PREFACE

The 2019 6th International Conference on Coastal and Ocean Engineering (ICCOE 2019) was held in Bangkok, Thailand, April 25-28, 2019. ICCOE 2019 is dedicated to issues related to Coastal and Ocean Engineering.

The objective of the conference is to bring together interested academics and industry experts in the field of Coastal and Ocean Engineering to a common forum. Three keynote speeches were delivered. Prof. Chou Loke Ming from Tropical Marine Science Institute, National University of Singapore, presented a keynote speech “Coral Reef Restoration and Habitat Creation to Mitigate Urbanization and Climate Change Impacts”. Assoc. Prof. Baoping Cai from China University of Petroleum gave the keynote speech “Development of Subsea Systems for Offshore Oil and Gas Equipment, with Fault Diagnosis Issues”. Professor Koh Hock Lye from Sunway University, Malaysia gave the third keynote speech “Sustainability in Theory and in Practice: A Three-decade Reflection on Education for Sustainability”. In addition, an Invited Speech “Convection-permitting Projections of Future Changes in Hydroclimatic Characteristics” was given by Dr. Shuo Wang from Hong Kong Polytechnic University.

Papers were presented in four sessions of the conference: 1) Marine Ecology and Environment, 2) Ocean Engineering and Physics, 3) Wastewater Treatment, and 4) Health and Renewable Energy Application. 29 presentations were given by participants from 10 countries.

This proceedings present a selection from papers submitted to the conference. All papers were subjected to peer-review by conference committee members and international reviewers. The papers selected depended on their quality and their relevance to the conference. This volume presents the recent advances in Coastal and Ocean Engineering and related areas, such as beach erosion and sediment transport, marine ecology and environment, climate change and sea level rise, lowland development and reclamation, subsea pipeline, coastal infrastructure development, deep sea engineering, and ocean activities platform.

We would like to thank all the authors who have contributed to this volume, and also the organizing committee, reviewers, speakers, chairpersons, sponsors and all the conference participants for their support to ICCOE 2019.

Prof. Chou Loke Ming
Tropical Marine Science Institute, National University of Singapore, Singapore
May 5, 2019
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Peer review statement

All papers published in this volume of *IOP Conference Series: Earth and Environmental Science* have been peer reviewed through processes administered by the proceedings Editors. Reviews were conducted by expert referees to the professional and scientific standards expected of a proceedings journal published by IOP Publishing.
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Chapter 1:
Ships and Marine Engineering
Steady State Analysis and Optimization for Autonomous Underwater Vehicle

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Abstract. Autonomous Underwater Vehicle (AUV) is widely used in ocean engineering field. Steady state analysis of AUV is considered as a necessary part in the design process. Motivated by improving the stability, a downwash filter based feedback optimization methodology for AUV is presented. First, combined with AUV structural parameters, the AUV four-degree of freedom motion space mathematical model is calculated to provide a mathematical basis for rock-bottom steady state control system. Second, the transfer function of the filter is derived according to the zero-pole and trajectory map of the AUV ontic system. Finally, the AUV optimized control system is established by series downwash filter. The simulation analysis result shows that the stability of the system is greatly increased after the whole system is connected to the filter.

1. Introduction

In contemporary society, land resources are becoming increasingly in shortage. Countries around the world are turning their attention to the sea. The development and utilization of the ocean are the future trend. Therefore, AUVs have been developed rapidly [1]. From the 1970s and 1980s, subsea surveys have emerged. AUVs, which is safe and efficient, can be widely used in all aspects of marine-related work, such as marine environmental survey and marine geological exploration. The United States, Germany, Russia, the United Kingdom, France, Italy, Japan, China and other countries have been developing a variety of AUVs [2]. AUVs are considered as the swords for countries to develop and use the ocean. With the rapid development and advancement of correlative technology, various AUVs will be applied in marine-related research work.

To copy with extremely complex subsea environment, the anti-environmental interference ability, rapid response ability and strong robustness ability should be involved in the consideration of design [3]. Therefore, the steady state design and analysis of AUV are considered to be significant. In this paper, AUV stability design is carried out by two aspects [4]: AUV's ontological structure steady state analysis and internal control system optimization. The ontological structure is the natural property of AUV, and its stability is directly reflected by the AUV's anti-rollover ability and self-return capability [5]. Stability analysis of the AUV mechanical structure is used as the standard for the design of the ontological structure. In order to further improve the stability and robust control of AUV, the downwash filter is designed, and the control model is analysed by the impulse response function to verify the AUV control effect [6-7].

The reminder of this paper is organized as follows. In Section 2, the kinematics model of AUV is established. In Section 3, the hydrodynamic model of AUV is established. Section 4 shows steady state analysis and optimization for AUV. Section 5 summarizes the paper.
2. Kinematic model of AUV

The object of analysis is the self-developed detection type AUV, which is a non-open frame structure, as shown in Figure.1, and the following assumptions are usually made in studying its equation of motion:

(1) AUV is seen as a rigid body;
(2) Considering the acceleration component of the center of gravity after ignoring the influence of the Earth's rotation;
(3) The force acting on the AUV ontological structure is only considered as the effects of gravity, hydrostatic force, and hydrodynamic force;
(4) Fluid dynamic coefficient or parameter is constant.

Figure 1. The self-developed AUV

The ground coordinate system \((E - \xi\eta\zeta)\) is used as the inertial coordinate system, \(E - \xi\) points to the center of the earth, \(E - \eta\) points to the geographic east, and \(E - \zeta\) points to the geographic south. As shown in Figure 2, the vehicle coordinate system \((O - xyz)\) is established, which is also known as the moving coordinate system. \(O - x\) is consistent with the AUV main symmetry axis, \(O - z\) is parallel to the front thruster pointing to the AUV bottom surface, and \(O - y\) conforms to the right hand rectangular coordinate system.

Figure 2. Coordinate system definition

The spatial position of the AUV is described as the three components of the moving coordinate system origin \(O\) in the static coordinate system, i.e. \(\{\xi O, \eta O, \zeta O\}\), and the three attitude angles of the dynamic coordinate system for the static coordinate system: horizontal inclination angle \(\phi\), pitch angle \(\theta\), and yaw angle \(\psi\). In the ground coordinate system \(\{E\}\), the speed of the AUV is recorded as \(V\), and the angular velocity is recorded as \(\Omega\). The projections of \(V\) and \(\Omega\) on the three coordinate axes of the moving coordinate system \(\{O\}\) are recorded as \(u, v, w\) and \(p, q, r\), respectively. The complex motion state of the AUV is decomposed into motions in two planes, as shown in Figure 2 (b) and (c), namely horizontal plane motion and vertical plane motion, respectively. In the low-speed navigation case, the coupling effects of motion in two planes can be ignored.

3. Hydrodynamic model of AUV
3.1. AUV external force

The calculation method of the external force \( F \) and external torque \( M \) using [8],

\[
F = F_F + B + P + \sum_{i=1}^{n} T_i
\]

\[
M = M_F + M_B + M_P + \sum_{i=1}^{n} M_t
\]

Where \( F_F \) is the hydrodynamic force of AUV, \( B \) is buoyancy of AUV, \( P \) is gravity of AUV, \( \sum_{i=1}^{n} T_i \) is thrust of AUV, and \( M_F, M_B, M_P, \sum_{i=1}^{n} M_t \) represent their corresponding torque.

3.2. AUV space motion equation

Only the four-degree of freedom space motion equation is considered. Based on the analysis in the above part, the space motion equation of AUV can be obtained using [9-10].

\[
\begin{align*}
\mathbf{m}(u-\nu r + wq) &= T_y - (P - B) + X_u X_{uu} u^2 \\
\mathbf{m}(w-\nu q + v p) &= T_z - (P - B) \cos \theta \cos \phi + Z_w w + Z_q q \\
I_x q + (I_y - I_z) r p &= M_y - z_c B \sin \theta + M_o + M_q w + M_q q \\
I_z r + (I_y - I_z) p q &= M_z + N_r v + N_v r + N_r v + N_v r
\end{align*}
\]

Where \( \mathbf{m} \) is the quality of the AUV. The \( X_u, X_{uu}, Z_w, Z_q, M_w, M_q, \) represent hydrodynamic coefficient. which are set as empirical values. The \( I_x, I_z, I_y \) represent the corresponding axis’s rotational inertia of AUV rotation.

4. Steady state analysis and optimization for AUV

4.1. AUV motion state space model

The AUV spatial equation of state can be generated by using Eq. (1)-(6) [11].

\[
\begin{bmatrix}
\dot{x}_1(t) \\
\dot{x}_2(t) \\
\dot{x}_3(t) \\
\dot{x}_4(t)
\end{bmatrix} =
\begin{bmatrix}
a_{11} & a_{12} & a_{13} & a_{14} \\
a_{21} & a_{22} & a_{23} & a_{24} \\
a_{31} & a_{32} & a_{33} & a_{34} \\
a_{41} & a_{42} & a_{43} & a_{44}
\end{bmatrix}
\begin{bmatrix}
x_1(t) \\
x_2(t) \\
x_3(t) \\
x_4(t)
\end{bmatrix} +
\begin{bmatrix}
b_1 \\
b_2 \\
b_3 \\
b_4
\end{bmatrix}
\begin{bmatrix}
u_1(t) \\
u_2(t)
\end{bmatrix}
\]

\[
\begin{bmatrix}
y_1(t) \\
y_2(t)
\end{bmatrix} =
\begin{bmatrix}
c_{11} & c_{21} & c_{13} & c_{14} \\
c_{21} & c_{22} & c_{23} & c_{24}
\end{bmatrix}
\begin{bmatrix}
x_1(t) \\
x_2(t) \\
x_3(t) \\
x_4(t)
\end{bmatrix}
\]

Where \( x_1(t) \) is swing angle, (rad); \( x_2(t) \) is swing angular velocity, (rad/s); \( x_3(t) \) is pitch angular velocity, (rad/s); \( x_4(t) \) is pitch angle, (rad); \( u_i(t) \) is thrust difference of rear thruster,
(N); \( u_2(t) \) is thrust difference of front thruster, \( u_1(t) \) is output swing angle (rad); \( y_2(t) \) is output swing angular velocity (rad/s).

Assume that the AUV underwater navigation speed is 2 knots/s, and the AUV ontic design parameters set \{A\} and \{B\} are calculated by using self-developed AUV parameters.

\[
A = \begin{bmatrix}
    a_{11} & a_{12} & a_{13} & a_{14} \\
    a_{21} & a_{22} & a_{23} & a_{24} \\
    a_{31} & a_{32} & a_{33} & a_{34} \\
    a_{41} & a_{42} & a_{43} & a_{44}
\end{bmatrix} = \begin{bmatrix}
-0.0058 & -0.0968 & 0.0082 & 0.0015 \\
0.0080 & -0.0150 & -0.0018 & 0.0000 \\
-0.0250 & 0.0880 & -0.0050 & 0.0000 \\
0.0000 & 0.0805 & 1.0000 & 0.0000
\end{bmatrix}
\]

\[
B = \begin{bmatrix}
    b_{11} & b_{12} \\
    b_{12} & b_{22} \\
    b_{13} & b_{33} \\
    b_{14} & b_{42}
\end{bmatrix} = \begin{bmatrix}
0.00002 & 0.00000 \\
-0.00500 & 0.00075 \\
0.05200 & 0.01300 \\
0.00000 & 0.00000
\end{bmatrix}
\]

4.2. **AUV ontic stability analysis**

4.2.1. **Analysis of system Pole-Zero**

The zero-pole map of the AUV ontological structure can be calculated by using the Eq. (7)-(8) [12], as shown in Figure.3.

![Zero-pole map of AUV ontic](image)

**Figure 3.** Zero-pole map of AUV ontic

The map shows that there is a pair of conjugate poles in the AUV state space model, indicating the system is progressively stable and comply with design requirements. However, since the pole is close to the imaginary axis, the system damping is relatively small, and the robust performance is low. Once the damping of the system is improved, the AUV’s ability will be improved to cope with complex sea conditions.

4.2.2. **Unit impulse response analysis of AUV system**

The stable performance of the AUV system under interference conditions is analyzed by loading the unit impulse. The unit impulse response of the system is shown in Figure 4.
The adaptive ability of the previous time after the interference is considered extremely important in the actual voyage of the AUV. Figure 4 (a) illustrates the system has certain robustness. The whole system tends to stabilize after being disturbed by the unit impulse for about 150 seconds, indicating that the pitch angle and the swing angle keep oscillating at first, and then the oscillation decreases and stabilizes gradually. It can also be seen that AUV has a weak anti-interference ability in a short time, considering the first 20 seconds of the system unit impulse response, as shown in Figure 4 (b). In order to improve the stability of the AUV system, the system is to be corrected by designing a feedback closed-loop.

4.3. Downwash filter optimization design
It is necessary to ensure that the root trajectory cannot move further to the left half plane in the Zero-pole map, and the downwash filter is designed to improve the stability of the AUV system [13]. The transfer function by using,

$$G(s) = \frac{s}{s + \alpha}$$

The zero point of the system is fixed at the origin, therefore, the root trace of the downwash filter is limited to the near origin. Set the $\alpha = 0.2$, and the system is desired to start to converge when the impulse response is 5 seconds so that the time constant $s$ is 5. The downwash filter-based closed-loop model is connected in series to the AUV system to form a new system, which zero-pole root trajectory map is shown in Figure. 5.

**Figure 4.** Unit impulse response of AUV

**Figure 5.** Closed-loop model zero-pole root trajectory map
The trajectory map indicates that the series downwash filter system is also stable. According to the general requirements of the stability system, when the system damping ratio $\xi = 0.3$, the system gain is 1.74, as shown in Figure 6.

![Root Locus](image)

**Figure 6.** $G(s) = 1.74$

The closed loop system with a series of downwash filter is designed to obtain a steady-state curve of the thrust difference input and the swing angle output, as shown in Figure 7.

![Response curve](image)

(a) Swing angle stability response curve    (b) Parameter stability response curve

**Figure 7.** Response curve

It can be seen from Figure 7 (a), the system starts to converge in about 5 seconds, then tends to be stable. Besides, the system oscillation is reduced, the stability is improved, and the robustness is improved. The stability curves of other parameters are shown in Figure 7 (b). It proves that the feedback system of the series downwash filter has strong robustness and stability, which can adapt to the complex underwater environment and ensures the safe operation of the robot system.

5. **Conclusions**

In this paper, the AUV kinematics and hydrodynamics are analyzed. The AUV spatial motion is decomposed into horizontal and vertical plane motions, and the AUV kinematics equation is derived from two planes. Combined with AUV structural parameters as well as gravity buoyancy and characteristics, the AUV external force analysis is carried out, and the AUV four-degree of freedom
motion space mathematical model is further proposed providing a mathematical basis for the AUV underlying steady state control. The AUV ontology space motion equation is derived, and the state space model is established. Then the unit impulse interference signal is applied to the ontology structure model to verify that the AUV mechanical structure has certain stability control ability. After receiving external interference for about 150 seconds, the whole system is stabilised, indicating that the structural design meets the stability criteria.

In order to further improve the stability of the AUV, the internal control system is optimized. The zero-pole and trajectory map of the AUV system is established, which is used as a guide to backward determine the key optimization parameters of the system stability. The AUV downwash filter is designed, and the stability control is realized through the intervention of the rock-bottom control program. The simulation analysis shows that the stability of the system is significantly improved after the whole system was connected to the filter.

6. References


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Calculation on the Uprighting Process of a damaged-capsized ship

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Abstract. Righting a damaged-capsized ship can be very complicated. In this paper, flooding quantity and three-dimensional mathematical model of righting force are introduced in order to solve the buoyancy and stability of a capsized ship. Through the simulation, the upraising process of a damaged-capsized ship with air cushion was researched. Computation result shows that the proportion between the maximum righting moment in the opposite direction and the maximum righting moment is 2.891. Trim decreased gradually during the upraising process, so it reached the minimum value when the ship returned to equilibrium location. The quantity of flooding water increased slowly in the later process when the opening was beneath the water level. For each calculation, the maximum shear force was located at the same position, which does not increase with flooding quantity.

1. Introduction
Capsized ship should be righted as earlier as possible, because vessel accidents have an impact on politics, military affairs, economy, ecology and culture[1-5]. Generally, capsized ships should be righted before refloating[6]. Buoyancy and stability must be calculated before righted. In the process, the gravity center changes during adjusting load or flooding, then the buoyancy and stability is affected by the addition weight[7,8]. cabin method was proposed to lower the influence of sudden ingress of water by Vermee et al, the flooding quantity mathematical model and computer program were tested by experiments[9]. The relationship between flooding quantity and pressure of the vessel cabin and the effection of instant flooding water were identified through calculation and experiments[10]. Damaged ship may capsise under wind, wave and current circumstances because a dramatic decrease of ship dynamic stability[11]. Through the research on the causes of SS Heraklion accident, Papanikolaou found that too small freeboard result in the capsized accident[12]. Maimun studied the of roll motion damaged passenger in random transverse waves by experimental and numerical methods, the results show that wave height and loadage are the main influence factors. Must wave height a maximum of 0.2 m when the height of gravity center is 1.3 m[13]. In order to design the scheme of floating crane ship righting a capsized ship better, Hanming Liu used Analytical Hierarchy Process(AHP) to analyze the contents, principle, and conditions of project, then a total frame of the scheme was put forward as a theoretical basis[14]. According to the principle of righting a capsized ship, xiangke Huang established a mathematical model of righting fulcrums internal angle, righting force, righting force functional point and righting force point internal angle[15]. Zhao discussed the possibility of righting a capsized ship with pontoon or floating crane according to different states of the ship[16,17]. Uprising scheme had been set up on the basis of primary data, grounding, environmental factors; then the calculate method of righting force was obtained[18].
pocket is a factor which can affect reserve buoyancy and stability during righting a capsized ship. Drobyshevski solved righting force of a capsized ship with or without air pocket by theoretical method, the righting force of ship with the angle of heel at 90° was calculated[19]. To simplify the calculation, some engineers only consider the changes of heeling angle and draft. The heeling angle was found directly related to the stability and height above water surface in compartment by Xiaolu Wu, and the uprighting scheme of vertical righting force was made. Matlab software was further used to solve righting force of floating crane[20]. Take a capsized ship with the angle of heel at 90° as an example, Yaohui Tong discussed righting and refloating a sunk ship with buoy used in both side. The method is suitable for capsized ship with angle of heel less than 90°[21]. Zhiwei Dong and Peikun Cong introduced the salvage engineering of righting Liaolvdu 7 with floating crane and buoy used in both side[22]. Costa Concordia ran aground and overturned after striking an underwater rock. Divers used special anchor chain to keep it in place. 2100 tonnes of oil was drained and chimney was removed for salvage engineering. 12 Hydraulic Jacks righted the inclined ship, and 15 enormous caissons played a supplementary role in the process[23,24].

At present many Chinese salvage companies are small in scale, the economic and technical strength is relatively weak. They are lack of independent key technologies. So traditional technology methods are applied to make salvage scheme, such as empirical algorithm or semi-empirical algorithms, but the accurate datas are not obtained.

For quickly and accurately calculating the distribution of cussion, flooding quantity, stress condition of a damaged and capsized ship, the paper established the flooding quantity model with cussion and righting force model. Uprighting process was simulated by GHS software, the causes and mechanism of all influencing factors were solved.

2. Analysis and calculation of damaged compartment

There are three typical classifications of damaged compartments during uprighting process according to floatation, stability, free surface, cussion[25]. Case 1: During the process, damaged compartments underwater cannot be repaired. Case 2: During the process, the damaged compartments can be repaired, but they cannot be fully drained. Free water in the slack tank can not be drained. Case 3: During the process, the damaged compartments are not repaired. The breach is not sealed, and water continues flowing inside or out the compartments. As a result, the total amount of water changes during the course, the insubmersibility of the ship must be considered. Case 3 is relatively complex. This paper researched and analyzed the uprighting process of a damaged and capsized ship.

In order to generally analyze the uprighting process of a capsized ship, mechanics model was build up in this paper[26].

\[
\begin{align*}
\Delta + F - W &= 0 \\
M_x &= M_{w_2} + M_{\Delta x} + M_{F_2} \\
&= (x_G W + z_D A + z_F F) \sin(\theta) + (x_G W - x_D A - x_F F) \cos(\phi) \cos(\theta) \\
M_z &= M_{w_2} + M_{\Delta z} + M_{F_2} \\
&= (x_G W - x_D A - x_F F) \sin(\phi) \cos(\theta) - (y_G W + y_D A + y_F F) \sin(\theta)
\end{align*}
\]

Where, \( \phi \) is heeling angle; \( \theta \) is trim angle; \( x_G, y_G, z_G \) are the coordinates of the gravity center along the coordinate axis; \( M_{w_2}, M_{F_2}, M_{w_2} \) are the moment of the gravity along the coordinate axis; \( x_D, y_D, z_D \) are the center of buoyancy along the coordinate axis; \( M_{\Delta x}, M_{\Delta y}, M_{\Delta z} \) are the moment of the buoyancy along the coordinate axis; \( x_F, y_F, z_F \) are the moment of righting force along the coordinate axis; \( M_{F_2}, M_{F_2}, M_{F_2} \) are the righting force moment along the coordinate axis.

During uprighting process, if a cussion in damaged ship, flooding quantity at any time can be solved based on[27].
\[
q_i = \begin{cases} 
\mu A \left\{ \frac{2}{\rho (1-k^2)} \left[ \rho g \frac{V(t)}{S} + P_1 - P_2 \right] \right\}^{\frac{1}{2}} \\
-\mu A \left\{ \frac{2}{\rho (1-k^2)} \left[ \rho g \frac{V(t)}{S} + P_1 - P_2 \right] \right\}^{\frac{1}{2}} 
\end{cases} 
\]

\[
h_{i-1} < h_i \quad \text{or} \quad h_{i-1} > h_i \quad i = 1, 2, 3, ..., n \tag{2}
\]

Here, \( \mu \) is the flow velocity; \( A \) is the area of the opening; \( S \) is the area of free surface; \( g \) is the acceleration due to gravity; \( V(t) \) is the volume of flooding water; \( P_1 \) is air pressure of the damaged compartment; \( P_2 \) is the pressure of the opening; \( \rho \) is the density of the water; \( k \) is the function of \( \mu \), \( S \) and \( A \).

### 3. Analysis of examples

The above is the theoretical analysis model of a damaged and capsized ship. Take case 3 for example, this paper simulated the uprighting process of a damaged and capsized ship. Figure 1 is the hull model diagram. Table 1 is the principal dimensions of the intact ship.

![Figure 1. Hull and compartments](image)

<table>
<thead>
<tr>
<th>Overall length /m</th>
<th>Moulded breadth /m</th>
<th>Moulded depth /m</th>
<th>Draft /m</th>
</tr>
</thead>
<tbody>
<tr>
<td>105</td>
<td>16</td>
<td>10</td>
<td>7.391</td>
</tr>
</tbody>
</table>

The ship capsized on water due to contingency. Compartment on the bow is damaged through underwater inspection. The uprighting process was simulated in this paper. The ship listed 170.48° to starboard, had a −1.03° trim, and an origin draft of −4.012 m. There is a air pocket in the damaged compartment during uprighting process.

### 4. Results and discussions

#### 4.1. Righting moment

Stability determines the difficulty of righting a capsized ship. Negative stability values represent the ship in upright condition. Negative stability is helpful for the uprighting process without righting force sometimes. Based on calculation, the proportion between the maximum righting moment in the opposite direction and the maximum righting moment is 2.891. In later phases of the process, righting force moment in the opposite direction is also needed to avoid the ship being damaged again or capsizing again.
4.2. **Trim angle**

Trim angle changes during uprighting process. Sometimes, free water in damaged compartment greatly intensifies the problem. In figure 3, trim angle decreases gradually during uprighting process. The variation of trim angle is 1.09, which is relatively big for salvage engineering, so the hull strength should be calculated accurately.

![Figure 3. Trim variation during uprighting](image)

4.3. **Flooding quantity**

Flooding quantity is varies with ship body state and uprighting process. There is an air cussion in the damaged compartment, so flooding quantity is small, which is has little effect on draft.

![Figure 4. Variation of total flooding quantity of damaged compartments during uprighting](image)

4.4. **Displacement**

Reserve buoyancy ensures the insubmersibility. Floating condition changes when water is flow into or out of the damaged compartments during uprighting process. In figure 5, total displacement represents the displacement of the enclosed space of the ship, and ship’s displacement represents the displacement of the ship during uprighting process. The ship was not sink according to total displacement curve and ship’s displacement curve. Water in the damaged compartments drained automatically in the beginning of the process. The quantity of flooding water increased slowly in the
later process when the opening was beneath the water level.

![Figure 5. Displacement variations during uprighting](image)

### 4.5. The maximum shear force

Hull's longitudinal strength calculation is the key to salvage scheme. Serious accidents will occur when the hull's longitudinal load exceeds the approved stress. In figure 6, the changes of the maximum shear force along the longitudinal direction is obtained. For this numerical example, the position of the maximum shear force is no change, which is 88.9 m in the back of the original point. Flooding quantity is small, but the maximum shear force is very big. The flooding quantity variation tendency is not agree with the maximum shear force.

![Figure 6. Variation of the maximum shear force during uprighting](image)

### 5. Conclusion

Refloat a damaged ship is a complex engineering. Especially, the opening can not repaired, and there is an air pocket in the damaged compartment. The flooding quantity and righting force model were introduced based on the ship's hydrostatical theory. Uprighting process is simulated by apply GHS software. Reserve buoyancy should be calculated before engineering to ensure capsized ship floating on water. The uprighting process is composed of numerous static states. It can be concluded from the position of damaged compartment, flooding quantity, displacement, that the position of damaged compartment is more likely to affect the process when the trim angle is small. The maximum shear force appears in a fixed position which may be associated with the location of damaged compartment and flooding quantity. Then, this paper will make a comparative study on simulation results of intact capsized ship and other types of damaged ships. Simultaneously, considering that trim angle varied within a small range during process. In the future research, the influence of trim angle varied within a large range on shear force should be studied.

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Influence of Large-scaled Reclamation on the Cold Wave in Coastal Area of Jiangsu Province

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Abstract. Taking the large-scaled reclamation in Jiangsu Sea as study background, on the premise of large-scaled area (East China Sea) having a good accuracy. The cold wave of Jiangsu Sea is simulated by the nested model of SWAN, and analysing the Hs (significant wave height) near the reclamation area extracted from the results. It is found that the cold wave field around the reclamation area is influenced remarkably by large-scaled reclamation. The large change of wave field is mainly concentrated in the southern part of central Jiangsu province, and it has a significant influence on the wave field. Due to the effect of covering, the wave height of the southern sea area of Dongsha, Gaoni and Lengjiasha reclamation area decreases greatly, and the decline can be as much as 30%. Due to the extent of reclamation is small, the change of wave field in the northern part of Jiangsu province is not obvious, and the range of change is not obvious.

1. Introduction
The coastal coastline of Jiangsu is about 954 kilometers, and the unique topography and landform of Jiangsu region has accumulated a large number of tidal mudflat resource. According to the comprehensive survey and evaluation of Jiangsu's coastal ocean in 2008 (part of Jiangsu 908 special project), the areas of unenclosed tidal mudflat is more than 7.5 million mu, accounting for about 1/4 of the national total areas [1-2]. Jiangsu province has put forward the plan of exploitation and utilization of reclamation on tidal flats in Jiangsu coastal area. Meanwhile, many problems brought by reclamation on tidal flats have been put on the research agenda.

The implementation of large-scale tideland reclamation project will change the topography of coastal areas and have an impact on the coastal environment, thus leading to change the hydrodynamic environment and silting environment. And on the one hand, it will bring a large number of land and economic benefits to human beings. On the other hand, it will also have a certain impact on the natural environment [3-4]. Cold wave wind is one of the disastrous extreme weather which has great influence on the coastal water in Jiangsu province. It's a mass movement of cold air. Cold wave wind is usually accompanied by the phenomenon of sharp cooling, strong wind, big wave and so on. As the cold wave
lasts for a long time, the wind wave caused by cold wave wind grows sufficiently, which seriously threatens people's life and property safety and offshore engineering facilities in coastal areas [5]. Therefore, it is of great practical significance to research the influence of large-scale reclamation on cold wave. In this paper, SWAN wave numerical model was used to establish a wind wave model in the east China sea, to simulate the wave field before and after the coastal reclamation planning in Jiangsu under cold wave and windy weather, focusing on the variation of wave height in the coastal area before and after the reclamation project implementation, and to analyze the impact of large-scale reclamation on the wave field.

