PORT ENGINEERING AND COASTAL STRUCTURES
A REVIEW OF THE INFLUENCES OF THE DREDGED MARINE SAND EXTRACTED FROM PERSIAN GULF (SHAHID RAJAI PORT) ON DURABILITY AND RESISTANCE PARAMETERS OF THE ROLLER COMPACTED CONCRETE PAVEMENT

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1. Introduction and Background

Overusing the natural resources leaves the future generations only with waste and pollutions [1]. In order to avoid such impacts serious efforts are required in various industries. Construction industry is a major counterparts of the World’s activities [2]. Although it is the main source of income for about 30% of population [3], it plays a significant role in the environmental issues, such as excavation and landscape deterioration, etc. As the most used material, concrete has a significant effect on the environment [4, 5, 6].

In a standard concrete about 0.75 % of the mix is filled with sand and gravel. Due to the growing increase of usage, the extraction of natural resources and mines will cause considerable impacts. Nevertheless, in some areas (e.g. islands and Persian Gulf marginal lands) these resources are rare, involving transportation cost and pollutions as well. Therefore, finding other sources of aggregates is very effective. From transportation point of view dredging the sediment from the coastal areas is a sensitive matter as it is conditional in order to provide passages with appropriate depth for loading or/unloading the ships. However, these deposed and left open dredges cause air and land pollution. Combining the latest with the former shows that using the dredging materials, as a replacement for the whole or a percentage of the aggregates is a perfect solution. Regarding various aspects related to this new source of materials several studies have contributed. Examples of which are being followed as: Limeira et al. (2012); Moradi et al. (2018); Etxeberria et al. (2016); Liu et al. (2016); [7, 8, 9,10].

2. Experiments and the Tests Setup

The experimental works are categorized in three main sets. First is the mix design and programming, in addition to the required tests on the ingredients and the basic materials. The second category is providing the specimens, (mixing, casting, curing etc.). The final is the mechanical and permeability tests on the specimens.

3. Outcomes of the Laboratory Tests

The experiments are excluded from this text but the final ones. As shown in figure 3. and 4. In the last stage of these tests, the data of each mixing plan (sulfate and non-sulfate) is the average of three 10*10*10 cm sample cubes. In the obtained graphs, the average compressive strength of sulfate samples has a slight difference with non-sulfate samples. In fact, in the middle (six months) and long-term (one year), despite the change in strength, there was no significant effect on the compressive strength of the test specimens. Also, in all three types of Portland cement, the mixing plan of the 15% dredged sand has the highest strength to the control concrete, and in special Pozzolona Portland cement and Portland cement type 5, the mixing plan containing 25% dredged sand has also shown more strength than the control concrete and after that, the strength such as the test of strength of the 15 * 15 * 15 cm specimens were on a downward path.
4. Discussion and Conclusions

In the present study, we have tried to due to lack of mines to extract the materials in addition to using existing materials in the region and also high costs and long-term non-performance of flexible asphaltic pavements compared with rigid pavements due to reduction of air pollution and environmental hazards of Sea dredged sand, use this material to produce a useful product. In this regard, the research results are as follows:

- According to the existing conditions, the replacement of 15% DMS instead of mine sand has the maximum strength and durability of test specimens in all three types of Portland cement.
- Adding higher percentages of DMS from 50 to 100 percent has disturbed the proportional distribution of fine and coarse aggregates in concrete, due to the uniformity of aggregates and the non-overlapping and interlocking between aggregate, lead to reducing the strength.
- Reducing trend of strength by using high percentages of up to 100% of DMS, although apparent, is a good idea if used in less-favored structures in remote areas of the islands and Persian Gulf ports, which have the potential to convey and supply mineral materials.
- The results of the sulfate test indicate that there is no effect on the compressive strength of the sulfate specimens in the medium (six months), and the test has only a slight weight loss of 10 grams, which can be due to the dissolution of some salt in the dredged sand in sulfate solution or human precision and measuring device.

5. References

INVESTIGATION OF THE PERFORMANCE OF GEOTEXTILES AS THE FILTER LAYER IN RUBBLE-MOUND BREAKWATERS

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1. Introduction

Today, using geotextile in engineering has become prevalent, owing to some of their features such as: durability, resistance, permeability and incorruptibility against destructive factors within soil. Whereof geotextiles work simultaneously as a water drainage and a soil conservator against being washed up, it is allowed to use them as a filtering material. Geotextiles are also very easy to carry, simple to use and reasonable in a matter of price [1].

The most common types of breakwaters are the rubble-mound breakwaters with a long lifespan which can moderate wave energy very well. For a better and more efficient use of these types of breakwaters, some important properties must be optimized, for example reducing the amount of pore water pressure in core layer [2].

Since pore water pressure is related to permeability, it can be reduced by a better replacement of filtering materials in breakwaters. To improve the efficiency of this process, the material of the filter layer can be changed. In this study, it has been tried to use a number of geotextiles as the filter layer to find out their effect on the reduction of the pore water pressure of the core layer of breakwaters.

2. The Replacement of the Granular Filter Layer by Geotextile

In this study, the cross sections of a constructed breakwater (located in Anzali free zone) are used and GeoStudio 2012 is employed for modeling. First, the main as-built section is modeled. Then, it is modeled by removing half of the thickness of filter layer and inserting a layer of geotextile instead, removing the entire filter layer and inserting one, three and five layers of geotextile in a row, respectively. Figure 1 shows the models.

At the end, by comparing the entire results, it is concluded that by using five layers of geotextile, the pore water pressure has been efficiently dropped. The geotextile which is used in this research is a type of GMH300 with a permeability of 0.0023 m/s and a thickness of 2.8 mm.

3. Calibration of the Coefficient in the Wave Height Equation in a Porous Layer

The theoretical wave height in each point of a breakwater (which is known as a porous layer) can be normally calculated by the following equation [4].
Where $H$ is the wave height in a porous layer, $H_0$ is the input wave height, $K$ is a coefficient which is assumed to be 2 for granular materials, $X$ is the distance from the start point, $D_{50}$ is the medium nominal diameter of grains and $d$ is the water depth.

At first step, to calculate the wave height in each point, $D_{50}$ for every layer of breakwater must be calculated by equations 2 to 5.

$$r = \frac{1}{2}D_{50}$$

$$V = \frac{4}{3}\pi r^3$$

$$\rho = \frac{V}{M}$$

$$D_{50} = \sqrt[3]{\frac{6M}{\pi\rho}}$$

Where $V$ is the nominal volume, $\rho$ is the density and $M$ is the mass of grains.

Finally, by inserting the data in equation 1, calculating the theoretical pressure heads and comparing them with the estimated pressure heads from GeoSudio, it is figured out that the differences between them are noticeable as it is shown in table 1.

<table>
<thead>
<tr>
<th>$H_0$</th>
<th>$H_t$</th>
<th>$\Delta$</th>
</tr>
</thead>
<tbody>
<tr>
<td>3.6066</td>
<td>11.49</td>
<td>7.8834</td>
</tr>
<tr>
<td>3</td>
<td>10.49</td>
<td>7.49</td>
</tr>
<tr>
<td>2.5812</td>
<td>8.0384</td>
<td>5.4572</td>
</tr>
<tr>
<td>1.4061</td>
<td>11.4607</td>
<td>10.0546</td>
</tr>
<tr>
<td>2.4377</td>
<td>6.6696</td>
<td>4.2319</td>
</tr>
<tr>
<td>2.3555</td>
<td>4.0569</td>
<td>1.7214</td>
</tr>
<tr>
<td>2.0665</td>
<td>1.1175</td>
<td>1.089</td>
</tr>
<tr>
<td>1.4549</td>
<td>-4.0073</td>
<td>5.4622</td>
</tr>
<tr>
<td>1.0024</td>
<td>-4.7467</td>
<td>5.7491</td>
</tr>
<tr>
<td>1.31</td>
<td>-3.0694</td>
<td>4.3794</td>
</tr>
<tr>
<td>1.2414</td>
<td>2.9611</td>
<td>1.7197</td>
</tr>
<tr>
<td>1.1788</td>
<td>4.2857</td>
<td>3.1609</td>
</tr>
<tr>
<td>0.7683</td>
<td>7.0011</td>
<td>6.2328</td>
</tr>
</tbody>
</table>

Since the differences are extremely considerable, the equation 1 has a major deficiency which might be solved by calibrating the coefficient. To overcome the problem, the equation 1 should be used recessively. Figure 2 shows the $K$ fluctuations.

Therefore, the $K$ coefficient is calibrated around 0.25. In such case we can expect to have a better result.

4. Results and Discussion

In this paper the pore water pressure parameter has been studied and it is comprehended that changing the material of the filter layer helps reducing the amount of pore water pressure in the core layer. This purpose can be successfully achieved by using five layers of geotextile (GMH300) instead of traditional filter layers.

The coefficient in the wave height equation was not good-defined to overcome the differences between the theoretical answers and the estimated ones. This study has reached a new coefficient by comparing the answers and using the equation 1 recessively. Considering the boundary conditions, the coefficient is calibrated around 0.25. Thus, the conclusions are satisfying up to here.

5. References


IDENTIFICATION AND RANKING OF THE CLAIM CAUSES IN THE CONSTRUCTION PROJECTS OF BUSHEHR PORTS AND MARINE ORGANIZATION

Seyyed Abdossalam Hosseini and Nahmat Khodaie

1. Introduction

Generally, a claim is defined as the seeking of change by one of the parties involved in the construction process [1]. Also, claims are described as the assertion of the right to money, property or remedy [2]. Construction claims are considered by many project participants to be one of the most disruptive and unpleasant events of a project [3]. Once a claim has been presented, the owner and contractor can come to an agreement concerning the claim and, thereby, create a change order or a modification, or they may disagree and create a construction contract dispute. Analyzing the various types and causes of claims is an important task to resolving these claims [4, 5, and 6]. Claims may result in cost overruns, schedule delays and may jeopardize the working relationships amongst the contracting parties [7]. Due to conflicts and differences over claims, the construction industry is plagued by an adversarial atmosphere between clients and contractors [8]. The claim management process in the construction industry has to be clear and be understood by all parties especially the contractor in order to know how to manage them [9].

Several attempts were made in the literature to study the types of construction claims and determine their main causes and ways of avoiding them. Al-Khalil and Al-Ghafly determined the most important causes of delay claims in public utility projects in Saudi Arabia based on the frequency and severity of these causes [10]. Scott conducted a survey to investigate the causes and mechanisms that are used to prepare and evaluate delay claims in United Kingdom [11].

2. Research methodology

The first step in this study was to identify the causes for claims in construction projects of Bushehr port and marine organization. The primary list, containing 142 reasons of the claims, was produced using the ideas of the invited representatives of client, contractors and consultants, in a brainstorming meeting. The identified reasons were ranked based on the weighted average of the responses collected at the meeting. A questionnaire was then developed containing the first 40 root causes. The questionnaires were sent to the construction professionals, who frequently deal with the studied construction projects. The responded questionnaires were analyzed using SPSS software, and the identified main causes of claims were ranked according to their relative importance. Finally, new recommendations are suggested to minimize claims and improve the performance of the construction projects.

<table>
<thead>
<tr>
<th>Index</th>
<th>Cause Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>X1</td>
<td>Lack of documentation at the right time</td>
</tr>
<tr>
<td>X2</td>
<td>Change orders during construction issued by the client</td>
</tr>
<tr>
<td>X3</td>
<td>Price Fluctuation and inflation</td>
</tr>
<tr>
<td>X4</td>
<td>Not to use the consultant’s expert price</td>
</tr>
<tr>
<td>X5</td>
<td>Client tendency to the lowest suggested price</td>
</tr>
<tr>
<td>X6</td>
<td>Increase of the volume of works more than the limits of general conditions of contracts</td>
</tr>
<tr>
<td>X7</td>
<td>Lack of specific list price for repair works</td>
</tr>
<tr>
<td>X8</td>
<td>Nonexistence of claim management methodologies</td>
</tr>
<tr>
<td>X9</td>
<td>Variations in the work qualities</td>
</tr>
<tr>
<td>X10</td>
<td>Mistakes in the project time schedule</td>
</tr>
<tr>
<td>X11</td>
<td>Time-limits on preparing tender documents</td>
</tr>
<tr>
<td>X12</td>
<td>Insufficient knowledge about law and circulars</td>
</tr>
<tr>
<td>X13</td>
<td>Disagreement over price of the new works</td>
</tr>
<tr>
<td>X14</td>
<td>Lack of Funding proportional to cash flow</td>
</tr>
<tr>
<td>X15</td>
<td>Not to follow the required steps to confirm the project estimation</td>
</tr>
<tr>
<td>X16</td>
<td>Quantitative variations in drawings and materials</td>
</tr>
<tr>
<td>X17</td>
<td>Insufficient information on relationship management</td>
</tr>
<tr>
<td>X18</td>
<td>Inaccuracy of the project study and design phases</td>
</tr>
<tr>
<td>X19</td>
<td>Lack of timely decisions on new prices</td>
</tr>
<tr>
<td>X20</td>
<td>Increase of the project cost more than the limit of 25 of the primary cost</td>
</tr>
<tr>
<td>X21</td>
<td>Mistakes in project estimation</td>
</tr>
<tr>
<td>X22</td>
<td>Change in client’s needs and decisions</td>
</tr>
<tr>
<td>X23</td>
<td>Payment delays</td>
</tr>
<tr>
<td>X24</td>
<td>Requiring the additional works to be done based on the basic price list</td>
</tr>
<tr>
<td>X25</td>
<td>Requiring the additional works to be done based on the basic price list</td>
</tr>
<tr>
<td>X26</td>
<td>Project extension due to the delays</td>
</tr>
<tr>
<td>X27</td>
<td>Change in roles of client authorities</td>
</tr>
</tbody>
</table>

Table 1. Identified causes presented in the questionnaire
Lack of claim management knowledge of contractors
Changes relative to the primary conditions
Entrance of inspection organization in tender process
Improprity of existing contracts for repair works
Orders of provincial and local governments
Client nonobligatory to the contract contents
Inaccuracy in examination of contractors documents during bidding
Poor consideration of site condition and ignoring bidders comments on this subject
Changes in client view points during construction phase
Lack of specific price list for marine structures
Unacceptable delays at project completion stage
Weakness of general conditions of contracts to analyze contractor delays

3. Results

In the designed questionnaire, each cause of claim is evaluated from three viewpoints: occurrence possibility, time effect and cost effect. For every condition of cause and the stated viewpoint, five probable levels of importance are defined as follow: 1) very low, 2) low, 3) medium, 4) high, 5) very high. Table 1 presents the identified causes presented in the questionnaire which are obtained with narrowing the primary list produced in the brainstorming meeting. Figure 1 shows the average importance level of the causes for the three indexes of occurrence factor, time and cost effects based on the analysis of the completed questionnaires. According to the ranking results, the following causes are respectively ranked as the first five important causes of claims in the construction projects of Bushehr ports and marine organization:

(X25) Project extension due to the delays, (X39) Unacceptable delays at project completion stage, (X5) Client tendency to the lowest suggested price, (X2) Change orders during construction issued by the client and (X34) Inaccuracy in examination of contractors’ documents during bidding.

Figure 1. The average importance level of the causes for the three indexes of occurrence factor, time and cost effects

4. References

PLANNING FOR SPACE & SPATIAL DEVELOPMENT OF PORTS
CASE STUDY: ANZALI PORT

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2) Faculty of Navigation and Marine Engineering, Chabahar Maritime University, Chabahar, Iran, narimanmehrgans@gmail.com

1. Introduction

Threats come from sea shipping to ports, the process of moving towards larger ships, ship specialization and ultimately, the threat of community-based ports. A well-known theory that describes how to progress the port infrastructure in terms of time and place is the Anyport model. This model Pattern the port development process, indicates Which side will they go to?

