Local scour around a monopile in reciprocating tidal current considering the effect of water depth

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Abstract. Despite most of sea-crossing bridges and offshore wind farms being located in areas dominated by tides, researches about the effects of tidal current on the scour process around these buildings are still limited to this date. A three-dimensional sediment transport numerical model was established to investigate the local scour around a monopile under reciprocating tidal flow. The standard turbulence equation RNG (Renormalization-group) κ-ε was adopted to close the N-S equation and the finite difference method was applied to discretize the governing equation in the numerical model. Tidal currents were adapted by bidirectionally reversing sinusoidal currents. It showed in the results the scour pits grew periodically in the process of repeated scour and deposition with the change of the flow velocity and direction. Besides, it implied that tidal scour depth around the monopile deepened with the water depth, but the impact of water depth diminished gradually. The change of the maximum scour depth was less than 5% if water depth is more than 4 times pile diameter. It indicated that the scour depth around the pile will be almost no longer influenced by water depth if water depth is more than 4 times pile diameter. Therefore, the research results emphasize the significance of selecting suitable water depth conditions for the design process of offshore structures.

1. Introduction
With the explosion of offshore wind farms, sea-crossing bridges, and deep-water ports, researches for erosion of buildings on marine environment become more and more significant for protecting offshore structures and improving the practicability and economy of structural design.[1]

Many scholars had carried out researches on local scour of structures under unidirectional current[2][3], wave[4][5] and combined wave-current conditions[6][7] et al., most of them dealing with the scour development at bridge piers. Actually, the area where the offshore structures are located is dominated by unsteady reciprocating tidal flow, in which flow velocity and direction changed periodically. For this kind of hydraulic condition, Escarameia et al.[8] studied the effects of velocity, flow direction, water depth and tidal cycle on the equilibrium scour depth around the pile. The experimental results showed that the equilibrium scour depth increased with the increase of velocity when the water depth is equal to the pile diameter. If the water depth decreases by 50% with a constant velocity, the maximum scour depth around the pile would be dropped by 83%. However, cases with water depth exceeding the pile diameter were not investigated. According to Froude Similarity Criterion, Schendel et al.[9] simulated cylindrical scour in real ocean environment with a laboratory flume. The results showed that equilibrium scour depth under unidirectional flow is same as maximum scour depth for tidal current when velocity of unidirectional flow is about 15-20% larger than the root mean square velocity of tidal current, while water depth was kept to be constant. Therefore, the effect
of water depth was not considered in Schendel[9]'s experiment. McGovern et al.[10] divided the velocity of a tidal cycle into \( U< U_c, \ U= U_c, \ U> U_c \) (\( U \) represents inlet velocity and \( U_c \) represents the critical velocity of sediment) to analyze the scouring process around the pile with scour time lasting for two tidal cycles. The experimental results implied that the scour depth and scour rate for reversing are both smaller than that of unidirectional flow, due to the greater flow intensity of unidirectional flow than that of tidal current (the velocity of unidirectional flow was consistent with the peak velocity of tidal current). However, the test lasted only two tide cycles and the scour process did not reach a stable state. Yao et al.[11] held a similar perspective to those of McGovern[10]. Yao used a sinusoidal current to simulate the tidal current and find that the scouring process of reciprocating tidal flow is slower than that of unidirectional flow.

Up to now, systematic researches on the effect of water depth on the scour of pile induced by reciprocating tidal flow is rare. However, most of the deep-water ports[12], offshore wind turbines[13] and sea-crossing bridges[14] are built in area where water depth exceeds the pile diameter and the range of water depth is quite wide. For that reason, a set of hydraulic numerical model was conducted to analyze the scour process of the pile under different water depths induced by reciprocating tidal current. The purposes of this research include (1) to gain further comprehension on the scouring and infilling processes induced by reciprocating tidal currents by the simulation of appropriately scaled sinusoidal tidal signals and (2) to obtain the relationship between the water depth and the maximum scour depth around the pile in tidal dominated area. As far as the authors know, this is the first systematic comparison that takes tidal currents with multiple different water depths into account. Furthermore, the results of this study were based on a verified model, ensuring reliable model effects.

