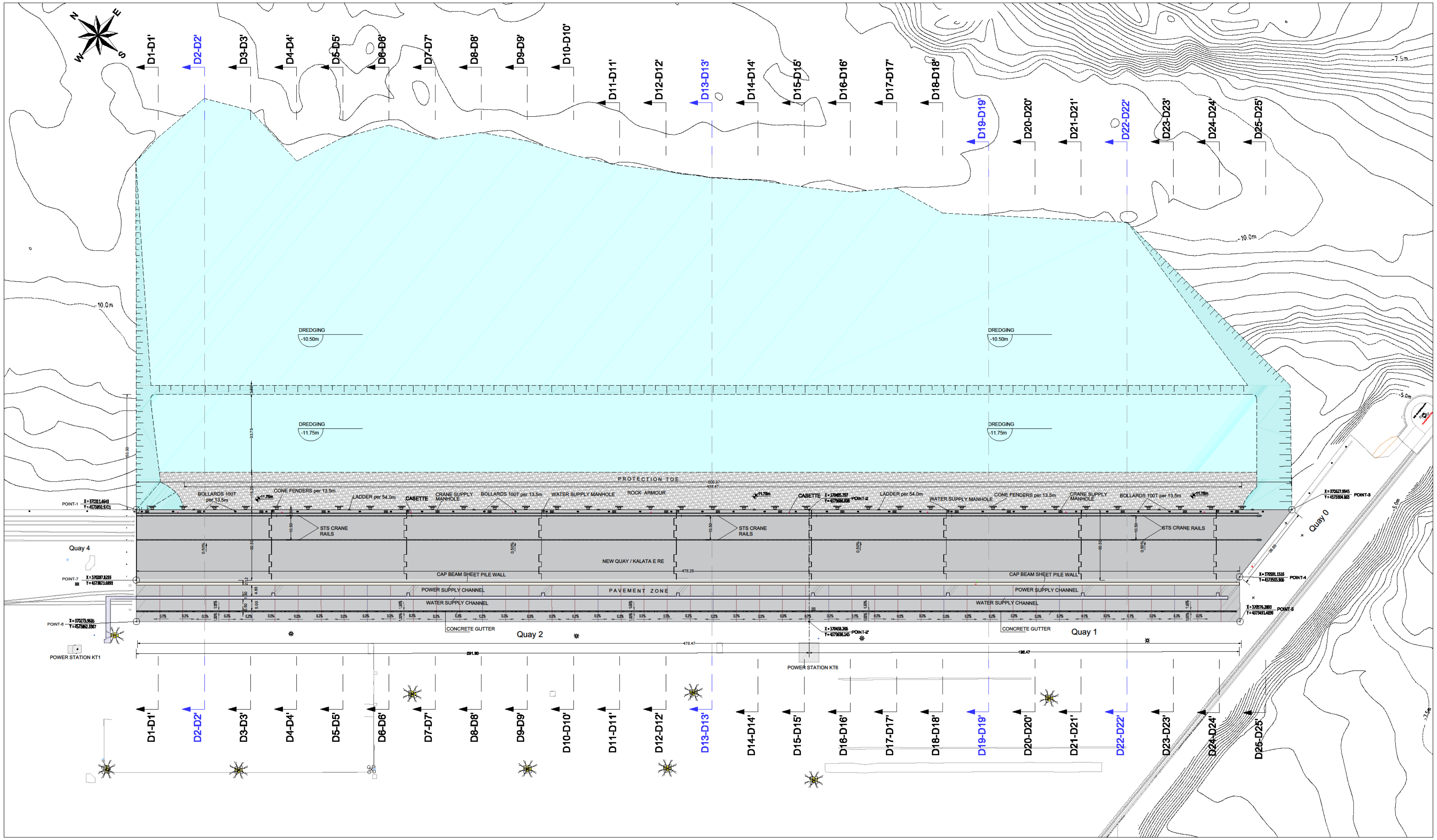


Figure 47 - New harbour works Layout

