AN INSIGHT INTO THE SETTLEMENT MECHANISMS OF RUBBLE MOUND BREAKWATER ON MUDFLAT USING DEM

Hamed Bayesteh¹ and Roham Mansouri Boroujeni²

1) Ph.D, Department of Civil Engineering, University of Qom, Qom, Iran, h.bayesteh@qom.ac.ir
2) M.sc, Department of Civil Engineering, University of Qom, Qom, Iran, rohammb71@gmail.com

1. Introduction

Because mudflats are compressible, substantial settlement will occur during construction of a rubble mound breakwater at such a site. The use of geosynthetics as reinforcement is common during breakwater construction [1]. A suitable numerical model should be selected to provide insight to the mechanisms of settlement of such a reinforced rubble mound breakwater. Although numerical methods based on continuum mechanics have been used to model the behavior of embankment systems, they cannot provide insight into settlement because they cannot approximate the individual movement of the particles. rubble mound breakwaters consist of rockfill, which is naturally discontinuous (Figure 1). Moreover, basing the numerical method on discontinuum mechanics to simulate the behavior of rockfill has not been achieved. The aim of the current study was to evaluate the control mechanisms of settlement of rubble mound breakwaters and the effect of geosynthetics on their performance. A full-scale case study of a rubble mound breakwater constructed on soft soil is considered for which the effect of the pattern, size and number of layers of the geogrid on performance was monitored. A series of numerical simulations using the discrete element method were done to evaluate settlement caused by submersion of the particles into the seabed. The effects of geosynthetic parameters such as stiffness, number of layers and length of geosynthetic material were analyzed. The results showed that the submersion of the individual particles of rockfill into the seabed is the main mechanism of settlement in an unreinforced breakwater (Fig. 1). This value is higher than for classic settlement (elastic or consolidation) [2]. However a reinforced geogrid acts as a separator and prepares a continuous base for the rockfill to prevent individual movement. Classic settlement is main mechanism of settlement which controls the behavior. It was shown that the length and number of geogrid layers had greater influence over decreasing settlement than the tensile strength of the geogrid.

2. Geometry of the Model

In order to model the geometry of the breakwater, a trapezoid with a 7 m height, 7 m crest width and side slopes 1:1 was considered (Figure 2). As the rockfill materials usually are rough, they are resistance to rolling. The particles used in this study were circular so the rolling resistance linear contact law was used to simulate the rockfill particles [2]. The parameters of a model are usually calibrated using known macro-mechanical responses. For calibration the rockfill particle, the particles deposited from a hopper above the base wall (Figure 3).

3. Settlement Due to Submerging Mechanism

The total settlement was 4.2 m which is compatible with the monitored data in an Iran projects. This finding shows that a large deformations and large strain can be simulated
with DEM [4]. As described in the theoretical background section, this large value of the settlement was induced due to the individual submerging of the rockfill, because the breakwater media is naturally discontinuum. Fig. 4 plots a part of the displacement vectors of the particles after settlement of the breakwater. It can be seen that displacement vectors for rockfill are towards the seabed while for seabed particles, the displacement vectors are laterally or upward due to the soil heaving. Submerging of the rubble mound into seabed leads to lateral movement of the seabed particles and inducing shear band.

Figure 4. Settlement due to the submerging of the breakwater particles

4. Settlement Due to Continuum Mechanism

In order to have an insight to the continuum settlement of a rubble mound breakwater, it is necessary to prevent individual settlement of the rockfill. As described in Figure 1, using geosynthetic below a rubble mound section, cause uniformity between rockfill and can acts as a continuum media. The results show that using the geosynthetic reinforcement leads to the decrease of the breakwater settlements at the seabed. This function leads to prepare a continuum media which is prevent separate submerging of the rockfill into seabed. Thus, the induced settlement in this situation (0.65 m), was due to the continuum settlement according to the subsoil geotechnical parameters. It is clear that the vertical force due to the self-weight of the rubble mound creates a tensile force in geogrid reinforcement which leads to transfer vertical load to horizontal load. Figure 5 shows the force chains for the reinforced rubble mound breakwater. By decreasing the length of the geogrid, the settlement increases. The results show that although there is not deference between the 45 m and 30 m length of the geogrid, but the value of the seabed heaving increase by reducing in the geogrid length.

Figure 5. Force chains for a reinforced rubble mound breakwater

5. Conclusions

In this paper, in order to have an insight to mechanism of settlement a rubble mound breakwater, a discrete element simulation was performed. According to the discontinuum based of DEM, the results indicated that this method can simulate the submerging particle into seabed. The induced force chains in the model show that the local large forces on the seabed particles, lead to push away the subsoil particle so the individually large settlement of the rockfill occurs. A series of simulations were conducted to evaluate the effect of the geogrid length, the reinforcement layers and the geogrid tensile strength on the performance of reinforced breakwater. The results shows that by increasing the length of the geogrid, the settlement decrease. Whereas the vertical load (due to the self-weight of the breakwater) transferred to a horizontal load through a longer length of the geogrid, the settlement reduces. Also, by increasing the geogrid layers, the settlement decreases. Indeed the more geogrid layers lead to reduce the tensile force in one geogrid. Redistribution of the contact force in the geogrid, causes suitable local interaction between seabed and lower geogrid layer and causes reduction in the settlement. By the way, the overall tendency is the reduction of the breakwater settlement with the increase of the geogrid length can be concluded. But there is an upper bond for the geogrid tensile strength for preventing over reinforcement.

Figure 6. Effect of the geogrid length on the breakwater settlement

6. References

EXPERIMENTS ON SCOUR AROUND SUBMARINE PIPES AND STUDY OF KC NUMBER UNDER SOLITARY WAVES

Hassan Vosoughi and Hooman Hajikandi

1. Introduction

Pipelines are a very opportune means to transport oil, gas, water, waste water or other hydrocarbons from the sea bed or river crossing is current in water environments. Pipelines are widely used coastal structures, and scour around them can influence their stability. Pipelines installed on sandy sea beds in coastal areas are exposed to wave and current action.

Sumer and Fredsøe [3] demonstrated that the relative scour depth, S/D, is remarkably well correlated with the Keulegan–Carpenter (KC) number. The scour increases with increasing KC number. S/D can be expressed in terms of the KC number as follows:

\[
\frac{S}{D} = 0.1\sqrt{KC}
\]

Çevik and Yüksel [1] studied scour below submarine pipelines in shoaling regions for normal-incidence regular waves using four different rigid pipes. They conducted tests of both horizontal beds and 1/5 and 1/10 sloping beds in shoaling regions. They investigated the effects of several parameters on the non-dimensional scour depth. They identified the wave height, wave period and pipe diameter as the dominant parameters in the development of scour. As each of these parameters increases, the non-dimensional scour depth increases. They proposed an expression for the equilibrium scour depth in terms of the KC number for both sloping (1/5 and 1/10) beds and horizontal bottoms.

\[
\frac{S}{D} = 0.11KC^{0.45}
\]

Kızılöz et al. [2] studied scour around rigid submarine pipelines under irregular wave attack on horizontal and (1/10) sloping beaches and relative scour in regular and irregular wave attack. Sumer and Fredsøe [4] studied the influence of irregular waves on scour using the JONSWAP wave spectrum. They proposed the following equation for the scour depth in the case of irregular waves:

\[
\frac{S}{D} = 0.14\sqrt{KC} = 0.1\sqrt{\frac{U^2T}{D}}
\]

Most of the experimental studies of scour around submarine pipelines to date have been conducted under conditions of regular and irregular wave attack. The purpose of this study is to model the local scour depth around a fixed submarine pipeline under solitary wave attack on horizontal and sloping conditions.

2. Experimental Setup and Procedure

Figure 1 shows a schematic of the experimental setup. As it is shown in Figure 1, a sluice gate is installed in the middle of the flume, approximately 5 meters from the inlet pipe. Prior to each experiment, water is reserved upstream the gate. The opening velocity of the gate is controlled by means of a rotary-motor. Total 40 experimental runs are performed for two conditions of horizontal flume and 0.6% longitudinally slope flume. All the experiments are conducted for two identical sediment layers the mean sediment size of which are 3.1 mm and 5.8 mm respectively. The Thickness of the sediment layer is 15 cm before the beginning of the experiments. Rigid PVC model pipe with diameter \( d_p = 4\) cm were placed 1 mm from the channel sides. To avoid wall effects, all measurements were made in the middle of the cylinder axis. At the end of the flume a tailgate with adjustable height was used to create desired depth in the flume.

![Figure 1. Experimental setup-Sectional](image-url)

At the beginning of each experiment the inlet flow rate was controlled and adjusted to provide the desired normal depth above the sediment recess. The sediment recess was located 0.5 m from the sluice gate. In order to provide a flat bed, artificial bed was made in the flume before and after the sediment recess. By closing the sluice gate, water is stored upstream the gate and when the water head reaches the desired head it is released by sudden opening of the gate.
3. Results
The results of extensive experimental runs on scour around submarine pipes under solitary waves generated by suddenly releasing of flow from an upstream sluice gate. The effect of relevant parameters including depth of flow before the sluice gate, channel slope, wave Froude number and sediment gradation on the scour hole is investigated. In this study, 40 random wave tests were conducted to study the scour depth below submarine pipelines and exposed to normal incident solitary wave attack. In this study proposed an expression for the equilibrium scour depth in terms of the KC number for both sloping beds and horizontal bottoms.

Relative scour depth increases with increasing wave height, wave period, and pipe diameter and scour depth can be expressed by the KC number for a horizontal and sloping bed. The following expression is proposed for a fixed pipe in contact with a live bed under solitary wave attack.

Figure 2 shows the variation of relative scour depth with KC number. This figure includes regular wave data of Sumer and Fredsøe [3] and irregular wave data of Kızılöz et al. [2] and those of this study which are the results of solitary wave data. The present data are located above KC numbers very different from the regular and irregular wave data Sumer and Fredsøe [3] and Kızılöz et al. [2].

Founded on the present experimental data, a new equation (Eq. 4) is suggested for dependence of S/D on KC, relating the maximum equilibrium scour depth for the live bed condition for a rigid pipe fixed initially in contact with the bed under solitary wave attack.

\[
\frac{S}{D} = 0.15KC^{0.83}
\]  

![Figure 2. Variation of maximum scour depth with the Kleugan-Carpenter number. Experiment: Present solitary wave data. Regular data from Sumer and Fredsøe (1990). Irregular wave data from Kızılöz et al. (2013).](image)

4. Reference


IMPLEMENTING THE “SELF-UPENDING CONCEPT” FOR THE JACKETS IN PERSIAN GULF WITH THE NEW ARRANGEMENT OF BUOYANCY TANKS

Seyed Ali Amid1, Majid Sohrabpour, PhD2 and Sara Allahyaribeik3

1) Marine Systems Technologists Limited, Tehran, Iran, ali.amid@yahoo.com
2) Marine Systems Technologists Limited, Tehran, Iran, sohrabpr@gmail.com
3) Science and Research Branch, Islamic Azad University, Tehran, Iran

1. Introduction

The Crane-assisted upending procedure for installation of jacket platforms requires heavy floating cranes. The alternative way to install jacket platforms is the “self-upending” procedure, in which belt-shaped Buoyancy tanks are deployed to self-upend the jackets. This belt-shaped buoyancy tanks comprise different tanks welded together. Hence, they cannot be deployed to self-upend other jackets which have different dimensions. The new positioning of buoyancy tanks on Jackets has been presented in this paper which makes the self-upending procedure much more cost-effective thanks to the Modularization of Buoyancy Tanks. By virtue of this method, buoyancy tanks are no longer welded together; therefore, they can be used to self-upend the other jackets with different dimensions. Furthermore, before positioning jackets on the seabed, the stability of floating jackets increases due to the new arrangement of Buoyancy tanks on jacket. Parametric study demonstrated that, in order for the jacket to assume its vertical position soon after launching, center of gravity of jacket must be located 1.92 meter below the center of buoyancy. Considering the jacket located horizontally on launching barge, two buoyancy tanks with dimensions of 2 m outside diameter and wall thickness of 1.5 cm control the transverse stability of jacket during self-upending. Four buoyancy tanks are deployed to comply the minimum bottom clearance criteria. Table 1 shows the jacket launch results.

Table 1. Launch results- jacket example A

<table>
<thead>
<tr>
<th>Time(sec)</th>
<th>Pitch</th>
<th>Roll</th>
<th>Yaw</th>
<th>Mudline clearance(m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>148</td>
<td>-56.9</td>
<td>-7</td>
<td>0</td>
<td>5.45</td>
</tr>
<tr>
<td>156</td>
<td>-75</td>
<td>-6.4</td>
<td>0</td>
<td>6</td>
</tr>
<tr>
<td>164</td>
<td>-86</td>
<td>2.6</td>
<td>1.8</td>
<td>7.57</td>
</tr>
<tr>
<td>189</td>
<td>-82</td>
<td>1.3</td>
<td>2</td>
<td>7.18</td>
</tr>
</tbody>
</table>

Four phases are considered in the launch analysis, tipping position occurs at time 114, separation occurs at 124 sec, and the final position of jacket occurs at 189 sec. Therefore, it takes 65 sec for the jacket to be self-upended. The minimum bottom clearance during the launching process is 5.45 meters occurring 148 seconds after the launch is commenced. Table 2 shows jacket’s final orientation with its metacentric properties.

Table 2. Jacket’s final position- jacket example A

<table>
<thead>
<tr>
<th>Jacket final orientation</th>
<th>Metacentric properties</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pitch</td>
<td>Roll</td>
</tr>
<tr>
<td>-82 deg</td>
<td>1.3 deg</td>
</tr>
</tbody>
</table>

Transverse as well as longitudinal metacentric heights are 4.17 and 6.34 m correspondingly, which are well above the minimum criteria of 0.5 and 0 m [3].

2. Example A: Jacket in 37.6m Water Depth

Self-upending concept has been implemented for 662 tons Jacket in Persian Gulf; the Software Sacs is implemented to perform 3-D Time-domain Launch and Self-upending Analysis [1]. Figure 1 shows the 662-ton jacket with its modular buoyancy tanks designed according to DNV-RP-C202 [2] as stiffened circular cylindrical shells. It has been concluded that, in order for the jacket to assume its vertical position soon after launching, center of gravity of jacket must be located 1.92 meter below the center of buoyancy. Considering the jacket located horizontally on launching barge, two buoyancy tanks with dimensions of 2 m outside diameter and wall thickness of 1.5 cm control the transverse stability of jacket during self-upending. One, with the same dimensions, perpendicular to the longitudinal axes of launching barge, assists jacket reaches its vertical position- pitch angle of 82 degrees. Four buoyancy tanks are deployed to comply the minimum bottom clearance criteria. Table 1 shows the jacket launch results.
3. Example B: Jacket in 70m Water Depth

Launch and Self-Upending analyses have been implemented for a larger jacket of 1828 tons dry weight in Persian Gulf. Buoyancy tanks with outer diameters of 2.5 and 2 m and wall thickness of 1.5 cm with a total weight of 176 Tons are implemented in the design. As it could be observed in figure 2, the arrangement of buoyancy tanks is the same as the 662-ton jacket. Table 3 shows the jacket launch results.

<table>
<thead>
<tr>
<th>Time(sec)</th>
<th>Pitch</th>
<th>Roll</th>
<th>Yaw</th>
<th>Mudline clearance(m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>279</td>
<td>-3</td>
<td>-2</td>
<td>0.2</td>
<td>44</td>
</tr>
<tr>
<td>291</td>
<td>-41.5</td>
<td>-7.4</td>
<td>-0.4</td>
<td>21</td>
</tr>
<tr>
<td>325</td>
<td>-79</td>
<td>-12</td>
<td>-2.8</td>
<td>5.4</td>
</tr>
<tr>
<td>348</td>
<td>-82</td>
<td>3.5</td>
<td>0.0</td>
<td>7.39</td>
</tr>
</tbody>
</table>

Table 3. Launch results- jacket example B

Table 4 shows the jacket’s final orientation with its metacentric properties.

<table>
<thead>
<tr>
<th>Pitch deg</th>
<th>Roll deg</th>
<th>Yaw deg</th>
<th>MC m</th>
<th>LGM m</th>
<th>TGM m</th>
<th>BG m</th>
<th>RB %</th>
</tr>
</thead>
<tbody>
<tr>
<td>-82</td>
<td>3.5</td>
<td>0</td>
<td>7.39</td>
<td>4.26</td>
<td>3.22</td>
<td>2.64</td>
<td>21%</td>
</tr>
</tbody>
</table>

Figure 2. Jacket model example B

4. Conclusion

Implementing the self-upending concept with the new arrangement of buoyancy tanks is quite attractive due to the following reasons:

1) The amount of time it takes both jackets to be self-upended is 69 seconds.
2) Several more launch simulations were performed to investigate the sensitivity of varying jackets parameters—that is—weight and CG sensitivity. Some of which are depicted in table 5.
3) Damaged cases are also considered, in which three buoyancy tanks of the jacket example A were allowed to free flood at the point of submersion. In this case of failure, the Pitch angle of Jacket A would be 79.7 degrees and longitudinal and transverse metacentric heights would be 4.74 and 1.48 meters correspondingly. It shows the considerable stability of the self-upended floating Jacket even with the failure of 3 compartments thanks to the new arrangement of buoyancy tanks.
4) This new arrangement of buoyancy tanks presents and suggests Cost-effective [4] and Secure Installation of jacket platforms through self-upending concept. Buoyancy tanks are selected and designed in an optimal as well as modular manners with the same dimensions, so that they could be used for other projects.
5) Before the positioning of self-upended floating jacket on the sea bed, Current with the velocity of 1.4 m/s has been applied to investigate the behavior of the floating jacket example A. In this case the velocity of floating jacket in x direction would be 0.7 m/s, which can be completely kept in the position by tug boats.
6) In order for the jackets to assume their nearly-vertical positions, there is no need to flood any compartment of the jackets because of the new arrangement; however, before positioning the jackets on the sea bed, two legs of the jacket example A and two legs of the jacket example B must be flooded with the ratio of 60 and 30% correspondingly without any assistance of the floating cranes.

