3rd International Conference on Transportation Geotechnics

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Workshop 2: Harbour Geotechnics

Edited by

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Workshop 2: Harbour Geotechnics

EXTENDED ABSTRACTS BOOK

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Workshop 2: Harbour Geotechnics

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Workshop 2: Harbour Geotechnics

Preface

The Portuguese Geotechnical Society (SPG), the University of Minho and the International Society for Soil Mechanics and Geotechnical Engineering (ISSMGE) organized the international Workshop "Harbour Geotechnics", that took place at the Port of Leixões in the 4th September 2016. This workshop was part of the 3rd International Conference on Transportation Geotechnics (3rd ICTG).

Geotechnics and harbour infrastructures are intimately connected, as the necessities of maritime infrastructures have motivated many advances and innovations in the scope of geotechnical engineering, thus bringing economic feasibility to such projects. The main objective of this Workshop was to gather international experts connected to research and teaching or to the industry that are involved in the several types of harbour geotechnical solutions. This brought about interesting opportunities for networking and discussion about ongoing works in the domain of harbour geotechnics. The Workshop was also an opportunity for the presentation of the most recent research works, new technological developments and new applications in the scope of harbour geotechnics. The topics of analysis included the evolution of geotechnical solutions at Port of Leixões, seismic design, soil fluidification, coastal geoscience mapping and case studies including stone columns, vibroflotation, driven piles and rubble-mound breakwater solutions.

The Editors

Yoshiaki Kikuchi
Alexandre Pinto
José Cerejeira

Workshop 2: Harbour Geotechnics

Workshop 2: Harbour Geotechnics

Table of contents

Workshop 2: Harbour Geotechnics

3rd International Conference on Transportation Geotechnics
4 September 2016 | Guimarães | Portugal

Workshop 2: Harbour Geotechnics

Geotechnical Problems and Their Solutions in Japanese Port Construction

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1 Introduction

There are about 1000 ports in Japan and most of them are constructed as sea ports, because Japan is surrounded by sea. There are three major bays such as Tokyo bay, Ise bay, Osaka bay, and most major ports are located in these bays. Thick soft clay layers cover the surface of the site for these major ports. And thickness of soft clay layers is more than thirty meters. In this meaning, port construction engineers have worked to overcome soft soil problems. Japan is famous as an earthquake country, but Japan is also facing soft soil problems. In this presentation, geotechnical conditions of Japanese ports are introduced at first, then geotechnical characteristics of Japanese soft clays are introduced, thirdly ground improvement methods mainly used in Japan are introduced. Finally some examples conducted in Japan against soft soils are introduced.

2 Geotechnical Conditions of Japanese Ports

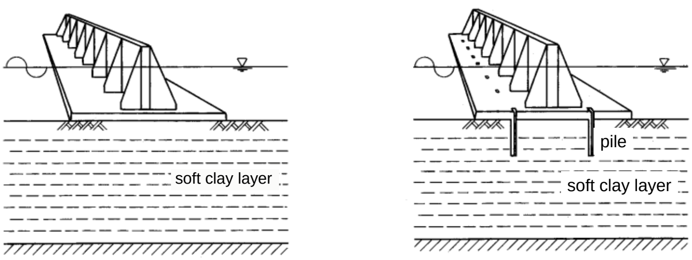
Kansai International Airport (KIA) is located 5 km from shore. The original sea depth of the site of KIA was about -20 m. Estimated total settlement of KIA is from 16 m to 20 m. This a kind of extreme evidence. But normally consolidated clay layers in Japanese port construction area are very compressible. Most important ports in Japan are located in large bays and surface of their ground is covered with about 30m of alluvial normally consolidated clays. They are categorized in silty-clay, but liquid limit of them is around 50 to 100 %. Ground improvement against large settlement problem is usually used in Japan. Popular ground improvement technics are sand compaction pile method for clay (high replacement ratio), deep mixing method, vertical drains, sand compaction pile method for clay (low replacement ratio).

3 Examples of Geotechnologies used in Japanese Port Construction

As mentioned in 2, Japanese geotechnical engineers have worked on soft and high compressible clays. In the presentation, I will introduce several examples of geotechnical technologies used in port construction in Japan. Here shows some of them. Japan is famous for earthquakes. And from this reason, design and construction technologies against earthquake are highly improved. And these topics will be introduced another speaker.

3.1 Soft landing breakwater

A soft landing breakwater was originally proposed for sites where the ground conditions are not good but wave conditions are rather mild. A key feature of the soft landing breakwater is its light self weight. As originally conceived, the horizontal resistance of this breakwater depends on the cohesion between the base plate and clay surface (Fig. 1 a)). Usually, however, the horizontal resistance of this type is inadequate for wave forces, and piles are therefore used to improve its resistance capacity. This modification is called the piled type (Fig. 1 b)). The mechanism of horizontal resistance in this type is rather complex because horizontal loads are borne not only by the base plate but also by the piles, and furthermore, the presence of piles may change the resistance mechanism of the cohesion between the base plate and the ground surface. A simplified design method for the piled type was firstly proposed in 1991, and was suited only for limited conditions. Due to this limitation, the applicability of the structure was confined to sites where the wave conditions is are very mild and the sea depth is very shallow. An improved design method was therefore needed to overcome these limitations. A new and more rational design method for the structure was proposed.



a) Original type (flat type) b) Piled type

Figure 1: Image of soft landing breakwater

3.2 New Geo-material - Super Geo-Material (SGM) -

Effective use of dredged clay has been one of the most important issues for port construction engineers in Japan, due to the maintenance of navigation channels producing a large amount of dredged clay. Light weight geo-materials, which are made from dredged clay, cement, and lightening materials such as air foam or expanded polystyrol (EPS) beads were developed for reducing earth pressure to water front structures which are deteriorated or are being improved. Total settlement will be decreased when it is used for reclamation, and earth pressure to the retaining structure will be reduced when this material is used for backfilling. The mechanical properties of the material were studied and the mixing method of the material along with the construction method using the material were also studied. A fourth runway of Haneda airport was constructed at the site of the mouth of the Tama River. One third of this runway was constructed on an open pier to prevent flooding from the river. The rest of the runway is to be constructed on reclaimed land. The retaining structure at this conjunction point should be able to sustain large earth pressure and minimize settlement of reclaimed land next to the structure. Super Geo-Materials (SGM), which is one of the light weight geo-materials made from cement treated clay with air foam, were discussed for possible use. The durability of this material was one of the biggest issues at that time. As SGM is applied to waterfront construction, the characteristics of this material can be influenced by seawater. There is a risk that some air in the SGM may be replaced with water over time and a prediction of the change of unit weight with time is required. Much knowledge has been accumulated on making the strength of the clay higher. Yet, there is much uncertainty about the increasing of the unit weight by absorption of water. Then the mechanism of absorption SGM was investigated. The absorption rate of SGM under the wet sand condition was found very slow. From this conclusion, SGM was used for the backfill of the conjunction point of the runway.