2. SWAN model

SWAN model [6] describes the random wave field with two-dimensional dynamic spectral density $N(\sigma, \theta)$, which is related to two-dimensional energy spectral density $E(\sigma, \theta)$, and the relation between $N(\sigma, \theta)$ and $E(\sigma, \theta)$ is $N(\sigma, \theta) = E(\sigma, \theta)/\sigma$. Where $\sigma$ is frequency and $\theta$ is wave direction.

In the cartesian coordinate system, the balance equation of the motion spectrum is expressed as:

$$\frac{\partial}{\partial t}N + \frac{\partial}{\partial x}c_x N + \frac{\partial}{\partial y}c_y N + \frac{\partial}{\partial \sigma}c_\sigma N + \frac{\partial}{\partial \theta}c_\theta N = \frac{S_{\text{sw}}}{\sigma}$$

Where: the first item on the left represents the rate of change of $N$ over time; the second and third items represent the propagation of $N$ in the direction of $x$ and $y$; the fourth item is the change of relative frequency caused by the change of water depth and water flow; the fifth item is the refraction caused by water depth and water flow ($C_x$, $C_y$, $C_\sigma$, $C_\theta$ represent the wave propagation velocity in the direction of $x$, $y$, $\sigma$, and $\theta$ respectively); On the right side of the equation, $S_{\text{sw}}$ is the source sink term which is represented by energy spectral density. And it can reasonably simulates the complex wave field under the environment of tidal current, topography and wind field. SWAN model takes into account the wave shallower, reflection, refraction, bottom friction, crushing and non-linear wave effects, etc. It adopts the fully implicit finite difference scheme and is unconditionally stable. This model has been successfully applied to wave prediction of coastal, estuary and offshore waters [7-15]. The version used in this paper is SWAN40.91.

3. Parameter selection and calculation conditions

In this chapter, the numerical simulation of cold wave and wind wave during December 20-22, 2008 was performed by SWAN model. The frigid wind field datum is obtained from CCMP satellite remote sensing of sea surface wind field datum. The offshore calculated domain is 20.4 °N to 41.9 °N and 117.1 °E to 131.4 °E in China. The model uses the unstructured triangular mesh with 70147 elements and 35814 nodes, direction is divided into 36 sections, resolution is 10 °. The simulation time is from 0 o'clock on December 17, 2008 to 18 o'clock on December 26, 2008, and the simulation time step is 600s. The simulation results were verified by Jason-1 track datum. The calculation area of large-scale wave mathematical model is shown in figure 1. During cold wave wind, the satellite was in the cycle 256, and orbits T127, T138, and T153 passed over the simulated area at about 21:00 on 22nd, 8:00 on 23rd and 13:00 on 23rd respectively (as shown in Figure2).
In order to further verify the accuracy of the simulation results, the simulated wave height is verified with the measured data of the wave station in the cold wave model. The Xiangshui wave station is located near the waste Yellow River estuary along the coast of Jiangsu province. The depth of the buoy is about 8m. The data is collected every hour. The buoy records the wave element process near the measured wave point during cold wave wind. The simulation results during the occurrence of cold wave wind and the measured effective wave height of the Xiangshui station were compared and analyzed. The results are shown in Figure 4. It can be seen from the verification results in Figure 3 and Figure 4 that the effective wave height calculated by SWAN model has a good accuracy, that is, the large-range mathematical model which has been established can reflect the wave field distribution in the east China sea during the period of cold wave wind, which can provide reliable spectral boundary for the simulation of large cold wave in a small range.
4. Impact of coastal beach reclamation planning in Jiangsu on cold wave and wind wave field

In order to study the effect of large-scale reclamation on offshore wave field in cold wave and windy weather, on the background of large-scale beach reclamation planning project in Jiangsu sea, on the basis of accurate simulation of large cold wave in east China sea, the SWAN model was used to self-nest to calculate the wave field before and after the large-scale reclamation in Jiangsu coastal area. The small scale research area is a little larger than the Jiangsu coastal area (The range is from 29.5 ° N to 35.8 ° N and 119.1 ° E to 124.3 ° E), the model adopted unstructured triangular mesh which is in accordance with the coastline of high degree. The grid node number is 48172, grid cell number is 94814, grid spacing decreases from the coast to the nearshore. In order to better depict the reclamation area and conduct grid encryption for the Jiangsu coastal area, the minimum grid spacing is 500m, which not only saves the calculation time, but also can fully improve the simulation accuracy of coastal areas of Jiangsu. The grid section diagram of the simulated area is shown in Figure 5.

In order to fully research scale reclamation planning engineering impact on wind wave characteristics in Jiangsu sea a series of characteristic points have been selected in Jiangsu coastal area surrounding the enclose tideland(as shown in Figure 6). And the feature points and reclamation area were positively associated with the degree of distribution density, the larger reclamation area is, the more intensive the feature points layout are, and the smaller the reclamation area is, the sparser feature points are. According to the distance between the feature points and the reclamation area, the feature points were divided into three layers. The first layer, A1–A25, was mainly distributed in the nearshore sea area near the reclamation area. B1–B19 are the second layer, slightly farther from the reclamation area than the first layer. C1–C15 are the third layer, which is distributed in the outermost sea area.

In order to fully depict the impact of large-scale reclamation on the cold wave, three typical moments are selected from the simulation results as the analysis objects. The typical moment includes the moment when the cold wave and the wind start to affect the study area, the moment when the influence is greatest and the moment when the influence will disappear. Figure 7 shows the wave field distribution of cold wave and strong wind at typical time. According to the wave field of cold wave wind at each typical moment, the cold wave wind had the biggest impact on the whole reclamation area of Jiangsu at around at 22 o’clock on the 21st. In order to further analyze the impact of coastal beach reclamation on wind and waves in Jiangsu province, the wave height data of the pre-reclamation and post-reclamation wave feature stations were extracted respectively at 22 o’clock on the 21st according to the model calculation results, and the statistical analysis was conducted, as shown in table 1.
Figure 6. The distribution of feature points of surrounding sea area after reclamation

Figure 7a. The distribution of Hs and direction at 13:00 on day 21.

Figure 7b. The distribution of Hs and wave direction at 22:00 od day 21

Figure 7c. The distribution of Hs and wave direction at 21:00 od day 22.
<table>
<thead>
<tr>
<th>Stations</th>
<th>Pre-reclamation</th>
<th>Post-reclamation</th>
<th>Variations</th>
<th>Rate of change</th>
<th>Stations</th>
<th>Pre-reclamation</th>
<th>Post-reclamation</th>
<th>Variations</th>
<th>Rate of change</th>
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<td>0.94</td>
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<td>1.57</td>
<td>1.58</td>
<td>0.01</td>
<td>0.45%</td>
</tr>
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<td>A2</td>
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<td>0.94</td>
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<td>0.91%</td>
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<td>1.14</td>
<td>0.92</td>
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<td>-19.5%</td>
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<tr>
<td>A3</td>
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<td>1.06</td>
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<td>1.95</td>
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<td>A5</td>
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<td>-0.04</td>
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<td>B10</td>
<td>2.24</td>
<td>2.19</td>
<td>-0.04</td>
<td>-1.98%</td>
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<td>A6</td>
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<td>2.16</td>
<td>0.06</td>
<td>2.99%</td>
<td>B11</td>
<td>2.09</td>
<td>2.11</td>
<td>0.02</td>
<td>1.15%</td>
</tr>
<tr>
<td>A7</td>
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<td>-0.02</td>
<td>-1.18%</td>
<td>B12</td>
<td>2.18</td>
<td>2.18</td>
<td>0.00</td>
<td>0.20%</td>
</tr>
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<td>A8</td>
<td>1.14</td>
<td>1.01</td>
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<td>-10.9%</td>
<td>B13</td>
<td>2.18</td>
<td>2.15</td>
<td>-0.03</td>
<td>-1.57%</td>
</tr>
<tr>
<td>A9</td>
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<td>-22.7%</td>
<td>B14</td>
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<td>-0.01</td>
<td>-0.48%</td>
</tr>
<tr>
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<td>-0.11</td>
<td>-9.34%</td>
<td>B15</td>
<td>1.82</td>
<td>1.86</td>
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</tr>
<tr>
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<td>1.42</td>
<td>1.49</td>
<td>0.07</td>
<td>4.88%</td>
<td>B16</td>
<td>1.77</td>
<td>1.80</td>
<td>0.02</td>
<td>1.35%</td>
</tr>
<tr>
<td>A12</td>
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<td>1.18</td>
<td>0.00</td>
<td>0.07%</td>
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<td>1.43</td>
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</tr>
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<td>-2.55%</td>
<td>B18</td>
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<td>1.38</td>
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<td>0.23%</td>
</tr>
<tr>
<td>A14</td>
<td>1.64</td>
<td>1.71</td>
<td>0.07</td>
<td>4.23%</td>
<td>B19</td>
<td>1.30</td>
<td>1.30</td>
<td>-0.01</td>
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</tr>
<tr>
<td>A15</td>
<td>1.73</td>
<td>1.72</td>
<td>-0.01</td>
<td>-0.59%</td>
<td>C1</td>
<td>1.65</td>
<td>1.64</td>
<td>-0.01</td>
<td>-0.48%</td>
</tr>
<tr>
<td>A16</td>
<td>1.63</td>
<td>1.55</td>
<td>-0.08</td>
<td>-4.96%</td>
<td>C2</td>
<td>2.33</td>
<td>2.31</td>
<td>-0.02</td>
<td>-0.78%</td>
</tr>
<tr>
<td>A17</td>
<td>1.78</td>
<td>1.74</td>
<td>-0.03</td>
<td>-1.86%</td>
<td>C3</td>
<td>2.50</td>
<td>2.50</td>
<td>0.00</td>
<td>0.07%</td>
</tr>
<tr>
<td>A18</td>
<td>1.97</td>
<td>1.95</td>
<td>-0.02</td>
<td>-1.05%</td>
<td>C4</td>
<td>3.18</td>
<td>3.18</td>
<td>0.00</td>
<td>-0.06%</td>
</tr>
<tr>
<td>A19</td>
<td>1.49</td>
<td>1.54</td>
<td>0.05</td>
<td>3.15%</td>
<td>C5</td>
<td>2.83</td>
<td>2.84</td>
<td>0.02</td>
<td>0.58%</td>
</tr>
<tr>
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<td>1.18</td>
<td>1.28</td>
<td>0.10</td>
<td>8.30%</td>
<td>C6</td>
<td>2.12</td>
<td>2.12</td>
<td>-0.01</td>
<td>-0.41%</td>
</tr>
<tr>
<td>A21</td>
<td>1.21</td>
<td>1.27</td>
<td>0.07</td>
<td>5.58%</td>
<td>C7</td>
<td>1.58</td>
<td>1.57</td>
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</tr>
<tr>
<td>A22</td>
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<td>0.88</td>
<td>-0.01</td>
<td>-0.84%</td>
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<td>2.43</td>
<td>2.43</td>
<td>0.00</td>
<td>0.07%</td>
</tr>
<tr>
<td>A23</td>
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<td>0.93</td>
<td>-0.03</td>
<td>-3.53%</td>
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<td>2.69</td>
<td>2.71</td>
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</tr>
<tr>
<td>A24</td>
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<td>0.82</td>
<td>-0.01</td>
<td>-0.96%</td>
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<td>2.58</td>
<td>2.60</td>
<td>0.01</td>
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</tr>
<tr>
<td>A25</td>
<td>0.58</td>
<td>0.59</td>
<td>0.01</td>
<td>1.74%</td>
<td>C11</td>
<td>2.49</td>
<td>2.47</td>
<td>-0.01</td>
<td>-0.60%</td>
</tr>
<tr>
<td>B1</td>
<td>1.09</td>
<td>1.03</td>
<td>-0.06</td>
<td>-5.43%</td>
<td>C12</td>
<td>2.70</td>
<td>2.66</td>
<td>-0.04</td>
<td>-1.37%</td>
</tr>
<tr>
<td>B2</td>
<td>1.44</td>
<td>1.50</td>
<td>0.06</td>
<td>3.92%</td>
<td>C13</td>
<td>2.39</td>
<td>2.40</td>
<td>0.02</td>
<td>0.74%</td>
</tr>
<tr>
<td>B3</td>
<td>1.63</td>
<td>1.47</td>
<td>-0.17</td>
<td>-10.2%</td>
<td>C14</td>
<td>1.90</td>
<td>1.91</td>
<td>0.00</td>
<td>0.26%</td>
</tr>
<tr>
<td>B4</td>
<td>2.63</td>
<td>2.65</td>
<td>0.02</td>
<td>0.81%</td>
<td>C15</td>
<td>1.85</td>
<td>1.84</td>
<td>-0.01</td>
<td>-0.42%</td>
</tr>
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<td></td>
<td></td>
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<td></td>
</tr>
</tbody>
</table>

It can be seen from Figure 7 that with the development of time, the wave direction near the coast of Jiangsu did not change much except for Haizhou bay during the occurrence of cold wave and high wind, which the direction was almost NNE. The wave direction in Haizhou bay has significant changes over time. At 7 o'clock on the 22nd, the direction was NNE. It is nearly vertical to the northern coast of Haizhou bay and has a great influence on the sea area. The wave height has a certain degree of increase, but owing to the water depth of Haizhou bay is shallow, and the area of beach reclamation is small, the wave height increment is finite (such as A19 - A21 and B15 to B17). The reclamation area in the southern part of central Jiangsu province, especially in Lengjiasha, Dongsha...
and Gaoni, had a great influence on the propagation of wind and wave. The wave height in the southern reclamation area was generally reduced, and the sea area closer to the reclamation area was covered by the reclamation area significantly. The height of wave decreased, with a decrease of -32.98% in the characteristic point A4 and -22.69% in the characteristic point A9. Waves are not blocked, it can propagate directly into the bay formed in the reclamation area. Reflection occurred in the bay, and the wave height of the characteristic point A13 increased.

In order to show the influence of tideland reclamation on the cold wave more intuitively, the wave height changes at the feature points of pre-reclamation and post-reclamation were taken, and the feature stations were numbered from A1 to C15 in order of 1 to 59, and Figure 8 was drawn.

![Figure 8](image)

Figure 8. The variation of Hs of feature points station before and after reclamation.

5. Conclusion
While reclamation brings social and economic benefits, it will also lead to a series of irreversible negative effects. The changes of Marine environment caused by coastal beach reclamation are of great significance to the economic and social development of coastal areas. In this paper, based on the planning of the large-scale tidal mudflat reclamation in Jiangsu province, the variation of the cold wave and wind wave field caused by the large-scale tidal mudflat reclamation was analyzed by means of numerical simulation. The large change of wave field is mainly concentrated in the southern part of central Jiangsu province, which is because the area of reclamation area in this area is large, and it has a significant influence on the wave field. Due to the effect of covering, the wave height of the southern sea area of Dongsha, Gaoni and Lengjiasha reclamation area decreases greatly, and the decline can be as much as 30%. Due to the extent of reclamation is small, the change of wave field in the northern part of Jiangsu province is not obvious, and the range of change is not obvious.

6. References

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Numerical Simulation of the Influence of Large-scale Structures on Wave Force of Adjacent Small-scale Bars in Composite Structures

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Abstract. In order to calculate the wave loads on the small and medium scale bars of composite structures accurately, not only the action of incident wave but also the wave diffraction caused by near large-scale structure and the effect of radiation on wave load of small scale bars should be considered. In this paper, based on the VOF method, the three-dimensional wave numerical flume of the wave acting on the composite structure is constructed in Fluent. The incident wave force, diffraction wave force and total force acting on the composite structure are obtained by numerical calculation. Compared with the results obtained by using the Morison formula alone, the effectiveness of the wave loads calculated by the model is verified. The results show that under certain wave conditions, the wave diffraction caused by large-scale structures can not be ignored. In addition, the influence of the incident wave number, the diameter of the pile and the position of the pile on the wave load of the small-scale pile is also studied. The results can provide a scientific reference for the accurate calculation of wave loads on small-scale piles in marine engineering with composite structures.

1. Introduction
Pile structure of various scales is a common structural form in marine engineering. Scholars at home and abroad have done a great deal of researches on wave loads on pile structures. MacCamy et al. [1] gave an analytical solution to the problem of diffraction of vertical cylinders in finite water depth. Linton et al. [2] studied the interaction of waves on vertical pile groups. Yilmaz et al. [3] studied the diffraction of truncated cylinders. Existing researches on Truss Spar platform and TLP platform ignore the diffraction and radiation due to the existence of large scale main structure usually when using Morison formula to solve the stress of small-scale members or tension legs of the platform. Geng B L et al. [4] calculated the wave loads of small and medium diameter cylinders in diffracted wave field by using Morison formula. On the assumption of linear theory, Jiang S C et al. [5] established an analytical solution of vertical cylindrical wave diffraction under finite water depth. Wu and Chwang et al. [6] studied the diffraction problem of submerged two-dimensional porous horizontal thin plates. Yu and Chwang et al. [7] studied the wave distribution when the wave passed through the submerged disk. Sakar et al. [8] studied the scattering velocity potential and the radiation velocity potential of the truncated cylinder. Techet et al. [9] gave the boundary conditions of wave radiation problem of large diameter cylinder and the wave force calculation formula. Y.Drobyshhevski et al. [10] obtained the analytical solution of hydrodynamic characteristics of truncated cylindrical platform by means of asymptotic matching method, Hu J M et al. [11] numerically simulated the diffraction problem of a
body in a numerical wave pool.

Considering the deficiency of the research on wave loads of small and medium scale piles in composite structures at present, this paper takes the composite structures composed of large-scale cylinder and small-scale pile as the research object, and considers the effect of diffraction. In this paper, the wave loads on composite structures are studied. In this paper, based on Fluent software, N-S equation is used as control equation, VOF method is used to control free surface, UDF secondary development function is used to realize physical wave generation, and PISO velocity coupling mode is adopted. The method of adding additional source term to the momentum equation is used to reduce the reflection of distant walls, and a three-dimensional numerical wave flume which can produce stable linear regular waves is established. The wave force acting on a small scale member is obtained by numerical calculation, and the influence of the incident wave number, the diameter of the upper cylinder and the position of the pile on the diffraction is also analyzed. The research results in this paper will provide a good scientific basis for the parameter design of composite marine structures.

2. Mathematical Model

In this paper, the fluid in the numerical wave tank is set up as incompressible viscous fluid. The governing equations include continuity equation and Navier-Stokes equation.

Continuity equation:

\[ \frac{\partial \rho}{\partial t} + \frac{\partial (\rho u)}{\partial x} + \frac{\partial (\rho v)}{\partial y} + \frac{\partial (\rho w)}{\partial z} = 0 \]  

(1)

Momentum conservation equation:

\[ \frac{\partial u}{\partial t} + u \frac{\partial u}{\partial x} + v \frac{\partial u}{\partial y} + w \frac{\partial u}{\partial z} = - \frac{1}{\rho} \frac{\partial p}{\partial x} + \nu \left( \frac{\partial^2 u}{\partial x^2} + \frac{\partial^2 u}{\partial y^2} + \frac{\partial^2 u}{\partial z^2} \right) + g_x \]  

(2)

\[ \frac{\partial v}{\partial t} + u \frac{\partial v}{\partial x} + v \frac{\partial v}{\partial y} + w \frac{\partial v}{\partial z} = - \frac{1}{\rho} \frac{\partial p}{\partial y} + \nu \left( \frac{\partial^2 v}{\partial x^2} + \frac{\partial^2 v}{\partial y^2} + \frac{\partial^2 v}{\partial z^2} \right) + g_y \]  

(3)

\[ \frac{\partial w}{\partial t} + u \frac{\partial w}{\partial x} + v \frac{\partial w}{\partial y} + w \frac{\partial w}{\partial z} = - \frac{1}{\rho} \frac{\partial p}{\partial z} + \nu \left( \frac{\partial^2 w}{\partial x^2} + \frac{\partial^2 w}{\partial y^2} + \frac{\partial^2 w}{\partial z^2} \right) + g_z \]  

(4)

In the formula, \( u, v \) and \( w \) are the velocities in \( x \), \( y \) and \( z \) directions respectively; \( g_x, g_y, \) and \( g_z \) are volume accelerations in \( x \), \( y \) and \( z \) directions respectively; \( p \) is fluid pressure, \( \rho \) is fluid density, \( \nu \) is the viscosity coefficient of fluid motion.

3. Numerical simulation

3.1. Geometric Model.

The geometric diagram of the three-dimensional numerical wave flume is shown in Figure 1. The overall size of the tank is 20m long, 4m high and 4m wide. The initial water depth of the flume is 2m, above the water surface is air, and the wave absorption area is 16~20m at the end of the flume. The radius of the upper large-scale cylinder of the composite structure is 0.4m and the draught of the cylinder is 1m. The superstructure is supported by a small scale pile with a radius of 0.1m and a pile length of 1m. The pile is in the same axis as the upper cylinder. The left wall of the three dimensional flume is used to simulate the wave-making plate. The coordinate system of the 3D model is arranged as shown in Figure 2, and the wave propagates in a positive direction along the \( x \) axis.
3.2. Parameter Setting.
The structure grid is divided in ICEM, and the motion law of the plate in the numerical wave tank and the wave elimination source terms in the wave absorption area are compiled by UDF. The left wall of the tank is equivalent to the wave-making plate. The moving mesh model is used to realize the motion of the wall. The pressure inlet boundary is set at the top of the flume to define the pressure condition and other scalar properties of the inlet, and the other boundary is wall boundary. The free water surface is obtained by VOF method, the pressure and velocity terms are calculated by Piso algorithm, the grid updating method of Layering is used to realize the wave-making of push plate. The time step is 0.01s and the simulated time is 20s.

3.3. Model verification.
In order to verify the validity of the model, the numerical results are compared with the theoretical results. The incident regular wave height is 0.15m and the period is 1.6s. It can be seen from the diagram that the simulated value is in good agreement with the calculated value of Morison formula. It can be preliminarily concluded that FLUENT has considerable accuracy in numerical simulation of wave interaction with small-scale vertical column structures.
4. Result and analysis

4.1. Influence of incident wave number on wave force of pile.

The regular incident wave with different incident wave number $ka$ is considered. Figure 4 is comparisons of incident wave force, diffraction wave force and total force $e$ of the pile with different incident wave number $ka$.

It can be seen from the diagram that the amplitude of the total force is larger than that of the incident wave force when the $ka$ values are different, because the phase of the diffraction force is basically the same as the incident force. When $ka<1.0$, the diffraction wave force is smaller than the total force, and the total wave force is mainly dependent on the incident wave force, and the diffraction wave force can be neglected. When $ka>1.0$, the effect of diffraction wave becomes more and more obvious and cannot be ignored. At $ka=2.5$, the diffraction wave force is about equal to the incident wave force, and the amplitude of the total force is about twice that of the incident wave force. When $ka>2.5$, the diffraction wave force is larger than the incident wave force, which dominates the total force.

4.2. Influence of upper cylinder size on wave force of pile.

When the radius of the upper cylinder is different multiple of $a$, let the radius of the different cylinder be $R=na$, and $n$ be equal to 1, 2, 3, 4 respectively. The influence of diffraction field on the wave load of a small pile with different radius is investigated. Consider the regular wave incidence of the wave number $ka=0.8$ with a wave amplitude of 0.75. Figure 5 shows comparisons of incident wave force,
diffraction wave force and total force amplitude of the pile with different radius R of the upper cylinder.

![Figure 5](image)

**Figure 5.** Comparisons of incident, diffraction and total waves forces with different upper cylindrical radius

Since the incident wave field is invariant, the incident wave force is the same, and its amplitude is 8.8N. It can be seen from the diagram that the diffraction field becomes stronger with the increase of the radius of the upper large-scale cylinder. The diffraction wave force on the bottom pile also increases, but the amplitude of the increase in wave force gradually slowed down. At the same time, due to the difference between the phase of diffraction force and the incident force, the amplitude of total force is smaller than the incident wave force most of the time.

### 4.3 Influence of upper cylinder size on wave force of pile.

For ease of representation, the projection of the composite structure on the XOZ plane is represented by polar coordinates (r, θ), as shown in Figure 6. In the calculation, the incident condition is invariant, the radius of the large scale cylinder is a, and the other dimensions of the structure are the same as before. Because the incident wave goes in the positive direction of x, according to symmetry, the wave load on the pile at the symmetrical position on the left and right sides of the x axis is the same, so only the force of the pile at different positions $\theta \in [0^\circ, 180^\circ]$ is studied here. Figure 7 shows comparisons of the diffraction wave force and the total wave force at different values of r and θ.

![Figure 6](image)

**Figure 6.** Projection of the pile at XOZ plane
Because the incident wave condition is invariant, the amplitude of incident wave force is still constant, its value is 8.8N. It can be seen from Figure 7 that the amplitude of force and diffraction wave force are smaller than the incident wave force. For the total force, the amplitude of the total force increases with the increase of the distance r. The amplitude of the total force is larger than that of the diffraction wave force when r is 0.5a and 0.75a, and the diffraction wave force is larger than the total force at the back side of $\theta \in [135^\circ, 180^\circ]$ when r is 0.25a. At the front side of $\theta \in [0^\circ, 90^\circ]$, the amplitude of total force is larger than that of diffraction wave force. The amplitude of total force reaches maximum at $\theta=0^\circ$ and minimum at $\theta=135^\circ$. On the contrary, the diffraction force reaches the maximum at $\theta=135^\circ$ and the minimum at $\theta=0^\circ$. The diffraction wave force decreases with the increase of the distance r. When r is 0.25a, the diffraction wave force is almost equal to the total force due to the phase difference.

5. Conclusion
In this paper, based on the VOF method, the numerical wave environment of wave action on composite structures is constructed in Fluent. The wave loads on small and medium scale bars in diffraction field are analyzed by a series of numerical examples, the conclusions are as follows: (1)

![Figure 7. Comparisons of diffraction and total wave forces with different r and $\theta$](image-url)
under the action of regular wave, when \( ka > 2.5 \). The diffraction wave action is greater than the incident wave action, which dominates the total force, and the magnitude of the total force is about twice of the incident wave force. (2) the size of the upper part directly changes the shape of the diffraction field in the wave field. The wave force of the diffraction field on the pile increases with the increase of the upper cylinder size. (3) the wave force of the small-scale pile will be greatly affected by the different positions of the small-scale pile in the diffraction field, and for the total force, the wave force will be greatly affected by the different positions of the small-scale piles in the diffraction field. The amplitude of the resultant force increases with the increase of distance \( r \). The diffraction wave force decreases with the increase of distance \( r \).

6. Reference

Assessment of coastal vulnerability to sea level rise:
a case study of Prachuap Khiri Khan, Thailand

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Abstract. The coastal zone of Thailand will get the effect from the eustatic sea level rise due to climate change. It is necessary to protect the coastal area from it. Therefore, coastal vulnerability index (CVI) was chosen to be a tool to identify the vulnerability of coastal zone in Prachuap Khiri Khan which is the study area. The physical and socioeconomic variables for CVI in this study were a coastal slope, rate of shoreline changes, geomorphology, signification wave height, tidal range, sea level changes, population density, coastal structure and land use. The CVI values was classified in 5 classes of vulnerabilities in the format of the coastal vulnerability map. The results presented that most of the coastal area in Prachuap Khiri Khan had very low to moderate vulnerability to sea level rise. It can be the preliminary analysis to develop the coastal management for this area in the future.