Then Rodrigue and Notteboom (2005) added Development-Regionalization to these steps. In this research, the process of growth and development of Anzali Port clause with Anyport model is examined. In the next step, the analysis of the growth and progress of this port from the past to the present has been done according to the Anyport model And then it refers to the differences and similarities that this port has been developing with the model. Then there are two types of development, concentrated and de concentrate will be examined. Considering port constraints based on the Anyport model, a framework for decision making is proposed with consideration of the basic constraints. These constraints include Geographical, economic and supportive. The decision framework for Location development of the port includes port planning, Location, study and decision making. In this research, Bandar Anzali has been studied and in the end it becomes clear that this port needs a centralized or decentralized development.

2. Anyport Model

The Anyport model is a well-known theory that developed by Bird in 1963. This Model is an initial attempt to categorize port development, which is still used as a reference. The steps Bird used for his model include: primitive, marginal quay extension, elaboration dock, marginal quay elaboration, simple lineal quayage and specialized quayage. Bird also recognized the various parts of the port that could be some writer believe that the six phases proposed by Bird can be categorized into three broad categories: Setting, Expansion, Specialization (Rodrique et al, 2009).

Three major steps can be identified in the port development process identified by ANYPOR:

2.1. Setting

The initial setting of a port is strongly dependent on geographical considerations. A standard evolution of a port starts from the original port, most of the time a fishing port with trading and shipbuilding activities, which includes several quays (1). For many centuries until the industrial revolution, ports remained rather rudimentary in terms of their terminal facilities. Port-related activities were mainly focused on warehousing and wholesaling, located on sites directly adjacent to the port. The port district was a key element of urban centrality.

2.2. Expansion

The industrial revolution triggered several changes that impacted on port activities. Quays were expanded and jetties were constructed to handle the growing amounts of freight and passengers as well as larger ships (2). As the size of ships expanded, shipbuilding became an activity that required the construction of docks (3). Further, the integration of rail lines with port terminals enabled access to vast hinterlands with a proportional growth in maritime traffic. Port-related activities also expanded to include industrial activities. This expansion mainly occurred downstream towards deeper draft areas.

2.3. Specialization

The next phase involved the construction of specialized piers to handle freight such as containers, ores, grain, petroleum and coal (4), which expanded warehousing needs significantly. Larger high-capacity ships often required dredging or the construction of long jetties granting access to greater depths. This evolution implied for several ports a migration of their activities away from their original setting and an increase of their handling capacities. In turn, original port sites, commonly located adjacent to downtown areas, became obsolete and were abandoned (Rimmer, 1967).

Figure 1. Development process according to Anyport Model
3. Location Development of the Port

Seaport development is divided into two categories:

- Concentrated: That is the development and expansion of the port at original site or near the place
- De concentrate: Here the development and expansion of the port moves towards a new position, while the original site of port is preserved (Chan et al., 2010).

Four stages of decision-making have been taken to help develop the port’s space:

4. ToC Analysis for Port Development

In this study, following the conformity of Anzali port with the stages of the Anyport model, Theory of Constraints has been used to determine the decision framework. In this study TOC has been used to determine the decision framework and whether port development should be separate from the original site or not.

Therefore, it is necessary to first identify the effective constraints on port development decision-making, as shown in Table 1 of these constraints.

5. Results

By combining the idea of the anyport model and TOC, a port development space was formed. It was shown to be a concentrate development for Anzali port to achieve more goals is more suitable in terms of geographical, supportive, economic constraints.

The Anyport model examines the port infrastructure’s evolution in terms of time and position. It can also be considered as the basis for comparing the true development of ports. The process of formation and expansion of the Anzali port was the same in any of the three stages of the Anyport model but because of the weakness in hinterland infrastructures, such as railways, it makes it difficult to enter the port of Anzali to the regional development stage (regionalization).

![Figure 2. Framework of decision for location development of the port (Chan et al., 2010)](image)

### Table 1. Development Constraints of the port’s spatial

<table>
<thead>
<tr>
<th>Elements</th>
<th>Secondary Constraints</th>
<th>Main Constraints</th>
</tr>
</thead>
<tbody>
<tr>
<td>Earthquake Areas</td>
<td>Shore</td>
<td>Geographical Constraints</td>
</tr>
<tr>
<td>Ability To Progress</td>
<td>Air</td>
<td></td>
</tr>
<tr>
<td>Inappropriate Weather</td>
<td>Sea</td>
<td></td>
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<tr>
<td>Depth Of Water</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Hinterland</td>
<td>Freight Resources</td>
<td>Economical Constraints</td>
</tr>
<tr>
<td>Cargo Transfer</td>
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</tr>
<tr>
<td>Tendency</td>
<td>Volume Of Freight</td>
<td></td>
</tr>
<tr>
<td>Transportation</td>
<td>Supportive Options</td>
<td>Supportive Constraints</td>
</tr>
<tr>
<td>Human Resources</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Environmental Effects</td>
<td>Green Production</td>
<td></td>
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<tr>
<td>Control Power</td>
<td>Government Planning</td>
<td></td>
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<tr>
<td>Terminal Operators</td>
<td>Port Operations</td>
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</tr>
</tbody>
</table>

### Table 2. Spatial & space development of Anzali port

<table>
<thead>
<tr>
<th>Development Constraints of the Port’s Location</th>
<th>Concentrated Development</th>
<th>Objectives</th>
<th>Development Constraints of the Port’s Location</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 Allow Access To Large Vessels</td>
<td></td>
<td></td>
<td>Geographical Constraints</td>
</tr>
<tr>
<td>1 Enough Space To Build The Terminal</td>
<td></td>
<td></td>
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<tr>
<td>1 Lower Risk In Navigation And Port Operations</td>
<td></td>
<td></td>
<td>Economical Constraint</td>
</tr>
<tr>
<td>1 Proximity To Cargo Resources</td>
<td></td>
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<tr>
<td>1 Cargo Growth</td>
<td></td>
<td></td>
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<tr>
<td>1 Maket Access</td>
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<td></td>
<td>Supportive Constraints</td>
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<tr>
<td>1 Minimum Environmental Impact</td>
<td></td>
<td></td>
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<tr>
<td>2 Workforce For Daily Operations</td>
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<td></td>
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<tr>
<td>2 Flexibility In Decision Making</td>
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<td>11 17 Results</td>
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</tbody>
</table>

6. References

A NOVEL DESIGN OF CAISSON QUAY WALL BASED ON PILES AND INVESTIGATING THE SEISMIC PERFORMANCE USING FINITE DIFFERENCE TIME HISTORY ANALYSIS

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1. Introduction

Quay walls, which are an important part of maritime transportation system, function as a protective structure for the soil behind and all port facilities built on it, including cargo and passenger shipping equipment; Thus optimal and innovative design of these structures is a high-priority issue in coastal engineering and important to the national interest. Caissons are among the largest berthing structures which are used for the construction of quays, breakwaters, coastal walls, bridges, etc. caisson type quay wall, is one of the most widely used kinds of quays, specially due to its structural and economic advantages, including working as a breakwater, relatively easy and quick implementation, and also much less need for repair and maintenance compared to traditional pile-supported wharves which are highly vulnerable to corrosion, deformation of the piles due to chemical factors and dynamic loading. On the other hand, caisson walls demand a rigid and firm foundation to resist their significant weight and consequently cannot be built everywhere. The present study suggests a novel design in order to make it possible to use the advantages of caisson quay walls even in lands with inappropriate soils [1-2].

Figure 1. Caisson quay wall on loose foundation.

2. Damage of Caisson-type Quay Walls

After the 1995 Kobe earthquake, Japan PHRI\(^1\) conducted an extensive survey on the seismic performance of 24 ports. It had been observed that the peak ground accelerations (PGA) recorded at the Kobe port were 0.54 g and 0.45 g in the horizontal and vertical directions, respectively. The field investigation reports showed that the characteristic damage patterns of a caisson-type quay wall are large lateral movements, tilting, and settlements of caissons and ground movements of the backfill in the form of lateral movements and settlements of the apron.

3. Proposed Method

In the proposed method, which can be used both for construction of new quay walls and repair and rehabilitation of damaged pile-supported wharves, a number of piles would be driven almost at the seabed elevation and then prefabricated concrete caissons will be placed on them by proper procedures, and be filled with sand and gravels. In case of repairing the existing damaged pile-supported wharves, old piles may be cut at seabed elevation and used to build a new quay wall on. Gravity load of caissons are transferred to the lower layers this way and lateral loads caused by earthquakes would be better resisted. The caisson quay wall of Kobe has been selected as the basic model to simulate the proposed method on.

4. Seismic Analysis

The need to assess the seismic behavior of these structures is obvious considering the high risk of earthquake in Iran and most coastal sites. Results of dynamic analyses performed by FLAC software, was validated using reliable data and the role of piles in improving the seismic performance of the wharf is well demonstrated by comparing the seismic performance of the wall with piles and without them. Live and dead loads, soil pressure, and earthquake forces were determined and considered in the simulated model which was firstly designed statically and then analyzed dynamically with 5 real earthquake acceleration records in vertical and horizontal directions.

Figure 2. Simulated caisson and soil model in FLAC.

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\(^1\) Port and Harbor Research Institute
FLAC 2D is an explicit finite difference computational software for performing soil–structure interaction analyses under static and seismic loading conditions. The reason behind selecting this program is that it has various constitutive models for soil and structural elements, and thus, can be used for analyzing and simulating the behavior of structure on the ground or located in the soil. In addition, FLAC 2D offers the free-field boundary condition and damping properties for the seismic analysis [3-4].

Figure 3. One of the excreted earthquake records.

The main parameters that were utilized to verify the model and evaluate the proposed method were horizontal and vertical displacements, a sample of which is shown in Figure 4.

Figure 4. Horizontal displacement contour.

The soil beneath the caisson was replaced with inappropriate soft sand and the model was analyzed again. After modeling three piles below the caisson the model was investigated by performing seismic analyses(Figure 5).

Figure 5. Caisson quay wall placed on 3 piles.

It was observed that horizontal and vertical displacements of the caisson were reduced 53% and 61%, respectively with the aid of added piles.

5. Further Sensitivity Analyses

Some sensitivity analyses were also performed on various parameters such as diameter of the piles, shear modulus and internal friction angle of the soil, to evaluate their impact on structural responses and horizontal and vertical displacements to provide optimized design criteria for the proposed quay. Results of one of these sensitivity analyses can be seen in Figure 6.

Figure 6. Displacement- PGA diagram for different pile diameters.

6. Liquefaction Evaluation

Finn and Byrne model [5] has been used to carry out coupled dynamic groundwater flow calculations to investigate the effects of seismically induced pore water pressures and estimate the degree and extent of liquefaction that can occur in the foundation and reclaimed soil during an earthquake excitation and ensure the seismic safety of the structures. The pore water pressure contour during earthquake can be observed in Figure 7.

Figure 7. Pour water pressure contour during earthquake.

7. References


CONSTRUCTION OF BREAKWATER BY GEOTEXTILE SANDBAGS: TECHNICAL SPECIFICATIONS

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1. Introduction

Although rock is the most popular material for construction of the coastal structures, such as breakwaters and various types of shore protections, there are difficulties in providing appropriate rock materials in some areas based on the economic and environmental conditions. In these cases, substitute materials such as concrete blocks and sandbags could be the alternatives of rock materials. In the plentitude of sand, for example when a project includes dredging, the sandbag is one of the best options from economic and environmental viewpoints.

Due to development of various types of the geotextiles, these materials have become one of the best selections to be used in making sandbags. In such cases, determination of the type of geotextile, technical specification of it and dimensions of the sandbags are very important.

Specification of the final product mostly depends on the bag size and texture quality. In this study some experimental tests have been designed and conducted to examine effective parameters. On this basis some recommendations have been proposed for the geotextile properties and size of sandbags for using in breakwater and shore protection structures.

2. Historical Usage of Sandbags

Geotextile Sandbags have been considered and used in different branches of civil engineering. In the coastal engineering, sandbags have been used mostly in construction shore protection structures. Building a temporary dam for blocking one of Pluimpot estuary’s channels in Netherlands in 1957 is one of the earliest usage of sandbags in coastal zone [1]. Clifton Springs port in Australia is an example of sandbag usage in breakwaters. In 2004, the second breakwater of this port comprised of 100 meters length and 2 meters height has been made of 2.5 m³ sandbags [2]. Sediment trap in Maroochydore-Queensland of Australia is an example of sandbag usage in shore protection structures [3]. Sandbags have been rarely used for construction of breakwaters. Thus, there is not specific recommendation for determination of their technical specification in breakwaters.

3. Recommended Specifications

Sewing, sand-filling, delivery and positioning the sandbags needs various equipment and facilities. Based on the available construction equipment and materials in Iran, it is recommended to use 1 m³ sandbag for breakwater core, which has had good results for one of the actual cases. To make this size of sandbag, a piece of 6.7 m² geotextile is needed. When the sandbags are not exposed to sunshine radiation, Polypropylene geotextile is better than Polyester because it better keeps its strength in water. As it will be explained in full paper non-woven geotextiles have better behavior during construction rather than woven ones. However, the woven geotextile must be considered for suitable positions too because it is cheaper than non-woven with the same strength.

The bags are under several forces in different phases such as filling, lifting, dumping and load due to the traffic of trucks on the core. After construction obviously the main force is just the weight of the upper sandbags and surcharges and of course the waves’ load.

Based on the theoretical computations and historical studies, the technical specifications of the recommended geotextile for 1 m³ sandbag is described in Table 1.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Type</td>
<td>Non-woven or Woven</td>
<td>Non-woven is recommended</td>
</tr>
<tr>
<td>Material</td>
<td>Polypropylene</td>
<td>-</td>
</tr>
<tr>
<td>Mass (gr/m²)</td>
<td>600</td>
<td>ASTM D5261</td>
</tr>
<tr>
<td>Tensile Strength (KN/m)</td>
<td>30</td>
<td>ASTM D4595</td>
</tr>
<tr>
<td>Punching Strength (N)</td>
<td>4000</td>
<td>ASTM D6241</td>
</tr>
<tr>
<td>Sewing Strength (KN/m)</td>
<td>24</td>
<td>ASTM D4595</td>
</tr>
</tbody>
</table>

Sandbags often are sewed and filled at the beach. During loading, unloading and installation, it is necessary to keep sandbags safe. Using inappropriate equipment may
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There should not be sharp materials such as shell pieces and woods in the sand. Generally, any touch by sharp objects should be avoided in the site. In case of using dump trucks for sandbags transportation, it is necessary to remove or cover any sharp edge of trucks and other instruments. The dumping bed can be covered by appropriate type of geo-textile. In case of using excavators, it is necessary to use proper hoe to avoid damaging of the sandbags (Figure 1).

4. Experimental Tests and Results

Different sizes and different types of geotextile were tested. Figure 2 shows some of tested sandbags. Evaluation of the size, specification of the used geotextile and quality of stitches were the main issues in these tests.

There are some parameters for optimizing the size of bags including price, operational equipment and speed of operation. Bigger bags contain more sands resulting in heavier bags that need stronger geo-textiles. In case of using inappropriate geotextile, it is most likely to face tearing of bags during drop or even transportation. Figure 3 shows an example of inappropriate used geotextile related to the size of the bag. Cost of bags strongly depends on the quality of the used geotextile. It worth to mention that small bags also cannot be good economic choice because of more needed amount of geotextile per 1 m³ of sand.

Considering all effective parameters, 1 m³ bag was recommended in this work as mentioned before. The in-situ tests consisted of a series of drop tests, tension and drag tests. Figure 4 depicts how a dropping test was conducted. In this test every bag has been dropped several times starting from 2 meters up to a maximum of 7 meters. Some of bags failed in drop tests. The most effective reason of drop failure was inappropriate texture

5. Conclusion

In this study, technical specifications of sandbags for using in construction of breakwaters were presented. 1 m³ sandbags with Polypropylene geotextile was found to be the appropriate selection. Some strength parameters were also proposed based on the theoretical and experimental findings. The designed sandbags were also rechecked by some in-situ tests that were in agreement with the presented criteria.