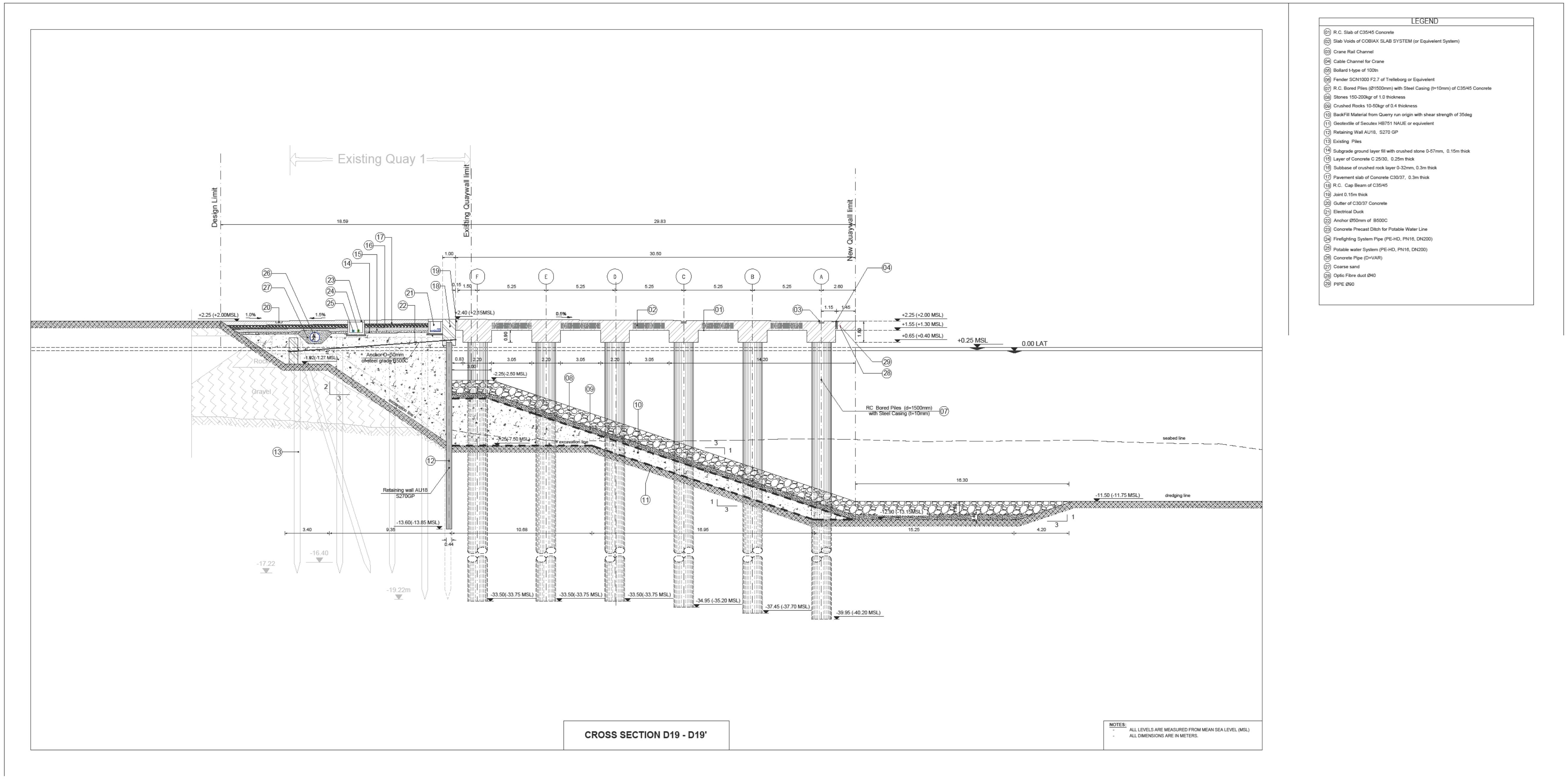


Figure 48 - Croos Section D-19 of Quay nr.1 After reconstruction

