SOME ASPECTS OF LOCAL SCOUR MECHANICS AROUND SUBSEA PIPELINES

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To my parents and my wife Chunhui.
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ABSTRACT

The research carried out in this thesis is concentrated on physical experiments of local scour around subsea pipelines. The main findings are listed as following:

In chapter three a series of experiments performed in the recirculating Mini O-Tube (MOT) flume at UWA concerning the onset of scour below pipelines in steady current is reported. The experiments were performed on model seabed with two different lengths to investigate the effects of variations in sediment supply on onset of scour. In each of the experiments the flow velocity and pipeline embedment were recorded simultaneously and continuously. The test results are compared with an existing empirical formula for a partially buried pipeline on a flat seabed. It is found that when the upstream sediment supply is limited, onset of scour occurs at a velocity well below that predicted by the existing empirical formula, following significant local scour around the pipe. This result suggests the possibility of onset of scour for embedded pipelines in the field where sediment supply upstream of the pipeline is interrupted by rock outcrops or seabed slopes.

To interpret the experiments, a series of numerical simulations have also been conducted to investigate the dynamic pressure difference between upstream and downstream of the pipeline prior to onset of scour. The numerical results show that despite local scour significantly altering the surrounding seabed for an embedded pipeline, the pressure gradient across the pipeline is similar in magnitude to that measured for a flat seabed and is sufficient to initiate piping.

In chapter four scour around a fixed pipeline under currents, waves, and combined waves and current conditions has been studied experimentally using both the MOT and Large O-Tube (LOT) facilities at UWA. The model pipe diameter ranges from 50 mm to 196 mm. Comparisons of equilibrium scour depth and scour width have been made between present experimental data and existing empirical formulae and data in all three flow conditions. The empirical formula for equilibrium scour depth in steady current has been updated based on the existing data. A new method of predicting the equilibrium scour depth in combined waves and current is also proposed based on the current component of equilibrium scour depth and the wave component equilibrium scour depth. Bed features (e.g. sand ripples) which occur in the model tests are discussed in detail using three dimensional scour profile measurements. These results show that different regimes of bed forms occur, but the average scour hole shape is generally not affected by bed forms. Secondly, the wall boundary effect between the pipeline model and o-tube
side walls is discussed and quantified using experimental results and three dimensional numerical simulations. Collectively, the investigations of sand ripples and the wall effect provide a better understanding of the physical experiments and laboratory data.

In chapter five the time scale of local scour below pipelines is discussed under current alone, wave alone and combined waves and current conditions, based on the physical experiments presented in Chapter four and existing published data. The fitting coefficient relating to the shape of scour depth time development curve is found based on the statistical analysis. After comparison with the previous predicted methods and various types of Shields parameter, the dimensionless time scale of the scour process is found to be mainly dependent on the maximum Shields parameter in all three flow conditions, leading to a universal predictive formula for current, wave and combined waves and current conditions. Collectively, the experimental results concerning time scale and equilibrium scour depth presented in this study allow, for the first time, for predications of pipeline scour to be made in any combination of collinear wave and current conditions directed perpendicular to the pipeline.

In chapter six the time scale of scour below a pipeline is investigated multiple consecutive independent environmental conditions and continuously changing (ramp-up) flow conditions. A predictive model is developed to predict the rate of scour for these changing flow conditions. The predictive model is validated through comparison with experiments. The results show that the predictive formula result in a good fit with the measured data.

In chapter seven local scour around two identical pipelines in a tandem arrangement is investigated experimentally. The tests were carried out under both live-bed and clear water conditions for steady current and for live-bed waves conditions. The effects of the gap ratio between pipelines is investigated both in terms of scour depth and time scale in live bed and clear water conditions. It is found that the smaller the gap ratio ($G/D$), the larger the interaction between two tandem pipelines. When $0 < G/D < 3$ the equilibrium scour depth of the downstream pipeline is slightly larger than the upstream pipeline; whereas if $G/D > 3$ the equilibrium scour depth of the downstream pipeline is slightly smaller than the upstream pipe. The time scale of scour below two tandem pipelines were studied for each individual pipe. It is found that the time scale predictive formula proposed in Chapter five for a single pipeline fit the measured data for two pipeline systems. Finally, the wall effect and sand ripples are discussed.
LIST OF SYMBOLS

\( ADV \) \hspace{1cm} \text{Acoustic Doppler Velocimeter} \\
\( a_s \) \hspace{1cm} \text{rate of increase in flow velocity} \\
\( C_2 \) \hspace{1cm} \text{inertial resistance factor} \\
\( C, D \) \hspace{1cm} \text{prescribed matrices in the momentum equation} \\
\( C_p \) \hspace{1cm} \text{pressure coefficient} \\
\( D \) \hspace{1cm} \text{diameter of pipeline} \\
\( d_{50} \) \hspace{1cm} \text{average grain size} \\
\( e \) \hspace{1cm} \text{burial depth of pipeline} \\
\( f_w \) \hspace{1cm} \text{friction coefficient for the wave boundary layer} \\
\( g \) \hspace{1cm} \text{acceleration due to gravity} \\
\( KC \) \hspace{1cm} \text{Keulegan-Carpenter number} \\
\( k_s \) \hspace{1cm} \text{Nikuradse equivalent sand grain roughness} \\
\( m \) \hspace{1cm} \text{velocity ratio} \; m = \frac{U_c}{(U_c + U_w)} \\
\( N \) \hspace{1cm} \text{sample size} \\
\( n \) \hspace{1cm} \text{porosity of the sediment} \\
\( p \) \hspace{1cm} \text{fitting coefficient} \\
\( p \) \hspace{1cm} \text{local pressure} \\
\( p_{\text{upstream}} \) \hspace{1cm} \text{pressure at the upstream side of the pipe} \\
\( p_{\text{downstream}} \) \hspace{1cm} \text{pressure at the downstream side of the pipe} \\
\( p_\infty \) \hspace{1cm} \text{pressure at far field} \\
\( q_{\text{over}} \) \hspace{1cm} \text{volumetric transport rate over the pipe} \\
\( q_{\text{lee}} \) \hspace{1cm} \text{volumetric transport rate due to the lee wake vortex} \\
\( q_{\text{in}} \) \hspace{1cm} \text{volumetric transport rate from free stream flow} \\
\( q_{\text{out}} \) \hspace{1cm} \text{volumetric transport rate downstream side of the pipeline} \\
\( RE \) \hspace{1cm} \text{Reynold’s number for the near-bed wave-orbital motion} \\
\( Re_c \) \hspace{1cm} \text{pipeline Reynolds number in current} \\
\( Re_w \) \hspace{1cm} \text{pipeline Reynolds number in wave} \\
\( R^2 \) \hspace{1cm} \text{square of the correlation coefficient between the real physical experimental data and fitted empirical formula} \\
\( \bar{R}^2 \) \hspace{1cm} \text{mean R-square} \\
\( \bar{R}^2_{\text{max}} \) \hspace{1cm} \text{maximum mean R-square}
\( S_e \)  
equilibrium scour depth
\( S_{\text{max}} \)  
maximum scour depth
\( s \)  
specific gravity of sediment grains
\( S_1 \)  
equilibrium scour depth corresponding to the initial flow conditions in two flow conditions cases
\( S_2 \)  
equilibrium scour depth corresponding to the final flow conditions in two flow conditions cases
\( S_{\text{initial}} \)  
initial scour depth corresponding to the initial flow condition
\( S_{\text{final}} \)  
final scour depth corresponding to the final flow condition
\( \text{sp} \)  
Seepage flow path
\( SSE \)  
sum of error squared of the prediction
\( \bar{SSE} \)  
mean SSE
\( \bar{SSE}_{\text{min}} \)  
minimum mean SSE
\( T \)  
characteristic time scale of the scour process
\( T_1 \)  
time scale corresponding to the initial flow conditions in two flow conditions cases
\( T_2 \)  
time scale corresponding to the final flow conditions in two flow conditions cases
\( T_b \)  
time scale based on the hyperbola equation
\( T_{bf} \)  
time scale of the backfilling process
\( T_f \)  
time scale when \( p = 1 \)
\( T_p \)  
time scale when \( p = 0.6 \)
\( T_w \)  
wave period
\( t \)  
time
\( t_i \)  
time elapsed for second flow conditions to achieve the scour depth of first flow condition
\( U_c \)  
velocity induced by current component at the height of D
\( U_w \)  
velocity induced by wave component
\( U_{wc} \)  
velocity induced by combined wave and current
\( U_0 \)  
instantaneous velocity in one time period under combined waves and current flow condition
\( u_{*c} \)  
friction velocity based on grain roughness in current
\( u_{*w} \)  
friction velocity based on grain roughness in wave
\( u_i \)  
mean velocity of fluid
\( u_i, u_j \) fluctuating velocity components
\( W_{c1} \) equilibrium scour width at upstream side of pipeline under steady current
\( W_{c2} \) equilibrium scour width at downstream side of pipeline under steady current
\( W_w \) equilibrium scour width under waves
\( W_{wc1} \) equilibrium scour width at upstream side of pipeline under combined waves and current
\( W_{wc2} \) equilibrium scour width at pipeline downstream side of pipeline under combined waves and current
\( x_i, x_j \) coordinates in horizontal and vertical directions
\( X \) seepage flow path
\( X_s \) distance from separation position to pipeline centre
\( \hat{y}_i \) predicted value
\( \bar{y} \) mean value
\( y_i \) real data
\( y^+ \) dimensionless wall distance
\( z \) height above seabed
\( z_0 \) bed roughness length