1. Introduction
Coastal zones are the plentiful ecosystems that provide many benefits to human. Coastal zones are the dynamic areas that have changed by increasing populations and the development activities including the industry. According to the report from Intergovernmental Panel for Climate Change (IPCC) [1], global sea level is rising in this century due to climate change. From the previous study, it indicated that sea level has risen in the Gulf of Thailand and Andaman sea average rate of 6.5 mm/yr [2]. Moreover, the projected beach loss in Thailand from the future sea level rise may reach a maximum of 71.8% or 39.77 km² [3]. Prachuap Khiri Khan was selected to the study area because this province has the longest coastline in Thailand. Many tourist attraction, two of the royal palace and famous beaches such as Huahin beach and Suan-son beaches located in Prachuap Khiri Khan. Moreover, there are several beachfront hotels and communities along the coastline. Therefore, it is necessary to plan the countermeasure and protect the coasts from the impacts of sea level rise.

Coastal vulnerability assessment is a spatial tool to identify the susceptible of the coast from the hazard. Coastal vulnerability index (CVI) was first developed for assessing the susceptibility to erosion of the coast in the United States from sea level rise [4]. There are many studies about coastal vulnerability assessment in Thailand which have different variables. In this study, CVI was considered by physical, economic and social parameters which were a coastal slope, rate of shoreline changes, geomorphology, signification wave height, tidal range, sea level changes, population density, coastal structure and land use. The main purposes of this study were to develop the coastal vulnerability index for sea level rise to identify the vulnerability of coastal zone in Prachuap Khiri Khan and to analyze the coastal vulnerability index presented as the coastal vulnerability map.

2. Study area
Prachuap Khiri Khan is the province in the western part of Thailand which located on the Gulf of
Thailand (Figure 1). It has 22 coastal sub-districts with 246.83 km lengths of coastline that are 178.27 km lengths of sandy beaches (Figure 2), 8.86 km lengths of sandy mud beaches, 1.85 km lengths of muddy beaches, 0.89 km lengths of rocky beaches, 55.13 km lengths of headlands and 1.83 km lengths of river mouths. There are 3.72 km lengths of shoreline with low-rate of erosion (0-1 m/yr), 0.22 km lengths of moderate-rate of erosion (1-5 m/yr) and 0.55 km lengths of high-rate of erosion (1-5 m/yr) [5]. Prachuap Khiri Khan got the effect of south-west monsoon in the rainy season (May-October) and north-east monsoon from China that made this area becomes cooler in the winter season (October-February).

3. Methodology

This study aims to analyze the coastal vulnerability index by considering the physical and socioeconomic variables (Table 1) before divides the data in the ranges as in Table 2. The coastal vulnerability index (CVI) was calculated by the square root of product mean [6] which was used in many research as in equation (1) and shows the results as a coastal vulnerability map. The method of this study is shown in Figure 3.

\[ CVI = \sqrt{a \times b \times c \times d \times e \times f \times g \times h \times i} \]

Where \( a = \) coastal slope, \( b = \) shoreline changes, \( c = \) geomorphology, \( d = \) signification wave height, \( e = \) tidal range, \( f = \) sea level changes, \( g = \) population density, \( h = \) coastal structure, \( i = \) land use and \( n = \) number of variables.
Figure 3. Research methodology

Table 1. Variables used in this study.

<table>
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<th>Variables</th>
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<td>Shoreline changes</td>
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<td>Geomorphology</td>
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<td>Tidal range</td>
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<td>The bureau of registration administration, Department of Provincial Administration</td>
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Table 2. Coastal vulnerability index classification

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<td></td>
</tr>
<tr>
<td>Conserved forest, Beach Forest, Mangrove, Pine forest, Grassland, Grove</td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Agriculture, Aquaculture, Salt pan, Mine</td>
<td>3</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Tourists attraction, Roads, Miscellaneous area</td>
<td>4</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Royal area, Community area, Industrial zone, Government bureau, Business quarter, Religious place, School, Hospital, Historical monuments</td>
<td>5</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

4. Results and discussions

4.1 Coastal vulnerability to sea level rise in Prachuap Khiri Khan

As shown in Figure 4, the coastal vulnerability index had the minimum values as 1.82 and maximum values as 136.93. Therefore, the CVI values in this study were separated in 5 class intervals as very low (1-28.4), low (28.4-55.8), moderate (55.8-83.2), high (83.2-110.6) and very high (110.6-138).
4.2 Percentage of vulnerable area in Prachuap Khiri Khan

Prachuap Khiri Khan has 116.04 km² of coastal area which consists of 84.39 km² or 72.72 percentage of very low vulnerability area, 29.87 km² or 25.74 percentage of low vulnerability area, 1.53 km² or 1.32 percentage of moderate vulnerability area, 0.21 km² or 0.18 percentage of high vulnerability area and 0.04 km² or 0.03 percentage of very high vulnerability area (Figure 5). Most of the coastal areas in Prachuap Khiri Khan had very low to moderate vulnerability to sea level rise because there are the coastal structures placed in these areas with a very low rate of erosion, high coastal slope, low of the population density and the open space kind of land use. As can be seen in Figure 6, the area of very high vulnerability was located in Prachuap Khiri Khan sub-district which has the communities with

---

*Figure 4. Coastal vulnerability map*
very high population density. It located behind the famous tourist attraction which is Thammikaram temple on Chong-krajok mountain. Moreover, there is the government bureau located on the flatland with no coastal structure. Then, these factors led to the very high vulnerability.

**Figure 5.** Represent the percentage of vulnerable area in Prachuap Khiri Khan

**Figure 6.** The very high vulnerability area in Prachuap Khiri Khan sub-district

5. **Conclusions and recommendations**

The coastal vulnerability index was chosen to be the tool for analyzing the coastal vulnerability to sea level rise in Prachuap Khiri Khan. In this study, the physical and socioeconomic variables were determined as a coastal slope, the rate of shoreline changes, geomorphology, signification wave height, tidal range, sea level changes, population density, coastal structure and land use. The CVI values were separated in 5 class intervals as very low (1-28.4), low (28.4-55.8), moderate (55.8-83.2), high (83.2-110.6) and very high (110.6-138). There were 72.72 percentage of very low vulnerability area, 25.74 percentage of low vulnerability area, 1.32 percentage of moderate vulnerability area, 0.18...
percentage of high vulnerability area and 0.03 percentage of very high vulnerability of the total area of coastal zone in Prachuap Khiri Khan.

Most of the coastal zones in Prachuap Khiri Khan had very low to moderate vulnerability to sea level rise. The variables leading to low vulnerability were the rate of erosion, coastal slope, land use, population density and coastal structure. The coastal structure and land use were the important variables for analyzing the coastal vulnerability because the coastal structure affected the rate of erosion and the land use was directly related to the population density.

This study is the preliminary analysis of the coastal vulnerability index for this study area due to the limitation of the data that affected the accuracy and precision of some variables. Therefore, the accuracy of coastal vulnerability assessment can be improved by developing the coastal vulnerability index and updating the data of variables or determining other variables in the future study. However, the preliminary planning and countermeasure can be applied from the result of this study in the management of the coastal zone in Prachuap Khiri Khan.

6. References


[5] Department of Marine and Coastal Resources Thailand 2018 Information of marine and coastal resources in Prachuap Khiri Khan pp 56-90 (in Thai)


Acknowledgements

This research was supported by Advancing Co-Design of Integrated Strategies with Adaptation to Climate Change in Thailand Project (ADAP-T). The authors would like to express grateful thanks to Department of Marine and Coastal Resources Thailand, Meteorological Department of Thailand, Department of Marine and Hydrographics Department, Royal Thai Navy for the data information.
Automatic Geometric Correction of Complex Sea Condition Remote Sensing Image Based on Decision Tree Classification

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Abstract. The geometric correction of ocean remote sensing image is a prerequisite for its data application. In this paper, to solve the problem that the sea island is sparse, cloud interference is severe, the control point is difficult to obtain, an automatic correction technique based on decision tree classification is proposed. In this paper, the image is processed by the method of super-pixel segmentation first. Then, the spectral and texture features in the superpixels are selected, including the energy value, the entropy and the correlation value of the gray level co-occurrence matrix and the normalized water index. Finally, the tree image classification model is used to classify the image superpixels, and the clear sky area which will be matched directly with the reference image can be extracted. Through the template matching and polynomial geometric model, the geometric correction of the remote sensing image is automatically corrected. Through the experiment of Landsat8 OLI_TIRS image, compared with the classification results of the other two classification methods, the final precision is better than the other two methods. Therefore, the technical process proposed in this paper can be applied to the geometric correction of complex sea condition remote sensing images.

1. Introduction

With the continuous development of earth observation technology, the application of remote sensing is more and more widely, of which ocean remote sensing is widely used in waterway, fishery, meteorological and other fields. The geometric correction of remote sensing image is a prerequisite for its application. And a large number of remote sensing images are constantly updated, the match method that the control points are selected manually has been unable to meet the needs of data production. Automatic remote sensing image geometric correction technology can solve such problems. The traditional method of automatic correction of remote sensing images by using the rich feature points on land has good effect. However, in the ocean environment, due to the large area of seawater, the island is sparse, cloud interference is great, resulting in match point can not be obtained, fine correction can not be completed automatically.

Geometric correction requires the more obvious control points in remote sensing images. In order to get control points in cloudy and less island images, it is first necessary to eliminate cloud interference. Second, islands or coastline which have obvious features need to be searched. Finally, by using the feature-specific islands and coastline, the images are matched with the reference images to complete a remote sensing image correction. In order to eliminate cloud interference, Chen uses the tree-like structure to extract the characteristics of the sub-block image. The second moments and the first order difference of the block image are calculated and the cloud and the object are separated. But the tree
structure is fixed, only applies to the removal of cloud interference in a particular remote sensing image. And the block division results in a large difference in pixel characteristics between each block, which has a greater effect on the accuracy of the subsequent classification results. The equal size segmentation method results in a large difference in the characteristics of each block pixel, which has a great influence on the accuracy of the classification result. Liu Pengyu [2] used the gray-level co-occurrence matrix and Gabor filter to extract the texture features of the image, and comprehensively describes the contents of the image, so that the cloud, land and water were classified. Chen [3] used the decision tree to classify the remote sensing images, and concluded that the decision tree method is simple and intuitive, and the overall classification accuracy is 90.65%. However, for ocean with less islands in remote sensing images, islands are often difficult to classify, and broken clouds and small island reefs are prone to misclassification.

In this paper, first, the image are divided into sub-regions by superpixel image segmentation. The pixels in each sub-region are similar under the measure of brightness and texture. Based on the characteristics of the brightness and texture of ocean remote sensing images, use the image super pixel block to train to get the decision tree model. From the classification results, the super-pixel blocks of test image whose island or coastline characteristics is obvious are obtained, which can be matched with reference image. Then a series of control points after the template matching, and the geometric correction of the test image is carried out by using the polynomial model. Thus the automatic geometric correction process is completed. This paper is based on the OLI_TIRS image of Landset8 in Ryukyu Islands.

2. Geometric correction process
Through the analysis of ocean remote sensing images, The following four types exist in the image.

![Figure 1.](image)

The clear sky island images can match the benchmark image well, because this situation that the cloud interference is small, coastline characteristics is obvious is similar to the reference image. So it is necessary to automatically extract the range of the clear sky in a remote sensing image. This process requires the segmentation and classification of the images. For the adaptive segmentation method, this paper adopts the super-pixel segmentation. For the classification method, the decision tree is used to adapt the classification.

2.1. Superpixel image segmentation
In a remote sensing image automatically extract the scope of clear sky islands. This process requires the image to be segmented and sorted. For the adaptive segmentation method, this paper adopts the super-pixel segmentation. For the classification method, the decision tree is used to adapt the classification.

The super-pixel segmentation method is a simple linear iterative clustering method. The advantages of this method are computationally fast, the edge tracking effect is better[4], the input parameters are few, only one parameter k, which represents the estimated number of super-pixel segmentation.

The simple linear iterative clustering algorithm is as follows:

Step 1: Convert the RGB image to CIELab color space. Given the number of super-pixels to be segmented, the number k is 2000 and the image size is 4237 × 4205 pixels, denoted as N, then the seed points step distance S is calculated according to the following formula.

\[ S = \sqrt{\frac{N}{k}} \]
The seed points are evenly distributed over the image in steps.

Step 2: Perturb the seed point: Select the point with the smallest gradient change as the new seed point in the neighborhood of the 3 × 3 pixel of the seed point to prevent interference from the noise. Each seed point is a cluster center.

\[ C_k = [l_k, a_k, b_k, x_k, y_k]^T \]

The \( l_k \) is the brightness of the seed point, the \( a_k \) is the red and green axis value, the \( b_k \) is the yellow blue axis value, and \( x_k, y_k \) is the spatial coordinate of the seed point.

Step 3: Traverse all cluster centers \( C_i \), then traverse the pixels \( i \) with a size range of 2S centered around the seed points, calculate the distance measure of spatial characteristics of the pixels and the color characteristic:

The characteristic of space distance:

\[ d_s = \sqrt{(x_j - x_i)^2 + (y_j - y_i)^2} \]

The characteristic of color distance:

\[ d_c = \sqrt{(l_j - l_i)^2 + (a_j - a_i)^2 + (b_j - b_i)^2} \]

Set the factor \( m \) to represent the maximum value of the color distance. Integrated distance measure \( D' \):

\[ D' = \sqrt{\left(\frac{d_s}{S}\right)^2 + \left(\frac{d_c}{m}\right)^2} \]

Compare each pixel point to the surrounding cluster center, labeling the smallest distance of the superpixel.

2.2. Classification characteristics selection

There is a large gap between the ocean and the land in spectral characteristics. In order to highlight the characteristics of the water, the differential water index (NDWI) is selected as the spectral classification characteristics.

\[ NDWI = \frac{(\text{Band2} - \text{Band4})}{(\text{Band2} + \text{Band4})} \]

In this paper, the gray level co-occurrence matrix (GLCM) is used, which is defined as the probability that two pixels whose distances is \( d \) and the direction is \( x \) appears in the image. Because the image of the image is characterized by the texture of the cloud. Through the \((d, \theta)\) value, a lot of GLCM can be combined to analyze the spatial distribution pattern of image gray level. In this paper, the three measures are selected as energy, entropy and correlation.

Energy is the measure of image uniformity, the more uniform the image, the larger the value:

\[ Energy = \sum_{a,b} G_{\theta,d}^2(a,b) \]

Entropy is the measure of the amount of image information, the image is close to random or the noise is large, entropy will be larger:

\[ Correlation = \frac{\Sigma_{a,b}[(ab)G_{\theta,d}^2(a,b)]}{\hat{\delta}_x \hat{\delta}_y} - \mu_x \mu_y \]

Where \( \mu \) is the mean and \( x \) is the standard deviation.

\[ \mu_x = \sum_a a \sum_{b} G_{\theta,d}(a,b) \]

\[ \mu_y = \sum_b b \sum_{a} G_{\theta,d}(a,b) \]

\[ \hat{\delta}_x = \sum_a (a - \mu_x)^2 \sum_{b} G_{\theta,d}(a,b) \]
2.3. Decision Tree Classification

The decision tree is a simple but widely used classifier proposed by Breiman [5] and others, and the decision tree is constructed by training data, which can efficiently classify unknown data. There are two advantages: 1) The decision tree model can be read well and descriptive, and it is helpful for manual analysis. 2) High efficiency, decision tree only needs to be constructed ones and used repeatedly. The maximum number of calculations for each forecast is not more than the depth of the decision tree.

A decision tree includes a root node (Rootnode) on behalf of the input variable, a series of internal nodes (Internal nodes) on behalf of the branch and the terminal node (Terminal nodes) on behalf of the leaves.

In this paper, the Classification and Regression Tree (CART) is used to implement the decision tree. The principle is as follows:

The \( \{x_1, x_2, \ldots, x_n\} \) represents the n attributes of a single sample, and y represents the classes. The CART algorithm divides the dimension space into a non-overlapping rectangle by recursive way. The primary judgment at the tree node is called a branch, which corresponds to dividing the training sample into subsets. The branches at the root node correspond to the total training samples. Each subsequent decision is a training subset partitioning process, so the The process of tree construction is actually a property query to generate a division rules.

The Gini indicator is commonly used to measuring impurity. Assuming the number of samples is \( C \), the Gini impurity of the node can be defined as:

\[
Gini(A) = 1 - \sum_{i=1}^{C} p_i^2
\]

The \( p_i \) represents the probability of belonging to class \( i \). Or use the Entropy Impurity.

\[
EI(A) = - \sum_{i=1}^{C} p_i \log_2 p_i
\]

If all the samples of the model are from the same class, then the impurity is zero, otherwise it is a positive value. When all the classes appear with equal probability, the entropy is max.

2.4. Template match

Use the result which is classified by decision tree to match the reference image to obtain the control points.

Because of the obvious characteristics of the remote sensing image of the clear sky, the water brightness is low, and the land brightness is higher, so the template matching can be more accurate corresponding to the pixel.

The superpixel block classified as the clear sky is matched with the reference image. The difference between the superpixel image and the reference image is measured by the correlation, the better the matching, the greater the matching value. The correlation measure is calculated as follows:

\[
d_{corr}(H_1, H_2) = \frac{\sum_i H'_1(i) \cdot H'_2(i)}{\sqrt{\sum_i H'^2_1(i) \cdot H'^2_2(i)}}
\]

The \( H_1, H_2 \) represents the superpixel block to be matched and the reference image.

After obtaining the matching point, use the quadratic polynomial model to correct the images.

3. Results and analysis

3.1. Experimental results
The experimental data are OLI_TIRS images of Landset8, and the latitude and longitude range is 128.46° E ~ 129.75° E 28.39° N ~ 29.53° N. The area is Ryukyu Islands, the islands are extremely sparse and the cloud coverage is 40%.

The image whose size is 4237 × 4205 pixel is divided into 2000 superpixels, the following figure shows the results of superpixel segmentation. As can be seen from the results, each superpixel image type is more consistent. Then, five OLI_TIRS remote sensing images near the Ryukyu Island are processed with superpixel segmentation.

![Figure 2. The result of superpixel segmentation.](image)

The average NDWI of each superpixel and the energy, entropy and correlation of the gray level co-occurrence matrix are calculated and classified with decision tree. The following model is obtained by training the data.

![Figure 3. The decision tree classification model.](image)

The decision tree directly reflects the classification of each characteristic value. By analyzing the structure of the decision tree, it is known that the water index of the island land and the cloud is low, and the water body and the non-water body are distinguished to a great extent. The entropy of the texture feature greatly reflects the amount of information of the image, and the entropy of thick cloud is large because it contains a large random texture. The correlation value of sparse cloud is small, because its distribution is extremely uneven and the internal correlation is small. The island image has a certain correlation within, but its distribution is not uniform, so the correlation of island images texture is larger and the energy value is lower.

According to the model obtained by training, the results are tested with the data as follows:
Obtain the superpixel of the island as follows:

![Superpixel of the island](image)

**Figure 4.** The result of classification.

**Figure 5.** Superpixel of the island.

The model is used to match the blue sky island super pixel with the reference image, and the four islands are used as the matching control points, leaving one island as the precision test point.

### 3.2. Accuracy evaluation

The classification model of the decision tree is compared with the Naive Bayesian classification method and the K-means clustering method. The evaluation criterion is the amount of the super-pixels of clear sky and wrong classification. The evaluation of the island extraction is the NASA Coastline vector data.

The following are the results of the Naive Bayesian model and the K-means classification method.

<table>
<thead>
<tr>
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<th>Decision Tree</th>
<th>Bayesian Belief</th>
<th>k-Means</th>
</tr>
</thead>
<tbody>
<tr>
<td>Count of island super pixels</td>
<td>17</td>
<td>26</td>
<td>4</td>
</tr>
<tr>
<td>Count of misclassification</td>
<td>2</td>
<td>13</td>
<td>1</td>
</tr>
</tbody>
</table>

**Table 1.** The result of classification.

The number of clear sky islands classified with Bayesian classification are more, but most of them are covered the cloud, so the template matching method to match these superpixel will occur, k-Means method to get the number of clear blue islands, resulting in The number of control points is small, can not cover the entire image, the final geometric precision correction accuracy.

In this paper, 5 pair of test points of 5 images are randomly selected to verify the corrected image geometric accuracy. The following is the correction accuracy.
Table 2. The result of geometric accuracy.

<table>
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<tr>
<th>Test point number</th>
<th>Basemap test point</th>
<th>Corrected image</th>
<th>Relative error (m)</th>
</tr>
</thead>
<tbody>
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<td>Longitude</td>
<td>Latitude</td>
<td>Longitude</td>
</tr>
<tr>
<td>1</td>
<td>110.5953676</td>
<td>20.1104276</td>
<td>110.5945106</td>
</tr>
<tr>
<td>2</td>
<td>117.8379696</td>
<td>15.12165387</td>
<td>117.8378905</td>
</tr>
<tr>
<td>3</td>
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</tr>
<tr>
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</tr>
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<td>5</td>
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<td>35.70248423</td>
<td>126.5559381</td>
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<tr>
<td>Average</td>
<td></td>
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</tr>
</tbody>
</table>

4. Conclusion

Through the research and experimentation, this paper proposes the use of decision tree classification to extract the superpixel blocks of ocean islands with large cloud cover. This method has ability to search the area of the image that can match the reference image to a great extent and to achieve automatic geometric correction of the sparse island images.

At the same time, the superpixel segmentation is used to make the initial clustering of the region with similar internal features and spatial features, which is beneficial to the improvement of the accuracy of the classification result, which has a great advantage compared with the traditional block segmentation. The spectral and texture features of the islands are also full used. The decision tree model is analyzed intuitively. The island image has more texture information than the cloud and the ocean, and the internal correlation is strong and the brightness distribution is not uniform. However, there are some problems need to be solved in the next step. The adaptability of the template matching is poor, and it is susceptible to cloud interference. The island matching method needs to be improved. Non-uniform control points can not guarantee the geometric accuracy of the whole image, the quadratic polynomial model may not be able to meet the situation of non-uniform control points, and the geometric adjustment model needs to be improved.

5. References


Acknowledgement

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Numerical Study of the Kinematic Motions of a Solitary Wave Hitting an Inclined Plate

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Abstract. In this study, we numerically investigated the vortex patterns and free-surface deformations of a solitary wave interacting with a bottom-mounted plate at various inclinations in shallow water. Assuming two-dimensional, incompressible, and non-breaking conditions, we considered a viscous fluid based on the streamfunction-vorticity algorithm used in Navier–Stokes equations. We solved the governing equations by finite analysis, and used an averaged two-time step method to consider nonlinear conditions on the free surface. To fit the irregular boundaries and reveal the fine details of the vortex phenomenon, we applied a transient boundary-fitted grid along with a local-grid-refinement technique and examined the vortex mechanics around the submerged barrier. We then compared the numerical solutions obtained for the flow patterns with existing experimental observations to ensure the efficiency and accuracy of the numerical model. In addition, we systematically simulated other cases using this model to explore kinematic phenomena such as streamlines, equi-vorticity lines, streak lines, timelines, and path lines.

1. Introduction
The interaction of a linear wave with an infinitely thin vertical plate has been studied by two research groups, i.e., Porter and Evans [1] for a single plate and Evans and Morris [2] for multiple tandem plates. This scenario differs from those that extend vertically upward. Thus, the response of a wave that encounters an inclined plate is another interesting problem worthy of investigation. Along the coast, the side surfaces of breakwaters frequently encounter wave motions that are tilted. Another example is that of the special coastal flood protection facility in Venice, Italy known as the MOSE system [3], which can be rotated from a submerged bottom position into an inclined angle above water-level to prevent rising water from entering the city. Furthermore, breakwaters with the proper inclined angle can be more effective in attenuating incoming waves. Rao et al. [4] experimentally studied the influence of a submerged inclined plate on wave transmission. The authors found that a plate-type breakwater with inclinations of +60° and −60° (using 0° as the reference vertical position) can reduce, for a large range of relative depths, the monochromatic wave height by about 40%. Recently, Koutandos and Karambas [5] investigated the interactions between waves and a partially immersed vertical/inclined thin breakwater and concluded that the lowest transmission coefficient mostly occurred when the plate was either at an inclination angle of −30° or +30°, rather than a 0° inclination. From the above studies, we know that inclination is a vital design factor in the effectiveness of a breakwater.
Some researchers have studied the encounter of a solitary wave with a thin vertical plate. For example, using the edge-layer theory, Sugimoto et al. [6] investigated the reflection and transmission of a solitary wave passing over a plate with no thickness. Later, this theory was extended by Jeffery and Ramollo [7] in their study of internal wave propagation. In addition to wave nonlinearity, the vortex shedding caused by the interaction of a solitary wave with structures has also been the focus of many studies (e.g., [8] [9] [10]). Considering a propagating solitary wave against plates, Liu and Al-Banaa [11] used the RANS (Reynolds-averaged Navier–Stokes) model to numerically study the problem of a solitary wave encountering a partially submerged vertical plate. These authors also performed particle image velocimetry (PIV) measurements to visualize wave-induced flow motions. A more detailed experimental investigation of solitary wave propagation over a bottom-mounted plate was conducted by Lin et al. [12], in which PIV measurements and a particle tracing technique were used to reveal the flow patterns. Recently, Jaf and Wang [13] adopted the Fourier integral method to derive analytical solutions for a solitary wave propagating over a submerged vertical plate and then performed experimental measurements to verify their solutions. The authors also examined the effect of breakwater height on the peaks of reflected and transmitted waves. However, studies of the effect of inclined barriers on approaching solitary waves have been very limited. Zaghian et al. [14] experimentally analyzed a solitary wave interacting with a tilted plate using the PIV method. In this paper, we extend Chang’s modeling approach [15] to study the interaction of a solitary wave with an inclined submerged rigid plate and investigate the influence of the inclination on the interaction. Our numerical results compare well with the experimental particle tracing visualizations. By numerical simulation, we identified and herein present more of the kinematics of the fluid motions, including the streamlines, equi-vorticity lines, streak lines, and path lines.

