

Article

Experimental Study on Local Scour at the Monopile Foundation of an Offshore Wind Turbine under the Combined Action of Wave–Current–Vibration

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Abstract: Monopile foundations are the most widely used offshore wind turbine foundations. The experiments were conducted to investigate the influencing factors of local scour around the monopile under the action of wave–current–vibration. The study analyzed the characteristics of local scour, including the maximum scour depth, the development of scour hole shape, and the shape of the scour hole profile. The dimensionless influencing factors (vibration intensity, Froude number, Keulegan–Carpenter number, and combined wave–current parameter) are subsequently analyzed. An empirical formula is developed to predict the local scour depth of a monopile under the combined influence of wave–current–vibration. The formula provides a theoretical underpinning for engineering design.

Keywords: wave–current–vibration interaction; offshore wind turbine; monopile; local scour; empirical formula



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1. Introduction

In recent years, the safety and reliability of wind turbine foundations has become a research priority due to the large-scale construction of offshore wind turbines. The environment for offshore wind turbine foundations is significantly harsher than its onshore counterpart. The challenging marine environment poses significant obstacles to the design and construction of offshore wind turbine foundations, and results in severe local scour around these structures, ultimately leading to instability and failure. As a result, local scour remains a long-standing issue in marine science research. The offshore wind turbine foundation bears both the complex coupling effect of wind, wave, and current, as well as the dynamic and static loads generated by the upper rotating blades, transferring them to the seabed soil. This transfer has a significant impact on the soil properties around the foundation [1]. Additionally, strong sediment movement caused by waves and currents is a further factor leading to local scour. According to Yu [2], offshore wind turbines experience between 10^6 and 10^8 vibration loads during their service life. These loads are primarily caused by strong winds, waves, and rotating blades. Long-term vibrations can cause particles to migrate and soil to become denser around the monopile [3]. This can not only lead to changes in mechanical properties, like soil stiffness, that can weaken the pile foundation's bearing capacity but also affect how the bed sediment moves in the eddy current environment and further affects local scouring around the pile foundation [4]. Offshore wind turbine foundations come in various types, including monopile, gravity, mono-caisson, tripod-pile/caisson, jacket-pile/caisson, high-rise pile cap, and others [5]. Monopile foundations are particularly common in offshore wind turbine construction due to their excellent applicability, simple structure, high bearing capacity, and low construction cost [6,7]. Monopile foundations have a good application prospect for offshore wind power,

although many scholars have studied the design and protection of offshore wind power foundations [8–11]. However, research on the local scouring characteristics of monopile foundations under the joint action of many factors is still far from enough; especially, the sediment movement rules around the monopile considering the superimposed transverse vibration load is still not clear. And there are fewer studies on the local scour depth of monopile foundations under the joint action of multiple dynamics.

Firstly, offshore wind turbine pile foundations are unavoidably influenced by water currents and waves, inducing alterations in the surrounding conditions of the original wind turbine pile. As a result, the exposure of the offshore generator foundation may imperil the safety of the offshore wind turbine. Currently, numerous scholars have extensively researched local scour of pile foundations caused by waves and currents. Sumer [12] demonstrated that the depth of scouring under wave action changes with the Shields number, and when the Shields number is greater than the critical value causing sediment suspension on the pile side, the scour depth becomes independent of the Shields number. Qi and Gao [13] performed a local scour and pore pressure response test on a large-diameter monopile under the combined action of wave and current, using a wave–current flume. The test findings indicate that the combination of waves and current enhances the formation of local scour around individual piles. The effect on the equilibrium scour depth is significant, and it becomes more apparent under clear water scour. Zhang [14] also carried out experiments under three conditions: current-only, wave-only, and combined wave–current. The study’s results indicate that scour depth is greater under combined wave–current conditions than under current-only conditions, and that this increases with flow velocity and decreasing water depth. Gautam [15] discovered that equilibrium scour depth did not notably differ for scour under waves alone, waves with weak currents, and waves with mild currents when developed in a low KC number environment. The equilibrium scour depth significantly increases when waves are combined with strong currents. For large KC numbers, the equilibrium scour depth is found to have increased with an increase in KC number as well as the combined wave–current parameter (U_{cw}).

Secondly, the offshore wind turbine pile foundation is subjected to numerous lateral vibration and other lateral loads from external sources, leading to reduced soil stiffness surrounding the foundation and subsequent local scour of the pile foundation. Therefore, the current research focus should be on the settlement and convection characteristics of sand surrounding the pile foundation under complex loading and traditional scour research’s hydrodynamic loads. Shi [16] used numerical methods to study the unsteady flow around a monopile under the influence of waves and vibrations. The calculation results show that monopile vibrations cause a great disturbance to the unsteady flow around the pile. Yu [17] conducted an experimental investigation of a vibrating monopile inside a cohesionless saturated sand bed. The experimental results indicate that the lower the vibration frequency, the more prominent is the delayed attenuation process during the oscillation cycle. Additionally, the pore pressure amplitude decay rate increases with increasing frequency as the depth of the seabed grows, and the pile side pore pressure shows a trend of increasing and then decreasing with frequency.

Under the influence of vibration load, the soil surrounding the monopile undergoes compaction and densification, causing a change in the soil stiffness [18,19]. Currently, few studies have investigated the effects of vibration load on the scour process of monopiles. However, the local scour depth of the pile’s foundation is closely associated with the cumulative settlement resulting from vibration load. Therefore, it is crucial to investigate the principles of scour growth around a monopile experiencing vibration loading. Al-Hammadi and Simons [20] analyzed the local scour process of pile foundations under vibration loading. Their research concludes that the scour hole is broader and less deep when compared to the equilibrium scour hole without vibration loading. Initially, the compaction of the sand bed results in a decrease in the scour rate during the early stages of scour, although this does not impact the equilibrium scour depth. The alternate application of water current and vibration load significantly increases the local scour depth and width

of the monopile. Guan [21] conducted a study on the effects of lateral vibration loads on the local scour of monopiles. The research found that vibration loads can increase the initial scour velocity, mainly due to soil compaction and pile–soil interactions. Furthermore, raising the frequency and amplitude of vibration can lead to decreased equilibrium scour depth and slope of the scour hole. These effects are attributed to sediment ratchet convection occurring on the surface of the scour hole. Qin [22] investigated how monopile vibration, water flow, and intermittent operation of OWTs (offshore wind turbines) work together to affect the scour around the monopile foundation. The results show that coupling of a large-amplitude monopile vibration and water flow will exacerbate the scour, while the coupling of a small-amplitude monopile vibration and water flow will retard the scour.

The research results above demonstrate that physical model testing on vibration mainly focuses on the settlement and convective motion characteristics of sand around a monopile under pure vibration, and the law of tracking the trajectory of sediment particles is studied through numerical simulation tests. For the vibration-induced scour test, a local scour test of a monopile under the influence of current and vibration was conducted, providing reference significance for studying the local scour of a monopile under the influence of wave–current–vibration. However, very few studies [23] have been conducted on the existing local scour of a monopile under the influence of wave–current–vibration in a marine environment. Therefore, it is of great significance to carry out physical model test research on the local scour of a monopile under the combined action of wave–current–vibration by arranging a vibration loading device in a wave–current flume, in order to explore the local scour characteristics and scour depth development law of monopiles, and to fit the empirical formula for predicting the maximum scour depth of monopiles.

In summary, this study uses a wave flume to test the local scour characteristics of a monopile foundation subjected to combined wave–current–vibration. Additionally, a generalized test is conducted to investigate the coupling mechanism between sand movement deformation and wave–current–structure–seabed action under vibration loading. Based on the test data, an empirical formula has been developed to predict the maximum scour depth. This formula offers a theoretical foundation for the design and safeguarding of monopile structures in actual complex environments.

2. Experimental Research Program

2.1. Experimental Layout

This experiment was carried out in the wave–current flume of the Hydraulic Experiment Center of Changsha University of Science and Technology. The test arrangement is shown in Figure 1. The length $\times$ width $\times$ height of the wave–current flume is 60 m $\times$ 1.5 m $\times$ 1.8 m. The flume is emptied by arranging the slope section and the flat slope section. The length $\times$ width $\times$ height of the flat slope section is 24 m $\times$ 1.5 m $\times$ 0.6 m, the length $\times$ width $\times$ height of the slope section is 6 m $\times$ 1.5 m $\times$ 0.6 m, the slope foot is 5.71° , and the sand in the sand trough experimental area with length $\times$ width $\times$ height of 4.5 m $\times$ 1.5 m $\times$ 0.6 m is reserved between the flat slope sections. Non-cohesive sand with a median particle size of $d_{50} = 0.334$ mm, a relative density $\rho = 2.65 \times 10^3$ kg/m³, and a unit weight of $\gamma = 14.58$ KN/m³ is laid in the sand trough test area. The model circular piles with a diameter of $D = 0.09$ m are buried between the sand troughs. The model piles are connected with the exciter to carry out cyclic vibration loading. One end of the flume is equipped with a push plate wave-making system and a cyclic current-making system. At the other end, a slope-type energy dissipation network is added to reduce the influence of wave reflection on the test, so as to ensure that the wave-making and current-making system can work for a long time and maintain the required wave and current types. The ratio of the distance between the side walls of the flume to the diameter of the model pile is $70.55/8.9 \approx 7.927 > 3$, which is in line with the wave model test specification. Referring to the suggestion of Whitehouse [24], the ratio of the width of the flume to the diameter of the model pile should be greater than 6 in the clear water scour, and the ratio of this test is $1.5/0.09 \approx 16.85 > 6$. Therefore, the side wall effect can be ignored in this test. Several

pre-experiments were carried out before the formal test to observe the test results to ensure the stable operation of the wave-making and current-making system. The working water depth of this test is 0.5 m, which is greater than 4 times the pile diameter D , and the influence of water depth on local scour can be ignored [25]. The experimental wave height data were collected by a WG-50 wave height meter (RBR Ltd., Ottawa, ON, Canada). A three-dimensional Doppler profile current meter (Nortek AS, Oslo, Norway) was used to collect velocity data. A ULS-100 terrain scanner (2G Robotics Inc., Waterloo, ON, Canada) is used to collect the local scour terrain data, and the three-dimensional terrain scanner can be translated and fixed through the multi-functional measuring frame to accurately collect the terrain data. The vibration load of the monopile is loaded by an HEV-500 high energy vibrator (Nanjing Foneng Technology Industry Co. Ltd., Nanjing, China), and a D050 strain displacement sensor from Yangzhou Jingming (Yangzhou Jingming Testing Technology Co. Ltd., Yangzhou, China) is installed to collect and calibrate the frequency and amplitude of pile movement. A scale is attached around the pile body to record the development of erosion depth in real time.