\( \alpha \) soil permeability
\( \gamma \) specific weight of water
\( \varepsilon \) void fraction, porosity
\( \theta_c \) current Shields parameter
\( \theta_w \) wave Shields parameter
\( k \) von Karman’s constant (=0.41)
\( \rho \) fluid density
\( \tau_m \) mean shear stress
\( \tau_c \) current shear stress
\( \tau_w \) wave shear stress
\( \tau_{max} \) maximum shear stress
\( \nu \) kinematic viscosity of water
\( |v| \) magnitude of the velocity
\( \varphi \) phase angle
\[ \phi \quad \text{angle between current and wave directions} \]
\[ \omega \quad \text{angular frequency of the oscillating flow} \]
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1. INTRODUCTION

1.1 Research motivations

The rapid increase in the demand for hydrocarbons, coupled with the depletion of offshore oil and gas resources in shallow water, has motivated fossil fuel developers to move further offshore into deeper waters. To transport hydrocarbons from these deep-water developments, seabed pipelines are generally acknowledged as a reliable option to bridge storage units and offshore fields. Nevertheless, the cost of thousands of kilometres subsea pipeline is significant, and the consequences of rupture are large. Therefore, proper understanding of scour below subsea pipelines is very important, especially with respect to how scour effects pipeline stability.

When a subsea pipeline is placed in a marine environment, the presence of the structure will alter the flow pattern in its immediate vicinity. The flow around the pipeline is generally accelerated, which can lead to sediment movement close to the structure.

Although extensive research efforts have been devoted to understanding scour around subsea pipelines over the last four decades, there are still some significant uncertainties. For instance, field survey data for subsea pipelines reveals that the onset of scour below pipeline may happen at velocities below the critical conditions predicted by existing theory, which directly affects the potential pipeline self-burial. Another example is the time scale of scour below subsea pipelines, for which the existing theory is only valid under steady current flow conditions and waves only flow conditions. Predicting time scale of scour below subsea pipeline under other flow conditions, such as combined waves and current, multiple flow conditions and continuously changing flow conditions, is important to develop engineering models to predict scour, but has not been studied in a systematic manner. Moreover, with more complex structures such as producing wells, injection wells and processing systems installed on the seabed, multiple pipelines are being installed in close proximity. However, few existing studies have focused on scour around multiple pipelines compared with the single pipeline.
1.2 Research goals

A series of scour problems are addressed in this thesis, these focus on:

1. Revisiting the mechanics of the onset of scour below subsea pipelines in steady current, accounting for the effects of sediment supply on the potential for the onset of scour.

2. Investigating the scour around subsea pipelines in two-dimensions at relatively large scale so as to evaluate the feasibility of existing empirical formulae to predict the scour depth and scour width in steady current conditions, waves conditions, and combined waves and current conditions.

3. Investigating the time scale of local scour below pipelines in current alone, wave alone and combined wave and current. Especially for the combined wave and current conditions, no study yet is available in such area.

4. Investigating the time scale of local scour below pipelines in other flow conditions, such as multiple independent flow conditions and continuously changing flow conditions.

5. Investigating the local scour around two identical pipelines in a tandem arrangement.

1.3 Methodology

1.3.1 Physical experimental approach

In order to investigate the onset of scour, equilibrium scour depth and time scale around subsea pipelines, the unique O-tube experimental facilities at the University of Western Australia were used, including the ‘Mini O-tube (MOT)’ and ‘Large O-tube (MOT)’.

The O-tubes are fully enclosed circulating water channels with a rectangular test section and a propeller-type pump driven by a motor. The MOT facility is 2.5 m in breadth, 0.3 m in height and 6.4 m in length. The Mini O-tube (MOT) test facility comprises of a motor-impeller system, unplasticized polyvinyl chloride (uPVC) tube sections, two honeycomb transitions at each end of the test section, and one straight test section (the main components are indicated in Figure 1-1). The impeller in the MOT is driven by a 5.5 KW three phase induction motor with rated speed 2885 RPM (Revolution Per Minute). The rotational speed of motor is controlled by the Danfoss VLT 2800 frequency converter which is managed in LABVIEW software on a local computer. The diameter of the uPVC
Chapter 1 Introduction

tube is 0.17 m and there is a tapered section of 0.4 m length to connect the circular uPVC tube to each end of the rectangular test section. The test cross-section of the MOT is 0.2 m by 0.3 m with length of 1.8 m. The bottom 0.1 m of the working section can be filled with sediment or a false floor. To reduce turbulence levels, the length of the honeycomb flow laminar is 0.2 m and composed of 81 PVC tubes in the form of a $9 \times 9$ array. The maximum steady flow velocity that can be generated in the facility is approximately 2.0 m/s (measured 0.05 m above the false floor).

Correspondingly, the Large O-tube flume facility is approximately 5 times larger than the MOT test facility. The LOT facility is located at Shenton Park Hydraulics Laboratory of the school of Civil, Environmental and Mining Engineering, University of Western Australia. The test section of the LOT is 17.6 m in length, 1.4 m in depth and 1.0 m in width (a sketch of the facility and model pipe is drawn in Figure 1-2). A honeycomb laminator is installed at each end of the test section to reduce the level of turbulence and scale of vortices introduced into the test section. The honeycomb is composed of 400 stainless steel tubes in the form of a $20 \times 20$ array. The LOT is driven by a large diameter impellor (900 mm) equipped with a brushless 580KW AC motor, which is able to generate a maximum steady current velocity of up to 3 m/s and an oscillatory flow velocity of up to 1 m/s – 2.5 m/s, respectively, for wave periods of 5 s ~ 13 s. By controlling the rotation direction and speed of the impeller in the LABVIEW software on the local computer, the facilities are able to generate combined steady current and/or regular or irregular oscillatory flow at seabed level at a relatively large scale. A detailed description of the LOT facility and its capabilities can be found in An et al. (2013).

At the bottom of the ocean, the near seabed flow inducted by waves and combined waves and current is mainly in horizontal direction and the vertical component is close to zero. Moreover, the wave length in the field is normally much larger than the diameter of subsea pipelines. Therefore, it is valid to use oscillatory flow and combined current and oscillatory flow to simulate the near seabed flow inducted by the waves and combined waves for the purpose of investigating pipeline local scour. In this thesis, the experiments under oscillatory flow and combined current and oscillatory flow conducted in the LOT and MOT facilities are expressed as “waves” and “combined waves and current” experiments to consistent the wording with the previous investigation. In this study, the LOT facility was used to investigate the time scale below subsea pipelines in steady current flow conditions, wave only flow conditions, combined waves and current flow conditions and multiple independent flow conditions with 196 mm diameter model pipe.
Furthermore, experimental investigation of scour around two tandem pipelines was also conducted in the LOT facility with 150 mm diameter model pipelines. The MOT facility was utilised to verify the new facility with traditional wave flume. Moreover, the MOT facility was also used in the study of the onset of scour below subsea pipelines and time scale of scour around subsea pipeline under continually changing flow conditions. Partial physical experiments related to the time scale in steady current flow conditions, wave flow conditions and combined wave and current flow conditions were also conducted in the MOT facility.

Besides the facilities, innovative measurement equipment was also adopted in this study. In the LOT facility a handheld 3D camera was used to obtain 3D morphological information; meanwhile the scour profile data was obtained by a 3D laser scanner in the MOT facility. With the 3D scour profile information the bed features such as sand ripples and sand waves were investigated in this study.

1.3.2 Numerical approach

Although the experimental environment is more and more close to the marine situation using the new innovative O-tube facilities and useful data is obtained using the measurement equipment, not all the information about scour around subsea pipelines can be acquired in the physical experiments. Therefore, numerical simulations have been conducted in this study to supplement the experiments. The numerical simulations have been carried out using ANSYS Fluent software.

To investigate the seepage flow path in the uneven seabed around embedded pipeline, the water flow field above the seabed is simulated by solving the two-dimensional Reynolds-averaged Navier-Stokes equations with shear stress transport (SST) $k - \omega$ turbulence model. The Laplace’s equation is used to calculate the seepage flow below the seabed.

To identify the incomplete horseshoe vortex at the junction between pipe and side wall, a similar numerical model with the onset of scour study is adopted, but in three-dimensional manner.

1.4 Outline

This thesis comprises eight chapters. The seven chapters following this introductory chapter are arranged as following:
Chapter 1 Introduction

In Chapter 2 previous investigations related to the onset of scour, the equilibrium scour depth and width, time scale, group pipelines scour and numerical simulation are summarised.

In chapter 3 the effect of limited sediment supply on the onset of scour below subsea pipelines is investigated experimentally and numerically.