Table 5. Weight tolerance for jacket example A

5. References

RELIABILITY ANALYSIS OF M5-GP PREDICTION MODELS FOR UPLIFT CAPACITY OF SUCTION CAISSONS

Ali Derakhshani

1) Department of Civil Engineering, Faculty of Engineering, Shahed University, Tehran, Iran
adera@shahed.ac.ir

1. Introduction

Different models for predicting the uplift capacity are available in the technical literature. The recommended models for uplift capacity prediction do not account for the input uncertainties affecting the results.

Generally, uncertainty in the inputs, is effective on the reliability of the system responses. Suction caisson is among the widely used engineering systems that its uplift capacity is influenced by uncertainties.

To improve the reliability of the designs, the input uncertainties need to be considered directly [1]. For this purpose, various approaches have been used for analyzing the engineering systems ranging from engineering judgments to the complicated statistical and intelligent methods.

The fuzzy sets theory can be considered as the most general method to be used for uncertainty analysis in engineering [2]. Examples of successful civil engineering studies that utilized the fuzzy sets theory include the uncertainty analyses in water pipeline networks [3, 4], structural engineering [5, 6] and geotechnical engineering [7].

2. Methodology

The uncertainties in the inputs can influence the estimated suction caisson uplift capacity. For the reliable design, analysis of the possible uncertainties and their influence on the uplift capacity is necessary. In this paper, the uncertainty analysis of the suction caisson uplift capacity is done based on the Fuzzy sets theory using fuzzy numbers.

First, the uplift capacity of the suction caisson is estimated using the various approaches without consideration of uncertainty of input parameters. Then the suction caisson uplift capacity is evaluated considering the input uncertainties using the suggested fuzzy approach. Comparison of the results of different models shows that how the input uncertainties are spread out over their relationships.

3. Data

The governing parameters with considerable influence on the uplift capacity of suction caissons ($Q$) are shown in Figure 1. The database used in this study consists of the experimental test results reported by Rahman, Wang [8]. This database has been employed in different researches conducted to develop models for predicting the uplift capacity of suction caissons. One recent approach is to employ M5 model tree together with GP (M5-GP) as used by Derakhshani [9] to propose two robust methods (Table 1).

Figure 1. Suction caisson geometry.

Table 1. M5GP models [9].

<table>
<thead>
<tr>
<th>Model</th>
<th>Formula</th>
</tr>
</thead>
<tbody>
<tr>
<td>M5GP1</td>
<td>$Q = \frac{\left(\frac{d}{L} + 1\right)(\frac{d}{L} + \theta)^{\alpha}}{T_{2}^{0.8}} \times S_{c}$ for $S_{u} \leq 12.28$</td>
</tr>
<tr>
<td></td>
<td>$Q = 1.036 \times S_{c}$ for $S_{u} &gt; 12.28$</td>
</tr>
<tr>
<td>M5GP2</td>
<td>$Q = 0.272 \times \frac{\left(\frac{d}{L} + \theta\right)}{T_{2}^{0.8}} \times S_{c}$ for $S_{u} \leq 12.28$</td>
</tr>
<tr>
<td></td>
<td>$Q = 1.104 \times \frac{\left(\frac{d}{L} + 1\right)(\frac{d}{L} + \theta)^{0.8}}{T_{2}^{0.8}} \times S_{c}$ for $S_{u} &gt; 12.28$</td>
</tr>
</tbody>
</table>
4. Results and Discussion

Membership functions of M5GP models for CC and RMSE statistical measures are provided in Figure 2. Based on the results given in this Figure, it can be found that although the performance of the two M5GP models is good regarding the crisp (most likely) values, the responses are vulnerable to the input uncertainties.

![Figure 2. Membership functions of M5GP models: a) CC, b) RMSE.](image)

5. Conclusions

Among different prediction models recommended for the uplift capacity of suction caissons, the recent hybrid M5GP models were investigated regarding the influence of input uncertainties. Reliability of the predictions made by these two methods was evaluated and compared by the fuzzy statistical indices including correlation coefficient (CC) and root mean squared error (RMSE). It is inferred that, although these models are remarkably accurate, they may be vulnerable to uncertainties of predictors.

6. References

CHALLENGES OF GAS EXPORT PIPELINE PROJECTS

Maryam Sarajian¹ and Mohammad Reza Bahaari²

¹) School of Civil Engineering, University of Tehran, Tehran, Iran, maryam.sarajian@ut.ac.ir
²) School of Civil Engineering, University of Tehran, Tehran, Iran, mbahari@ut.ac.ir

1. Introduction

In the following research, gas challenges and solutions adopted for gas export pipelines will be studied. In recent decades, many successful projects such as Nord Stream, South Stream, Langeled and Trans-Mediterranean pipelines have been constructed, include lots of creative ideas. Many of those can be implemented in similar projects.

Gas exports to Oman and India are expected to be implemented in coming years. So it would be beneficial to become familiar with experiences of similar projects because future projects may encounter these obstacles too. For instance, in the Nord Stream pipeline, which transports the natural gas from Russia to Germany, existing cables and pipelines, munitions dump, cultural heritage, fishing activities, logistic and shipwreck sites were constraints which have been solved with creative ideas and solutions.

At the present time, more than 50 percent of world's energy consumption is oil and gas. Disparity between consumption and domestic production, makes oil and gas trading more important, for example, about 60 percent of proven natural gas reserves are in Russia and the Middle East while the European Union’s share is just 0.7 percent [1].

Figure 1. Distribution of world's energy consumption in 2016 [1].

A continuing increase in the demand for natural gas within the EU is expected, coupled with a decline in the EU’s own productive capacity and reserves. As a consequence, imports will supply a greater share of total EU consumption. Natural gas import requirements are expected to rise from 314 bcm (billion cubic meters) per annum, corresponding to 58 percent of total demand, in 2005 to 509 bcm, corresponding to 81 percent of total demand, in 2025 [2].

On the other hand, a country like Iran has 18 percent of world's natural gas reserves, as Figure 2 illustrates only small amount of these reserves have been consumed. With the big advantage of being located close the EU and having the large natural gas reserves, Iran can provide a significant proportion of the EU's gas imports. Also due to Iran's geographical location, proximity to markets and the dependency of Iran's budget on oil and gas export earnings, a further increase in exporting is expected.

Usually, the natural gas is transported in a form of gas (NG) through pipelines or in a form of liquefied natural gas (LNG) by specially designed ships. Small amounts of natural gas are also exported on trucks as LNG and as compressed natural gas (CNG) [3], see Figure 3.

Figure 2. Iran's reserves to production (R/P) ratios [1].

So far, Iran does not have the infrastructure to export or import liquefied natural gas (LNG) [4]. The most natural gas transported via pipeline is sweet and less corrosive. Exports or imports through transmission pipelines have distinct advantages in terms of carbon dioxide emissions, traffic and energy efficiency compared to LNG transports [2]. Also LNG transports need infrastructures such as LNG ports and regasification terminals. From an economical
point of view, transmission pipeline systems could be more efficient than LNG transports. There are many factors in the choice of the exporting medium and methods. These factors are specified at the conceptual design stage.

If the pipelines pass through the seas or oceans, special methods and economic considerations must be taken. Investment, type of contracts, political conflicts, route and diameter selection, environmental concerns, seabed conditions, national and international legislation etc., are some issues that can cause difficulties.

Generally, the aforementioned challenges can be divided into four groups: management, engineering, installation and operation.

Figure 3. Major trade movements in 2016 (billion cubic meters) [1].

1.1. Iran-Oman Gas Pipeline Project

Based on an agreement signed in 2013, Iran will export 28 million cubic meters of gas to Oman per day via a subsea pipeline through Persian Gulf for 15 year period. The length of the subsea pipeline will be approximately 200 kilometers with 36 inches diameter. It can be expected that the aforementioned project will be commissioned by 2020 [5]. Some of the Iranian gas volumes will be exported as LNG from Oman and the remaining gas will be used for Oman's domestically demand [4, 5].

The pipeline route will establish a direct link between Iran and Oman besides it will not pass through another country's waters. By achieving this purpose, the length will be shorter, although the pipeline should be laid in deeper sector [5].

Type of contracts, financing and procurement, owner and operator of the pipeline system, maintenance and repair systems are main challenges of this project. For instance, is it possible for the other gas purchasers to rent this pipeline for transporting gas?

1.2. Iran-India Gas Pipeline Project

The aim of the project is to supply gas from Iran to India. Indirect route from Iran to Oman then from Oman to India instead of the direct route from Iran to India is an alternative option [Figure 4]. The estimated length of pipeline is around 1300-1400 km. Maximum water depth will be 3600 meters [6].

Figure 4. Potential pipeline routes to supply gas to India [6].

2. Conclusions

Megaprojects face immense technical, legal, political, economic, and social challenges and sometimes require innovative solutions to mitigate environmental risk. For instance, lack of technology, expertise or facilities is one of the limitation. To overcome this challenge, indigenous infrastructure must be upgraded or borrowed. The shareholders, foreign investors or bank loans can finance project cost. An extensive and transparent communication strategy must be adopted to achieve agreement on political and legal issues. Finally, the potential impact of the project must be evaluated to avoid possible negative effects on the environment.

3. References

1. Introduction

Offshore pipelines are typically designed to operate in high pressure and high temperature loading conditions. Operational loads potentially lead to pipe expansion, whereas seabed resistance force act to resist pipeline movement. Lateral deflection of exposed pipelines subject to high axial loads is known as “lateral buckling” phenomena. Well-ordered deformation can be a challenge during the design phase and uncontrolled movements may result in structural integrity problems and catastrophic failure [1]. Thermo-mechanical behavior strongly depends on pipe-soil interaction [2].

Influence of various assumptions for pipe-soil interaction models on lateral buckling performance is investigated in this paper.

2. FEM

A series of FE analyses in ABAQUS software is utilized for investigating pipeline response in different conditions. Deformable pipe element (PIPE1H) is used for modeling the pipe over an analytical rigid seabed. Gravity load and internal pressure loads are applied to the pipeline. Consequently, cyclic thermal loads are added for seven shut-down and start-up conditions. The project characteristics is presented in Table 1.

<table>
<thead>
<tr>
<th>Modeling Parameters</th>
<th>Unit</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pipeline Outer Diameter (OD)</td>
<td>in</td>
<td>14</td>
</tr>
<tr>
<td>Pipeline Wall Thickness (WT)</td>
<td>mm</td>
<td>19.8</td>
</tr>
<tr>
<td>Pipeline Total Length</td>
<td>km</td>
<td>4.881</td>
</tr>
<tr>
<td>Maximum Operating Temperature</td>
<td>oC</td>
<td>120</td>
</tr>
<tr>
<td>Seabed Ambient Temperature</td>
<td>oC</td>
<td>12</td>
</tr>
<tr>
<td>Maximum Operating Pressure</td>
<td>bar</td>
<td>200</td>
</tr>
<tr>
<td>Pipeline Material Modulus</td>
<td>MPa</td>
<td>205,000</td>
</tr>
<tr>
<td>Coefficient of Thermal Expansion</td>
<td>1/°C</td>
<td>1.3 ×105</td>
</tr>
<tr>
<td>Pipe-Soil Axial Friction</td>
<td></td>
<td>0.5</td>
</tr>
<tr>
<td>Pipe-Soil Lateral Friction</td>
<td></td>
<td>0.5</td>
</tr>
</tbody>
</table>

A practical initial curvature patented by Statoil [3] is utilized in this paper for creating three initial imperfections with equal intervals along the pipe. Initial strain equal to 0.2% is considered at each imperfection location as result of residual curvature from reel-laying installation operation [4].

3. Pipe-soil Interaction Models

Uncoupled axial and lateral friction model besides coulombic friction are used in this paper for pipe-soil interaction response. Considered PSI models represent a lightweight pipeline response. Failure surface illustrated in Figure 1 based on lateral and axial friction and normal force on the pipe-soil interface. “Friction model” shows a rectangular face which means distinct values of friction in axial and lateral directions are specified during the simulation. This “Friction model” model is defined via a friction subroutine in ABAQUS software [5,6]. On the other hand, “Coulomb model” shows the situation that resultant vector of friction is calculated based on two lateral and axial forces and friction coefficient is considered the resultant slip of the pipe on seabed [5]. Coulomb friction assumption has been used widely for lateral buckling analyses [6]. Failure surface for “coulomb model” is presented in an oval shape that is the default response in ABAQUS software [7].

Figure 2 shows four typical models for seabed resistance force. The elasto-plastic interaction model is
considered for axial friction (Friction model 2). Nonetheless, lateral restriction force can be different. Soil berms during the lateral deformation may generate different brittle breakout. So, all four possible behaviors are assumed for lateral response. Besides, one model with coulombic friction (Figure 1) is provided. Lateral buckling response is compared for these five situations in following parts of this paper.

Lateral displacement increases during each heat-up and decrease in each cool-down. Maximum displacement in each cycle occurred at full heat-up condition (operation phase). The growth of maximum lateral deflection is observed in subsequent cycles because of changes in initial curvature after each cycle. Almost equivalent response is detected for four friction models. However, coulomb friction model represent lower range of lateral displacements.

4. Results

Effect of different models of lateral buckling response was investigated in diverse analyses. Lateral deflection of the pipeline at the middle point is shown in Figure 3 as an example for comparing friction models. These lateral deflections demonstrate the situation that the buckle occurred at the middle section of the pipeline. Buckled pipe in “Friction model” represents sharper configuration which is a more realistic model [8]. The maximum stress value occurred at the crown of the buckled zone. Lateral displacement value governs final curvature and induced stress on the pipeline section.

5. Conclusion

The extreme lateral deflection of “Friction model” option is about 1.16 times larger than “Coulomb model” for the case study presented in this paper.

The buckle shape for “Friction model” option is happened in mode 3; however, it is occurred in mode 5 for “Coulomb model” case.

Nearly 49% buckle dimension development is observed after seven cycles of start-up and shut-down cycles.

Based on presented results for “Friction model” 1 to 4, it can be grasped that brittle breakout and elastic slip values have a minor influence on the final size and configuration of the buckled pipe (about 5.3%).

6. References

1. Introduction

Desalination plants create drinkable freshwater from ocean water in areas where there are not sufficient supplies of freshwater.

Subsea pipelines are considered to be the most efficient and economical way to transport liquids from offshore to land or other areas. Loads on a marine pipelines can be divided into the categories: functional, environmental, accidental and installation loads [1]. A marine pipeline is exposed to different loads during installation such as tension, bending, and high external hydrostatic pressures which are becoming greater problems in increasing water depths. Pipeline installation process of SAKO desalination plant is numerically analyzed in this paper using OrcaFlex software.

2. SAKO Project Overview

SAKO plant is a power and desalination complex located in the Northern coast of Persian Gulf, Bandar Abbas, Iran. The project consists of desalination units with total capacity of 1,000,000 m3/day freshwater and a power plant rated at 1000MW. Approximate location of the project is shown in Figure 1.

![Figure 1. Approximate location of SAKO project.](image)

This project is consists of 6 intake pipelines and 3 outfall pipelines. The internal diameter of pipeline is 2.5 meter. Other characteristics of the pipeline are introduced in Table 1.

<table>
<thead>
<tr>
<th>Material type</th>
<th>High Density Polyethylene</th>
</tr>
</thead>
<tbody>
<tr>
<td>Grade</td>
<td>PE100</td>
</tr>
<tr>
<td>Profile No.</td>
<td>SM530, SM550, SM515</td>
</tr>
<tr>
<td>Wall Thickness</td>
<td>78, 87, 126 mm</td>
</tr>
</tbody>
</table>

3. Pipeline Installation Method

Pipeline installation is one of the important stages of offshore field development. The choice of installation method is influenced by the water depth, pipeline type and material, time and cost among other things. There are four major pipeline installation methods: S-Lay, J-Lay, Reel Lay, Towing [2].

Here the surface towing method is used (Figure 2). In towing lay, the pipe is made up at some remote location onshore, transported to the offshore installation site by towing, and layed down. The buoyancy of the pipe is selected and designed to verify that a controlled-depth towing can be performed [3].

![Figure 2. General View of Pipe String Towing Route.](image)

In the present project, guide piles are employed to anchor pipe strings against environmental loads during positioning and installation procedure as well as providing a directional route; then by gradual penetration of seawater inside the pipes, pipes become immersed and finally lay on the sea bed.
4. Numerical Modelling

Numerical modeling was performed using OrcaFlex software. OrcaFlex is a fully 3D non-linear time domain finite element. A lumped mass element is used which greatly simplifies the mathematical formulation and allows quick and efficient development of the program to include additional force terms and constraints on the system in response to new engineering requirements [4].

According to the met-ocean study of the area, significant wave height of 0.5 m, wave period of 4 sec and current speed of 1.2 m/s were considered for pipeline’s installation period. The maximum modelling depth was assumed to be 19.14 m relative to MSL.

Model includes a string with length of 300 m which is linked to a tugboat. Several airbags are installed on the pipe to control the stresses occurring during installation process and Guide piles are used with spacing of 45m (Figure 3).