3.3 Reuse of by-products as geo-materials in port construction in Japan

As port constructions are usually of a large scale, they need a large amount of construction materials in a short time. Most of these geo-materials had been provided from virgin natural materials until the early 1990s in Japan. On the other hand, some port constructions produce a large amount of dredged soil. Generally dredged soils are low quality materials for reclamation, because they contain a lot of water and are weak in shear strength. This method of utilization is not recommended. Industrial zones have been located in port areas in Japan, and factories such as iron-foundry plants or heat power plants in industrial zones have produced large amounts of slag or coal ash. Japan has anticipated the utilization of such by-products in port areas. Large amounts of by-products such as dredged soil, slag, and coal ash which can be used for geo-materials have been produced in port areas. However, it is usually difficult to intensively use them in their natural states, because they are low quality materials with whose original forms have large variations in terms of characteristics. If these materials can be changed to high quality materials with minimum treatment, they will be intensively used for geo-materials for port construction and reduce the use of natural resources. This kind of usage is promising for sustainability and maintains the environment. Methods for reusing of industrial and construction by- products have been studied in Japan. Outline of the by-products used in port construction is presented here. And, engineering issues and state of the art of using fly ash, and iron slag which have intensively used in port construction are introduced.

3rd International Conference on Transportation Geotechnics
4 September 2016 | Guimarães | Portugal

Workshop 2: Harbour Geotechnics

Offshore vibro replacement for large depths and challenging soil conditions - Recent cases from Europe and South America

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1 Introduction

The use of deep vibratory methods for the improvement of the bearing capacity, reduction of settlement and liquefaction mitigation of weak soils that are unsuitable as foundation for offshore structures dates back over more than 50 years. During this long period of application, a lot of experience was gained with this technology and enormous progress was made pushing forward boundaries and limitations for its application. The continuous development has been experienced not only regarding design methods and standards, but also equipment to carry them out in practice.

As a result of permanent technological progress Keller Group has developed the Alpha-S System that permits offshore vibro replacement stone column construction using the bottom feed dry method. With this system, stone columns can be installed to large depths, with the current record being at approx. 50 m, in challenging soil conditions. Keller´s gravel-jet, sophisticated material feeding system, improves handling, productivity and quality control.
In the last decade, the unique features of vibro replacement performed by Alpha-S System were used in numerous offshore and port projects in Europe and South America, in order to facilitate the building process in complicated soil conditions and to improve the level of safety and efficiency. In this extended abstract selected details of some of those projects will be presented with objective to explain how vibro replacement can be used as an integral part of the solution and to solve detailed problems. Based on these practical examples, the basic framework for the design and execution of vibro replacement will be outlined.

2 Alpha-S System

For the installation of longer stone columns, stones or gravel reliably need to be transported to large depths into the seabed. This can only be achieved using a bottom-feed system. The complete setup for the stone column operation comprises crane with vibrator string, gravel-sender and gravel-jet, double-locked chambers and the Alpha-S vibrator.
Alpha-S System has been developed with automatic transport of the gravel to the tip of vibrator. Keller´s hydraulic gravel-sender and gravel-jet system pumps through a 200 mm diameter hose to the top of vibrator using high velocity water. A fully automated and

computerized system is used to control and monitor the transportation of stones from gravel-jet system to the tip of vibrator.
The advantages of Alpha-S System can be listed as follows:

- Increase in length of vibrator string and hence increased achievable column length and depth of installation.
- High productivity and efficiency.
- No need for alignment of additional auxiliary equipment for gravel supply.
- Closed system of gravel transportation minimizes the wastage of stones.
- Fully automatic transfer of gravel results in better quality control.
- GPS System on the operator´s cabin to reliably locate the compaction points under water.

3 Reconstruction of ASMAR Shipyard Talcahuano, Chile

ASMAR is one of the most important shipyards in Chile and in South America, located in Concepcion Bay, Talcahuano. Due to damages caused by a strong earthquake and tsunami of February 2010 (magnitude 8,8 Richter), reconstruction and restauration of facilities was necessary to reestablish port and shipyard activities.

All types of structural and geotechnical pathologies including liquefaction were detected. Soil improvement by offshore vibro replacement was adopted as a part of the global project solution which permitted to take advantage of the existing dock/berth structure despite observed damages, and avoid its demolition; *consequently new construction activities and costs were significantly reduced.*

Almost all benefits of vibro replacement and Alpha-S System were exploited. Soil improvement was performed up to 30,0 m below seabed, in order to increase bearing capacity and shear strength, and to mitigate liquefaction risks of soils formed by very soft silts and loose silty sands. According to the causes of liquefaction, its mitigation by means of stone columns was achieved due to the superposition of following positive effects, as has been stated by several researchers:

1) Soil densification and increase of the in-situ lateral stress (increase in CRR).
2) Reinforcement of the soil with the stiffer columns of compacted gravel (reduction of CSR).
3) Increment of drainage of earthquake-induced excess pore water pressures from the in-situ soils (reduction of CSR).

In total 95.000 m of stone columns were installed, 52.000 m from the existing berth structure and 43.000 m from the barge (see Figure 1).

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Figure 2: Stone columns performed from existing berth structure and from barge – typical section.

Figure 2: ASMAR Shipyard – stone column installation from the barge.

Offshore Stone Columns to improve alluvial soils for caissons quay wall and land fill foundations

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ABSTRACT

The presentation describes several alternatives considered in project of the quay and reclamation area of the "Recinto y Atraque en el Dique del Este del Puerto de Valencia", which led to a solution consisting on building a mooring quay at – 16 Z.H using concrete caissons, placed over a selected stone prism, 5 meters thick (Figure 3).

With an initial scenario based upon the geological data available two alternatives were studied. One consisting on dredging up to -28,0 Z.H so that the sand seabed level could be reached or another one on which the dredging works would be performed just up to -21,0 Z.H and a different solution would be considered to enhance the bearing capacity of the existing clay layers in the seabed (Figure 3).

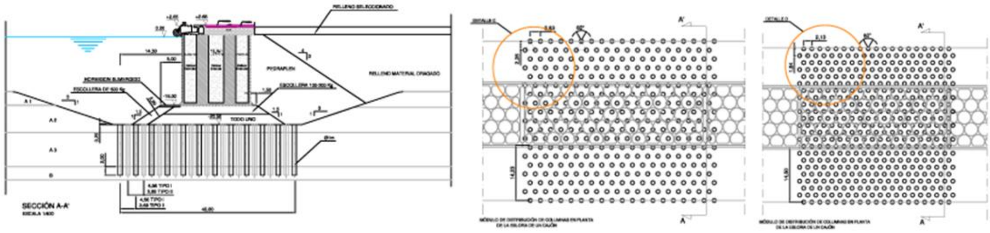

Figure 3: Solution found for the caisson foundation

The solution found was to execute 1 meter diameter gravel columns (from -31,0 Z.H up to -21,0 Z.H) with the objective of replacing 20% of the existing seabed materials, consisting mostly of clays. The gravel columns, in general have stayed from 1,5 up to 2,0 meters below the existing sand level at -25,0~28,0 Z.H . Two methods used to perform this task are mentioned with detail has well as some particular points regarding the off-shore works. Finally the results of such a solution are presented as a consequence of the continuous monitoring and observation of the project since the solution was implemented (Figure 3and Figure 4).

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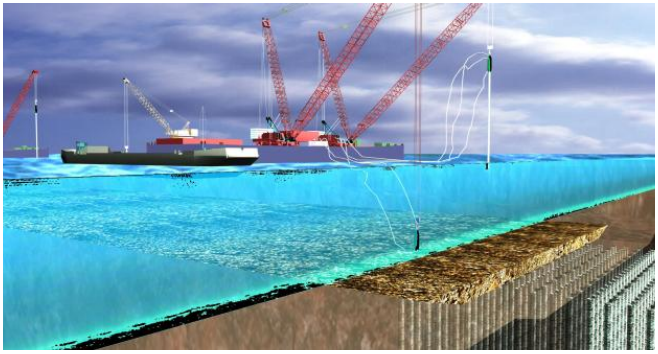

Figure 4: Stone column execution method

It is also described the procedure taken to fill the reclamation area next to the quay wall, consisting on a area of 330.000,00 m^2. As all reclamation area is founded on very soft clay layer measures had to be taken when filling with sandy/rocky material coming from the dredging works. Inspections were continuously carried out while filling operations took place so that materials from the seabed would not mix with the ones coming from dredgers when repulsed by rainbow system.