In chapter 4 pipeline equilibrium scour depth and scour width in current/waves and combined waves and current are revisited based on new experimental results and reinterpretation of existing results. Scale effects and the effect of bed features on subsea pipelines scour are also reviewed in this chapter.

In chapter 5 the time scale of local scour below single pipelines in current/waves and combined waves and current is investigated based on new experimental results and reinterpretation of existing results.

In chapter 6 the time scale of local scour below single pipeline in multiple independent flow conditions and continually increasing flow conditions is studied experimentally.

In chapter 7 local scour around two tandem pipelines is investigated experimentally.

In chapter 8 conclusions and suggestions on future work are presented.
Chapter 1 Introduction

References

Figure 1-1 Sketch of the Mini O-tube facility.

Figure 1-2 Sketch of the Large O-tube facility. (Draper et al., 2014)
2. LITERATURE REVIEW

With the increasing development of offshore oil and gas projects, local scour below subsea pipelines has attracted a large amount of research over the last three decades. A comprehensive review on local scour below a pipeline is provided by Sumer and Fredsøe (2002). Although great progress has been achieved on the topic in the past, several aspects of scour around pipelines still remain unknown, such as the onset of scour on an uneven seabed, time scale in combined waves and current and scour around two pipelines, etc.

The purpose of this chapter is to summarise the existing work related to the present study on the scour processes around subsea pipelines, including the onset of scour, the equilibrium scour depth and width, time scale, scour around groups of pipelines and numerical simulation. Although a major part of this review overlaps with existing publications such as Sumer and Fredsøe (2002) and Whitehouse (1998), it is provided here for the completeness and for convenience. In addition, the present review covers some new research over the last decade that are not covered in the previous summary literature.

2.1 Onset of scour

Interaction between a submarine pipeline and an erodible seabed has attracted much attention because of its importance in offshore engineering. Of particular interest is the ‘onset of scour’, which defines the critical point at which sediment is washed away beneath a pipeline due to a seepage pressure gradient exceeding the submerged weight of sand particles. The onset of scour results in tunnel erosion below a pipeline, followed by spanning of the pipeline and subsequent self-burial (Sumer and Fredsoe, 2002) or pipeline breakout. Predicting the onset of scour is therefore of significant engineering importance.

For the case of a pipeline partially embedded in a flat seabed, a number of investigations have been reported regarding the onset of scour. Chiew (1990) conducted a series of physical experiments and found that piping (backwards erosion of sediment) leads to the onset of scour. More specifically, they found that in steady current, piping occurs when the dynamic pressure difference upstream and downstream of the pipeline induces a pressure gradient in the soil which exceeds the floatation gradient of the sediment immediately downstream of the pipe. Sumer et al. (2001) also investigated this
mechanism and presented an empirical expression for the onset of scour in steady current for a pipeline partially buried in a flat seabed. This expression is given as

\[
\frac{U_{cr}^2}{gD(1-n)(s-1)} = 0.025 \exp \left[9 \left(\frac{e}{D}\right)^{0.5}\right], \tag{2-1}
\]

where \(U_{cr}\) is a ‘critical’ undisturbed steady current velocity above which the onset of scour due to piping will occur; \(g\) is acceleration due to gravity; \(D\) is pipe diameter; \(n\) is porosity of the sediment; \(s\) is specific gravity of sediment grains, and \(e\) is burial depth of a pipeline.

The expression in Eq.(2-1) provides a valuable prediction of the onset of scour for a pipeline on a flat seabed. However, it suggests that the onset of scour is unlikely in weak ambient field conditions (smaller than the critical velocity \(U_{cr}\)) for pipelines on most soils if they are embedded to certain level. Field survey data for subsea pipelines, (Leckie et al., 2015) however, tends to suggest that the onset of scour may still happen even if the initial (as laid) pipeline embedment is relatively large (Westgate, 2013) or the ambient flow is relatively weak. This implies that other mechanics may contribute to the onset of scour.

\section*{2.2 Two-dimensional scour}

\subsection*{2.2.1 Scour depth}

Once the onset of scour happens under a pipeline, the seabed material under the pipeline will be eroded and the tunnel erosion leads to a growing scour hole under the pipeline until the scour process reaches the fully-developed stage. The scour depth measured vertically below the centre of pipe corresponding to the fully-developed stage is called the equilibrium scour depth.

\subsection*{2.2.1.1 Scour depth in steady current}

Scour depth has been studied extensively in the steady current case (Bijker and Leeuwestein, 1984; Hansen et al., 1986; Kjeldsen et al., 1973; Mao, 1988). In these studies the pipeline was fixed in position, with the bottom of the pipeline at the same level as the level of the undisturbed bed.

Kjeldsen et al. (1973) established an empirical relation between the equilibrium scour depth, \(S_c\), the pipe diameter, \(D\), and the mean flow velocity, \(U\).
\[ S_c = 0.972 \left( \frac{U^2}{2g} \right)^{0.2} D^{0.8}. \] (2-2)

Bijker and Leeuwestein (1984) stated that the scour depth depends on the depth-averaged flow velocity, pipe diameter, flow depth, height of pipeline above bed level and grain size. Using results from a series of model tests, including those conducted by Kjeldsen et al. (1973), they proposed a slightly different empirical equation for computing scour depth below submarine pipelines:

\[ S_c = 0.929 \left( \frac{U^2}{2g} \right)^{0.26} D^{0.78} d_{50}^{-0.04}, \] (2-3)

where \( d_{50} \) is the average grain size.

Sumer and Fredsøe (1990) indicated that in steady currents, the fully developed stage of the scour process is weakly dependent on the current Shields parameter \( \theta_c \) in live-bed flow conditions, while high dependence was found in clear-water flow conditions. They also found that the dependence on the relative roughness and Reynolds number is insignificant. For all practical purposes, Sumer and Fredsøe (2002) indicated that the following relation can be used as the design equation to predict the equilibrium scour depth in steady currents:

\[ \frac{S_c}{D} = 0.6 \pm 0.1. \] (2-4)

2.2.1.2 Scour depth in waves

To investigate scour below a pipeline under wave only conditions, there is one additional parameter that is relevant, namely the Keulegan-Carpenter number, \( KC \), which is defined by:

\[ KC = \frac{U_w T_w}{D}, \] (2-5)

in which \( T_w \) = the wave period and \( U_w \) = the maximum outer flow velocity in wave. Sumer and Fredsøe (1990) presented data on equilibrium scour depth plotted as a function of \( KC \) number. The plot indicates that there is a strong correlation between the equilibrium scour depth and the \( KC \) number. Based on the data, a design equation was established for the equilibrium scour depth:

\[ \frac{S_w}{D} = 0.1 \sqrt{KC}. \] (2-6)

This equation was found to be applicable from \( KC \) numbers around 2 to very large \( KC \) numbers (\( KC = 1000 \)).
2.2.1.3 Scour depth in combined waves and current

In the field, current and waves normally coexist, Sumer and Fredsøe (1996) presented the following empirical expression to predict the scour depth in combined waves and current:

$$\frac{S_{wc}}{D} = \frac{S_c}{D} F,$$

in which $S_c$ = the scour depth in the current-alone case, and $F$ = a function of $KC$ and ratio of velocity inducted by current component to the total velocity $m = \frac{U_c}{(U_c + U_m)}$, and given by the following empirical equations:

$$F = \begin{cases} \frac{2}{3} (KC)^a \exp(2.3b) & 0 < m \leq 0.7 \\ 1 & 0.7 < m \leq 1 \end{cases}, \quad (2-7)$$

$$a = \begin{cases} 0.557 - 0.912(m - 0.25)^2 & 0 \leq m \leq 0.4 \\ 2.14m + 1.46 & 0.4 \leq m \leq 0.7' \end{cases}, \quad (2-8)$$

$$b = \begin{cases} -1.14 + 2.24(m - 0.25)^2 & 0 \leq m \leq 0.4 \\ 3.3m - 2.5 & 0.4 \leq m \leq 0.7' \end{cases}, \quad (2-9)$$

Caution must be exercised in the implementation of the preceding empirical equations when $KC$ is larger than the upper boundary of the $KC$ range tested in the study ($KC = 50$) for $m \neq 0$. Also, it may be noted that the preceding results have been obtained under live-bed ($\theta > \theta_{cr}$) conditions.

All in all, most of the previous investigations about the scour depth under a pipeline were typically based on testing of model pipeline with relatively small diameters. Scaling effects may exist in these small diameter tests, however it is unclear how significant these effects are. There have not been many systematic studies on this issue.

Moreover, most of the equilibrium scour depth investigations were conducted under live-bed flow conditions, namely $\theta > \theta_{cr}$. Under live-bed conditions, the sediment transport prevails over the entire bed. Therefore, the initially flat bed may deform into various types of bed features, ranging in size from small ripples up to major sandwaves. However, no systematic study is available to demonstrate the effect of bed features on the scour around subsea pipelines in combined waves and current.