2. Governing equations and numerical method

In a viscous incompressible fluid, we applied the streamfunction-vorticity formulations of the Navier–Stokes equation to establish the governing equations of the flow field. The vorticity ($\Omega$) is defined as the curl of the velocity that reduces to a scalar for a two-dimensional flow, and the streamfunction ($\psi$) is defined to satisfy the following continuity equation:

$$\frac{\partial \Omega}{\partial t} + u \frac{\partial \Omega}{\partial x} + v \frac{\partial \Omega}{\partial y} = \nu \left( \frac{\partial^2 \Omega}{\partial x^2} + \frac{\partial^2 \Omega}{\partial y^2} \right)$$

(1)

$$\frac{\partial^2 \psi}{\partial x^2} + \frac{\partial^2 \psi}{\partial y^2} = -\Omega$$

(2)

Figure 1. Schematic diagram of the problem

Figure 1 shows a schematic diagram of the problem. If not stated otherwise, all variables are expressed in non-dimensional quantities that were normalized based on the reference length (still water depth, $H$), and reference velocity (linear-long-wave celerity, $\sqrt{gH}$).
As the incident wave condition, we impose the analytical solitary wave solutions of Grimshaw (1971) [16], with the initial height $A_0$ starting at $X_0$ and moving in the positive $x$ direction. The celerity $C$ and initial wave profile $\zeta_f$ are expressed as follows:

$$C = 1 + \frac{1}{2} A_0 - \frac{3}{4} A_0^2 + \frac{5}{8} A_0^3,$$

$$\zeta_f = A_0 \text{sech}^2 k \left[ 1 - \frac{3}{4} A_0 \text{tanh}^2 k + A_0 \left( \frac{5}{8} \text{tanh}^2 k - \frac{101}{80} \text{sech}^2 k \text{tanh}^2 k \right) \right]$$

where $k = \frac{2\pi}{x_0} \left( \frac{1}{2} A_0 + \frac{\text{H}}{2\text{H}} A_0^3 \right) (x - X_0)$. The initial free-surface boundary values for the streamfunction are $\psi_f = C \zeta_f$. The vorticity values for the whole computational domain are initially zero. Equations (1) and (2) can be solved using the initial and associated boundary conditions. We employ a transient curvilinear coordinate grid system to solve for the moving free surface. We transform the physical domain in space and time, expressed as $(x, y; t)$, into a computational domain corresponding to $(\xi, \eta; \tau)$, respectively. On the free surface $(\zeta)$, the dynamic condition assumes that the water-surface pressure is uniform with atmospheric pressure. This can be further derived from the Navier–Stokes equation and expressed as follows:

$$\psi_{\xi} (Jg_{12}^1) + \psi_{\eta} (Jg_{12}^2) + \psi_{\zeta} + \psi_{\tau} = \Omega \left[ (u-x_\xi \zeta_{\xi} + (v - \zeta_{\eta}) \zeta_{\eta}) - (v - \zeta_{\xi}) x_\xi \right]$$

$$+ \frac{J}{\text{Re}} \left( g^{12} \Omega_{\xi} + g^{22} \Omega_{\eta} \right) = 0$$

where $\text{Re}$ in equation. (5) is defined as $H \sqrt{\frac{gH}{\nu}}$ (here, $\nu$ is the kinematic viscosity coefficient), and

$$\psi_{\xi} = -\left( \frac{x_{\xi}}{J} \right), \psi_{\eta} = -\left( \frac{y_{\eta}}{J} \right), \psi_{\zeta} = \psi_{\tau} = \frac{x_{\xi} y_{\eta} + y_{\xi} x_{\eta}}{J}, \quad g^{11} = \left( \frac{x_{\xi}^2 + y_{\eta}^2}{J} \right), \quad g^{12} = \left( \frac{x_{\xi} y_{\eta} + y_{\xi} x_{\eta}}{J} \right), \quad g^{22} = \frac{x_{\xi}^2 + y_{\eta}^2}{J}, \quad f_1 = \left( Jg_{11}^1 \right) + \left( Jg_{12}^2 \right) / J, \quad f_2 = \left( Jg_{22}^2 \right) / J, \quad \text{and} \ J = x_{\xi} y_{\eta} - x_{\eta} y_{\xi}.$$

Another free-surface kinematic description yields the condition:

$$\psi_{\xi \zeta} + \psi_{\eta \zeta} x_\xi = x_\xi y_\eta.$$

Both the sea-bed bottom and bottom-mounted rigid plate, which are connected, are impermeable, therefore the streamfunction can be specified along the solid surface as $\psi_0 = 0$. The vorticity at the solid walls must satisfy the non-slip condition. Nalysamy (1986) [17] derived the formulation of wall vorticity for a Cartesian grid, which can be re-derived and expressed in a generalized curvilinear form as follows:

$$\Omega_0 = 2g^{22} (\psi_0 - \psi_1 + \psi_{0\eta}) - (g^{11} \psi_{0\xi} + 2g^{12} \psi_{0\eta} + f_1 \psi_{0\eta} + f_2 \psi_{0\eta}),$$

where the subscript “0” denotes a grid node on the wall and “1” is the node adjacent to “0.” The open boundary conditions are as follows:

$$\left. \Omega \right|_{\zeta} \pm \sqrt{1 + \zeta^2} \left. \partial_{\zeta} \right|_{x_\xi} = 0,$$

where $\Omega$ is a dummy variable that can represent $\psi$, $\Omega$, and $\zeta$; the “+” sign represents the downstream boundary, and the “−” sign the upstream boundary.

This numerical model employs a system comprising a boundary-fitted grid and a locally refined grid [15] to solve the governing equations by finite analysis (Chen and Chen, 1984 [18]). The reader is referred to these publications for details of the numerical methods we used to generate the grids and discretize the problem.

3. Results
First, we plotted the numerical streamlines and compared them with the experimental particle tracking visualizations. Figures 1 and 2 show cases with plate inclinations of $+60^\circ$ and $-60^\circ$, respectively, for a solitary wave with height $A_0 = 0.25, X_0 = -15$ (at $t = 0$), moving in the positive $x$ direction to pass over a thin inclined plate with dimensionless width, 0.06, and height $S = 0.5$ for $Re = 58,500$. In our numerical simulations, we ignored the influence of the plate width, and assumed a zero-thickness plate in our calculations. For the $+60^\circ$ plate (see figure 2), the positive tilt makes a stream-wise flow that produces a primary vortex behind the plate within the acute triangle zone. For the $-60^\circ$ case (figure 3), the flow is hindered and trapped in front of the plate, and a thinner primary vortex is attached to the lee side of the plate. Overall, the numerical results and experimental observations are in good qualitative agreement.

![Figure 2](image_url)

**Figure 2** Numerical streamline pattern (left) and experimental particle tracking visualization (right) of a solitary wave with $A_0 = 0.25$ encountering an inclined plate with a $+60^\circ$ angle and a height of 0.5.

![Figure 3](image_url)

**Figure 3** Numerical streamline pattern (left) and experimental particle tracking visualization (right) of a solitary wave with $A_0 = 0.25$ encountering an inclined plate with a $+60^\circ$ angle and a height of 0.5.

After validating the results of this numerical simulation, we applied them to calculate and compare the fluid motions of cases with $A_0 = 0.4, S = 0.5$ at three different inclined angles (i.e., $0^\circ$, $+30^\circ$, and $-30^\circ$). Vorticity is one of the dominant variables in this model. The vortex evolutions can reflect the intensity of the fluid-velocity gradients. Figure 4 shows the instant equi-vorticity lines at $t = 11$ and $t = 20$ for three different angles, respectively. At $t = 11$, we can see that all their flows separate from the plate tips to form vortex bubbles of different sizes. When the time reaches $t = 20$, we can see that intensive vorticities adhere to the right bottom corner and left tip in the case of $\theta = 0^\circ$. However, for
the case of $\theta = +30^\circ$, the vorticity is concentrated at the bottom bed of the right corner, and the vorticity of $\theta = -30^\circ$ is concentrated on the right-side tip of the plate.

**Figure 4** Vortical patterns at three different inclinations.

**Figure 5** Pathline patterns at $t = 50$ for three different inclinations.

Pathlines are trajectories of fluid particles in a flow over a certain period. The pathline is an integration of a particle’s velocity with time from the Lagrangian viewpoint, which is the model we employed to trace the particle movements. Figure 5 presents the pathlines of five particles initially positioned at $x = 0.0$ and $y = -0.4, -0.3, -0.2, -0.1,$ and $0.0$ and traced to $t = 50$. From the figure, we can see that the water particles from the same initial position will differ greatly in their movement trajectories with changes in the angle of the plate.
The streak line is the line of all fluid particles that have passed continuously through a particular point in space. In an unsteady flow, the streak line at one point in time may sway into a very different position at the next time point. To observe the evolution of the streak lines, we selected five points (x = 0; y = −0.4, −0.3, −0.2, −0.1, and 0) between the plate and the free surface at which to continuously release particles. Figure 6 shows the progression of the five streak lines, which are marked with different colours.

From these plots, we can see that at t = 20 the tracked streak lines move to very different positions in response to the plate angles. The streak particles at θ = −30° are strongly sucked into the main vortex. Timelines are the lines generated by a set of fluid particles from a given starting instant as the particles move. The changes in the shape of the timelines reflect the variations in the properties of the fluid with time. Prior to x = 0, we set five timelines at x = −1.193, −0.943, −0.693, −0.443, and −0.193, assigned them different colours, and traced their movements. We used 200 uniformly distributed virtual particles for each initial timeline. Figure 7 shows timeline plots of two selected instants (t = 11, and t = 15). At t = 11, the leading timeline starts to overshoot the plates at θ = 0° and θ = −30°, which indicates the initiation of the vortex. After that, subsequent timelines eventually merge into the circulation areas of the vortex. The θ = +30° case extends the overshoot further downstream, thus slowing the time in which these timelines are involved in the eddy current. However, the θ = +30° case causes these timelines to enter the period of vortex scrolling earlier.

Figure 6 Streakline patterns for three different inclinations
4. Conclusion
The results of this study identify the vortical motions of a solitary wave passing over a bottom-mounted plate set at different inclinations. We found that the inclination of a plate can significantly influence the flow-motion kinematics.

5. References
Acknowledgments

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Chapter 2: Marine Management and Port Engineering
Prediction of quantitative risk assessment and risk level criteria based on the data from oil tanker collision simulation at Kyauk Phyu Deep Sea Port, Myanmar

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Abstract. The rates of ship accidents in waterways have been massively increased by many reasons including ship to ship collision. Collisions of crude oil tankers threat the health, safety and marine environment to its related surroundings. This study presents the QRA for double hull crude oil tanker collision in Kyauk Phyu Deep Sea Port of Myanmar navigational waterway by following the FSA guide lines. In order to present the useful simplified risk assessment method, the risk is predicted including frequencies, consequences analysis with the virtual ship collision model which is constructed based on the local situations. Furthermore risk acceptance criteria are also illustrated by F-N, F-T and F-P curves. The maximum acceptable risk levels are also conducted for social risk (1*10⁻³ fatalities per year), environmental risk (0.7*10⁻³ spills per year) and property risk (4*10⁻¹ USD per year) respectively. Ultimately, this paper aims to develop the IMO’s FSA into simplified method and hopes to assist the local authorities’ decision making process in case of tanker incident occurs in this area.

1. Introduction
Nowadays, the delivering of world trade is mostly used by waterborne transportation so that it is the vital signs of global economy. At the same time, the safety of life, cargo and ships at sea is one of the most important factors for maritime industry. However, it has a numerous number of legislation and guidelines for safety, many accidental issues are still rarely happened yet including the oil spill by tanker accidents. As a result, not only they have severe damage consequences to the health safety of its related human and living things but also have huge economic consequences for local resident. According to ITOPF record, figure 1 below shows that the causes of oil spill (> 7tons) incidents global record in percentage from 1970 to 2017 [1]. From this 47 years record, it is obvious that the most frequent cause of oil pollution from tanker ships is the ship collision. There are numerous negative impacts to many sectors after causing the ship to ship collision incident. When the ship collision was occurred, firstly it makes adverse impact to its related environment as well as the marine wildlife. Likewise, it injuries and dies the crew and passengers which are an irrecoverable losses in all of the losses. Reviewing the previous studies, Dong Y and Frangpol [2] presented the probabilistic ship collision risk and sustainability assessment into economic, societal and environmental matrices with the probability distribution using Monte Carlo Simulation. This study also assessed the expected utility related to different risks interval in different attitudes. Youssef and Ince [3] developed a quantitative risk assessment of double hull oil tanker ship collision model which is collided with 30 different ships by using LS-DYNA nonlinear finite element method in order to predict the structural damages of each collision condition. And also estimated the environmental consequences in terms of monetary value.
and finally the total risk are assessed. Vidmar and Perkovic [4] studied the safety assessment of crude oil tanker. This study systematically analyzed the overall risk management state and discussed the risk acceptance criteria for three risks: potential loss of life (PLL), potential loss of containment (PLC) and potential loss of property (PLL) in recent decades with F-N curves for different tanker sizes. The main objective of this study is to analyze the ship collision frequency, consequences and risk for deep sea port of Myanmar waterway area and conduct the risk standard level for this area based on the IMO’s FSA guidelines.

Figure 1. Significant causes of oil spill (>7 tons) incidents in percentage from 1970 to 2017 [1].

2. Material and methods
During this few years, China formally introduced the “Belt and Road Initiative” across Eurasia to connect China with Europe, the Middle East, and South Asia. With this project, Myanmar is also one of the integral components of the 21st century Maritime Silk Road and the Silk Road Economic Belt. Kyauk Phyu special economic zone project is one of the demonstrable evidence by Chinese investments which is located in the Rakhine State, western part of Myanmar [5]. The Kyauk Phyu deep sea port project include oil terminal, already operated since 2013, which handles the maximum 300,000 dwt crude oil tankers from Middle East and transport to Yuan province in china by passing through the crude oil pipeline that avoids the Malacca Straits and it would save about 5000 km sailing distance for sea going vessels to China. The deep sea port project will consist of the container terminal with on total 10 berths. It is planning to accommodate the maximum 6000 TEU containers and bulk carriers. This port will be one of the biggest infrastructure projects in Myanmar’s history and would be beneficial for the country’s economic growth. On the other hands, it needs to consider negative impact to local people and environment such that oil spill in its navigational waterway due to unexpected conditions [6]. That is why this paper is based on this local situation data and then identifies the virtual hazardous scenario to determine the risk in this port. In this study, the beginning step is to identify the hazardous scenario by the FSA approach [7]. So, the four different oil tankers model will be initially established and then ship to ship collision model will be simulated in the LS-DYNA nonlinear finite different element. In the risk assessment study, the two data are needed to identify such as frequency and consequences. That is why the ship collision frequency calculation will be approached in the next step based on the local environmental and traffic input data of the study area by following the Pederson’s ship [8] model. Likewise, the ship collision consequences will also be derived in third step including environmental losses, property losses and societal losses in terms of monetary value. In order to calculate the economic losses, Monte Carlo simulation will be applied that taken 1000000 random number will be initialized and give the expected costs with the probability distribution for each matrices. Finally, F-N curve will be illustrated for three risks: PLL, PLC and PLP for each tanker class and the suitable risk acceptance standard will be conducted for the study area.

3. Collision hazardous identification
In this study, the four different double hull oil tankers namely as Panamax, Aframax, Suezmax and VLCC [9] are collided by the Container ship and Bulk carrier in each collision scenario. At the same time, the striking ships’ bow structure model will be chosen as 22328 dwt Container vessel and 24497 dwt Bulk Carrier respectively. By using these particulars of each vessel, the vessel hull form for each tanker and striking ship bow structure are created by the Solidwork engineering drawing software.
When the hull form is completed, each model are simulated by the LS-DYNA explicit dynamic from the ANSYS 14.0 software [9] in order to get the output of collision structural failure to inner and outer hull, penetration results and determine whether it will be spilled oil or not. In each simulation, the striking bow structures collide with 8.5 knots and 90° collision angle to struck ship oil tankers. The required input data for ship’s material and density are taken as program default values and the mesh size is set as 1.5m. The simulation output of Panamax and Container ship collision can be seen in the figure 2 which is selected from the 8 collision scenarios. From these collision simulations, the inner and outer hull of the all collision models are breached with a speed of 8.5k knots so that is why the damaged area for inner and outer shell can be estimated and simultaneously it can be determined that each scenario either oil spill happens or not. The oil leakage amount for marine pollution considers as 10% of oil volume which carried in damaged tank [10]. Table.1 lists the resulted damaged area and oil leakage amount for each accidental case.

<table>
<thead>
<tr>
<th>Struck ship</th>
<th>Striking Ship</th>
<th>Spill amt (ton)</th>
<th>Std Deviation</th>
<th>Damage Area (m²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Panamax</td>
<td>Container</td>
<td>608.703</td>
<td>108.03</td>
<td>1155</td>
</tr>
<tr>
<td>Panamax</td>
<td>Bulk</td>
<td>608.703</td>
<td>108.03</td>
<td>1330.5</td>
</tr>
<tr>
<td>Aframax</td>
<td>Container</td>
<td>934.985</td>
<td>189.9</td>
<td>679.5</td>
</tr>
<tr>
<td>Aframax</td>
<td>Bulk</td>
<td>934.985</td>
<td>189.9</td>
<td>618</td>
</tr>
<tr>
<td>Suezmax</td>
<td>Container</td>
<td>1297.644</td>
<td>257.5</td>
<td>631.5</td>
</tr>
<tr>
<td>Suezmax</td>
<td>Bulk</td>
<td>1297.644</td>
<td>257.5</td>
<td>471</td>
</tr>
<tr>
<td>VLCC</td>
<td>Container</td>
<td>1767.802</td>
<td>285.5</td>
<td>370.5</td>
</tr>
<tr>
<td>VLCC</td>
<td>Bulk</td>
<td>1767.802</td>
<td>285.5</td>
<td>354.5</td>
</tr>
</tbody>
</table>

4. Collision frequency analysis
Many researchers conducted the enormous models to compute the ship-ship collision accident frequency $F_{\text{collision}}$ [8,10,11]. It is obvious that ship collision accidents mostly occur in the crossing waterway area. Therefore, the Pedersen’s model [8] is applied in this study to determine number of possible ship-ship collisions ($N_a$) over the considered risk area. So, the ship-ship collision frequency can be calculated as:

$$F_{\text{collision}} = N_a \cdot P_c \quad (1)$$

The causation probability $P_c$ is the probability of accidents caused by human errors and technical faults. This parameter can be estimated from the accidental data from different locations and then extended to the required analyzed area. Another approach is to calculate the value of $P_c$ by analyzing the human mis-operation and external faults establishing with event tree. In this study, the first method...
is used to get the causation probability \( P_c \). Based on the collision statistics in Bay of Bangel waterways \([12]\) has estimated that for \( P_c \) value is 1.44E-04. According to the specific navigation condition of the representative ships in the port area and above model, the number of collision between various ships can be computed and then the ship-ship collision frequency of the representative ships in this port area can be obtained \([13]\).

Furthermore, another three kinds of frequencies namely oil spill frequency \( F_{q\text{spill}} \), hull damage frequency \( F_{q\text{hull}} \) and crew casualty frequency \( F_{q\text{fat}} \) are considered based on the collision accident frequency. At first, oil spill frequency is considered as the product of collision accident’s frequency and oil tanker spill probability in collision case. The probability of spill in collision is assumed as the value of 0.175 based on the report data from ITOPF, IMO, LMIS and MAIB such kinds of association \([14]\). As for the calculation of hull damage frequency, the hull breaching probability is initially derived by following the formula of hull damage penetration probability from MARPOL regulation 32 \([15]\) and it can be denoted as:

\[
P_{ST} = 1 - \left[ 0.749 + \left\{ 5 - 44.4 \left( \frac{y}{B_s} - 0.05 \right) \right\} \left( \frac{y}{B_s} - 0.05 \right) \right]
\]  

(2)

In here, \( P_{ST} \) means the probability of side damage penetration; \( y \) is the distance between two side shell and \( B_s \) denotes the ship’s breadth. The frequency of Ship’s crew fatalities is considered upon the COWI analysis \([16]\) and the expected number of crew fatalities \( N_{\text{fat}} \) rate during this analyzed year is 0.01 and then frequency of crew can be determined by equation (3). All of the estimated results are listed in the table 2.

\[
F_{q\text{fat}} = \frac{N_{\text{fat}}}{N_{\text{crew}}}
\]  

(3)

<table>
<thead>
<tr>
<th>Struck ship</th>
<th>( D_j )</th>
<th>( N_a )</th>
<th>( P_c )</th>
<th>( P_{ST} )</th>
<th>( T_{fleet} )</th>
<th>( F_{q\text{collision}} )</th>
<th>( F_{q\text{spill}} )</th>
<th>( F_{q\text{hull}} )</th>
<th>( F_{q\text{fat}} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Panamax</td>
<td>267.83</td>
<td>9.83E-03</td>
<td>1.44E-04</td>
<td>0.188</td>
<td>9786.81</td>
<td>1.42E-06</td>
<td>2.49E-07</td>
<td>2.66E-07</td>
<td>3.98E-04</td>
</tr>
<tr>
<td>Aframax</td>
<td>288.70</td>
<td>1.04E-02</td>
<td>1.44E-04</td>
<td>0.247</td>
<td>17580.49</td>
<td>1.49E-06</td>
<td>2.61E-07</td>
<td>3.69E-07</td>
<td>3.57E-04</td>
</tr>
<tr>
<td>Suezmax</td>
<td>314.68</td>
<td>1.16E-02</td>
<td>1.44E-04</td>
<td>0.241</td>
<td>10724.44</td>
<td>1.67E-06</td>
<td>2.92E-07</td>
<td>4.03E-07</td>
<td>3.13E-04</td>
</tr>
<tr>
<td>VLCC</td>
<td>373.68</td>
<td>1.47E-02</td>
<td>1.44E-04</td>
<td>0.213</td>
<td>16046.47</td>
<td>2.12E-06</td>
<td>3.71E-07</td>
<td>4.51E-07</td>
<td>3.13E-04</td>
</tr>
</tbody>
</table>

### 5. Collision Consequences Analysis

The consequences analysis is the compulsory part in quantitative risk assessment. The costs for time variant consequences are considered as the equation (4) in which \( FV \) defines the value at future time \( t \), \( PV \) means the value at present time \( n \) and the interest rate is assumed as 2%. In this section, the consequences costs are mainly considered as environmental, property and crew casualty losses which monetary values are further elaborated into environmental clean-up cost, ship repair cost, cargo oil loss, operation time loss, crew injury loss and fatality.

\[
FV = PV \times (1 + i)^{t-n}
\]  

(4)

Environmental damage costs \( C_{\text{ENV}} \) due to oil pollution can be determined based on the amount of spilled oil volume for impacted area clean-up and compensation \([17]\) which can be calculated as:

\[
C_{\text{ENV}} = f_{\text{env}} \times f_{\text{spill}} \times 42301Q_{\text{spill}}^{0.7233}
\]  

(5)

Where, \( Q_{\text{spill}} \) = amount of spilled oil in ton, \( f_{\text{env}} = \) environmental modelling factor (mean 1 and COV 0.2) and \( f_{\text{spill}} = \) oil spill modelling factor (mean 0.3 and COV 0.2) \([18]\). Property damage costs are considered based on the three situations such as ship repair cost \( C_{\text{REP}} \), oil cargo losses \( C_{\text{CARGO}} \) and down time losses \( C_{\text{TIME}} \). The approximate ship repair times compared to the damaged area are followed by the COWI analysis \([16]\).

\[
C_{\text{REP}} = f_{\text{rep}} \times A \times c_{\text{rep}}
\]  

(6)

\[
C_{\text{CARGO}} = f_{\text{cargo}} \times f_{\text{spill}} \times Q_{\text{spill}} \times c_{\text{oil}}
\]  

(7)

\[
C_{\text{TIME}} = f_{\text{time}} \times c_{\text{time}} \times Dt
\]  

(8)
From the above equations, \( A \) = struck ship damaged area, \( c_{\text{rep}} \) = ship repair cost per area, \( c_{\text{oil}} \) = oil price per ton, \( c_{\text{time}} \) = ship operation cost per day, \( Dt \) = down time, \( f_{\text{rep}}, f_{\text{cargo}}, f_{\text{time}} \) = modelling factor with mean 1 and COV 0.2. Every collision accidents, the cost of crew’s fatalities \( C_{\text{FAT}} \) and injury losses \( C_{\text{INJ}} \) should be mainly considered as one part by the equation (9) and equation (10). The input values for fatalities modelling factor \( f_{\text{fat}} \) and injury factor \( f_{\text{inj}} \) are assumed as the same value of the above modelling factor. Then, the expected number of crew fatalities \( N_{\text{fat}} \) rate in collision case is taken as 0.387 and 5.413 for injuries \( N_{\text{inj}} \) respectively [19]. Value of life \( V_{\text{life}} \), injury cost per person \( c_{\text{inj}} \) and others input data are described in the table 4.

\[
C_{\text{FAT}} = f_{\text{fat}} \times N_{\text{fat}} \times V_{\text{life}}
\]

\[
C_{\text{INJ}} = f_{\text{inj}} \times N_{\text{inj}} \times c_{\text{inj}}
\]

\[
TEV = C_{\text{ENV}} + C_{\text{RE}} + C_{\text{CARGO}} + C_{\text{TIME}} + C_{\text{FAT}} + C_{\text{INJ}}
\]

Finally, the total economic value \( TEV \) for ship collision will be calculated as equation (11). And table 3 summarizes the input value which used in the consequences calculation and each economic damage costs are computed by using the method of Monte Carlo simulation. The costs of each risk in every activity can be determined by this simulation which supposed the probability distribution function and finally through one simulation can determine the costs deviation. This study used the Monte Carlo simulation lognormal distribution with 100000 random variables and runs the simulation into 50 iterations. The estimated consequences costs for each scenario are finally listed in the table 4.

### Table 3. The input value used in the economic losses calculation.

<table>
<thead>
<tr>
<th>Variables</th>
<th>Mean ($)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Value of statistical life ( V_{\text{life}} ) [12]</td>
<td>3000000</td>
</tr>
<tr>
<td>Injury costs per person ( c_{\text{inj}} ) [13]</td>
<td>50000</td>
</tr>
<tr>
<td>Repair cost per m(^2) ( c_{\text{rep}} ) [25]</td>
<td>216</td>
</tr>
<tr>
<td>Crude oil price per ton ( c_{\text{oil}} )</td>
<td>371</td>
</tr>
<tr>
<td>Daily cost per day ( c_{\text{time}} ) [5]</td>
<td>20068</td>
</tr>
<tr>
<td>Value of Panamax [12]</td>
<td>50000000</td>
</tr>
<tr>
<td>Value of Aframax [12]</td>
<td>65000000</td>
</tr>
<tr>
<td>Value of Suezmax [12]</td>
<td>85000000</td>
</tr>
<tr>
<td>Value of VLCC [12]</td>
<td>130000000</td>
</tr>
</tbody>
</table>

### Table 4. Results of collision consequences costs for each scenario.

<table>
<thead>
<tr>
<th></th>
<th>( C_{\text{ENV}} ) ($)</th>
<th>( C_{\text{RE}} ) ($)</th>
<th>( C_{\text{TIME}} ) ($)</th>
<th>( C_{\text{CARGO}} ) ($)</th>
<th>( C_{\text{INJ}} ) ($)</th>
<th>( C_{\text{FAT}} ) ($)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pan&amp;Con</td>
<td>1.41E+06</td>
<td>2.60E+05</td>
<td>4.20E+05</td>
<td>2.25E+05</td>
<td>2.70E+05</td>
<td>1.16E+06</td>
</tr>
<tr>
<td>Pan&amp;Bulk</td>
<td>1.41E+06</td>
<td>4.03E+05</td>
<td>4.20E+05</td>
<td>2.25E+05</td>
<td>2.70E+05</td>
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<td>Af&amp;C</td>
<td>1.92E+06</td>
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<td>3.47E+05</td>
<td>2.70E+05</td>
<td>1.16E+06</td>
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<tr>
<td>Af&amp;B</td>
<td>1.92E+06</td>
<td>1.78E+05</td>
<td>4.20E+05</td>
<td>3.47E+05</td>
<td>2.70E+05</td>
<td>1.16E+06</td>
</tr>
<tr>
<td>Sue&amp;C</td>
<td>2.44E+06</td>
<td>1.49E+05</td>
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<td>4.80E+05</td>
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<td>1.16E+06</td>
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<td>Sue&amp;B</td>
<td>2.44E+06</td>
<td>1.07E+05</td>
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<td>VLCC&amp;C</td>
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<td>4.20E+05</td>
<td>6.55E+05</td>
<td>2.70E+05</td>
<td>1.16E+06</td>
</tr>
</tbody>
</table>

### 6. Risk assessment and risk acceptance criteria

In this portion, the final risks for each of the accident are assessed by the product of accident frequency and consequences. Moreover, others potential such as Potential Loss of Life (PLL), Potential Loss of Cargo (PLC) and Potential Loss of Property (PLP) are also computed in this part. The results are listed in the table 5. The risk control option is one of the important parts in the risk assessment study so that this paper considers the risk outputs into negligible, acceptable and intolerable region according to the ALARP principle [20-22]. Initially, the societal risks are presented
by F-N curves which are the curves of the cumulative frequency (F) via number of crew fatalities and injuries of four oil tankers associated with the model incidents. Similarly, Frequency versus number of spill oil (F-T) and frequency versus loss of property in USD (F-P) plots are also illustrated to predict the level of risk for each different oil tanker. A benchmark FN criterion considers as the acceptable frequency (F) per ship year of accident exceeding N fatalities and it can be expressed as:

$$F(N) = F_A N^{-b}$$  \hspace{1cm} (12)

Here, The FN criterion slope (b) could be varying from different places with the range between -1 and -2. For the marine based risk criterion, the slope line is set to be -1 by following the IMO guideline [23]. The acceptable frequency ($F_{1A}$) per ship year for the intercept with N=1 axis and maximum number of fatalities in one accident $N_T$ is expressed as:

$$F_{1A} = \frac{PLL_A}{\sum_{N=1}^{N_T} N}$$  \hspace{1cm} (13)

<table>
<thead>
<tr>
<th>Struck ship</th>
<th>Striking Ship</th>
<th>Total risk ($/ship yr)</th>
<th>Potential Loss of Cargo (PLC) (ton/ship yr)</th>
<th>Potential Loss of Life (PLL) (fatalities/ship yr)</th>
<th>Potential Loss of Property (PLP) ($/ship yr)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Panamax</td>
<td>Container</td>
<td>5.3179</td>
<td>1.52E-04</td>
<td>2.31E-04</td>
<td>0.69E-01</td>
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<tr>
<td></td>
<td>Bulk</td>
<td>5.3804</td>
<td>1.52E-04</td>
<td>2.31E-04</td>
<td>1.07E-01</td>
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<tr>
<td>Aframax</td>
<td>Container</td>
<td>6.3906</td>
<td>2.44E-04</td>
<td>1.29E-04</td>
<td>0.64E-01</td>
</tr>
<tr>
<td></td>
<td>Bulk</td>
<td>6.4</td>
<td>2.44E-04</td>
<td>1.29E-04</td>
<td>0.66E-01</td>
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<tr>
<td>Suezmax</td>
<td>Container</td>
<td>8.2147</td>
<td>3.79E-04</td>
<td>2.11E-04</td>
<td>0.60E-01</td>
</tr>
<tr>
<td></td>
<td>Bulk</td>
<td>8.145</td>
<td>3.79E-04</td>
<td>2.11E-04</td>
<td>0.43E-01</td>
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<tr>
<td>VLCC</td>
<td>Container</td>
<td>13.791</td>
<td>6.55E-04</td>
<td>1.41E-04</td>
<td>4.29E-01</td>
</tr>
<tr>
<td></td>
<td>Bulk</td>
<td>13.606</td>
<td>6.55E-04</td>
<td>1.41E-04</td>
<td>3.89E-01</td>
</tr>
</tbody>
</table>

The value of potential loss of life $PLL_A$ are taken as the average value of four tankers simulation as $2.1*10^{-4}$. The maximum number of fatalities in collision accident is assumed as 6. And then, the $F_{1A}$ value is given an output result of $1*10^{-4}$. The next step is setting an ALARP border with upper and lower limits which centred on the acceptable frequency $F_{1A}$ benchmark [23]. The final risk criteria can be determined as:

- Upper criterion (intolerable risk) $F(N) > 10 F_A N^{-1}$
- Lower criterion (negligible risk) $F(N) < 0.1 F_A N^{-1}$

With the same approach for calculating of risk criteria for crew fatalities, oil pollution risk criteria are estimated with the mean value of potential loss of cargo oil PLC in collision accident is taken as $6.55*10^{-4}$, maximum number of spill ton in one accident is about 1200 ton and final output of $F_{2A}$ is $0.7*10^{-4}$. Similarly, the risk criteria for property loss can be computed as the above consideration. $F_{3A}$ value for property losses is computed as $4*10^{-2}$ with the mean potential loss of property PLP is $1.69*10^{-1}$ and maximum property losses is the value of $7*10^6$ USD respectively. The estimated criteria for the above three categories are drawn on the figure 3.