Such inspections were carried out both by diver and surveyor teams, contrasting survey data with the real situation below sea level. Conclusions were taken from data analysis and afterwards transmitted to dredger that knew from that moment where to repulse the dredged material on its next dredging/dumping cycle.

With such procedure it was possible to cover all the soft clay layer of the reclamation area foundation with a 1,50 ~2,00 meter sandy/rocky layer. From that on, this situation permitted that all the reclamation area tasks could be *safely* carried out since the danger of mixing existing seabed clays and competent dredged materials brought had been greatly mitigated by the layer previously created.

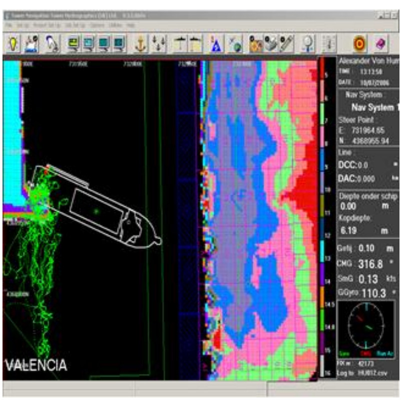

Figure 5: Data screen form dredger while repulsing

Figure 6: Repulsing of dredged material

The remain material to fulfill the reclamation area needs, regarding filling material, was performed with dumper trucks in a controlled way. Any uprising material was taken off by split barge to a designated disposal area.

Neither any significant settlement nor singular phenomena related with any instability on the reclamation area has been reported until now.

Conclusion

Soft clays and limes are difficult materials to handle with, principally when it comes to foundation solution in marine works. Solutions may lead to scenarios both costly and environmentally aggressive. Alternatives are possible but require carefulness and constant monitoring during and after implementation. However, when taking the necessary measures and procedures intended results may be achieved. The role of previous geological tests campaign is central and absolutely critical to the success of any solution.

Workshop 2: Harbour Geotechnics

Landfill treatment in La Guaira Port, Venezuela

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1 Abstract

The objective of this communication is to present the new Container Terminal in La Guaira Port, more precisely to describe the procedure used in the landfill treatment to improve the load capacity of the soil. The structure of the pier is composed by a concrete platform supported by piles with an extension of 693m. Since the area where containers are moved and parked consists of a hydraulic landfill situated on a seismic zone, it was necessary to define what kind of treatment would me more suitable to overcome the possibility of a liquefaction phenomenon. Since the embankment was made with sandy soils the chosen procedure was vibroflotation.

2 Introduction

The Port of La Guaira is situated in the central coast of Venezuela, north of the capital Caracas. This infrastructure has a water level protected by a breakwater 1.300m long and several piers. In operational terms it is characterized by fractional general cargo and containers movements in the North Quay, with strong operating constraints resulting from the embankment with only 50m wide.

To improve the port performance the national state-owned company that manages the Port – Bolipuertos – signed a contract with the Portuguese company Teixeira Duarte in EPC mode (Engineering – Procurement – Construction). The contract established under the Portugal –Venezuela Agreement included:

1. Execution of a pier;
2. Container Park (rehabilitation and area conquered to the sea);
3. Port equipment (STS's and RTG's);
4. Administrative buildings;
5. Complete formation in all port operations, from the vessel arrival to the shipment of goods (intern or extern exportation);

Workshop 2: Harbour Geotechnics

Figure 7: New Container Terminal in La Guaira Port

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The geometry of the pier consists of modules of 60m length, each one formed by 8 transverse alignments distanced 7.50m. Each transverse alignment is supported by 5 concrete piles with 1.20m diameter. The transverse alignments are connected by beams of 1.70m high and 1.30m wide.

2.1 Soil Treatment

The expansion and modernization project of the new container terminal was made with an hydraulic landfill conquered to the sea. To achieve this it was necessary to materialize two marginal retentions (West-East) and a frontal prism, longitudinal to the pier building, retaining the embankment. It becomes important to refer that this hydraulic landfill was materialized through the existing sandy soils dredged in the area of the Port of La Guaira. This led to an increase of the port draft (-15.20) thereby allowing access to Post-Panamax vessels.

The project was developed along with the Contractor in order to provide a solution involving a combination of precast and concreted “in situ” elements in order to enable the advancing of the pier, from East to West.Through this process it was possible to materialize the hydraulic embankment in stages and hence the treatment of the soil, thus ensuring the necessary conditions for the implementation of service networks and paving the treated area.

Workshop 2: Harbour Geotechnics

Figure 8: New Container Terminal in La Guaira Port

Figure 9: Execution of vibroflotation

The chosen solution to carry out the treatment of the landfill was vibroflotation, given the existing conditions and materials (in the materialization of the landfill were used 1.1 million m3 of sand). Thus, after the execution of part of the landfill several tests were performed since it was necessary to determine the characteristics and depths of treatment, including the mesh, the up speed of the equipment and the treatment time per level. It is important to refer that SPT and CPTU tests were performed as well as particle size analysis and plate bearing tests. These were made prior and during the treatment in order to control the improvement process of vibroflotation.

This treatment led to an average increase in relative density. It is important to refer that were introduced 8650m3 of additional material (sand and stone aggregates).
In the vibroflotation were used three cranes with four “needles” in continuous work 24h/24h, 6 days a week.

3 Conclusion

The need for landfill treatment depends both on the type of use and the risk of liquefaction. The choice of the treatment (replacement soils, preload deep vibration, dynamic compaction, drain installation or rigid structures) will have to take into account several factors:

1. Nature of soil to be treated;
2. Desired improvement;
3. Deadline of achieving the results;
4. Cost/Benefit analysis of the each kind of treatment;
5. Local conditions for execution.

Workshop 2: Harbour Geotechnics

Engineering, Procurement and Construction of the Jetty for the Marine Operations Support Dock in Soyo, Angola

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1 Introduction

The Jetty was built in the natural gas processing plant of Angola LNG in Soyo, province of Zaire do Norte, in Angola. It is located at Diogo Cão Bay, in the end of Congo River, one of the largest rivers in the world, which limits the border between Angola and the Democratic Republic of Congo, 500 km north of Luanda.

Located inside the MOSD - Marine Operations Support Dock complex to support marine operations related with gas transportation, it was built 294m long and 8.1m wide, perpendicular to the existing quay. It has capacity for the mooring of several vessels, namely large Svitzer tugboats.

Teixeira Duarte, Engenharia e Construções S.A., was the company responsible for this project in E P C scheme. As this was a job of an urgent nature it was necessary to develop a construction method that would simultaneously comply with the tender base project and with the required execution period of 18 months.

2 Structural Solution and Geology

The Jetty is made of a structure with a reinforced concrete deck with 55 spans of 5.325m, supported by 29m deep metal piles driven by vibration, filled with sand and a concrete plug on the top to materialize the connection to the deck, (Figure1). The structure was designed in modules, so that its application would be easy and fast, thus avoiding the restrains imposed by the tide level.