6. References

EXPERIMENTAL INVESTIGATION OF THE EFFECT OF HYDRAULIC PARAMETERS ON THE WAVE REFLECTION FROM THE ICELANDIC BERM BREAKWATER

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1. Introduction

In terms of design and operation, the berm breakwaters are divided into two types of homogeneous and multi-layered berm breakwaters. In the homogeneous berm breakwater, the armor layer consists of only one grading, but in the multi layer berm breakwater, the armor layer consists of several stone classes. A multi berm breakwater (IBB) is a statically stable breakwater that only a small amount of reshape is permitted, also stones are divided into classes depending on their sizes. The philosophy of choosing different classes of stone is the best usage of dedicated quarry. The largest class of rock is called class I is located in front of the berm and in some cases in the upper slope of the berm which has the greatest impact wave. Also Smaller stone classes are placed in the lower parts of the slope and inner layers where the effective of the wave is less.

Hydraulic parameters are one of the most important factors in choosing the type and geometric characteristics of coastal structures. Among the hydraulic parameters, wave reflection is one of the most important parameters. Coastal structures can reflect part of the incident wave energy. The interaction of incident and reflection waves creates a turbulent environment in front of the structure and it makes difficult to navigate the vessels in this area. In addition, turbulent flows created around the structure can cause erosion and scour on the toe of the structure. The reflection of the wave is expressed by the reflection coefficient \( C_r \) as following:

\[
C_r = \frac{H_r}{H_i}
\]

Where, \( H_i \) is the incident wave height and \( H_r \) is the reflected wave height.

A number of researchers have worked on conventional rubble mound breakwater and proposed some experimental formulae for estimating reflection coefficient, e.g., Postma, Hughes, Muttray, Zanuttigh and Siguradarson [1-5]. However, the investigation of the wave reflection in IBB has been rarely done.

2. Experimental Set up and Structure Design

Experimental model have been carried out on an IBB in a wave flume at the hydraulic laboratory of Tarbiat Modares University. The wave flume is 16 m long, 1 m wide and 1 m deep. In all tests, an irregular waves with a JONSWAP spectrum were used with a peak enhancement factor \( \tau = 3.3 \). Along the flume, four wave gauges have been used to record water level fluctuations. Three wave gauges were installed in front of the breakwater to separate the incident and reflected waves by using Mansard and Funke method [6]. Figure 1 shows the initial cross section of the IBB that situated at the end of the flume. The material properties in different layers for all tests are as following: Nominal stone diameter \( D_{950} \) (m) for classes I, II and III is respectively (0.027, 0.02 and 0.012), stone gradation factor \( f_g \) for classes I, II and III is respectively (1.2, 1.5 and 1.5) and mass density for all classes is equal 2650 kg/m³. The range of dimensionless parameters covered in the present study is as following: stability number \( H_o = 1.3-2.3 \), stability index \( H_o T_o = 23.7-67.4 \), wave steepness \( S_o = 0.016-0.067 \) and breaker parameter \( \xi = 3.4-5.3 \). To minimize the scale effect, Van der Meer proposed that Reynolds number as following [7]:

\[
R_e = D_{50} \sqrt{gH_o / \nu} > (1-4) \times 10^4
\]

![Figure 1. Initial cross section of IBB.](image)

3. Results

In this research, the effects of environmental parameters such as wave height and wave period are investigated on the wave reflection from an IBB. The results are illustrated as dimensional and non-dimensional graphs. Finally, a relationship is proposed based on the non-dimensional parameter. It is noticeable that the IBB was designed in all tests as non-overtopped structure. The effect of wave height on wave reflection is shown in Fig. 2. According to Figure 2 wave height does not have a considerable effect on wave reflection. Muttray et al. express that the effect of wave breaking and the effect of permeability are approximately balanced. Therefore, the reflection coefficient is almost independent of the effect of wave height [3].
Figure 3 is shown the influence of wave period on the reflection coefficient. According to Fig. 3, it is observed that the wave period has a significant influence on the reflection coefficient. If the wave period with same wave height increases, the reflection coefficient will increase.

Figure 2. Influence of wave height on reflection coefficient.

Figure 3. Influence of wave period on reflection coefficient.

Most relationships to the wave reflection are defined as a function of the breaker parameter $\xi = \tan(\alpha) / \sqrt{S_o}$. Where $\xi =$ breaker parameter, $S_o =$ wave steepness and $\alpha =$ front slope angle. In the figure 4, the effect of wave steepness on wave reflection is investigated.

Figure 4. Influence of wave steepness on reflection coefficient.

As shown in Figure 4, with increasing wave steepness, wave reflection is reduced. But, there is a lot of scatter between data, which is mainly due to the dependence of the wave steepness to the wave height. Therefore, it should be used an approach independent of wave height in related to wave reflection. Muttray et al. is assumed that wave reflection is a function of $T^2 / d$ or dimensionless parameter $L_0 / d$. According to assuming Muttray et al., Figure 5 is plotted.

Figure 5. Dimensionless parameter $L_0/d$ versus reflection coefficient.

Now, the wave reflection for an IBB can be calculated according to the following formula:

$$ C_r = 7.6 \times (L_0 / d)^{0.44} $$

(3)

The square of the correlation factor between experimental and predicted formula (Eq. (3)) is about 92%, while the E error is about 5.2%. These statistical quantities represent very good compliance. The results of this study indicate that the wave steepness in the breaker parameter (due to wave height) is not a suitable parameter for the wave reflection coefficient analysis.

4. Acknowledgments

The authors wish to express their sincere thanks to Iran National Science Foundation (INSF), who contributed financially to the project.

5. References


USE MARINE DREDGED SAND FOR PORT AREA CONSTRUCTION: A CASE STUDY OF SHAHID RAJEE PORT, IRAN

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1. Introduction

Nowadays, mismanagement of dredged materials and its consequent accumulation at the shore results in both environmental and economic drawbacks. The annual worldwide estimations of dredging operations are between 15 and 20 billion tons [1]. Dredged marine sands (DMS) are generally very soft soils with low shear strength, usually treated as waste [2]. The sustainable development, environmental issues and shortage of the construction materials lead to finding a beneficial solution for reusing dredged material. Currently, one of the largest dredging projects in northern coast of Persian Gulf is the developing project of Shahid Rajaee port. In total, 11.5 million cubic meter of natural coastal sediment were carried out by cutter suction dredger, and disposed in foreshore area next to western part of the calmness basin.

The geocell-reinforcement technique is one of suggested method in terms of its soil confinement and increases the strength and stiffness property of the soft soil [3].

In this paper application of geocell reinforcement has been examined. Also, the effect of cover soil layers over geocell was studied, using sand and gravel. In order to evaluate the possibility of constructing low volume roads with marine sediments, plate loading tests were conducted.

2. Materials and Methods

2.1. Geocell

The geocell used in this study were made of heat-bonded non-woven (HBNW) polypropylene geotextiles with 21.3 kN/m Tensile strength. Single cells were 110 mm long, 100 mm wide and 100 mm height (Figure 1).

![Figure 1. Geometric characteristics of geocell](image)

2.2. Soils

Two types of soils were used in this study. The first type, which formed the higher volume of material used, was the sand derived from the dredging process of Shahid Rajaee port which has been used in different layers of the models. The second type of soil was well-graded gravel which was obtained from a mine near Bandar Abbas city and was used only in the cover layer. Particle size distributing of both materials is presented in Figure 2.

![Figure 2. Dredged sand and well-graded gravel grain size distribution](image)

2.3. Plate Load Test

In order to determine the bearing capacity of test backfills was used plate load test. Figure 3 illustrates the schematic of testing box.

![Figure 3. Plate load test setup](image)
Backfills were made by manually compacting the dredged sand, with tamper in 50 mm lifts up to 350 mm in reinforced cases and 450 mm in unreinforced cases. Then geocells placed and dredged sand filled with accuracy in cells. Finally, a 50 mm thick sand or gravel cover layer, placed and compacted. All lifts were compacted to 70% of relative density with 4% moisture content. The specifications of the four prepared backfills are shown in Table 1.

Table 1. Lab tests specifications

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Notation</th>
<th>Geocell height (mm)</th>
<th>50-mm cover layer type</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>UR-S</td>
<td>100</td>
<td>Dredged sand</td>
</tr>
<tr>
<td>2</td>
<td>GR-S</td>
<td>100</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>UR-W</td>
<td>100</td>
<td>Well graded soil</td>
</tr>
<tr>
<td>4</td>
<td>GR-W</td>
<td>100</td>
<td></td>
</tr>
</tbody>
</table>

Loading process included four stress levels (250, 500, 750 and 1000 kPa) consisting of 10 cycles each. A circular plate with a diameter of 150 mm was used in this study.

3. Results and Discussion

Figure 4 plots the total settlement at the end of each repeated stress level and Figure 5 plots the plastic settlement at each loading cycle.

For a better comparison of results, the maximum force, total and plastic settlement at failure and the number of loading cycles are summarized in Table 2.

Table 2. Results of PLT and performance ratings.

<table>
<thead>
<tr>
<th>Backfill name</th>
<th>UR-S</th>
<th>GR-S</th>
<th>UR-W</th>
<th>GR-W</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maximum stress (kPa)</td>
<td>416</td>
<td>725</td>
<td>520</td>
<td>960</td>
</tr>
<tr>
<td>Settlement at failure (mm)</td>
<td>4.6</td>
<td>9.0</td>
<td>15.5</td>
<td>14.9</td>
</tr>
<tr>
<td>Plastic settlement (mm)</td>
<td>3.5</td>
<td>7.0</td>
<td>12.5</td>
<td>12.0</td>
</tr>
<tr>
<td>Number of load cycles</td>
<td>10</td>
<td>20</td>
<td>20</td>
<td>30</td>
</tr>
<tr>
<td>Tolerable stress level</td>
<td>1</td>
<td>2</td>
<td>2</td>
<td>3</td>
</tr>
<tr>
<td>Bearing capacity ratio (BCR)</td>
<td>1.74</td>
<td>1.25</td>
<td>2.32</td>
<td></td>
</tr>
<tr>
<td>Performance rating</td>
<td>4</td>
<td>2</td>
<td>3</td>
<td>1</td>
</tr>
</tbody>
</table>

According to the results, only geocell reinforcement backfills can carry standard truck wheel load (550 kPa). Geocell can increase ultimate strength of backfills with a sand cover layer by 70% (from 416 kPa to 725 kPa) while in backfill with a gravel cover layer showed 80% increase (from 520 kPa to 960 kPa) in ultimate strength. The gravel cover layer in unreinforced backfills increases the ultimate strength by 25 percent (from 416 kPa to 520 kPa).

Reinforcement endured 10 more load cycles before failure compared to un-reinforcement backfill. Base on Table 2, bearing capacity ratio (BCR) increased up to 2.3 and have best when geocell reinforcement and gravel cover layer were used together. Using geocell as reinforcement for sand backfills, improved the stress-settlement behavior. Dredged sand can be used as backfill material for yards and access roads when reinforced with geocell and covered with a layer of well-graded gravel.

4. Acknowledgement

The authors gratefully acknowledge the support provided by Shahid Rajaee Port and Persian Gulf branch of Road, Housing & Urban Development Research Center (BHRC).

5. References


1. Introduction

Coastal structures are designed and constructed to protect coastal regions against storm waves and high water levels during storm surges. Reliable prediction of wave overtopping plays a significant role in the design and safety assessment of these structures. Excessive wave overtopping can harm people, damage vehicles, and damage properties on or close to the breakwaters. Commonly, the mean wave overtopping rate should be below an acceptable rate under the design conditions. Construction of a higher structure produces more protection than a lower one, but it would be costly and greatly impair the harbor aesthetics [1].

For the design, safety assessment and rehabilitation of coastal structures, reliable predictions of wave overtopping are required. Several design formulae exist for simplified types of dikes, rubble-mound breakwaters and vertical breakwaters [2]. The prediction of wave overtopping has been studied by various researchers like Franco et al., (1994) [3], EurOtop (2007) [4], Goda (2009) [5].

In this study we use the method of decision tree for developing the previous and predicting the wave overtopping rate for vertical breakwaters. The proposed model was trained and validated using the selected small scale data from the CLASH database [6]. The results were also compared to those of previous empirical formulae.

2. Decision Tree

A decision tree is a decision support tool that uses a tree-like graph or model of decisions and their possible consequences, including chance event outcomes, resource costs, and utility. It is one way to display an algorithm that only contains conditional control statements. Decision tree has two algorithms: MARS [8] and M5 [9,10]. The new writing for M5 algorithm presented by Wang and Witten, (1997) [11] that called M5’ algorithm. In this study M5’ algorithm is used.

3. Description of the Parameters Affecting the Overtopping Rate:

Overtopping discharge is mostly related to the wave run-up. When the rising water caused by incident waves reaches the crest of the structure and pass over it, overtopping occurs [7]. Several structures and hydraulic parameters affect the overtopping rate on vertical breakwaters such as crest freeboard ($R_c$), Significant wave height ($H_{m0}$, toe) and The mean period ($T_{m, toe}$).

4. CLASH Database:

The international CLASH project of the European Union (Crest Level Assessment of coastal Structures by full scale monitoring, neural network prediction and Hazard analysis on permissible wave overtopping, www.clash-eu.org) [12] focuses on the modelling and predictions of wave overtopping for a wide variety of coastal structures, both in prototype and laboratory conditions.

5. Modeling:

In this study M5’ model tree is used for wave overtopping modeling. First, Multiple models were built by CLASH datasets for vertical breakwaters that eventually, more accurate model selected. Then, the function of presented formula has been compared with previous with the help of the statistical charts and tables.

The formula is presented in this study by M5’ model tree:

\[
\frac{R}{H_s} \leq 1.092 ; \quad q = \frac{g \cdot H_s}{\sqrt{g \cdot H_s}} \quad \text{exp}(-0.0897 \frac{H}{H_s}) - (2.0977 \frac{R}{H_s}) - (13.9577S_{sa}) - 2.88
\]

and

\[
\frac{R}{H_s} > 1.092 ; \quad q = \frac{g \cdot H_s}{\sqrt{g \cdot H_s}} \quad \text{exp}(-1.2833 \frac{H}{H_s}) - (0.8947 \frac{R}{H_s}) - (23.8144S_{sa}) - 5.0644
\]

\[
S_{sa} = \frac{2\pi H_{sa}}{gT_s^2}
\]

The statistical indices such as $X_G$ (Geometric mean), $\delta_{G_0}$ (Geometric deviation), BIAS (Deviation index) and RMSE (root mean square error) have been used to evaluate the accuracy of the models.
Table 1. Calculated errors for estimated values by different models for vertical breakwaters.

<table>
<thead>
<tr>
<th>Error index</th>
<th>Franco</th>
<th>EurOtop</th>
<th>Goda</th>
<th>M5'</th>
</tr>
</thead>
<tbody>
<tr>
<td>XG</td>
<td>3.627</td>
<td>3.23</td>
<td>3.83</td>
<td>2.82</td>
</tr>
<tr>
<td>BIAS</td>
<td>15.9</td>
<td>13.46</td>
<td>15.27</td>
<td>11.38</td>
</tr>
<tr>
<td>RMSE</td>
<td>0.00011</td>
<td>0.000035</td>
<td>-0.00017</td>
<td>-0.00018</td>
</tr>
</tbody>
</table>

Figure 1. Comparison of the measured and predicted overtopping rates by the M5' model tree.

This chart shows three inclined lines demonstrate conditions that the predicted rates are 10 times, equal, and 0.1 times of the measured rate.

6. Result

The obtained results were also compared with those of previous models. The accuracy of the model was evaluated by statistical measures, and it was shown that the developed model (M5') is more accurate than previous models.

7. References

1. Introduction

Concrete armour units specially the Third generation armour units, the units are placed randomly in a single layer, are more cost efficient for rubble mound breakwaters. Often the lack of indigenous technical knowledge and implementation of this type of armor is unnoticed. Dezhpod® armour unit has been launched by Sazeh Pardazi Iran Consulting Engineers [1] as the first Iranian concrete armour, based on processing experiences in recent years of design, construction and operation of onshore and offshore structures formed in Iran. The development studies of this armour began about seven years ago. Dezhpod® is used in a single layer and its stability is based on the interlocking between adjacent blocks. The high interlocking quality of Dezhpod® has been proven in laboratory tests, and its suitable geometry showed satisfying hydraulic and structural stability. The ease of fabrication and placement are the other advantages of this armour unit. As Dezhpod® is randomly placed in a single layer, it is comparable to the other single layer armour units such as Xbloc® [2].