### 7. Conclusion

This paper performed a QRA method for crude oil tankers collision with other vessels in Kyauk Phyu deep sea port of Myanmar territorial water. Following the FSA procedure, the double hull oil tankers full scale model is collided with bow structure of bulk and container vessels. Then, LS-DYNA explicit dynamic method in ANSYS 14 software was applied to determine the hull breaching area and oil leakage amount. The estimated collision frequency $F_{q collision}$ in this model is slightly different with the IMO’s struck ship collision frequency because IMO refers only to the number of events between 1980 to 2007 period. The collision accident frequency for struck ships is gradually decreased in this recent decade. Most of the unit costs used in consequences analysis are based on the local exchange rates and economy rates and also input 100000 random variables. Thus it is the most suitable way to consider the consequences analysis. Later, the risk acceptance criteria was considered by following the ALARP...
principle for potential loss of life, potential loss of cargo and potential loss of property which were illustrated in F-N, F-T and F-P curve. In here, The estimated risk standard for societal and property losses are nearly same as the Vidmar’s and IMO results [4,7]. However, the environmental risk level is quite different. Since, the oil spill amount in Vidmar’s model is taken as the global data with the range between 2000 ~ 15000 ton and then these values are used in the PLC calculation which gives the result of 1543 ton/ship year. But, the PLC value is based on the model oil spill amount (10% of damage tank) so that the result is obviously smaller than the Vidmar’s model. As a further research and next step in this area it should be consider the cost benefit analysis to identify and compare the risk reduction. The cost benefit analysis was not considered in this study because some input data are inconvenient to collect for this research area within the short time duration. As a final goal, this study conducted the simplified quantitative collision risk assessment model and modified the FSA procedure in methodology. This study hopes to assist the future oil tanker collision risk assessment study and to become guidance for the prediction of risk in the Kyauk Phyu Deep Sea Port for local authorities.

8. References


[9] ANSYS Workbench release 10.0, 2005 ANSYS workbench products release notes for 10.0 ANSYS, Inc. and ANSYS Europe, ansysinfo@ansys.com


Acknowledgments
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Abstract. Biological phosphorus removal (BPR), possesses the significant advantages of low operational costs and little environmental impacts, is an economical and sustainable process to remove P by recycling the activated sludge through anaerobic and aerobic processes. For the BPR system, the hydraulic retention times (HRT) in aerobic and anaerobic processes are the most significant controlling parameters which can directly affect the effluent quality, especially the effluent phosphorus removal efficiencies. In this study, six lab-scale sequencing batch reactors (SBRs) were operated to conduct the single-factor experiments. 13 experimental runs designed by a 2-factor and 5-level response surface methodology (RSM) using Central composite design (CCD) were used to optimize the relationship between anaerobic HRT (X1, h) and aerobic HRT (X2, h) and two most important responses, COD removal efficiency (CRE, Y1, %) and P removal efficiency (PRE, Y2, %). High squared regression coefficients R2 (> 0.99) and adjusted R2 (> 0.99) indicated a high degree of correlation between the predicted and actual responses, which means that the model could fit the response well. Experimental validation by operating under the optimal combination of the two operational HRTs were conducted. Good correlation between the predicted and experiments values provides confidence in the obtained models.

1. Introduction

Biological phosphorus removal (BPR) is one of the most economical and sustainable processes to remove P from wastewater achieved by recirculating activated sludge through anaerobic and aerobic processes [1-3]. Although many satisfactory results have already been reported in lab- and pilot-experiments [4,5], its practical implementation still faces many challenges, both in scientific research and practical engineered application. The hydraulic retention times (HRT) in aerobic and anaerobic process are the most significant controlling parameters which can directly affect the wastewater treatment, especially the phosphorus removal efficiency. On the other hand, the design of HRT also imposes a significant effect on the infrastructure and operational costs in an engineered bioreactor [6]. However, in terms of the investigations from previous studies, the controlling of the HRTs in anaerobic and aerobic reactions changed widely. In consideration of the controlling aerobic and anaerobic HRTs in SBRs during BPR process, Long et al. [7] used a 4 h anaerobic HRT and a 7 h aerobic HRT with an influent chemical organic demand (COD) of 600 mg/L and P of 30 mg/L, the specific controlling conditions were 12 h per cycle: a 4 h anaerobic period, 7 h of aeration, a 50 min settlement, 5 min of decanting, and 5 min of idling. Zhang et al. [8] controlled a 2 h anaerobic HRT and a 170 min aerobic HRT with an influent COD of 214 mg/L and P of 20 mg/L. While almost
similar controlling strategies of 2 h anaerobic and 3 h aerobic period were conducted when an influent COD/P of 300 mg/L/12 mg/L were performed [9]. Furthermore, Yang et al. [10] used a 2 h anaerobic HRT and a 4 h aerobic HRT with different influential loadings: COD/P of 150 mg/L/6 mg/L; COD/P of 300 mg/L/13 mg/L; and COD/P of 400 mg/L/20 mg/L. From the literatures in the previous studies, it could be seen that the systematic analyses on why their anaerobic and aerobic HRTs in SBR are chosen and whether they are the optimal HRTs are not clearly stated in these papers. Thus, the controlling and optimization of the HRTs in aerobic and anaerobic processes are significant for mechanism study and engineering application.

The original single factor optimization method cannot account for the mutual effects of all the factors involved and requires a large number of experiments [11]. Response surface methodology (RSM) is a useful statistical technique for researching complex variable processes based on the Central composite design (CCD), which can effectively describe the interactions between independent experimental factors and response parameters [12]. By using a two-factor and five-level CCD, batch experiments on the study of the optimization of the HRTs in anaerobic and aerobic processes were conducted. The objective of this paper is to optimize the HRTs in different anaerobic and aerobic of the SBRs to achieve improved nutrient removal efficiencies during biological wastewater treatment process.

2. Material and Methods

2.1. Sludge cultivation and seed microorganisms

In this study, the applied bench-scale sequencing batch reactors (SBRs) are designed with a diameter of 15 cm, a height of 35 cm. The working volume of the SBR is 5.0 L. The schematic diagrams of SBR is shown in Fig. 1. Each SBR was inoculated to maintain a mixed liquor suspended solids (MLSS) of 4000 mg/L with activated sludge from a municipal wastewater treatment plant in Harbin. The temperature controlled in SBRs were 20.0 ± 0.5 °C. The constant temperature for the SBRs was maintained by a precision thermostatic bath circulator with the digital temperature controller (Ningbo Tianheng Instrument factory, Ningbo, China). Air was supplied from the bottom of the reactor by aerators and the dissolved oxygen (DO) concentration was maintained at 2-6 mg/L during aerobic phase. The DO concentration in each SBR was measured and controlled precisely by a DO probe with a DO industrial intelligent controller (SUP-DM2800, Hangzhou Sinomeasure automation Technology Co., LTD, China). Electric mechanical stirrer was performed to prevent sludge settling, which were run constantly except the time for settling, feeding and decanting: 40 min of sludge settling period, 15 min of decanting period, and 5 min of idling idle phase. The anaerobic period and aerobic period were performed in Table 1. Six identical SBRs with different anaerobic and aerobic HRTs during operation was operated to conduct the single-factor experiments. During operation, the pH was maintained at 7.50 ± 0.05 by two automatic titration units (SC-200A, ChangSha Sichen Instrument Technology Co., LTD, China) dosing 1 M HCl and NaOH to avoid the phosphate precipitation.

During operation period, the synthetic wastewater used as the influent of the tested SBRs were as follows: 256.4 mg/L sodium acetate (in chemical oxygen demand), 38.2 mg/L NH₄Cl, 21.95 mg/L KH₂PO₄, 40 mg/L CaCl₂, 75 mg/L MgSO₄. After about 2 months’ cultivation, the sludge characteristics and effluent concentrations in the SBRs were maintained in steady states. Each measurement was performed in triplicate.

Table 1 Operation and control parameters for the anaerobic-aerobic SBR systems

<table>
<thead>
<tr>
<th>Parameters</th>
<th>#1</th>
<th>#2</th>
<th>#3</th>
<th>#4</th>
<th>#5</th>
<th>#6</th>
</tr>
</thead>
<tbody>
<tr>
<td>Effective volume of SBRs (L)</td>
<td>5.00</td>
<td>5.00</td>
<td>5.00</td>
<td>5.00</td>
<td>5.00</td>
<td>5.00</td>
</tr>
<tr>
<td>DO in anaerobic tank (mg/L)</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td>–</td>
</tr>
<tr>
<td>DO in aerobic tank (mg/L)</td>
<td>2-6</td>
<td>2-6</td>
<td>2-6</td>
<td>2-6</td>
<td>2-6</td>
<td>2-6</td>
</tr>
<tr>
<td>HRT in anaerobic tank (h)</td>
<td>0.5</td>
<td>1.0</td>
<td>1.5</td>
<td>2.0</td>
<td>2.5</td>
<td>3.0</td>
</tr>
<tr>
<td>HRT in aerobic tank (h)</td>
<td>2.0</td>
<td>3.0</td>
<td>4.0</td>
<td>5.0</td>
<td>6.0</td>
<td>7.0</td>
</tr>
<tr>
<td>HRT in setting tank (h)</td>
<td>2.0</td>
<td>2.0</td>
<td>2.0</td>
<td>2.0</td>
<td>2.0</td>
<td>2.0</td>
</tr>
<tr>
<td>MLSS in the main reactor (mg/L)</td>
<td>4000</td>
<td>4000</td>
<td>4000</td>
<td>4000</td>
<td>4000</td>
<td>4000</td>
</tr>
</tbody>
</table>
2.2. Central composite design and statistical analysis

RSM is a mathematical and statistical technique used to analyse the mutual relationships between the response and the independent variables [13], which were widely used to optimize operating parameters for a system. Furthermore, this optimization method is expected to describe the entire effects of the selected parameters on the process [12]. The results obtained from the orthogonal experiments are therefore analysed using RSM. In this study, 13 experimental runs designed by a two-factor and five-level RSM using CCD were used to optimize the relationship between anaerobic HRT ($X_1$, h) and aerobic HRT ($X_2$, h) and the most important response, COD removal efficiency (CRE, $Y_1$, %) and P removal efficiency (PRE, $Y_2$, %). Eq. (1) showed the relationship between the uncoded and coded values:

$$ x_i = \frac{X_i - X'_i}{\Delta X_i} \quad (1) $$

The second-order polynomial model is presented in Eq. (2):

$$ Y = \beta_0 + \beta_1 x_1 + \beta_2 x_2 + \beta_{12} x_1 x_2 + \beta_{11} x_1^2 + \beta_{22} x_2^2 \quad (2) $$

where $x_i$ is the coded value of the independent variable; $X_i$ is the uncoded value of the independent variable; $X'_i$ is the uncoded value of the independent variable at the centre point; $\Delta X_i$ is the step change value; $Y$ is the response variable; $\beta_0$, $\beta_1$, $\beta_2$, $\beta_{12}$, $\beta_{11}$, and $\beta_{22}$ represent the regression coefficients from the experimental data.

<table>
<thead>
<tr>
<th>Independent variables</th>
<th>Low</th>
<th>High</th>
<th>-alpha</th>
<th>+alpha</th>
</tr>
</thead>
<tbody>
<tr>
<td>Anaerobic HRT (h), $x_1$</td>
<td>1</td>
<td>2</td>
<td>0.792893</td>
<td>2.20711</td>
</tr>
<tr>
<td>Aerobic HRT (h), $x_2$</td>
<td>3</td>
<td>5</td>
<td>2.58579</td>
<td>5.41421</td>
</tr>
</tbody>
</table>

Table 2: Experimental ranges and levels of the independent variables.
2.3. Analytical methods
During the operation period, the measurements of COD, the soluble chemical oxygen demand (SCOD), MLSS, mixed liquor volatile suspended solids (MLVSS), sludge volume index (SVI), total phosphorus (TP), $\text{PO}_4^{3-}$-P were measured in accordance with the standard methods [14]. DO and pH were measured by probes (Germany WTW Company pH/Oxi 340i main engine, pH meter, Germany). Before examination, the collected samples were filtered through 0.45 µm filters for analyses. Analyses of COD, SCOD, MLSS, MLVSS, SVI, TP, $\text{PO}_4^{3-}$-P were determined every day. All the experiments were performed in triplicate. All the experiments were conducted at room temperature.

3. Results and discussion

3.1. Analysis of single factor test results
Six lab-scale SBRs (Table 1) performed under 20 ± 0.5 °C generally reached steady-state after one-month inoculation. In order to provide a basis for the subsequent orthogonal experiments, single-factor experiments were used to study the optimal operating ranges of HRTs in anaerobic and aerobic processes.

The results of the single factor tests shown in Fig. 2a indicated that the CRE and PRE in effluents were altered by varying the anaerobic and aerobic HRTs in SBRs. In Fig. 2a, enhanced CRE and PRE were observed with the increases in anaerobic HRT from 0.5 h to 1.5 h, then decreased from 2.0 h to 3.0 h. For the anaerobic HRT in SBR, 1.5 h was considered to be the best anaerobic HRT to control. In Fig. 2b, results showed that enhanced CRE and PRE were showed with an increased aerobic HRT of 4 h. Results of this study demonstrated that the anaerobic and aerobic HRTs were indeed significant parameters in COD and P removal during BPR process. Results obtained in this study demonstrated that when the anaerobic HRT was controlled from 4.2 h to 4.8 h, and the aerobic HRT was controlled from 1.35-1.45 h, better COD removal efficiency could be obtained from an alternating anaerobic and aerobic SBR system. According to the observation, a higher HRTs controlled in the alternating anaerobic/aerobic SBR might deteriorate the effluent COD. This phenomenon might be induced by the production of soluble microbial products (SMP) and biomass-associated products (BAP), more BAP production had significant effects on higher COD concentrations in effluent [15].

![Figure 2](image)

Figure 2. Single factor test results of the CRE and PRE under different anaerobic HRT and aerobic HRT in SBRs

3.2. Optimization of operating variables and their reciprocal analysis
The design matrix and results obtained based on the experimental CCD design are showed in Table 3. Linear fitting of the predicted and the actual data (a) CRE (%) and (b) PRE (%) (Fig. 3) demonstrated that there is a good agreement between the predicted and the actual data
Figure 3. Linear fitting of the predicted and the actual data (a) CRE (%) and (b) PRE (%).

Table 3 Response surface CCD and experiments.

<table>
<thead>
<tr>
<th>Run</th>
<th>(X_1)</th>
<th>(X_2)</th>
<th>(Y_{\text{CRE}}) (%)</th>
<th>(Y_{\text{PRE}}) (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1.5</td>
<td>5.41421</td>
<td>93.92</td>
<td>97.23</td>
</tr>
<tr>
<td>2</td>
<td>1.5</td>
<td>4</td>
<td>94.34</td>
<td>97.53</td>
</tr>
<tr>
<td>3</td>
<td>1.5</td>
<td>4</td>
<td>94.34</td>
<td>97.53</td>
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<td>4</td>
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<td>5</td>
<td>2</td>
<td>3</td>
<td>91.23</td>
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<td>97.53</td>
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<td>7</td>
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<td>8</td>
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<td>93.21</td>
<td>96.23</td>
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<tr>
<td>10</td>
<td>1</td>
<td>5</td>
<td>94.32</td>
<td>94.44</td>
</tr>
<tr>
<td>11</td>
<td>2</td>
<td>5</td>
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<td>1</td>
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<td>91.21</td>
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<tr>
<td>13</td>
<td>1.5</td>
<td>4</td>
<td>94.34</td>
<td>97.53</td>
</tr>
</tbody>
</table>

Second-order polynomial models for coded responses of \(Y_{\text{CRE}}\) (CRE, %) were established as described in Eq. (3):

\[
Y_{\text{coded}} = 94.34 - 0.100x_1 + 1.44x_2 - 0.17x_1x_2 - 0.52x_1^2 - 1.25x_2^2
\]  
(3)

Second-order polynomial models for actual responses of \(Y_{\text{CRE}}\) (CRE, %) were established as described in Eq. (4):

\[
Y_{\text{actual}} = 62.26916 + 7.36503x_1 + 11.91052x_2 - 0.33500x_1x_2 - 2.07499x_1^2 - 1.24626x_2^2
\]  
(4)

Table 4 shows the analysis of variance for the experimental model equations to examine the significance and the adequacy of the second-order polynomial equation. \(P<0.0001\) indicated a high significance of the corresponding variable. A high squared regression coefficient, \(R^2\) of 0.9981 and adjusted \(R^2\) of 0.9967 indicated a high degree of correlation between the predicted and actual responses, indicating that the model could fit the response well. To represent the interaction between the independent variables and determine the optimal levels of each independent variable for observing the optimal response levels, the 2D contour curves and 3D response surface plots were depicted in Fig.
4. Results obtained in this study demonstrated that when the anaerobic HRT was controlled from 4.2 h to 4.8 h, and the aerobic HRT was controlled from 1.35-1.45 h, better COD removal efficiency could be obtained from an alternating anaerobic and aerobic SBR system. It could be observed that a higher HRTs controlled in SBR might deteriorate the effluent COD, which might induce by more BAP generated as the major effluent components at a long HRT and thus had significant effects on higher COD concentrations in effluent [15]. This conclusion was consistent with the single-factor experiments in this study.

Table 4 Analysis of variance (ANOVA) results for the response surface quadratic mode.

<table>
<thead>
<tr>
<th>Source</th>
<th>Statistics</th>
<th>df</th>
<th>Mean Square</th>
<th>F-value</th>
<th>P-value</th>
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<tr>
<td>Model</td>
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<td>5.69</td>
<td>719.61</td>
<td>&lt;0.0001</td>
</tr>
<tr>
<td>$x_1$</td>
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<td>0.080</td>
<td>10.11</td>
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<tr>
<td>$x_2$</td>
<td>16.54</td>
<td>1</td>
<td>16.54</td>
<td>2093.07</td>
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<tr>
<td>$x_1 \times x_2$</td>
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<td>1</td>
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<tr>
<td>$x_1^2$</td>
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<td>1.87</td>
<td>236.86</td>
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<td>$x_2^2$</td>
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<td>Pure Error</td>
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<td>0.000</td>
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<tr>
<td>Cor Total</td>
<td>28.49</td>
<td>12</td>
<td></td>
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</tr>
</tbody>
</table>

Figure 4. The effect of the anaerobic and aerobic HRTs and the response $Y_1$ (CRE, %): (a) 2D contour curves and (b) 3D response surface plots.

Second-order polynomial models for coded responses of $Y_2$ (PRE, %) were established as described in Eq. (5):

$$Y_{\text{coded}} = +97.53+1.39x_1+0.47x_2+0.16x_1x_2+1.69x_1^2-0.53x_2^2$$

(5)

Second-order polynomial models for actual responses of $Y_2$ (PRE, %) were established as described in Eq. (6):

$$Y_{\text{actual}} = +65.7825+24.43233x_1+5.20350x_2-0.33x_1x_2-6.77998x_1^2-0.530000x_2^2$$

(6)

The analysis of variance for the experimental model equations to examine the significance and the adequacy of the second-order polynomial equation was showed in Table 5. $P<0.0001$ demonstrated a high significance of the corresponding variable. $R^2$ of 0.9960 and adjusted $R^2$ of 0.9932 proved that the
high degree of correlation between the predicted and actual responses, indicating that the model could fit the response well. In Fig. 5, the contour 2D curves and the response surface 3D plot were depicted to represent the interaction between the independent variables and to determine the optimal levels of each independent variable for observing the optimal response levels.

Table 5 Analysis of variance (ANOVA) results for the response surface quadratic mode.

<table>
<thead>
<tr>
<th>Source</th>
<th>Sum of squares</th>
<th>df</th>
<th>Mean Square</th>
<th>F-value</th>
<th>P-value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Model</td>
<td>37.90</td>
<td>5</td>
<td>7.58</td>
<td>350.10</td>
<td>&lt; 0.0001 significant</td>
</tr>
<tr>
<td>$x_1$</td>
<td>15.37</td>
<td>1</td>
<td>15.37</td>
<td>710.05</td>
<td></td>
</tr>
<tr>
<td>$x_2$</td>
<td>1.76</td>
<td>1</td>
<td>1.76</td>
<td>81.10</td>
<td></td>
</tr>
<tr>
<td>$x_1 x_2$</td>
<td>0.11</td>
<td>1</td>
<td>0.11</td>
<td>5.03</td>
<td></td>
</tr>
<tr>
<td>$x_1^2$</td>
<td>19.99</td>
<td>1</td>
<td>19.99</td>
<td>923.16</td>
<td></td>
</tr>
<tr>
<td>$x_2^2$</td>
<td>1.95</td>
<td>1</td>
<td>1.95</td>
<td>90.26</td>
<td></td>
</tr>
<tr>
<td>Residual</td>
<td>0.15</td>
<td>7</td>
<td>0.022</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Lack of Fit</td>
<td>0.15</td>
<td>3</td>
<td>0.051</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Pure Error</td>
<td>0.000</td>
<td>4</td>
<td>0.000</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Cor Total</td>
<td>38.05</td>
<td>12</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Figure 5. The effect of the anaerobic and aerobic HRTs and the response $Y_2$ (PRE, %): (a) 2D contour curves and (b) 3D response surface plots.

3.3. Validation of the models

From Eqs. (4) and (5), the optimal actual values of $x_1$ and $x_2$ were determined to be 1.4 h and 4.6 h, and the maximum predicted COD was 94.78%; and a maximal PRE of 97.89% was obtained when the anaerobic HRT and aerobic HRT were controlled at 1.7 h and 4.4 h. To validate the optimal combination of the two operational HRTs, the optimal conditions predicted by RSM was used to test the predictive model. Triplicate tests under the optimized condition were conducted. Good correlation between the predicted and experiments values provides confidence in the obtained models.

4. Conclusion

In this study, 13 experimental runs designed by a 2-factor and 5-level RSM using CCD were conducted to design and study the optimal levels of the anaerobic HRT and aerobic HRT in an alternating anaerobic/aerobic SBR system. A good agreement between the predicted and the actual data were observed. For the established models, the second-order polynomial models for actual responses of $Y_1$ (CRE, %) and $Y_2$ (PRE, %) were obtained. Results demonstrated that when the anaerobic HRT was controlled from 4.2 to 4.8 h and the aerobic HRT was controlled from 1.35 to 1.45
h, better COD removal efficiency could be obtained from an alternating anaerobic and aerobic SBR system. A high HRT controlled in anaerobic and aerobic process might deteriorate the effluent COD due to more BAP production in effluent. High $R^2$ and adjusted $R^2 (>0.99)$ demonstrated that a high degree of correlation between the predicted and actual responses, which further indicated that the model could fit the response well. To validate the optimal combination of the two operational HRTs, the optimal conditions predicted by RSM was used to test the predictive model. Good correlation between the predicted and experiments values provides confidence in the obtained models.

5. References

Acknowledgments
The authors gratefully acknowledge the financial support by the National Nature Science Foundation of China (Grant No. 51708154), the State Key Laboratory of Urban Water Resource and Environment (Grant No. HC201621-01), the Key Laboratory of Research center for Eco-Environmental Science, Chinese Academy of Sciences (Grant No. kf2018002).
Evaluation of Tsunami Scouring on Subsea Pipelines

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Abstract. It is important that subsea pipelines are designed and maintained to withstand earthquakes and tsunamis, especially in earthquake-prone regions such as Japan. Tsunami scouring of the seabed may reduce the amount of soil covering a subsea pipeline, exposing it to harmful wave action. In this study, we investigated the feasibility of a subsea pipeline by calculating the tsunami scouring volume for representative tsunamis via numerical analysis. As a case study, we determined how subsea pipelines in the Kashima-nada sea area, Japan, would be affected. The results obtained indicate that for both a level 2 tsunami and a level 1 tsunami, scouring does not significantly affect the pipeline and no construction is subsequently required to restore the soil cover.

1. Introduction
Japan is in an earthquake-prone region; hence, it is vital that gas pipelines in this region are designed and maintained to withstand earthquakes. Japan has constructed onshore pipelines that are designed to be aseismic in the case of earthquakes and ground displacement due to liquefaction. However, subsea gas pipelines constructed in the open ocean must be designed to also withstand tsunami action. There are two types of subsea pipelines: (1) pipelines constructed in trenches and buried with backfill sand to protect against damage from anchors and wave forces and (2) pipelines placed directly on the seabed. Buried pipelines can be impacted by tsunamis in many ways however one of the most common impacts is pipeline exposure due to sediment scouring. Once the pipeline becomes exposed it is at risk of anchor strikes, and deformation and vibration damage due to wave action.

It is assumed that the extent of tsunami driven sediment scouring depends on the size of the tsunami; hence, the effects on subsea pipelines differ in the scale of the tsunami scour. Thus, it is estimated that the durability of the subsea pipeline differs according to the scale of the tsunami. Furthermore, the Japanese government has proposed that tsunami disaster prevention should change to reflect the scale of the threat (i.e., the potential size of the tsunami) that exists in Japan.

Therefore, in this study we examined two types of tsunamis, a large-scale low-frequency tsunami and a small-scale high-frequency tsunami, and calculated the depth of seabed sediment transported by each event. The effects of tsunami-driven scouring on subsea gas pipelines placed in the Ibaraki area were then determined.