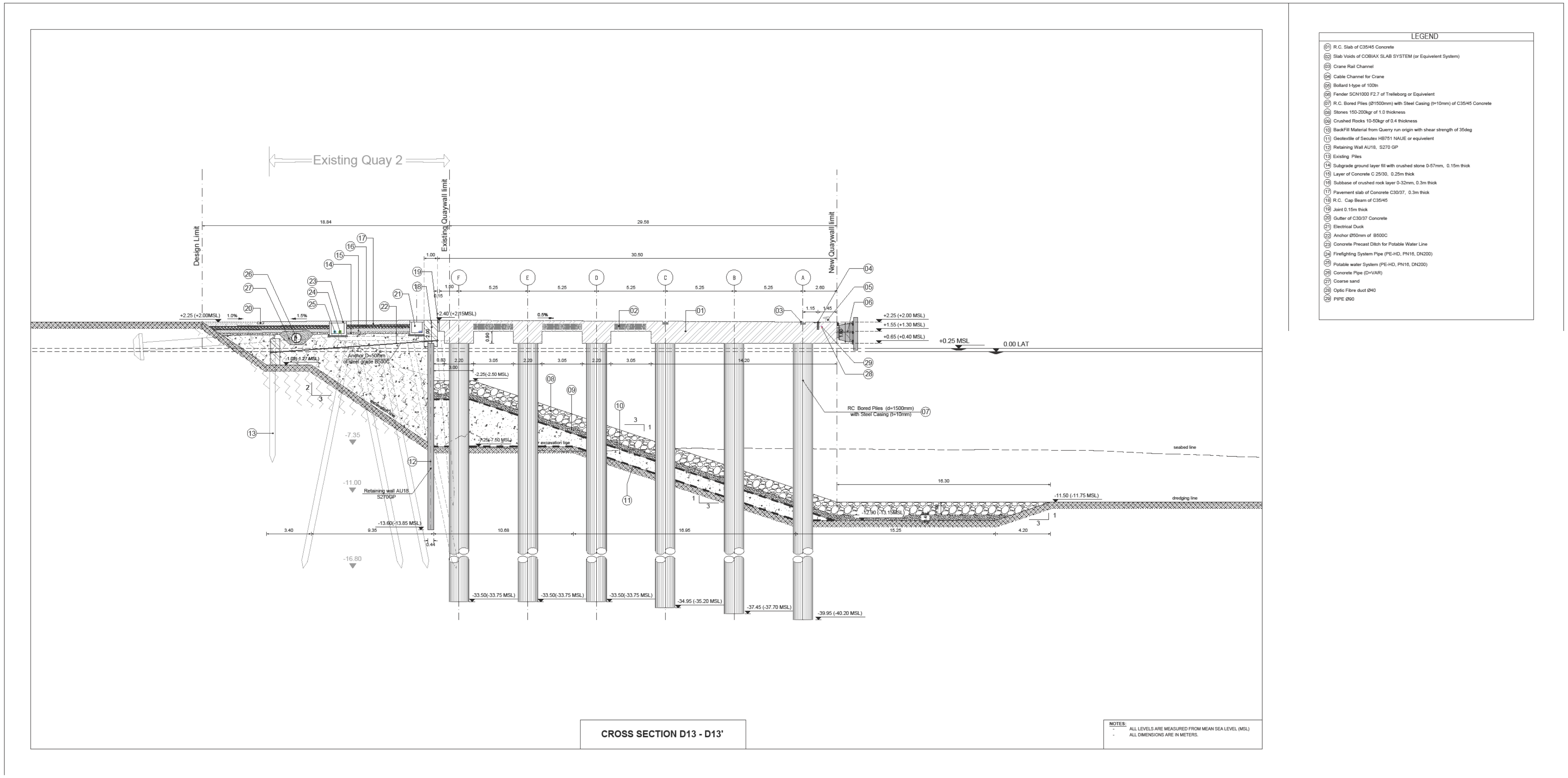


Figure 49 - Cross Section D-13 of Quays nr.2 After