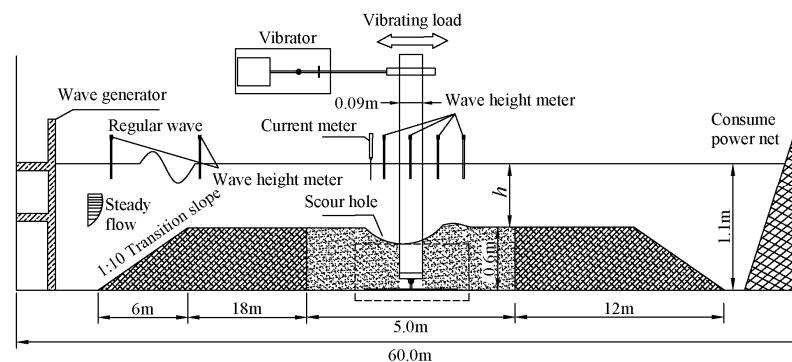


Figure 1. Experimental layout diagram.

In this paper, the wave-making system is calibrated by the wave height meter before the start of the test to ensure that the waves produced by the computer during the test are consistent with the requirements, and it is convenient to collect the wave height data during the test. During the test, a total of six wave height meters were arranged to collect the incident wave height and the wave height change around the pile.

2.2. Key Parameters of the Experimental

The main parameters such as vibration frequency f_v , amplitude A_v , wave height H , wave period T , and flow velocity U_c required for the test are mainly considered and selected from the following aspects: (1) All the above parameters are within the normal working range of the test equipment. (2) Because an offshore wind turbine is subjected to complex environmental loads, these environmental loads have the characteristics of low frequency. When the inclination angle of the offshore wind turbine pile exceeds 0.5° , the offshore wind turbine will not be able to work normally. The selected test vibration frequency and amplitude should be low frequency and low amplitude. Based on the above principles, the vibration frequency $f_v = 3, 6, 9$ Hz and the amplitude $A_v = 2, 4, 6$ mm are determined. The amplitude reference position is the horizontal displacement distance at the loading point of the pile top exciter. (3) Before the test, the important parameters such as the Froude number Fr , KC number, U_{cw} number, and Shields number θ of each wave–flow condition are calculated, and it is confirmed that they are within a reasonable range. In order to make the difference between each working condition obvious and facilitate data processing and analysis, the wave height $H = 5.4, 8.7, 11.2, 8.3, 9.1$ cm, the wave period $T = 1, 1.3, 1.6$ s, in order to prevent breaking during wave propagation; all wave parameters meet the following requirements of Equation (1). Moreover, the flow velocity of the reference point at $1D$ from the bed surface under the flow condition is $U_c = 0.213$ m/s. (4) Since the

construction environment of offshore wind turbines is an offshore area, the water depth is much larger than the pile diameter of offshore wind turbines. Therefore, the water depth of the test is selected as a fixed value $h = 50$ cm, which is similar to the actual environment of offshore wind turbines.

$$H/L \leq 0.142 \tanh(kh), \quad (1)$$

In the formula: H is the wave height; h is the water depth; L is the wavelength; k is the wave number.

2.3. Experimental Scheme

The test is an experimental study on the local scour characteristics of a monopile under the combined action of wave–flow–vibration. The local scour of a monopile under the action of wave–flow is a comparative test of local scour of a monopile under the combined action of wave–flow–vibration. The test has a total of five groups, and the test duration is 3.5 h. The local scour test of a monopile under the combined action of wave–current–vibration is the main research objective of this paper, which is divided into the following two parts: one group of local scour tests of the monopile under the action of current–vibration, 25 groups of local scour tests of the monopile under the action of wave–current–vibration. Before the start of the test, the flow velocity, wave height, wave period, vibration frequency and amplitude were calibrated, and the calibration files of each working condition were saved to control the exciter, wave-making, and flow-making system.

During the test, the development of scour depth was recorded by reading the scale around the pile and the critical scale of the sand bed. The scale was read every 1 min at 10 min before the start, every 5 min at 10 min~30 min, every 10 min at 30 min~120 min, every 20 min at 120 min~210 min, and every 30 min after 210 min. After the completion of the test and at the characteristic time points, shading cloth is used to cover the treatment, and then the underwater terrain scanner is used to scan and collect the sand bed terrain. Each scanning position is kept in the same position to ensure that it will not affect the scanning results. Finally, the required terrain is obtained by processing.

2.3.1. Wave–Current Test Conditions

Table 1 is the wave parameter table, and Table 2 is the working conditions table of the local scour test of a monopile under the combined action of wave and current. The wave–current tests were carried out with five wave heights, three wave periods, one flow rate, a total of six working conditions, including one group of pure flow conditions and five groups of wave–current conditions. The parameters in the table are obtained as follows:

Table 1. Table of wave parameters.

Working Condition	Wave Height H/cm	Wave Period T/s	KC	U_{wm}	θ	θ/θ_c
1	5.4	1.3	1.07	0.074	0.012	0.321
2	8.7	1.3	1.78	0.123	0.024	0.639
3	11.2	1.3	2.23	0.155	0.035	0.930
4	8.3	1.6	2.45	0.138	0.027	0.724
5	9.1	1	0.96	0.087	0.014	0.382

Table 2. Wave–current test conditions table.

Working Condition	Incidence Velocity $U_c/\text{m}\cdot\text{s}^{-1}$	Wave Height H/cm	Wave Period T/s	KC	U_{cw}	Fr	S/cm	S/D	θ	θ/θ_c
1	0.213	0	0	0	0	0	7.5	0.83	0.014	0.38
2	0.213	5.4	1.3	1.07	0.74	0.28	7.7	0.86	0.026	0.70
3	0.213	8.7	1.3	1.78	0.63	0.31	7.3	0.81	0.038	1.02

Table 2. Cont.

Working Condition	Incidence Velocity $U_c/\text{m}\cdot\text{s}^{-1}$	Wave Height H/cm	Wave Period T/s	KC	U_{cw}	Fr	S/cm	S/D	θ	θ/θ_c
4	0.213	11.2	1.3	2.23	0.58	0.33	7.2	0.8	0.049	1.31
5	0.213	8.3	1.6	2.45	0.61	0.32	7.5	0.83	0.041	1.1
6	0.213	9.1	1	0.96	0.71	0.29	7.7	0.86	0.028	0.96

U_c is the reference point velocity at 9 cm from the sand bed surface (1D position) under the action of water flow, U_{wm} is the maximum velocity of the reference point at 9 cm from the sand bed surface (1D position) under the action of wave, S is the maximum measured scour depth on the side of the monopile, and S/D is the relative scour depth. In the table, KC , U_{cw} , and Fr are dimensionless parameters. Among them, the Froude number Fr is one of the important parameters affecting the local scour of the monopile foundation. The KC number is one of the important parameters describing the local scour process of pile foundation under wave action, which shows the relative ratio relationship between viscous force and inertial force. U_{cw} is a dimensionless number reflecting the relative strength of water flow velocity and wave flow velocity under the combined action of wave and current. The calculation method of each parameter is as follows: $KC = U_{wm}T/D$, $U_{cw} = U_c/(U_c + U_{wm})$, $Fr = U_a/\sqrt{gD}$. U_a is the 1/4 period water particle velocity at the reference point under the combined action of wave and current, and its calculation formula is:

$$U_a = \frac{1}{T/4}U_a = \frac{4}{T}\int_0^{T/4}\left(U_c + U_{wm}\sin\left(\frac{2\pi t}{T}\right)\right)dt = U_c + \frac{2}{\pi}U_{wm}, \quad (2)$$

θ is the Shields number, which reflects the ratio of the force of water flow to the resistance of bed sand to movement. θ_{cr} is the critical Shields number, which is the starting parameter of sediment. It can distinguish between clear water erosion and mobile bed erosion, and its calculation formula is:

$$\theta = \frac{U_f^2}{g(\rho_s/\rho_w - 1)d_{50}}, \quad (3)$$

$$\theta_{cr} = \frac{0.3}{1 + 1.2D_*} + 0.055[1 - \exp(-0.02D_*)], \quad (4)$$

In the calculation formula, U_f is the maximum frictional velocity at the reference point, and D_* is defined as the dimensionless sediment particle size, which is defined as $D_* = d_{50}[(s - 1)g/\nu^2]^{1/3}$, where s is the sand–water density ratio, ν is the kinematic viscosity coefficient, and d_{50} is the median particle size. The critical Shields number under the wave–current and wave action conditions of this test is calculated by Equation (4).

The wave–current test conditions in Table 2 include both clear water erosion and mobile bed erosion. When $\theta/\theta_c < 1$, only the local sediment of the monopile moves, and this condition is clear water erosion. When $\theta/\theta_c > 1$, the sand bed indicates that the sediment is generally starting to move, and this condition is the mobile bed erosion.

2.3.2. Wave–Current–Vibration Test Conditions

Table 3 is the working conditions table of multi-stage vibration and wave–current load alternating development, and Table 4 is the working conditions table of monopile local scour tests under the combined action of wave–current–vibration. The wave–current–vibration tests carried out one group of local brush tests of a monopile under the combined action of current and vibration, 25 groups of local scour tests of a monopile under the combined action of wave–current–vibration, and two groups of multi-stage vibration and wave–current load alternating development test conditions, a total of 28 groups of test

conditions. This test condition explores the local scour law of monopiles under wave–current–vibration, and provides data for the fitting formula of local scour depth of a monopile under wave–current–vibration.

Table 3. Multi-stage vibration and wave–current load alternating development conditions.

Working Condition	Incidence Velocity $U_c/\text{m}\cdot\text{s}^{-1}$	Vibration Frequency f_v/Hz	Amplitude A_v/mm	Wave Height H/cm	Wave Period T/s
1	0.213	6	4	0	0
2	0.213	5	5	8.7	1.3

Table 4. Wave–current–vibration test conditions.