2. Numerical model

2.1. Governing equations of the flow

Considering the flow of an incompressible viscous fluid around a pile, the hydrodynamics are solved by the RANS (Reynolds-averaged Navier-Stokes) equations.

\[
\frac{\partial \mathbf{u}}{\partial t} + \mathbf{u} \cdot \nabla \mathbf{u} = -\frac{1}{\rho} \nabla p + \mathbf{v} \nabla \mathbf{u} \nabla + \mathbf{\tau}_j
\]  \hspace{1cm} (1)

where \( x_i \ (i = 1, 2 \text{ and } 3) \) denotes the Cartesian coordinate, \( u_j \) denotes the velocity in \( x_i \) direction, \( v \) denotes the kinematic viscosity, \( p \) denotes the pressure, \( \rho \) denotes the fluid density, \( t \) denotes the time, and \( \tau_{ij} \) denotes the Reynolds stress which defined as

\[
\tau_{ij} = \nu_i (\frac{\partial u_j}{\partial x_i} + \frac{\partial u_i}{\partial x_j}) - \frac{2}{3} \delta_{ij} \kappa
\]  \hspace{1cm} (2)

where \( \delta_{ij} \) is the Kronecker delta, \( \kappa \) is the turbulent energy, \( \nu \) is the turbulent viscosity.

The sediment transport simulation is sensitive to turbulence modeling[15]. The RNG \( k-\varepsilon \) model is quite suitable in simulating low-turbulence-intensity flows in strong shear regions[16][17] and was, therefore, adopted in the present study. The governing equations of the RNG \( k-\varepsilon \) turbulence model are:

\[
\frac{\partial k}{\partial t} + \mathbf{u} \cdot \nabla k = \frac{\partial}{\partial x_i} \left[ (\nu + \frac{\nu}{\sigma_k}) \frac{\partial k}{\partial x_i} \right] + \tau_{ij} \frac{\partial u_i}{\partial x_j} \varepsilon
\]  \hspace{1cm} (3)

\[
\frac{\partial \varepsilon}{\partial t} + \mathbf{u} \cdot \nabla \frac{\varepsilon}{\rho} = \frac{\partial}{\partial x_i} \left[ (\nu + \frac{\nu}{\sigma_\varepsilon}) \frac{\partial \varepsilon}{\partial x_i} \right] + C_1 \frac{\varepsilon}{k} \tau_{ij} \frac{\partial u_i}{\partial x_j} - C_2 \rho \frac{\varepsilon^2}{k}
\]  \hspace{1cm} (4)
where $\varepsilon$ is the dissipation of turbulent energy, $C_{\mu}$, $C_{\nu}$, $C_{s}$, $\sigma_s$, and $\sigma_e$ are model coefficients with values usually setting to 0.085, 1.42, 1.68, 0.7179 and 0.7179.

2.2. Sediment transport model

The bed near the pile is subjected to strong shearing actions. When the shear stress exceeds a critical value, the bed erosion occurs. This criterion can be expressed in dimensionless form

$$\theta \geq \theta_c, \theta = \frac{\tau}{g d_{50} (\rho_s - \rho)}$$

where the Shields parameter $\theta$ is the nondimensionalized shear stress $\tau$, $g$ is the magnitude of the gravity acceleration, $d_{50}$ is the median diameter of the sand grain and $\rho_s$ is the sediment density and. The critical Shields number $\theta_c$ can be calculated by a formula of Soulsby [18]

$$\theta_c = \frac{0.3}{1 + 1.2 d_s} + 0.055 \left[1 - \exp(-0.02d_s)\right]$$

The nondimensional grain size $d_s$ is calculated as

$$d_s = d_{50} \left[\frac{\rho (\rho - \rho_s) g}{\mu^2}\right]^{1/3}$$

in which $\mu$ is the fluid dynamic viscosity. The critical friction velocity $U_{fr}$ can be given by the critical shear stress on bed $\tau_c$ when $\theta = \theta_c$

$$\tau_c = \rho U_{fr}^2$$

The transport rate of bedload can then be predicted by a model of van Rijn [19]

$$q_b = 0.053 d_s^{0.3} \left(\frac{\theta}{\theta_c} - 1\right)^{2.11} \left[\frac{\rho_s - \rho}{\rho}\right]^{0.5} d_{50}^{0.5}$$

For entrainment, the velocity at which the sediment leaves the bed surface can be calculated by the formula [20],

$$u_{ent} = n_b \alpha_s d_s^{0.5} (\theta - \theta_c)^{1.5} \sqrt{gd (\rho_s / \rho - 1)}$$

where $\alpha_s$ is the entrainment coefficient, $n_b$ is the outward normal vector of the bed surface. In the deposition, the settling velocity [18] is used,

$$u_{sett} = \frac{g \cdot \mu}{\rho d} \left[(10.36^2 + 1.049 d_s^{0.5}) - 10.36\right]$$