This procedure is modeled for all three types of cross-section and results are mentioned in Table 2.

Table 2. Results of three types of cross-section.

<table>
<thead>
<tr>
<th>Profile type</th>
<th>Length (m)</th>
<th>Max depth (m)</th>
<th>Max von mises stress (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SM515</td>
<td>130</td>
<td>9.05</td>
<td>7.4</td>
</tr>
<tr>
<td>SM550</td>
<td>300</td>
<td>17.23</td>
<td>8.38</td>
</tr>
<tr>
<td>SM530</td>
<td>300</td>
<td>19.14</td>
<td>9.4</td>
</tr>
</tbody>
</table>

The installation process is presented in the Figure 4 in several time steps during installation.

5. Conclusions

1- OrcaFlex software is capable of modeling of marine environmental conditions such as wave and current loads.
2- In this study, sea water gradually flows inside the pipelines causing them to move from sea surface to sea bed which is similar to what happens in reality during installation process.
3- Pipeline’s structural specifications are also reasonably modeled.
4- The bed slope is also modeled.
5- Modeling results showed that there will be maximum stress of 9.4 MPa in the pipelines.
6- Airbags and guide piles were modeled as well to control the installation process.

6. References

REDDUCING HEAVE RESPONSE AMPLITUDE OPERATOR OF A SEMI-SUBMERSIBLE PLATFORM USING PORO-ELASTIC PLATES

Arefe Emami¹ and Ahmad Reza Mostafa Gharabaghi²

¹) Faculty of Civil Engineering, Sahand University of Technology, Tabriz, Iran, a_emami@sut.ac.ir
²) Faculty of Civil Engineering, Sahand University of Technology, Tabriz, Iran, mgharabaghi@sut.ac.ir

1. Introduction

In this paper, the attempt has been made to analytically solve the coupling problem of a monochromatic linear wave with a semi-submersible platform which is modified by attaching Poro-Elastic Plates (PEPs) at the bottom of its pontoons. The PEP is considered to be homogenous, isotropic and saturated. The region around the semi-sub is divided into different parts and their related governing equations with appropriate boundary conditions (BCs) are developed. By applying Eigenfunction Expansion Method (EEM), these equations with their related BCs for each region are solved numerically. In order to verify the developed model, it is applied to a typical GVA4000 semi-submersible drilling rig and its heave Response Amplitude Operator (RAO) is extracted and compared with available experimental data. Furthermore, the PEP is attached to the rig and its heave RAO is estimated and compared with the original case. It is concluded that using PEP reduces the peak value of the heave motion response as well as the resonance frequency. Therefore, it can be a suitable method in order to improve the performance of a semi-submersible platform particularly its heave motion response.

2. Semi-Submersible Platform with Poro-Elastic Plate

A Semi-submersible drilling platform is a floating mobile offshore drilling unit (MODU) which is designed for drilling in water depths beyond the capacity of jack up drilling rigs. They are typically made from two submerged pontoons with four or more columns connecting the pontoons to the hull. One of the main problems of semi-submersible drilling rigs is their heave motion response which can often restrict their operability range or even lead to damage to their risers and mooring systems. There are several attempts in order to reduce the heave motion response of these platforms which can be categorized to the modification of their geometry, increasing their draft, installing heave plates or using truss type columns. As a new concept, in this study, the PEPs are attached to the lower part of rig's pontoons in order to reduce its heave motion response. The PEP is a type of flexible material which can deform due to the fluid flow through its porous structure. The interaction between fluid flow and linear porous material deformation was originally proposed by Biot (1941) [1]. In literature, such material was used for some type of submerged structures. Lan and Lee (2010), Lan et al. (2011, 2012, 2014), and Lan and Hsu (2014) analytically studied the interaction of waves with submerged breakwaters made from poro-elastic material [2-6].

3. Mathematical Equations

In this study, a typical GVA4000 drilling semi-submersible rig was modified by a layer of PEP at the bottom of its pontoons. The rig is assumed to be located at water depth of \( h_1 \) under a monochromatic small amplitude wave train propagating in the +x direction (Figure 1). It is assumed that the fluid is incompressible, homogeneous, inviscid and poro-elastic material is homogeneous, isotropic and saturated. It is necessary to calculate the exciting forces, added mass, damping coefficient and stiffness in order to determine the modified rig's heave motion RAO.

The exciting force can be determined by using incident and radiated wave potentials. The added mass and damping coefficient are calculated by radiated wave potential. The incident wave potential is obtained with assuming the harmonic wave in the absence of platform and with the aid of the Laplace equation (Eq. 1) [7]:

\[
\Phi_i = -\frac{A_g}{\omega} \frac{\cosh(\kappa z + h_1)}{\cosh(\kappa h)} \exp(i k x) \tag{1}
\]

The radiation potential is obtained by assuming the oscillation of the structure in the absence of waves [7]. As shown in Figure 1, the domain around the rig is divided into five separate regions. Each region has its own governing equations with appropriate BCs. The governing equation for Regions I, II-1, III and IV is Laplace equation which can be solved by satisfying the relative dynamic and kinematic boundary conditions which are consisted of the free surface, seabed and the normal velocity over the structure BC's. The governing equations for Region II-2 are the continuity and momentum equations for the PEP. The continuity equation can be written based on Verruijt’s equation [8] and the momentum equations for PEP can be obtained by Biot’s theory [9]. These equations should be solved simultaneously in order to satisfy the interface boundary conditions between two adjacent regions. The unknown coefficients are derived by using EEM and with the aid of the continuity conditions for pressure, normal velocity and other boundary conditions that are illustrated in Figure 1. Finally, they are solved by the LU decomposition method.
4. Verification of the Developed Model
In order to verify the developed analytical model, the results obtained for the RAO of heave motion of a GVA4000 drilling semi-submersible platform without PEP is compared with the experimental data [10]. The input parameters are \( a=40.28 \text{m}, s=20.5 \text{m}, s_1=13 \text{m}, l=20.91 \text{m} \) and \( h_1=1000 \text{m} \). As shown in Figure 2, the results of experimental data and analytical method are close enough to be considered acceptable.

5. Results
In the next step, the PEPs are attached at the bottom of GVA4000’s pontoons in order to study its effect on the heave motion response. The input parameters of PEPs are the PEP’s porosity=0.4, PEP’s Poisson ratio=0.333, fluid kinematic viscosity=1.12x10^{-9} \text{m}^2/\text{s}, fluid pore compression=4.35x10^{-14}, turbulent drag coefficient=0.2, added mass coefficient=0.015, elastic solid density=2650 \text{kg/m}^3, fluid density=1000 \text{kg/m}^3, PEP’s thickness=0.5 \text{m}, shear module=5 \times 10^8 \text{N/m}^2, and intrinsic permeability= 2.28 \times 10^{-6} \text{m}^2.

The convergence is satisfied by applying 20 modes in EEM. Then, the heave RAO is calculated and compared with the original case in Figure 3. It is noticed that the heave motion response of the modified semi-submersible platform reduced significantly after installing the PEPs.

6. Conclusion
The main objective of the present study was to investigate an approach for reducing the heave motion response of the semi-submersible platform. Therefore, the PEPs are installed at the bottom of its pontoons. The EEM was used to solve the governing equations with related BC’s. Results showed that the heave motion response of the modified semi-submersible decreased significantly compared to its original case. Indeed, the PEPs not only reduced the resonance amplitude but also the resonant frequency displaced to the lower frequency. This research offers an applicable way for reducing heave motion response of a semi-submersible platform with easy installation on the all semi-submersible platforms even in operation.

7. References
INTERACTION JACKED AND RISER IN THE FATIGUE DAMAGE OF THE RISER USING TIME HISTORY ANALYSIS METHOD

Hamid Anbarestani1 and Naser Shabakhty2

1) MSc, Department of Marine industry, Science and research branch, Islamic Azad University; Tehran, Iran, hamid.anbarestani70@gmail.com
2) Assistant professor, School of Civil Engineering, Iran University of Science and Technology; shabakhty@iust.ac.ir

1. Introduction
The life of the offshore platform under fatigue damage is estimated using the time history of the structural stress [1]. This research was conducted on the basis of the API code for one of the platforms installed in the South Pars region. The result shows, fatigue damage is an important failure mode for riser and it may effect on the lifetime of riser.

2. Sea State Scatter Diagram
Fatigue damages in riser are combination of damage arise from several sea states and incorporating all these sea states are time consuming task in the time history analysis. Therefore, sea state scatter diagram should be divided into several blocks which each one is concentrated in the center of block to reduce the fatigue analysis time... To calculate the wave height and period for center of this block, the weighted average method is used. Therefore, for wave height the following equation,

\[ H_s = \frac{\sum H_s \times N_i}{\sum N_i} \]

and the following expression can be used for wave period.

\[ T_p = \frac{\sum T_p \times N_i}{\sum N_i} \]

Where in these equations \( N_i \) is the number of occurrence of sea state considered in each block. The scatter diagram is divided to four block and wave heights and periods are estimated and shown in Table 1.

Table 1. Sea state in fatigue analysis

<table>
<thead>
<tr>
<th>Sea state</th>
<th>( H_s )</th>
<th>( T_p )</th>
<th>( P_l )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Block 1</td>
<td>0.67</td>
<td>2.7</td>
<td>0.315</td>
</tr>
<tr>
<td>Block 2</td>
<td>1.1</td>
<td>4.56</td>
<td>0.49</td>
</tr>
<tr>
<td>Block 3</td>
<td>1.25</td>
<td>6.25</td>
<td>0.114</td>
</tr>
<tr>
<td>Block 4</td>
<td>2.7</td>
<td>8.6</td>
<td>0.079</td>
</tr>
</tbody>
</table>

According to the centreal sea states, we obtained the hydrodynamic forces on the riser for 20 minutes and axial and flexural stresses for in-plane and out-off plane (\( \sigma_{ipb} \) and \( \sigma_{opb} \)) are estimate for eight points around intersection of the riser. Hot Spot stress is estimated by incorporating the stress concentration factor (SCF) for each component by the following expression.

\[ \sigma_{HS} = \sigma_{a} \times SCF_A + \sigma_{ipb} \times SCF_{IP} + \sigma_{opb} \times SCF_{OP} \]

This hot spot stress should be considered for at least 8 points around intersection of riser, to determine the highest point that fatigue damage may occurs.

1) \( \sigma_1 = SCF_A \sigma_X \times SCF_{IP} \sigma_M \)
2) \( \sigma_2 = 0.5 SCF_A \sigma_X + 0.5 \sqrt{2} SCF_{IP} \sigma_M - 0.5 \sqrt{2} SCF_{OP} \sigma_M \)
3) \( \sigma_3 = SCF_A \sigma_X - SCF_{OP} \sigma_M \)
4) \( \sigma_4 = 0.5 SCF_A \sigma_X - 0.5 \sqrt{2} SCF_{IP} \sigma_M + 0.5 \sqrt{2} SCF_{OP} \sigma_M \)
5) \( \sigma_5 = SCF_A \sigma_X - SCF_{OP} \sigma_M \)
6) \( \sigma_6 = 0.5 SCF_A \sigma_X + 0.5 \sqrt{2} SCF_{IP} \sigma_M + 0.5 \sqrt{2} SCF_{OP} \sigma_M \)
7) \( \sigma_7 = SCF_A \sigma_X + SCF_{OP} \sigma_M \)
8) \( \sigma_8 = 0.5 SCF_A \sigma_X + 0.5 \sqrt{2} SCF_{IP} \sigma_M + 0.5 \sqrt{2} SCF_{OP} \sigma_M \)

3. Riser Fatigue Estimation
To estimate the fatigue damage, we should determine the number of stress cycle. We used rain-flow counting method and the final fatigue damage is calculated by the Palmgren-Miner method by the following equation,

\[ DFAT = \sum \frac{n_i}{N_i} \leq 1 \]

Where, \( n_i \) is the number of stress cycle in riser, \( N_i \) is the number of stress cycles to failure which is derived from API Regulations. The total damage should be less than one to prevent fatigue failure. However, this damage should be divided to the safety factor recommended in this regulation. Figure 1 shows the recommended stress number to failure \( N_i \) in API [2].

This stress number to failure can be shown by the following equation:

\[ N(s) = 2 \times 10^6 \times \left( \frac{\Delta \sigma}{\Delta \sigma_{ref}} \right)^{-m} \]

\( \Delta \sigma \) is existing stress range in riser and \( \Delta \sigma_{ref} \) shows the reference stress range given by codes and, \( m \) is the slope of S-N curve given in API code.
4. Conclusion Summary
Comparing and examining of two riser and platform interaction models, it shows when the stiffness of platform increase to ignore the riser-platform interaction, the period of the structure decreases. By reducing the period of the structure, the vibration impacts on riser reduced, so the fatigue life of the riser increases. Therefore, for fatigue riser evaluation, interaction of platform and riser is an important issue and should be taken into account.

5. References

COMPARISON BETWEEN CONDITIONAL AND UN-CONDITIONAL FAILURE PROBABILITY OF CORRODED GAS TRANSMISSION PIPELINES CONSIDERING STOCHASTIC PROCESS FOR INTERNAL PROSSURE AND CRACK GROWTH RATE

Mohammad Mahdi Shabani¹, Mohammad Daghigh² and Reza Taravati³

1) Offshore Structural Engineering, Petroleum University of Technology, Isfahan, Iran, m.shabani@mnc.put.ac.ir
2) Assistant Professor and Ph.D. in Offshore Engineering, Pars Oil and Gas Company (POGC), mdaghigh@gmail.com & daghigh@pogc.ir
3) Technical Inspector, Iran, Iran, reza_taravati@yahoo.com

1. Introduction

Pipelines are the safest and most economical form of natural gas transmission which are remarkable for their efficiency and low cost [1]. Generally, subsea pipelines carry oil and gas products from wellhead to the riser base [2].

According to study of Shabani [3], corrosion is the most deteriorative factor on pipeline health and safety. So identifying and recognizing corrosion and its resultant are essential and necessary for pipeline integrity management [4]. Shabani et. al present a Stochastic-Based Approach for determining risk of corroded pipelines considering unconditional form of corrosion attacks.

This paper deals with RA of corroded gas transmission pipelines considering all probable failure modes of corrosion attacks. Reliability level of pipeline is evaluated in three different modes. All pipeline geometrical parameters are modeled as inherent uncertain parameters. Relationship between crack dimension is modeled using correlations. Crack growth rate and internal pressure are modeled using Poisson Square Wave Process and Ferry Borges-Castanheta, respectively (see Figure 1).

2. Reliability Assessment

Non-deterministic approaches are used to evaluate uncertainties in both load and resistance parameters. Reliability of a component can be defined as probability that component meets some specified demands under specific environmental conditions[6]. There are too many methods (techniques) for determining POF. Zhou et. al [5] recommends to use IS technique for reliability assessment of corroded pipelines. IS technique is a modified form of Monte-Carlo sampling method by FORM.

Based on study of Zhou et. al[5], three different failure modes are taken into account in this study which are:

- Small leak
- Large Leak
- Rupture

According to study of Shabani et. al, ASME B31.8 presents the best model for estimating burst capacity of corroded pipelines that is:

\[ P_{\text{burst}} = 1.1 \cdot \frac{2 \cdot \sqrt{\text{SMYS}}}{D} \left( 1 - \left( \frac{Q}{2} \right) \right) \]  \hspace{1cm} (1)

\[ P_{\text{burst}} = 1.1 \cdot \frac{2 \cdot \sqrt{\text{SMYS}}}{D} \left( 1 - \frac{1}{2} \left( \frac{Q}{2} \right) \right) \]  \hspace{1cm} (2)

Where \( d \) is depth of corroded region and \( Q \) is Fulia factor.

3. Methodology

All pipeline parameters are modeled using recommended distribution by Shabani et. al [7]. Based on the ILI data, correlation coefficients and coefficient of variation is determined. As there is no clear pattern for varying pressure and crack growth rate using stochastic processes, and also internal pressure is the most effective factor on reliability level of corroded pipelines. Therefore, internal pressure and crack growth rate are modeled by Ferry Borges and Poisson Square Wave Process, respectively (see Figure 1).

Figure 1. Poisson Square Wave Process model of internal pressure
For figuring out that results are trustable, a criterion should be applied to control failure probability which is converging of results. This paper uses coefficient of variation of calculated failure probabilities. Normally, it set up five percent and for harsh condition it set up two percent; this paper uses harsh condition.

Based on studies of Zhou et. al [5] and Shabani et. al [7] definition of conditional and un-conditional form of corrosion attack is such below:

- **Conditional:**
  - Small leak: when small leak occurs pipeline doesn’t fall in large leak and rupture:
    \[ g_{SL} \leq 0, g_{LL} > 0, g_{rupture} > 0 \]  
  \[ (9) \]
  - Large leak: pipeline falls in large leak, however pipeline doesn’t suffer from rupture and small leak:
    \[ (g_{SL} > 0, g_{LL} \leq 0, g_{rupture} > 0) \]  
  \[ (10) \]
  - Rupture: pipeline bursting happens while large leak occurred:
    \[ (g_{SL} > 0, g_{LL} \leq 0, g_{rupture} \leq 0) \]  
  \[ (11) \]

- **Un-Conditional:**
  \[ P_{rupture\_total} = P_{rupture} + P_{rupture\_SL} + P_{rupture\_LL} \]  
  \[ (12) \]

Let assume priority of each failure mode such below:

![Figure 2. Procedure of corrosion defect growth](image)

If we neglect interaction between each failure mode (i.e. consider each failure mode independently), then un-conditional form of corrosion attack formula changes to following form:

\[ P_{rupture\_total} = P_{rupture} \left( e^\tau \right) + P_{rupture\_LL} \left( e^\tau \right) + P_{rupture\_SL} \left( e^\tau \right) \]  
\[ P_{rupture} \left( e^\tau \right) = P_{rupture} \left( e^\tau \right) \cdot P_{SL} \left( e^\tau \right) \]  
\[ (13) \]

4. Results

Using described method, un-conditional POF of the pipeline for different service times is determined and discussed as follows:

Figure 3.a indicates that small leak has no significant impact on pipeline integrity in first ten years, but due to larger POF than target POF its affect should be considered in RBI plan.