2.2.2 Scour width

For the steady current alone cases, Sumer and Fredsøe (2002) suggested the width of scour hole at the upstream side of the pipeline is 2D and 4D at the downstream side of the pipeline.
Chapter 2 Literature review

For the wave alone case, lee-wake erosion occurs on both sides of the pipe. The width of the lee-wake is dependent on the KC number. Therefore, Sumer and Fredsøe (2002) presented the following empirical formula to calculate the scour width under wave only conditions

\[
\frac{W_w}{D} = 0.35KC^{0.65},
\]

in which \(W_w\) is the width measured from the center of the pipeline to the end of the scour hole.

For combined waves and current, Sumer and Fredsøe (1996) indicated the following:

1. The scour widths \(W_{wc1}\) and \(W_{wc2}\) (the upstream and downstream widths, respectively) reduce to that suggested by Eq. (2-11) as \(m \to 0\) (the wave case).
2. Furthermore, the net effect of superimposing a current on waves is to make the downstream width of the scour hole larger and the upstream width smaller (slightly), apparently due to the effect of the lee-wake.
3. It appears that the scour widths approach constant values for \(m\) larger than 0.5-0.7, namely

\[
\frac{W_{wc1}}{D} \to \approx 2, \quad \frac{W_{wc2}}{D} \to \approx 4, \text{as } m \to 1
\]

4. The above result is consistent with the corresponding findings related to the scour depth.

It may be noted that the results summarized above correspond to the live-bed condition, \(\theta > \theta_{cr}\).

Again, the previous investigations related to the scour width around pipeline were typically based on testing of model pipeline with relatively small diameter. Large scale model tests would be ideal to check scale effects.

Like the research on scour depth, most of the scour width investigations were conducted in the traditional wave flume with limited length of pipeline and two side walls. Wall effect occurred at the junction between pipeline and the side wall, but these have not been investigated extensively.

2.3 Time scale

2.3.1 Time scale in stationary flow conditions

Due to the complexity of the scouring process, the majority of the literature has concentrated on determining the maximum equilibrium scour depth for given flow and
Chapter 2 Literature review

sediment conditions, while relatively few researchers have considered the evolution of scour in time.

Shen (1965) was one of the early researchers to investigate the scour around piles and concluded that the Froude number was the most significant parameter in determining the scour depth, Shen (1965) provide empirical curves of scour depth as a function of time.

For a pipeline originally placed on the seabed, the scour depth develops towards the fully developed stage through a transitional period. Fredsøe et al. (1992) indicated that the time variation of the scour depth can be approximately represented by:

\[ S(t) = S_e \left(1 - e^{-\frac{t}{T_f}}\right), \]  

(2-13)

where \( S(t) \) is the time-varying scour depth, \( t \) is time, \( S_e \) is the equilibrium scour depth and \( T_f \) is the characteristic time-scale of the scour process (so that in this case, the scour depth is \( \sim 0.63 \) times the equilibrium value after scour has occurred for a duration equal to the time scale). Using continuity arguments and an assumption that the scour hole scales with pipe diameter (Mao, 1986; Whitehouse, 1998). Fredsøe et al. (1992) reasoned that the time scale could be non-dimensionalised such that

\[ T_f^* = \left(\frac{g(s - 1)d^3}{D^2}\right)^{\frac{1}{2}} T_f, \]  

(2-14)

where \( D \) is the diameter of the pipeline, \( s \) is the relative density of the sediment, \( d \) is the mean sediment grains size, and \( g \) is the acceleration due to gravity. Based on an empirical fit to limited experimental data available in the literature at the time in both current and wave only cases, Fredsøe et al. (1992) found that Eq. (2-13) could explain the scour process reasonably well when the non-dimensional timescale across all of the experiments was calculated according to:

\[ T_f^* = \frac{1}{50} \theta^{-5/3}, \]  

(2-15)

where \( \theta \) is the dimensionless free field shear stress, known as the Shields parameter, and is given by \( \tau/(\rho g(s - 1)d) \), where \( \rho \) is the density of water and \( \tau \) is the shear stress. In current only conditions \( \theta \) (\( \equiv \theta_c \)) is calculated using the mean shear stress, and in wave only conditions \( \theta \) (\( \equiv \theta_w \)) it is calculated using the maximum wave shear stress (Fredsøe et al., 1992).

A more general expression than that used by Fredsøe et al. (1992) was suggested by Whitehouse (1998).
where \( p \) is an empirical coefficient and \( T_p \) is the time scale associated with this exponent. (Note there is an error in the original formulation given in Whitehouse (1998), where \(-t/T_p\) is incorrectly raised to the exponent \( p \)). This expression is consistent with Fredsøe et al. (1992) when \( p = 1 \).

The basic scour profile in time for a pipeline is similar to that for scour around the structures, such as piles. Briaud et al. (1999) proposed a different method, referred as SRICOS, to predict the scour depth \( S \) versus time \( t \) curve at a cylindrical bridge pier for a constant velocity flow. The SRICOS method consists of:

1. Collecting Shelby tube samples near the bridge pier
2. Testing them to obtain the erosion rate \( \dot{z} \) (mm/h) versus hydraulic shear stress \( \tau \) (N/m\(^2\)) curve
3. Calculating the maximum hydraulic shear stress \( \tau_{\text{max}} \) around the pier before scour starts
4. Reading the initial erosion rate \( \dot{z}_i \) (mm/h) corresponding to \( \tau_{\text{max}} \) on the \( \dot{z} \) versus \( \tau \) curve.
5. Calculating the maximum depth of scour \( S_{\text{max}} \)
6. Constructing the scour depth \( z \) versus time \( t \) curve using a hyperbolic model.
7. Reading the scour depth corresponding to the duration of the flood on the \( z \) versus \( t \) curve.

Briaud et al. (1999) found that the temporal developments of scour depth around a vertical pile can be described by the following hyperbolic function:

\[
S(t) = S_e \left( 1 - \exp \left( - \left( \frac{t}{T_p} \right)^p \right) \right)
\]  

(2-16)

The maximum hydraulic shear stress \( \tau_{\text{max}} \) exerted by the water on the riverbed was obtained by performing a series of three-dimensional numerical simulations of water flowing past a cylindrical pier of diameter \( D \) on a flat river bottom and with a large water depth. The results of several runs led to the following equation (Briaud et al., 1999):

\[
\tau_{\text{max}} = 0.094 \rho U_e^2 \left( \frac{1}{\log R_e} - \frac{1}{10} \right),
\]  

(2-18)

where \( \rho \) is the density of water; \( R_e \) is the pier Reynolds number (= \( U_eD/\nu \), \( \nu \) the kinematic viscosity of the water).
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The maximum scour depth was obtained by performing a series of 43 model scale flume tests (Briaud et al., 1999). The results of these experiments and a review of other work lead to the following equation, which appears to be equally valid for clays and sands:

\[ S_{\text{max}}(\text{mm}) = 0.18R_e^{0.635}. \]  

(2-19)

This method is also extended to account for accumulative effects of different velocities and soil layers by Briaud et al. (2001).

Although this approach works well for piles, the algorithms of \( \tau_{\text{max}} \) and \( S_{\text{max}} \) are based on vertical cylinder tests data. The applicability of this method for horizontal cylinders, such as pipelines, has not be validated.

Collectively a significant amount of research on equilibrium scour depth (in particular the results of Sumer and Fredsøe (1990)) and the time scale analysis in Fredsøe et al. (1992) allow for the evolution of scour to be predicted for fixed pipelines subjected to perpendicular current only and wave only conditions. However, the evolution of scour beneath a pipeline in any combination of perpendicular combined waves and current conditions cannot yet be predicted confidently because of a lack of systematic study on the time scale in combined waves and current conditions. Because of that, the engineering model is incomplete to predict the scour history around pipeline for a long time period (Harris et al., 2010). Despite the lack of investigations, Whitehouse (1998) has hypothesised that Eq.(2-15) may be applicable in combined waves and current conditions if \( \theta \) is calculated as the combined waves and current shear stress; however questions remain as to the appropriateness of this hypothesis and to whether the mean combined shear stress, the maximum combined shear stress or a different formulation (such as the linear combination \( \theta_w(1 - m) + \theta_c \), which has been recently adopted by Cheng et al. (2014) with \( m \) is most appropriate to use.

2.3.2 Time scale in multi-flow conditions

In order to predict the scour development in non-steady flow, a time stepping approach has been introduced Whitehouse (1998). The principle of this time stepping approach is that an increment in scour depth \( \Delta S \) may be calculated for a time interval \( \Delta t \) when a quasi-steady flow is assumed to exist (a reasonable approximation if the flow conditions changes gradually). Then it is assumed

\[ \Delta S(t) = \frac{dS(t)}{dt} \Delta t. \]  

(2-20)

From Eq. (2-16) (see also (Whitehouse, 1998)): 

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\[ t = T \left[ -\ln \left( \frac{S_e - S(t)}{S_e} \right) \right]^{1/p}, \]  
(2-21)

\[ 1 = \frac{T}{p} \left[ -\ln \left( \frac{S_e - S(t)}{S_e} \right) \right]^{1/p-1} \frac{dS(t)/dt}{S_e - S(t)}, \]  
(2-22)

\[ \frac{dS(t)}{dt} = \frac{p(S_e - S(t))}{T \left[ -\ln \left( \frac{S_e - S(t)}{S_e} \right) \right]^{1/p-1}}. \]  
(2-23)

Thus the calculation of the scour depth increment $dS$ during any time step requires knowledge of the length of the time step $\Delta t$, the degree of scour which has already occurred $S$, the ultimate scour depth $S_e$, the time-scale $T$ and the empirical constant $p$.