Working Condition	Incidence Velocity $U_c/\text{m}\cdot\text{s}^{-1}$	Vibration Frequency f_v/Hz	Amplitude A_v/mm	Wave Height H/cm	Wave Period T/s	S/cm	S/D
1	0.213	6	4	0	0	4.1	0.456
2	0.213	3	4	5.4	1.3	5.4	0.6
3	0.213	6	4	5.4	1.3	4.5	0.5
4	0.213	9	4	5.4	1.3	4.1	0.456
5	0.213	6	2	5.4	1.3	4.4	0.489
6	0.213	6	6	5.4	1.3	3.2	0.356
7	0.213	3	4	8.7	1.3	5.1	0.567
8	0.213	6	4	8.7	1.3	4.6	0.589
9	0.213	9	4	8.7	1.3	4	0.444
10	0.213	6	2	8.7	1.3	4.9	0.544
11	0.213	6	6	8.7	1.3	4.4	0.489
12	0.213	3	4	11.2	1.3	6.3	0.7
13	0.213	6	4	11.2	1.3	6	0.667
14	0.213	9	4	11.2	1.3	5.75	0.689
15	0.213	6	2	11.2	1.3	6.85	0.761
16	0.213	6	6	11.2	1.3	5	0.556
17	0.213	3	4	8.3	1.6	5.93	0.659
18	0.213	6	4	8.3	1.6	4.6	0.511
19	0.213	9	4	8.3	1.6	4.2	0.467
20	0.213	6	2	8.3	1.6	5.7	0.633
21	0.213	6	6	8.3	1.6	4.4	0.489
22	0.213	3	4	9.1	1	5.8	0.644
23	0.213	6	4	9.1	1	4.7	0.522
24	0.213	9	4	9.1	1	4	0.444
25	0.213	6	2	9.1	1	5.2	0.578
26	0.213	6	6	9.1	1	4.1	0.456

3. Study on Scour Characteristics around a Monopile under Wave–Current–Vibration Interaction

3.1. The Variation Law of Wave Height and Reference Point Velocity

The regular wave conditions used in the test are as follows: when the wave period is 1.3 s, the wave heights are 5.4 cm, 8.7 cm, and 11.2 cm, respectively; when the wave period is 1.6 s, the wave height is 8.3 cm; when the wave period is 1 s, the wave height is 9.1 cm. The undisturbed flow velocity at the distance of $1D$ (9 cm) from the bed surface is taken as the reference point velocity [13]. The horizontal velocity at this reference point represents the velocity of the water near the bottom. The measured velocity at the reference point of each working condition of the wave is: 0.074 m/s, 0.123 m/s, 0.155 m/s, 0.138 m/s, 0.087 m/s. The pure flow only uses one velocity, and the measured velocity at the reference point is 0.213 m/s. In the test, an ADV current meter is used to measure the flow velocity of the reference point, and the contact wave height meter is used to measure the wave height. The pre-experiment is carried out before the test, and it is determined that the

wave-making system and the current-making system can stabilize the wave-making and current-making for a long time to meet the accuracy required for the test.

3.1.1. The Variation Law of Velocity and Wave Height at the Reference Point under Wave Action

- Water particle trajectory at the reference point;

The small amplitude wave theory holds that the trajectory of water particles at each position is an ellipse under finite water depth, and its long half-axis is defined as $a = A \cos kh(Z_0 + h) / \sin kh$. The short half-axis is defined as $b = A \sin kh(Z_0 + h) / \sin kh$, A is the amplitude of the water point, and the long and short half-axes of the water point trajectory gradually decrease from the water surface to the bed surface. Therefore, the parameters of the water particle trajectory at the reference point of each working condition of the wave in this experiment are shown in Table 5.

Table 5. Parameter table of water particle trajectory at the reference point.

Wave Period T/s	Wave Height H/cm	Long Half-Axis a/cm	Short Half-Axis b/cm	Moving Trajectory
1.3	5.4	1.530	0.367	ellipse
1.3	8.7	2.468	0.591	ellipse
1.3	11.2	3.174	0.761	ellipse
1.6	8.3	3.496	0.635	ellipse
1	9.1	1.241	0.443	ellipse

- Wave height curve and horizontal velocity change at the reference point;

The free surface curve of the pure wave under various working conditions is shown in Figure 2 below. It can be seen from the figure that the wave crest, wave period, wave trough, and other forms are relatively regular under the wave conditions used in the test. Because the size of the wave flume used in the test is large, the pile diameter is relatively small, and the reflection of the wave is less. Therefore, after a long time of wave loading, the waveform and parameters can still be maintained well, so that the test can carry out a long time of wave loading.

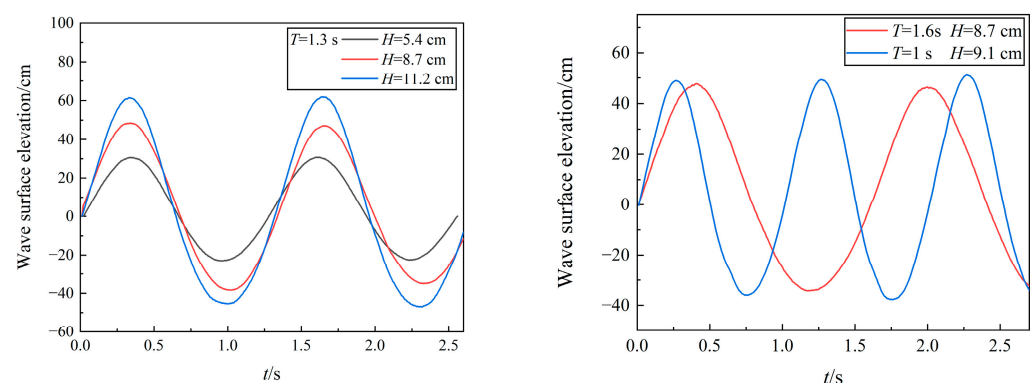


Figure 2. Free surface curve of pure wave condition.

The horizontal velocity at the reference point of each working condition under the action of waves is shown in Figure 3. It can be seen from the diagram that the phase of the horizontal velocity at the reference point is the same as that of the free surface curve, the velocity changes periodically, the peak velocity is the same as the trough velocity, and the direction is opposite. The trajectory of the water particle is elliptical, which conforms to the small amplitude wave theory. Figure 4 shows the comparison between the calculated value and the theoretical value of the wave reference point flow velocity. It can be seen from the

figure that the theoretical value of the test flow velocity is relatively close to the calculated value, and the test conditions meet the requirements.

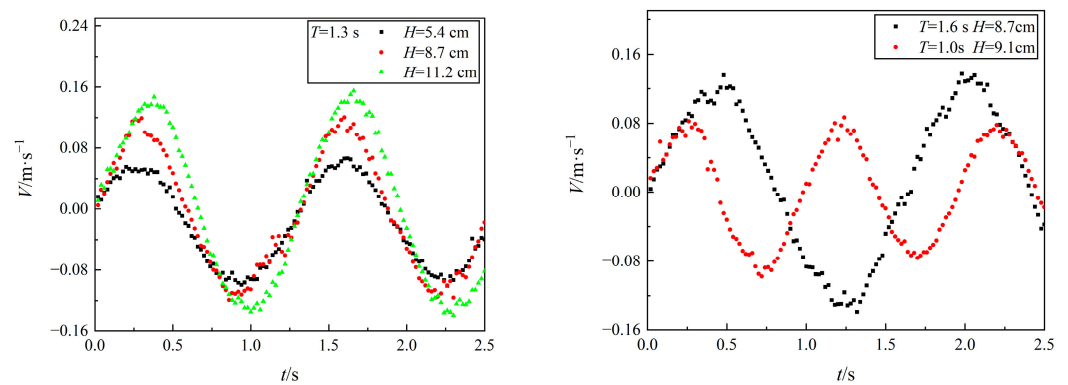


Figure 3. The horizontal velocity at the reference point of wave condition.

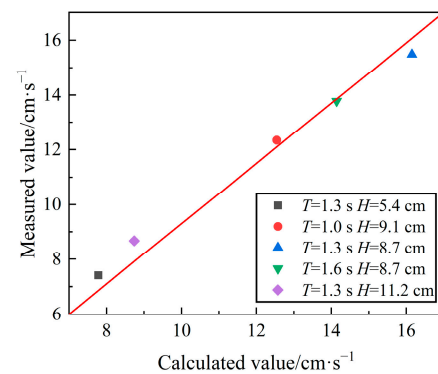


Figure 4. The calculated value of wave reference point velocity is compared with the theoretical value.

3.1.2. The Variation Law of Velocity and Wave Height at the Reference Point under the Action of Wave–Current

The free surface curve under various working conditions of the wave–current is shown in Figure 5 below. It can be seen from the diagram that after the wave superposition, for the free surface curve, compared with the pure wave under the same working condition, the peak and trough are obviously flatter, the wave height is reduced, and the wave period is increased, but the waveform can still be maintained well, so the wave–current test can be loaded for a long time.

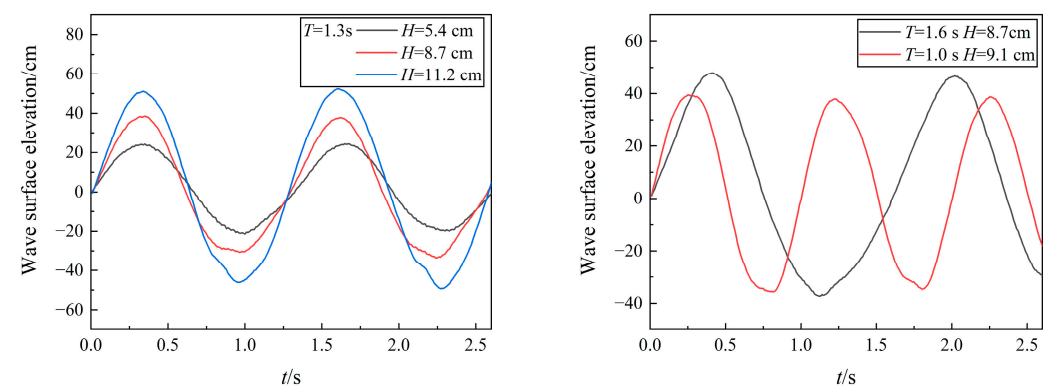


Figure 5. Free surface curve of wave–current condition.