2. Representative tsunamis and pipeline performance requirements
Currently, no standards or design guidelines exist, in Japan or abroad, for safety checks related to tsunami scouring of pipelines on the ocean floor. Therefore, to determine the effects of tsunami scouring on subsea pipelines, we referred to the earthquake resistance design guidelines for high-pressure gas pipelines on land [1] and a survey on tsunami disaster prevention in order to select representative tsunamis and propose performance requirements for subsea pipelines.
The current guidelines adopt a two-step design procedure, as shown in the Proposal on Earthquake Resistance for Civil Engineering Structures [2]. In this design, level 1 earthquake motion is defined as an earthquake event whose occurrence probability one to two times within the service period, and level 2 earthquake motion is defined as an earthquake whose occurrence probability is very low but that occurs close to a fault and impact is extremely strong. The steps of this method comprehensively determine pipeline performance requirements according to the importance of the structure and its influence at the time of the disaster. According to the earthquake resistance design guidelines for high-pressure gas pipelines, after a level 1 earthquake, the gas pipelines should not require repairs and should continue to function without problems. After a level 2 earthquake, the gas pipelines may deform but should not leak gas (Table 1).

For the representative tsunamis, proposals such as the "Special investigation meeting on earthquake and tsunami countermeasures based on the 2011 Tohoku tsunami lessons" [3] are referred to in the design of the structural and evacuation plan. In the investigation, tsunamis are divided into two classes: large-scale tsunamis (level 2) that occur with very low frequency but can cause an extremely large amount of damage and small-scale tsunamis (level 1) that occur with high frequency but do not produce high wave levels or cause excessive damage. The structural performance requirements are determined for each tsunami type individually.

Thus, based on these earthquake resistance design guidelines for high-pressure gas pipelines and recommendations on tsunami disaster prevention, in this study, we determined the pipeline performance requirements and selected representative tsunamis for evaluation as shown in Table 2. Because the pipelines are installed in a trench on the ocean floor to minimize the risk of damage due to ship anchors and fishery activities, the performance requirements for the level 1 tsunami specify that the soil covering the pipeline should not necessitate restoration after scouring. For a level 2 tsunami, the performance requirement is that the pipeline should not be exposed by tsunami scouring. Although leakage will not occur immediately if the pipeline is exposed after a level 2 tsunami, harmful deformation may occur owing to the tsunami wave force, therefore, we determine these conditions to ensure pipeline safety. The representative tsunamis used in this study are detailed in Sections 5.1.2 and 5.2.2.

### Table 1. Performance requirements of a pipeline based on the earthquake resistance design guidelines for high-pressure gas pipelines.

<table>
<thead>
<tr>
<th>Representative earthquake</th>
<th>Performance requirement</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Level 1 earthquake</strong></td>
<td>Probability of occurrence 1–2 times within the service period</td>
</tr>
<tr>
<td></td>
<td>Gas pipelines should not require repair and should function without problems</td>
</tr>
<tr>
<td><strong>Level 2 earthquake</strong></td>
<td>Very low probability but occurs close to the fault and has an extremely strong impact</td>
</tr>
<tr>
<td></td>
<td>Gas pipelines may deform but may not leak gas</td>
</tr>
</tbody>
</table>

### Table 2. Performance requirements of a pipeline and tsunami levels determined in this study.

<table>
<thead>
<tr>
<th>Representative tsunami</th>
<th>Performance requirement</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Level 1 tsunami</strong></td>
<td>High frequency of occurrence but does not produce high wave levels</td>
</tr>
<tr>
<td></td>
<td>No work needed to restore the soil cover after scouring</td>
</tr>
<tr>
<td><strong>Level 2 tsunami</strong></td>
<td>Very low frequency of occurrence but could cause excessive damage</td>
</tr>
<tr>
<td></td>
<td>Pipelines should not be exposed after tsunami scouring</td>
</tr>
</tbody>
</table>

3. **Overview of the numerical model for tsunami sediment transport**

In this study, we used a numerical model presented by Takahashi et al. [4] to gage sediment transport due to tsunami. The model was developed to reproduce the changes in the seabed observed at Kesennuma bay, Japan, based on the Chilean tsunami of 1960 (Figure 1).
As shown in Figure 2, we first calculated the tsunami height and then the sediment transport using the tsunami height. By repeating this procedure for each analysis step, the time series for tsunami sediment scouring was estimated.

Tsunami flux flow calculation is a general method for calculating the differential nonlinear long-wave motion equation composed of the continuity equation (equation (1)) and the momentum conservation equation (equation (2)): 

\[ \frac{\partial \eta}{\partial t} + \frac{\partial M}{\partial x} + \frac{\partial N}{\partial y} = 0 \] 

(1)

\[ \frac{\partial M}{\partial t} + \frac{\partial}{\partial x} \left( \frac{M^2}{D} \right) + \frac{\partial}{\partial y} \left( \frac{MN}{D} \right) + gD \frac{\partial \eta}{\partial x} + \frac{gN^2}{D} M \sqrt{M^2 + N^2} = 0 \]

\[ \frac{\partial N}{\partial t} + \frac{\partial}{\partial x} \left( \frac{MN}{D} \right) + \frac{\partial}{\partial y} \left( \frac{N^2}{D} \right) + gD \frac{\partial \eta}{\partial y} + \frac{gN^1}{D} N \sqrt{M^2 + N^2} = 0 \]

(2)

where \( t \) is time, \( x \) and \( y \) are the horizontal position coordinates, \( D \) is the depth \( (D = h + \eta, \) where \( h \) is the seabed, and \( \eta \) is the sea surface), \( g \) is the gravitational acceleration, \( n \) is the Manning roughness coefficient, and \( M \) and \( N \) are the flow flux in the \( x \) and \( y \) directions, which are calculated by integrating the horizontal flow velocities \( u \) and \( v \) from the seabed to the sea surface. The sediment transport calculation is composed of the continuous equation of the bed load layer (equation (3)) and the continuous equation of the suspended load layer (equation (4)): 

\[ \frac{\partial Z}{\partial t} + \frac{1}{1 - \lambda} \left( \frac{\partial Q_x}{\partial x} + \frac{\partial Q_y}{\partial y} + w_{ex} \right) = 0 \] 

(3)

\[ \frac{\partial C_x M}{\partial x} + \frac{\partial C_y N}{\partial y} + w_{ex} + \frac{\partial C_x h_s}{\partial t} = 0 \] 

(4)

where \( Z \) is the bed level and \( \lambda \) is the porosity, \( Q_x \) and \( Q_y \) are the bed load rates in the \( x \) and \( y \) directions, \( C_s \) is the mean concentration of the suspended load, \( h_s \) is the depth of the load layer, and \( w_{ex} \) is the exchange load rate between the suspended and bed load layers are calculated using equations (5) and (6) obtained from the hydraulic experiment of Takahashi et al. [4]: 

\[ Q = 21 \sqrt{sgd} \tau_*^{3/2} \] 

\[ Q_x = \frac{MQ}{\sqrt{M^2 + N^2}} + \varepsilon_s |\frac{\partial h}{\partial x}|, \quad Q_y = \frac{NQ}{\sqrt{M^2 + N^2}} + \varepsilon_s |\frac{\partial h}{\partial y}| \] 

(5)

\[ w_{ex} = 0.012 \sqrt{sgd} \tau_*^{3/2} - \omega_s C_s \] 

(6)

where \( s \) is the specific gravity of water \( (= \rho / \sigma - 1): \rho \) is the sea water density and \( \sigma \) is the sand density, \( \tau_* \) is the critical Shields number, \( \varepsilon_s \) is the vertical diffusion coefficient, and \( w_s \) is the settling velocity. The Shields number is the non-dimensional rate of bed load transport limit (limit sweeping power), which is expressed by equation (7): 

\[ \tau_* = \frac{u_{*s}^2}{(\sigma / \rho - 1)gd} \] 

(7)

where \( u_{*s} \) is the critical friction velocity. In the case of sand, the critical Shields coefficient is 0.034–0.05 from Iwagaki et al. [5].
4. Validation of the numerical model and material parameters

4.1. Conditions

Previous studies [6] have confirmed that the numerical analysis method used in this study has adequate validity for determining sediment transport resulting from tsunamis. However, there is no established method for setting data related to the target area of the numerical analysis, such as, the boundary conditions, water level, and altitude; hence, it is necessary to confirm their validities beforehand. Therefore, we used the data from the 2011 Tohoku Tsunami to calculate tsunami propagation for the Kashima Nada area, and compared observation records and conditions to confirm the validity of the results obtained. However, because the changes in seabed topography during the 2011 Tohoku tsunami are still being analyzed, our calculation results for sediment transport were not compared with available observations.

As shown in Figure 3, the calculation of tsunami propagation involved a sea area including the Kashima-Nada measuring an area of 1,458 km \times 1,239 km. As shown in Table 3, terrain data such as water depth and altitude of the target area were the same as those adopted in the Ibaraki coastal tsunami flooded area study report [7]. The earthquake faulting of the Tohoku tsunami was the same as the model adopted by the Study Group on the Cabinet Office's Great Earthquake [8], and the initial vertical seabed displacement due to fault data and fault movement are shown in Table 4 and Figure 4, respectively.

As the calculation of tsunami propagation utilizes the difference method, it is necessary to divide the target region into computation lattices of appropriate sizes. In this study, the region was divided into a calculation lattice from a maximum of 2,160 m to a minimum of 40 m, considering the trade-off
between computation time and accuracy (Figure 3). Land side and offshore side boundary conditions were assumed to be under perfect reflection and free transmission conditions, respectively, and the tide level was set as T.P. - 0.52 m at the time of the occurrence of the Tohoku tsunami. The calculation interval was 0.2 s, and the calculation duration was 6 h from the fault movement.

**Figure 3.** Location map of the study area (showing entire area and nested calculation area).

**Figure 4.** Initial vertical seabed displacement for Tohoku earthquake.

**Table 3.** Material parameters for validation of the numerical model.

<table>
<thead>
<tr>
<th>Index</th>
<th>Calculation conditions</th>
</tr>
</thead>
<tbody>
<tr>
<td>Calculation method</td>
<td>Basic equation Model sediment transport from Takahashi et al. (1999) [4]</td>
</tr>
<tr>
<td>Difference method</td>
<td>Spatial difference: Staggered grid</td>
</tr>
<tr>
<td>Coordinate system</td>
<td>Temporal difference: Leapfrog integration</td>
</tr>
<tr>
<td>Calculation area</td>
<td>Plane Cartesian coordinate system No.9</td>
</tr>
<tr>
<td>Calculation mesh spacing</td>
<td>2,160 m→720 m→240 m→120 m→40 m</td>
</tr>
<tr>
<td>Number of meshes</td>
<td>1305×1899</td>
</tr>
<tr>
<td>Terrain condition</td>
<td>Terrain data</td>
</tr>
<tr>
<td></td>
<td>Based on the survey report from the central disaster prevention council and Ibaraki</td>
</tr>
<tr>
<td></td>
<td>coastal tsunami inundation area</td>
</tr>
<tr>
<td>Fault model</td>
<td>Tohoku Region Pacific Offshore Earthquake Cabinet Office model (2012) [8]</td>
</tr>
<tr>
<td>Ground displacement</td>
<td>Method of Okada (1992) [9]</td>
</tr>
<tr>
<td>Initial condition</td>
<td>Provide the above ground variation amount with respect to the sea level T.P.-0.52 m</td>
</tr>
<tr>
<td>Initial tide level</td>
<td>Free transmission condition</td>
</tr>
<tr>
<td>Boundary condition (coast side)</td>
<td>Perfect reflection condition</td>
</tr>
<tr>
<td>Boundary condition (land side)</td>
<td>Calculation time</td>
</tr>
<tr>
<td>Calculation time interval</td>
<td>6 h</td>
</tr>
<tr>
<td>Calculation time</td>
<td>0.2 s</td>
</tr>
</tbody>
</table>
### Table 4. Conditions of the Tohoku tsunami fault.

<table>
<thead>
<tr>
<th>Occurrence year</th>
<th>2011</th>
</tr>
</thead>
<tbody>
<tr>
<td>Magnitude</td>
<td>8.2</td>
</tr>
<tr>
<td>Epicenter</td>
<td></td>
</tr>
<tr>
<td>Latitude:</td>
<td>34.84</td>
</tr>
<tr>
<td>Longitude:</td>
<td>139.76</td>
</tr>
<tr>
<td>Number of fault elements</td>
<td>4,575</td>
</tr>
<tr>
<td>Fault area (km²)</td>
<td>119,974</td>
</tr>
<tr>
<td>Depth 0–10 km</td>
<td>6.50</td>
</tr>
<tr>
<td>Depth 10–16 km</td>
<td>13.81</td>
</tr>
<tr>
<td>Depth 16–32 km</td>
<td>11.81</td>
</tr>
<tr>
<td>Depth over 32 km</td>
<td>2.78</td>
</tr>
</tbody>
</table>

### 4.2. Results and comparison with observed tsunami height

Figure 5 shows the maximum wave height distribution over the entire calculation period of tsunami propagation. The maximum wave height is larger in coastal areas than in offshore areas by approximately 4 m. In addition, because the maximum wave height distribution cannot be explained only by the distance from the fault, located to the northeast of the calculation area, it is inferred that the water depth, altitude, shape of the coastline, etc., also have a large effect. Figure 6 shows the time histories of wave heights at the P1, P2, and P3 points along the coast. The period of the tsunami was approximately 30 min, and it was confirmed that a large amplitude tsunami approached several times.

The Tohoku tsunami Joint Study Group gathered a wide range of experts to contribute to the reconstruction plan of the disaster area, a future disaster prevention plan, and reconsideration of the tsunami disaster prevention plans for other tsunami prone areas [10]. Here, we compared our calculation results with the results of field surveys conducted by this group. Figure 7 compares the maximum calculated wave height with time and the trace height of the tsunami obtained by field survey for all 60 points along the coast. For some points, although the calculation result and field survey result showed a deviation of approximately 1.5 m, they generally showed the same pattern. For all 60 sites, we obtained the maximum tsunami height from both the calculation and field survey using the \( K \) and \( \kappa \) indices proposed by Aida et al. [11] as described in equations (8) and (9). We quantitatively evaluated the spatial fitness of the tsunami trace height:

\[
\log K = \frac{1}{n} \sum_{i=1}^{n} \log K_i
\]

\[
\log \kappa = \left[ \frac{1}{n} \left( \sum_{i=1}^{n} (\log K_i)^2 - n(\log K)^2 \right) \right]^{1/2}
\]

where \( n \) is the number of points where the trace height of the tsunami is compared, and \( K \) is the height of the tsunami trace obtained by the field survey for numerical analysis results at each site. The evaluation results using the index of Aida et al. are shown in Table 5. The geometric mean, \( K \), shows the average correspondence between the height of the tsunami trace height and the calculated value when it is closer to “1”; the calculated value thus corresponds well with the tsunami trace height. The geometric standard deviation shows the variation in correspondence between the tsunami trace and the calculated value; the smaller the geometric standard deviation, the better the calculated value corresponds to the tsunami trace height. \( K \) and \( \kappa \) are 1.1 and 1.3, respectively, and range from 0.8 to 1.2, which is generally considered to indicate adequate compatibility, as indicated in the guidance on
tsunami river run-up analysis [12]. Therefore, according to spatial fitness, the calculation result and the results of the field surveys correspond well. As shown above, the 2011 Tohoku tsunami propagation was calculated, and consistency between the calculation and observation results was confirmed; therefore, the target area of numerical analysis, boundary conditions, water level, and altitude data are considered to be valid.

![Image](image1.png)

**Figure 5.** Modeled maximum height for Tohoku tsunami.

![Image](image2.png)

**Figure 6.** Time histories of wave heights at the P1, P2, and P3 points along the coast. Contours and color map show seabed altitudes.

![Image](image3.png)

**Figure 7.** Comparison of recorded wave height and modeled wave height.

<table>
<thead>
<tr>
<th>Index</th>
<th>Result</th>
<th>Required range</th>
</tr>
</thead>
<tbody>
<tr>
<td>Geometric mean (K)</td>
<td>1.06</td>
<td>0.8&lt;K&lt;1.2</td>
</tr>
<tr>
<td>Geometric standard deviation (κ)</td>
<td>1.29</td>
<td>κ&lt;1.6</td>
</tr>
</tbody>
</table>

### 5. Analysis of tsunami scouring and pipeline evaluation

#### 5.1. Large-scale and low-frequency tsunamis (Level 2 tsunamis)

**5.1.1. Conditions.** The entire tsunami propagation area for the scouring calculation described in Section 4.2 would result in a large-scale numerical analysis; therefore, the calculation area is reduced with the smallest calculation lattice being 40 m, as shown in Figure 3. In addition, the input boundary condition is a tsunami level waveform at multiple points, and interpolation is performed between each point in a linear manner. The Manning roughness coefficient was 0.025 [13], the sediment grain size was 0.2 mm, porosity was 0.4 [14], and the saturated suspended sediment concentration was 1 % [15].
5.1.2. Tsunami selection and outline. Based on the Ibaraki Coastal Tsunami Inundation Forecast Area Report conducted by Ibaraki tsunami disaster prevention measure study, we selected the Tohoku tsunami and Empo-boso-oki tsunami as level 2 tsunamis. The Cabinet Office fault model [16] used in Section 4.1 was used for the Tohoku tsunami. This model for Empo-boso-oki tsunami, which describes the largest class of tsunami magnitude (Mt 8.6–9.0) proposed by the earthquake research promotion headquarters, was adjusted to 1.5 times the slip amount of the model. Table 6 and Figure 8 show the initial displacement amounts in the vertical direction of the seabed ground accompanying fault data and fault movement, respectively.

Table 6. Conditions of the Empo-boso-oki tsunami fault.

<table>
<thead>
<tr>
<th>Condition</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Occurrence year</td>
<td>1677</td>
</tr>
<tr>
<td>Magnitude</td>
<td>8.8</td>
</tr>
<tr>
<td>Epicenter</td>
<td>Latitude: 40.31, Longitude: 144.40</td>
</tr>
<tr>
<td>Number of fault elements</td>
<td>972</td>
</tr>
<tr>
<td>Fault area (km²)</td>
<td>26,118</td>
</tr>
<tr>
<td>Depth 0–10 km</td>
<td>5.00</td>
</tr>
<tr>
<td>Depth 10–16 km</td>
<td>5.42</td>
</tr>
<tr>
<td>Depth 16–32 km</td>
<td>7.04</td>
</tr>
<tr>
<td>Depth over 32 km</td>
<td>6.78</td>
</tr>
</tbody>
</table>

5.1.3. Model results. For the Tohoku tsunami and the Empo-boso-oki tsunami, the tsunami propagation calculations and sediment transport calculations were performed with time from the occurrence of the tsunami until convergence, revealing topographical changes due to the sediment movement. The Tohoku tsunami showed larger topographical changes (Figure 9). Approximately 0.5–0.6 m of sediment was transported from the coast to an offshore area of approximately 40 m depth 1 h after the first wave of the tsunami arrived. Thereafter, accumulation occurred in the scoured area over time and sand was gradually re-buried. The final topographical change in this area was approximately -0.1–0.3 m 6 h after the tsunami converged. At the periphery of the structure such as breakwaters in the coastal area, large amounts of sand were locally transported 1 h after the tsunami occurred and deposition began after another 3 h. This trend continued after 6 h, indicating a different behavior to that offshore. Figure 10 shows the maximum amount of scouring and deposition from tsunami occurrence up to 6 h after the tsunami.

Figure 11 shows the maximum amount of scouring and sedimentation for the Empo-boso-oki tsunami and the final topography changes 6 h after the tsunami. As with the Tohoku tsunami, large sediment scouring occurred from the coastal area to an offshore area of approximately 40 m depth, with a maximum scoured amount of 0.5 m. Local scouring and sedimentation in the surrounding areas were also confirmed in the periphery of the coastal area and with the same tendency as the Tohoku tsunami. As the majority of the areas where the subsea pipeline may be laid have water depths ranging from 20–40 m in general, the maximum scouring volume, amount of deposition, and final landform changes...
for the entire area are shown in Table 7. The maximum scouring amount for the Tohoku and Empo-
bose-oki tsunami was 0.6 m and 0.4 m, respectively. By assuming 1.8 m of sand cover, which is
commonly used when laying land-based high-pressure gas pipelines, we confirmed that the modeled
scouring amount was much lower than this, and would not expose the subsea pipeline.
Therefore, we confirmed that the performance requirements set in this study were met. Although more
scouring occurred in the periphery of the structures such as breakwaters in the coastal area than in the
offshore area, considering that the influence of sedimentation in the coastal area, would likely increase
the amount of sand covering, lay the pipeline can be laid avoiding these areas; thus, such large
scouring should not be problematic.

**Figure 9.** Temporal variation of modeled topographical change: 30 min, 1h, 2h, 3h, 4h, and
6h after Tohoku tsunami.

**Figure 10.** Modeled maximum net erosion and deposition for Tohoku tsunami. Contours are seabed altitudes.
Figure 11. Modeled maximum net erosion, deposition, and final topography change for Empo-boso-oki tsunami. Contours are seabed altitudes.

Table 7. Modeled maximum net erosion, deposition, and final topography change for level 2 tsunamis at shallower depths of 20 m (except around structures).

<table>
<thead>
<tr>
<th>Tsunami</th>
<th>Maximum scouring (m)</th>
<th>Maximum sediment deposition (m)</th>
<th>Final topography change (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tohoku</td>
<td>0.6</td>
<td>0.5</td>
<td>-0.1–0.3</td>
</tr>
<tr>
<td>Empo-boso-oki</td>
<td>0.5</td>
<td>0.4</td>
<td>-0.4–0.2</td>
</tr>
</tbody>
</table>

5.2. Small-scale and high-frequency tsunamis (Level 1 tsunamis)

5.2.1. Conditions. As described in Section 5.1, calculations were made for the smallest grid size of 40 m, as shown in Figure 3. The input boundary condition was a tsunami level waveform at multiple points, and interpolation was performed between each point by linear interpolation. The same values as in Section 5.1 were used for the roughness coefficient, particle diameter of the seabed, porosity, and saturated floating sand concentration.

5.2.2. Tsunami selection and outline. The Chilean earthquake tsunami, Genroku earthquake tsunami, Meiji sanriku earthquake tsunami, Miyagi-oki earthquake tsunami and Fukushima-east-oki earthquake tsunami were selected based on the Ibaraki Coast Tsunami Inundation Area Forecast. For the fault model, the Chilean tsunami used the Takaoka model [17], the Genroku tsunami used the Kasahara model [18], the Meiji sanriku tsunami used the Tanioka model [19], the Miyagi tsunami used the Aida model [20], and the Fukushima east-oki tsunami used the Abe model [21]. Figures 12 and 13 and Table 8 show the initial displacement amounts in the vertical direction of the seabed ground accompanying fault data and fault movement, respectively.

The Chilean tsunami was a distant tsunami that propagated over long distances from the Chilean coast to the Japanese coast. For this reason, it is necessary to calculate a tsunami propagation from the Chilean coast to the Japanese coast using a global model of the polar coordinate system and then calculate tsunami propagation of the planar orthogonal coordinate system covering Japan's ocean area [22]. In this study, the calculation in the plane orthogonal coordinate system was conducted using the tsunami propagation calculation results obtained from the global model employed in the Ibaraki Coastal Tsunami Inundation Area Forecast.
Table 8. Condition for each representative earthquake fault. (D is depth; \( \theta \) is strike angle; \( \delta \) is dip angle; \( \lambda \) is rake angle; L is length; W is width; U is slip amount; \( \mu \) is elastic modulus.)

<table>
<thead>
<tr>
<th>Tsunami name</th>
<th>Occurrence year</th>
<th>Magnitude</th>
<th>Epicenter Latitude</th>
<th>Epicenter Longitude</th>
<th>( D ) (km)</th>
<th>( \theta ) (deg)</th>
<th>( \delta ) (deg)</th>
<th>( \lambda ) (deg)</th>
<th>L (km)</th>
<th>W (km)</th>
<th>U (m)</th>
<th>( \mu ) (10^{11} \text{dyne/cm}^2)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Chile</td>
<td>1960</td>
<td>9.5</td>
<td>38.29</td>
<td>73.05</td>
<td>1</td>
<td>10</td>
<td>10</td>
<td>90</td>
<td>800</td>
<td>55</td>
<td>6.7</td>
<td>-</td>
</tr>
<tr>
<td>Genroku</td>
<td>1703</td>
<td>8.2</td>
<td>34.84</td>
<td>139.76</td>
<td>0</td>
<td>315</td>
<td>30</td>
<td>153</td>
<td>85</td>
<td>55</td>
<td>6.7</td>
<td>3.5</td>
</tr>
<tr>
<td>Meiji sanriku</td>
<td>1896</td>
<td>8.5</td>
<td>40.31</td>
<td>144.40</td>
<td>0</td>
<td>190</td>
<td>20</td>
<td>90</td>
<td>210</td>
<td>50</td>
<td>10.6</td>
<td>3.5</td>
</tr>
<tr>
<td>Miyagi oki</td>
<td>1978</td>
<td>7.4</td>
<td>38.39</td>
<td>142.37</td>
<td>25</td>
<td>190</td>
<td>20</td>
<td>76</td>
<td>26</td>
<td>65</td>
<td>2.0</td>
<td>7.0</td>
</tr>
<tr>
<td>Fukushima east oki</td>
<td>1938</td>
<td>7.5</td>
<td>36.93</td>
<td>142.05</td>
<td>20</td>
<td>200</td>
<td>10</td>
<td>95</td>
<td>100</td>
<td>60</td>
<td>2.3</td>
<td>5.0</td>
</tr>
</tbody>
</table>

Figure 12. Initial vertical seabed displacement for Chilean earthquake.

Figure 13. Initial vertical seabed displacement for each earthquake.

5.2.3. Model results. As in Section 5.2.2, tsunami propagation calculations and sediment transportation calculations were performed for the five tsunamis from tsunami occurrence until convergence after 6 h. Figure 14 shows the maximum scouring volume and Table 9 shows the maximum scouring volume, maximum deposition volume, and the final topographical change after 6 h. The maximum scouring volume of the Chilean tsunami was approximately 0.05 m in the coastal area at a depth of \( \leq 10 \) m and approximately 0.03 to 0.04 m in the area where the subsea pipeline may be laid at a depth of 20 m or more. The maximum scouring volume of the Genroku tsunami was 0.04–0.05 m at a water depth of \( \leq 10 \) m and 0.02–0.03 m at a water depth of \( \geq 20 \) m. The maximum scouring volume of the Meiji sanriku tsunami was approximately 0.05 m at a depth of \( \leq 10 \) m but \( \leq 0.01 \) m at a water depth of \( \geq 20 \) m. The maximum deposition volume was 0.02–0.03 m at a water depth of 20 m or more.
m, resulting in minimal scouring. The maximum scouring volume of the Miyagi oki tsunami was ≤ 0.01 m for the entire study area; almost no tsunami scouring occurred. The maximum scouring volume of the Fukushima east-oki tsunami was 0.01–0.02 m at a depth of ≤ 20 m, excluding the peripheral structure, indicating almost no tsunami scouring. Therefore, level 1 tsunamis resulted in maximum tsunami scouring of approximately 0.05 m. Thus, even when sand coverage of the subsea pipeline is reduced by tsunami scouring, it is sufficient for measures such as anchor protection and does not require restoration, thereby meeting the performance requirements of the pipeline.

Figure 14. Modeled maximum net erosion for each level 1 tsunami. Contours are seabed altitudes.

Table 9. Modeled maximum net erosion, deposition, and final topography change for level 1 tsunamis at shallower depths of 20 m (except around structures).