The breakwater armour unit Xbloc® has been launched by Delta Marine Consultants and is used in some Iranian projects. Unfortunately, some examples of this armour failure has been observed that were attributed by breakage of the armour units. For instance Figure 1 shows examples of Xbloc® Armour units in Anzali port, Iran, which is broken due to wave impact.

![Figure 1: Breakage of Xbloc® armour units](image)

The observed armour breakage for Xbloc® in above mentioned project, made the Dezhpod® developers to investigate the structural resistance of this armour unit.

In this study a set of structural analysis was performed to investigate the tensile stress distribution in Dezhpod® armour unit comparing to its well-known competitor Xbloc®. To this aim, three-dimensional (3D) Finite Element (FE) model of both armour units are built. Results of the numerical analysis showed that Dezhpod® armour unit has better performance in distributing the tensile stresses and higher resistance to cracking.

2. Finite Element Modeling and Results

The static structural response of Dezhpod® as well as Xbloc® unit through investigating the tensile stress distribution has been investigated with a three-dimensional (3D) Finite Element (FE) model using the computer program ABAQUS 6.14. The 3D FE models of both armour units have the same properties: unit volume 4m³, solid density 2400 kg/m³, Young’s Modulus 30 kN/mm², Poisson ratio 0.20. These particulars are comparable to those used by Hakenberg et al. [3] and Muttray et al. [4]. The armour units have been exposed to flexure, torsion and combined flexure and torsion. The load cases are the same as the ones used by Muttray et al. [4] to study the Xbloc®. These static load cases are outlined in Figure 2.

![Figure 2: Static load cases for FE analysis](image)
The numerical analysis in this study focused on the assessment of the tensile stresses in the FE models, because these stresses are the governing factors for initial damage and cracking in concrete armour units. The stresses and strains that are computed in each of the elements are linearly related to the displacement of the corresponding nodes of the elements. The graphical presentation of the FE models used in this study along with the critical paths corresponding to the load cases are given in Figure 3.

Both finite element models were analyzed under the action of the four load cases and results are presented in Figure 4. This figure presents the tensile stresses along the critical paths shown in Figure 3. As depicted from Figure 4, in all load cases Dezhpod® gives lower values of stresses along the critical paths. Although in Figure 4(c) and 4(e), for Dezhpod®, stresses are raised locally due to the armour geometry, the overall stress values are lower than Xbloc®.

3. Conclusion
A set of numerical analysis is performed to investigate the tensile stress distribution along the critical paths on a 4m³ Dezhpod® armour unit and comparing the state of stresses with Xbloc® armour unit with the same volume and material properties. Results show smoother stress distribution in Dezhpod® armour unit, and lower values of stresses in comparison with Xbloc®. It can be concluded that Dezhpod® unit has higher resistance under static loads and cracking.

4. References
1. Introduction
Where executive and geotechnical conditions allow, one of the most common quay structures is concrete bored pile wall. Two techniques are used for this purpose: Secant piles and contiguous piles [1-4].

Contiguous piled walls process involves the construction of a run of piles with pre-calculated gaps dependent on soils structure and surrounding ground conditions. In contrast, in secant piled walls, primary piles which are composed of plain concrete or even plastic concrete are constructed initially and the main secondary reinforced pile will be executed between them while cutting a portion of the primary ones. As a result, no gap will remain between piles.

Construction of secant-pile wall is more complicated than the other method and required especial equipment. Consequently, Iranian contractors as well as consulting engineers used to prefer contiguous-pile wall. This, nowadays, leads to a significant problem in some of Iranian ports like Anzali and Amirabad: crown settlement and/or creating extended hole under pavement due to soil escaping from pile gaps. This issue will intensify when the adjacent piles have tolerances of pile position and vertical alignment.

In this paper some solutions are proposed to limit material escaping from pile gaps in existing quaywall based on project condition. In addition, some techniques which can be employed for new berth construction is presented.

2. Description of the Problem
Although backfill and subsoil escaping is also common in steel–pile wall, its rate is significantly less than those cases observed in bored concrete-pile wall. This is because steel piles in quaywalls are usually equipped with interlock which can tighten the gaps appropriately. Even with no interlock element, steel piles drive particularly close enough to each other to limit material escaping. But cast-in-situ concrete piles have more executive tolerance and therefore open gaps between them are more probable than steel-pile walls (see Figure 1).

These gaps result in backfill escaping and gradually form large cavities behind the wall. One of these huge holes which have been formed under utility trench behind the main wall of old Anzali quaywall is shown in Figure 2.

These cavities should be filled as soon as possible with non-shrinkage material to prevent future damage of trench, pavement, etc.

Figure 1. Open spaces between concrete piles, Anzali old quaywall.

Figure 2. A large cavity formed under utility trench due to material escaping, Anzali Old quaywall.

3. Material Escaping Remediation
3.1. New Berths
Although contiguous piled walls are common in urban excavation, the best recommendation is using secant-pile wall for new maritime structures. Available experiences show that even perfect construction could not prevent material escaping completely in contiguous pile walls.

If contiguous piled wall is unavoidable, gap tightening should be carried out perfectly. A traditional method is employing tightening pile, a small diameter plain concrete pile cast in situ between two main piles behind their gap. However as they are executed as the same manner, there are some spaces between tightening and main piles. In this condition a modification is using a wedge added to steel casing to occupy the space between piles (see Figure 3).
Figure 3. Using wedge added to tightening pile casing.

Injection grouting or even jet grouting behind the gaps are other methods which are appropriate when the soil in front of the piles is not excavated/dredged yet. Injection grouting is useful for granular soil such as sand and sand-gravel mixture. Jet grouting may be applicable in extended rage of soil grading. However jet grouting should be used with caution because it may damage concrete piles. Moreover, the quality control for these method is usually with some difficulties.

Another tightening method which used successfully is driving a large H/U Pile behind the pile gap, drilling the space between main piles and the H/U steel pile and finally concreting it. It is obvious that the soil around the piles should not be removed until tightening is completed. More explanation will be presented in the next section.

3.2. Existing Quaywalls

For existing contiguous-piled walls which are subjected to material escaping, remediation methods are so complicated because:
- The soil in front of piles which act as a barrier does not exist.
- The space behind the piles is hardly accessible to setup and handle required equipment.
- Unpredicted obstruction may be occurred during executive processes.
- Remediation methods are not usually under control.

Where backside of the wall is accessible from the top, the following method may be used:
- Driving a U-shape sheetpile or H-shape steel profile (see Figure 4)
- Drilling the space between two adjacent piles and the profile by conventional method like air-lifting.
- backfilling the hole by concrete or filter material

The main executive challenge for this method is the pile gaps where the concrete may escape from them. Therefore, temporary tightening is necessary. Manual placement of bags filled by dry concrete, between piles may be a fast solution.

However, the perfect method is tightening the gap by an under-water comprehensive concrete work (see Figure 5):

- Placement permanent forms alongside the gap;
- Pouring grouting mortar from the bottom until it overflow from the top of the form.
Grouting mortar is a fine-grain concrete mixed with additives such as Superplasticizers etc.

The latter method is going to be used for the old Anzali Berth. In spite of several difficulty encountered during execution, this method seems to be more reliable than the other method and its quality can be controlled during construction.

4. References
DESIGN OF AN OFFSHORE PORT BASED ON COASTAL SEDIMENT AND ENVIRONMENTAL ASPECTS

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1. Introduction
Sand deposition at the port entrance and erosion along the down-drift side of the port are the most important points to be taken into consideration for designing the layout of small fishing ports constructed on sandy beaches [1]. Breakwaters trap sediment moving along the coast, and at least in the first year after construction of the port, no (or hardly any) sediment will pass the breakwater and serious erosion may occur at this side of the fishing port. On the other hand, sediment will deposit just after the construction of the port at the up-drift side and sand deposition at the approach channel and at the entrance of the port will occur [2].

Cage culture is one of the most commonly used methods which recently has been considered for fish farming in many countries. In this regard, Iran Fisheries Organization has a plan to develop fish cage cultures at some locations in the Caspian Sea. Larim site which is located at the 30km east of the Babolsar port is one of these selected locations. Marine section of each site is going to have two main parts: 1- An offshore part including cage, feeding barge, … and 2- An onshore port which supports the service and feeding vessels. Many small ports constructed along the southern coasts of the Caspian Sea suffer from different modes of sediment problems. An offshore port connected to the shore by an open bridge which doesn’t block the littoral drift and so could be proposed as an alternative for connected small nearshore ports and a solution for relative sediment problems.

The purpose of this study is to propose an offshore port layout to reduce the sedimentation problems for the Larim service port located in Mazandaran province, north of Iran. To achieve this goal, a series of numerical modeling of waves, currents, sediment transport and coastline evolution have been conducted by LITPACK and Mike21 for 1D and 2D simulation respectively. Also the functional design and dimensional ratios of main parts have been controlled by the relative design references advice.

2. Project Site Conditions
The Project site is located in south-eastern of the Caspian Sea almost 30 km east of the Babolsar port. The hydrography counter lines are parallel to the coastline. As shown in Figure 1, predominant wave direction is NW, the calm period (H<sub>s</sub> < 0.5m) is 32.7% and the maximum wave height is 5.5m at the depth of 20m. Bed slope is uniform and about 0.6%. Based on the local sediment sampling at the depth of -5m, mean grain diameter of bed sediments (D<sub>50</sub>) is about 0.2mm. Site project hydrography shows a shoreline parallel sandbar formed at the depth of around -4m. This bar is a clear sign of cross-shore sand transport and should be considered in port layout design (Figure 2).

3. 1D Sediment Transport Modeling
Based on 1D numerical modeling executed by LITPACK, the net and gross longshore sediment transport rate is estimated approximately 110,000 (from west to east) and 160,000 m³/year respectively. Also surf-zone width is calculated 350m (depth -4.5m) and 450m (depth -5m) corresponding to 80% and 90% of littoral drift rate respectively. Sediment transport model calibration has been done by using historical shoreline evolution behind the Fereydounkenar and Neka breakwaters.

4. Different Scenarios of the Port Layout
To design a small service port at the Larim site, three scenarios have been considered from view point of the port function on littoral sediment transport blockage (see Figure 3). The first scenario is based on partially blockage of littoral drift. The proposed layout is a conventional short port which several years after evolution of the up-drift shoreline will be located in the surfzone. On the other hand the breakwaters have to be extended beyond the sandbar. Then in this layout the basin entrance is located at -5m depth and the length of main and secondary arm is 575 and 360m respectively. The entrance shoaling is expected after a few years but can be decreased by mechanically bypassing of up-drift sediments.
Completely blockage of longshore sediment transport by a long rubble mound jetty is the second scenario. Based on the Litline results, extending the port to -6m depth by a 650m long breakwater will guarantee the sand not to be deposited at the port entrance within a period of 20 years after construction. The up-drift sedimentation and down-drift serious erosion and consequently morphological and environmental problems are the most important defects of this alternative. An offshore port is proposed as the third scenario to reduce the morphological and environmental effects of the hard-blocking coastal structures. Although the offshore port function is similar to 1D offshore detached breakwaters, the two dimensional effects of the basin should be considered on the future shoreline changes.

6. Similar Examples of Existing Offshore Port
Two samples of offshore ports are shown in Figure 5. Kunnuy fishing port is located in Japan as an offshore fishing port to prevent coastal erosion at down-drift of the port by allowing longshore sediment transport. For Kudankulam power plant in west of India, an offshore breakwater has been constructed as a surface water intake.

7. Conclusion
Different scenarios have been proposed as the port layout alternatives for the Larim service port. Sedimentary, morphological and environmentally concerns of the small-short ports especially in the Caspian Sea, guide the layout to an offshore port connected to the shore by a 360m length pile and deck access road. The detail design of the project components and exact cost estimation showed that the pile and deck structural system could be considered as an effective alternative for rubble mound breakwaters in the Caspian sea with almost the same (or not very higher) construction cost. Rock mines excavation and transportation problems, long distance of the borrow materials to the project site for rubble mound structures, besides the morphological and environmental advantage of the offshore port, make the offshore port to be introduced as an economically-technically layout for small ports in the Caspian Sea in Iran. Finally, the sediment monitoring is strongly recommended for offshore ports.

8. References
STRATEGIC PLANNING IN SMALL PORTS (CASE STUDY OF SAJAFI PORT)

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1. Introduction
Unlike large global ports which play a key role in many supply chains, small regional ports are usually more stagnant and have not reached the same level of development as the larger ports. Port development is mostly focused on large ports and small ports have gradually lost their national or regional roles. However, these ports have many potentials and can serve as a part of the whole to help the Port and Maritime Organization (PMO) to achieve the desired long-term goals. Investigation and study of strategic orientations of small ports along Persian Gulf, Oman Sea and Caspian Sea is under consideration by PMO. The present paper focuses on the strategic plans for development of Sajafi Port in Khuzestan Province as a case study [1,2].

Sajafi port is one of the small ports situated near Hendijan city in Khuzestan Province. The aerial view of the port is shown in Figure 1.

2. Methodology
In order to determine the strategic orientation of the small port, first the basic information is gathered by desk studies, site visits and interviews with the stakeholders. According to the available data, the relative advantages (the advantages that depend on the nature of the port itself or are achieved in comparison to other small ports), structural bottlenecks (obstacles that affect and prevent the development and operation of the port) and the prevailing tendencies (affairs that affect the future function of the port in some way) are studied. The mentioned issues are investigated in four categories of physical, operational, managerial, and socio-economic considerations. Based on the analysis of the existing situation and considering the perspective of the PMO [3], the objective will be to determine the strategic orientation of the desired port. The SWOT matrix is used for this purpose [2].

3. Analyses of the Current Status of Sajafi Port
The division of port related issues of Sajafi port in comparison with other small ports of Khuzestan Province (Arvand Kenar, Chebdeh and Shadegan ports), are presented in Table 1.

<table>
<thead>
<tr>
<th>Relative Advantages</th>
<th>Structural Bottlenecks</th>
<th>Prevailing Tendencies</th>
</tr>
</thead>
<tbody>
<tr>
<td>Longer Wharf</td>
<td>Maximum distance to the nearest transportations infrastructures</td>
<td>Tendency to import general tax free products (products transferred by small vessels without paying any tax to the customs duty)</td>
</tr>
<tr>
<td>Fairly suitable port area</td>
<td>Maximum distance to the nearest industrial/commercial centers</td>
<td>Tendency of vessel owners to use neighbor small ports</td>
</tr>
<tr>
<td>Enough space in hinterland for future development</td>
<td>Low quality of access road</td>
<td></td>
</tr>
<tr>
<td>Neighboring with the fishery wharf</td>
<td>Proximity to the vessel repair yard</td>
<td></td>
</tr>
<tr>
<td>Proximity to the vessel repair yard</td>
<td>Maximum distance to the nearest industrial/commercial centers</td>
<td></td>
</tr>
<tr>
<td>Maximum distance to the nearest industrial/commercial centers</td>
<td>Environmental restrictions in Zohreh River</td>
<td></td>
</tr>
<tr>
<td>Low quality of access road</td>
<td>Maximum distance to main marine navigation routs</td>
<td></td>
</tr>
<tr>
<td>Maximum depth in front of the wharf and sedimentation problems</td>
<td>-Low depth in front of the wharf and sedimentation problems</td>
<td></td>
</tr>
<tr>
<td>Proximity to rival small ports (Deylam, Genaveh and Shadegan)</td>
<td>Proximity to rival small ports (Deylam, Genaveh and Shadegan)</td>
<td></td>
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<tr>
<td>Limitation of port infrastructures</td>
<td>Limitation of port infrastructures</td>
<td></td>
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<tr>
<td>Environmental restrictions in Zohreh River</td>
<td>Environmental restrictions in Zohreh River</td>
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<tr>
<td>Tendency to import general tax free products (products transferred by small vessels without paying any tax to the customs duty)</td>
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</tr>
</tbody>
</table>
4. Strategic Position of Sajafi Port

The scores derived from the internal and external factor evaluation matrices (IFE and EFE) should be imported to the vertical and horizontal dimensions of the SWOT matrix. The SWOT method, analyzes the strengths, weaknesses, opportunities and threats associated with the port in a systematic way and reflects the appropriate strategies for the port. Four categories of ST (Strength-Threat), WT (Weakness-Threat), WO (Weakness-Opportunity), and SO (Strength-Opportunity) strategies are achieved according to the results.