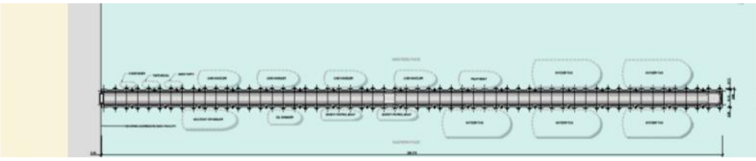

Figure 1: Jetty overview.

Workshop 2: Harbour Geotechnics

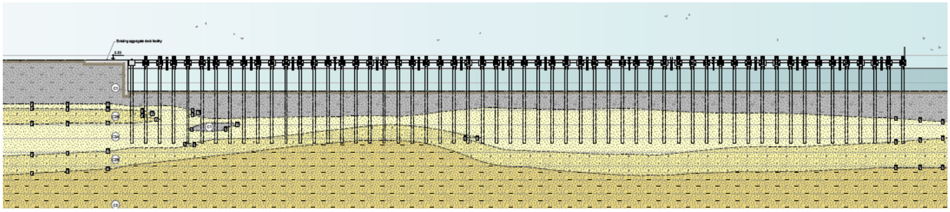

Figure 2: Jetty longitudinal section and geological layers.

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Recent formations were identified from the Holocene and Pleistocene (Miocene) periods, composed of medium to fine sands, sometimes coarse, very muddy in the most superficial layers and sometimes clayey in the underlying layers, characterized by a gradual increase of the compactness (Figure 2).

3 Constructive Solution

The geotechnical details noticed by the initial and complementary surveys and the poor accessibilities, inherent to the worksite, imposed the development and detailed definition of the whole constructive process of the job, long before the starting of its execution.

The adopted constructive solution consisted in the use of a 50m long and 8m wide provisory metal jetty, which gave support to the final jetty construction in reinforced concrete. The provisory jetty allowed the creation of several work fronts, always with a privileged connection to the land, thus facilitating the simultaneous performance of all fixed jetty execution stages (Figure 3).

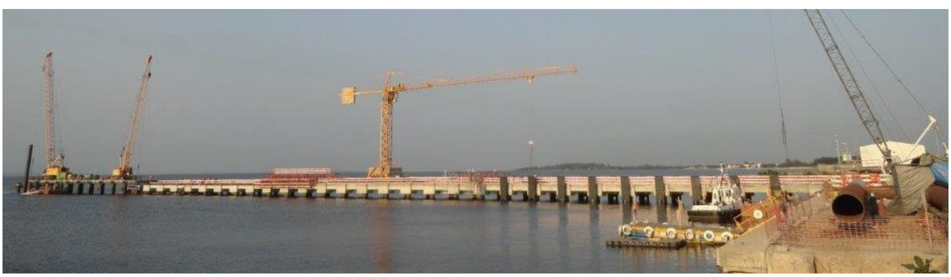

Figure 3: Jetty under construction. Stg.1 - Pile driving. Stg.2 - Temporary jetty movement. Stg.3 - Precast assembly. Stg.4 – Cantilever system. Stg.5 – Fenders assembly with a tower crane on tracks.

4 Final Considerations

The genesis of the construction and the available resources were closely linked to the constructive solutions developed during the project stage and later on, during its execution. Always resorting to trial runs and tests carried out on site.

Workshop 2: Harbour Geotechnics

The idea of a temporary metal jetty combined with a reinforced concrete precast modular cantilevered system allowed the work execution above the tide level, as well as to achieve the required efficiency rates, always working on unquestionable safety and quality levels (Figure 4).

Figure 4: General overview of the works in the final stage.

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Workshop 2: Harbour Geotechnics

On the conception, design and contracting of important port infrastructures- Some examples

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1 Abstract

During the 1970s and the early 1980s, several ship repair yards sprang up in the Mediterranean area (Spain, France, Italy and Malta) and in the Arabian Gulf (Bahrain, Saudi Arabia, Iran and Dubai), chiefly to take advantage of their location at the end of ships' return journey with ballast from all routes passing the main source of the world oil supply. Similar initiatives were implemented in Latin America (Ecuador, Venezuela and Brazil) and in Africa.

Portuguese engineering[1] played a major role in carrying out preliminary studies, designs and the supervision of the construction of the largest shipyards undertaken in the world at that time, notably: Lisnave at Margueira, in Almada and Setenave at Mitrena, in Setúbal, both in Portugal; Astilleros Españoles in Cadiz, Spain; ASRY, the Arab Shipbuilding and Repair Yard, in Bahrain; and JSRY, the Jeddah Shipbuilding and Repair Yard in Saudi Arabia.

A shipyard built from scratch is undoubtedly a venture in which marine works are preponderant and may account for 65% of the total cost, thereby justifying the need for sound experience in the design of these works in order to achieve their better adaptation to natural – generally extremely adverse – conditions. A shipyard generally requires all types of marine works (protection and sheltering works, dredging and fills, drydocks, quays, jetties and dolphins) and complex geotechnica

l works (soil treatment and consolidation, cofferdams and lowering the water table).
The launch of a venture of this kind, and other port infrastructures, is thus always a great challenge to the creativity of the designer in his quest for construction solutions that not only meet demanding operational, environmental and safety requirements but also optimize construction costs and periods.

The Eurocode 7 (EC7) classifies geotechnical structures into three groups according to their complexity, experience, geotechnical information and the risk of damage. In turn, the Portuguese legislation on the development of public works projects (ordinance 701-H/2008 of 29 July) classifies port works in four categories (I to IV) according to the degree of difficulty of design and the complexity of the project. Dry docks and locks belong to the

[1] Initially in the form of PROFABRIL and then PROMAN since the 1980s

category IV and other port infrastructure to category III. On the other hand, EC7 gives them the highest classification.

The conception, design and contracting of the most important port infrastructures requires special care from all stakeholders and some cases, based on the long and diverse experience of the author, will be described in the presentation.

2 Conception and Design

It is generally in the initial phase of a project, when the location and the conceptual design of works are studied, that there is opportunity to achieve major economies, or, conversely, avoid heavy losses. With the selection of the basic engineering solution, the functional features and the general arrangement of the facilities are established. The physical characteristics of the site location and the proposed civil and infrastructure works are established, forming the basis of investment costs and operating costs.

The next phase of the basic design, in which the works and equipment are defined quantitatively and qualitatively, also offers the opportunity to assess and benchmark findings of the previous phase, refining the functional requirements of the project and adaptation to local physical conditions. For this purpose, it is essential to completely characterize local conditions (topographical, geological and geotechnical surveys, etc.) in the study and design phases. If the works and equipment are well conceived and designed, the costs of the following stages will be more strictly controlled.

While in the initial study and design phase, costs are still relatively small, they significantly increase in the construction phase, when shortcomings in localization, conception and design are always difficult and sometimes impossible to overcome in terms of cost and time.

All too often, the responsible entities reduce the initial investments (in studies and designs), despite being comparatively small, though admittedly time-consuming. This can create major difficulties and setbacks in the later stages, or even compromise the viability of the project.

The above applies to works of any nature. In the specific case of port facilities, it gains special relevance, since the always expensive works are usually located in areas where conditions are very complex and require research, specialized studies and design by experienced personnel and, of course, time. Often they are located in estuaries of rivers where the geotechnical conditions are not very favourable for the deployment of heavy structures and the creation of large areas subjected to the high loads and tight deformability requirements required of modern port facilities. The structural design of works is particularly difficult due to the need for special structures and adequate soil treatment techniques.