The change of horizontal velocity at the reference point under various working conditions of wave–flow is shown in Figure 6. Comparing Figure 5 with Figure 6, it can be seen that the horizontal velocity at the reference point is the same as the phase of the free surface curve under the combined action of waves. The maximum horizontal velocity in the figure reaches 0.357 m/s, and the minimum velocity is 0.052 m/s. Because the flow velocity is 0.213 m/s, which is much larger than the wave horizontal velocity, there is no reverse velocity, and the peak velocity and the trough velocity are symmetrically distributed along the flow velocity of 0.213 m/s. By adding the flow velocity at the reference point to the flow velocity, it can be found that the wave–current velocity is not a simple linear addition of the flow velocity under the action of water flow and waves, which is consistent with the phenomenon found by Qi [13]. The superposition of water flow on the wave action will increase the size and strength of the horseshoe vortex around the pile, and reduce the critical KC number of the horseshoe vortex, so that the horseshoe vortex also exists at a smaller KC number, and is accompanied by upward pore water pressure generated by the trough, which makes the initial transport of sediment easier, and ultimately affects the scouring process around the pile.

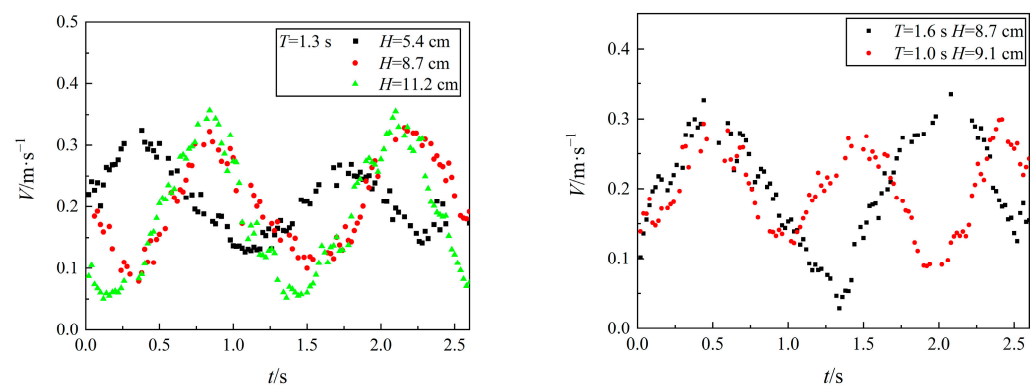


Figure 6. The horizontal flow velocity at each reference point in the wave–flow condition.

3.2. Study on the Development of the Local Scour Depth of a Monopile under the Combined Action of Wave–Current–Vibration

Due to the difference in wave height and wave period between the test conditions, the hydrodynamic characteristics around the monopile are different. The difference in vibration frequency and amplitude leads to a difference in sediment particle movement around the monopile, which leads to the great difference in the scouring process. The most obvious difference is in the development process of local scour depth. The change trend of local scour depth with time is a gradual process, which is one of the important indexes to reflect the scour development characteristics of a monopile under certain flow conditions and vibration loads. Before the test, a scale is pasted on the side wall of the monopile to facilitate the reading of the scour depth. During the test, the scale is read according to the plan, and the scale data are statistically processed after the test to obtain the erosion duration development curve. In this section, the influence of vibration parameters on local scour depth under different wave–current conditions is analyzed, and the difference between clear water scour and live-bed scour is compared. The development characteristics of local scour depth under wave–current–vibration are analyzed in detail.

3.2.1. The Influence of Vibration Frequency on the Duration of Scour Depth Development

Figure 7 is the local scour time curve of the monopile caused by different vibration frequencies under the action of wave–current–vibration. It can be clearly seen from the diagram that the local scour development model of the monopile under different vibration frequencies can also be divided into a rapid development stage, a stable development stage, and a balanced development stage. Under the same wave–current conditions, the equilibrium scour depth and scour rate of the monopile are the largest when the vibration frequency is small, reaching $0.66D$ at $t = 210$ min. The scour depth under the vibration load of f_6a_4 is $0.511D$, and the scour depth under the vibration load of f_9a_4 is $0.467D$, while the final scour depth of pure wave–current without vibration load is $0.833D$, which indicates that the vibration load will reduce the quasi-equilibrium scour depth, and with the increase in vibration frequency, the quasi-equilibrium scour depth will gradually decrease. This is because the increase in vibration frequency will increase the backfill efficiency of the sand bed around the monopile, resulting in a decrease in the quasi-balanced scour depth [23].

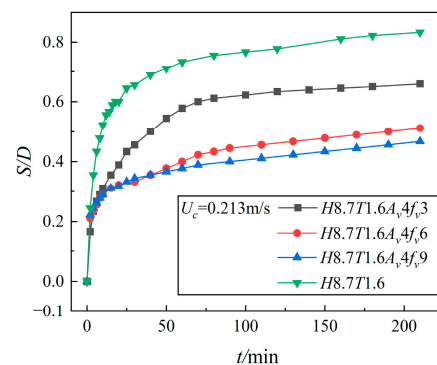


Figure 7. Effect of vibration frequency on scour duration.

3.2.2. The Influence of Amplitude on the Duration of Scour Depth Development

Figure 8 is the local scour time curve of the monopile caused by different amplitudes under the action of wave–current–vibration. It can be clearly seen from the figure that the influence of amplitude on the scour development curve is similar to that of vibration frequency. As the vibration frequency increases, the quasi-equilibrium scour depth decreases. This is because the increase in the amplitude will increase the influence of the sand bed around the monopile, which will increase the backfill efficiency of the scour hole and reduce the quasi-equilibrium scour depth [23].

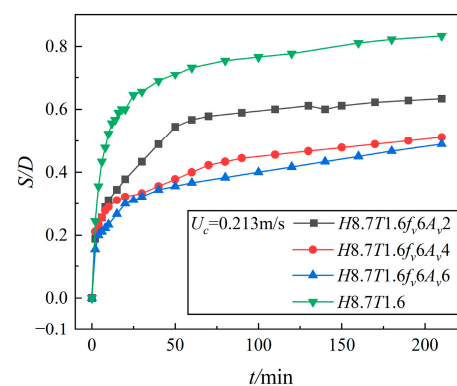


Figure 8. Effect of amplitude on scour duration.

3.3. Development Process and Morphological Characteristics of the Scour Hole

The exploration of the development law of local scour holes changing with time under the combined action of wave–current–vibration is an important part of studying the scour mechanism. In this paper, through the scour test under the action of convection–vibration and the scour test under the action of wave–current–vibration, $t = 20$ min, 60 min, 210 min, and 450 min are selected as typical moments. At typical moments, the topographic data are scanned by the underwater topographic scanner, and the topographic map is obtained after processing. The development process of the scour hole and the shape of the scour hole are qualitatively described through the topographic map, so as to obtain the characteristics of the local scour development process of the monopile under the action of current–vibration and wave–current–vibration. Taking $U_c = 0.213$ m/s, $f_v = 6$ Hz, $A_v = 4$ mm (working condition 1 in Table 3), and $U_c = 0.213$ m/s, $H = 8.7$ cm, $T = 1.3$ s, $f_v = 6$ Hz, $A_v = 4$ mm (working condition 8 in Table 4) as examples, the development process and morphological characteristics of local scour of the monopile under convection–vibration and wave–current–vibration are analyzed, respectively.

In the process of local scour of a monopile under wave–current load, horseshoe vortex, and descending flow are the main factors causing local scour around the pile. It can be seen from Figure 9 below that under the action of pure flow, the forward water flow moves to the front of the pile. Under the obstruction of the pile, the velocity potential energy of the water flow is transformed into pressure potential energy, and the flow velocity in the vertical direction of the water flow presents a logarithmic distribution. The resulting pressure gradient also shows a near-logarithmic distribution with a large pressure potential energy on the upper part of the monopile and a small pressure potential energy on the lower part. Under the action of this pressure difference, a downward flow is formed. When the downward flow passes through the monopile, the flow section decreases due to the existence of the monopile, and the flow will accelerate to bypass the monopile. The downward flow will touch the bottom in front of the pile, accompanied by the movement of the flow, forming a circular horseshoe vortex, rotating around the bottom of the monopile, carrying a large amount of sediment to the downstream movement. The flow field structure around the monopile under the action of wave and current is shown in Figure 10. Similar to the flow field under the action of pure current, it is also composed of a descending flow in front of the pile, an annular horseshoe vortex at the bottom of the ring pile and a trailing vortex behind the pile. However, due to the obvious periodicity of the wave, the intensity of the vortex formed is also cyclical, and the strength of the sediment around the monopile is also cyclical. In the process of local scour of the monopile under the combined action of current–vibration or wave–current–vibration in this experiment, there is not only scour under the action of the above-mentioned falling flow and vortex, but also densification of sediment and ratchet convection. The action of vibration has an effect on the formation and strength of the falling flow and vortex, so this section discusses the local scour characteristics of the monopile under this complex action.

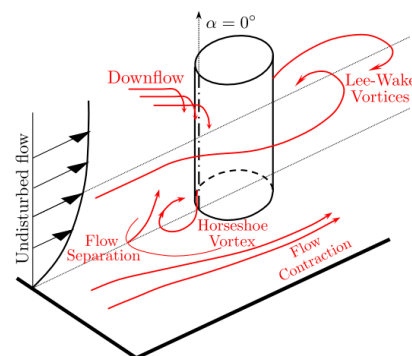


Figure 9. Flow field diagram around a monopile under pure flow [26].

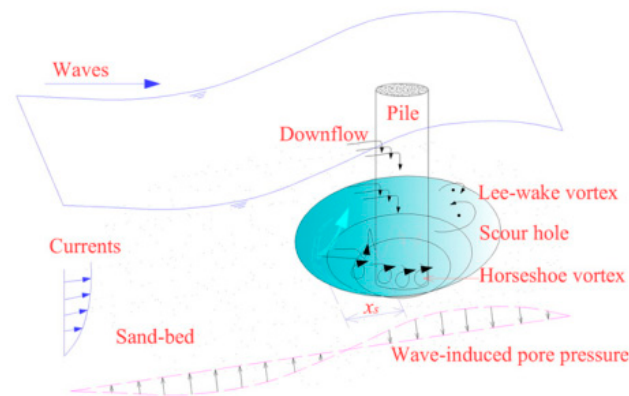


Figure 10. Flow field diagram around a monopile under wave–current interaction [13].