In order to provide a more precise analysis, the strategic orientation of the port is once investigated separately for physical, operational, managerial and socio-economic factors and eventually the integration of all influential factors are considered. The results are summarized as follows.

Assessing the physical factors led to ST position. The most important physical factors that threaten the port are the large distance from the transportation systems and the shortages in the infrastructures, such as storage areas and marine navigation equipment. These threats can be controlled with the help of strengths such as the relatively high annual income and high vessel traffic in the port.

Assessing the operational factors led to ineffective zone of WT in the SWOT matrix. This important result have several reasons including inadequate port equipment, high sedimentation and low depth of Zohreh River, environmental restrictions in dredging of the river mouth, the uncertainty about the rate of loading and unloading and the tendency to import tax free goods.

In the managerial viewpoint, the port is in the ST position. Inadequate office space, many employees and lack of compliance with the laws related to the import of goods have led to such a situation for Sajafi port. Increasing the office space and the establishment of commercial rules in the port can lead the port management position to the SO zone.

From the socio-economic point of view, the port is situated in SO zone. The droughts and agricultural decline has expanded the resident’s tendency to port-related activities and has led to the dependence of the people of the city of Hendijan to Sajafi port. Radar charts of SWOT matrix for different factors are shown in Figure 2.

5. Conclusion

The strategic orientation of Sajafi port considering the integration of all influential factors is in the ST position (Figure 2). Considering the total strengths, weaknesses, opportunities and threats of Sajafi port, it can be concluded that the development of the port is not cost-effective due to the natural status of the river and the possibility of competitiveness of the port is low compared to other nearby small ports. The strategy of this port is to preserve and organize the existing commercial activities and to revitalize light fishing activities. Some of the short-term actions in this port could be organizing the legal system of import and export, upgrading navigational and berthing equipment, as well as improving the quality of the access road to the port.

6. References


DESIGN CONSIDERATION OF PILES AGAINST LATERAL SPREADING CAUSED BY LIQUEFACTION

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1. Introduction

The liquefaction is response of loose saturated sand to cyclic loading of earthquake in which the sand loses its strength as a result of excess pore water pressure and tends to liquefy. Accordingly, there is a high potential for lateral movement of soil on slope and severe damage to structures in coastal ports during earthquake as reported around the world [1].

Generally, excessive pore water pressure in vulnerable soils during earthquake has twofold unwanted effects on structures as:

- lateral and vertical strength reduction of soil
- additional lateral load due to the lateral spreading/movement of soils on slope

In this study, the flexural mode of failure in marine piled-structure, as one of the most important cases of maritime structures is investigate and local strengthening of vertical piles in a real case is examined. Upper and lower band of lateral strength of soil is considered to adopt strengthening length of piles.

2. Soil Strength Reduction

Strength reduction of liquefied and its adjacent non-liquefied soil due to the excess of pore water pressure should be considered in evaluation of inertia force on structures. This phenomena is mostly taken into account via p-multiplier reduction factor of P-Y curves or developing a new P-Y curves like soft cohesive soils [2]. In the present study, strength reduction factor according to the JRA in terms of factor of safety against liquefaction and dynamic shear strength ratios for liquefied soils has been used [1]. Additionally, reduction factor for adjacent non-liquefied soils with approach presented in [3] has been considered.

3. Lateral Spreading

Lateral spreading is lateral movement of liquefiable soil during earthquake that can exert a massive extra kinematic load on piles. Nevertheless, according to the JRA (Japan Road Association), the peak ground inertia force and kinematic loading due to the lateral spreading does not occur at the same time, and, consequently, they are not considered simultaneously on structures [1]. Two main approaches of kinematic lateral spreading load, as demonstrated schematically in Figure 1, are applying appropriate displacement to fixed point of P-Y springs or equivalent static load [1]. In this study, the latter one is considered based on JRA 2002.

\[ q_{nl} = C_{nl} \sigma_v \left( H_{nl} \leq z \leq H_{nl} + H_{nl} \right) \]  

Figure 1. Lateral spreading kinematic load [1].

The lateral pressure of liquefied layers \( q_{nl} \) and non-liquefied \( q_{nl} \) above liquefiable layers are considered as:

\[ q_{nl} = C_{nl} \sigma_v \left( H_{nl} \leq z \leq H_{nl} + H_{nl} \right) \]  

where

\[ \begin{align*}
  C_{nl} & = \begin{cases} 
  1.0 & 0 \leq z \leq 50m \\
  0.5 & 50m < z \leq 100m \\
  0.0 & 100m < z
  \end{cases} \\
  \sigma_v & = \begin{cases} 
  0 & 0 \leq I_z \leq 5 \\
  0.2 & 5 < I_z \leq 20 \\
  1 & 20 < I_z
  \end{cases}
\]  

4. Local Strengthening of Piles: a Real Case Study

A real case study of an operational oil platform in south of Iran (Qeshm island)\(^1\) in 8 meters of water depth, located at the end of a jetty access bridge (Figure 2), is investigated in this study.

\(^1\) Owner: National Iranian Oil Products Distribution Company (NIOPDC).
The stress in embedded part of pile is so high that increasing neither pile diameter nor thickness can solve the problem. Additionally, increasing pile diameter absorbs more kinematic lateral spreading load and is inappropriate herein. On the other hand, available soil improvement choices in seabed were not economical due to the size of project and depth of liquefiable soil. Furthermore, filling pile with concrete in total length was not considered due to the increasing of seismic mass of the structure and cost. Accordingly, local strengthening of pile using reinforced concrete is employed in this project. Two upper and lower band of soil strength were considered based on PLOA [5], and the envelop of bending diagram, as depicted in Figure 5, was regarded to find the length to be strengthened by reinforced concrete.

Finally, the composite section is designed based on the maximum bending values of envelop as depicted in Figure 6. This figure clearly shows that the maximum axial force-bending demand is lower than the capacity of the composite sections. Furthermore, composite section will be applied in the model between 9 and 23 below seabed beyond which the steel pile capacity is sufficient. It should be noted that, while in the present study the flexural failure mechanism in the embedded part of piles was investigated, the global buckling of the long pile in the presence of the lateral spreading has been also taken into account for free span of pile columns.

5. References
COMPARISON OF STRUCTURAL DESIGN PROCEDURES OF RCC PAVEMENTS DUE TO HEAVY LOADED APRONS (CONTAINER TERMINALS)

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1. Introduction
Roller-compacted concrete (RCC) or cement bound granular mixtures (CBGM or CBM) is an ultra-tough, zero-slump concrete pavements. It is placed with asphalt pavers to form a nonreinforced, concrete pavement. RCC successfully and economically combines strength and durability with ease of construction especially in heavy-duty industrial pavements (e.g. ports and multimodal terminals). Ports and heavy industrial facilities are large, open areas with few obstructions that may delay the construction process, making them ideal candidates for RCC. Pavements for port and other heavy industrial facilities must be strong and durable because container handling equipment can have wheel loads of 14 to 28 tons or more per tire with traffic speeds typically less than 48 km/hr which are uncommon.

The selection of the pavement course materials and thicknesses is a major part of the pavement design process. There are already several manuals providing guidance on this aspect of container terminal pavement design such as BPA, OCDI, USACE and RCC-PAVE computer program that are the most common pavement design methods used for this application [1-7]. This paper compares the pavements designed by each manual in view of technical and construction procedures and reveals the differences in each procedure.

2. Design Basis
To gain the abovementioned aims, a heavy loaded pavement is analyzed with various procedures. An 8 wheel Rubber Tired Gantry (RTG) crane has been selected to transmit full container as block layout (6 containers with truck lane) in 1 over 5 high stacking. The travel speed of RTG without load is about 8 km/hr. The distances between axles are 2.5, 6.7 and 2.5 m and the gantry span is 23.6 m. The self-weight and capacity of RTG are approximated to 131.5 and 41 tons, respectively. It means that the maximum wheel load is about 28 tons. Number of passes of RTG is assum ed 365,000 throughout design life. The contact area of a tire of handling plant is assumed to be circular with a contact pressure equal to that of the tire pressure. Container handling equipment such as RTG with pneumatic tires is normally operated at a tire pressure of approximately 1.0 MPa.

To compare the results, subgrade is sand or sand-gravel mixtures relatively free of plastic fines. The California Bearing Ratio (CBR) is considered 10% which is equivalent to subgrade reaction modulus of about 54 MN/m3.

The concrete pavement a mixture of coarse and fine stones, cement and water, similar to common concrete but with approximately 40% as much cement and water as normal concrete. It has an average 28-days characteristic compressive cylinder strength of 12 MPa and flexural strength or modulus of rupture of about 2.5 MPa and tensile strength of about 1.5 MPa. The elastic modulus and Poisson’s ration of concrete is about 30,000 MPa and 0.15, respectively. This type of concrete is equal to Cement Bound Material 4 (CBM4).

It is important to note that CBM4 comprises all inclusive elements of RCC but with a reduced strength. For example the compressive strength of RCC can be more than 32 MPa.

On the other hand in RCC, surface appearance and texture are generally not of great importance and the surface smoothness typically has a 9.5 mm maximum variance for a 3 m straight edge. Therefore unsurfaced RCC pavement can be allowed in low traffic speeds areas such as ports. Nevertheless concrete block paving is a preferred container terminal surfacing material in many regions especially Europe because it combines the benefits of the durability of concrete with the flexibility of asphalt. The driving factor was the anticipated settlements and the ability to remove and relay areas of pavers from time to time to maintain the required terminal surface level. Thickness of pavers is usually 80 mm and even roles directly in pavement structure design by Material Equivalent Factor (equals to 0.87*80=70 mm CBM4) [6].
3. Design Procedures

The existing design manuals are based upon computing stresses and/or strains at critical locations within and directly below concrete pavements, comparing those computed stresses/strains with values which the pavement construction materials are known to be able to sustain successfully, sometimes referred to as permissible stresses/strains and thereby providing pavements of sufficient structural capacity. The permissible stresses/strains are established by using a Transfer Function, which is a relationship between the number of repetitions of a loading event and the corresponding permissible stress/strain. Transfer functions are entirely empirical relationships which have been derived by observing the way in which pavements have performed historically and are usually informed by scientific fatigue relationships. Each design procedure has particular formulations to calculate the concrete pavement thickness.

In PBA [4], the geometry configuration of all wheels should be determined for each vehicle. Based on effective depth, proximity factor and dynamic load factors, single equivalent wheel load (SEWL) is calculated and thereafter regarding load repetition proportional to damaging effect of wheel loads, CBM4 thickness is approximated. If abovementioned RTG specifications and subgrade condition are considered, this procedure results in SEWL 32.2 ton, about 1,400,000 repetitions and consequently a 330 mm CBM4.

OCDI and RCC-Pave computer program have a similar technique to design the pavement. In this method, the fatigue characteristics of concrete pavements are calculated based on the wheel load stresses imposed on concrete and their numbers of load repetition during design working life. The relation between the above mentioned characteristics and the degree of fatigue as a failure criterion is proposed to set the thickness of RCC. Allowable numbers of wheel load have been shown in Figure 1 depending on stress ratio (ratio of critical applied flexural stress to flexural strength). The degree of fatigue of a concrete slab is calculated from $F_{n} = \sum \frac{n}{N}$ (n = wheel load repetition, N = Allowable numbers of wheel load).

Using the degree of fatigue as the failure criterion of a concrete slab, concrete thickness is set so that the degree of fatigue is equal to 1.0 or less. Each procedure uses a method to calculate the flexural stress due to wheel loading. A 28 ton wheel load of RTG causes a 560 mm and 540 mm respectively for OCDI and RCC-Pave thickness is approximated. If abovementioned RTG specifications and subgrade condition are considered, this procedure results in SEWL 32.2 ton, about 1,400,000 repetitions and consequently a 330 mm CBM4.

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The USACE procedure for RCC pavement design is similar to the procedure for conventional concrete pavements. The vehicle loading is expressed as an equivalent number of repetitions of an 8.2 ton single-axle loading, and, as a further simplification, the range of equivalent repetitions of the basic loading (i.e., traffic) is designated by a numerical scale defined as the pavement design index. USACE applies only a chart to determine the pavement thickness regarding to subgrade reaction, flexural strength and pavement design index. This method consequences in a RCC pavement with a thickness not more than 250 mm.

![Figure 1. Fatigue relationship for RCC.](image)

4. Conclusion

The following important conclusions can be drawn:

- Tensile and flexural strengths are the key characteristics factors of RCC respectively for BPA and OCDI design procedures.
- For a specific vehicle loading and subgrade condition together with a determined concrete pavement, The BPA and USACE lead to a near range of pavement thickness and two other methods show a notable increase about 90 percent in concrete thickness.

Although the examined codes introduced to design of heavy duty pavements, the results show noticeable deviation to each other and the design procedure can directly affect on pavement design and consequently the design economy. Therefore especially in national ports it is needed to codify a unique standard to design heavy duty pavements.

5. References

DESIGN OF BREAKWATER FOR A NEW MULTI-CARGO PORT IN BLACK SEA

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1. Introduction

Filyos Port is planned as Turkey’s third largest port for multi-cargo handling including containers, dry bulk and break bulk. It is located at central Black Sea coastline of the country, presently a greenfield area with long sandy beach. When completed, it will have total container throughput capacity of 1.5 million TEU’s and 20 million ton bulk cargo.

![Figure 1. Project location.](image1)

The Port will have a 2,700 meters long main breakwater placed mostly at 10 meters water depth and 1,500 meters long secondary breakwater protecting the berths from harsh Black Sea wave climate. Various armor layer design options with tetrapod, accropode and xbloc units were investigated in the project and the final design was developed with xblocs, which is also a first application with these units in the country.

Construction of the port is presently ongoing and in the first stage breakwater construction is progressing ahead of other activities to provide a sheltered work area for the berth constructions to start.

In this paper, basic design criteria employed and main issues addressed in developing the design of the breakwaters will be discussed.

2. Geotechnical Conditions

The seabed in the project area is mostly sand in the region close to the shore however soft clay layers at sea bottom starts to appear as one goes towards offshore. At the location of the main breakwater which is approximately at 10 meters water depth, thickness of the soft silty-clay layer reaches to 5 meters in average. The cohesion of the soft clay layers are determined to be less than 20 kPa.

Geotechnical investigations showed that thick and hard to medium hard sand layers exist under the soft clay layer. Slope stability, bearing capacity and settlement analysis carried out for the breakwater showed that a safe and stable breakwater cannot be built without improving the parameters of the soft clay layer underneath the breakwater. After assessing various alternatives of improvement, replacement method was selected as the most economical approach. It was decided to dredge all soft material under the breakwater and replace with quarried rocks to create a stable foundation for the structure.

3. Design Wave Height

The design wave parameters for the breakwater were based on the offshore wave parameters as described in the Turkish Wave Atlas for the Black Sea [1]. Numerical modelling studies were conducted for transformation of waves as propagate from deep water to the breakwater location. The main breakwater was designed for Hs=7.0 m. at deepest sections.