Also, Figure 3.b indicates that ASME considers larger in service time in comparison to initial reliability index for the pipeline.

5. Conclusion

An attempt was made to determine better form of corrosion attacks failure modes for reliability based design (inspection). According to numerical results, SL has no significant impact on pipeline safety, but POF of SL reaches to three percent in 15 years. Furthermore, results indicated that ASME considers larger POF for servicing time in comparison to initial POF at design step. Also, using un-conditional form for pipeline rupture could cause larger POF. In other words, higher degree of safety could be planned in detailed design stage and in RBI plan.

6. References


Seismic Behavior of Hunchbacked Block-Type Gravity Quay Walls

Babak Ebrahimian, Amir R. Zarnousheh Farahani and Ali Noorzad

1) Faculty of Civil, Water and Environmental Engineering, Shahid Beheshti University, Tehran, Iran, b.ebrahimian@sbu.ac.ir, ebrahimian.babak@gmail.com
2) Faculty of Civil, Water and Environmental Engineering, Shahid Beheshti University, Tehran, Iran, a.zarnoosheh@sbu.ac.ir
3) Faculty of Civil, Water and Environmental Engineering, Shahid Beheshti University, Tehran, Iran, a_noorzad@sbu.ac.ir

1. Introduction

Herein, the seismic behavior of hunchbacked block-type gravity quay walls is investigated using 2D fully nonlinear dynamic time-history analyses. They take into account material and geometric non-linearities. Both static and seismic conditions are considered. The obtained numerical results are compared with those of 1g shaking table tests [1]. Comparisons are proposed in terms of acceleration, displacement, and total lateral earth pressure time histories across the height of two types of quay walls with different configurations, as shown in Figure 1. Quay wall Type I has a larger hunch than that of Type II.

![Figure 1. Schematic cross section of quay wall: (a) Type I, (b) Type II in prototype scale and location of instrumentations (TEP: Total Earth Pressure; ACC: Acceleration; LVDT: Displacement).](image)

2. Numerical Modeling Procedure

Numerical simulations are conducted by an explicit finite difference code incorporating hysteretic Mohr-Coulomb constitutive model to describe the stress-strain response of soil and the Rayleigh damping to increase the level of hysteretic damping in the model [2]. The Seed modulus reduction curve is employed to consider the soil non-linear behavior before yielding [2]. Contact conditions between wall and adjacent soil are modeled via special interface elements allowing for slipping and gapping through the Coulomb frictional law. Element size is selected small enough to allow the seismic wave propagation throughout the numerical model. The sea water is simulated through the hydrostatic pressures applied to the front side of the wall. Correspondingly, the hydrodynamic effects are exerted by the Westergaard’s added masses on the seaward face of the wall. In dynamic analyses, the free-field condition is applied to the lateral boundaries eliminating the wave reflection into the model [2]. To avoid spurious oscillations at very small deformations and high frequency components of motions, 5% of Rayleigh damping, centered at a frequency of around 2.5 Hz (close to the fundamental frequency of the system), is considered in the dynamic analyses. It is noted that liquefaction cannot be triggered during shaking and the developed excess pore water pressures are negligible due to the presence of coarse grained backfill soil and very dense seabed sand foundation.

3. Results and Discussion

For the applied input motions demonstrated in Figure 2, the predicted and measured responses are illustrated in Figures 3-7. According to Figure 3, overall agreement is achieved between the predicted and measured horizontal accelerations, with the predicted values giving somewhat higher amplitudes. Figure 4 shows that the displacement of quay wall head increases incrementally during seismic loading. Quay wall Type I moves seaward about 30 cm and has very small amount of vertical settlement, showing wall translational sliding on its base, whereas the backfill settle up to about 55 cm behind the wall. Quay wall Type II moves extensively toward the sea by amount of about 100 cm, accompanied by a maximum settlement of about 4.5 cm. For the latter wall, the backfill has maximum settlement of 100 cm. The very small differences between the predicted and measured vertical displacement of quay wall head are likely attributed to the minor rocking motion of the walls, Figure 4. Figure 5 shows the wall movement and backfill deformation patterns in wall Type II. Particularly, the wall on the very dense sandy soil foundation significantly slides and slightly rotates seaward. Apron settlement immediately behind the broken-back
quay wall is large. The deformation patterns obtained from numerical simulations are in proper agreements with those of model experiments [1]. Lateral earth pressures on walls are calculated in Figure 6 and compared with the measured ones. It is seen that, in general, both the magnitude and the trend of all time histories are in reasonable agreement. It is confirmed that quay walls with steeper broken-back angle have better seismic performance and are preferred to the vertical-back block quay walls in high seismicity regions.

Figure 2. Horizontal acceleration time histories applied at the base of numerical model for quay wall: (a) Type I, (b) Type II.

Figure 3. Predicted versus measured horizontal acceleration time histories at head of quay wall: (a) Type I, (b) Type II.

Figure 4. Predicted versus measured values of: (a) horizontal, and (b) vertical displacements time histories at head of quay walls Types I and II.

Figure 5. Computed deformed configuration of quay wall type II at the end shaking

4. References
EFFECT OF THE WATER SALINITY ON THE CONSOLIDATION AND MECHANICAL BEHAVIOR OF THE PERSIAN GULF MARINE CLAYS: A CASE STUDY

Ali Bayat¹ and Hamed Bayesteh²

¹) M.sc, Department of Civil Engineering, University of Qom, Qom, Iran, a.bayat@stu.qom.ac.ir
²) PhD, Department of Civil Engineering, University of Qom, Qom, Iran, h.bayesteh@qom.ac.ir

1. Introduction

One of the most important indicators of the development of countries is the development of the marine infrastructure and the expansion of development activities in coastal strips, which has become popular in Iran in recent years. On the other hand, the soils of coastal areas have unique characteristics due to their formation and their structural nature. High water content, high porosity, low resistance and high corrosive salt content are the most important engineering properties in marine soft soils. One of the soils that is always considered is clay that is scattered in many coastal regions of the country, such as Khuzestan and Bushehr. The behavior of clay is the function of minerals and pore water. The chemical status of pore water may be significantly altered by the exchange of exchangeable cations in clays that in most cases affect its engineering characteristics. Most engineering problems in clay take place due to the physical-chemical changes of the pore water. On the other hand, the environmental conditions and climate change are such that changes in the pore water salinity in the adjacent clay, therefore it is necessary to recognize this interaction in the soil of each region. The effect of change in the water salinity on the marine clay behavior in the Persian Gulf area has not been well investigated yet. Therefore, it is necessary to study the effect of water salinity on the above-sea and the compressibility and the parameters of the consolidation of marine clay as well as parameters such as horizontal and vertical permeability coefficients in geotechnical terms. This issue should be investigated in southern soils of the country that are in opposition to salt changes to make. In this research, the clay-water approach in natural conditions has been selected from the southern coast of the country and the effects of salinity changes on different levels of stress have been investigated on the consolidation behavior marine clay, by considering the soft clay located in the Bushehr city.

2. Materials and Methods

The clays were collected from the coasts of Bushehr province, as indicated in Figure 1.

Figure 1. Sampling site in Bushehr province

In order to know the mechanical and microstructural properties of this soil, XRD analysis and Field Emission scanning electron microscopy (FESEM) were prepared from this clay and related experiments were performed. In Figure 2, the results of the XRD test as FESEM images are shown, which states that the nature of the sample is carbonate and given that 50% of the soil is formed by calcium carbonate. According to previous studies this soil is in the category of calcium Silicate [1]. The FESEM image states that carbonate soil has a needle-like structure [2].

Figure 2. XRD analysis and FESEM photo on clay samples

A test program was developed with two methods of water leakage with different salinity in soil, as well as intrinsic salinity changes in soil. In Table 1, the types of soil and water are summarized. Soil electrical conductivity has been used to express the amount of water salinity. To prepare the sweet and semi-sweet soil according to the number of times listed in Table 1, add distilled water to the
soil site with a ratio of 1 to 10 and mix it for 2 hours, each time for a period of 10 minutes at a rotational speed of 2000 rpm centrifuges. Seawater was used to create salt water and in order to carry out the test in real conditions, it was used as seawater. The salt water used is the same as seawater. To investigate changes in the characteristic of the clay due to the water change as well as the nature of the soil (sweetening), the Atterberg limits have been tested according to the standard ASTM D4318[3] and the results of variations with regard to the different conditions of water and soil are reported in Figures 3.

The results show that water changes do not have a significant effect on the nature of plastic behavior of soil in the short time. In all different states of soil with distilled water, there is an increase in the liquid limit, which in fact indicates a tendency to absorb more water in this state. [4]

To investigate the changes in soil consolidation behavior, the specimens were prepared in a very loose wet weight at the place of 1.8 gr/cm³ in an extension ring with a diameter of 51 mm and a height of 20 mm. For this purpose, firstly mix the tested soil with suitable water (for example, the soil with sea water) with excessive water of liquid limit and in the pipe and the ring we are looking for and allow to be grainy in the ring. Then the consolidation test has done. Some Results changes in soil consolidation behavior in different conditions of water and soil, which are normalized with the normalized $e/e_l$ are plotted in Figure 4.

The soil and water profiles of each test are: 1) in situ soil with intermediate water salinity, 2) in situ soil with sea water salinity, 3) washed soil to E 1.53 mmohs/cm with distilled water and 4) washed soil to E 0.3 mmohs/cm with sea water. The results show that according to the osmotic theory, the water moves from a lower concentration medium to a higher concentration environment, so that when seawater is added to the soil, since all 2 concentrations are the same, they do not significantly change the particle arrangement. The strong bonds of the present are not broken down and cause a lower settlement than when the semisweet and distilled water enters the environment, because the semisweet water changes the structure of the soil from the complex to the dispersed state, weakening the vanadium bonds between the particles he does. The results of unconfined compressive strength (UCS) test on samples with the change of water conditions of the project are shown in Figure 6. The results show that formation of a stronger bonds between clay particles lead to increase strength.

3. Conclusions

Change in water salinity has major effect in volume change behavior of marine clays. The amount of water salinity, in the short time, has little effect on the Atterberg limits except in distilled water. The presence of distilled water in the environment increases the Atterberg limits of soil types because the tendency to absorb water due to unbalanced negative loads in the soil environment increases. Changes in the amount of water salinity cause tangible changes in the soil liquid limit. At a constant stress level about 16 kg /cm², the lowest level of seagrass is due to the presence of strong seawater seams due to strong bonds and the highest sewage in soils with sea water because during the sweetening process the breaks have broken down and the soil structure has changed from the complex to the dispersed.

4. Reference


DEVELOPING A MULTI-OBJECTIVE OPTIMIZATION ALGORITHM FOR PREDICTING HULL DIMENSIONS OF SEMI-SUBMERSIBLE PLATFORM

Arefe Emami¹ and Ahmad Reza Mostafa Gharabaghi²

¹) Faculty of Civil Engineering, Sahand University of Technology, Tabriz, Iran, a_emami@sut.ac.ir
²) Faculty of Civil Engineering Faculty, Sahand University of Technology, Tabriz, Iran, mgharabaghi@sut.ac.ir

1. Introduction

One of the main approaches in order to improve the performance of semi-submersible drilling platforms is its hull geometry optimization. Developing a method for optimal design of this type of platform that achieves the best answer in a short time according to certain objectives, is incredibly significant. In this paper, by developing Grid Search (GS) algorithm as multi-objective optimization, the hull dimensions of a typical semi-submersible platform was estimated. The objectives were minimizing hull weight and its heave motion response. In this algorithm, design constraints of semi-submersible platform such as stability, air gap, draught, geometrical limitations, hull weight, and heave motion response were considered. Then, by implementing GS’ algorithm, the optimal hull dimensions of the studied platform were estimated. It achieved valid and reliable results with low computational time. Moreover, it is a more easy method to use for multi-objective functions in the floating or semi-floating structures.

2. Semi-Submersible Platform

Increasing need to the energy sources such as oil and gas, has encouraged its production from deep waters. Floating platforms are the most favorable units used for gas and oil production in deep waters. They are massive structures which usually are classified based on their objective such as drilling rigs, production platforms, storage platforms and drilling/storage/offloading platforms [1]. Semi-submersibles drilling platforms are a type of floating mobile offshore drilling units (MODUs) designed for drilling in deep waters. They are typically made from two submerged pontoons with four or more columns connecting the pontoons to the hull. One of the main problems of this type of platform is its heave motion response which is much larger than the other types of deep water platforms. One of the major methods in order to reduce these motions is the optimal design of its hull-dimensions. There are different methods in order to find the optimal dimensions of its hull such as applying the optimization algorithms. In literature, there are several investigations to find the optimal form or dimensions of its hull such as Akagi and Ito (1984), Birk and Claus (2001, 2008), Gallala (2013) [2-5]. In this paper a simple and logical multi objective optimization algorithm is applied for prediction of the hull dimensions of a typical semisubmersible platform. Indeed, the goal of this article is to provide a method that enables solving multi-objective problems using a simple way and achieving valid results with devoting less time and energy.

3. Grid Search Algorithm

The Grid Search algorithm (GS) is one of the multi-dimensional grid methods that has a centroid where the optimum point has been located on there. Indeed, GS’ algorithm searches a multi variable function on a computational grid and after finding the optimum point, it saves the information in its memory (Figure 1) [6]. This algorithm is a simple and reliable method which has been used extensively in Biomedical Engineering for single objective optimization. In this paper, GS’ algorithm is implemented for multi-objective functions for prediction of the hull dimensions of the semisubmersible platform for the first time.

4. Optimization of Semi-submersible

For prediction of the dimensions of the semi-submersible platforms’ hull, a certain shape of the main parts of the platform is required. Since the main goal of this paper is prediction of optimum hull dimensions of semisubmersible platform using GS’ algorithm, two geometrical panels are taken into account: the first geometrical shape is similar to the case related to the Gallala’s work (2013) and it is used to confirm the efficiency of the GS’ algorithm [5], another panel is related to the Iran Amirkabir semi-submersible drilling platform that the GS’ algorithm was applied to find its optimal geometry. Iran Amirkabir platform is a GVA4000 type platform which was designed and manufactured for drilling of oil and gas at water depth of 1000 m in Caspian Sea, north of Iran [7]. The GS’ algorithm is defined according to some constraints. The most important constraints for the
A semi-submersible platform includes: hull weight, stability constraints, air gap constraints that must be considered because of the risk of wave slamming, geometrical constraints which due to the logical output must be considered, and heave response constraints. It should be noted that in this algorithm, the main parameters are the hull weight and buoyancy of the semi-submersible platform’s hull form and its heave motion which is calculated by Morrison equation and Froude-krylove relation.

5. Validation of Grid Search Algorithm
To prove the accuracy and efficiency of the developed GS’ algorithm for optimal design of floating platform’s hull, a proposed semi-submersible platform by Gallala (2013) is considered. Then GS’ algorithm is implemented as a single objective algorithm with the goal of minimizing hull weight of semi-submersible platform. Finally, its results are compared by Gallala’s work (Table 1). The results show that both of optimization algorithms have a very close answers. So, GS’ algorithm is suitable and has enough efficiency for optimizing the geometry of semisubmersible platform’s hull.

Table 1. The comparison of optimization methods for semisubmersible platform

<table>
<thead>
<tr>
<th>Parameters (m)</th>
<th>Optimization with Grid Search algorithm</th>
<th>Optimization with GRG Method</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pontoon length</td>
<td>87</td>
<td>87</td>
</tr>
<tr>
<td>Pontoon height</td>
<td>8</td>
<td>0.04</td>
</tr>
<tr>
<td>Pontoon breadth</td>
<td>12.6</td>
<td>12.06</td>
</tr>
<tr>
<td>Column length</td>
<td>9</td>
<td>9.11</td>
</tr>
<tr>
<td>Column breadth</td>
<td>11</td>
<td>9.92</td>
</tr>
<tr>
<td>Column height</td>
<td>28.7</td>
<td>27.96</td>
</tr>
<tr>
<td>Distance between columns</td>
<td>74</td>
<td>74</td>
</tr>
</tbody>
</table>

6. Development of GS’ algorithm
In the next step, the GS’ algorithm was developed as multi-objective optimization method for minimizing the hull weight, and heave motion response (Figure 2). Initial parameters are based on the Iran Amirkabir drilling platform’s data. By applying design variables and criteria, optimization algorithm is implemented. It is noteworthy to mention that calculations have been done by a personal computer device SONY with Core i5.

7. Results
The results obtained from GS’ algorithm have been compared by hull dimensions of Iran-Amirkabir drilling platform (Table 2). It has given close answers to Iran-Amirkabir platform’s hull dimensions. Also, the short computational time to calculate optimal hull dimensions of semisubmersible platform is observed.