3.3.1. The Development Process of the Scour Hole under the Combined Action of Current and Vibration

Under the influence of a horseshoe vortex and falling water flow, the sediment around the monopile will move, resulting in some sediment being transported to the pile by the vortex. During the transportation process, the sediment will gradually settle with the decrease in the strength of the wake vortex, and gradually accumulate behind the pile to form a large dune. There is only a ring-like scour hole in front of the pile without deposition, and the area behind the pile is alternately changed with erosion and deposition. Under the combined action of current and vibration, in addition to the influence of the horseshoe vortex and falling water flow, it is also affected by the densification and ratchet convection motion under the action of vibration load, resulting in the sand bed sediment around the monopile sliding into the scour hole, which will affect the scouring process around the monopile under the action of water current.

Figure 11a–d is the topographic map at typical times of the sand bed around the monopile under the action of $U_c = 0.213$ m/s, $f_v = 6$ Hz, $A_v = 4$ mm (working condition 1 in Table 3). In the first 20 min of the scour, the scour around the monopile develops rapidly, and the sediment transport rate is high at this time. A large amount of sediment moves rapidly to the back of the pile under the action of the horseshoe vortex and tail vortex to settle, forming a sand ripple connected with erosion and deposition. The vibration load causes the sediment around the monopile to slide into the scour hole, increasing the width of the scour hole and slowing down the slope of the scour hole. Under the coupling effect of water current and vibration load, the scour hole topography shown in the figure is formed. When $t = 60$ min, the scour depth around the monopile increases slowly, while the width of the semi-annular-like scour hole increases rapidly, and the height of the dune behind the pile also increases further. The maximum scour depth is in front of the pile. When $t = 210$ min, the development of scour depth and width slows down, and the development of the scour hole is basically mature. At this stage of 210 min~450 min, the scour depth develops slowly, the width of the scour hole increases slowly, and the scour depth reaches a quasi-equilibrium state.

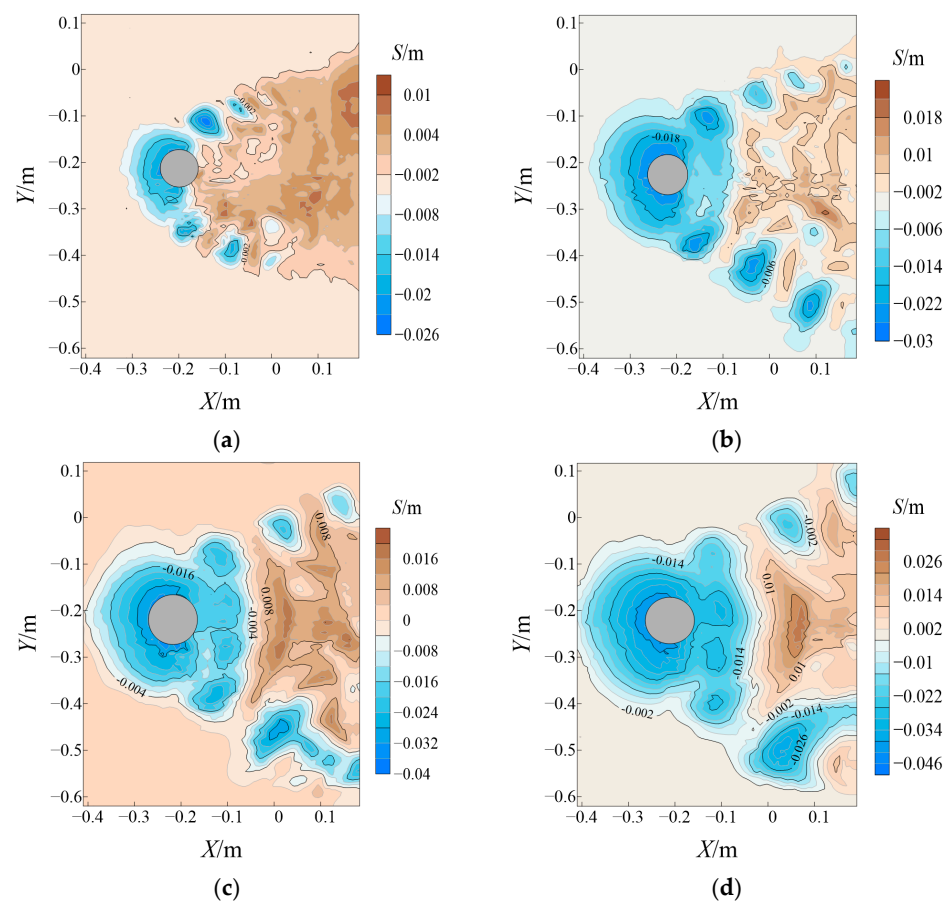


Figure 11. The development process of the scour hole under the action of current and vibration: (a) $t = 20$ min; (b) $t = 60$ min; (c) $t = 210$ min; (d) $t = 450$ min.

3.3.2. The Development Process of the Scour Hole under the Action of Current and Vibration

Figure 12a–d is the typical time topographic map of the sand bed around the monopile under the action of $U_c = 0.213$ m/s, $H = 8.7$ cm, $T = 1.3$ s, $f_v = 6$ Hz, $A_v = 4$ mm (working condition 8 in Table 4). Under the combined action of wave–current–vibration, the scour hole around the monopile develops rapidly under the action of periodic sand lifting of waves, sediment carrying of water current, densification of vibration load, and ratchet convection. In the first 20 min, the erosion around the monopile develops rapidly. It can be observed that the sediment around the monopile is rolled up under the action of the vortex, and continues to move backward under the action of the water flow and wake vortex. At $t = 60$ min, it can be clearly observed that the maximum scour depth is in front of the pile, the width of the scour hole develops rapidly, and the dunes behind the pile move obviously backward. When $t = 210$ min, the sand bed appears as a whole, and the scour hole also develops from a semi-ring-like shape in front of the pile to a ring-like shape around the pile. This is because $\theta/\theta_c = 1.02 > 1$ in this working condition, which belongs to the live-bed scour, but it is close to the critical Shields number, so the formation time of sand ripple is slow. At $t = 450$ min, the scour around the pile reaches a quasi-equilibrium state. From Figure 12d, it can be observed that the height of the dune decreases significantly. This is because under the action of wave–current, the dune behind the pile gradually moves backward, and the main peak position is outside the topographic map. The scour terrain in the quasi-equilibrium state is highly symmetrical, and the maximum scour hole is located on the side of the pile.

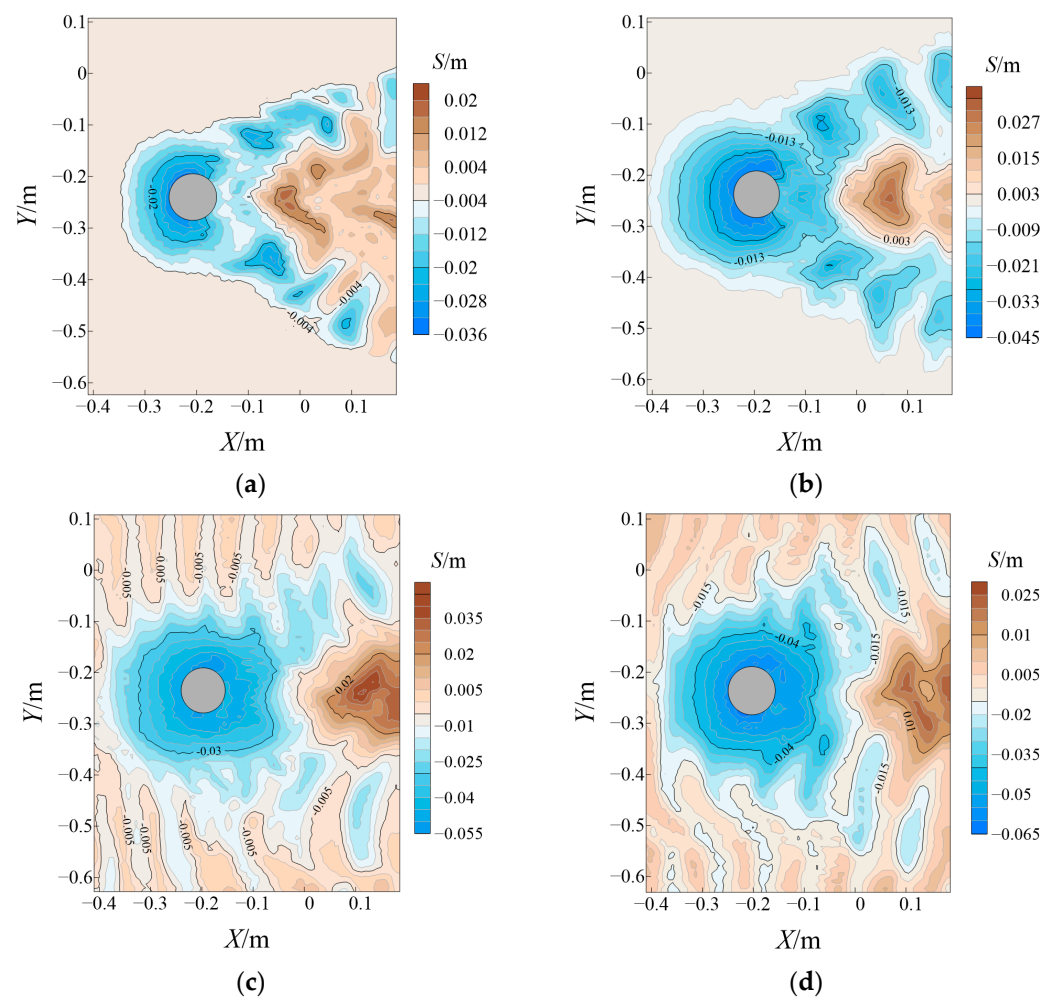


Figure 12. The development process of the scour hole under the combined action of wave–current–vibration: (a) $t = 20$ min; (b) $t = 60$ min; (c) $t = 210$ min; (d) $t = 450$ min.