![Figure 2. Sample output from wave modelling studies.](image2)

4. Comparison of Concrete Armour Units

Many breakwaters with concrete armour units have been constructed in the Black Sea coastline of Turkey in the past. In those projects initially cubes and tetrapods have been used. In 1990’s various projects with antifer cubes were realized however tetrapods stayed as the most favored units both in public and private sector projects.
Filyos Port’s breakwater was initially planned and tendered with tetrapod units however EPC Contractor decided to assess the overall construction economy (cost + time) by comparing design alternatives with tetrapods, accropodes and xblocs. The final decision was made in favor of xblocs and breakwater design was prepared by using 10 m³, 8 m³ and 4 m³ units at different depths.

In the design of the breakwater protection with xblocs, the guidelines issued by the product licensor Delta Marine Consultants were used [2] together with various study results published on the subject [3-5].

5. Hydraulic Design and Tests
Stability tests for the designed breakwater sections were done in the laboratory facilities of Ministry of Transportation. A series of wave conditions were used in the tests along with design water levels including sea level rise due to climate change. The designed breakwater sections were verified by the tests against allowable damage levels and provided input the final design of the structure.

6. Seismic Design
Being located in an earthquake prone area with relatively weak soil conditions, the seismic design of the breakwater was a challenging issue. The local codes required the breakwater to be designed against the Contingency Level Earthquake with a return period of 475 years and only controlled damage is allowed. This design earthquake corresponded to an equivalent seismic coefficient, kh of 0.16.

Extensive pseudo-static and full dynamic geotechnical analysis were conducted to determine the behavior of the breakwater under seismic events. Dynamic analysis included Finite Element Modeling of the breakwater with time domain analysis.

7. Conclusions
Design studies for the breakwaters of Filyos Port involved extensive comparative analysis of different concrete armour units with regard to material, labour, formwork, handling and placement costs. In this very first application in Turkey of single layer armour design with semi-random placement, construction speed and less dependency on heavy-lift equipment were the main factors effective on the final decision.

Wave and geotechnical modelling studies conducted during the study set examples for the similar project in the area to cope with the wave and seismic design conditions in weak soil conditions. Following the finalization of design works of Filyos Port, two another major port projects with xbloc units were initiated in the region.

8. References
[1] NATO TU WAVES, Turkish Wave Atlas
[4] Pearson, J., van der Meer, J.W., Bruce, T. and Franco, L., "Overtopping performance of different armour units for rubble mound breakwaters"
THE EFFECT OF SEA SAND DEREDGER ON THE RESISTANCE OF CONCRETE PIECES REINFORCED WITH METAL FIBERS

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1. Introduction and Background

The unique properties of concrete, especially reinforced concrete, and economic costs make it the most suitable material for most infrastructures of the world.

The aggregate is known as cost reduction factor of concrete production which is now a challenging topic for sustainable development. The most urgent issue in this regard is the environmental impacts of aggregate production. Many areas in the world including Persian Gulf are suffering from excessive deterioration and even inaccessibility to appropriate rock and minerals. The unconventional use of aggregate mines in the country and the limited source of these mines and their accessibility cost made us to consider a solution for this issue concerning the demolition of dredging materials on the coast and the shortage of materials in far distances of south Iran and margins of Persian Gulf. In this paper, it is tried to use metal fibers for reinforcing this concrete with various percentages of 4%–6% and 8% in addition to using sea dredging sand (prepared from Shahid Rajaee Port Complex of Bandar Abbas) as 25% substitute for fine aggregates in concrete. In so far as mostly in making reinforced concrete armatures, fabric is used instead of bar, therefore, the impact of sea sand on metal fibers for reinforcing this type of concrete will be studied in this application.

In most parts of the world, there is a legal restriction on utilization of pits and mines. For example, in parts of the southwest of England, the supply of aggregates is very low, so that in order to meet the demand, it is inevitable to import it from the Midlands, Ireland and Manche Channel. In a study in Spain, three different concrete treatments have been used for construction of the pavement and the impact of dredger sand as a sandstone in production of concrete has been analyzed. In Barcelona, another study has been conducted on making a cement mortar from three types of sand. The purpose of this research was to determine the effect of adding dredge sand on bonding and consistency of concrete and the effect of modified fine grains on compressive strength in the different mixing scenarios of concrete. In their study, Chapman et al concluded that the dredged material of the sea could be used in the manufacturing of concrete [1, 2, 3, 4].

using the dredging materials, as a replacement for the whole or a percentage of the aggregates is a perfect solution. Regarding various aspects related to this new source of materials several studies have contributed. Examples of which are being followed as: Limeira et al. (2010); Limeira et al. (2012); Moradi et al. (2018); Etxeberria et al. (2016); Liu et al. (2016) [5, 6, 7].

2. Experiments and the Tests Setup

The experimental works are categorized in three main sets. First is the mix design and programming, in addition to the required tests on the ingredients and the basic materials. The second category is providing the specimens, (mixing, casting, curing etc.). The final is the mechanical and permeability tests on the specimens.

3. Outcomes of the Laboratory Tests

In this section, the laboratory program, the specifications of concrete materials used in concrete and also the results of compressive strength test on the concrete tests made from sand of sea and reinforced with metal fabrics will be presented. The Persian Gulf dredged sand is consisted of various materials, a percentage of metals and different kinds of marine sediments that have been passed from sieve number 4 as replacement for fine-grained concrete. These materials with a density of 2.7 have high water absorption rate. In Figure 1, the optimum moisture content of dredge sand that in this moisture content reaches its maximum density is shown.

![Figure 1. Dry and wet processes of test and curing specimens](image)

![Figure 2. Optimum moisture curve of sea dredged sand](image)
The mixture patterns used in this type of concrete are presented in Table 1.

<table>
<thead>
<tr>
<th>Control type</th>
<th>25% DMS &amp; 4% Fiber</th>
<th>25% DMS &amp; 6% Fiber</th>
<th>25% DMS &amp; 8% Fiber</th>
</tr>
</thead>
<tbody>
<tr>
<td>C25 0.8</td>
<td>C25 0.6</td>
<td>C25 0.4</td>
<td>C25 0.2</td>
</tr>
</tbody>
</table>

Table 1: Specifications of the number specimens, materials used and the percentage of substitutes for sea dredged sand

In this test, end hook steel fiber of diameter 0.8 and length of 50 mm of 4%–6% and 8% as replacement for cement has been used. The used fiber is shown in Figure 3.

The cube test pieces of 15*15*15 cm have been tested in five mixture pattern for compressive strength. The fresh concrete temperature has been measured in range of 28 to 33 degrees Celsius and controlled 12 cm - concrete slump which increased by 15 cm by adding sea sand to concrete. It means that the concrete will be more consistent and easily condensed with increase of sea sand which is rounded and fine-grained and filling the space between coarse grains. It is worth noting that by adding metal fibers, the concrete's consistency is reduced and its slump decreases by about 7 cm. The results obtained from mean of data related to three cubic samples are the same. In this project, the compressive strength of the test specimens has been measured according to standard C39 ASTM [8] in 28 days whose results are depicted in Figure 4.

4. Discussion and Conclusions

As shown in the diagram, adding 25% dredging sand to concrete leads to its increased resistance compared to control concrete and by adding metal fibers up to 6 weight percent of cement, the compressive strength reaches its maximum and adding more fibers leads to decreased strength of concrete due to lack of compression and cohesion of materials and lack of uniform dispersion of fibers in concrete. One of the challenges of adding these fibers is their lack of flexibility and length and lack of uniform dispersion in concrete that leads to shrinkage of fiber and materials in a limited area and reduces the slump and concrete performance. Therefore, in order to reinforce concrete with this type of fiber, a special method and instrument should be used for mixing that does not lead to their entanglement.

5. References


1. Introduction
Generally, coastal structures could be impediments to natural flow of sand. Sedimentation and erosion close to small ports are acute problems and there is no general consensus on appropriate measures to mitigate their adverse effects. Some mitigating approaches include beach nourishment and sediment bypassing. Sand transfer systems are a means of transporting sand around an impediment in an attempt to reinstate the flows of sand that would occur naturally [1]. Sand bypassing systems operate on simple principles, and consist of dredging, transporting and depositing sand. Mechanical bypassing is an approach to transport sediment from the updrift to the downdrift side of ports or inlets. Bypassing can be achieved through dredging or excavator and truck delivery. Many sand bypassing systems have been designed and operated in the last decades. Boswood and Murray [2] list 53 different bypassing stations from around the world and document their environmental and system parameters. A pump or eductor is used for dredging operations with bypassing systems which could be a water-based mobile system: where the dredge pump is operated from a floating, mobile platform, or a land-based mobile system: where the dredge pump is generally operated by a crane, or a fixed system: where the dredge remains in a single location ([1], [2], [3]).

Since construction of Chamkhaleh port (the former port located in Guilan, The Caspian Sea), heavy sedimentation behind the west breakwater and rapid shoreline advance have been observed. Also the port basin was located at river mouth which itself was at the influence of two rivers discharge. Inaccurate layout of the breakwaters has caused a calm region for siltation and accumulation of fine sediments in the port basin and eventually led to disruption in the port operations in recent years (see Figure 1). The required offshore data providing important information for the detailed 2D morphological assessment and 1D littoral drift study of Chamkhaleh coastal area has been extracted from two wave hindcasting projects of ISWM I (covered 1992 to 2003 at 6-hour intervals), and monitoring and modeling study of northern coasts of Iran (covered 1983 to 2013 at 1-hour intervals). Comparison of hydrographic surveys (years: 1998, 2010, 2012, 2014, 2017), satellite images and aerial photos has shown the average annual net long shore sediment transport (LST) rate is about 100,000 m$^3$ from west to east ([4]). This paper presents some results of Chamkhaleh’s port development plan proposed by Karan Sazeh Pasargad consulting engineers Co.

Figure 1. Top: study area, Bottom: upstream coastal profile

2. Outline of the Project
The location of former port (at river mouth) has not been recognized appropriate for further development. Hence, different port layouts along the coast have been studied in detail [4]. With regard to special conditions in the Caspian Sea and uncertainties in prediction of long-term fluctuation of water level, any new project in this region should be adaptable enough to any probable changes. Finally, by considering different scenarios, it became clear that construction of a single-main-breakwater port with sand bypassing system would be a viable solution for future development plan of Chamkhaleh coastal area. At the development plan, effect of breakwaters layout on the port basin tranquility and sedimentation pattern has been studied numerically using DHI model. Change in orientation of the main breakwater near the port mouth along with the effect of reflected waves from the secondary breakwater could mitigate adverse influence of eddy formation and sedimentation close to the port entrance. Additionally, at the development plan a calm area close to the port has been considered in order to trap the littoral drift coming from upcoast. Figure 2 illustrates different
parts of the project. Also the breakwaters construction would be implemented by means of geotubes (filled up with pumping sands from the bed of upcoast and river mouth) protected by a layer of geotextile, armor and tetrapod.

3. Finding Design Capacities, Technical Specifications and Costs of the System

As previously mentioned offshore waves data (from 1983 to 2013) have been used to investigate the variation of longshore sediment transport rate and its distribution along coastal profiles. The results after calibration, have been stored in time series format indicating critical situations of LST during storms, seasonal variations and average annual rate of LST. Afterwards sediment transport along the coastal profile for each storm has been studied in detail using LITPACK model. Besides, the time spans between strong storms have been considered as criteria for finding the required time for operating the system in most critical situation. The system was designed to bypass a target rate of 100,000 – 210,000 m$^3$ of sand per year, but it has sufficient capacity to bypass up to 340,000 m$^3$ per year. During extreme events, such as a storm, the system was also designed to be able to bypass a capacity of 20,000 m$^3$ (sand) in seven days. The bypassing system would only require working in a few months of year, and even in some months the system could be completely turned off. It should be also noted that in practice, normally the slurry (mixture of sand and water) is pumped on a 1/3 ratio, so capacity of pumps in the aggregate should be larger than three times of the rates mentioned above.

The main equipment of the system encompasses three submersible pumps (capacity: two pumps 44 kW -140 m$^3$/hour and one pump 60 kW -350 m$^3$/hour), a booster pump and control panel, an excavator along with water jet ring and its pump, pipes and valves, a floater and two diesel generators. The initial costs of the system amounts to 1.2 million Euros.

4. Conclusion

The results of this pre-feasibility study show that the proposed sand bypassing system adaptable to the port layout, the sand trap and Caspian sea level fluctuation is a viable alternative for Chamkhaleh coastal development plan. The study, provided vigorous data needed for designing specifications of the systems and estimating its initial costs, however, some robust studies should be conducted in order to see all aspects of the project.

5. References

GEOTECHNICAL VARIATION IN MARITIME WORKS AND NECESSITY OF PERFORMING GEOTECHNICAL INVESTIGATION FOR EACH SITE SEPARATELY

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1. Introduction
Gathering sufficient geotechnical information from a project area is one of the most important parts of the design phase of marine structures. Since significant variation in both layering and geotechnical parameters alongside of shore area is expected, lack of comprehensive information about subsoil usually leads to unforeseen problems during construction phase and sometimes causes great change in the project layout and/or structural system. Moreover, marine structures construction and maintenance is more complicated than the other ones and usually required more time and money. Therefore, appropriate planning for geotechnical investigation proportional to the project cost, is of crucial importance to save money as well as the safety.

In this paper, in order to show how much the subsurface can be variable in a shoreline area and to show the importance of adequate site investigation, the results of some geotechnical studies performed for a part of Persian Gulf coastal area in Hormozgan province, Iran, are assessed here. As a result, necessity of new geotechnical investigation for a new project near the ones that have field investigation is obvious. In the following, some recommendations for geotechnical investigation derived from international maritime codes are presented.

2. Problem Description
In order to construct marine structures including protection dikes and breakwaters and also some berths, four sites which are close to each other were explored separately. Figure 1 shows these sites location, case 1 to 4. They are located alongside the shoreline with 3.5km long and extended to the sea up to 1km.

The first case includes 10 boreholes with lengths of 20-30 meters [1], and the second one comprises 7 boreholes with lengths of 10-30 meters [2]. The third and fourth cases include 6 boreholes with a length of 20 meters [3] and 5 boreholes with lengths of 25 meters respectively [4]. A variety of in-situ and laboratory tests have been carried out during the investigation (e.g. S.P.T, Relative Humidity, Atterberg Limits, Soil Specific Gravity, Direct Shear, Triaxial, Consolidation and Uniaxial).

3. Subsurface Variability
Based on 4 series geotechnical investigation carried out at the project location, soil layering profile alongside the shoreline is illustrated in Figure 2. As shown in this figure, in the case 1, the surface layer is composed of medium dense silty sand (SM). Thereafter, there is a fine-grained layer of CL and ML, with increasing consistency with depth. The last layer is a sandy soil (SM) with moderate to high density. In the case 2, subsoil consists of fine-grained (clay and silt) layers that their consistency increases with depth. In addition, medium to dense Silty and clayey Sand (SM / SC) is also observed in the middle layers. In the case 3, a soft clay surface layer underlain by a dense silty Sand (SM) layer which is observed up to the end of the boreholes. A smooth clay layer (CL) layer is also observed in the case 4. Below this layer, thin lens of Coral followed by a SM soil with moderate density is observed.

In addition to the variability alongside the shoreline, the cross-section profiles also revealed that as the distance from the coast increases, the thickness and density of layers changes. For example, the cross-section profile of the case 1 is shown in Figure 3. The result shows reduction in SPT numbers and increase in the thickness of loose layers with increasing the distance to the coast.

Results show that although geotechnical trend is predictable for the site near the mentioned cases (for example the area between case 3 and 4), some details may not be predicted exactly without performing special
geotechnical investigation for that site. For example, the thickness of shallow soft layer which affect the breakwater settlement and the amount of material penetration varies significantly from case to case. Moreover, soil relative density in second layer changes rapidly and can affect, for one, on the penetrating length of deep foundation which are common for berths in soft soils. Both of these cases may have great influence, at least on the project cost because breakwater and the berths are the main structures in a marine projects.