For example, the cost of a drydock may represent about 35% or more of the total cost of building a shipyard. The cost of building a dry dock with a drained bottom slab, can be 20 to 30% lower than a dry dock with an anchored bottom slab.
To highlight the importance of conception studies of this type of works, and also the verification by an independent checker, the cases of several shipyards and port infrastructures, with very different local conditions, will be described in the presentation.

3 Contracting Procedure

One of two forms of contract is usually adopted:

- A traditional design and construction contract involve a three-party arrangement between the Owner, the Engineer and the Contractor. The Owner enters into an agreement with the Engineer to prepare the design of the works and another with the Contractor to carry out the construction of the works according to the design produced by the Engineer and approved by the Owner. The Owner thus warrants the sufficiency of the design and assumes any liability for defects before the Contractor. The Owner may find it convenient to engage an independent professional adviser to check the design produced by the Engineer. The Contractor is then responsible for defective construction and workmanship, but is free from any liability for design defects. The Engineer, while responsible for design, does not assume any liability for defective construction. The Owner enters also into an agreement with the Engineer or another independent professional to supervise the construction of the works.
- In a design and build contract the Owner enters into a single agreement with the Contractor who will perform both the design and construction. The Contractor´s objective is to satisfy the Owner's broad project objectives and requirements rather than to adhere rigidly to the Engineer´s design plans and specifications. As well as being responsible for faulty workmanship in construction, the Contractor is also liable for any deficiencies in the design. The design and build Contractor may be composed of a joint venture of a contractor and an engineer, or the former engages the latter as sub-consultant. Either way, the Owner is looking to the Contractor for the full package of design and construction.

To highlight the advantages and disadvantages of these two procedures, the contracting and construction of several shipyards and port facilities projects will be described in the presentation.

4 Final Remarks

While in the pre implementation phase of an important port infrastructure, comprising the initial studies and designs, the costs are relatively small, they increase significantly in the construction phase. To reduce initial investment, despite being comparatively small, although admittedly time consuming, can create major difficulties and setbacks in the later stages, or even compromise the viability of the project. An efficient pre implementation phase is fundamental for the success of the project in either a traditional and construction contract or a design and build contract.

Design and build contracts without such pre implementation phase, especially those which do not the operation of facilities by the Contractor, can involve high risks of failure, either in terms of quality, to the extent that cost reduction may reduce the useful life of the infrastructure, thus compromising the viability of the project, or in terms required to satisfy the Owner´s project objectives.

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Workshop 2: Harbour Geotechnics

Sea waves and seabed interaction. Partial fluidification of breakwaters foundations

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1 Introduction

The north breakwater of Nazaré harbor, Portugal, was subject to rehabilitation works to reconstruct the structure's rotation head and part of the trunk, which were severely damaged during the 2012/2013 and 2013/2014 winter storms (Santos Ferreira *et al*, 2015). It was assumed for the rehabilitation design that the structure's deterioration was due to the wave action on the breakwater blocks, 40 ton tetrapods and 5-12 ton rock blocks, and to their consequent breaking and, or removal from the rotation head.
The original design, from 1980, considered that the maximum possible wave that could occur in the breakwater area was 7m based on the theoretical transformation of the wave towards the coast; there is evidence that, during the referred storms, the wave attacking the breakwater reached 9m high.

2 The original design of the breakwater

Since 1920, several studies and designs have been presented to build a harbor in Nazaré. Almost all the solutions considered the construction of breakwaters in the sea, usually occupying a part of today's Nazaré beach. None of these first studies considered the dredging, inside the shoreline, of the harbor basins. Originally, the river was parallel to the shoreline and a 90º turn (perpendicular to the shoreline) linked it to its small estuary where it ends (Figure 1a). The concept, when in the late years of 1970 was finally decided to build the harbor, was to use the area between the river and the shoreline, to dredge the basins, and to create the necessary earthfills. The river would finish inside the harbor's basin, but keeping it separated (Figure 1b).

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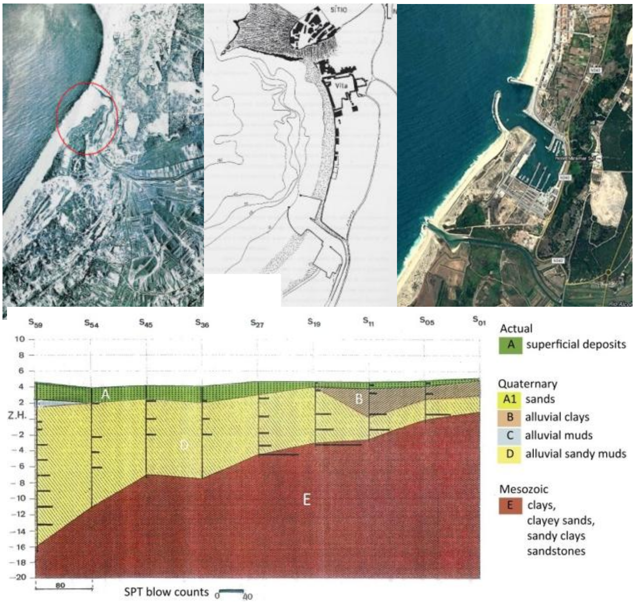

Figure 10: a) River and coast before the harbor's construction; b) Design solution; c) Final layout, with the new river estuary; d) Geotechnical cross section along the new estuary of the river, perpendicular to the shoreline.

Several studies of sediment transport along the coast were carried out, and the conclusion was that the onshore transport would not have a significant influence on the sediment deposition on the harbor and its entrance.

Nevertheless, during the harbor's construction, it became clear that the sediment carried by the river would deposit in the harbor entrance and navigation canal, due to the wider flow section and to the reduction of velocities, and so the decision to change the river estuary.

To support the design, a dense boring survey was carried out. It showed that the formations in the harbor area are, from down up, sandstones and clays, sandy or sandy mud layers, clays, sands and superficial deposits. From the interior to the seashore, the depth of the sandstone and clay top increases, as well as the thickness of the sandy muds. Beach sand was found only in a small area, as the survey was not extended to the beach itself. Figure 1d shows the geotechnical cross section 4, along the new estuary of the river, perpendicular to the shoreline.

As a consequence, the final layout adopted is presented in Figure 1c. The north breakwater roundhead was protected with two layers of 40 ton tetrapods taking into account the significant wave of 7m defined as stated above.

3 The destruction of the breakwater and its reconstruction

As stated above, the breakwater was heavily damaged during the 2012/2013 and 2013/2014 winter storms. The reconstruction design considered the actual structure’s destruction by the occurred 9m waves, which removed the protection tetrapods, as they were dimensioned for a 7m wave. The tetrapods in the rotation head were then replaced by 50 ton antifer blocks. During the reconstruction works, it was observed that only four rock blocks and three tetrapods were out of the head’s area. It was also verified all the blocks sunk into its sandy foundation soil in an imbricated way. This pointed out the need of a reevaluation of the breakwater’s rupture mechanism.