The time-stepping model uses Eq. (2-16) to calculate the scour occurring over the interval $0 \leq t \leq \Delta t_1$ where $\Delta t_1$ is the step size of the first time step. Subsequent time steps may then always utilise a non-zero value of $S(t)$. The conditions $S(t) > S_e$ will lead to negative scour which is not physically meaningful and so $S > S_e$ must give $dS/dt = 0$ within the model.

Although this time stepping approach appears reasonable, the lack of a predictive model for the time scale in combined waves and current conditions limits the application of the step model. Moreover, the model has not been tested and verified with experimental data for pipeline under varying velocity conditions.

2.4 Scour around two tandem pipelines

In offshore oil and gas engineering there are situations where two pipelines are laid in parallel. Although many studies have been conducted to investigate scour below a single pipeline, the scour behaviour around two pipelines is not yet properly understood. Few studies have focused on the scour process below two pipelines. Jensen et al. (1990) found that the vortex shedding from the pipelines occurs after the scour reaches a certain depth. Zhao and Cheng (2008) studied scour below a piggyback pipeline (comprised of two pipelines of different diameters) in steady current and found that the existence of a small pipeline on top of the big one increases the scour depth. For two close parallel pipelines local scour processes are affected by the spacing between the two pipelines. J.H. Westerhorstmann et al. (1992) investigated scour below two and three-pipelines in tandem arrangements with 0.5D and 1D spacing. It was found that for two pipelines system, spacing between pipelines of 0.5D resulted in less scour depth than full diameter
spacing. Furthermore, unidirectional flow with oscillatory motion resulted in less scour than unidirectional flow.

Harichandan and Roy (2012) performed numerical investigations for circular cylinders in tandem arrangement close to a wall at Reynolds number $Re = 100$ and $Re = 200$ for gap to pipe diameter ratio equal to 1 and 4. Rao et al. (2013) numerically studied the dynamics and stability of the flow past two cylinders sliding along a wall in a tandem configuration for Reynold numbers between 20 and 200, and gap to pipe diameter ratio from 0.1 to 10. Zhao et al. (2014) were the first to investigate local scour around two subsea pipelines tandem arrangement using a numerical model. The RANS equations were used to solve the flow past two pipelines and the conservation of sediment mass was used to predict the evolution of the sea bed profile. The numerical model was validated against the experimental data under both clear water and live bed scour conditions. The effects of the gap between the two pipelines on scour were studied by extensive numerical simulations with the gap to diameter ratio between the two pipelines ranging from 0.5 to 5.

Based on this earlier work of physical experiments, it is clear that pipelines in tandem arrangement have only been investigated in limited conditions ($0.5D$ to $1D$). For the testing at different spacing would be worthwhile.

### 2.5 Numerical model

The knowledge gained from the list of research studies through experimental and empirical studies has sparked the development of various numerical models for predicting 2D and 3D scour in recent years. Numerical modelling of local scour around offshore structures is being developed rapidly, with many universities and research institutes opting to progress along this trend. The development of numerical models are cost efficient with strong application in the offshore industry. Nevertheless, experiments are still important in identifying the fundamentals and acts as a basis for validating numerical models.

In this section, the existing research related to the present study is summarised, such as flow simulation around subsea pipelines and numerical modelling of the onset of scour below subsea pipelines.
Chapter 2 Literature review

2.5.1 Numerical modelling of flow around subsea pipelines

At first, several authors have developed potential-flow models for flow and scour around pipelines (Bernetti et al., 1990; Chao and Hennessy, 1972; Hansen et al., 1986; Li and Cheng, 1999). The potential-flow models generally have a good prediction about the flow at the upstream side of pipeline, however, the unsteady flow on the downstream side of the pipeline cannot be simulated properly.

With the increasing capacity of computers, models which calculate the flow around the pipeline by solving the Navier-Stokes equations have been developed. Different turbulence models have been adopted by various authors in their models. Flow models based on $k-\varepsilon$ turbulence were presented by (Brors, 1999; Leeuwenstein and Wind, 1984; Van Beek and Wind, 1990); Lee et al. (1994) presented a finite difference solution to the 2D Navier-Stokes equations with a Subgrid-Scale (SGS) model, qualitative comparison showed favourable agreement with the experimental measurement of Bearman and Zdravkovich (1978). Li et al. (1997) presented finite difference solutions to both 2D and 3D Navier-Stokes equations in Large Eddy Simulation (LES). They found that both 2D and 3D simulations agreed well with the measured data, and that the 3D simulation was marginally better than the 2D simulation. Lei et al. (1999) carried out both 2D and 3D direct numerical simulations (DNS) in studying the wall effects on the flow over a circular cylinder for a relatively low Reynolds numbers ($1.31 \times 10^4 < Re < 1.45 \times 10^4$). Liang and Cheng (2005b) compared four turbulence model at $Re = 7000$, which are the standard $k-\varepsilon$ model, the Wilcox low-Reynolds-number $k-\omega$ model and Smagorinsky’s SGS model, to simulate the flow around a circular cylinder placed above a solid wall. Wang and Tan (2008) studied the near-wake flow characteristics of a circular cylinder close to a flat bed for $Re = 1.2 \times 10^4$. Zhao et al. (2007) applied a $k-\omega$ model at $Re = 2 \times 10^4$. Ong et al. (2010) studied flows around a pipeline near the flat seabed at $Re = 3.6 \times 10^6$ by using 2D Unsteady Reynolds-Averaged Navier-Stokes (URANS) with a standard high Reynolds number $k-\varepsilon$ model. An et al. (2011) investigated a partially buried pipeline in a permeable seabed subject to combined oscillatory flow and steady current with a $k-\omega$ model at $Re = 3 \times 10^5$.

However, very little work has been done to simulate the incomplete horse vortex at the junction between pipeline and side wall in the experimental flume, which is quite important to understand the wall effect in the physical experiments.
2.5.2 **Numerical modelling of the onset of scour below the subsea pipeline**

Compared with the extensive numerical investigations about the scour around subsea pipelines, the numerical study on the onset of local scour below subsea pipeline is relatively rare.

Liang and Cheng (2005a) were the first to establish a numerical model of onset of local scour below offshore pipelines subject to steady currents. The pressure gradient that governs the seepage flow below pipelines is determined by solving the 2D continuity equation and Reynolds-averaged Navier-Stokes equations with the standard $k - \varepsilon$ turbulence closure in a general curvilinear coordinate system. The seepage flow is calculated by the Laplace equation and the free water surface is tracked in the model. The critical incoming flow velocity for the onset of scour is then calculated and the results are found to compare well with experimental data.

Zang et al. (2009) developed a numerical model to predict the onset of local scour below offshore pipelines in steady current only and waves only. In the model, the water flow field above the bed was determined by solving the 2D Reynolds-averaged Navier-Stokes equations with a $k - \omega$ turbulence closure. The seepage flow below the seabed was calculated by solving the Laplace equation with known pressure distribution along the common boundaries of the flow domains. The average pressure gradient along the buried pipeline surface is employed in the evaluation of the onset conditions with a calibration coefficient. Using this model, the influence of flow parameters, including water depth, embedment depth, boundary layer thickness, Reynolds number ($Re$) and Keuleagan-Carpenter ($KC$) number, on the pressure drop coefficient over the pipeline have been studied systematically.

Gao and Luo (2010) proposed a flow-pipe-seepage sequential coupling Finite Element Method (FEM) model to simulate the coupling between the water flow-field and soil seepage-field. A critical hydraulic gradient is obtained for oblique seepage failure of the sand in the direction tangent to the pipe. Parametric study is performed to investigate the effects of inflow velocity, pipeline embedment on the pressure-drop, and the effects of soil internal friction angle and pipeline embedment-to-diameter ratio on the critical flow velocity for pipeline spanning initiation. It is indicated that the dimensionless critical flow velocity changes approximately linearly with the soil internal friction angle for the submarine pipeline partially-embedded on a sandy seabed.
However, all the above investigations about the onset of scour below subsea pipelines are based on the flat initial seabed around the embedded pipe, no numerical study of the onset of scour for an embedded pipeline on a scoured seabed is found in literature yet.
Chapter 2 Literature review

References


Cheng, L., Yeow, K., Zang, Z. and Li, F., 2014. 3d scour below pipelines under waves and combined waves and currents. Coastal Engineering, 83(0): 137-149.