The suspended sediment is transported by advection along with the fluid, and can be calculated by solving its own transport equation:

$$\frac{\partial C_s}{\partial t} + \nabla \cdot (u_s C_s) = \nabla \cdot (D C_s)$$

where $C_s$ is the suspended sediment mass concentration, which is defined as the sediment mass per volume of fluid-sediment mixture; $D$ is the diffusivity; $u_s$ is the suspended sediment velocity. Ignore the interactions between suspended load; the velocity difference between the suspended load and the fluid-sediment mixture is mainly the settling velocity of grains; $u_s$ is calculated by

$$u_s = \bar{u} + \frac{u_{sett} C_s}{\rho_s}$$

in which $u$ is the velocity of the fluid–sediment mixture, which can be obtained by solving the continuity and the Navier–Stokes equations with the RNG $k - \varepsilon$ model.
3. Numerical model validation and setup

3.1. Model validation
In order to prove the accuracy of the numerical hydrodynamic model and sediment transport model, the flume test conducted by Schendel [9] was selected to verify the model. The experiment was conducted in a hydraulic flume, which was 18 m long and 1 m wide. The water depth is maintained at 0.5 m. And the inlet maximum velocity is 0.457 m/s. The tidal period is about 120 min. The pile is in the center of the flume positioned at (0, 0) with a diameter D of 0.15 m. The median size of sediment is \( d_{50} = 0.25 \) mm with a density \( \rho \) of 2650 kg/m\(^3\). Figure 1 and figure 2 indicated that the numerical simulation results presented a well agreement with the literature data. The final numerical results were slightly 6.7% smaller than the experimental results which is acceptable. The accuracy of the hydrodynamic model and sediment transport model has been validated as above. Thus it can be applied to simulate the sediment erosion under reciprocating tidal current.

Figure 1. Comparison of velocity over time between numerical simulation and experimental results.

Figure 2. Comparison of scour evolution over time between numerical simulation and experimental results around the pile

3.2. Model domain
As shown in figure 3, similar to Schendel [9]’s research, the numerical flume was set as 18 m long, 1 m wide and dept 1.5 times water depth. The flume provides a sediment pit with a length of 8 m and a thickness of 0.3 m. The median diameter of sediment was \( d_{50} = 0.75 \) mm with a density of \( \rho_s = 2650 \) kg/m\(^3\). The pile was positioned in the center of the flume with a diameter D of 0.15 m. The inlet velocity of tidal current was considered as a sine function of tidal period: \( U = U_{\text{max}} \sin((2\pi/T) \cdot t) \), where the maximum peak velocity \( U_{\text{max}} \) is set as twice the sediment critical velocity \( U_c \), \( t \) is the time, and tidal period \( T \) is 360 s. Each case was simulated for 4 cycles with a total time of 1440 s. The water depth was set at 1 ~ 6 times of the pile diameter to study the effect of different water depths. All detailed cases are presented in table 1.

3.3. Boundary conditions
For the inlet boundary, it was set to be velocity boundary, giving the sinusoidal tidal velocity. The outlet boundary was chosen as pressure boundary, in which pressure was set to be zero in the area above the water surface, while in the area below the water surface was given vertical hydrostatic pressure distribution. The top boundary of the model was considered to be the standard atmospheric pressure and the VOF methods were adopted to capture the deformation of the air-water interface. For the bottom boundary, the wall boundary condition was considered, which applies the no-slip condition at the boundary. The symmetry boundary condition was used at the two lateral boundaries to apply a zero-gradient condition at the boundary as well as a zero-velocity condition normal to the boundary.
4. Results and discussion

4.1. Scouring process under reciprocating tidal current

A periodical scour process of reciprocating tidal current with increasing scour depth could be found in figure 4. It could be also found that scour intensity in first cycle is maximum and scour depth at \( t=\frac{T}{2} \) is 0.738 times of scour depth at \( t=4T \). Figure 5 depicts scour rate over time. The scour rate is represented by the scour gradient between two adjacent time points. It illustrated in figure 5 that when the inlet velocity reaches the peak velocity \( U_{\text{max}}=0.684 \text{ m/s} \), the scour rate around the pile is 1.2 mm/s, which is the maximum value in 4 cycles. The scour depth increases slowly and the erosion rate decreases obviously since the second cycle began and the absolute values of scour rate are all below 0.5 mm/s. In next half cycles, the eroding velocity slows down obviously after reaching the peak velocity, which indicates that the scour pit will not be able to be eroded drastically when inlet velocity is less than peak velocity. The scour pit will remain the same depth as it once to be at peak velocity. This phenomenon is consistent with the experimental results of Chang et al.[21] and Link et al.[22]