Table 2. Optimal dimensions of Iran-Amirkabir platform

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Optimization</th>
<th>Reference (Amirkabir drilling platform)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Length of pontoons (m)</td>
<td>80.392</td>
<td>80.56</td>
</tr>
<tr>
<td>Width of pontoons (m)</td>
<td>18.124</td>
<td>18.68</td>
</tr>
<tr>
<td>Height of pontoons (m)</td>
<td>2.75</td>
<td>2.75</td>
</tr>
<tr>
<td>Radius of columns (m)</td>
<td>7</td>
<td>6.45</td>
</tr>
<tr>
<td>Height of columns (m)</td>
<td>20.55</td>
<td>21</td>
</tr>
<tr>
<td>The longitudinal GM value (GML) (m)</td>
<td>2.4232</td>
<td>2.48</td>
</tr>
<tr>
<td>The transversal GM value (GMT) (m)</td>
<td>2.4232</td>
<td>2.48</td>
</tr>
<tr>
<td>Min. Weight (ton)</td>
<td>8846.7</td>
<td>8825</td>
</tr>
<tr>
<td>Min. Heave Motion Response (m)</td>
<td>2.0371</td>
<td>1.8</td>
</tr>
<tr>
<td>Computational Time (sec)</td>
<td>2.8111e+4</td>
<td>Unknown</td>
</tr>
</tbody>
</table>

8. Conclusion
One of the methods to increase the efficiency of semisubmersible platforms is to minimize their motions and achieve more stability. Therefore, the hull dimensions of these platforms are usually optimized. In the present paper, the Grid search algorithm has developed as two-objective optimization for the prediction of the hull dimensions of semi-submersible platform including: minimizing the hull weight, and its heave motion response. The results obtained for the most suitable optimal dimensions for its hull with low computational time. The most significant advantages of this algorithm are simple expression, logical function and easier to use for multi-objective functions. So it is suggested for optimization of other floating platforms such as spar and tension leg platforms (TLP).

9. Reference
APPLICATION OF TUNED LIQUID COLUMN DAMPER FOR PITCH MOTION REDUCTION OF SEMISUBMERSIBLE FLOATING PLATFORMS

Hamidreza Feizian1 and Roozbeh Panahi2

1) PHD Candidate, Civil Engineering Department, Tarbiat Modares, Tehran, Iran, h.feizian@modares.ac.ir
2) Assistant Professor, Civil Engineering Department, Tarbiat Modares, Tehran, Iran, rpanahi@modares.ac.ir

1. Introduction

Offshore floating platforms have been employed worldwide in many applications for decades. For the sake of safety, human comfort and increasing productivity it is suitable to reduce motions and vibrations of platforms as much as possible. Many researches have been conducted on different methods to reduce the motions of floating platforms. Recently, utilizing passive methods such as Tuned Mass Dampers (TMD) and Tuned Liquid Column Dampers (TLCD) are being considered increasingly [1]. Among all passive methods, using TLCD is the most attractive one, because of its low cost, easy handling and few maintenance requirements [2, 3].

TLCD is a U-shaped tube, containing a liquid (commonly water). Usually, there is at least one orifice at the middle part of the tube causing a head loss when the liquid oscillates in the tube, resulting in energy dissipation.

Application of TLCD in floating platforms is mostly studied on Tension-Leg Platforms (TLPs), so far. Lee et al. incorporated a TLP with a huge TLCD for the first time on the topside in order to reduce the wave-induced vibration of the floating platform [4]. In order to calculate the response spectrum of the structure-TLCD system, they derived a transfer function for each DOF and multiplied it by the wave spectrum.

A TLP, equipped with an underwater TLCD; known as UWTLCD; was later studied experimentally by Lee and Juang [5]. The pontoons of the platform were used as the vertical parts of the TLCD while connected with a horizontal small diameter tube. This scheme was used to equip the platform with a TLCD without any space occupation on the topside.

2. Equations of Fluid Motions in TLCD and Generated Forces

A conventional U-shaped TLCD is shown in Figure 1. The damper is assumed to be attached to the main structure and they move in the X, Y and θ directions. Using a general form of the Bernoulli’s equation for a moving reference frame as done by Hochrainer the complete form of the equation of the water surface in the TLCD tube can be derived [6]:

\[ L_{ew} \ddot{w} + \frac{1}{2} \xi \dot{w} \dot{w} + 2gw = -B.\ddot{X} - (2w)\ddot{Y} - B \left( \ddot{\theta} \left( \frac{L_B}{2} + D \right) + g\dot{\theta} \right) \]  

(1)

where \( L_{ew} \) is defined as 
\[ L_{ew} = L - B \left( 1 - \frac{A_v}{A_h} \right) \] and L and B are the total length of internal fluid and width of the TLCD respectively as shown in Figure 1. \( A_v \) and \( A_h \) are the cross section area of vertical and horizontal tubes. \( w, \dot{w} \) and \( \ddot{w} \) are denoting the fluid displacement, velocity and acceleration in the vertical part of the tube. \( g \) is the gravity acceleration and \( \xi \) is head loss coefficient depending on the blocking ratio of the damper orifice. \( D \) is the distance of the rotation center from the center of the TLCD.

The forces generated by damper as produced by the fluid fluctuations within the TLCD can be calculated as described by Hochrainer and Xue et al. as below [6, 7]:

\[ F_x = -pA_vB \ddot{w} \]  

(2)

\[ F_y = -p(2A_v \dot{w} \dot{w}) \]  

(3)

\[ M_\theta = -pA_vB(H + D)\dot{w} \]  

\[ -pgA_v \left( \frac{2 - B}{2} + B \frac{A_h}{A_v} \right) D\dot{\theta} + Bw \]  

(4)

3. Numerical Modelling of Semisubmersible-TLCD System

The drilling rig, GVA4000 is selected to investigate the effect of utilizing TLCD on response mitigation of semisubmersibles.
Numerical modelling is carried out in two stages. The first stage is done in frequency-domain to provide hydrodynamic specifications of the floating structure. In the second stage, the position of the semisubmersible with and without a TLCD is calculated at each time-step under irregular waves. TLCD forces and moments are calculated and exerted to the structure at such steps using the aforementioned equations.

Simulation of the floating platform motions is performed by Ansys®AquaTM (17.0) and TLCD forces and moments are calculated in the time-domain stage by means of a fortran code. The fortran code is coupled with Ansys®AquaTM via an external dynamic library link file. It is good to remember that the software is based on boundary element method which simplifies calculations when compared with other methods e.g. the one proposed by Panahi et al.[8]

Numerical model of the floating platform (without TLCD) is verified by frequency-domain results of numerical and experimental simulations of Clauss et al. as shown in Figure 2 [9]. Comparison shows that the current numerical results are well matched with the experimental and numerical data of the mentioned study.

![Figure 2. GVA4000 pitch RAO – comparison of current numerical results with Clauss et al. [9]](image)

Time domain analysis of the floating platform has been done with active and inactive TLCD on the structure. Also, the effect of head loss coefficient ($\xi$) variation on damper efficiency is studied. The frequency of the applied TLCD is about 0.1 Hz and the ratio of $\frac{\text{TLCD}_{\text{Mass}}}{\text{Structure}_{\text{Mass}}}$ is approximately 4.5 percent. Irregular wave time series, based on JONSWAP spectrum are applied with a significant wave height of 5 meters and pick period of 11 seconds.

In order to better study the phenomenon, time domain results are transformed into response spectrum. Figure 3 shows the response spectrum in the range of natural frequency of the structure, without and with TLCD for different values of head loss coefficient. It can be concluded that application of TLCD has a significant mitigation effect on pitch response of the semisubmersible platform and greater values of $\xi$ would result in more efficiency of damper. It means that bigger blocking area of the damper orifice will cause more energy dissipation. It is noteworthy that the greatest value of head loss coefficient ($\xi=55$) is related to about 80 percent orifice blocking ratio, according to Wu et al. calibration tests [10]. It is also concluded that there is not a linear relation between head loss coefficient and mitigation effect of the damper. As it can be observed in Figure 3, by increasing the value of $\xi$ from 35 to 55, a small reduction in pitch response is achieved.

![Figure 3. GVA4000 pitch response spectrum without and with TLCD](image)

4. References
1. Introduction

Investigating local scour around an off shore pipeline has received most attention both experimentally and numerically in the last few years. So far, most of the studies on scour below the pipeline are related with a single pipeline. Offshore pipelines of different diameters are sometimes laid together as a bundle due to technical or economical attentions. A pipeline bundle comprises a large pipe and a few of small pipes. The pipelines in the bundle could either be in direct contact with each or be separated by small gaps. The most popular configuration of pipelines bundles consists one large pipeline with a small one installed exactly above the large one, as shown in Figure 1. It is supposed that the existence of a small pipeline and a gap between the two pipes will affect local scour below the piggyback pipelines. In this study the effects of the gap between the two pipelines in piggyback configuration on local scour profiles are investigated numerically.

2. Hydrodynamic Observation below a Single Pipeline

The numerical model developed in this study is validated against hydrodynamic model results available by Figan Htipoglu (2003-Ocean Engineering) on Flow around a cylinder in a steady current. Figure 2 Shows the streamlines near the pipeline. This agrees with the experimental observation reported by Hatipoglu (2003). The length of separation region was observed in the downstream of cylinder in the case of surface mounted cylinder (G/D=0) was 0.54 m.

3. Scour below a Single Pipeline

The water flume experiment reported by Mohammadi (2014) is simulated for this purpose in this study. In order to allow for a direct comparison with the experimental results, the computation is carried out under the conditions as close as possible to those specified in the physical experiment by Mohammadi (2014) by FLOW 3D. In Mohammadi’s experiments, the water depth was 0.15 m, the pipeline diameter was 0.4 m, the grain size was 0.00078 mm, the Shields parameter was 0.03. The pipeline diameter and sediment particle size used in the computations are the same as those in the experiment.

Figure 3 Shows the scour under the cylinder at 10 s. In the continue Figure 4 shows the scour to 600 s.
Figure 4. Scour at 600 s

4. Scour below a Piggyback Pipeline

Local scour below a piggyback pipeline is investigated using the validated numerical model by FLOW 3D. The configuration of the piggyback pipeline is shown in Figure 1. The large pipeline diameter (D) is 60mm, the small pipeline diameter (d) is 15mm, the flow velocity is 0.262 m/s and the sand size and computational domain are set the same as those in the single pipeline case studied before. The diameter ratio of the small pipeline to the large one is d/D = 0.25. Computations are carried out for the gap ratio of G/D = 0.0, 0.25.

Figure 5 shows the final result for scouring of these modes at 600 s.

(a) G/D = 0

(b) G/D = 0.25

Figure 5. G/D = 0.0, 0.25

5. Conclusion

It can be seen that local scour below piggyback pipelines with different diameters has been affected noticeably with the existence of the small pipeline. The scour depth below the piggyback pipeline is greater than single pipeline. It is assumed that the scour profile should reach to the scour profile for the single pipe case when G/D is very large. It can also see that the scour depth in front of the pipelines decreases with the increase of G/D.

6. References

CONSTRUCTION OF ARTIFICIAL ISLANDS BY USING STEEL CYLINDERS (CASE STUDY: HONG KONG-ZHOU-MACAO BRIDGE)

Khaled Pourali¹, Mohammad Javad Ketabdari²* and Arno Petrosian³

¹,²,³) Faculty of Marine Technology, AmirKabir University of Technology, Tehran, Iran, *ketabdar@aut.ac.ir

1. Introduction
Artificial islands refer to islands that are created by humans in aquatic environments without the involvement of natural agents. Construction of artificial islands may perform with goals such as extracting oil and natural resources, military, development of ports and coastal towns and for entertainment purposes. In this research, a new method for constructing an artificial island has been investigated. In this method, the large diameter steel pipes create a temporary dam against the water and wave forces. Building the island inside the enclosed area is the next step. Therefore the issue that has been addressed in this leading research is human advancement in coastal areas and near coastal areas by construction of artificial islands. It is possible to encourage governments and countries into the construction of such structures. For example, the countries of the southern margin of the Persian Gulf, such as Bahrain, Qatar, Oman and the United Arab Emirates, have been brought to the construction of artificial islands, due to the shortage of coasts and withdrawals from the single-product economy [1]. Because of the increase in the population of China to facilitate the flow of citizens and the link between the two cities of Hong Kong and Zhou Macau, two artificial islands as two spans of the submarine tunnel have been used [2].

2. Design and Implementation Consideration
In design of artificial islands, factors such as depth of water, wave height, ice conditions, tidal range, sea floor conditions, earthquake risk, borrowing resources, and environmental conditions are influential [3].

3. Construction Steps
3.1. Geometric Shape and Island Location
Figure 1 shows the submarine tunnel route in China. There are two openings of a submarine tunnel that longitudinally is 6.7 km. The project began in mid-December 2010 with the construction of a Western artificial island. The goal is to complete the island in 20 months or less. The arrangement of the western island is shown in Figure 2. As shown in this figure the construction of the island is divided into two phases. Each of these islands are made by 67 cylindrical steel that their thickness and weight are 16 mm and 450 tons respectively [4].

3.2. Manufacturing and Carrying Cylindrical Tubes
Cylindrical tubes made of corrosion-resistant and anti-abrasion steel in shipyard and offshore industries near the shore (Figure 3). The steel cylinders are designed to penetrate 15 meters in a permeable layer of alluvial layers with SPT N>8. The length of each of these cylinders is more than 45 meters. These cylinders are shipped to special vessels carrying large industrial shipments. Each vessel has a capacity of more than 10,000 tons.

3.3. Installing the Cylindrical Tubes
After being transferred the cylindrical tubes with special vessels from the workshops to the desired location (Figure 4), they are installed using marine cranes and other specially designed vibrating equipment at the desired location (Figure 5). On each of these cylinders, the guide lines are laid out to ensure that they are not removed from the installation line. After the installation process, the cylinders are filled with dredging ships that sucks the sea bed so that they fill the cylinders 2 meters above sea level from the bedding (Figure 6).
3.4. Preparing The Inside Of The Island

After the above steps, with the help of suction equipment, the water inside the enclosed area by steel cylinders is drained. Approximately the discharge of this water last for 90 days, Then the soft soil layer is removed to depth of approximately 20 meters. After that, the area is filled with sand dunes to a height of about 5 m (Figure 7).

4. Conclusions

Steel cylinders method can be used to construct artificial islands in deep water. One of the advantages of this method is the speed of its implementation. But this method is very expensive due to the need for special equipment. Furthermore the risks for using such a method include:

- Possibility of tilting cylinders during installation.
- Instability during construction, since each cylinder is connected with large parts by welding, so the likelihood of instability in these cylinders increases.

By creating additional reinforcing elements in each cylinder, it is possible to reduce the chance of their tilting.

5. References


A NOVEL MODELLING APPROACH FOR EARTHQUAKE-INDUCED SEABED LIQUEFACTION

V.S. Ozgur Kirca1,2, Giray Civak3 and B. Mutlu Sumer2

1) Faculty of Civil Engineering, Istanbul Technical University, Istanbul, Turkey, kircave@itu.edu.tr
2) BM Sumer Consultancy & Research, Istanbul, Turkey
3) Graduate School of Science, Engineering and Technology, Istanbul Technical University, Istanbul, Turkey, civakg@itu.edu.tr

1. Introduction

Under cyclic loading conditions, shear deformations gradually rearrange soil grains and the pore water pressure increases in saturated, undrained soils at the expense of pore volume. In case of the presence of sufficient time and room, the pore water pressure reaches such a level that exceeds initial effective stresses and because of disappearing stresses between individual grains, the soil acts like a fluid, loses the ability to bear any load thus it fails. The term “liquefaction” is used to define this phenomenon in engineering terminology. It has been recognized that soils that can be liquefied under cyclic conditions are basically limited to fine soils or composite soils such as silty or clayey sands. As it was mentioned in the definition, liquefaction susceptibility is closely related with sort of soil parameters and cyclic loading conditions as expected. Through the years many soil failures caused by earthquake induced liquefaction has been reported by engineers and scientists. For example; in 1999 Kocaeli earthquake, an extensive liquefaction caused sinking of breakwaters, large displacements of quay walls and huge settlements of backfills which were resulted in serious damages to coastal structures (Figure 1).

![Figure 1. Liquefaction induced lateral spreading case after 1999 Kocaeli Earthquake.](image)

There are limited comprehensive analysis methods for earthquake induced seabed liquefaction and ordinarily, specially prepared charts where the dimensionless parameter Cyclic Stress Ratio (CSR) is plotted versus corrected SPT blow counts gathered from Standard Penetration Tests to define relative density of the soil have been used by practitioners to assess liquefaction susceptibility, albeit as a first approximation [1].

In this study; an experimentally-validated mathematical model [2], which was originally developed for wave induced liquefaction, was modified and adapted to predict earthquake-induced seabed liquefaction. While earthquakes and waves both produce cyclic shear stresses (and accordingly cyclic shear deformations) in the seabed, the ones induced by earthquakes are more severe compared to that caused by waves. The early results obtained from this modified mathematical model were compared to the widely used CSR-SPT assessments to investigate liquefaction susceptibility.