Chapter 2 Literature review


3. EFFECT OF LIMITED SEDIMENT SUPPLY ON THE ONSET OF SCOUR BELOW SUBSEA PIPELINES

This Chapter summarizes the results of a series of experiments performed in a recirculating flume (the Mini O-tube) to investigate the onset of scour below subsea pipelines in steady currents. The experiments were performed on a model seabed that extended different lengths upstream of the pipeline to assess the effects of sediment supply on the potential for the onset of scour. In each experiment, the flow velocity and pipeline embedment were recorded continuously (and some experiments were repeated with three cameras inside a transparent pipeline to get a more detailed view of the scour process). These measurements have been compared with an existing empirical formula proposed for predicting the critical velocity at which onset of scour will occur for a pipeline partially buried in a flat seabed. In general, present experimental results show that when the upstream sediment supply is limited, the onset of scour may happen due to piping at a velocity well below that predicted by the existing empirical formula. This result suggests that there is potential for tunnel erosion to occur beneath embedded pipelines in the field where the sediment supply upstream of the pipeline may be interrupted by rock outcrops or seabed slopes.

A series of numerical simulations have also been conducted to investigate the dynamic pressure difference and seepage flow upstream and downstream of the pipeline prior to the onset of scour. The numerical results show that despite local scour significantly altering the surrounding sediment morphology for an embedded pipeline with limited sediment supply, the pressure gradient under the pipeline is still sufficient to cause piping compared with the flat seabed case, and the maximum pressure gradient point at the downstream side of the pipeline is consistent with the breakthrough point observed in the physical experiments.

3.1 Introduction and motivation

Interaction between a submarine pipeline and an erodible seabed has attracted much attention because of its importance in offshore engineering. Of particular interest is the ‘onset of scour’, which defines the point at which sediment is washed away beneath a
pipeline. The onset of scour results in tunnel erosion below a pipeline, followed by spanning of the pipeline and subsequent self-burial (Sumer and Fredsøe, 2002) or pipeline breakout. Predicting the onset of scour is therefore of significant importance in understanding and predicting the stability of a subsea pipeline.

For the case of a pipeline partially embedded on a flat seabed, a number of investigations have been reported regarding the onset of scour. Chiew (1990) conducted a series of physical experiments and found that piping (backwards erosion of sediment) leads to the onset of scour. More specifically, they found that in steady currents, piping occurs when pressure gradient in the soil inducted by the dynamic pressure difference upstream and downstream of the pipeline exceeds the floatation gradient of the sediment around the pipe. Sumer et al. (2001) investigated this mechanism in more detail and presented an empirical expression of onset of scour in steady current for a pipeline partially buried in a flat seabed. This expression is given as

\[ \frac{U_{cr}^2}{gD(1-n)(s-1)} = 0.025 \exp \left[ 9 \left( \frac{e}{D} \right)^{0.5} \right] \]  

(3-1)

where \( U_{cr} \) is a ‘critical’ undisturbed steady current velocity (measured at the level of the top of the pipeline) above which onset of scour due to piping will occur; \( g \) is acceleration due to gravity; \( D \) is pipe diameter; \( n \) is porosity of the sediment; \( s \) is specific gravity of sediment grains, and \( e \) is embedment depth of pipeline (measured vertically from bottom of pipeline to surrounding level of the seabed, when the seabed is flat; see Figure 3-9 (a)).

A few numerical studies on the onset of scour below subsea pipelines is also reported in literature. Liang and Cheng (2005a) were the first to establish a numerical model of the onset of scour below subsea pipelines subject to steady currents. The pressure gradient that governs drives seepage flow below the pipeline is determined by solving the two-dimensional Reynolds-averaged continuity and Navier-Stokes equations with the standard \( k - \varepsilon \) turbulence closure. The seepage flow is calculated via the Laplace equation and the free water surface is tracked in the model. The critical incoming flow velocity for the onset of scour is then calculated and the results are compared with experimental data. Zang et al. (2009) also developed a numerical model for the onset of scour by solving the flow field with \( k - \omega \) turbulence model. The average pressure gradient along the buried pipeline surface is employed in the evaluation of the onset conditions with a calibration coefficient. They numerically studied the influence of flow parameters, including water depth, embedment depth, boundary layer thickness, Reynolds number (\( Re \)) and Keulegan-Carpenter (\( KC \)) number, on the pressure
difference either side of the pipeline. Gao and Luo (2010) proposed a flow-pipe-seepage sequential coupling Finite Element Method (FEM) model to simulate the coupling between the water flow-field and soil seepage-field. Their modelling indicated that the dimensionless critical flow velocity changes approximately linearly with the soil internal friction angle for the submarine pipeline partially-embedded in a sandy seabed.

All the previous experimental and numerical studies of onset of scour are based on a flat seabed. The expression in Eq.(3-1) provides a valuable prediction of the onset of scour for a pipeline on a flat seabed. However, it suggests that the onset of scour is unlikely in weak ambient field conditions (smaller than the critical velocity $U_{cr}$) for pipelines on most soils if they are embedded to certain level. Field survey data for subsea pipelines (Leckie et al., 2015) suggests that the onset of scour may still happen even if the initial (as laid) pipeline embedment is relatively large (Westgate, 2013). This implies that other mechanics may contribute to the onset of scour.

In this paper onset of scour below subsea pipelines was revisited by conducting a series of experiments and subsequent numerical analysis subject to steady currents. In the physical experiments, specific attention is paid to the effect of upstream sediment supply on local scour, and the potential for the onset of scour leading to tunnel erosion. This may be applicable for pipelines in the field were surrounding rocky outcrops or upward-sloping seaboeds may limit upstream sediment supply. The numerical simulations were conducted to improve understanding of the pressure distribution around a pipeline with different scour profiles.

### 3.2 Physical experiments

#### 3.2.1 Experiment set-up

The physical experiments for the onset of scour below subsea pipelines were conducted in the MOT test facility. A brief description and sketch figure of the MOT facility can be found in chapter 1, and will not be repeated here. The steady current velocity profile at the centre of the working section within the bottom 0.1 m of the sediment seabed (or false floor) was found to be logarithmic in all experiments (see Figure 3-2 for an example).

The flow velocity during the tests was measured by a SonTek/YSI 16-MHz 3D MicroADV (Acoustic Doppler velocimeter) at 0.66 m upstream of the model pipeline and in line with the top of the model pipe. To determine the influence of the presence of velocimeter on the flow and scour profile, some experiments were repeated without the ADV, and the scour results were similar. This suggested that the presence of the ADV
had negligible influence on the flow and the seabed around the pipe. The scour process was recorded from both sides of the MOT test section and at an oblique angle (to capture the scour at the centre of the pipe) during all the experiments.

For most of the MOT experiments, a PVC model pipeline with 50 mm in diameter was fixed in the middle of the test section and extended the entire width of the test section (see Figure 3-1). The surface of model pipe was smooth. Whitehouse (1998) mentioned that the artificially high blockage in the laboratory model can be avoided if the rate of flume cross-sectional area to model cross-sectional area is larger than 6. However, Mao (1986) stated that the blockage effect of the pipeline on the flow was very limited, where such rate is less than 3.5. In this study, the rate of O-tube cross-sectional area to model pipeline cross-sectional area is about 4, and the results of validation tests (see Figure 3-5) is still consistent with previous scour investigations (Sumer et al., 2001). Therefore, the blockage effect appears to be limited in this study.

Three types of siliceous sand were used in the present study. Soil properties, together with critical shear stress $\tau_{cr}$ for incipient motion in steady current conditions, were measured at the beginning of the study. The relevant data are listed in Table 3-1. The particle size distribution of these three types of sand are also shown in Figure 3-3.

<table>
<thead>
<tr>
<th>Table 3-1 Key characteristics of test sample.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Source</td>
</tr>
<tr>
<td>Grain size $d_{50}$ (mm)</td>
</tr>
<tr>
<td>Grain size $d_{84.1}$ (mm)</td>
</tr>
<tr>
<td>Grain size $d_{15.9}$ (mm)</td>
</tr>
<tr>
<td>Geometric standard deviation $\sigma_g = (d_{84.1}/d_{15.9})^{1/2}$</td>
</tr>
<tr>
<td>Fines percentage $(&lt; 75 \mu m)$</td>
</tr>
<tr>
<td>Specific gravity of sediment grains $s$</td>
</tr>
<tr>
<td>Porosity of the sediment $n$</td>
</tr>
<tr>
<td>$U_{cr}$ at 0.05 m (m/s)</td>
</tr>
<tr>
<td>Critical shear stress, $\tau_{cr}$ (N/m²)</td>
</tr>
</tbody>
</table>

Note: the notation $d_x$ indicates the grain diameter for which $n$% of the grains by mass is finer.

The physical experiments were conducted with different sediment, different pipeline embedment $e$ and different length of seabed upstream of the pipeline. Essentially entire experiments were divided into two categories: type (1) standard experiments; and type (2) limited sediment supply experiments.

For the standard experiments, the sediment was placed across the whole (1.8 m long) test section, see Figure 3-1 (a), the length of sediment bed at the upstream side of the pipe is about 18 D. In contrast to the standard experiments, the limited sediment supply experiments were performed to investigate the onset of scour below the pipeline when the sediment supplied from upstream was interrupted. For the limited sediment supply
experiments a box was made of 9 mm thick transparent plastic and designed to sit flush with the false floor of the working section, as shown in Figure 3-1 (b). The box only allowed 0.47 m of the 1.8 m long working section to be filled with sediment, which is less than 5 D in front of the pipe, and so upstream sediment supply became limited quickly during the tests.