4 Proposed monitoring of the breakwater

The afterward rupture mechanism analysis suggested the probable rupture cause was a partial liquefaction of the foundation sands resultant of the cyclic pore water pressure increase due to the high waves. This mechanism has been studied by some authors, like (Tsui and Helfrich, 1983), (de Groot, et al., 2006) or (Schlütter, et al., 1996). Given the fluidification of foundation soils is not in accordance with the considerations of the new design, it was then decided to monitor the breakwater as schematised in Figure 2. A profile of waterpressure cells is installed 5m in front of the rotation head, as well as an accoustic wave profiler. Measurements are registered every second from all devices, duly synchronized. In the breakwater core three water pressure cells are installed in two profiles, so the horizontal and vertical gradients can be analyzed; these devices are also registered every second.

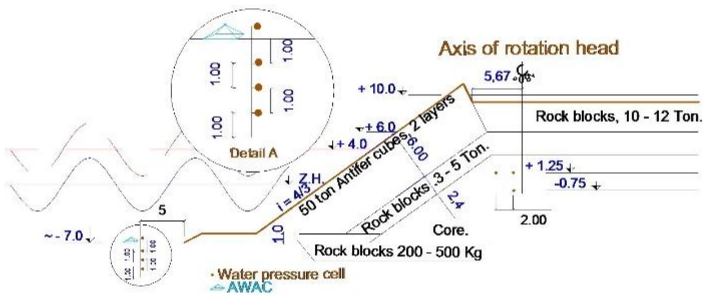

Figure 2: Instrumentation considered.

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Workshop 2: Harbour Geotechnics

Coastal geoscience mapping for harbour geotechnics: implications in maritime environments

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1 Coastal geoscience mapping: an overview

In general this work presents the role of the coastal geoengineering cluster and also develops an integrated coastal geoscience approach which is intrinsically linked to maritime environments. This study also shows the importance of a holistic perspective and methodological approach in urban coastal areas (e.g., Chaminé et al. 2016; Pires et al. 2016). One of the fields is the so-called coastal geoscience mapping for harbour geotechnics and the implications in maritime environments (e.g., Pires and Chaminé 2007; Santos-Ferreira et al. 2014; Pires et al. 2014, 2016; and references therein). Hence, it is essential to diagnose the geomaterials/blocks in situ concerning their degradation/deterioration level on the basis of the current status of the coastal protection structure (e.g., CIRIA et al. 2007; Pires et al. 2014, 2016; and references therein) in order to facilitate more efficient monitoring and maintenance, with economic benefits (Figure 1a to 1c). The coastal integrated system linked six stages allowing the production of detailed maps of the maritime environment: *(i)* high-resolution aerial imagery surveys; *(ii)* visual inspection and systematic monitoring; *(iii)* applied field datasheets; *(iv)* in situ evaluation; *(v)* field scanline surveys; and *(vi)* GIS mapping.

2 Coastal geoengineering features: implications in maritime environments

The model presented in Figure 1d shows the shoreline system and the littoral zone in an extremely simplified form. This is the ultimate and idealised model exposing different types of coastal contexts. This theoretical model embraces various factors, parameters, forcing conditions, several contexts and different areas of research. The shoreline system approaches all sorts of environments and forcing conditions, factors and even the constraints. It is possible to visualise rocky and sandy coasts, cliffs, shore platforms, coastal boulders and hydraulic structures (mixed environments) (for further details see: Pérez-Alberti et al. 2013; Pires et al. 2014, 2016). Moreover the rock quarrying source is represented, as well as the networking route (transport operation) from the extraction

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location to the construction site. Likewise it is possible to see the social perspective such as the fishing communities and activities. Being the shoreline, a dynamic system of moving sediment is involved, most of which is supplied by rivers; the river processes are also characterised here, bringing erosional debris from the continent and erosion of sea cliffs by wave action.

3 Insights about coastal geoscience mapping for harbour geotechnics

- GIS coastal mapping and modelling techniques reinforce geo-monitoring coastal plans. Moreover, the integrated approach applies different concepts to assess quality indicators for material armour layer and structure types.
- The coastal zone is a dynamic environment with a history of change and which will continue to change in the future. Consequently it was important to understand the importance of coastal systems and processes involved. In fact, the relationship between all the processes, elements and forcing conditions allowed the production of several thematic geoengineering maps, as well as a better understanding of the coastal morphodynamics.
- The application of regional coastal geoscience maps and local approaches outputs could help the government, local authorities and stakeholders to develop coastal management plans and to recommend strategies. With such data, it is possible to propose or recommend strategies for coastal and shoreline management based on several justifications in terms of social, economic, and environmental questions, or even provide a GIS-based planning support system reinforced by geo-cartographic decisions.
- The proposed integrated coastal geoengineering methodology is valid for any type of coast or maritime environment: i) GIS mapping encouraged an interdisciplinary framework and showed an innovative sequence of techniques, equipment and efficient approach to easily assessing maritime environments; ii) The strength lies in coupling GIS applications with photogrammetric techniques in order to create applied cartography and thematic maps. The output maps are used in hydraulic structures.
- Finally, all of these thematic areas are crucial to propose conceptual models and to shape the future of integrated coastal geoengineering management and harbour geotechnics field.

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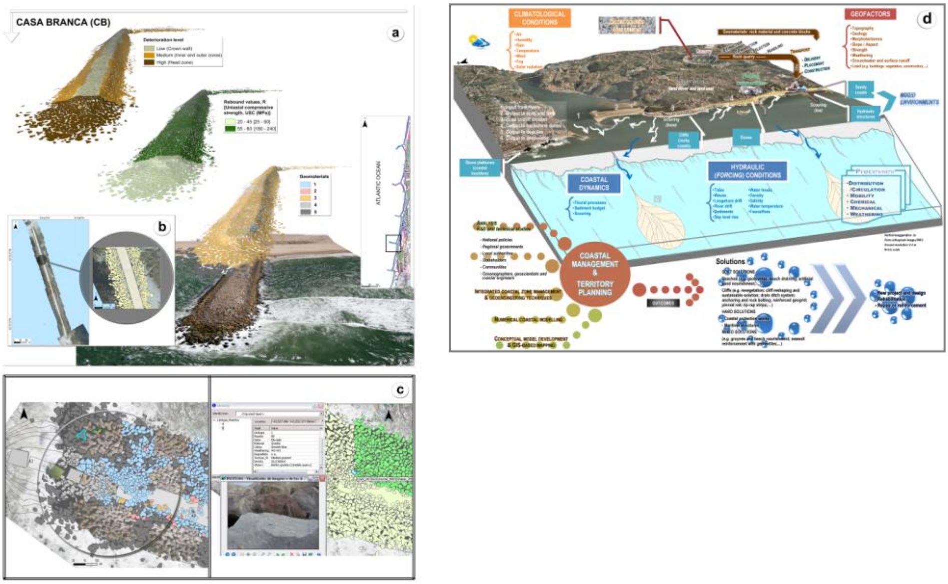

Figure 11: The coastal geoscience mapping and several applications in maritime environments: (a) to (c) Example of possible outputs in terms of the current status of the maritime structure (geomaterials/blocks characterisation or degradation/deterioration level evaluation (images adapted from Pires et al. 2016); (d) Holistic conceptual model proposal and the coastal geoengineering outlook.