- Design phase (Preliminary / Detail);
- Soil stratification (Simple / Complicated);
- Project Scale;
- Type of Marine Structures;

![Figure 2. Soil profile variation in 4 close project.](image)

**Table 1. Borehole layout & spacing [5-6]**

<table>
<thead>
<tr>
<th>Type of Marine Structures</th>
<th>Case 1 – Large scale area (MARINE ST.)</th>
<th>Case 2 – Small scale area (MARINE ST.)</th>
<th>Case 3 – Complicated stratification</th>
<th>Case 4 – Uniform stratification</th>
</tr>
</thead>
<tbody>
<tr>
<td>Code</td>
<td>300 – 500</td>
<td>50 – 100</td>
<td>50 – 100</td>
<td>10 – 30</td>
</tr>
<tr>
<td>Design Phase</td>
<td>Preliminary</td>
<td>Preliminary</td>
<td>Preliminary</td>
<td>Preliminary</td>
</tr>
<tr>
<td>Soil Condition</td>
<td>Very soft to Soft</td>
<td>Medium</td>
<td>Very soft to Soft</td>
<td>Medium to Stiff</td>
</tr>
<tr>
<td>Type of Required Structures</td>
<td>Large scale area (MARINE ST.)</td>
<td>Small scale area (MARINE ST.)</td>
<td>Complicated stratification</td>
<td>Uniform stratification</td>
</tr>
<tr>
<td>Max dist. From the Face Line of Structure</td>
<td>300 ~ 500</td>
<td>50 ~ 100</td>
<td>50 ~ 100</td>
<td>10 ~ 30</td>
</tr>
</tbody>
</table>

5. Conclusion

Based on the results of exploratory boreholes presented in this study, it is observed that the thickness and density of different layers of soil vary at different points in the soil layering profile and cross sections. Therefore, depending on the importance of the project, geotechnical studies and full understanding of the in-situ geotechnical conditions are of crucial importance.

Recently, because of high price of maritime geotechnical investigation, clients’ trend is to omit these investigation and used available data from the investigation carried out near the project area. Although it can be useful for preliminary design, it cannot predict all of the geotechnical challenges of a maritime project. It should be noted that projects costs also rise as well as site investigation price rise. Therefore geotechnical investigation is still a little portion of a project cost either marine project or other ones.

6. References


Figure 3. Case 1 cross section perpendicular to the shoreline
DEVELOPING FRAGILITY CURVES AS AN EFFICIENT METHOD FOR ASSESSING COMMON RETROFIT METHODS OF PILE-SUPPORTED WHARVES

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1. Introduction
The aim of this study is proposing fragility analysis as an efficient method to evaluate common retrofit methods (FRP and batter pile) for pile-supported wharves damaged by aging effect in harsh environment of sea. In this study, fragility curves are developed for the concrete pile-supported wharf located in Persian Gulf by Capacity Spectrum Method (CSM) [1]. To compare retrofit methods, the wharf is modeled in ABAQUS 6.12. Then, results are shown in four different conditions i.e. before aging effect (initial condition), after aging effect (damaged condition), and retrofitted conditions (FRP without batter pile and FRP with batter pile). The damaged condition is considered for the splash zone having the highest corrosion rate among the other zones. In this zone, the behavior of damaged concrete is modeled according to the physical modeling carried out for concrete material in Persian Gulf environment [2], and for rebar, the corrosion rate proposed by OCDI is considered [5]. To retrofit the damaged model, at first, FRP layer is wrapped around damaged areas. Then, the batter pile with the degree of 43º is attached by rubber isolator (Figure 1).

CSM is a Pushover (SPO) based method which measures performance points of structures under scaled selected spectrum. Here, this method is performed by choosing 8 records (Table 2), and the probability of the responses exceeding from the damage states, defined by PIANC [6], is obtained. The cross section of the wharf and property of soil layers are respectively displayed in Figure 1 and Table 1 [3]. The soil layer is modeled by Mohr-Coulomb theory.


![Figure 1. Wharf in four conditions [3]](image)

Table 1. Soil layers

<table>
<thead>
<tr>
<th>Soil layer</th>
<th>Density (kg/m³)</th>
<th>E (kg/m²)</th>
<th>φ'</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gravel</td>
<td>2000</td>
<td>17×107</td>
<td>30</td>
</tr>
<tr>
<td>Sand</td>
<td>1835.65</td>
<td>17×105</td>
<td>40</td>
</tr>
</tbody>
</table>

Table 2. Ground motion used for seismic analysis [4]

<table>
<thead>
<tr>
<th>Event</th>
<th>Year</th>
<th>M</th>
<th>R (Km)</th>
<th>PGA (g)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Chi-chi-1118</td>
<td>1999</td>
<td>7.6</td>
<td>26.8</td>
<td>0.147</td>
</tr>
<tr>
<td>Chi-chi-1132</td>
<td>1999</td>
<td>7.6</td>
<td>18.7</td>
<td>0.165</td>
</tr>
<tr>
<td>Chi-chi-1143</td>
<td>1999</td>
<td>7.6</td>
<td>25.9</td>
<td>0.639</td>
</tr>
<tr>
<td>Imperial-0200</td>
<td>1979</td>
<td>8.2</td>
<td>17.9</td>
<td>0.175</td>
</tr>
<tr>
<td>Kobe-1041</td>
<td>1995</td>
<td>6.9</td>
<td>26.4</td>
<td>0.345</td>
</tr>
<tr>
<td>Hills-0728</td>
<td>1987</td>
<td>6.8</td>
<td>27.1</td>
<td>0.167</td>
</tr>
<tr>
<td>Cape Mendocino</td>
<td>1992</td>
<td>7.0</td>
<td>16.5</td>
<td>0.116</td>
</tr>
<tr>
<td>Manjil</td>
<td>1990</td>
<td>7.3</td>
<td>39.0</td>
<td>0.350</td>
</tr>
</tbody>
</table>

2. Developing Fragility Curves
In order to develop fragility curves for all the models in the predefined damage states, SPO curves, base shear (Vb) versus deck displacement (Δddeck) (Figure 2), are converted to capacity curves, spectral acceleration (Sa) and spectral displacement (Sa) (1-2). Then, the performance points (x) are driven by intersection of capacity curves under scaled demand spectra i.e. the spectra of chosen recordings. The scaling should cover the whole range of structural responses from elastic to global instability. Furthermore, fragility curves are developed by lognormal distribution with the probability density function (PDF) in the predefined damage states (x) i.e. serviceability (I), repairable (II), and near collapse (III) (Table 3) (3-6) [1].

\[
S_{ai} = \frac{V_b}{W\times n_i} , \quad S_{a} = \frac{S_{a,deck}}{S_{a,deck}}
\]

(1),(2)

where \(a\) and \(P_i\) are respectively the modal mass coefficient and participation factors for the first natural mode of the structure and \(\Phi_{roof}\) the roof level amplitude of the first mode.

\[
f(x) = \frac{1}{\sqrt{2\pi}\lambda} \exp\left[-\frac{1}{2} \left(\frac{\ln(x-\lambda)}{\zeta} \right)^2 \right] \quad 0 \leq x < \infty
\]

(3)

Where \(\zeta\) and \(\lambda\) are the parameters of lognormal distribution of random displacement variable \(X\). These two parameters can be calculated by mean value (\(\mu\)) and
standard deviation ($\sigma$) of sample population ($x$) in each scaled level: (4-5)

$$\lambda = \ln \frac{1}{2}^\sigma$$, $\xi^2 = \ln[1 + \delta^2]$ (4, 5)

Where $\delta = \frac{\sigma}{\mu}$

Fragility curves for damage states of $s_i$ is the conditional probability of wharf responses exceeding damage states of $s_i$ at a specific PGA level (6) [3].

$$P[S > s_i|PGA] = P[X > x_i|PGA] = 1 - \Phi\left(\frac{\ln x_i - \lambda}{\xi}\right)$$ (6)

Where $\Phi$ is normal cumulative distribution function, $x_i$ is the upper bound for $s_i$ (i=I, II, III), and $\lambda$ and $\xi$ are the parameters mentioned above and dependent on PGA level (Figure 3-5).

3. Results

In this study, fragility analysis is suggested to assess the efficacy of two common retrofit approaches, and fragility curves are developed to show how much these methods improve the performance of structure in each damage state. It is shown that using the batter pile could almost alleviate the conditional probability of wharf responses exceeding serviceability state. For the other damage states, however, there are no discernible changes. Therefore, in this wharf, it might not be economical to apply batter pile.

4. References


DUCTILITY ENHANCEMENT OF CEMENT BASED SECTIONS REINFORCED WITH FRP BARS

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1. Introduction
The major cause of deterioration on steel-reinforced concrete structures exposed to marine environments, is corrosion of the reinforcing steel. This deterioration is most evident on substructure components (e.g., foundations, footers, pilings, etc.); however, reinforcing steel corrosion can also be present on superstructure components (e.g., bridge decks, beams, pile caps, etc.). Fiber-reinforced polymer (FRP) materials have emerged as an alternative for producing reinforcing bars for concrete structures. Fiber-reinforced polymer reinforcing bars offer advantages over steel reinforcement because they are noncorrosive [1]. The primary motivation of this paper is to investigate the ductility of concrete beams reinforced with brittle FRP composite.

2. Ductility
As concrete and FRP rebars are both brittle materials, ductility becomes a great concern [1]. Unless ductility requirements are satisfied, FRP materials cannot be used reliably in structural engineering applications. The term ductility describes the ability of a member to undergo large deformation without rupture before failure occurs. The ductility index is often defined as the ratio between the ultimate deflections (or curvature) over the deflection at yielding [2].

In some cases, the traditional definition of ductility cannot be directly applied to concrete structures reinforced with FRP reinforcement, it is necessary to develop a new approach as well as a set of ductility indices for evaluating the ductility performance of FRP reinforced members. Accordingly, two main approaches have been widely used. One of them is deformation-based approach. This approach was first introduced by Jaeger et al. [3]. According to this approach, the ductility index is equal to:

$$J_{\text{index}} = \frac{M_{\phi}}{M_{0.001}}$$

where $M$ and $\phi$ are the beam moment and curvature and the subscripts $u$ refer to the ultimate state, and 0.001 to the service state that corresponds to a concrete maximum compressive strain of 0.001 [3]. This factor must be greater than 4 for rectangular sections and greater than 6 for T-sections.

3. Numerical Model
The load-deflection behavior of reinforced concrete beams is calculated by a program in FORTRAN. The beams are discretized into multi-layered short elements.

The moment-curvature diagram of each element is calculated by applying the assumptions that plane sections remain plane and that the strain in the reinforcement is the same as that in the surrounding concrete. Any stress-strain diagram can be used for concrete and steel.

3.1. Validation
This methodology was validated with an experimental data from Meda et al. for conventional RC beam [4]. All samples were 4m long with a span of 3.6m. The height, depth and the width of the section were 300 mm, 260mm and 200mm, respectively. For tension reinforcement 2φ16 and 4φ16 and for the compression ones 2φ10 were used. In the studied experimental study the yield stress of reinforcing steel and compressive strength of concrete were 500Mpa and 49.7Mpa, respectively. It should be noted that the post-peak behaviour of concrete in tension has been neglected. Figure 1 compares the experimental records with the results of the numerical study. It can be seen that the test results are in good agreement with the numerical results.

![Figure 1. Load-Deflection curve for experimental [4] and numerical (green and red) model.](image)

4. Results and Discussion
In this paper, to overcome the lack of ductility of concrete sections reinforced with FRP bars, two approaches were considered; replacing conventional concrete with Polymer-Modified-Concrete (PMC) and using ECC.

Figure 2 and Figure 3 show compressive stress-strain curve for PMC (modified with SBR polymer) [5] and tensile stress-strain curve for ECC (used PVA fibers) [6], respectively.
The modulus of elasticity and ultimate strain of GFRP are 46GPa and 1.42%, respectively. Table 1 shows the results for the combination of PMC and GFRP. The cross-section of all models were considered to be 400mm×400mm. The J-index for all models were calculated and it was found that all values are greater than the minimum requirement, i.e. 4. Therefore, the mentioned beams are acceptable by codes from ductility point of view.

Table 1. Results of PMC and GFRP models.

<table>
<thead>
<tr>
<th>I-D</th>
<th>GFRP Area (mm²)</th>
<th>J-index</th>
<th>Failure mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>SBR-GFRP900</td>
<td>900</td>
<td>5.92</td>
<td>FRP rupture</td>
</tr>
<tr>
<td>SBR-GFRP2000</td>
<td>2000</td>
<td>15</td>
<td>FRP rupture</td>
</tr>
<tr>
<td>SBR-GFRP3000</td>
<td>3000</td>
<td>25.28</td>
<td>FRP rupture</td>
</tr>
<tr>
<td>SBR-GFRP7000</td>
<td>7000</td>
<td>27.84</td>
<td>Concrete crush</td>
</tr>
<tr>
<td>SBR-GFRP15000</td>
<td>15000</td>
<td>29.66</td>
<td>Concrete crush</td>
</tr>
</tbody>
</table>

Table 2 shows the details of models for considering ECC as a replacement for concrete and GFRP as a tensile rebar. All sections were 400mm×450mm with 10mm cover. Figure 4 shows the moment-curvature diagrams of these models.

To calculate ductility index (µ), a bilinear transformation method, based on ATC-40[7], was used. Figure 5 shows a bilinear graph using the mentioned approach. The graph shows that µ equals to 23.96. Therefore, to improve ductility of concrete sections reinforced by FRP bars ECC can also replace the conventional concrete.

5. References
ANALYTICAL APPROACH FOR ESTIMATION OF WAVE TRANSMISSION COEFFICIENT FOR $\pi$-SHAPE FLOATING BREAKWATER

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1. Introduction
This study presents a simplified analytical approach, based on power transmission theory [9], to estimate the transmission coefficient of a $\pi$-shape floating breakwater (FB) with finite width. In evaluating the transmitted wave power, this approach considers both the incident wave kinetic power and the heave oscillation of the FB. Additional power due to the acceleration of the floating body and the hydrodynamic mass increases the transmitted wave power behind the FB and consequently increases the transmission coefficient. The proposed theoretical approach is validated using laboratory-scale experimental data obtained from the literature for $\pi$-shape FB. The results of the proposed approach are in good to excellent agreement with those of experimental studies. In addition, the reliability of the proposed approach is assessed by comparing its results with those of other theoretical models. The effects of sea depth, relative draft, and incident wave height on the magnitude of the transmission coefficient are examined. It is found out that effect of the incident wave height distinguishes the proposed model from others in the existing literature.

2. Methodology
Wave decay on FB can be related to the ratio between incoming wave height $H_i$ and transmitted wave height $H_t$. As a wave passes a FB, it decays and attains new height, $H_t$. [1, 6, 8]. Such wave attenuation can be expressed by the wave transmission coefficient $K_t$ as

$$K_t = \frac{H_t}{H_i} \tag{1}$$

The incident wave energy includes potential, kinetic, and wave-induced pressure energy. Under the assumptions of linear wave theory, the energy transport (wave power) in the wave propagation direction is estimated by considering only the power resulting from the work done by the wave-induced pressure ignoring the transport of kinetic energy (wave kinetic power), owing to approximation to a certain order of [2]. However, in the presence of FB, wave kinetic power should be considered because their heaving behavior significantly affects the total transmitted kinetic power. Wave kinetic power accumulates with the kinetic power generated from the heaving oscillation of the FBs. Both kinetic powers increase the total transmitted power and hence, the transmission coefficient. Therefore, the total incident wave power $P_{tot}$ comprises incident wave kinetic power $P_{I.1}$ in addition to wave-induced pressure power $P_{I.2}$.

Part of these two wave powers is transmitted from beneath the FB draft $D$ to the lee side (see Figure 1). The transmitted part includes the transmitted wave kinetic power $P_{T.1}$ and wave-induced pressure power ($P_{T.2}$). In addition to these two transmitted powers, $P_{T.3}$ (kinetic power per unit FB width resulting from the heaving motion of the FB) is transmitted in the $x$ direction to the lee side. This power consists of two parts: the kinetic power of the heaving body of the FB and the kinetic power of the hydrodynamic mass that accelerates simultaneously with the floating body.

Figure 1. Process of power transmission theory.