To limit local scour at the transition area between the edge of the test section and the sand, a ramp with 1:2 was placed within the sand for the standard experiments (see Figure 3-1 (a)). This was effective at limiting local scour at the entrance of the test section, and the size of an associated bed from. For the limited sediment supply experiments, the scour profile at the upstream of the pipeline roughly remained smooth throughout the experiments (see Figure 3-7), and so no attempt was made to use sloping false bed in this second type of experiment.

To view under the pipeline during the onset of scour, some type (2) experiments were repeated with a transparent 50 mm diameter model pipeline with three micro cameras inside. Due to the limited distance between the camera lens and the bottom of pipeline is limited, a slender mirror is attached to the inner wall of the transparent model pipe. A schematic sketch of cameras pipeline was shown in Figure 3-4. Each camera had a view area of 70 mm along the bottom of the model pipe.
Chapter 3 Effect of limited sediment supply on onset of scour below subsea pipelines

Table 3-2 Test conditions for the onset of scour tests.

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Test type</th>
<th>Sand type</th>
<th>Initial $e/D$</th>
<th>Flow velocity $u_e$ (m/s)</th>
<th>Sand size $d_{50}$ (mm)</th>
<th>Specific gravity $s$</th>
<th>Porosity $n$</th>
<th>Shields parameter $\theta_e$</th>
<th>Pipe Reynolds number $Re_c(\times 10^4)$</th>
<th>$U_e^2/(g(s-1)(1-n))$</th>
<th>Run time (min)</th>
<th>Time to onset (s)</th>
<th>Note</th>
</tr>
</thead>
<tbody>
<tr>
<td>O1</td>
<td>1</td>
<td>A</td>
<td>0.2</td>
<td>0.66</td>
<td>0.24</td>
<td>2.67</td>
<td>0.384</td>
<td>0.28</td>
<td>3.30</td>
<td>0.86</td>
<td>3</td>
<td>60</td>
<td>--</td>
</tr>
<tr>
<td>O2</td>
<td>1</td>
<td>A</td>
<td>0.2</td>
<td>0.58</td>
<td>0.24</td>
<td>2.67</td>
<td>0.384</td>
<td>0.22</td>
<td>2.90</td>
<td>0.66</td>
<td>30</td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td>O3</td>
<td>1</td>
<td>A</td>
<td>0.2</td>
<td>0.52</td>
<td>0.24</td>
<td>2.67</td>
<td>0.384</td>
<td>0.17</td>
<td>2.60</td>
<td>0.53</td>
<td>40</td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td>O4</td>
<td>1</td>
<td>B</td>
<td>0.2</td>
<td>0.58</td>
<td>0.60</td>
<td>2.65</td>
<td>0.39</td>
<td>0.12</td>
<td>2.90</td>
<td>0.68</td>
<td>20</td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td>O5</td>
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<td>B</td>
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<td>0.69</td>
<td>0.60</td>
<td>2.65</td>
<td>0.39</td>
<td>0.17</td>
<td>3.44</td>
<td>0.95</td>
<td>40</td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td>O6</td>
<td>1</td>
<td>B</td>
<td>0.2</td>
<td>0.71</td>
<td>0.60</td>
<td>2.65</td>
<td>0.39</td>
<td>0.18</td>
<td>3.55</td>
<td>1.01</td>
<td>50</td>
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<td>--</td>
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<td>O7</td>
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<td>B</td>
<td>0.2</td>
<td>0.83</td>
<td>0.60</td>
<td>2.65</td>
<td>0.39</td>
<td>0.25</td>
<td>4.15</td>
<td>1.39</td>
<td>5</td>
<td>&lt;30</td>
<td>--</td>
</tr>
<tr>
<td>O8</td>
<td>1</td>
<td>B</td>
<td>0.1</td>
<td>0.32</td>
<td>0.60</td>
<td>2.65</td>
<td>0.39</td>
<td>0.03</td>
<td>1.60</td>
<td>0.21</td>
<td>10</td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td>O9</td>
<td>1</td>
<td>B</td>
<td>0.1</td>
<td>0.39</td>
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<td>2.65</td>
<td>0.39</td>
<td>0.05</td>
<td>1.97</td>
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<td>10</td>
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<td>--</td>
</tr>
<tr>
<td>O10</td>
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<td>B</td>
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<td>0.46</td>
<td>0.60</td>
<td>2.65</td>
<td>0.39</td>
<td>0.07</td>
<td>2.31</td>
<td>0.43</td>
<td>2</td>
<td>&lt;30</td>
<td>--</td>
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<tr>
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<td>0.69</td>
<td>0.60</td>
<td>2.65</td>
<td>0.39</td>
<td>0.17</td>
<td>3.44</td>
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<td>3</td>
<td>234</td>
<td>--</td>
</tr>
<tr>
<td>O12</td>
<td>2</td>
<td>B</td>
<td>0.2</td>
<td>0.64</td>
<td>0.60</td>
<td>2.65</td>
<td>0.39</td>
<td>0.14</td>
<td>3.18</td>
<td>0.81</td>
<td>30</td>
<td>265/467**</td>
<td>Repeat*</td>
</tr>
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<td>2</td>
<td>B</td>
<td>0.2</td>
<td>0.56</td>
<td>0.60</td>
<td>2.65</td>
<td>0.39</td>
<td>0.11</td>
<td>2.82</td>
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<td>30</td>
<td>508/1157**</td>
<td>Repeat*</td>
</tr>
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<td>2</td>
<td>B</td>
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<td>2.65</td>
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<td>2.54</td>
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<td>30</td>
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<td>--</td>
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<td>0.45</td>
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<td>0.07</td>
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<td>30</td>
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<td>2.65</td>
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<td>0.09</td>
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<td>O17</td>
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<td>B</td>
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<td>0.48</td>
<td>0.60</td>
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<td>B</td>
<td>0.1</td>
<td>0.39</td>
<td>0.60</td>
<td>2.65</td>
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<td>0.05</td>
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<td>A</td>
<td>0.1</td>
<td>0.50</td>
<td>0.24</td>
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<td>0.16</td>
<td>2.48</td>
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<td>1</td>
<td>&lt;20</td>
<td>--</td>
</tr>
<tr>
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<td>A</td>
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<td>0.58</td>
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<td>&lt;10</td>
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<td>0.65</td>
<td>0.24</td>
<td>2.67</td>
<td>0.384</td>
<td>0.27</td>
<td>3.24</td>
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<td>1</td>
<td>&lt;20</td>
<td>--</td>
</tr>
<tr>
<td>O22</td>
<td>2</td>
<td>A</td>
<td>0.2</td>
<td>1.17</td>
<td>0.24</td>
<td>2.67</td>
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<td>0.92</td>
<td>5.87</td>
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<td>&lt;20</td>
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<td>A</td>
<td>0.2</td>
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<td>0.2</td>
<td>0.79</td>
<td>0.24</td>
<td>2.67</td>
<td>0.384</td>
<td>0.41</td>
<td>3.97</td>
<td>1.24</td>
<td>1</td>
<td>&lt;10</td>
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</tr>
<tr>
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<td>A</td>
<td>0.2</td>
<td>0.72</td>
<td>0.24</td>
<td>2.67</td>
<td>0.384</td>
<td>0.33</td>
<td>3.59</td>
<td>1.01</td>
<td>5</td>
<td>213/90**</td>
<td>Repeat*</td>
</tr>
<tr>
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<td>A</td>
<td>0.2</td>
<td>0.60</td>
<td>0.24</td>
<td>2.67</td>
<td>0.384</td>
<td>0.23</td>
<td>3.00</td>
<td>0.71</td>
<td>3</td>
<td>102</td>
<td>--</td>
</tr>
<tr>
<td>O27</td>
<td>2</td>
<td>A</td>
<td>0.2</td>
<td>0.52</td>
<td>0.24</td>
<td>2.67</td>
<td>0.384</td>
<td>0.17</td>
<td>2.59</td>
<td>0.53</td>
<td>15</td>
<td>722/840**</td>
<td>Repeat*</td>
</tr>
<tr>
<td>O28</td>
<td>2</td>
<td>C</td>
<td>0.2</td>
<td>0.38</td>
<td>0.22</td>
<td>2.72</td>
<td>0.57</td>
<td>0.10</td>
<td>1.88</td>
<td>0.39</td>
<td>1</td>
<td>30</td>
<td>--</td>
</tr>
</tbody>
</table>

Note: model pipe diameter is 50 mm. * the same conditions was repeated with camera pipe. ** the time to onset in the repeat test
3.2.2 Testing conditions

A complete list of test conditions and results is given in Table 3-2, in which \( U_c \) is the undisturbed steady current flow velocity measured at the level of the top of the pipeline; the embedment depth of pipeline is measured vertically from bottom of pipeline to surrounding level of the seabed, when the seabed is flat (see Figure 3-9); and \( \theta_c \) is the Shields parameter in steady current flow conditions defined by

\[
\theta_c = \frac{u^2_c}{g(s - 1)d_{50}}, \quad (3-2)
\]

where \( u_c \) is current friction velocity which can be determined based on the logarithmic velocity profile

\[
U_c(z) = \frac{u_c}{\kappa} \ln \left( \frac{Z}{z_0} \right), \quad (3-3)
\]

where \( z \) is the height above seabed; \( z_0 \) is bed roughness length; \( \kappa \) is von Karman’s constant ( = 0.41). The bed roughness length \( z_0 \) is estimated using the expression of Christoffersen and Jonsson (1985) given by

\[
z_0 = \frac{k_s}{30} \left[ 1 - \exp \left( \frac{-u_c k_s}{27v} \right) \right] + \frac{v}{9u_c}, \quad (3-4)
\]

where \( v \) is the kinematic viscosity of water and \( k_s \) is the Nikuradse roughness which is taken as \( 2.5d_{50} \) in this study.