<table>
<thead>
<tr>
<th>Tsunami</th>
<th>Maximum scouring (m)</th>
<th>Maximum sediment deposition (m)</th>
<th>Final topography change (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Chile</td>
<td>0.03–0.04</td>
<td>0.03–0.04</td>
<td>0.03–0.04</td>
</tr>
<tr>
<td>Genroku</td>
<td>0.03</td>
<td>0.02–0.03</td>
<td>0.01–0.02</td>
</tr>
<tr>
<td>Meiji sanriku</td>
<td>≤ 0.01</td>
<td>≤ 0.01</td>
<td>≤ 0.01</td>
</tr>
<tr>
<td>Miyagi-oki</td>
<td>≤ 0.01</td>
<td>≤ 0.01</td>
<td>≤ 0.01</td>
</tr>
<tr>
<td>Fukushima east-oki</td>
<td>0.01–0.02</td>
<td>≤ 0.01</td>
<td>≤ 0.01</td>
</tr>
</tbody>
</table>
6. Conclusion
In this study, the influence of tsunami scouring caused by representative earthquakes for a proposed pipeline off the coast of Ibaraki prefecture was confirmed by numerical analysis. With regard to the assumed tsunami, we set two levels from the viewpoint of the scale of the tsunami and the frequency of occurrence, with reference to the guidelines that set the policies for tsunami countermeasures and high-pressure gas pipeline earthquake design guidelines. The results obtained by numerical analysis are as follows.
For a level 2 tsunami, although tsunami scouring occurs, the scouring volume is lower than the soil covering amount, so the subsea pipeline is not exposed, and it was confirmed that it meets the performance requirement set in this research. Further, for a level 1 tsunami, the tsunami scouring is at most 0.05 m, and even when the coverage of the subsea pipeline decreases due to the tsunami scouring, the soil covering necessary for measures such as anchoring from the ship is sufficient. We confirmed that it meets the performance requirement that it is secured and no construction is required to restore the soil cover after the earthquake.

7. References
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Thinking from a sea bauxite accident

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Abstract. In this paper, a typical case of transporting bauxite ore on the ship is studied. From the perspective of maritime safety, the bauxite is studied in the classification of goods in the IMSBC rules and the improvement measures in the current transportation process are proposed to provide the maritime transport, so as to provide reference for maritime transport and reduce the recurrence of such accidents.

1. Introduction
In recent years, with the rise of domestic industry demand and insufficient resources. The large import of bauxite has become a necessary trend. However, in the process of transporting the bauxite from the oversea, many accidents occurred.
On January 2nd, 2015, the JUPITER ship of the Bahamas flag bulk carrier, which was built in 2006 and occupied by GEARBULK. The ship loaded 46,400 tons of bauxite mine and was on its way from Southeast Asia to China and sank from the sea 150 nautical miles in Vung Tau, Vietnam. According to the maritime investigation and analysis, the ship carried out the requirements of such goods in strict accordance with the IMSBC rules [1]. And after the loading is completed, all processes are recorded in the logbook rigorously and carefully [2]. Leave the wharf without confirmation. When the ship is on the way to the voyage, 18 people were died and a major safety incident occurred in there.
After investigation, although the ship encountered heavy wind and waves in the coastal areas at that time, but there was no danger of overturning with the design and construction specifications of the ship, At least the shipwreck finally happened. What is the cause leads the navigation danger? The cause of the accident will be analyzed in detail below.

2. The accident analysis
Alumina bauxite is also known as aluminous soil or bauxite. It has high value to the industry. The ore composed of gibbsite, boehmite or diaspore, which is the main mineral making the aluminum products in China.
Since 2010, the domestic economy has developed rapidly and the demand for aluminum ore has increased greatly. Therefore, overseas imports have become an inevitable trend. Bauxite resources are widely distributed around the world, so domestic imports are diversified [3]. At the time of increasing imports of ore, there is a danger of shipping. After a preliminary understanding of the basic information, we started a comprehensive analysis to find that which one leads the question. It is dangerous to start a comprehensive analysis of which causes.

2.1. Human factors
Throughout the loading and transportation process, through the maritime investigation, it was learned that the ship's personnel were performing normal strength operations without fatigue. Therefore, it can be judged that the accident has nothing to do with human factors.
2.2. Ship management
Check the cargo management information provided by the ship.

2.2.1. Before loading
- The ship first trained the crew on the safety awareness of the transported goods, and arranged for the commissioner to review the dumping site of the ore, carefully inspect the ship's equipment and ensure that everything is normal.
- The owner of cargo has provided the ship with relevant information and proof of the goods at a considerable time in advance, and the data obtained by the ship has also been reviewed and approved by the authoritative official organization.
- According to the data, the ship has formulated a stowage plan in strict accordance with the cargo category in the IMSBC rules, and determined the loading sequence and the loading rate based on the damage strength of the ship.
- Control the source of supply. Accompanied by experienced tripartite personnel, the ship’s officer carried out a safe sampling of the loaded goods. The goods were completely carried out in accordance with the requirements of the IMSBC Rules. The sampling was carried out in three zones, three times, under the designated area. After the sampling is completed, the test shows that the goods are not contaminated by other substances and are basically consistent with the attributes of the goods described in the data. During the verification period, the weather was in good condition and the cargo was not affected by the weather. The verification is completed.

2.2.2. Loading period
During the loading period, the weather was good and there were no unforeseen circumstances, such as rain, storm and so on. According to the loading requirements, workers completed the loading task regularly and safely.
When the loading plan is about to be completed, the ship has carried out, such as trimming, a series of operations

2.2.3. Sea navigation
The shipment of the goods was completed. According to the weather forecast given by the gas guidance company, the ship planned a safe route, and was driven from Malaysia to the country under the driving of experienced senior crew.

Therefore, it can be judged that the accident has nothing to do with ship management.

2.3. Goods themselves attributes
Check out the current IMSBC rules as shown in Table 1:

<table>
<thead>
<tr>
<th>Table 1. Basic data of bauxite</th>
</tr>
</thead>
<tbody>
<tr>
<td>Angle of repose</td>
</tr>
<tr>
<td>Not applicable</td>
</tr>
<tr>
<td>Size</td>
</tr>
<tr>
<td>70% to 90% agglomerate 2.5mm to 500mm</td>
</tr>
</tbody>
</table>
Description: Brownish yellow clay and earthy minerals, water content: 0% to 10%, insoluble in water. As can be seen from the above, bauxite is defined in the IMSBC rules as a solid bulk cargo that is neither fluidizable nor chemical hazardous. During the shipment, no other requirements requiring special attention were given in the rules. This means that such goods can be transported safely as normal solid bulk cargo.

In the case of everything that seems to be safe, there was an accident. What caused the accident? In the process of analyzing the problem, we found two factors, we should take a caution about such circumstances:

2.3.1. Ore particle size

When domestic buyer purchase bauxite ore, the imported ore is marked with some particle size requirements in order to reduce the production cost of aluminum. Therefore, the goods are pretreated by the supplier before shipment, first, the machine remove particles and agglomerates exceeding 10 cm by coarse filtration. Then, the large particles are pulverized by the corresponding machine to make the particle size smaller. Therefore, when transported by barge to the cargo ship, the percentage of the size of the cargo powdery particles seen by the ship is likely to have exceeded the range defined by the particle size of Class C bauxite in the IMSBC Code, That is to say, the powder particles from 2.5mm to 500mm may be greater than thirty percent.

2.3.2. Ore moisture content

- The domestically imported bauxite is produced in Malaysia. The local climate is tropical rain forest climate. The air is humid all year round and the rainfall is huge. The production area has just experienced heavy rainfall during the monsoon season. This leads to an increase in the water content of the ore.
- The bauxite imported from Malaysia is mainly composed of red mud, which contains a large amount of iron oxide (about 60%). This material is highly fluid and may itself be in a liquid state.
- In order to reduce the environmental hazard caused by the dust of the goods and response to the needs of the government, the workers carry out high-pressure flushing when the goods were crushed, so that the water content of the goods was further increased. It can be judged from the above three points that when the goods are finally delivered to the cargo ship, the surface of the goods appears to be dry, but the inside actually contains more water, and even in some places, it is muddy. After testing by the authority, the moisture content of the goods has exceeded 10%, and some locations have reached about 12%.

3. The Accident Analysis Conclusion

According to the above analysis, the bauxite transported, in this voyage, from Malaysia to China has increased the content of fine granular ore during transportation and contains a certain amount of water. The goods as a whole have not met Class C goods in the IMSBC Code, and the goods at this time have shown a tendency to fluidize and thus have characteristics similar to those of Class A goods. At the same time, in the process of shipping, encountering wind and waves, small particles of minerals sinking and compacting (as shown in Figure 1), water produces free liquid so heavy that surface cargo develops flow [4]. Meanwhile, the decrease or loss of ship stability results in ship capsizing.

Figure 1 Schematic diagram of the flow of bauxite
Therefore, it can be inferred that the cause of this danger is that the cargo has a tendency to fluidize under certain circumstances. It is similar to class A cargo, that is, fluidization goods. As far as the above information is concerned, it should be recognized that the fine-grained cargo of any moisture content specified in the IMSBC Code is potentially mobile and therefore needs to be tested [5]. According to the current version of the IMSBC Code Appendix 3, paragraph 2.1.

Many fine-grained cargoes are easy to flow if they have a high water content. Therefore, wet goods containing a certain proportion of fine particles should be tested before loading.” At the same time, from November 9, 2011, the Ministry of Transport issued the “Regulations on the Safety Management of Fluidized Transporting Solid Bulk Goods”, which further clarified the definition of easy-to-fluid solid bulk cargo. The goods mentioned above are in full compliance with the standards defined for the easy-to-fluid cargo. This makes it clearer that if the goods have the conditions described above, the shipment management should be carried out in accordance with the shipping standards for the Class A goods.

4. The response plan

4.1. Measures for ship safe loading

The owner shall provide proof of the ship's cargo to the ship’s officers before loading the cargo, which shall include not only the information required by the IMSBC Code, but also a valid proof of the strength of the cargo granules, the exact composition of the cargo, and the moisture content of the cargo. The shipowner may encounter one of the following situations during the verification process and should treat it separately.

During the certificate verification process, the ore's water content shall not exceed 10%, and the rest of the requirements shall be in full compliance with the standards defined by the IMSBC Rules for Class C goods. If the final inspection is correct, the goods may be shipped in Class C cargo.

If the particle size and water content provided on the certificate are just at the edge of their definition of Class C goods, they are discussed in two cases.

If the cargo granularity meets the requirements, but the water content is higher than 10%, the owner shall be notified to exchange the goods, or the goods shall be air-dried, and the water content shall be reduced to within the standard before loading.

If the particle size does not meet the requirements, but the water content is less than 10%, according to the current version of the IMSBC Rules Section 1.3.1.1, Chapter VII, Section 7-5, paragraph 1.7.5.

"1.3.1.1 When it is assessed that the solid bulk cargo to be transported, as defined in 1.7 below, may have the hazard as defined in the Group A or Group B cargo, it is necessary to consult the port of discharge and the competent authority of the flag State. The three competent authorities will provide temporary and appropriate conditions for the carriage of the goods.

1.7.5 Fluidizable cargo means a cargo containing a certain proportion of fine particles and a certain amount of moisture. Such cargoes may be fluidized if they are shipped with water content exceeding their transportable moisture limit.”

The above goods have signs of suspected Class A goods. As described in the rules, temporary suitable conditions should be established for the goods before shipment.

If the data provided on the certificate is completely inconsistent with the definition of Class C goods in the IMSBC Code, it shall be shipped in accordance with the shipping standard of Class A goods. When the surveyors carry out the sampling test in the on-site, the way, such as CAN TEST approved in the rules [6] , hand kneading method, baking test and other methods, should detect the moisture content of the goods (in the test process must pay attention to the limitations of various test methods, and the above several detection methods can be used interchangeably, can also be used alone). If the test result is within the scope of service, the goods are loaded in Class A. If the test result is unqualified, that is, if the water content is too high, the goods are directly rejected.

Shipment operations may also be carried out when the relevant authority officially evaluates that the goods do not have the characteristics of Class A goods.
Even if the certificate provided meets the standard, the shipowner should carefully review the accuracy and timeliness of the certificate, the authenticity of the goods, and whether there is a means of renaming the goods to cover up its true nature, etc. whether there is a concealment or deception of the goods. If abnormal conditions are found, the goods should be refused and wait for the more authoritative inspection agency to complete the inspection before deliberation.

4.2. Management of the ship on the voyage

During the period of transporting, the ship should try to optimize the route and collect meteorological information in time to prevent the ship from tilting. Especially in Southeast Asia, tropical cyclones are more in summer and climate change is too fast. It is easy to cause danger of navigation. In case of emergency, there must be sufficient countermeasures [7]. If the conditions permit, the goods can be unpacked and aired, and the goods should be inspected regularly for safety.

5. Conclusion

In summary, the article analyzes the classification of bauxite in the IMSBC rules. It is concluded that bauxite should be transported in a manner that is easy to fluidize when it meets certain condition, which can reduce the similar sea transporting of dangerous accident.

6. References

Research on integrated power generation control system based on wind, rainwater and energy store battery

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Abstract: The exploitation and utilization of new energy is the current trend of the energy industry. By analyzing the characteristics of wind energy, rainwater energy, graphene and energy store battery, an overall scheme of wind, rainwater and energy store integrated power generation system for urban buildings is designed, so is its control system. The design of control system includes hardware design and software design. By this control system, it achieves the goal of charge control and discharge control. This system takes the microcontroller as the data acquisition and controller, the data which has processed is displayed in the screen, by this SX1278 wireless communication module, and the data is transmitted to the computer.

1. Introduction
With the energy crisis caused by the intense consumption of fossil energy in the world and the fashionable of the low-carbon economy, it is extremely urgent to find alternative energy sources and develop clean energy. At present, the energy supply and demand gap is constantly increasing in China, making its external dependence increasingly higher, which has become a factor that threatens national security [1]. Under the current increasingly severe pressure on energy conservation and emission reduction, accelerating the development of renewable energy has become a top priority in China [2]. At the same time, water pollution is serious, resulting in significant degradation of water ecology, water quality is generally deteriorating, various sudden water pollution incidents occur frequently, and water shortages are becoming increasingly prominent.

At present, the single power generation mode of wind energy or rain energy has been extensively researched and developed. However, wind or rain power generation is greatly affected by the environment, and it is extremely unstable compared to the traditional power generation method. The top of the city building has rich wind resources; furthermore the top of the building can collect a lot of...
rainwater for hydropower generation. Graphene has a unique property as a new type of nanomaterial that graphene can generate electricity when it interacts with rainwater. In view of the shortcomings of the single power generation mode, this paper proposes an efficient integrated power generation system based on wind, rainwater and energy store battery, which suitable for urban buildings by the research of wind power generation technology, hydropower generation technology, energy storage technology and graphene power generation technology. Due to the difference in power generation modes, it is necessary to design an effective control system to realize the comprehensive operation, energy storage and grid connection of the integrated power generation system [3]. In this paper, by the research of the integrated power generation system, a control system that based on single-chip microcomputer is proposed to ensure the normal operation of the integrated power generation system.

2. The integrated power generation system based on wind, rainwater and energy store battery

At present, wind power generation technology is relatively mature, while rainwater power generation is mostly a method of hydroelectric power generation after rainwater is collected. Rainwater has a certain kinetic energy when it falls from a high altitude. In order to make full use of wind energy and rain energy, wind power generation is mainly used, and a rainwater collecting tank is arranged on the wind turbine blade to constitute a wind turbine blade of the wind-rain linkage generator, thereby generating electricity. It has been found that rainwater can generate electricity when it flows through the surface of graphene film or through the three-dimensional structure prepared by graphene oxide [4-6].

By the in-depth research of wind power, rain energy, and graphene respective power generation technologies and energy storage technologies, a integrated power generation system which consists of wind energy, rainwater energy and energy storage fits for urban buildings are proposed. The working principle diagram of the integrated power generation system is shown in Figure 1.
Figure 1. Working principle diagram of integrated power generation system based on wind, rainwater and energy store battery.

The working principle of this power generation system:

1) Small wind turbines are placed on the top of urban buildings. They can take advantage of the abundant wind energy at the top of the buildings to generate wind power in rainless and windy weather.

2) In the rainy weather, the huge roof at the top of the city building can collect a large amount of rainwater, collect and filter the rainwater, then use the tubular hydroelectric generator arranged in the drainage pipeline to generate hydropower.

3) The rainwater collecting trough is covered on one side of the wind turbine blade, and the other side is still smooth, which constitutes the wind turbine blade of the wind-rain linkage generator. In the weather of wind and rain, the wind turbine blades of the wind-rain linkage generator can collect the flushing rainwater on one side, and apply the potential energy and kinetic energy of the rainwater to the wind turbine blades, so that the wind turbine blades rotate and convert into the mechanical energy of the wind turbine blades. The adjustment mechanism is adjusted so that the rotation direction of the wind turbine blade is the same as the rotation direction when the wind force acts alone. In this way, the kinetic energy and potential energy of the wind energy and the rain energy are converted into the mechanical energy of the wind turbine blade, and then converted into electrical energy.

4) The use of graphene materials with rainwater to generate electrical energy can be used for secondary power generation. A graphene film is plated on the smooth surface of the wind turbine blade of the wind-rain linkage generator to generate electrical energy. A sandwich layer structure that containing graphene oxide sheets is arranged in the drainage pipe for the urban buildings rainwater, the purified rainwater flows through the sandwich layer structure then generate electrical energy.

5) The converted electrical energy is processed and stored in a battery. It can convert into chemical energy. The battery can be connected to an electrical device or connected to a step-up and grid-connected circuit. Convert chemical energy into electrical energy according to electricity demand, directly use or connect to the grid by the DC/AC converter [7].

The rainwater flowing out of the sandwich layer structure enters the hydroelectric generator, at last the rainwater enters the reservoir after power generation, which can meet the needs of urban non-potable water, such as greening, air dust removal or car washing. The innovation of this power generation system is that it not only utilizes natural wind and rainwater to generate electricity, but also reduces the sulfur cycle to protect the environment, and solves the water shortage problem to some extent.

Wind power generation, hydropower generation and graphene power generation are the main sources of this electricity, converting wind and rain energy into electrical energy. The controller mainly regulates the charging control of the battery by the power generating device. The computer collects and monitors the information, and detects the running state of each unit to switch the switch and control the field device. When the input power is detected to meet the requirements of the powered device, it can be directly used by the powered device or stored in the battery. The inverter mainly provides a stable and reliable working voltage for the AC load. The surplus power can be stepped-up and connected to the grid where power is needed to create social value. When the power generated by the power generation group is insufficient, the power is supplied by the battery to satisfy the normal
use of the power device. The integrated power generation system framework diagram is shown in Figure 2.

![Diagram of integrated power generation system](image)

1-Wind Power Current; 2-Hydroelectric current; 3-Rainwater-graphene generation current; 4-Control charging current; 5-Rainwater-graphene charging current; 6-Battery voltage; 7-Battery output voltage; 8-Inverter output voltage;

**Figure 2.** Frames of integrated power generation system based on wind, rainwater and energy store battery.

### 3. The design of control system

In the integrated power generation system based on wind, rainwater and energy store battery, the core is the control system, and the normal operation process of the power generation system requires the control system to adjust. The main functions realized by the control system are charge or discharge control of the battery and the control of the wind power generation unit, the hydropower unit, and the rainwater-graphene power generation group.

#### 3.1 The design of hardware in control system

The MCU has rich instructions, flexible software programming, good accuracy and real-time performance, small control system, low power consumption, and easy data acquisition and control functions which by the use of the expansion of peripheral circuits [8]. The control system uses a single-chip microcomputer as a control chip, and realizes charging control and discharge control of the system by combining software programming methods. Considering the characteristics of the power supply object, the AVR microcontroller mega16 is selected as the control core of the system. The core of AVR has a rich instruction set and 32 general purpose working registers. All registers are directly connected to the arithmetic logic unit (ALU), allowing one instruction to simultaneously access two independent registers in one clock cycle. This architecture greatly increases the efficiency of code and has up to 10 times the data throughput rate of a typical CISC microcontroller. The ATmega16 is a low power 8-bit CMOS microcontroller based on an enhanced AVR RISC architecture. Thanks to its advanced instruction set and single clock cycle instruction execution time, ATmega16's data throughput is as high as 1 MIPS/MHz, which can alleviate the contradiction between power consumption and processing speed. The pins of the ATmega16 are shown in Figure 3.
Figure 3. Pin diagram of ATmega16.

Among them, port A (PA7...PA0) is the analog input of the A/D converter. Port B (PB7...PB0) is an 8-bit bidirectional I/O port with a programmable internal pull-up resistor. Port D (PD7...PD0) is an 8-bit bidirectional I/O port with a programmable internal pull-up resistor.

In this control system, the system analog input has 8 channels of battery voltage, wind power current, hydroelectric current, rainwater-graphene generation current and other currents. The digital output includes 8 channels of battery overcharge, over discharge protection and system working status indication. The control system structure is shown in Figure 4.

Figure 4. Control system structure.

In order to ensure the normal operation of the system, each power supply unit must be tested and controlled [9]. For the signal acquisition of the 8 input signals in the system, the system selects the
Hall sensor to detect the current, and uses the transmitter to convert the output signal of the sensor into a signal that can be recognized by the controller. For the output signal of the battery charge or discharge protection control, the system uses relays for protection.

3.2 The design of software in control system

ATmega16 has a complete set of programming and system development tools, including: C compiler, macro assembly, program debugger / software simulator, emulator and evaluation board. ICCAVR is a tool for developing MCU programs in accordance with the ANSI standard C language. The designed control system software is written in C language under the mega16 MCU development software ICCAVR environment. The AVR JTAG emulator is used as the program loading and debugging operation. The whole program is mainly divided into data acquisition and processing module, status display module, battery charging control module and battery discharge control module, SX1278 wireless data communication module and scheduling policy control module. Figure 5 shows Software flow chart.

The data acquisition and processing module is mainly responsible for collecting and processing various parameters of the recorded controlled system. The A/D conversion has the function of converting an analog signal into a digital signal that can be recognized by a single chip microcomputer. Firstly, the sampling measurement value of each parameter is obtained by A/D conversion, and then stored in the corresponding registration unit after processed by digital filtering. The program uses continuous acquisition of multiple sets of data, and performs arithmetic average operation on the measured values of each parameter as the current collection. The final value is stored. The current data indicating that the indicator light is on or off is processed and displayed on the status display screen. The single-chip microcomputer outputs an electric signal to realize the closing or opening of the control relay. Data is transmitted to the computer via SX1278 wireless data communication.
The working state of the battery has a great influence on the performance and working life of the battery, so it is very important to correctly judge the working state of the battery. There are three states when the battery is operating: over-discharge, normal, and over-charge. Reasonable control of the battery charging method is the key to the service life of the system. In the entire system, the charge or discharge control of the battery is determined by the current state of the battery. When the battery is in the over-discharge state, the system cuts off the power supply of the battery by the control relay to prevent the battery from being further discharged to protect the battery and emit an audible and visual alarm signal displayed by the indicator light. When the battery is in the overcharge state, it is necessary to issue an alarm signal in time to step up the surplus power into the grid. If the weather conditions are not good enough to generate enough power, start the battery to supply power to the load. When the battery terminal voltage drops to the set value, stop the power supply to the load.

In the judgment of the state of the battery, a threshold control method is adopted, which can prevent frequent relay action and alarm due to battery voltage instability. The status of the battery can be displayed by the indicator of the status display module. The over-discharge, over-charge, and normal status are displayed by three indicators. The status switch also has a buzzer as an alarm.

4. Conclusion

1) Based on the working principle of wind energy, rain energy, graphene and energy storage, a integrated power generation system based on wind, rainwater and energy store battery which suitable
for urban buildings is designed. It is feasible in principle. This integrated power generation system can make full use of natural wind, rain, and has the green environmental characteristics.

2) The MCU is used as the data acquisition and controller, and the control system is designed. By the use of the SX1278 wireless data communication module, wireless communication and real-time monitoring are realized.

3) The threshold control method is used to determine the state of the battery, and the charging control and discharge control of the system are realized by the software programming.

4) The design of control system software adopts modular, which making the development speed faster. In addition, by the use of modular the program become more concise, easier to modify, and has a strong program portability.

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Wave parameters influence on breakwater stability

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Abstract. This research paper experimentally explored the wave parameters effects on the breakwater armour layer stability. Breakwater stability mainly depends on the armour layer that guards the inner layers against the wave attack and the wave condition affects the breakwater stability. The wave height, length and steepness effect were investigated during the test program. Different formulas were available in the literature to predict the breakwater stability. The laboratory results were thus employed to assess the applicability of existing design equations. Experimental results showed that under normal wave condition (Hs) the breakwater was stable, whereas under storm condition (Hs of 1.2 designed wave height) the roundhead reached the failure state. The results approved the impact of wave parameters on the breakwater stability and damage progression. Results revealed that the existing design formulas underestimate the structure stability.

1. Introduction

Breakwater as a coastal protection structure is frequently applied to protect the coastal area. The wave attack produces forces on the breakwaters due to the associated phenomena as breaking, reflection, refraction and resulting rip currents. The breakwater work is to dissipate the wave energy. Therefore, the resulted forces have a great impact on the stability of the breakwater armour layer. Breakwaters are always designed applying empirical formulas to determine the armour stone size, weight, density and/or breakwater slope with regarding to the wave conditions.

It is commonly agreed that any breakwater structure should be tested before construction using a physical modeling tool in order to examine its stability. The reliability of the rubble-mound type is governed by the weight and interlocking nature of the armour unit's material and the cross section of the structure. Mostly the conventional and low crested structures are composed of armour layers of different units, bedding layer of different smaller material and toe protection. The breakwater is usually built with different material size, larger material facing the wave attack to smaller material in the core. Consequently, investigating the design may result in reducing the construction costs, the size and reducing the weight of the armour units.

The armour layer stability was the focus of different researches, e.g. [1-9]. Mase et al. [10] Investigated the effects of wave attack, wave breaking and wave steepness on the stability of the wave dissipating blocks. Van Gent and van der Werf [6] argued that the wave steepness, width, and thickness of the toe appear to affect the toe damage. Accordingly, these parameters need to be considered during the design process to derive accurate predictions of the damage to the toe structure. Kramer [11] believed that under slightly oblique waves, low crested structures could be exposed to be damaged. The stability of the rock armour layer with wave attack was recently investigated by [12 and 13].

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The wave attack and overtopping may cause unit movement of the armour and bedding. This phenomenon is known as the hydraulic instability. The movements can take the form of rocking, displacement of units out of the armour units. Hanzawa et al. [14] reported that the larger the wave steepness, the larger the stability which is the opposite tendency of [1]. The stability number is depending on wave steepness in a direct relation, [10].

The hydraulic design of the breakwater is mainly based on the wave conditions. The structure stability depends on the allowable damage for the different breakwater layers and on the material used for the armour, the core, bedding and toe layers. The damage percentage for rock structures are defined according to [15] as "the normalized eroded volume in an active volume from the middle of the crest to one $H_s$ below still water level". A statically stable structure has stability number, $N_s$, between 1 and 4. The damage in terms of displaced units is generally given as the relative displacement, $D$, [16] defined as the proportion of displaced units relative to the total number of units, or preferably, to the number of units within a specific zone around still water level (SWL).

Structure stability depends on the surrounding seabed, structural outer shape, characteristics of materials, and hydrodynamic parameters. Consequently, it was concluded that, the necessary armour rock size is in general influenced by the acceptable damage level and physical characteristics such as structural parameters, materials and hydrodynamics [11]. Helgason and Burcharth [3] proved that the effect of rock density is correctly described by traditional stability formulae for rock armour in case of structure slopes 1:2 and most likely for flatter slopes.

According to vidal et. al. [17], the stability number for non-overtopped structures is 1.4 and for structures with the crest at the still-water level is 1.6. They also defined the initiation of damage, $S$ in range of 0.5 to 1.5, for $2.01 < \left( \frac{R_c}{D_{n50}} \right) < 2.41$, where $R_c$ is the free board and $D_{n50}$ is the nominal diameter of the armour units.

Many researches and design equations are available in the literature. The wave condition is the most important factor affects the breakwater stability. The accuracy and reliability of the existing equations is very essential as it has a great effect on the construction cost. The present laboratory experiments aimed to test the stone armour layers and their hydraulic stability under different wave conditions including the tide. The effect of the wave height, length and the wave steepness were assessed. The experiments results were employed to benchmark of the available existing designing formulas.