The transmitted wave (at the lee side of the FB) carries a total power that equals the total transmitted power ($P_{tot}$ = $P_{T.1}$ + $P_{T.2}$ + $P_{T.3}$). The transmitted wave becomes the incident wave toward the coastline with a height of $H_i$, which is attained after attenuation of the seaside incident wave. The lee-side incident wave carries a total power $P_{LS}$ that comprises the wave-induced pressure power and wave kinetic power. $P_{LS}$ is a function of $H_i$, and once the value of $P_{LS}$ is found (i.e., $P_{LS}$ = $P_{tot}$), the value of $H_t$ can be obtained and transmission coefficient $K_t$ can be calculated using Eq. (1).

3. Model Validation
The proposed approach was evaluated using laboratory data that was obtained from an experimental study on the
hydrodynamic performance of π-shape FBs conducted by Koutandos et al. (2005) [4]. The experimental data is shown in Table 1.

Table 1. Full-Scale Properties and Wave Conditions for Experimental Study.

<table>
<thead>
<tr>
<th>Draft D (m)</th>
<th>Width B (m)</th>
<th>Wave Height Hi (m)</th>
<th>Wave Period T (s)</th>
<th>Water depth d (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.5</td>
<td>10</td>
<td>1.5</td>
<td>14.9, 12.4, 9.5, 7.5, 6.4, 5.4</td>
<td>10</td>
</tr>
</tbody>
</table>

In addition, earlier theoretical approaches from Macagno (1954), Kriebel and Bollmann (1996), and Ruol et al. (2013) [3, 5, 7] were evaluated using the same laboratory data; the results were compared with those of the approach proposed in this study.

Figure 2 compares the experimental and theoretical transmission coefficients. The curves show the relationship between relative FB width $B/L$ and transmission coefficient $K_t$.

4. Discussion

4.1. Effect of Draft and Water Depth

The results show that as the draft increases (a higher value of $D/d$), the transmission coefficient decreases, indicating better blocking wave power transfer and successful additional attenuation of the transferred wave height. For a constant relative draft value, another important finding is the reduction of the transmission coefficient with respect to increasing sea depth. Thus, the FB’s wave reduction performance is better in deep water owing to the change in wave orbital velocity with respect to changes in water depth.

4.2. Effect Incident Wave Height

Based on the proposed approach, as the incident wave height increases, so does the transmission coefficient of the FB under the same wave period condition. This result is expected, as incident wave height $H_i$ is a factor that affects the calculation of transmission coefficient $K_t$. The kinetic power resulting from the heaving movement of the FB and its hydrodynamic mass is a function of $H_i$, and the kinetic power increases with wave height, leading to additional power transmission.

5. Conclusion

The results obtained using the proposed approach agreed with the small-scale results in the literature for waves with long periods and low steepness, in accordance with linear wave theory. Some scatter is to be expected, because it is difficult to adequately consider the effect of mooring stiffness in a simple approach. Part of the scatter is also attributed to scale effects that are likely to influence the transmission behavior, especially for very high waves, ignoring overtopping. Therefore, this approach may be inaccurate when applied to waves higher than the freeboard of the FB, especially in the case of full-scale structures.

6. References

CONDITION ASSESSMENT OF HARBOR AND OFFSHORE PLATFORMS USING FIBER OPTIC SENSORS

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In this research work FBG has been used as a new kind of sensing element for health monitoring in the large composite and concrete civil infrastructures. The core problems focus on the novel proposed expert system using FBG sensors application. As a new kind of sensor for structural health monitoring system, Popularization and generalization of FBG is still a challenge for researchers.

This research work shows that FBG sensors have become one of the key sensors in structural health monitoring (SHM) and will take the place of some conventional electrical sensors.

In this work, the performances of a proposed solution, based on expert fiber Bragg-grating were tested and directly compared both in laboratory and in field conditions. The results are presented and discussed, aiming at the assessment of generalization the main characteristics of this technology for quay wall and other similar type, structural health monitoring applications, and also taking into account the principal requirements in-field of civil engineering applications.

The efficiency of a monitoring method based on long-gauge sensors is illustrated through an application at the Shahid Rajaee port in Iran. For developing the present expert system the knowledge domain is organized so that the information can be structured in the computer program for effective use and originated from source of knowledge should be the domain expert. To design and develop knowledge based expert system, the specific knowledge domain or the subject domain is acquired. In the proposed scheme the change of reference FBGs signal with expert system, implies that something within the subsea concrete structure has altered and diagnosis is made.

A distributed on-line temperature and strain fiber sensing system based on the fiber Bragg grating (FBG) technology is presented and investigated experimentally for expert monitoring of the Harbour and Getty Structures.

The implementation of fiber optic sensing (FOS) technology is the solution to incapability of traditional electrical stain gauges in a large sensing network for structural health monitoring of different building and civil engineering structures. It is Difficult and sometimes not possible to measure a fault or crack in the offshore or onshore construction for various reasons, including climate and environmental conditions.

The present work survey the principles and a criterion of the diagnosis signal processing and introduces these achievements to an expert system technique using Fiber Bragg Grating (FBG) sensors to monitor the dynamic strain and temperature of Shahid Rajaee harbour.

This article aims to ensure the protection, safety and health monitoring of the concrete structure quay wall in the Shahid Rajaee harbour using proposed method of measuring defected parts, incorporating the Fiber Bragg Grating sensors and extracting knowledge from expert system.
References


THE EFFECT OF EXPOSURE CONDITIONS AND CONSTRUCTION METHODS ON THE CHLORIDE DIFFUSION INTO CONCRETE IN THE PERSIAN GULF REGION

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1. Introduction

The reinforced concrete structures present in marine environment are susceptible to reinforcement corrosion due to the presence of chlorides. The chlorides disposed in marine environment come mainly from seawater. Its contact with concrete structures can happen directly by seawater or through the marine aerosol. After contact, the chlorides are deposited on the surface of concrete and can penetrate it through different mechanisms, depending on the characteristics of materials, construction methods and the exposure conditions in which it operates [1,3].

Due to different characteristics of attack, resulting mainly from different accesses of oxygen and humidity, the marine environment is divided in different zones of aggressiveness. These zones are segmented having the sea level as reference, and are defined as follows [2]:

- Atmospheric zone: Concrete suffers the action from marine aerosol, however the structure is not affected directly by water splashes. The winds can carry the salts in the form of solid particles or as droplets of saline solution. The quantity of salt present decreases as a function of the distance from the sea, suffering influence of speed and prevailing wind directions.

- Splash zone: Zone immediately above the maximum level of intertidal variation and concrete is directly affected by water splashes. The height of the splash zone is a function of the wave height and speed/direction of wind. The most significant damage is produced by reinforcement corrosion by chlorides. The splash zone is subjected to cycles of wetting and drying and this fact becomes more significant as water evaporates and the salt remains into the concrete.

- Tidal zone: It is the concrete zone between the min/max levels of tide. This region is also subjected to the wetting and drying cycles action. Degradation occurs due to the action of aggressive salts (chemical attack), reinforcement corrosion, waves’ abrasive action and other substances in suspension, and attack of microorganisms.

Ghods et al. [4] consider that exposure zones play an important role in the service life design of concrete structures that should be concerned as a main input parameter in models. This means that in many cases, a model that is related to only one type of exposure zone relative to the chloride attack, should not be generally used. Thus, the expected life of a structure in a marine environment needs to be considerable different depending on the 4 exposure zones previously mentioned.

The aim of this study is assessment of the basic parameters in long term chloride penetration in the Persian Gulf region. So, five year-old concrete jetties located in Imam Khomeini Port Complex were investigated depending on two parameters, a) construction method: in-situ and precast, b) exposure conditions: atmospheric, splash and tidal zones. Also some accelerated durability tests were carried out on standard samples with same concrete mixture in laboratory. Finally, corrosion initiation time was estimated by five service life prediction models.

2. Description of Structures and Materials

With a history of more than 80 years, Imam Khomeini Port is considered as one of the largest and the most important commercial port of Iran which handles about half of all non-oil trades of the country. Bandar Imam Khomeini (BIK) is located on the tidal lands of the northern parts of Khure Musa which provides a safe and protected and approach channel for vessels up to 150000 ton. The under investigated structures are located in southern part of BIK zone and are called Eastern and Western jetties. These on-shore structures were built in 1927-41 and were totally rehabilitated in 2004-07 (fig. 1).

Based on the official documentary information, investigated parts of structures have the same concrete mixture and were repaired five years ago. The concrete mixtures containing type I( PM) blended cement from
Bushehr company and silica fume (SF) with 89% SiO₂ and pozzolanic activity index 103%. A mix ratio of 1:2.1:2.6 by mass was used with water to binder ratios of 0.35. Because of low w/b ratio in mixture, polymer-based superplasticizer was used to achieve desired workability.

3. Experimental Program and Results

In order to analyze long term chloride penetration into concrete and predict corrosion initiation time, two series of experiments were performed. The first series were on standard samples with similar concrete mixture of field structures in laboratory that is presented in table 1.

<table>
<thead>
<tr>
<th>Compressive strength (28-day) (MPa)</th>
<th>Surface Resistivity (KΏ.cm)</th>
<th>Rapid Chloride Migration Test (mm²/year)</th>
<th>Rapid Chloride Penetration Test (Coulomb)</th>
</tr>
</thead>
<tbody>
<tr>
<td>BS-1881 part 11E</td>
<td>55</td>
<td>NT Build-492</td>
<td>ASTM C1202</td>
</tr>
<tr>
<td>36</td>
<td>169</td>
<td>1365</td>
<td></td>
</tr>
</tbody>
</table>

The field surveys are applied on three exposure conditions of precast (P) and In-situ (I) structural members included: atmospheric (A), splash (S) and tidal (T) zones. Also, each measurement is repeated at two point (1, 2) in specified structural members. Additionally, abbreviations PTB, ITS and PTC is related to previous study on samples included: atmospheric (A), splash (S) and tidal (T) zones. Also, the differences of Cₛ, generally indicates how concrete quality and exposure conditions can significantly affect the chloride penetration. As it well known in previous studies, in tidal zone near high water level due to wet/dry cycles, resulting in a successive supply of chlorides by wetting with sea water, and salt crystallization by drying.

4. Discussion

Equations governing the diffusion are based on Fick’s laws in the form of differential equation and can be solved in various forms. Some consider this equation as constant diffusion coefficient (crank solution) [3] that is called apparent diffusion coefficient used for estimations. It is generally noted from figure 3.top, Dₚₑₚ parameter is more or less the same at the majority of samples and the exposure conditions didn’t indicate considerable effect on it after five years of exposure.

5. Conclusions

As a main conclusion, the ingress of chlorides in concrete is different in each exposure zone and influenced by microclimatic factors such as temperature, relative humidity, wind and solar radiation. However, it is noteworthy that most of the mathematical models of service life prediction available in literature do not consider all these variations in their formulations. The difficulty of monitoring these variations in the locals of the concrete structure exposure is recognized, but this effect should be considered in models and technical standards related to the life of these structures.

6. References

PHYSICAL MODELING OF RUBBLE MOUND BREAKWATER DEFORMATION ON SOFT SEABED

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1. Introduction
Soft seabed in some coastal areas causes noticeable problems in the course of construction of marine structures such as rubble mound breakwaters. Pouring rubbles for constructing this type of breakwaters, rock particles sink into the soft soil and a foundation for the breakwater cannot be developed [1] causing excessive settlement, instability of breakwater and waste of materials. Except that the individual penetration of rock particles into the soft soil, other phenomena such as immediate deformation and consolidation of the seabed, flow of soft material into pores of rock fill and erosion of the seabed effect the total waste of rock material [2].

Before choosing the best technique to encounter the problem, it is essential to make an accurate estimation of breakwater deformation and volume loss. There are numerous projects that underestimating the rock material volume required for construction lead to lack of resources, contracting problems, funding difficulties and at least delay in construction process because of need for redesign and change the approach. On the other hand, overestimating the volume loss can lead the project to an expensive improvement technique.

In this paper at the first step, data gathering from several projects with considerable waste of rock material was carried out and as case histories took into account. Then, a 1g physical modeling test procedure was suggested for a more detailed study on the subject.

2. Case Histories
Many coastal projects in the south and north of Iran have encountered with soft beds problem. Here the available records of 7 breakwaters and/or access roads have been presented: Hendijan, Mohammad Ameri, Nakhle-Nakhoda, Shenas, Shah-Abdullah, Bandar-Abbas, and Deylam [3-9]. The thickness of the very soft layer was variable from 3 to 12 m in various projects and the maximum settlement reported was about 8 m.

There are often some shortcomings and ambiguities in the reported information:
- Time dependent data for deformation records during construction and afterward usually is not available.
- The geotechnical characteristics of the seabed which have been documented in the geotechnical report of the project, usually has been carried out before construction and in a wide area, and the soil parameters at the exact recording position of settlements are not available. Even sometimes enough studies have not been performed in the project area.
- Generally, reported data is related to the settlement and the amount of sinking stones in seabed and total volume loss of the materials have not been presented.

Considering the above, some physical modeling tests were proposed.

3. Proposed 1g Physical Modeling Tests
The amount of stone material required to construct a breakwater can be obtained from multiplication of the designed section volume to the specific gravity measured for the rock material (\( W_{initial} \)). But during construction, the pre-designed section changes and consuming material becomes much higher than the initial estimation (\( W_{Total} \)). The deformed cross-section can be divided into the following areas (see Figure 1):

- Material placed in the deformed bed. 
- Material sanked into bed.
- Uplift area in the pre-designed body.
- Extra material due to the toe deformation.
- Extra material added to the initial estimation to reach the design surface.

Between the above areas, the following relationships can be written:

\[
W_{Total} = W_{initial} + W_{Extra}
\]

\[
W_{Total} = W_{initial} + W_{Over} + W_{Settled} + W_{Penetrated} - W_{Uplift}
\]

Then:

\[
W_{Extra} = W_{Over} + W_{Penetrated} - W_{Uplift}
\]
Figure 1. Different area definitions of deformed breakwater modeled in 1g physical modeling box. 

$W_{\text{Extra}}$ is known during construction. Also, by integrating from the deformed surface of the soft seabed, $W_{\text{Settled}}$, $W_{\text{Uplift}}$ and $W_{\text{Over}}$ can be obtained and, according to Equation 3, sinking stoned into the bed ($W_{\text{Penetrated}}$) can be calculated, too. In this way, all areas of Figure 1 will be known.

A sample of the tests performed inside the physical modeling chamber of Kharazmi University is shown in Figure 2. Time dependent vertical deformation of bed surface for the test is presented in Figure 3 and the deformed shape and consuming material for each sector have been presented in Figure 4 and Table 1, respectively.

Figure 2. 1g physical modeling test performed inside the physical modeling chamber of Kharazmi University.

Table 1. Consuming relative to design material for two average undrained shear strengths of bed ($C_{u,\text{ave}}$).

<table>
<thead>
<tr>
<th>$C_{u,\text{ave}}$ (kPa)</th>
<th>$W_{\text{Settled}}$</th>
<th>$W_{\text{Uplift}}$</th>
<th>$W_{\text{Over}}$</th>
<th>$W_{\text{Extra}}$</th>
<th>$W_{\text{Penetrated}}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.25</td>
<td>5.9</td>
<td>2.2</td>
<td>1.1</td>
<td>11.6</td>
<td>6.7</td>
</tr>
<tr>
<td>1.10</td>
<td>6.2</td>
<td>0.4</td>
<td>0.9</td>
<td>15.7</td>
<td>9.0</td>
</tr>
</tbody>
</table>

In the proposed 1g physical modeling tests, by measuring and calculating each of the above quantities, the role of each sector in the total amount of consuming stone materials can be studied. Also, their relationship with design parameters such as undrained shear strength of clay, geometric dimensions of the breakwater and the type of rock materials used in the construction can be investigated.

Figure 3. Time dependent vertical deformation of bed surface (Gauge numbering is done from the right).

Figure 4. Deformed shape diagram used for integration and calculation of areas.

4. Acknowledgements

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5. References