2. Methodology

The method used in this study basically covers the comparison of the results obtained from a series of parametric model runs with the estimated ones from recent CSR-SPT liquefaction susceptibility assessment procedures. Primarily, earthquake-soil interaction was analyzed, then the shear stresses caused by cyclic loadings during an earthquake were used as the model input. Below equation (1) where; g is gravitational acceleration and \( \gamma' \) is the total specific weight of the soil, was used with soil depth \( z \) dependent proper reduction coefficients \( r_d \) to translate maximum cyclic ground acceleration during an earthquake \( a_{max} \) into average equivalent cyclic shear stresses, \( \tau_a \), in the seabed [3].

\[
\tau_a = 0.65 \frac{\gamma' z}{g} a_{max} r_d \quad (1)
\]

The modified one-dimensional mathematical model basically simulates the pore pressure build-up under cyclic loading conditions depending on the intensity of shear stresses/deformations. More than a hundred parametric runs containing different cyclic loading conditions were done for various relative densities \( D_r \) and pore water pressure levels where liquefaction occurs defined to calculate Cyclic Stress Ratio’s by using Equation 2 shown below [3].

\[
CSR = \left( \frac{\tau_a}{\sigma'_0} \right) = 0.65 \frac{a_{max}}{g} \frac{\sigma'_0}{\sigma'_0} r_d \quad (2)
\]

Because of the incapability of obtaining undisturbed specimens to be analysed in the laboratory, empirical approach based in-situ penetration test results gained more popularity among engineers, and therefore, SPT
based assessments were referred to in this study. Table below summarizes the approximated relation between density indexes and numbers of corrected SPT blow counts [4].

Table 1. SPT with corrected blow counts.

<table>
<thead>
<tr>
<th>Corrected SPT blow count</th>
<th>Relative Density (Dr)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
<td>0.15</td>
</tr>
<tr>
<td>8</td>
<td>0.35</td>
</tr>
<tr>
<td>15</td>
<td>0.50</td>
</tr>
<tr>
<td>25</td>
<td>0.65</td>
</tr>
<tr>
<td>42</td>
<td>0.85</td>
</tr>
<tr>
<td>58</td>
<td>1.00</td>
</tr>
</tbody>
</table>

Furthermore, a sensitivity analysis was conducted by changing the values of related soil parameters. As an addition exercise, the results obtained from the adapted mathematical model for pore pressure levels were compared to the ones measured during a centrifugal shaking table test set up to investigate earthquake induced liquefaction.

3. Results and Discussions

In this study, pore pressure build-up in the seabed under cyclic loading conditions was modelled via one dimensional experimentally validated mathematical model and then, results were compared to recent and widely used Cyclic Stress Ratio (CSR) and Cyclic Resistance Ratio (CRR) approaches to investigate liquefaction susceptibility. Figure 2 below briefly compares both model and traditionally used SPT based earthquake induced liquefaction potential assessment results.

![Figure 2. Simplified base curve of CRR from SPT data together with modelled liquefaction results.](image)

In conclusion, early results showed that the adapted model has a potential to estimate earthquake-induced pore pressure buildup and seabed liquefaction, especially for soils with 30 or less SPT blow counts.

4. References


AXIAL COMPRESSION BEARING CAPACITY OF DRIVEN OFFSHORE PILES IN THE PERSIAN GULF – A CASE STUDY

Babak Ebrahimian 1 and Amir Hossein Shamshirgaran 2

1) Faculty of Civil, Water and Environmental Engineering, Shahid Beheshti University, Tehran, Iran, Email: b_ebrahimian@sbu.ac.ir, ebrahimian.babak@gmail.com
2) Faculty of Civil, Water and Environmental Engineering, Shahid Beheshti University, Tehran, Iran, Email: shamshirgaran.amir@gmail.com

1. Introduction

Cone penetration test (CPT) is broadly employed in the design of offshore piles. The reliability, high-quality results and continuous recording of soil resistance in depth are the CPT advantages which result in excellent performance of CPT rather than the other in situ tests. Moreover, the shapes of CPT and pile as well as the failure mechanisms developed during penetration are similar. These significant characteristics have motivated many researchers to propose direct estimation methods of pile bearing capacity using CPT data. Herein, the axial compression bearing capacity of an offshore steel-pipe pile driven in marine clay deposits of the Persian Gulf-South Pars field is estimated using three property-based static analysis methods including API (2011), FBV (Fugro, 1996) and NGI (2005) as well as ten popular direct CPT-based methods including Aoki & Velloso (1975), Penpile (1975), Shmertmann (1978), de Ruiter & Beringen (European/Dutch, 1979), Tumay & Fakhroo (Cone-m, 1981), Bustamanate & Gianeseli (LCPC/LCP, 1982), Price & Wardle (1982), Esfami & Fellenius (Unicone, 1997), Jardine et al. (ICP, 2005), and Niazi & Mayne (Enhanced Unicone, 2015). The question is why these methods have been selected? A majority of offshore piles worldwide has been designed based on API standard as the most common design code for offshore structures. In addition, the selected CPT-based methods in this study can be classified in two groups. The first group contains the more commonly used CPT-based methods which were mainly developed before the year 2000 and the second consists of the more recently developed CPT-based methods which were included in the commentary of the new 22nd Edition of the API RP 2A Recommendation. It is mentioned that the direct CPT methods developed based on older types of mechanical CPT equipment with no pore pressure measurement, apply total stress values. The total stress approaches govern the short-term behavior of piles capacity, whereas property-based static analysis methods apply effective stress values, and govern the long-term behavior of piles capacity. Determining the pile capacity in clay necessitates using CPTu sounding with pore pressure measurements. Details of the above CPT-based methods have been given in [2] and hence not reproduced here due to space constraints. For verification purposes, the results of the aforementioned methods are compared against the PDA records obtained from dynamic load tests conducted at various pile depths during jacket installation. The paper presents the predictive performance of the above thirteen methods. The considered pile is a tubular steel pile with around 90 m length, 1.52 m diameter, and 50.88 mm wall thickness. Water depth at the location of jacket structure is nearly 75 m. Soil and CPT data relevant to the location of corresponding offshore structure are shown in Figure 1. The profiles have a general trend of increasing linearly with depth; however, the values fluctuate in some occasional cohesionless granular lenses. In the South Pars field, the clayey soil is very soft to soft at above 20 m depth, stiff at 20–70 m depth, and very stiff to hard beyond 70 m depth. This layering pattern is dominant; it means that no considerable variation is seen in the entire field [1].

2. Results and Discussion

The calculated curves of skin friction, end and ultimate compression bearing capacities obtained from different methods are presented in Figure 2. The results have been generated through a series of Spreadsheets which were developed and precisely verified by the authors. It is seen that the results demonstrate a very wide range of variation in the predicted capacities. The methods yield skin friction, end and ultimate bearing capacities between 13400-72600 MN, 4100-16200 MN and 19600-78600 MN, respectively. The calculations confirm that the end bearing contributes very little to the total ultimate bearing capacity.
In order to evaluate the performance and applicability of different methods in predicting the axial compression capacity of piles in clay, PDA data and CAPWAP analysis results in EOD (End Of Driving) and BOR (Beginning Of Restrike) conditions are employed and depicted on Figure 2. It is observed that the EOD PDA data are mainly settled on the lower bound of bearing capacity curves. BOR results have been gained after 21 hours, 9 days, 29 hours and 263 days in PDA 2-BOR, PDA 4-BOR, PDA 6-BOR and PDA 9-BOR, respectively, to study the time and set-up effects on the long term behavior of soil-pile system. PDA 9-BOR has been chosen as a reference of measured pile capacity ($Q_m$) in long term and shown in Figure 3. As illustrated, the methods with green and red columns are close to and far from the PDA result, respectively. According to Figure 3, Eslami & Fellenius (Unicone, 1997), Price & Wardle (1982), Shmertmann (1978) and Jardine et al. (ICP, 2005) methods propose the best predictions among all methods. In contrary, API (2011), Aoki & Velloso (1975), NGI (2005), Fugro (1996), Penpile (1975) and Bustamanate & Gianeseli (LCPC/LCP, 1982) present the worse consistency with PDA result. Figure 3 confirms that API method shows the poorest performance and CPT-based methods generally provide more reliable estimates of pile capacity in clay than the API method.

3. References


A COMPARISON BETWEEN THE LATERAL RESPONSE OF MONOPILE IN CALCAREOUS AND SILICA SANDS BY CENTRIFUGE MODELING

Farzad Memari\(^1\), Mohammad Reza Rasouli\(^2\) and Majid Moradi\(^3\)

1) M.Sc. Student, University of Tehran, Tehran, Iran, fnemari@ut.ac.ir
2) Ph.D. Candidate, University of Tehran, Tehran, Iran, rezarasouli@ut.ac.ir
3) Associate Professor, University of Tehran, Tehran, Iran, mmoradi@ut.ac.ir

1. Introduction
By increasing the demand for renewable, sustainable and greener energy resources, the offshore wind farm industry is experiencing quick growth in many countries, such as the UK and Germany. It is expected for near-term (2020), and long-term (2050) offshore wind turbines (OWTs) play an important, paramount role in reducing greenhouse gas emission. The most common foundation type for offshore wind turbines is a single large diameter pile, termed a mono-pile, which the turbine is located on. Furthermore, they are used as breasting and mooring dolphins in port structures. As the diameter of such piles is envisaged to increase in future installations, there are concerns that current design methods are not applicable. Although many types of research have done on the lateral response of mono-piles, still a paucity of comprehensive data exists to make general rules. Such piles are always subjected to significant lateral loads due to the wind, mooring force and wave actions (Figure 1). This paper describes the results of a centrifuge modeling study of the response of mono-piles embedded in calcareous and silica sands subjected to monotonic lateral loading. The experimental program involved a comprehensive set of centrifuge modeling tests on reconstituted samples of calcareous and silica sands with a similar grain size distribution tested by utilizing the Geotechnical Centrifuge of the University of Tehran at similar conditions for comparison purposes. In this study, the main characteristics of mono-pile static lateral behavior are investigated.

2. Location of the Soil Materials
A comparison is made between the lateral response of mono-piles embedded in calcareous sand retrieved from northern coastal of Hormuz Island in the Persian Gulf and silica sand obtained from Firuzkuh, north of Iran (Figure 2). The Firuzkuh silica sand was selected to compare with Hormuz calcareous sand because it is sand widely used in Iranian geotechnical researches.

3. Soil Characteristics
The properties of the soil materials which were tested are shown in Table 1.

<table>
<thead>
<tr>
<th>Soil</th>
<th>( C_u )</th>
<th>( C_C )</th>
<th>( \varphi_{\min} )</th>
<th>( \varphi_{\max} )</th>
<th>( G_s )</th>
<th>( CaCO_3 )</th>
</tr>
</thead>
<tbody>
<tr>
<td>F161</td>
<td>1.54</td>
<td>1.13</td>
<td>0.58</td>
<td>0.865</td>
<td>2.65</td>
<td>1.03%</td>
</tr>
<tr>
<td>Hormuz</td>
<td>1.49</td>
<td>1.11</td>
<td>0.64</td>
<td>0.92</td>
<td>2.78</td>
<td>95.3%</td>
</tr>
</tbody>
</table>

Details of the soil fabric can be observed through a scanning electronic microscope (SEM) on a sample or a thin of the sample. To determine the shape of soil grains, the SEM images of the soil materials were prepared as well. Images of soil samples are shown in Figures 3.a and 3.b. These figures are showing clearly the difference in particle shape. The calcareous sand has fragile, angular and hollow particles as opposed to the hard, rounded silica grains.

Figure 1. The response of a pile to lateral load.

Figure 2. Location of Hormuz Island and Firuzkuh

Figure 3. Images of soil samples.
4. Centrifuge Modeling

As geotechnical and marine issues often have great dimensions and their modeling involves applying small exponential coefficients, there are a considerable amount of modeling-caused errors (scale effect) in them. If it is possible to equalize stress conditions at corresponding points of the model and real situations, the problem caused by scale effect errors will be largely negligible. The geotechnical centrifuge is a device that compensates the reduction of the stress caused by model shrinkage and reduces modeling errors through the local increase of gravitational acceleration by revolution. Soil models are placed on a swinging platform at the end of the centrifuge arm and then accelerated so that they are subjected to an inertial radial acceleration field of N times earth’s gravity g acting normal to the surface of the platform. In centrifuges, linear dimensions of a model are reduced \((1/N)\) based on the ratio of gravitational acceleration to acceleration of gravity (N). General scaling factors for different quantities in Ng space may be derived and summarized in Table 2.

### Table 2. Scaling factors for centrifuge modeling.

<table>
<thead>
<tr>
<th>Quantity</th>
<th>Scale Factor (Model / Prototype)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Acceleration</td>
<td>(N)</td>
</tr>
<tr>
<td>Stress and Strain</td>
<td>1</td>
</tr>
<tr>
<td>Length</td>
<td>(1/N)</td>
</tr>
<tr>
<td>Area</td>
<td>(1/N^2)</td>
</tr>
<tr>
<td>Volume</td>
<td>(1/N^3)</td>
</tr>
<tr>
<td>Mass</td>
<td>(1/N^3)</td>
</tr>
<tr>
<td>Force</td>
<td>(1/N^2)</td>
</tr>
<tr>
<td>Energy</td>
<td>(1/N^3)</td>
</tr>
</tbody>
</table>

#### 4.1. Geotechnical Centrifuge at the University of Tehran

The Geotechnical Centrifuge at the University of Tehran has components such as A) floating basket, B) centrifuge beam, C) counterweights, D) hydraulic rotary connections and electronic sliding connections, E) driving system (driver), F) aerodynamic hood, G) automatic control system of balance in rotation and other minor components. It is shown in Figure 4.

5. Experimental Testing

Concerning the limited space in a centrifuge, driving system and its components should occupy minimum volume. As there is no appropriate loading system in a centrifuge, it was decided to consider a lateral loading setup with a mechanical stepper motor for imposing static loading in proportion to soil chamber. Performance mechanism of the system is in a way that the rotation of the stepper motor is transferred to two pulleys and a belt round them. This motion is imposed to the end of a ball screw. The ball screw converts the rotational movement created in the motor into a translational movement.

Here, a stainless steel pipe with a 51 mm diameter was considered for modeling mono-pile to examine and compare the response of mono-piles embedded in calcareous and silica sands subjected to monotonic lateral loading. Lateral loads were applied on piles with similar embedded depth and load eccentricity. They were tested by the geotechnical centrifuge facility, and all the experiments were performed under the same acceleration.

6. References

DAMAGE DETECTION IN JOINTS LOCATION OF OFFSHORE JACKET PLATFORMS

Amin Rahimzadeh1, Ahmad Reza Mustafa Gharabaghi2 and Mohammad Reza Chenaghlo3

1) Faculty of Civil Engineering, Sahand University of Technology, Tabriz, Iran, a_rahimzadeh@sut.ac.ir
2) Faculty of Civil Engineering, Sahand University of Technology, Tabriz, Iran, mgharabaghi@sut.ac.ir
3) Faculty of Civil Engineering, Sahand University of Technology, Tabriz, Iran, mrchenaghlo@sut.ac.ir

1. Introduction
The fixed offshore jacket platforms are always at exposure of the severe environment condition of sea. Because of the high importance of these structures, early detection of damages, when they are minor, and their repair can prevent from further serious dangers. According to a report from the damages caused to the jacket platforms in the NFS1 from 1974 to 2016, mostly damages are of crack type at the joints [1]. The purpose of the paper is to detect damages in jacket platforms with respect to minor damages at the joints. Among different methods of damage detection, signal processing methods have better performance in health monitoring of complex structures with nonlinear behavior. By studying the researches of Kim and Melhem, Byissa et al., Rakowski, Zhu and Huang et al. [2-6], it is found that the wavelet transform method has a high ability to detect minor damages. In this paper, the wavelet packet transform is used to extract the wavelet packet energy rate for damage detection at the joints. However, these detects have errors. Therefore, a target function formulated based on the wavelet packet energy rate index (WPERI). Then, the LSE2 optimization optimizes target function to obtain a new index (WPTLSE) to reduce the errors and improve the results.

2. Modeling and Verification
For modeling, the SPD16 platform of the South Pars Gas Field Development Phase 12 in the Persian Gulf was simulated in Abaqus. According to Figure 1, for substructure modeling, the Wire element based on the cross section characteristics, and for deck modeling, Shell element based on its loading characteristics is used. For the pile-soil interaction the equivalent length of pile with fixity point at the end was used. Water effect is considered as an added mass and hydrodynamic damping.

To verify the structure modeling, the natural frequencies of the first three modes were compared to the original designed structure frequency that has acceptable difference below 8%.

3. Identification Method
The principles of damage detection methods are often based on the comparison of a structural system in two different states (damaged and undamaged). Accordingly, structure signals must be available in the two phases, and then processed. Wavelet packet transform is one of the signal processing approaches provides level by level decomposition of signal. However, processed signal properties should be extracted to be comparable in two different states.

Wavelet packet energy, \( E_j \), at the level \( j \) of signal decomposition is the extracted property calculated according to the following equation:

\[
E_j = \frac{1}{2} \sum_{-\infty}^{+\infty} f_j^m(t)dt = \sum_{n=-\infty}^{+\infty} \sum_{a=1}^{2^j} f_j^m(t) f_j^a(t)dt
\]

And the wavelet packet energy rate (WPERI) at the level \( j \) is calculated according to Equation 2:

\[
WPERI = \sum_{j=1}^{2^{j+1}} \left( \left( E_{j+1}^{sd} \right)^{1/2} - \left( E_{j+1}^{wd} \right)^{1/2} \right)
\]

In this equation, \( sd \) and \( wd \) refer to damaged and undamaged states respectively. \( f_j^i \) is the \( i \) th component of the \( j \) th level of decomposition. Lotfollahi Yaghin et al.
used wavelet packet energy rate as a damage index by applying a damage threshold (WPERI-UL, WPERI) [7]. In this paper, this index is used as WPERI and is compared to the proposed index.