3.3.3. The Development Process of the Scour Hole under the Action of Current and Vibration

Figures 13 and 14 are the longitudinal profiles of the scour hole parallel to the current direction and passing through the center of the monopile under the combined action of current–vibration and wave–current–vibration, respectively. It can be seen from Figure 13 that with the continuous loading of current–vibration, the depth and width of the scour hole around the monopile and the height of the dune behind the tail continue to increase, and the sand peak of the dune behind the tail continues to move downstream. With the continuous loading of current–vibration, although the depth of the scour hole continues to develop, the shape of the front wall at each time during the development process is relatively consistent with the slope. And the shape of the back wall is also relatively consistent, but the back wall will continue to steepen during the development process. This is due to the gradual backward movement and increase in the sand dunes behind the tail, so that the effect of ratcheting intensity on the sand dunes on the back wall is reduced. Therefore, with the development of loading time, the slope of the front wall of the pile remains relatively unchanged, and the slope of the back wall of the pile continues to increase. It can be seen from Figure 14 that under the continuous action of wave–current–vibration load, the development law of the front wall of the scour hole and the back wall of the scour hole is similar to that under the action of current and vibration. The shape of the front wall at each time is relatively consistent with the slope, and the shape of the back wall changes with the development of time. Also, the sand dunes behind the pile are sand ripples, and this is due

to the live-bed scour under the combined action of wave–current–vibration. The back wall of the pile is relatively longer than the front wall, so there are sand ripples.

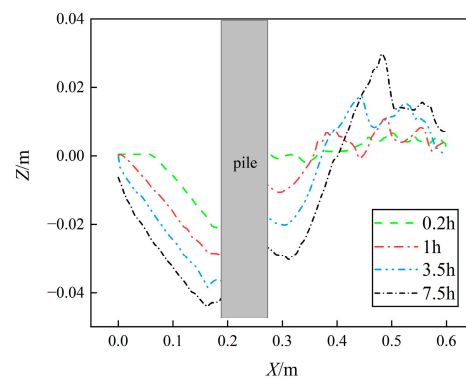


Figure 13. Longitudinal profile of scour hole development under current and vibration action.

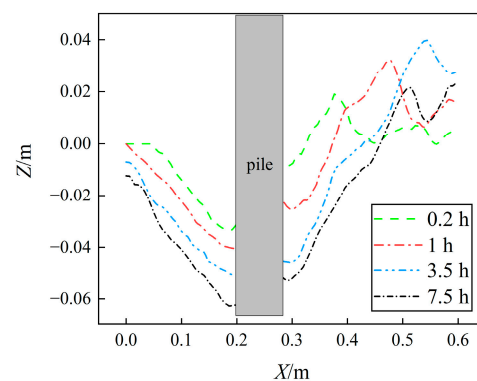


Figure 14. Longitudinal profile of scour hole development under wave–current–vibration.

3.4. The Influence of Vibration Load on the Profile of the Scour Hole

Figure 15 shows the longitudinal section of the scour hole under the influence of different vibration loads under the combined action of wave–current–vibration. It can be observed from Figure 15a that under the same wave–current and vibration frequency conditions, with the increase in vibration amplitude, the width of the scour hole in front of the pile increases significantly, the depth of the scour hole changes little, the slope of the front wall decreases, and the wall behind the pile gradually moves backward with the increase in the amplitude. It can be seen from the whole that the increase in amplitude will significantly increase the width of the scour hole, and with the increase in amplitude, the height of the dune behind the pile shows a step-like increase, and the dune as a whole shows a streamlined structure. It can be observed from Figure 15b that the influence of vibration frequency on the scour hole around the pile shows different characteristics from the amplitude. Under the same wave–current and amplitude conditions, with the increase in vibration frequency, the scour hole in front of the pile is obviously backfilled, the depth of the scour hole is obviously reduced, the width of the scour hole changes little, and the slope of the front wall is obviously slowed down. When the sand dunes behind the pile are acted on at a small frequency, the shape of the back wall remains highly consistent. With a continuous increase in the vibration frequency, the sand dunes behind the pile are affected by the vibration, and the sand dunes near the pile are obviously collapsed, and the slope of the back wall is obviously slowed down. It can be known as a whole that an increase in vibration frequency will backfill the scour hole and significantly reduce the depth of the scour hole.

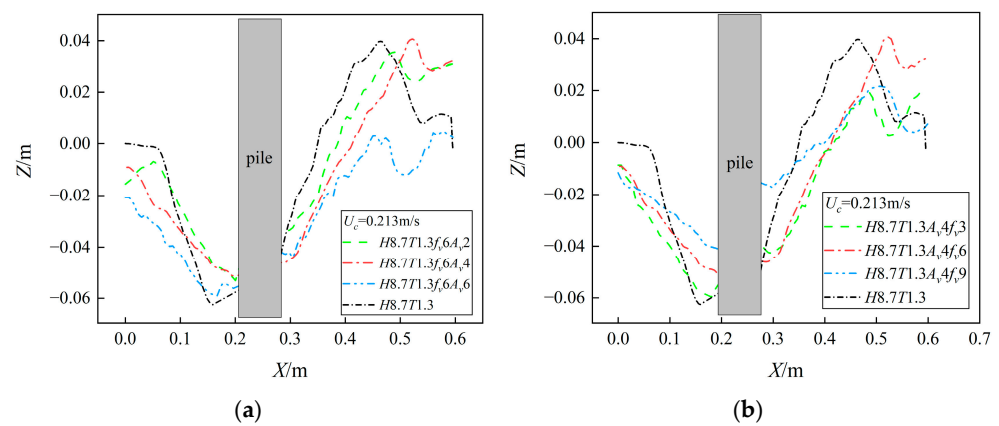


Figure 15. Effect of vibration load on scour hole profile: (a) Longitudinal profile of scour hole under different amplitudes; (b) Longitudinal profile of scour hole under different vibration frequencies.

4. Analysis of Influencing Factors on the Local Scour Depth of a Monopile under the Combined Action of Wave–Current–Vibration

4.1. The Influence of Vibration Intensity on the Maximum Scour Depth

Vibration frequency and amplitude have an effect on the local scour depth of a monopile. The vibration frequency and amplitude of offshore wind turbines also change during normal operation. Therefore, a comprehensive parameter for vibration intensity Λ [27,28] is introduced to measure the influence of vibration load on local scour. The calculation formula is as follows:

$$\Lambda = \frac{A_v \times (2\pi f_v)^2}{g}, \quad (5)$$

In this formula: A_v is the vibration frequency; f_v is the amplitude; the value of π is 3.14; g is the gravitational acceleration, which is 9.81 m/s^2 .

Figure 16 shows the relationship between vibration intensity and relative scour depth under different wave conditions. The five curves in the diagram represent five different wave and current conditions. According to the Hilbert number, the scouring test can be divided into two types: live-bed scouring and clear water scouring. It can be seen from Figure 16 that under the condition of clear water scour, the relative scour depth shows a decreasing trend with the increase in vibration intensity. Under the condition of live-bed scour, when $\theta/\theta_c \leq 1.1$, the relationship between relative scour depth and vibration intensity is similar to that of clear water scour. With the increase in the relative Shields number, when $\theta/\theta_c > 1.1$ (red curve and green curve), the relative scour depth increases first, then decreases, and finally increases with the increase in vibration intensity.

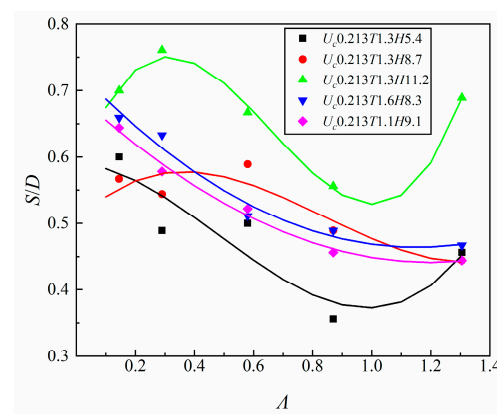


Figure 16. The influence of vibration intensity on the maximum scour depth.

4.2. The Influence of the Froude Number Fr on the Maximum Scour Depth

The Froude number Fr is one of the important parameters affecting the local scour of a monopile foundation. The Froude number under wave and current conditions is defined as $Fr = U_a / \sqrt{gD}$; this parameter is closely related to the flow field structure (horseshoe vortex, wake vortex) around the monopile under the combined action of wave–current–vibration. The local relative scour depth of a monopile under wave–current–vibration is fitted with the test data of Qi and Gao [13] and Sumer [29,30], and the curve shown in Figure 17 is obtained. After the dimensionless processing of the test data, it is found that the test data are located below the fitting curve, but the growth law is similar to the fitting curve. The relative scour depth S/D shows an increasing trend with the increase in the Fr number, and the backfill caused by vibration is the main reason for the test data to be located below the fitting curve.

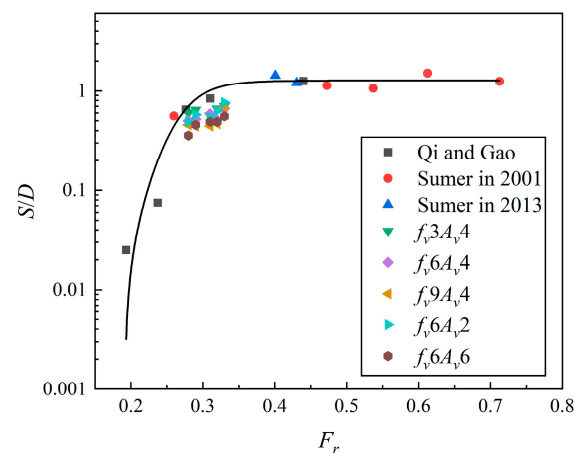


Figure 17. Relationship between Fr and relative scour depth [13,29,30].