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Workshop 2: Harbour Geotechnics

Seismic Design of Port Facilities in Japan

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1 Earthquake Damage and Progress in Seismic Design

Seismic behavior of port structures which have been damaged during past large earthquakes in Japan was clarified by attentive investigation and further research on those experiences. As analysis techniques to predict seismic behavior and damage have been examined on the basis of results of those, earthquake-resistant technology of the port structures have been developing with reflecting the research outcome to design method and the technical standards.
1964 Niigata earthquake made liquefaction phenomena of sand ground widely known; apartment buildings fell down with liquefaction of ground and movie of sand boiling was filmed at Niigata Airport which was seriously affected by the earthquake. After this earthquake, mechanism of liquefaction was widely studied and Liquefaction prediction method using particle size distribution and N value was established in 1970.
In 1968 Tokachi-oki earthquake, seismic ground motion records were observed in some ports by strong-motion earthquake observation in Japanese ports. Arranging the seismic motion records with collected damage information of port structures, relational expression of horizontal design seismic coefficient to maximum acceleration was accomplished for seismic coefficient method of quay wall structures in 1975. In addition, the seismic motion record of larger than 200Gal in amplitude was observed in Hachinohe Port during the earthquake, and it has been utilized in many researches and actual structure designs.
Significant damage of many kinds of structures during 1995 Hyogoken-nanbu earthquake greatly affected subsequent seismic design of those. Performance-based design taking into account seismic deformation of facilities was introduced to design standards of port structures since it is difficult not to allow any deformation for all facilities under huge seismic excitation such as the Hyogoken-nanbu earthquake. Therefore, in design of high-earthquake resistance quay walls as important facilities, it was decided to employ numerical analysis by effective stress finite element method to handle liquefaction phenomena of ground. Assuming two levels of ground motions, acceleration time history of the Level 2 ground motion have been utilized in design of high-earthquake resistance wharfs. Further study has been conducted on evaluation of seismic ground motion and it is achieved to introduce seismic ground motion considering source, propagation path, and site amplification in revision of the technical standards in 2007.

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2 Current Seismic Design of Quay Walls

2.1 Design Ground Motions

In General, strong ground motions are determined by three effects, namely, the source effect, the path effect and the site effect. The source effect is defined as the effect of the rupture process of the earthquake. The path effect is defined as the effect of the materials along the propagation path from the source to the bedrock beneath the site. The site effect is defined as the effect of sediments below the site down to the bedrock. The bedrock is defined as a layer with a shear wave velocity over 3000 m/s in this design standard (In many cases it corresponds to fresh granite in Japan). As shown in Figure 1, existence of sediments below the site has significant effects on the amplitude, the frequency content and the duration of strong ground motions.

The Level 1 ground motion is defined as a ground motion with high probability of occurrence at the site during the design working life and is determined appropriately as a stochastic time history based on the results of earthquake observation. The Level 2 ground motion is defined as the largest ground motion among ground motions at the site from scenario earthquakes and is determined appropriately as a time history based on results of earthquake observation and source parameters of the scenario earthquake.

Figure 1: The topography around the Port of Sakai, west Japan (left) and the velocity waveforms for the fault-normal component recorded around the port during the 2000 Tottori-ken Seibu earthquake (M_J7.3).

2.2 Earthquake Resistant Design of Quay Walls

For the Level 1 ground motion, ordinary functions of quay walls shall be maintained. For gravity type quay walls, three failure modes, namely, the sliding of the quay wall, the overturning of the quay wall and the lack of bearing capacity of the underlying ground, should not occur against seismic coefficient of the level 1 ground motion, comparing loads including seismic load to resistance for each mode.

Performance verification of gravity type quay walls for the L1 ground motion is carried out with a pseudo static approach. The procedure for evaluating the seismic coefficient can be described as Figure 2. The seismic coefficient is evaluated considering seismic response properties of the ground and the structure, duration time of ground motion, and allowable horizontal residual displacement.

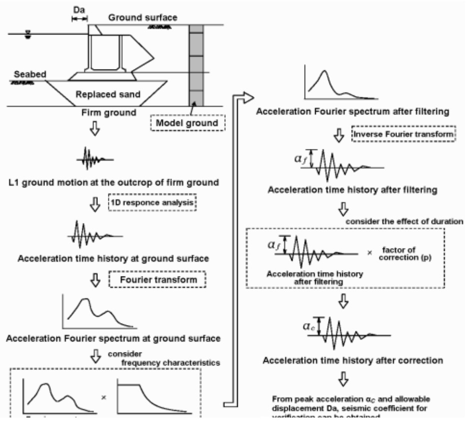

Figure 2: Procedure for the evaluation of seismic coefficient

In the case of a high seismic resistant quay wall, only a slight restoration work is allowable after the L2 earthquake. The performance of seismic-resistant quay walls of gravity type for the L2 ground motion is most typically evaluated with a two-dimensional effective-stress finite element analysis. In the performance verification of port facilities, the decrease of effective stress in the ground due to the excess pore water pressure is not negligible in cases of L2 earthquakes. The decrease of effective stress leads to a change in the stress-strain relation and the damping characteristics of the soil. To evaluate the residual deformation of a quay wall, it is necessary to employ a nonlinear constitutive model for stress-strain relation of soil.

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Workshop 2: Harbour Geotechnics

Punta Langosteira Harbour– A Coruña (Spain)

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1 Introduction

During the last two decades, the Port Authority of A Coruña has developed an enormous work of design and construction to expand its facilities and develop a new basin. This port is located in the north coast of Spain, exposed to the harshest design conditions, Hs[m]=15.1 and Tp[yr]=140.

The new outer harbour of A Coruña is the result of an exhaustive multidisciplinary project in order to solve the problems existing in the current port of A Coruña, which can be summarized as depletion of space, with no capacity of enlargement and the physical and environmental risks associated with the handling and storage of certain dry and liquid bulks, exacerbated by the proximity to the city.

The works were awarded to the joint venture composed of DRAGADOS, SATO, COPASA and FPS, and began on March 11, 2005 with a term of 78 months, so its completion was originally planned for September 11, 2011. Finally and after the granting of an extension due to the delays caused by two storms (Becky and Quirin in November 2010 and February 2011), the works were completed on 28 December 2011. The Port sheltering was completed during the years 2013 and 2014 by the Port Authority of A Coruña, developing the design of the secondary breakwater of the port. The works are under construction, with the completion planned in 2016.

2 Description of the project

This ambitious project involved the construction of a rubble-mound breakwater as the most significant element. The breakwater is approximately 3,400 m long, with variable depth in its development, reaching 40 m in some sections, protected with blocks of 150 t, and crowned with a shoulder elevation of +25 m and is topped with a sloping nose composed of high-density blocks of 178 and 195 t.

The work is completed with a spur breakwater of 390 m length, which will on its inner side also serve as moorings for oil tankers and a sloping breakwater 215 m long. The new port design is completed by a dock for solid bulk cargo, 900 m long and 22 m depth.

The basin thus formed will have an area of 230 ha of sheltered water and have 150 ha of wharves, of which 91 will be reclaimed land.

The new port is operating since summer 2012 and has reached a movement of more than 700,000 t in 2015.

Since that date various works are still taking place, emphasizing the ratings and the construction of warehouses by contractors:

Secondary Breakwater: 1,300 m long, this breakwater in slope protected on the outside by a monolayer of cubipods 25t on trunk and 30t in jog and a double layer of 45t cubipods at the head breakwater.

Gallery for pipes: it consists of a structure of reinforced concrete spandrel on two parallel alignments of pillars that emerge from a foundation based on way of the breakwater, in order to protect connecting pipelines from the berths of liquid bulks along the breakwater of new installations.

3 Project of auscultation and monitoring of breakwater

The Port Authority of A Coruña has been performing a number of periodic measurements on a leveling established net in order to monitor the possible movements of the main structures that make up the new port.