Reconstruction

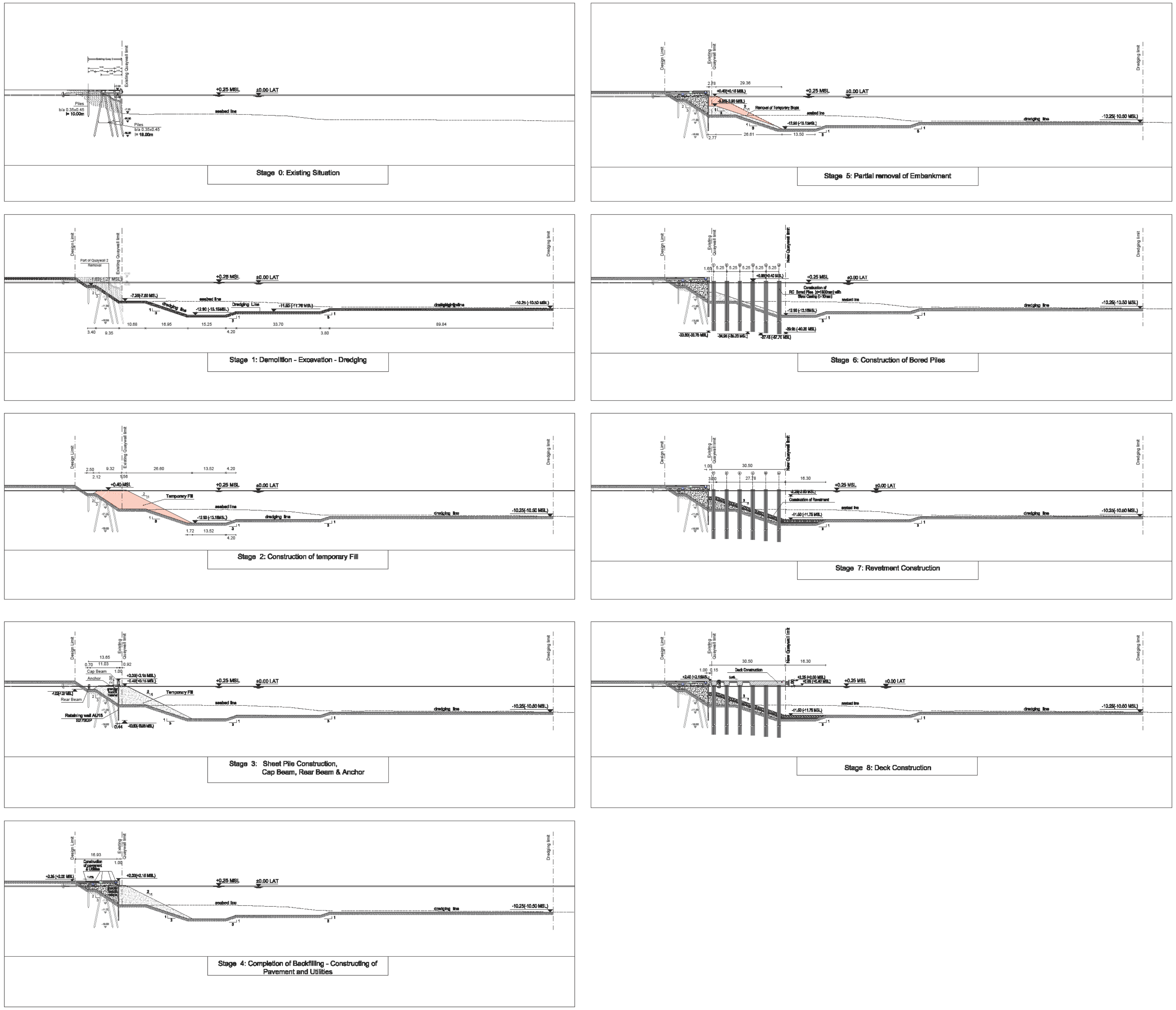


Figure 50 - Construction Sequencing