Moreover, with the periodic changes of flow velocity and direction, the scour process almost stops when the flow reverses. Once the scour flow velocity at bed reaches the critical velocity of sediment, the scour process continues. When the flow is reversed, the reverse flow carries the sediment which is deposited on the downstream side into the upstream scour pit, leading to the decrease of the maximum scour depth. The infilling process was the result of negative flow. At this time, the scour rate is less than zero, which can be proved by the negative scour rate in figure 5. According to figure 4 and figure 5, the positive flow eroded the upstream area around the pile and made the downstream side deposited by sand. Then the sediment on the downstream side is backfilled into the upstream scour pit by reverse flow. The depth of the scour pit is gradually deepened in the process of repeated scour and deposition.

4.2. Tidal scour under different water depths

As water flows into the front of pile, the boundary layer at the bottom of the flow was blocked by the pile and a negative pressure gradient was generated at the front of the pile. This pressure gradient caused downflow, which induced the vortex at the bottom of the pile and thus increased the flow velocity. Then, local scour occurred around the pile. The deeper the flow is, the thicker the bottom boundary layer was, the greater the negative pressure gradient was. Thus, the vortex at the bottom around the pile increases, which leads to the intensification of local scour. Once the water depth increased to a certain extent, the velocity near the free surface was not affected by the boundary layer, and the thickness of the boundary layer was no longer increased. [23] At this time, the increase of water depth has no effect on the maximum scouring depth. The flow direction of the tidal current has the periodicity of reciprocating, which brings uncertainty to the effect of water depth on the maximum scour depth.

Table 1. Parameters of numerical cases.

| Cases   | Water depth \( h \) (m) | Inlet velocity \( U \) (m/s) | \( U_{\text{max}}/U_c \) |
|---------|-------------------------|-----------------------------|-----------------|
| Tide01  | 0.15                    | 0.610\text{sin}(2\pi/T \cdot t) | 2               |
| Tide02  | 0.30                    | 0.684\text{sin}(2\pi/T \cdot t) | 2               |
| Tide03  | 0.45                    | 0.732\text{sin}(2\pi/T \cdot t) | 2               |
| Tide04  | 0.60                    | 0.789\text{sin}(2\pi/T \cdot t) | 2               |
| Tide05  | 0.75                    | 0.796\text{sin}(2\pi/T \cdot t) | 2               |
| Tide06  | 0.90                    | 0.822\text{sin}(2\pi/T \cdot t) | 2               |
As the water depth raised, the horizontal velocity at the bottom of pile also increased (figure 6). In detail, the velocity around the pile grew up extremely with water depth raising from 1D to 4D. When the water depth exceeds 4D, the increment of velocity decreases. This is due to the fact that the thickness of the boundary layer no longer grew significantly with the increase of water depth. The increment of boundary layer is far less than that of water depth. Therefore, when the water depth continues to increase, the negative pressure gradient in the boundary layer did not change greatly, which leads to the small change of vortex and turbulence intensity at the bottom of the pile as well as the decrease of velocity increment. Especially for the flow close to the pile wall, the velocity at that area almost did not change after the water depth exceeded 4D.

The local scour around the pile was also impacted by the velocity distribution. Figure 7 showed that the maximum scour depth $|S|$ grew gradually with the increase of water depth. But the change rate of maximum scour depth $\Delta |S| / |S|$ was smaller than that of water depth $\Delta h / h$. When the water depth raised from 1D to 6D, the maximum scour depth $S$ increased from 0.105 m to 0.180 m (table 2), while the change rate of the maximum scour depth $\Delta |S| / |S|$ decreases, that is, the deepening trend of the maximum scour depth around the pile gradually slowed down with the increase of water depth. As the water depth raised from 4D to 5D, the change rate of the maximum scour depth $\Delta |S| / |S|$ is only 0.05 (table 2). Moreover, the maximum scour depth at 4D was 0.92 times as much as that at 6D (table 2). This was consistent with the previous analysis of the relationship between velocity and water depth. The increase of water depth led to the thickening of the boundary layer and the increase of velocity at the bottom of the pile, but the velocity at the bottom of the pile could not increase infinitely.
with the growing of water depth, which is inconsistent with Richardson[24]'s conclusion, which suggested that scour depth would increase with the water depth without limit. When the water depth exceeds 4D, the velocity at the bottom of the pile does not increase significantly as depicted in figure 6. So the change rate of the maximum scour depth decreases. Therefore, it can be considered that its effect on the maximum scour depth can be ignored if the depth of incoming water exceeds 4D.