4.3. The Influence of the KC Number on the Maximum Scour Depth

The Keulegan–Carpenter (KC) number is one of the important parameters to describe the local scour process of pile foundation under wave action, which shows the relative ratio between viscous force and inertial force. The change in KC reflects the change degree in wave height, so the local scour depth of the monopile in this test should be in a functional relationship with KC. Figure 18 is a fitting curve of relative scour depth S/D with KC number under different U_{cw} by fitting the data of Sumer [29,30], Qi and Gao [13], Rudolph and Bos [31]. The test data are classified according to three intervals: $U_{cw} = 0.4–0.5$, $U_{cw} = 0.5–0.7$, $U_{cw} = 0.7–0.85$. There are 25 groups of working conditions under the combined action of wave–current–vibration in this test. U_{cw} can be divided into two intervals, which are classified and represented in Figure 18. It can be seen from the figure that all the relevant data are located near the corresponding curve, but it can be clearly observed that as the vibration frequency and amplitude increase, the corresponding data points gradually shift downward and fall below the curve. This is because as the vibration frequency and amplitude increase, the densification of the sand bed around the pile and the ratcheting convection movement causes the backfill effect of the scour hole to increase [23], resulting in a decrease in the relative scour depth. The resulting data points deviate from the curve, but the overall data trend with KC. The growth trend is basically consistent with the fitting curve, and it shows an increasing trend as the KC number increases.

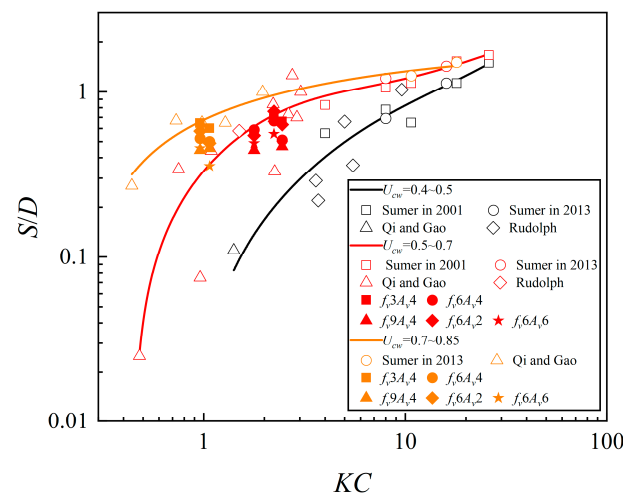


Figure 18. Relationship between KC number and relative scour depth [13,29–31].

4.4. Effect of U_{cw} on the Maximum Scour Depth

U_{cw} is a dimensionless number reflecting the relative strength of current velocity and wave–current velocity under the combined action of wave and current. It can be defined as $U_{cw} = U_c / (U_c + U_{wm})$, where U_c is the current velocity at the current reference point, and U_{wm} is the maximum horizontal velocity of the water particle at the wave reference point. Figure 19 shows the curves of relative scour depth S/D with U_{cw} under different vibration load parameters. The distribution of five curves with different colors in the figure represents the scour test under five groups of vibration load parameters. According to the Shields number, the scour test can be divided into two types: live-bed scour and clear water scour. It can be seen from the diagram that under the action of vibration load, with the increase in U_{cw} , the relative scour depth under clear water scour and live-bed scour shows a downward trend.

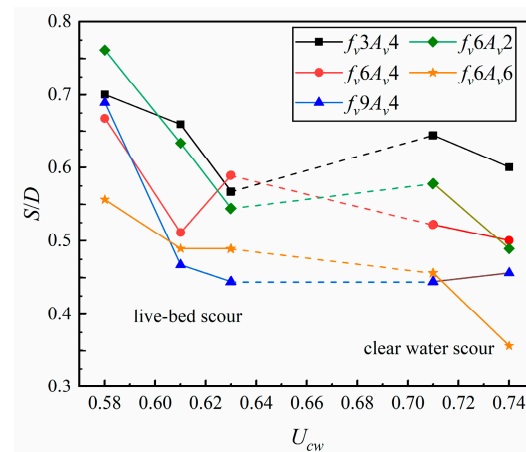


Figure 19. Relationship between U_{cw} and relative scour depth.

4.5. Empirical Formula of Local Scour Depth of a Monopile under Combined Action of Wave–Current–Vibration

There are many empirical equations for predicting the local scour depth of a monopile under the action of wave and current. The commonly used equation is the large KC number equation ($KC \geq 6$) (Equation (6)) proposed by Sumer [32]. Recently, Dogan [33] modified the local scour equations of fine piles and large-diameter piles under wave action. Compared with the estimation formula of local scour depth under pure wave action, the local scour depth of a pile foundation under wave–current action is also closely related to various dimensionless parameters such as the Fr number and U_{cw} . Qi and Gao [34] summarized

the empirical equation for predicting the local scour depth of a monopile based on multiple sets of test data ($0.4 < KC < 26$) (Equation (7)). However, the above empirical formula only considers the wave and water current, and lacks the empirical formula of local scour depth of a pile foundation under vibration load.

$$\frac{S}{D} = 1.3\{1 - \exp[-0.03(KC - 6)]\}, \quad (6)$$

$$\lg\left(\frac{S}{D}\right) = -0.8 \exp\left(\frac{0.14}{Fr}\right) + 1.11, (0.1 < Fr < 1.1, 0.4 < KC < 26), \quad (7)$$

Whitehouse [24] believed that the local scour depth of a pile foundation is related to the current conditions, sediment characteristics, pile size, water depth, and other factors. In this research, the monopile foundation under vibration load is taken as the research object, the vibration strength Λ is introduced, and the dimensionless parameter equation of scour depth S is established by Equation (8).

$$S = f(H, T, U_c, U_{wm}, d_{50}, D, h, g, \rho_s, \rho_w, f_v, A_v), \quad (8)$$

where S is the maximum scour depth, ρ_s is the volume weight of sediment, ρ_w is the volume weight of water, g is the acceleration of gravity, and the other parameters have been described above.

Within the parameter range covered in this study, further dimensionless processing is performed based on the Pi theorem. The final empirical prediction model is shown in Equation (9). The four dimensionless parameters of the Keulegan–Carpenter number (KC), vibration intensity (Λ), velocity ratio (U_{cw}), and Froude number (Fr) are used as multivariate regression model variables, where k_1, k_2, k_3, k_4, k_5 , and k_6 are the control coefficients of equations and variables. In this paper, a total of five scour depth data points without vibration, only wave–current action, and 25 scour depth data points under the combined wave–current–vibration action of this experiment are used as training data samples for fitting the formula coefficients. The final equation and coefficient values are shown in Equation (10).

$$\frac{S}{D} = \exp(k_1 \cdot KC + k_2 \cdot U_{cw} + k_3) \cdot Fr^{k_4} + k_5 \cdot \Lambda^{k_6}, \quad (9)$$

$$\frac{S}{D} = \exp(0.014 \cdot KC - 0.065 \cdot U_{cw} + 0.962) \cdot Fr^{0.949} - 0.182 \cdot \Lambda^{0.264}, \quad (10)$$

The results of Equation (10) in the case of no vibration are calculated and compared with the results of Equation (7) proposed by Qi and Gao [34] under the test conditions in this paper, and the obtained comparisons are shown in Table 6. From the table, it can be seen that the errors of the two formulas are below 15%, which can indicate that the results of the fitted formulae in the absence of vibration are also reliable.

Table 6. Comparison table of values calculated by Formulas (6) and (9).

Working Condition	KC	U_{wm}	Fr	Calculated by Formula (6) [34]	Calculated by Formula (9)	Data Comparison
1	1.07	0.74	0.28	0.618	0.756	−13.84%
2	1.78	0.63	0.31	0.713	0.847	−13.41%
3	2.23	0.58	0.33	0.771	0.908	−13.66%
4	2.45	0.61	0.32	0.743	0.883	−13.99%
5	0.96	0.71	0.29	0.651	0.782	−13.14%

The error between the measured value of the test and the calculated value of the Equation (10) is shown in Figure 20. It can be seen from the diagram that the formula can better predict the local scour depth of a monopile under the action of wave–current–

vibration. Only a preliminary exploration is carried out on the basis of existing test data. The empirical formula with wider application range and higher precision needs further research in the future.

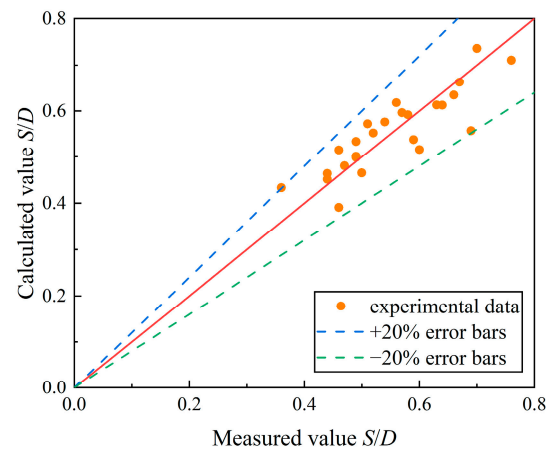


Figure 20. Comparison between calculated values and measured values.

5. Conclusions

In this paper, the experimental study on the local scour of a monopile under the combined action of wave–current–vibration is carried out by arranging the vibration load loading device in the wave–current flume. Firstly, the local scour characteristics of the monopile under the action of wave–current–vibration are tested, and the development of maximum scour depth, the development of the scour hole shape, and the shape of the scour hole profile are analyzed. Further, the influence of vibration intensity Λ and Froude number Fr , KC number, and U_{cw} on the local scour depth of a monopile is analyzed. Finally, the dimensionless influencing factors are analyzed, and an empirical formula for predicting the local scour depth of a monopile under the combined action of wave–current–vibration is fitted. The main conclusions are as follows:

- (1) The maximum scour depth under the combined action of wave–current–vibration will be significantly smaller than the maximum scour depth under wave action. Vibration load will reduce the quasi-equilibrium scour depth, and with the increase in vibration frequency and amplitude, the quasi-equilibrium scour depth around the monopile will gradually decrease. The local scour topography of the monopile under the combined action of current and vibration is highly symmetrical, and the maximum scour depth is on the side of the pile, with the scour hole type presenting a spoon-like shape. As the amplitude increases, the width of the scour hole will increase significantly. As the vibration frequency increases, the backfill will be more obvious, and the maximum scour depth in front of the pile will be significantly reduced.
- (2) Under the combined action of wave–current–vibration, the relative scour depth decreases with the increase in vibration intensity under the scouring of clear water. Under live-bed scour, when $\theta/\theta_c \leq 1.1$, the relationship between relative scour depth and vibration intensity is similar to that of clear water scour. When $\theta/\theta_c > 1.1$, the relative scour depth increases first, then decreases, and finally increases with the increase in vibration intensity.
- (3) S/D is the function of vibration intensity Λ and Froude number Fr , KC number, U_{cw} under the wave–current–vibration interaction. The relative scour depth S/D increases with the increase in Fr and KC , and has the same trend as the vertical pile fitting curve. However, with the increase in U_{cw} , the relative scour depth under clear water scour and live-bed scour shows a downward trend.
- (4) The four dimensionless parameters of the Keulegan–Carpenter number (KC), vibration intensity (Λ), velocity ratio (U_{cw}), and Froude number (Fr) are used as multivariate regression model variables. The prediction formula of the local scour depth of a

monopile under wave–current–vibration is obtained by fitting. A comparison is made between the measured and calculated values of the error in this formula. The results show that this formula can predict the local scour depth of a monopile under wave–current–vibration.