In this research, a two-step method for damage detection is designed. At first, by shaking an undamaged structure under an impact load, the displacement-time signals are taken in 20 seconds for 20 points of the structure at the joints location, where cross platform floor elements with leg elements as shown in Figure 1. Then, by causing damage to each joints, while other joints are undamaged, and re-shaking the structure under the same load, displacement-time outputs are obtained again at the same previous points. Here, all damages are considered as 5% decrease in cross-section along 2mm of the member connected to the joints. At this step, WPERI is calculated according to equation (2). In this paper, based on the signal frequencies and trial and error method, the db5 mother wavelet is used at the fifth decomposition level. Therefore, each joint will then have a WPERI index for different damages.

At the second, a data-based optimization, in which the information extracted from the system used as training data for target function is applied. In this step, the WPERI is used to train the target function and LSE optimization method is used to optimize target function. By considering the indices obtained from each joint in each damage as a component of a vector, each damage will be a vector \([X_i]\):

\[
\begin{bmatrix}
X_1 \\
X_2 \\
\vdots \\
X_i \\
\end{bmatrix} \rightarrow \text{Damage in } i_{th} \text{ mode}
\]

Now, any other damage in the structure will create a unique vector by WPERI as \(Y\). By weighing the vector \([X_i]\) with \(A_i\), a target function is formed based on these vectors according to equation (5). To minimize this target function, the LSE algorithm is used to find the weight combination of \(A_i\) that is more similar to \(Y\). This combination of \(A_i\) is introduced as WPTLSE index by using the threshold of damage and it is used to identify the location of the damage.

\[
A_i \rightarrow \begin{bmatrix}
X_1 \\
X_2 \\
\vdots \\
X_i \\
\end{bmatrix} - Y \quad \text{if} \quad 0 \leq A_i \leq 1
\]

(3)

4. Results

In this section, the performance of the proposed WPTLSE index is evaluated to detect the damage location by applying several damage scenarios. The scenarios were considered while two damages exist in the structure and the results are compared between WPTLSE and WPERI. In this paper, merely the combination of damage in No.1 joint with other joints has been investigated.

It is determined that in 38 possible cases of damage, the WPERI index detected 24 damages successfully and 71 errors. Where, WPTLSE detected 28 damages successfully and 14 errors. The following graph shows the improvement in the results of the WPTLSE index.

5. Conclusion

The purpose of this research was minor damage detection at the joints of fixed offshore jacket platform. With this aim, the method of wavelet packet transform was used as an appropriate tool for minor damage detection. Then, the wavelet packet energy rate was calculated and the WPERI index obtained by applying a threshold of damage. However, due to the number of errors identified in this method, the new index, WPTLSE, was introduced by creating a target function based on the wavelet packet energy rate and using the LSE algorithm. Several damage scenarios were applied to the structure in existing of two damages at the same time and then WPERI and WPTLSE indices were compared. The results indicate an increment in successful detection of damages, reduction in errors, and improvement in the accuracy of using the WPTLSE index.

6. References


LONG TERM STRESS RANGE DISTRIBUTION OF THE RISER OF AMIRKABIR SEMISUBMERSIBLE PLATFORM UNDER THE EFFECT OF WAVES

Zohreh Sadat Haghayeghi1 and Mohammad Javad Ketabdari2

1) Marine Engineering Department, Amirkabir University of Technology, Tehran, Iran. z.haghayeghi@aut.ac.ir
2) Marine Engineering Department, Amirkabir University of Technology, Tehran, Iran. ketabdar@aut.ac.ir

1. Introduction
Top-tensioned risers, are prone to high amplitude vibrations under the effect of waves, vortex shedding, vessel motion and internal flow. Due to the oscillating nature of forces acting on the riser, it experiences varying stress ranges which leads to fatigue damage. In this research the riser of Amirkabir semisubmersible rig was modeled to estimate the long-term stress range distribution over its length. For this purpose a FEM model of a tensioned Euler-Bernoulli beam was developed for simulation of riser’s motions. The vibration of riser under the effect of waves in the long-term was calculated in the time domain for different sea states from the wave scatter diagram. The riser has been analyzed in each sea state of the wave scatter diagram by 50 times to reduce the sampling error. Then a rainflow cycle counting method was applied to find the stress ranges and the number of their occurrences. Finally a Weibull distribution was fit to the stress ranges to find the distribution of stress ranges in the lifetime of riser.

2. Riser Analysis Methodology
The equation of motion of a riser connected to a semisubmersible rig as shown in Figure 1 can be written as [1]:

\[
EI \frac{\partial^4 u(z,t)}{\partial z^4} + T(z) \frac{\partial}{\partial z} \left( \frac{\partial^2 u(z,t)}{\partial z^2} + m \frac{\partial^2 u(z,t)}{\partial t^2} \right) = f(z,t)
\]  

(1)

where \( EI \) denotes the flexural rigidity, \( m \) mass, \( T(z) \) the variable tension along riser defined by:

\[
T(z) = T_t - W_s (1 - z)
\]  

(2)

%W_s is the submerged weight of riser, \( f(z,t) \) is the transverse force on the riser calculated via the following relation [2]:

\[
f_s(z,t) = \frac{1}{2} \rho D_u \left[ U_c + U_w - \bar{u} \right] \left[ U_c + U_w - \bar{u} \right] + \rho \frac{\pi D^2}{4} C_m (1 - \bar{u})
\]  

(3)

In this equation \( U_c \) is the marine current velocity along the riser:

\[
U_c = U_c^t \left( 1 - \frac{z}{R} \right)
\]  

(4)

and \( U_w \) is the wave particle velocity, \( \bar{u} \) and \( \bar{B} \) are riser velocity and acceleration respectively while \( C_d \) and \( C_n \) denote the drag and added mass coefficients.

The pinned-pinned boundary conditions has been applied to the FEM model of riser and the stress in the cross section of riser due to bending moment and top tension can be written as [3]:

\[
\sigma_z(z) = \frac{T_t}{A_e} + \frac{D_o}{2} \frac{\partial^2 u}{\partial z^2}
\]  

(5)

where \( A_e \) is the riser’s cross-sectional area. The model dimensions and parameters used in this study are listed in Table 1.

Table 1. Riser properties.

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Outer diameter (( D_o ))</td>
<td>0.53 m</td>
</tr>
<tr>
<td>Inner diameter (( D_i ))</td>
<td>0.48 m</td>
</tr>
<tr>
<td>Riser length</td>
<td>800 m</td>
</tr>
<tr>
<td>Modulus of elasticity</td>
<td>2.07 GPa</td>
</tr>
<tr>
<td>Water density</td>
<td>1025 Kg/m³</td>
</tr>
<tr>
<td>Steel density</td>
<td>7850 Kg/m³</td>
</tr>
<tr>
<td>Inner flow density</td>
<td>998 Kg/m³</td>
</tr>
<tr>
<td>Added mass coefficient (( C_m ))</td>
<td>2</td>
</tr>
<tr>
<td>Drag coefficient (( C_d ))</td>
<td>0.7</td>
</tr>
<tr>
<td>Current velocity at surface (( U_c^t ))</td>
<td>0.5 m/s</td>
</tr>
<tr>
<td>Top tension (( T_t ))</td>
<td>3000 KN</td>
</tr>
<tr>
<td>Water depth (( h ))</td>
<td>800 m</td>
</tr>
</tbody>
</table>

Figure 1. Schematic of riser in the marine environment.
3. Long-term Wave Data

Wave data used in this research was provided by KEPCO (Khazar Exploration and Production Company). The wave scatter table contains the characteristics of 87 possible sea states. Figure 2 shows the contour plot of probabilities of occurrence of sea states of the wave scatter table.

For each combination of $H_s$ and $T_p$, an irregular wave is produced from a Pierson-Moskowitz spectrum as [4]:
\[
S_H(\omega) = \frac{2H_s^3}{T_p^5} \exp\left(-\frac{2H_s^2}{T_p^2}\omega^2\right)
\]

Figure 2. The logarithmic contour plot of probability of occurrence of wave height and period.

4. Stress Time History Statistics

As previously stated, the riser was analyzed in each sea state 50 times to reduce the modelling error. The stress time histories in 200 points along the riser were selected to find the stress ranges and their numbers. Finally a Weibull distribution was fit to these stress ranges to find their long-term distribution. The distribution is as follows [5]:
\[
F_S(s) = 1 - \exp\left(-\frac{s}{A}\right)^B
\]

The result of this analysis can be directly applied in the prediction of the lifetime of the structure. The Weibull distribution for the long-term stress distribution of midpoint of riser is displayed in Figure 3.

This distribution can be used to acquire the probable hotspots. A more refined analysis may be required for places where the Weibull distribution has higher amounts of shape and scale parameters (A and B).

Figure 3. Weibull distribution fit to the stress ranges of midpoint of riser.

Figure 4. The parameters of Weibull distribution fit to stress ranges along the riser vertical axis.

The results of the analysis for 200 sections over the length of riser have been finally plotted in Figure 4.

5. References

1. Introduction

In recent years, various methods have been innovated and developed for attaining clean and renewable energies. In this regard different types of wind turbine at land and sea have been used for transforming kinetic energy to electrical energy. By considering cumulative power of wind at marine environment and also increasing cost of construction, implementation and operation in the environment, it is necessary to perform serious researches to optimize design costs. Among this, analysis of geometry parameters effects on applied forces on mooring and establishment of foundation in floating wind turbines and stability maintenance and optimization of mooring cost is very important. According to global statistics up to 2050, 12% of electricity generation in world will be obtained from renewable resources. The length of Iran shoreline is 5800 Km and the area of sea is 1900 Km² that cause a good availability to high potential of renewable energy of wind [1].

A few studies had been performed by researchers on stability and establishment of offshore platforms and wind turbines. Among them Larsen analyzed spar platform and optimization its geometry [2]. Jain and Agarwal studied mooring behavior at spar platform affected by waves [3]. Raeis Zade and Motahar stated that use of turbine with horizontal axis is optimal on Persian Gulf [4]. Ketabdari and Mirzaee Sefat studied interactions of linear waves with behavior of spar platform and modeled structural behavior against linear waves [5]. In present study, effect of geometry changes of different parts of wind turbines on applied forces and platform stability modeled and simulated by ANSYS AQUA Software and its effect on applied forces on mooring by considering JONSWAP wave spectrum are evaluated.

2. Force Estimation

Diffraction-dispersion theory was used for estimating applied forces on floating turbine structure. ANSYS AQWA Software uses 3D panel model for object with arbitrary shape and the mean of progress rate for estimating hydrodynamic forces and movements of floating turbine in waves. Panel method is a numerical method for estimating potential around object based on Green Integral theory [5]. In this method there is no limitation for form and shape of the body and it is assumed that oscillation amplitude of fluid respect to dimensions of object is small. Since every panel meet principles of potential theory, flow separation is ignored.

Here, the fluid is incompressible and non-rotational type and fluid field is defined as total potential function obtained by sum of functions:

$$\phi(x,y,z)e^{i\omega t} = \left(\phi_0 + \sum_{j=1}^{\infty} \phi_j x_j \right) e^{i\omega t}$$

This potential function should be applied on following Laplace equation:

$$\nabla^2 \Phi = \frac{\partial^2 \Phi}{\partial x^2} + \frac{\partial^2 \Phi}{\partial y^2} + \frac{\partial^2 \Phi}{\partial z^2}$$

In this study proportional to 25 m depth, for JONSWAP spectrum the Gamma spectrum factor of 3.3 and peak frequency of 0.125 Hz are considered to model real condition of Persian Gulf. It leads to a significant wave height (Hs) of 4 m for such a condition. In the present study a mini spar wind turbine with four catenary type mooring cables were used (see Figure 1). Results of these modeling for maximum tension in cables were presented. Findings show that tension in mooring cable is increased with increase of spar diameter which has exponential relation. In the case of analysis of effect of draft depth of spar on mooring force of floating wind turbine, several model with different depth including 7.5, 10, 12.5 and 15m were generated and studied (see Figure 2). Figure 3 shows the cables tension against spar draft. This shows the rate of increase in maximum mooring forces. On the other hand, induced forces on moorings with 100, 150 and 200 mm diameter were calculated in order to study applied forces from floating wind turbine with 2m diameter and 10m draft depth. This shows that average mooring forces increased up to 30% by increasing the mooring diameter (see Figure 4).
3. Conclusion

In this paper a mini spar wind turbine with four catenary type mooring cables were modeled in sea state of Persian Gulf by Ansys Aqwa software. The most important results are as follows:

- Mooring forces of floating turbine increased by increasing diameter of spar, while the rate of this increase decrease by increasing the diameter of spar.
- Relation between spar diameter and maximum tension is an exponential relation.
- Maximum value of mooring force increases by increasing the draft diameter of spar.
- The average value of mooring force which is generated in different models with different diameter of mooring is increased up to 30% by increasing the mooring diameter.
- By increasing the moorings diameter, their applied forces increase.

4. Reference

BEHAVIOUR OF SEASTAR TLP IN CASPIAN SEA WAVE CONDITION USING NUMERICAL MODELING

Ali Firoozpur1, Mohammad Javad Ketabdari2 and Farhood Azarsina3

1) Science and Research Branch of the Islamic Azad University, Tehran Iran, ali.firoozpur@gmail.com
2) Faculty of Marine Technology, Amirkabir University of Technology, Tehran-Iran, ketabdar@aut.ac.ir
3) Science and Research Branch of the Islamic Azad University, Tehran Iran, f.azarsina@srbiau.ac.ir

1. Introduction

SeaStar platform is a new generation of tension-leg platform, somewhat similar to Spar platform, which in addition to the benefits of Spar, benefits from the positive properties of tension-leg platform. SeaStar platform uses a cylindrical main body with three completely submerged appendages and provides the required buoyancy for load-bearing deck and equipment. Pre-stretched cables are connected to these three appendages [1]. Due to this remarkable buoyancy a SeaStar platform can handle up to about 1.8 times its own weight while for a Spar platform this is equal to 0.6. Also, less use of steel in this platform has made it very economical [2]. The movement of SeaStar platform to three degrees of freedom (Surge, Sway, Yaw) is comparative and the natural period is large while it is less in the other three degrees of freedom (Heave, roll, pitch) which are known as the hard movements of the platform [3]. The process results from initial stretching exercise sufficient to control the platform. Sreekumar et al. (2001) using linear diffraction radiation theory studied the behavior of a mini seastar TLP [4]. In 2006, Tabeshpour and his colleagues analyzed the hydrodynamics of a TLP under the influence of waves [5]. Abaiee et al. conducted numerical and laboratory studies on the seastar TLP platform in 2015 and presented and discussed the results [6]. Substantial benefits of the SeaStar platform as well as proving the existence of huge oil resources in deep waters of the Caspian Sea forced us to analyze this kind of platform and establish a positive step towards enhancing the country’s capacity in the field of oil extraction drilling. In this paper, the behavior of a mini SeaStar platform in deep water of Caspian Sea due to regular waves was investigated using Moses software. The RAO charts of the hard and soft movements of the platform are then estimated in different collision angles.

2. Numerical Modeling

To analyze the model, the SeaStar platform was implemented in the current environmental conditions of Amirkabir Semi-Submersible platform in a water depth of 700 m, wave height of 7 m and wave period of 9.4 sec using INCO wave modeling data in Figure 1 [7]. Figure 2 shows the SeaStar TLP geometry used in this research and its modeling in software.

Figure 1. a) Waves characteristics b) Bathymetry on the south coast of the Caspian Sea [7]

Figure 2. a) SeaStar platform geometry b) Its numerical modeling
3. Results

The RAO values of the modeled SeaStar TLP platform for 0, 45 and 90 degrees angles of the collision of waves are shown in Figures 3 and 4. As shown in these Figures, the RAO values for the Heave movement are almost linear and close to zero, which is due to the Tensioned type of platform moorings. Furthermore RAO of the rotational movement initially increases but decreases again after reaching its maximum. In fact the peak RAO of rotating movements occurs in the resonance vicinity where natural and wave frequency are close.

![Figure 3. SeaStar Surge, Sway and Heave RAO in different headings](image)

![Figure 4. SeaStar Roll, Pitch and Yaw RAO in 0 heading](image)

4. Conclusions

In this paper, the hydrodynamic behavior of the SeaStar TLP was studied. The purpose of the study was to investigate the behavior of the platform movements under the impact of collision waves in the Caspian Sea. In order to obtain the amplitude of the response from the 3D-diffraction theory, motions and wave responses are obtained using spectral analysis. The results indicates the proper behavior of the platform in the conditions of the Caspian Sea for operation. Therefore, it is recommended to use this platform at the deep points of the Caspian Sea.

5. References

1. Introduction

This paper is part of a collaborative project between ITMO Co. Iranian Sanat Sadaf Co. and Islamic Azad University (North Branch) to study the impact of hull and propeller fouling on vessel’s fuel consumption and emissions. It is well known that hull fouling, particularly in case of “hard or shell fouling”, can cause detrimental impact to the ship hydrodynamic performance. However, some cleaning practices may also lead to decreased lifetime of the fouling-control coating. The surface covered by sea adherence is about 80% of total ship hull and all sea adherences have to be cleaned before inspection. Underwater cleaning of the hull and the adhesive coating can therefore be performed in between dry-dockings in order to improve the ship’s efficiency. Furthermore, hull and propeller cleaning require different time to be performed and imply in distinct costs. Thus, there should be an optimal frequency of maintenance of hull and/or propeller depending on the ship specific fouling condition. The work presented here has consisted in performing several sea trials according to latest standards, in different configurations, in order to isolate the influence of the hull and the propeller underwater cleaning for a crude oil tanker.