**Table 2.** Effect of water depth on maximum scour depth (t=4T).

| Cases  | h(m) |  h \( \frac{D}{D} \) |  \( \frac{|S|}{D} \) |  \( \frac{|S|}{D_{\text{pre6}}} \) |  \( \frac{\Delta h}{h} \) |  \( \frac{\Delta |S|}{|S|} \) |
|--------|------|-----------------|-----------------|-----------------|-----------------|-----------------|
| Tide01 | 0.15 | 1 | 0.105 | 0.70 | 0.58 | - | - |
| Tide02 | 0.3 | 2 | 0.139 | 0.93 | 0.77 | 0.50 | 0.32 |
| Tide03 | 0.45 | 3 | 0.150 | 1.00 | 0.83 | 0.33 | 0.10 |
| Tide04 | 0.6 | 4 | 0.166 | 1.10 | 0.92 | 0.25 | 0.08 |
| Tide05 | 0.75 | 5 | 0.174 | 1.16 | 0.97 | 0.20 | 0.05 |
| Tide06 | 0.9 | 6 | 0.180 | 1.20 | 1.00 | 0.17 | 0.03 |

According to the scour patterns after 4 cycles, the scour patterns around the pile on different water depths were similar (figure 8). As the picture depicted, due to the reciprocating flow direction in both positive and negative, scour pits could be formed at both the upstream and downstream of the pile. The shape of the scour pit under tidal current was symmetrical about the pile. The contour lines were concentric rings with the pile as the center, radiating outward from the center. The scour depth increased from 1D to 6D, and the scope of scour pit was also expanded in turn. As the water depth grew from 1D to 4D, the size and shape of scour pit change obviously and the shape of scour pit changes slightly with flow depth changing from 4D to 6D, which is shown a well agreement with the variation trend of the horizontal velocity around the pile with the water depth.

Giving the fixed relative velocity of \( U_{\text{max}}/U_c=2 \), the maximum scour depth changed with water depth from 1D to 6D after 4 cycles of reciprocating tidal current as figure 9. The relationship between the maximum scour depth and water depth could be obtained by fitting the hyperbolic tangent function with the least-square method:

\[
\frac{|S|}{D} = 1.1 \tanh 0.64 \frac{h}{D} \tag{15}
\]

The determination coefficient \( R^2 \) of the fitting curve is \( R^2=0.96 \), which indicates that the relationship could reflect the variation trend of the maximum scour depth around the pile on reciprocating tidal current. It demonstrated from the relationship that if the water depth is greater than 4D, the change rate of the maximum scour depth is very small and the influence of water depth on the maximum scouring depth can be ignored.

**Figure 8.** Scour depth around pile (t=4T).

**Figure 9.** Relationship between maximum scour depth and water depth.
5. Conclusion
The RNG $k-\varepsilon$ turbulent model and van Rijn's [22] sediment transport formula were applied to simulate the sediment scour at a monopile induced by sinusoidal tidal flow. And the results might help to understand the characteristics of erosion by tidal current. The findings are summarized as follows.

1) The positive flow in tidal flow makes the upstream side of the pile scoured first and the sediment was backfilled into the scour pit on the upstream side due to the reverse flow with the erosion of downstream side. The depth of the scour pit is periodically deepened in the process of repeated scour and deposition with reciprocating flow.

2) The change rate of the maximum scour depth $\Delta|S|/|S|$ decreased gradually with the deepening of the water depth from 1D to 6D. And it was less than the growth rate of the water depth $\Delta h/h$. If the water depth was greater than 4D, the variation rate of the horizontal velocity, the range of the scour pit and scour rate all decreased and the effect of water depth on them could be neglected.

3) The scour characteristics of tidal scour around the monopile could be reflected by the established formula of the maximum scour depth and water depth. If the water depth is less than 4 times the diameter of the pile structure, the effect of water depth on the scour depth must be considered. If the ratio of water depth to diameter $h/D$ exceeds 4, the scour depth could be predicted with the scour depth at $h/D = 4$.

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