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References

1. He, K.; Ye, J. Seismic Dynamics of Offshore Wind Turbine-Seabed Foundation: Insights from a Numerical Study. *Renew. Energy* **2023**, *205*, 200–221. [\[CrossRef\]](#)
2. Yu, L.-Q.; Wang, L.-Z.; Guo, Z.; Bhattacharya, S.; Nikitas, G.; Li, L.-L.; Xing, Y.-L. Long-Term Dynamic Behavior of Monopile Supported Offshore Wind Turbines in Sand. *Theor. Appl. Mech. Lett.* **2015**, *5*, 80–84. [\[CrossRef\]](#)
3. Cuéllar, P.; Georgi, S.; Baefler, M.; Rücker, W. On the Quasi-Static Granular Convective Flow and Sand Densification around Pile Foundations under Cyclic Lateral Loading. *Granul. Matter* **2012**, *14*, 11–25. [\[CrossRef\]](#)
4. Nanda Kishore, Y.; Narasimha Rao, S.; Mani, J.S. The Behavior of Laterally Loaded Piles Subjected to Scour in Marine Environment. *KSCE J. Civ. Eng.* **2009**, *13*, 403–408. [\[CrossRef\]](#)
5. Shi, S.; Zhai, E.; Xu, C.; Iqbal, K.; Sun, Y.; Wang, S. Influence of Pile-Soil Interaction on Dynamic Properties and Response of Offshore Wind Turbine with Monopile Foundation in Sand Site. *Appl. Ocean Res.* **2022**, *126*, 103279. [\[CrossRef\]](#)
6. Van Der Tempel, J.; Diepeveen, N.F.B.; Salzmann, D.J.C.; De Vries, W.E. Design of Support Structures for Offshore Wind Turbines. In *WIT Transactions on State of the Art in Science and Engineering*; WIT Press: Southampton, UK, 2010; Volume 1, pp. 559–591, ISBN 978-1-84564-205-1.
7. Wang, X.; Li, S.; Li, J. Load Bearing Mechanism and Simplified Design Method of Hybrid Monopile Foundation for Offshore Wind Turbines. *Appl. Ocean Res.* **2022**, *126*, 103286. [\[CrossRef\]](#)
8. Matutano, C.; Negro, V.; López-Gutiérrez, J.-S.; Esteban, M.D. Scour Prediction and Scour Protections in Offshore Wind Farms. *Renew. Energy* **2013**, *57*, 358–365. [\[CrossRef\]](#)
9. Negro, V.; López-Gutiérrez, J.-S.; Esteban, M.D.; Matutano, C. Uncertainties in the Design of Support Structures and Foundations for Offshore Wind Turbines. *Renew. Energy* **2014**, *63*, 125–132. [\[CrossRef\]](#)
10. Luengo, J.; Negro, V.; García-Barba, J.; López-Gutiérrez, J.-S.; Esteban, M.D. New Detected Uncertainties in the Design of Foundations for Offshore Wind Turbines. *Renew. Energy* **2019**, *131*, 667–677. [\[CrossRef\]](#)
11. Wu, M.; Stratigaki, V.; Fazeres-Ferradosa, T.; Rosa-Santos, P.; Taveira-Pinto, F.; Troch, P. Experimental Uncertainty Analysis of Monopile Scour Protection Stability Tests. *Renew. Energy* **2023**, *210*, 174–187. [\[CrossRef\]](#)
12. Sumer, B.M.; Hatipoglu, F.; Fredsøe, J. Wave Scour around a Pile in Sand, Medium Dense, and Dense Silt. *J. Waterw. Port Coast. Ocean Eng.* **2007**, *133*, 14–27. [\[CrossRef\]](#)
13. Qi, W.-G.; Gao, F.-P. Physical Modeling of Local Scour Development around a Large-Diameter Monopile in Combined Waves and Current. *Coast. Eng.* **2014**, *83*, 72–81. [\[CrossRef\]](#)
14. Zhang, B.; Li, J.; Liu, W.; Zhang, H.; Shi, P.; Fu, X. Experimental Study on Dynamic Characteristics of a Monopile Foundation Based on Local Scour in Combined Waves and Current. *Ocean Eng.* **2022**, *266*, 113003. [\[CrossRef\]](#)
15. Gautam, S.; Dutta, D.; Bihs, H.; Afzal, M.S. Three-Dimensional Computational Fluid Dynamics Modelling of Scour around a Single Pile Due to Combined Action of the Waves and Current Using Level-Set Method. *Coast. Eng.* **2021**, *170*, 104002. [\[CrossRef\]](#)
16. Shi, Y.; Cheng, Y.; Huang, X.; Li, Y. Numerical Simulation of Unsteady Flow Field around Monopile under the Combined Influence of Wave and Vibration. *Ocean Eng.* **2023**, *276*, 114238. [\[CrossRef\]](#)
17. Yu, Z.; Cheng, Y.; Cheng, H. Experimental Investigation on Seabed Response Characteristics Considering Vibration Effect of Offshore Monopile Foundation. *Ocean Eng.* **2023**, *288*, 116000. [\[CrossRef\]](#)

18. Yang, W.; Tan, S.K.; Wang, H.; Yu, G. Rheological Properties of Bed Sediments Subjected to Shear and Vibration Loads. *J. Waterw. Port Coast. Ocean Eng.* **2014**, *140*, 109–113. [[CrossRef](#)]
19. Awad-Allah, M.F.; Yasufuku, N.; Abdel-Rahman, A.H. Cyclic Response of Wind Turbine on Piles in Unsaturated Sand. *Int. J. Phys. Model. Geotech.* **2017**, *17*, 161–176. [[CrossRef](#)]
20. Al-Hammadi, M.; Simons, R.R. Local Scour Mechanism around Dynamically Active Marine Structures in Noncohesive Sediments and Unidirectional Current. *J. Waterw. Port Coast. Ocean Eng.* **2020**, *146*, 04019026. [[CrossRef](#)]
21. Guan, D.; Chiew, Y.-M.; Melville, B.W.; Zheng, J. Current-Induced Scour at Monopile Foundations Subjected to Lateral Vibrations. *Coast. Eng.* **2019**, *144*, 15–21. [[CrossRef](#)]
22. Qin, B.; Xie, Y.; Yang, W.; Qu, R.; Geng, F. A Further Study on the Scour around the Monopile Foundation of Offshore Wind Turbines. *Sustain. Energy Technol. Assess.* **2023**, *57*, 103198. [[CrossRef](#)]
23. Cheng, Y.; Xie, H.; Zheng, Y.; Shi, L. Experimental Study on the Sediment Movement around Monopile under Vibration Action. *Ocean Eng.* **2023**, *288*, 116206. [[CrossRef](#)]
24. Whitehouse, R. *Scour at Marine Structures*; Thomas Telford Publishing: London, UK, 1998.
25. Cuéllar, P.; Baeßler, M.; Rücker, W. Ratcheting Convective Cells of Sand Grains around Offshore Piles under Cyclic Lateral Loads. *Granul. Matter* **2009**, *11*, 379. [[CrossRef](#)]
26. Yang, Y.; Melville, B.W.; Macky, G.H.; Shamseldin, A.Y. Experimental Study on Local Scour at Complex Bridge Pier under Combined Waves and Current. *Coast. Eng.* **2020**, *160*, 103730. [[CrossRef](#)]
27. Mawatari, Y.; Koide, T.; Tatemoto, Y.; Uchida, S.; Noda, K. Effect of Particle Diameter on Fluidization under Vibration. *Powder Technol.* **2002**, *123*, 69–74. [[CrossRef](#)]
28. Xu, C.; Zhu, J. Parametric Study of Fine Particle Fluidization under Mechanical Vibration. *Powder Technol.* **2006**, *161*, 135–144. [[CrossRef](#)]
29. Sumer, B.M.; Fredsøe, J. Scour around Pile in Combined Waves and Current. *J. Hydraul. Eng.* **2001**, *127*, 403–411. [[CrossRef](#)]
30. Sumer, B.M.; Petersen, T.U.; Locatelli, L.; Fredsøe, J.; Musumeci, R.E.; Foti, E. Backfilling of a Scour Hole around a Pile in Waves and Current. *J. Waterw. Port Coast. Ocean Eng.* **2013**, *139*, 9–23. [[CrossRef](#)]
31. Rudolph, D.; Bos, K.J. Scour around a Monopile under Combined Wavecurrent Conditions and Low KC-Numbers. In Proceedings of the 3rd International Conference on Scour and Erosion (ICSE-3), Amsterdam, The Netherlands, 1–3 November 2006; pp. 582–588.
32. Sumer, B.M.; Fredsøe, J.; Christiansen, N. Scour Around Vertical Pile in Waves. *J. Waterw. Port Coast. Ocean Eng.* **1992**, *118*, 15–31. [[CrossRef](#)]
33. Dogan, M. The Equilibrium Depth of Wave Scour around Both Slender and Large Piles. *Ocean Eng.* **2021**, *236*, 109474. [[CrossRef](#)]
34. Qi, W.; Gao, F. Equilibrium Scour Depth at Offshore Monopile Foundation in Combined Waves and Current. *Sci. China Technol. Sci.* **2014**, *57*, 1030–1039. [[CrossRef](#)]

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