Figure 1: Specially designed and fabricated robot for hull assessment and cleaning in Iran by authors

This project also is intended to provide a technical assessment of the emerging industry of hull cleaning robots to help the Iran Port and Maritime Organization with their goal of protection and conservation of ports and seawater environments. The project resulted in a comprehensive list of many hull cleaners that identified. In order to obtain effective and usable information for the PMO and the Department of Environment following objectives are defined:
1. Presentation of potential hull cleaning systems and important hull cleaning technologies
2. Study of constraints for evaluating hull cleaning systems
3. Survey on current hull cleaning technologies and ones currently in Development

Besides preventing direct surveying, marine fouling causes a decrease of ship’s speed and consequently increases the fuel consumption. For recuperation of ship's performance, it is necessary to dry-dock a ship and to clean off the marine growth on ships hull. This cleaning is always required before any other repairing/maintenance activities can follow on. Nowadays cleaning is done manually in dry-dock with an employment of different adapted methods like grit blasting or water jet. It has to be noticed that, in itself, it is a very contaminant operation (the resulting dust always contains painting particles), it is harmful for human operators health and it is a very uncomfortable job.

The use of the new technical solution allows cleaning a ship from sea adherence without the use of an expensive dry dock. In this case it would be possible to clean a ship much more often, for example, twice a year. Even in the case when it is necessary to make inspection, repair or restoration of a protective coat of a ship in a dry dock, it would be possible to make a ship's hull cleaning out of a dry dock first, which would allow using dry dock more efficiently [1].

A vessel's fuel performance usually begins decreasing after six months from dry-dock and continues to decrease rapidly. Underwater marine growth, barnacles, and/or sea grass can cost a ship-owner millions of extra dollars in time and fuel costs each year. To prevent spending additional dollars for fuel, a ship should be cleaned twice a year. A new VLCC tanker uses approximately 96 tons of bunker fuel per day and 610 barrels of fuel per 24-hour period. The cost per day for fuel alone would be approximately $30,000 with an additional $20,000 for operating expenses per day. On an average 15,000-mile cruise, the VLCC tanker would...
make the cruise in about 25 days with a clean hull. If the ship’s hull is fouled with marine growth not exceeding 0.5 inch, the same trip would take 28 days. The difference is the loss in speed of over 2 knots, which equates to 3 days. Those additional three days cost $90,000 in fuel consumption alone. The propeller is particularly vulnerable to marine fouling since it is an unpainted surface that must remain clean and shiny for proper operation. The U.S. Navy determined that propeller fouling, despite its small surface area, can generate energy losses amounting to half that of the hull so maintaining a clean propeller is critical. On military ships, the unpainted surfaces such as propellers, rudders, and sonar domes are cleaned twice as often as the hull surfaces. The propellers are also polished routinely to reduce friction and ensure that the propeller operates at optimum efficiency. Even with routine maintenance, surface roughness can occur as a result of erosion, corrosion etc. This roughness alone can increase fuel consumption up to 10 percent.

2. Hull Cleaning Intervals

The optimum interval between the periodic cleanings and inspections that comprise a preventive maintenance program will vary with the type of vessel, the location of the vessel, and its service profile (speed of operation, idle time, etc.). The type and condition of bottom coatings will also have an effect on the cleaning interval. Large vessels typically have several layers of coatings, up to 6 millimeters thick, and generally operate 4 to 6 months between hull cleanings. The location of the vessel also has a substantial influence on the rate of fouling since marine organisms flourish in warm tropical waters. The U.S. Navy has established geographic fouling zones, indicating the frequency with which the hull and unpainted surfaces (propellers, rudders, and sonar domes) should be cleaned for vessels operating within each geographic zone. In Navy Zones 1 and 3, propeller cleaning is recommended up to six times a year and hull cleaning is recommended up to three times a year.

3. Conclusions

In this paper, observe of main solutions of cleaning a ship is offered, allowing making decision about necessity of underwater ship hull cleaning by robotic devices. Such systems will prolong the service and enable excluding the use of dry dock for performance of this operation that will reduce many times the cost of ship maintenance. This research work examines current motivations and capabilities of hull cleaning robots.

4. References


THE EFFECTIVENESS OF THE SLIDING ISOLATORS IN CONTROLLING THE DYNAMIC RESPONSES OF THE JACKET TYPE PLATFORMS

Elham Mina1, Mohammad Taghi Ahmadi2 and Mehdi Shafiee Far3

1) Hydraulic Structures, Tarbiat Modarres University, Tehran, Iran, E.mina@modares.ac.ir
2) Hydraulic Structures, Tarbiat Modarres University, Tehran, Iran, mahmadi@modares.ac.ir
3) Marine Structures, Tarbiat Modarres University, Tehran, Iran, shafiee@modares.ac.ir

1. Introduction
Jack platforms have unique features and it is possible to establish a physical link between an oil platform and the seabed in certain areas such as in shallow waters. The greatest advantage of these platforms is their stability, and given the connection between these platforms and the seabed, the exertion of the wind and water forces results in the slight displacement of these platforms. The piers and offshore platforms are affected by complex destructive forces of wind, ocean currents, earthquake, and eddies. These forces result in the considerable displacements in the piers or platforms, which harm the safety and serviceability of the offshore structures in operation. In this research, a hydrodynamics software solution was used to simulate the dynamic interaction between the pier and hydrodynamic forces and to analyze the offshore and structural parameters. The present research objectives were to analyze the sea hydrodynamic parameters and the jacket platform rocking oscillations under the effect of irregular JONSWAP wave torque and to perform a numerical analysis of the seismic isolator performance in the process of controlling the dynamic responses of the offshore jacket platforms and the damped vibrations of the jacket-type platforms. The results were indicative of the effect of the sliding isolator on the offshore jacket platforms and its contribution to the decrease in the effect of vibration caused by waves and other dynamic forces on the superstructure. The sliding isolators considerably increase the energy dissipation capacity without considerably increasing the system hardness and the seismic rehabilitation of the structures is carried out through reducing the lateral earthquake and wind loads.

2. Methodology
In this research, a hydrodynamic software solution and the boundary element method were used to discretize and simulate the equations governing the fluid flow including the continuity and momentum equations. The sea hydrodynamic parameters were also analyzed in the frequency and time domains. In this regard, the effects of the seabed on the structure rocking oscillations and the hydrodynamic characteristics were studied around the given structure using the diffraction and Morrison theories. Moreover, the efficiency of the seismic isolators placed in the supports in controlling and reducing the vibrations of offshore jacket platforms was analyzed and discussed under dynamic loading. The Ansys Transient Structural and Ansys AQWA modules were also used to analyze the sea and structure parameters. In order to analyze the structural response to the irregular waves, the performance of the different wave parameters under hydrodynamic forces was studied along with the response of a four-legged jacket platform using the JONSWAP spectrum parameters. In this research, an offshore jacket platform located in the Persian Gulf was rehabilitated. This platform was situated in the South Pars field and sliding-rubber isolators were placed in the support structure. Sliding isolators (TLD) have been made from 12 to 41 inches in diameter. Slider manufacturing sliders are fabricated with a Teflon disc that mates with a stainless steel sliding surface. The platform is not attached to the soil [1] and wave-induced force is obtained using wave theory [2] to [5] for a regular wave. The environmental specifications and conditions of the region are as follows:

\[ \rho = 1025 \frac{kg}{m^3}, \ g = 9.81 \frac{m}{s^2}, \ H_0 = 6m, \]
\[ d = 150m, \ T = 8s, \lambda = 100m \]
\[ k = \frac{2\pi}{\lambda} = 0.062 m^{-1}, \omega = \frac{2\pi}{T} = 0.7853 \left( \frac{rad}{s} \right) \]

3. Analysis
3.1. Analysis of the Hydrodynamics
Structural Response and Wave
The waves were irregular random waves and it was substantially important to perform their time history analyses in assessing the offshore structures. In addition, the time histories of waves have two
major characteristics namely the significant wave height and the frequency content. According to the analysis result of the hydrodynamic force of the Jacket type platforms and the effectiveness of Sliding isolators in controlling the dynamic responses of the Jacket type platforms.

Figure 1. A view of the pressure and diffraction forces in the frequency domain

Figure 2. Maximum displacement of total hydrodynamic force of the Jacket type in direction x

Figure 3. Maximum displacement of total hydrodynamic force of the Jacket type in direction y

Figure 4. The Sliding isolators effected on displacement of pseudo-spectral

4. Conclusion

Based on the analysis results, the effect of the seismic isolators on the dynamic response of a platform was studied under stimulations caused by the hydrodynamic forces. The effect of the seismic isolator on the dynamic responses suggested that the seismic isolators significantly reduced the dynamic responses and improved the vibration performance of the jacket-type platforms. The decrease corresponding to the maximum displacement of the platform top deck, the maximum base shear of the platform, and the maximum acceleration of the top deck was 38%, and 42%, respectively. Also, the results showed that with the increasing of the depth, the impact of wave’s force and moment on the base of platform are reduced through exponential relationship. The amount of force and total moment are inclined to a fixed value. The reductions are due to the effective depth that is equal to half the wavelength; so that it reduces the amount of force and moment to a small amount.

5. References

AN INVESTIGATION ON RELIABILITY OF FINITE ELEMENT MODELS IN PREDICTING CAPACITY OF TUBULAR JOINTS

Behrouz Asgarian1 and Vahid Mokarram2

1) Professor, K.N. Toosi University of Technology, Tehran, Iran, asgarian@kntu.ac.ir
2) PhD candidate at Department of Civil and Environmental Engineering, Shiraz University, vahid_mokarram@shirazu.ac.ir

1. Introduction

Circular tubular sections are widely used in design and construction of fixed offshore platforms. Code-based load-carrying capacity of tubular joints are dominantly proposed according to experimental tests. However, due to high costs and difficulties attributed to the development of experimental tests, experimental database for predicting capacity of tubular joints are limited.

The engineering community needs to be supported with more insight into how reliable FE models can be developed for predicting the load-carrying capacity of tubular joints. FE models can enhance the database for future code-based capacity formulations. Moreover, FE models can be regarded as a promising tool for evaluating existing fixed offshore platforms. Because, effects of local joint flexibility and brace interaction are automatically incorporated in such models. In this paper, a nonlinear FE model is carefully constructed using shell elements in ANSYS [1] for investigating the pros and cons of such models. In this paper, a nonlinear FE model is carefully constructed using shell elements in ANSYS [1] for investigating the pros and cons of such models in evaluating the capacity of unreinforced tubular T-joints subjected to the actions of tensile, compressive and in-plane bending loads of the brace. Finally, a brief discussion on the effects of boundary conditions of the chord on capacity of the joints is presented. The results from FE models shall be compared with recommendations of latest edition of API standard [2].

2. Load-carrying Capacity of Y/T Joints

2.1. Characteristics of the FE Models

Geometrical characteristics of the considered joints considered in this paper are given in Table 1. All joints are modelled with ANSYS [1] Shell93 element, which is an 8-node structural shell element that can incorporate both material and geometrical nonlinearities. The element is particularly suitable for modelling curved shells. Rigid plates are attached to both ends of the chord. Boundary conditions (clamped or pinned ends) are satisfied in the nodes located at the center of these plates.

2.2. Joints Capacity under Axial Tensile Loading

Except for joints with small values of $\beta = D_b / D$ (where $D$ and $D_b$ represent chord diameter and brace diameter, respectively) if crack propagation mechanisms are not incorporated in FE models, the tensile load can be increased to reach a failure due to global plasticity. However, failure mode of such joints is attributed to crack propagations, which eventually result in rupture and separation of the brace from the chord.

2.3. Joints Capacity under Axial Compression Loading

Since crack propagation is not the failure mode of the joints under axial compression loads, FE models yield reliable estimations for load-carrying capacity of such joints. In fact, FE models show that local buckling and plastic deformations of the chord’s wall is the main failure mode of Y/T joints under compression.

2.4. Joints Capacity under In-Plane Bending Moment

Figure 1 and Table 2 shows excellent agreements between load-carrying capacities obtained from the FE models and those recommended by API [2]. Joint capacities obtained from API [2] provisions are between 93% (in joint $T_8$) and 99% (in joint $T_5$) of those obtained from the FE models. Yura’s [3] limit is an experimental deformation-based indicator which is suitable for finding the capacity of joints that have monotonic load-deformation curves. Since the curves corresponding to joints $T_5$, $T_9$ and $T_9$ are monotonic, the non-dimensional $M_{max}/F_{y}t_{c}D_{b}$ ratios that are reported in Table 2, have been obtained based on the $M_{max}$ corresponding to Yura’s [3] limit (see Figure 1).

By investigating Table 2, it can be confirmed that joints with larger values of $\gamma$ and smaller values of $\beta$ had the least local plasticity. In such cases, FE results match the best with capacities obtained from API [2]. Failure of Y/T joints subjected to in-plane bending moments occurs due to (1) failure of the chord’s wall under tensile stresses that are exerted on the wall as a result of the bending moment of the brace and, (2) local buckling and plasticity of the chord’s wall.

3. Effects of Boundary Conditions on Joints Capacity

In this section, the capacity of joint $T_5$ for two cases (1) chord with pinned ends, and (2) chord with clamped ends are obtained. The results show that whenever failure modes are not global capacity of the joint, FE results and API’s [2] predictions show excellent agreements. However, if the global plasticity of the system is the dominant mode of failure, the FE model yields capacity overestimations in comparison with API [2].
Table 1. Geometric characteristics of the considered T-joints.

<table>
<thead>
<tr>
<th>Joint ID</th>
<th>γ = Dc/Db</th>
<th>β = Dc/Lc</th>
<th>α = 2Lc/Dc</th>
<th>τ = tb/tc</th>
<th>tc (cm)</th>
<th>Lb (cm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>T1</td>
<td>12</td>
<td>0.25</td>
<td>12</td>
<td>0.8</td>
<td>3.175</td>
<td>200</td>
</tr>
<tr>
<td>T2</td>
<td>12</td>
<td>0.5</td>
<td>12</td>
<td>0.8</td>
<td>3.175</td>
<td>200</td>
</tr>
<tr>
<td>T3</td>
<td>12</td>
<td>0.75</td>
<td>12</td>
<td>0.8</td>
<td>3.175</td>
<td>200</td>
</tr>
<tr>
<td>T4</td>
<td>15</td>
<td>0.25</td>
<td>12</td>
<td>0.8</td>
<td>3.175</td>
<td>200</td>
</tr>
<tr>
<td>T5</td>
<td>15</td>
<td>0.5</td>
<td>12</td>
<td>0.8</td>
<td>3.175</td>
<td>200</td>
</tr>
<tr>
<td>T6</td>
<td>15</td>
<td>0.75</td>
<td>12</td>
<td>0.8</td>
<td>3.175</td>
<td>200</td>
</tr>
<tr>
<td>T7</td>
<td>18</td>
<td>0.25</td>
<td>12</td>
<td>0.8</td>
<td>3.175</td>
<td>200</td>
</tr>
<tr>
<td>T8</td>
<td>18</td>
<td>0.5</td>
<td>12</td>
<td>0.8</td>
<td>3.175</td>
<td>200</td>
</tr>
<tr>
<td>T9</td>
<td>18</td>
<td>0.75</td>
<td>12</td>
<td>0.8</td>
<td>3.175</td>
<td>200</td>
</tr>
</tbody>
</table>

Note: Dc, Db, Lc, tc, and tb represent chord diameter, brace diameter, chord length, chord thickness and brace thickness, respectively.

Table 2. Capacity of the considered joints under in-plane bending moment.

<table>
<thead>
<tr>
<th>Joint ID</th>
<th>API</th>
<th>Nonlinear FE model</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Mmax</td>
<td>Fmax</td>
</tr>
<tr>
<td>T2</td>
<td>6.14</td>
<td>3.09</td>
</tr>
<tr>
<td>T3</td>
<td>3.98</td>
<td>5.33</td>
</tr>
<tr>
<td>T4</td>
<td>7.05</td>
<td>3.29</td>
</tr>
<tr>
<td>T6</td>
<td>11.47</td>
<td>6.12</td>
</tr>
<tr>
<td>T8</td>
<td>7.97</td>
<td>4.07</td>
</tr>
<tr>
<td>T9</td>
<td>12.96</td>
<td>6.71</td>
</tr>
</tbody>
</table>

Note: Mmax, Mf, and Fc represents the maximum in-plane moment in the intersection region of the joint, plastic moment of the chord section and yield stress of the chord, respectively.

4. Conclusions

Reliability of FE models in predicting load-carrying capacity of tubular T-joints was investigated through considering tubular T-joints with different non-dimensional geometric parameters. Capacity of the considered T-joints were evaluated from FE models and compared against API’s predictions for three different loading conditions. For joints under tensile loads, crack propagation and separation of the brace from the chord is the main failure mode. Hence, findings of this paper suggest that FE models are not unless crack propagation mechanisms are properly addressed. It was also shown that parameter γ does not affect the capacity of T-joints under tensile actions. Moreover, the results of this study revealed that FE models are reliable in predicting load-carrying capacity of joints under compressive axial loads and in-plane bending moments. It could be concluded that local buckling and plasticity of joints under compressive actions of the brace could be accurately captured in nonlinear FE models with shell elements. It was also shown that global plasticity is not the failure mode of T-joints under in-plane bending moments. It indicates that, in such cases, FE models can predict the load-carrying capacity with high accuracy. The results presented in this paper show that FE models can effectively estimate load-carrying capacity of tubular joints if local buckling and local plasticity of the chord are the dominant failure mode. This conclusion was also confirmed by investigating the effects of boundary conditions in joint responses.

5. References