2. Material and Methods

A physical model was constructed at the Hydraulics Research Institute (HRI) of the National Water Research Center (NWRC), Egypt, to achieve the objective of this research. The model construction and setup are depicted in Figure 1. The wave basin of an area of 34.0-m x 31.0-m was equipped with a wave generating system capable of producing regular and irregular waves up to 15.0 cm wave height. The wave board was provided with active reflection compensation. This means that the motion of the wave board compensates for the waves reflected by the structure and preventing them to re-reflect at the wave board and propagate towards the model. A spending beach was placed around the basin boundaries to dissipate the transmitted waves. Moreover, the wave basin in front of the wave generator has slope of 1:10 to damp the waves.

![Figure 1. Model construction and setup.](image-url)
The breakwater was constructed from stone layers, with a side slope of 1:2. It had a crest level of +15 cm and a toe level of -26 cm. Specifications of the considered layers are given in Table 1. The armour material was placed directly on the top of the under-layer material of different breakwater sections randomly.

### Table 1. Layers specifications.

<table>
<thead>
<tr>
<th>Breakwater Layers</th>
<th>Model</th>
<th>Weight (gm)</th>
<th>Density (t/m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Armour (stones)</td>
<td></td>
<td>178 – 356</td>
<td>2.3</td>
</tr>
<tr>
<td>Under layer (stones)</td>
<td></td>
<td>15.0 - 37.0</td>
<td>2.4</td>
</tr>
<tr>
<td>Toe (stones)</td>
<td></td>
<td>15.0 - 37.0</td>
<td>2.4</td>
</tr>
<tr>
<td>Core (stones)</td>
<td></td>
<td>0.75 - 3.0</td>
<td>2.4</td>
</tr>
<tr>
<td>Sea water</td>
<td></td>
<td></td>
<td>1.00</td>
</tr>
</tbody>
</table>

The considered armour layer is stone blocks. The armour layer weight was ranged from 178 gm to 356 gm in the model. Throughout this study, JONSWAP spectrum was applied in all tests with 1000 wave. The bed was leveled to produce the required bed in front of the breakwater with a slope of 1:50, along with a water depth in front of the breakwater 25.75 cm. Wave conditions were measured in the deep water and in front of the structure toe. The significant wave height in this test ranged from 4 to 12 cm with a peak wave period varies between $T_p = 0.9$ and $1.56$ s perpendicular to the breakwater, Table 2.

### Table 2. Test program.

<table>
<thead>
<tr>
<th>No.</th>
<th>Significant Wave Height (cm)</th>
<th>Peak Period $T_p$ (s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test 1</td>
<td>4.0</td>
<td>0.90</td>
</tr>
<tr>
<td>Test 2</td>
<td>6.0</td>
<td>1.10</td>
</tr>
<tr>
<td>Test 3</td>
<td>8.0</td>
<td>1.27</td>
</tr>
<tr>
<td>Test 4</td>
<td>10.0</td>
<td>1.42</td>
</tr>
<tr>
<td>Test 5</td>
<td>12.0</td>
<td>1.56</td>
</tr>
</tbody>
</table>

Wave height meters (WHM) that were designed for dynamic fluid level measurements, were used to measure the incident wave heights in these experiments. The obtained time series at the toe were used for the analysis. The resulted damage due to wave attacks were recorded utilizing digital overlay photos during each test. The rocked units and moved stones that were displaced more than one-unit diameter, ($N_{od}$) were counted.

The characteristics of damage in this research was based on the eroded cross-section area, $A_{se}$, around sea water level (swl) as introduced by [18-20]. Where the eroded area was defined as $(S = A_e/D_{n50}^2)$ for trunk damage.

Different design equations for breakwater stability were deduced for rock armour units, the results were used to assess their applicability. Van Der Meer [21] introduced the following formula for surging wave to compute the breakwater stability of stone material with considering the breaking condition.

$$\frac{H_s}{\Delta D_{n50}} = 1.0 \ p^{-0.13} \ (\sqrt{\cot \infty} ) \left( \frac{S}{N} \right)^{\frac{1}{2}} \ e_m^p$$

(1)

Where $H_s$ is the significant wave height, $\Delta$ is the relative armour density, $D_{n50}$ is the nominal diameter of the armour units, $p$ is the breakwater permeability, $\alpha$ is the breakwater slope, $S$ is the damage level, $N$ is the number of waves, and $\xi_n$ is the wave steepness based on mean wave period.

Vidal et. al. [22] proposed a general equation for designing the breakwater stability of different sections. They introduced reduction factors for the different sections. The general equation with factors for the front trunk and head is presented as:
\[ N_s = 1.831 - 0.245 \left( \frac{R_c}{D_{n50}} \right) + 0.0119 \left( \frac{R_c}{D_{n50}} \right)^2 \]

Where \( N_s \) is the stability number, and \( R_c \) is the free board.

Van Gent et al. [23] developed an equation to predict the stone armour stability. They considered the structure porosity and wave period. However, they disregarded the breaking condition.

\[ \frac{H_s}{\Delta D_{n50}} = 1.75 \cot(\alpha)^{0.5} \left( 1 + \frac{D_{n50-core}}{D_{n50}} \right) \left( \frac{S}{\sqrt{N}} \right)^{1/5} \]

In which \( D_{n50-core} \) is the nominal diameter of the core units.

The breakwater was divided into zones to observe the damage, the number of armour layer stones was identified in each zone. The armour layer stability was investigated under different wave conditions representing calm and storm conditions, Figure 2. Stone movements and overtopping were monitored by photos and videos employing a digital camera in addition to visual inspections. The armour layer and toe were reconstructed after each test. The wave condition during the performed tests was recorded. The incident wave heights were measured using the WHM in the deep water and in front of the breakwater as shown in Figure 2. The breakwater stability was evaluated with respect to the designed conditions and allowable damages, for this breakwater the allowable damages percentage of \( (S = 0-5\%) \) along with stability \( N_s \) of 1.50.

![Figure 2. Model during operation.](image)

### 3. Results and Discussions

The changes to the armour layer due to the wave attack was examined in this study, where the stone movement was recorded using a precise digital video camera. After each test the moved armour units and rocked units were counted before reforming the armour layer for the next run. The damage was recorded as in table 3, and percentage damage was then computed applying guideline of [16]. The initiation of damage, \( S \) condition equals 2-3 (equal to damage % 0-5) which corresponding to a little movement and displaced stones in a section according to [24], while failure status occurs with movement of stones at \( S = 8 \).

<table>
<thead>
<tr>
<th>Run</th>
<th>( H_s ) (cm)</th>
<th>Head section</th>
<th>Trunk Section</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>No. of stones moved or rocked</td>
<td>% damage</td>
</tr>
<tr>
<td>1</td>
<td>4</td>
<td>0</td>
<td>0.00</td>
</tr>
<tr>
<td>2</td>
<td>6</td>
<td>0</td>
<td>0.00</td>
</tr>
<tr>
<td>3</td>
<td>8</td>
<td>1</td>
<td>0.52</td>
</tr>
<tr>
<td>4</td>
<td>10</td>
<td>1</td>
<td>0.52</td>
</tr>
<tr>
<td>5</td>
<td>12</td>
<td>4</td>
<td>2.05</td>
</tr>
</tbody>
</table>

### Table 3. Recorded damage.
Referring to Figures 3 and 4, the damage level and progression were increased with increasing the water height for both the head and trunk sections. Figure 4 shows direct relation between the stability number and the damage percentage, which confirms the results of [25]. In Figure 3, the percentage damage was directly proportional to the relative wave height. Moreover, the damage level for the head section was higher for $N_s$ of more than 1.5 and the damage progression was also higher for a head section than the trunk. Moreover, Figure 3 demonstrates that the different breakwater sections are stable under severe wave conditions, only the head section face failure stage at $N_s$ of 2.4.

**Figure 3.a.** Damage level.

**Figure 3.b.** Damage progression.

**Figure 4.** Damage progression versus stability number.
It is worthy to mention that the stability of the breakwater was higher than the designed stability, as
the breakwater was designed for a stability number of 1.5 with initial damage of two stones moving.
This result agreed well with the previous studies. The head of the breakwater was much vulnerable
due to the curvature and it was the least stable part especially in the case of emerged breakwaters, [22
and 24], where the interlocking was much less than other parts. The toe and bedding layers were stable
under different wave attack.

![Figure 5. Effect of Wave Length.](image)

The wave length and the damage percentage are depicted in Figure 5. According to this figure, for
long waves the damage was higher than short waves where little damage occurred. Figure 5 illustrates
that with short wave attack slight damage occurred for both the head and trunk sections. It is also
observed that with long waves the damage takes place at the head section. It is worthy to be noted that
armour units interlocking improved and increased with shortest wave duration, while decreased with
longer ones and they showed a high-performance consistency with the argument of [25].

![Figure 6. Effect of wave steepness.](image)

Figure 6 presents the impact of wave steepness on the breakwater stability, they were inversely
proportional, high stability corresponds to short wave steepness until wave steepness reached 0.034
then the relation became direct, having the same trend proposed by [24].

The present experimental results were employed to evaluate and benchmark some of existing design
formulas for the rock armour layer. The formulas of [22] and [24] produced the same trend for both
the head and trunk sections, Figures 7 and 8 respectively. The design equations underestimated the
stability of the breakwater for the trunk and the head section.

Equation of [22] produced consistent results with the current experiments for the breakwater stability.
Whilst, it was not sensitive to wave condition and it mainly was sensitive to the stone diameter and
free board.
The discrepancy between the observed and calculated values may be due to the methodology in definition or calculation of damages. Although the design equations gave the same trends, but they underestimated the breakwater stability for the considered case. This confirmed that the actual stability was usually much higher than the designed ones, [24]. Also, the conclusion of [26] noted that the measured stability numbers and predicted are not the same.

4. Conclusions

The breakwater stability under wave attack was experimentally investigated throughout this research. The effect of the wave parameters was analyzed and graphically presented. The laboratory data were employed to evaluate the existing design equations. The damage in the breakwater was occurred when the breakwater was exposed to wave storm with failure in head section. While the trunk was stable under all wave conditions. The results agree well and support the previous investigations. According to the experimental results, the following conclusions could be drawn:

- The damage level increased for long waves than shorter waves with a direct relationship.
- The wave steepness had a clear impact on the breakwater stability for both the trunk and head section
- The breakwater stability was found to be higher than the designed, which increasing the construction costs.
- The percentage damage of the armour layer for the rounded head is higher than for the trunk. It was found to be 2.05% while for the trunk section was found to be 0.64% with the severe wave condition of $H_s = 12$ cm.
Comparison between the present damage results and those calculated by the existing formulas, revealed that they underestimated the stability and they were in a good estimation for occurring damage.

Vidal's equation was only sensitive for free board and nominal diameter and did not represent the stability under the different wave attack conditions.

According to the study results, further investigations are needed as follows:

- More research investigation is needed tackling the breakwater design to develop more accurate design equation that gives the accurate required armour units specifications for cost reduction.
- Development is needed to adjust the existing equations to match with the real estimate of the breakwater function which is the breaking state.
- Monitoring of existing breakwater could improve the existing design concepts and formulas by two-way approach.
- Physical modeling is still essential in the coastal structure to resolve the uncertainty in the designing formula especially the stability part. More efforts are required in this issue to reduce the overall cost, such as the development of the mathematical model that is able to execute stability tests.

5. Notation

The following symbols were used in this paper:

- $A_e$: Eroded area ($m^2$)
- $D_{n50}$: Nominal diameter of the armour units ($m$)
- $D_{n50-core}$: Nominal diameter of the core units ($m$)
- $H_d$: Wave height in deep water ($m$)
- $H_s$: Significant wave height ($m$)
- $N$: Number of waves (-)
- $N_s$: Stability number (-)
- $N_{OD}$: Damage number (-)
- $P$: Breakwater permeability ($m/s$)
- $R_{fc}$: Free board ($m$)
- $S$: Damage level (%)
- $T_p$: Peak period (s)
- $\alpha$: Breakwater slope (-)
- $\Delta$: Relative armour density (-)
- $\eta$: Wave steepness based on mean wave period (-)

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Experimental Study on Wave Interaction with Multiple Row Pile Breakwater

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Abstract. Coastal works where the tranquillity requirements are low porous pile breakwaters are considered as a good cost-effective substitute for the conventional type of breakwaters. In this study, an experimental investigation has been carried out in a two-dimensional wave flume to study the wave interaction with vertical pile breakwater. At 50 cm still water depth (h), interactions between regular waves (wave period T = 1.5 sec, 1.6 sec, 1.8 sec and 2.0 sec) and the pile breakwater of two different porosity (n = 0.65 and 0.80) and three different structure heights (hb = 40 cm, 50 cm and 60 cm) have been studied experimentally. Experimental results reveal that, minimum transmission coefficient (Kt = 0.55) is obtained for breakwater with lowest porosity (n = 0.65) and with emerged condition (hb/h = 1.2) for short wave period (T = 1.5 sec). Minimum reflection coefficient is obtained for breakwater with highest porosity (n = 0.80) and with submerged condition (hb/h = 0.8). It is also noticed that porosity has an effect on wave energy loss coefficient.

1. Introduction

Porous structures offer an alternative to the conventional solid breakwater. Seawalls with porous toe, caissons on rubble foundation, rubble-mound breakwaters, and pile breakwaters are some kind of typical porous structures. In general, we define a porous medium as a rigid body containing pores that are randomly distributed throughout the body, thus forming a homogeneous matrix allowing fluid to pass through.

Vertical pile structures protect lee side wave attack by reflecting and dissipating wave energy through the viscosity-induced resistance in the porous media. It permits to exchange the water masses along the beaches, which minimize the pollution aspects. With the advantage of changeable vertical condition (emerged and submerged), it becomes more effective, the landside of the emerged types of this breakwater can be used for berthing purposes. The functional performance of the porous breakwater is evaluated by examining the wave reflection, transmission and wave energy dissipation caused by this breakwater. The reflected waves and the dissipated wave energy are strongly affected by water depth, wave properties such as period and height, and structural properties. The major structures properties are pile distribution, the height of the pile above or below the water surface, clearance of the toe of piles.

Many Experimental and theoretical studies have been done for determining the efficiency of vertical emerged and submerged permeable or impermeable types of breakwaters. A hollow framework-shaped test model was used. Three models each with three different porosities ranging from 0.40 to 0.80 were used in that experiment. Womera investigated the interaction between waves and rectangular submerged impermeable breakwater [1]. Akter and Rahman investigated the
experimentally hydrodynamic performance of rectangular porous breakwater both submerged and emerged way. Wave transmission coefficient, wave reflection coefficient and energy loss coefficient values were then analysed with respect to relative submergence, relative breakwater width, and porosity of breakwater. Minimum transmission coefficient was obtained for breakwater with minimum porosity. Minimum reflection coefficient was obtained for breakwater with the highest porosity. Wave energy loss coefficient increases with increasing porosity. Relative submergence has an effect on these coefficients too [2]. Koftics et al performed an experimental investigation on multiple row pile breakwaters. There were Three different configurations of the model, regarding the number of rows of piles, are examined; namely two, four and six rows with three different water level condition submerged, emerged and waterline. As expected, the emerged 6-row structure is the most efficient in wave attenuation, since the smaller transmission is found, namely transmission coefficient 0.40 [3]. Hayashi et al performed hydraulic research on the closely spaced pile breakwater both experimentally and theoretically. A laboratory experiment has been made on a model structure. The coefficients of wave transmission and wave reflection obtained from the experiments, one theory was also present and comparison between experimental and theoretical results was shown [4]. Afroze and Rahman performed both experimental and numerical study on a horizontal slotted submerged porous breakwater. From this study, it was found that Wave reflection coefficient increases as relative breakwater width decreases. Also, the reflection coefficient decreases as porosity increases. The transmission coefficient decreases as relative breakwater width increases. Wave energy loss coefficient increases as relative breakwater width increases and as the porosity decreases [5].

2. Experimental Setup

Experiments were carried out in a two-dimensional wave flume (21.3 meters long, 0.76 meters wide and 0.74 meters deep) at the Hydraulics and River Engineering Laboratory of Bangladesh University of Engineering and Technology (BUET). Multiple row pile breakwater had been made using the plastic hollow pipe as the pile, filling it with sand and concrete cap for stability against wave action. Piles are 9.8 cm in diameter. There was iron made grill structure in the bottom so that piles can stand in the flume which is made with ¼” diameter rod. The total length of the breakwater was 95 cm and width was 73 cm. There were pipes of three height 60cm, 40cm and 50 cm. The orientation of pipes was changed for gaining different porosity. After placing the iron made grill structure in the bottom of the flume pipes were placed with two different orientations for obtaining different porosity. Figure 1(a) and Figure 1(b) shows the plan view and longitudinal view of pile breakwater of porosity 0.65. Figure 2(a) and Figure 2(b) indicates the plan view and longitudinal view of pile breakwater of porosity 0.80. Pipes heights were changed for changing relative submergence and still water depth was maintained in the laboratory flume for all experimental runs as 50 cm. Regular waves with four different wave periods (T=1.5 s, 1.6 s, 1.8 s and 2.0 s) were generated from a flap type wave generator installed at the upstream end of the wave flume. To damp the transmitted wave after passing the breakwater a wave absorber was installed at the end of the wave flume. For each type of breakwater, three different heights of breakwater (h_b) as 40 cm, 50 cm and 60 cm were used, which made relative structure height (relative submergence), h_b/h = 0.8 (submerged), 1.0 (submerged) and 1.2 (emerged). Interaction of regular waves of four different wave periods with the breakwater of two different porosities and each breakwater having three different heights made twenty-four laboratory runs. For each experimental run, wave heights were measured at five locations, two were in front of the breakwater, the third one was over the breakwater and the last two were behind the breakwater. The detail of the experimental setup is shown in Figure 3, Figure 4 and Figure 5. Some pictures regarding data collection are shown in Figure 6.
Figure 1. Profile view of breakwater for porosity 0.65. (a) Plan view of breakwater for porosity 0.65 (b) Longitudinal view of breakwater for porosity 0.65

Figure 2. Profile view of breakwater for porosity 0.80. (a) Plan view of breakwater for porosity 0.80 (b) Longitudinal view of breakwater for porosity 0.80
Figure 3. Experimental setup in detail.

Figure 4. Experimental setup plan view for porosity 0.65.

Figure 5. Experimental setup plan view for porosity 0.80.

The maximum and the minimum wave heights ($H_{max}$ and $H_{min}$) at the wave generator side (upstream the breakwater) and the transmitted wave heights ($H_t$) at the wave absorber side (downstream the
breakwater) were measured to estimate the reflection and transmission coefficients \( (K_r \text{ and } K_t) \). Incident wave height \( (H_i) \) and reflected wave height \( (H_r) \) are calculated as \( H_i = (H_{\text{max}} + H_{\text{min}})/2 \) and \( H_r = (H_{\text{max}} - H_{\text{min}})/2 \) respectively, where \( H_{\text{max}} \) = maximum wave height measured at antinodes and \( H_{\text{min}} \) = minimum wave height measured at nodes. Then reflection coefficient, \( K_r = H_r/H_i \) and transmission coefficient, \( K_t = H_t/H_i \) are calculated, where \( H_t \) = Transmitted wave height. The energy loss coefficient, \( K_L \) can be calculated from the relation (Thornton and Calhoun 1972): \( K_r^2 + K_t^2 + K_L^2 = 1 \). The conventional method as used by Dean and Dalrymple [1991] has been adopted to separate the measured wave train into its incident and reflected wave components. For measuring maximum and minimum wave heights, two wave gauges were placed at fixed distances of \( L/4 \) and \( L/2 \) from the breakwater, where \( L \) is the wave length. At each position (antinode, \( L/4 \) and node, \( L/2 \)) data of water surface were collected for one-minute duration at five seconds interval. Then maximum or minimum wave heights \( (H_{\text{max}} \text{ or } H_{\text{min}}) \) in cm were calculated by taking difference between the maximum and minimum water surface reading at antinode and node respectively.

![Data collection pictures](image)

**Figure 6.** Data collection pictures (a) Wave propagation (b) reduction of wave height around breakwater (c) Wave height measuring tape (d) Longitudinal view of wave propagation.

### 3. Results and discussions

#### 3.1. Effect of relative submergence \( (h_b/h) \) and relative breakwater width \( (k \times B) \) on Transmission coefficient \( (K_t) \)

Figure 7(a) and 7(b) shows the relationship between the transmission coefficient \( (K_t) \) and relative breakwater width \( (k \times B = 2\pi B/L) \), where \( k \) is the wave number. The figures show the results for three different breakwater heights \( (h_b/h) \) as 0.8, 1.0 and 1.2 and also for two different values of breakwater porosity \( (n = 0.65, 0.80) \). It is seen that the transmission coefficient \( (K_t) \) decreases as \( k \times B \) increases.
This reveals that the breakwater width \((B)\) has influence in damping the transmitted wave the above-mentioned behaviour could be attributed to two reasons. First, the increase of breakwater width causes the increase of the friction between the breakwater surface and the transmitted waves, causing more wave energy loss. Second, as the wave becomes short, the water particle velocity and acceleration suddenly change and the turbulence caused due to this sudden change causes dissipation in the wave energy. Also, the transmission coefficient decreases as \(h_b/h\) increases. In Figure 7(a), when porosity \(n = 0.65\) the transmission coefficient decreases from 0.71 to 0.55 for \(h_b/h =1.2\), decreases from 0.79 to 0.65 for \(h_b/h = 1.0\), decreases from 0.89 to 0.85 for \(h_b/h = 0.8\) for increasing relative breakwater width \((k \times B)\) from 1.47 to 2.12.

![Figure 7(a)](image-a)

![Figure 7(b)](image-b)

**Figure 7.** Effect of relative breakwater width on transmission co-efficient (a) For porosity 0.65(b) For porosity 0.80

### 3.2. Effect of relative submergence \((h_b/h)\) and relative breakwater width \((k \times B)\) on reflection coefficient \((K_r)\)

Figure 8(a) and 8(b) shows the relationship between the wave reflection coefficient \((K_r)\) and the relative breakwater width \((k \times B =2\pi B/L)\), for \(h_b/h= 1.2, 1.0, 0.80\) and for porosity \(n = (0.65,0.80)\). The figure shows that \(K_r\) decreases as \(k \times B\) increases. This may be attributed to the increase of the wave energy loss as the width of the porous media increases. Also, the reflection coefficient \((K_r)\) increases as \(h_b/h\) increases.

Reflection coefficient decreases from 0.58 to 0.42 when \(h_b/h =1.2\), decreases from 0.45 to 0.27 when \(h_b/h =1.0\), and from 0.27 to 0.13 when \(h_b/h =0.8\) for porosity \(n = 0.65\) and for increasing \(k \times B\) from 1.47 to 2.12 which are shown in Figure 8(a).

Reflection coefficient decreases from 0.28 to 0.15 when \(h_b/h =1.2\), decreases from 0.19 to 0.12 when \(h_b/h =1.0\), decreases from 0.12 to 0.07 when \(h_b/h =0.8\) for porosity \(n = 0.80\) and increasing \(k \times B\) from 1.47 to 2.12 which are shown in Figure 8(b).
3.3. Effect of relative submergence ($\frac{h_b}{h}$) and relative breakwater width ($k \times B$) on Wave energy loss coefficient ($K_L$).

Figure 9(a) and 9(b) shows the relationship between the wave energy loss coefficient ($K_L$) and the relative breakwater width ($k \times B = 2\pi B/L$) when $h_b/h = 0.80, 1.0, 1.2$ and $n = 0.65$ and $0.80$. These figures show that $K_L$ increases as $k \times B$ increases. Also, the wave energy loss coefficient ($K_L$) increases as $h_b/h$ increases from 0.65 to 0.80.

In Figure 9(a), when porosity $n = 0.65$, the wave energy loss coefficient ($K_L$) increased from 0.39 to 0.72 when $h_b/h = 1.2$, from 0.42 to 0.71 when $h_b/h = 1.0$, from 0.37 to 0.51 when $h_b/h = 0.80$ for increasing $k \times B$ from 1.47 to 2.12.

The wave energy loss coefficient increased from 0.50 to 0.74 when $h_b/h = 1.2$, 0.45 to 0.67 when $h_b/h = 1.0$, from 0.211 to 0.36 when $h_b/h = 0.80$ for porosity $n = 0.80$ and increasing $k \times B$ from 1.47 to 2.12 as shown in Figure 9(b).

3.4. Effect of breakwater porosity on the transmission co-efficient ($K_t$), reflection co-efficient ($K_r$), wave energy loss coefficient ($K_L$):
Figure 10(a), 10(b) and 10(c) show the variation of the transmission coefficient, reflection coefficient and the wave energy loss coefficient for two different porosity ($n = 0.65, 0.80$) of the breakwater at relative submergence 1.2. at relative submergence of $h_b/h = 1.2$. It is seen that as the porosity of breakwater increases, most of the wave energy easily transmits rather dissipating and results in higher transmission coefficient. The reflection coefficient is seen to increase for less porous breakwater (when $n = 0.80$). The wave energy loss coefficient is less for higher porous breakwater (when $n = 0.80$).

Moreover, the transmission and the reflection coefficient decreases as the relative breakwater width ($k \times B$) increases, while the wave energy loss coefficient takes the opposite trend.

For, $h_b/h = 1.2$, in Figure 10(a) it is seen that the transmission coefficient increases as the porosity increases from $n = 0.65$ to $n = 0.80$ by about 15.5% to 18.18% with increasing $k \times B$ from 1.47 to 2.12. For the same case, the reflection coefficient decreases by 51.73% to 64.28% as seen in Figure 10(b) and wave energy loss coefficient decreases by 22% to 2.70% as shown in Figure 10(c).

![Figure 10](image)

**Figure 10.** Effect of relative breakwater porosity when relative submergence $h_b/h = 1.2$ (a) On transmission coefficient (b) On reflection coefficient (c) On wave energy loss coefficient

3.5. Relation among $K_t$, $K_r$ and $K_L$ with respect to relative submergence ($h_b/h$):

Figure 11(a) and 11(b) shows the effect of increasing relative submergence ($h_b/h$) on the transmission coefficient ($K_t$), reflection coefficient ($K_r$), and the wave energy loss coefficient ($K_L$) Here it observed that with increasing submergence (breakwater height increases from submerged to emerged) the transmission coefficient decreases, the reflection co-efficient increases, energy loss coefficient increases for any value of breakwater porosity.

Figure 11(a) shows the relation of $K_t$, $K_r$, and $K_L$ for porosity, $n = 0.65$ and wave period $T = 1.5$ sec. In this case, the transmission coefficient decreases 35.29% with increasing submergence from 0.8 to 1.2, reflection coefficient increases 223% and wave energy loss coefficient increases 41.17% with the increasing submergence from 0.8 to 1.2.

Figure11(b) shows the effect of increasing relative submergence $h_b/h$ on the transmission coefficient ($K_t$), reflection coefficient ($K_r$), and the wave energy loss coefficient ($K_L$) for porosity $n = 0.80$ and wave period, $T = 1.5$ sec. In this case, the transmission coefficient decreases 30.11%, reflection coefficient increases 114%, wave energy loss coefficient increases 91.89% with increasing submergence from 0.8 to 1.2.
4. Conclusions

Total twenty-four laboratory runs have been conducted in a two-dimensional wave flume. From this experimental run, it is revealed that the transmission coefficient decreases with an increase in relative submergence \( \frac{h_b}{h} \) and increases with an increase in breakwater porosity \( n \). The porous the breakwater the less the transmission coefficient. Experimental results reveal that, minimum transmission coefficient \( (K_t = 0.55) \) is obtained for breakwater with lowest porosity \( (n = 0.65) \) and with emerged condition (when \( h_b/h = 1.2 \) for short wave, i.e. when \( T = 1.5 \) sec. Minimum reflection coefficient \( (K_r =0.07) \) is obtained for breakwater with highest porosity \( (n =0.80) \) and with minimum submerged \( (h_b/h=0.8) \) condition. The higher the porosity the lower the reflection coefficient. Porosity and relative submergence of breakwater have an effect on wave energy loss coefficient also. From this study, it is revealed that Pile breakwater properties like porosity, relative submergence and breakwater width has an effect on its efficiency. Standard values of these three coefficients ensure a tranquil water basin for the berthing of vessels and less erosion in landside This study may help the coastal engineers for efficient design of the porous submerged emerged pile breakwater to be constructed for different purposes.

5. References

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