In the specific case of the breakwater, it is worth stressing the movements detected after strong storms of waves that occurred during January and February 2014, reaching record seats and movements of up to 18 cm.

The studies conducted on this issue indicate that the observed movements of the crown wall are only explained by the high expected deformability of a breakwater's body of these magnitudes, although it cannot be ruled out that it has been slightly influenced by any eventual reduction of a coefficient of global security in situations of rough weather or a possible migration of fines.

In order to refine the preliminary results obtained, a campaign has been designed with the aim of characterizing the real geotechnical parameters of the breakwater and the loads produced by incident waves and overrides on structures located at the coronation of the same; by monitoring the behavior of the reactions of the breakwater about actions that will be submitted in real working conditions, which will constitute an essay to scale 1:1.

The final objective of the research will allow to model the real conditions of work of breakwater, to predict its behaviour, to assess real safety coefficients of the current breakwater and its evolution to the useful lifelong of the structure.

4 Acknowledgements

It is only fair to add that EPTISA and ICEACSA companies have collaborated with APAC, both in design and control of execution of the works at Punta Langosteira.

Figure 12: New outer harbour, October 2010

Workshop 2: Harbour Geotechnics

Stability of submerged clay masses. A case study in a port.

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1 Introduction

The study of the stability of submerged clayey mass is one of the concerns of marine geotechnical engineering.

The case study analyzed in this work presents the rupture of the pier's platform of "Porto de Leixões", –Dock 2, North Pier Wall - verified in 1975 over 300m. The main purpose of this study is to analyze its causes according with the results of the boreholes and laboratorial and in situ tests carried out, as well as, the different methodologies and resources used in the first study (1989) and in the work currently developed.

2 Developed Work

Some natural sedimentary deposits present enough deformation to, in case of unfavorable inclination of the rigid inferior support, suffered displacements due to the existence of yield stresses in the contact of the firm rock stratum. In the analysis developed in 1989, it was studied the cinematic method (ex: Bishop Method), which depends, mainly, of the slope's geometry and the materials delimited by the slip curve chosen. However, this method is not sensible to decompression situations of the backfill (Figure 13). In fact, these methods could conduct to incorrect evaluations of the slope's safety since the existence of an inclined surface of the rock stratum is rather unfavorable to the slope's stability and this situation isn't detected by the cinematic methods (Figure 14). Thus, the Bishop's method was used, in the first place, to quantify sliding safety for a circular sliding curve and the Nonveiller's method was used to determine an arbitrary sliding curve.

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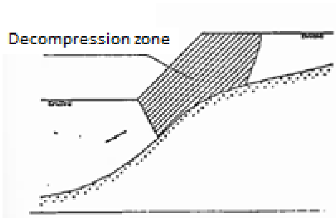


Figure 13: Decompression scheme destabilization

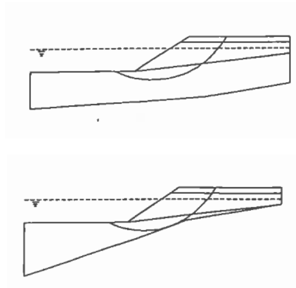


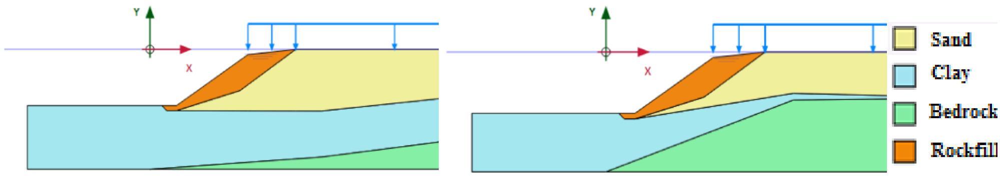
Figure 14: Low influenced of rock stratum in the safety coefficient applying the Bishop's method

The finite element method was also studied to obtain a good approximation of real stress field following a certain surface. In this case, the chosen curves did not correspond necessarily to minimum safety coefficient. In fact, according with the slip curve defined with the Bishop's method, the coefficient factor was calculated applying the finite elements method. The values obtained presented significant differences and were influenced by the firm stratum dip.

The main purpose of the work currently developed is compare the results obtained of the slope's stability, considering the use of commercial *software* – Slide and Plaxis. The Slide *software* is based on the cinematic method and, although determine expeditiously the minimum safety factor (and the respective failure surface) applying different methods (ex. Bishop, Janbu, Spencer, etc.), does not take into to account the elastic-plastic displacements. By the other side, the *software* Plaxis calculates the safety factor applying *the phi-c reduction* method and depends, besides the strength parameters, of the deformability parameters. In fact, this method reduces the strength parameters (cohesion and friction angle), and the safety factor (ΣM_{sf}) is determined considering the relation between the $\tan \phi'_{input}$ and $\tan \phi'_{reduced}$, which is equal to the relation between c'_{input} and $c'_{reduced}$.

The value and the slip curve obtained should be compared with results presented by the Slide *software*. Throughout the study, two sections (one is representative of the accident that occurred in 1975 - S5 - and the other present a different geometry and different strength parameters - S3) were defined in order to determine the minimum safety coefficient (Figure 15). Thus, the minimum value of the undrained shear strength that could lead to the rupture (equal to 24kPa) and a relation between the safety factor and the undrained shear strength was also established.

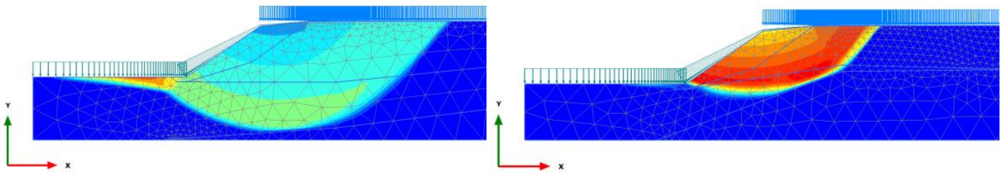
Workshop 2: Harbour Geotechnics



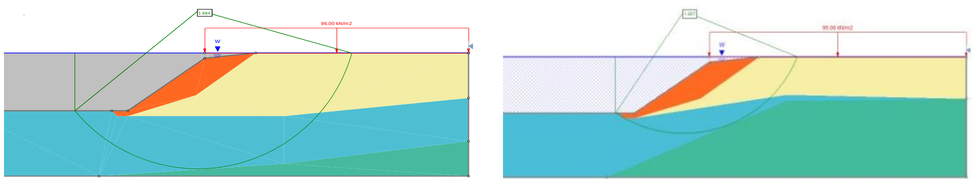
Section S3 Section S5

Figure 15: Geometric differences between the sections S3 an S5

The results obtained in the software Plaxis and Slide are presented in the followings figures:



SF_{Plaxis}=1.784 SF_{Plaxis}=1.545



SF_{Slide}=1.684 SF_{Slide}=1.207

Section S3 Section S5

Figure 16: Slip curve defined by Plaxis and Slide

3 Conclusions and Future work

The results obtained allow compare the values between the different modeling carried out in Plaxis and Slide and the conclusions obtained in the study developed in 1989. According with the results presented in this document, the safety factor is rather influenced by the thickness and the undrained shear behavior of the clay. The software Plaxis and Slide do not present exactly the same results but their values are not widely dispersed. The excavation of the clayed mass in the slope's face is currently studied, as well as, its influence in the safety factor in a long-term analysis.

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Workshop 2: Harbour Geotechnics

Author index