The pipe Reynolds number in steady current is

\[
Re_c = \frac{U_c D}{\nu} \quad (3-5)
\]

The viscosity of water is taken to be \( 1 \times 10^{-6} \text{ m}^2/\text{s} \) throughout.

Each of the experiments were run until at least one of the following three conditions occurred:

1) the onset of scour;
2) No onset of scour, but an equilibrium profile was achieved (like test O3, O6 and O18), or the pipeline was buried more than 80% (like test O2, O4 and O5);
3) In the type (1) standard experiments, the test had run for so long that the sediment supply from upstream started to reduce (observed as a lowering of the upstream seabed, like test O15).

Table 3-2 also shows the duration of each test in minutes, and the ‘time to onset’ in seconds, which is the time from the start of a test, to the time of the onset of scour was observed.
3.3 Mechanisms of the onset of scour

3.3.1 Findings from previous investigations

Figure 3-5 displays the curve defined by Eq.(3-1) together with the experimental results for steady current reported in Sumer et al. (2001). It should be noted that in Sumer et al. (2001) each experiment in steady current lasted on the order of 10 seconds and care was taken throughout to level the seabed and ensure no build-up of sand on the downstream side of the pipe. The similar procedure was followed in the type (1) standard experiments, tests O7 and O10 as the test of critical flow conditions corresponding to their embedment to demonstrate the onset of scour occurred in the MOT, which are also displayed in Figure 3-5. It indicates that present tests results collapse well with the previous investigation and gives confidence on the new MOT test facility.

3.3.2 Flat and uneven seabed onset of scour experiments

Based on the observations prior to the onset of scour, all tests in Table 3-2 for which the onset of scour was observed could be divided into two groups: (a) flat seabed onset of scour tests, in which the seabed around the embedded pipeline remained flat before onset of scour occurred (see Figure 3-7 (a) for example), and (b) uneven seabed onset of scour tests, in which the seabed morphology was no longer flat due to luff and lee wake erosion around the pipeline prior to the onset of scour (see Figure 3-7b).

Figure 3-6 presents the results in terms of these two groups, plotted together with Eq.(3-1). For most flat seabed onset of scour tests it is evident that the onset of scour was observed around or above the curve of Eq.(3-1). Moreover, in all these flat seabed onset of scour experiments the onset of scour generally happened very quickly (the steady current lasted no more than 30 s), which is also consistent with the observation in Sumer et al. (2001). Furthermore, compared with type (1) standard experiments (eg. Test O7 and Test O10) the onset of scour also happened very quickly on a flat seabed for type (2) limited sediment supply experiments (eg. Test O19-O24). This indicates that sediment supply from upstream side of pipeline has no effect on the flat seabed onset of scour tests, which is not a surprise because the onset of scour occurred so fast that sediment had not started to transport significantly forwards the pipeline. Moreover, it also indicates that the presence of the box has no influence on this type of the onset of scour tests. For instance, the consistent results were obtained without box (Test O10) and with box (Test O19).
Chapter 3 Onset of scour below subsea pipelines

In contrast to the flat seabed onset of scour tests, it is shown in Figure 3-6 that the uneven seabed onset of scour tests resulted in the onset of scour below the curve of Eq.(3-1). In addition, all the uneven seabed onset of scour tests take a longer time (larger than 30 s) for onset of scour compared with the flat seabed onset of scour tests. Moreover, it is observed that a scour hole was formed at the upstream side of the pipeline and sediment was built up at the downstream side of the pipeline before the onset of scour occurred. The general scour profile prior to onset of scour was similar to that shown in Figure 3-7b; this figure is a snapshot at the point the onset of scour occurred and shows the mixture of sand and water breaking through the sediment slope at the downstream side of the pipe. This breakthrough is consistent with the description of the piping process given in literature (Chiew, 1990; Sumer et al., 2001).

Another thing that should be noted is that most of the uneven seabed onset of scour tests are the type (2) limited sediment supply tests. It shows that the onset of scour can still happen when the combination of velocity and pipeline embedment is not sufficient to cause scour due to Eq.(3-1), if upstream sediment supply is limited. Apart from that, the onset of scour test O1 is special for some extent, which is a type(1) standard test, in other words, the upstream sediment supply was not limited artificially, however, the onset of scour happens below the curve defined by Eq.(3-1). During the time period (about 60s) before the onset of scour happened for test O1, it is observed that local scour around pipeline developed fairly fast, in contrast, the rate of sediment transport to the front of pipeline is relatively slow, both effects lead to a temporary sediment supply shortage, and immediately the onset of scour occurred on an uneven seabed. It is still followed the feature of other uneven seabed onset of scour tests.

3.3.3 Onset of scour below pipeline with limited sediment supply

Figure 3-8 (a) displays vortices around the pipeline resting on a plane bed, as observed by Mao (1987). Vortex A is formed in front of the pipe, Vortex B and C are formed at the rear of the pipe. For the flat seabed onset of scour tests, the onset of scour occurred very quickly due to the pressure different generated by flow between the upstream and downstream sides of the pipe. The vortices around the pipeline have no time to transport the sediment around them, therefore, the seabed around pipeline still remains flat prior to the onset of scour.

Figure 3-8 (b) is a convenient way to interpret changes in the seabed morphology around the pipeline prior to the onset of scour for the uneven onset of scour experiments.
Chapter 3 Onset of scour below subsea pipelines

On this figure several sediment transport fluxes $q_{in}$, $q_{out}$, $q_{luff}$, $q_{flyover}$ and $q_{lee}$ are drawn to define the volumetric transport rate at various locations around the pipe. Observations showed that the free stream flow transported sediment to the vicinity of pipeline (denoted by $q_{in}$) by means of bed load and suspended load in live bed conditions. Meanwhile, the vortex A formed at the upstream of pipeline entrained sediment, lifting it into suspension. If vortex A was sufficiently strong, sediment was advected over the pipeline leading to sediment transport $q_{luff}$, and combined with (all or some fraction of) the sediment supplied from the free stream, leading to $q_{flyover}$. At the downstream side of the pipe, vortex B transported sediment in bed load back towards the pipe, namely $q_{lee}$. The redundant sediment was transported to the downstream of the pipeline and washed away which can be labelled $q_{out}$. Based on the sediment conservation equation, the rate of volume erosion from the upstream of the pipeline per unit length along the pipeline, can therefore be written as:

$$\frac{d(V_{up})}{dt} = \frac{q_{in} + q_{luff} - q_{flyover}}{1 - n},$$

whilst the rate of volume erosion for the downstream of the pipeline per unit along the pipeline can be described as:

$$\frac{d(V_{down})}{dt} = \frac{q_{flyover} + q_{lee} - q_{out}}{1 - n}.$$  \hfill (3-7)

The magnitude of $V_{up}$ and $V_{down}$ can lead to direct changes in the scour profile around the pipe. In particular, if the upstream supply $q_{in}$ is not sufficient, a negative $V_{up}$ may occur, which leads to the formation of a scour depression in front of the pipe, whereas a positive $V_{down}$ causes local sediment build-up downstream of the pipe. Together these changes in the scour profile ultimately alter the embedment of the pipe.

3.3.4 Description of experiments with limited sediment supply

To describe the uneven onset of scour tests (i.e. O25, O26, O27, O28 etc.), the scour process can be divided into four stages.

**Stage 1:** In the first stage, vortex A eroded the sand bed at the corner between the pipeline and bed, leading to a local scour depression and a reduction in the local embedment at the upstream side of the pipe. On the downstream side of the pipe, vortex B transported sediment towards the pipeline and led to a local build-up of sediment. The net result of the upstream erosion and downstream sediment build up was to cause an
increase in the embedment of the pipeline (according to the definition of embedment in Figure 3-9 (b)).

**Stage 2:** In the second stage, no further increase in the scour depression at the upstream side of the pipeline was observed (suggesting $q_{luff} \sim 0$), whilst on the downstream side of the pipeline the sediment continued to build up. Overall, the rate at which the embedment of the pipeline increased was fastest during this stage until, at some point, the downstream build-up of sediment reached a maximum value. This coincided with an equilibrium downstream scour profile equivalent to that shown in Figure 3-7 (b).

**Stage 3:** In the third stage, there was a noticeable reduction in sediment supplied from upstream (i.e. $q_{in}$ reduced) and this was associated with renewed growth of the scour depression in front of the pipeline (suggesting that $q_{luff} > 0$). No change in the downstream profile was observed, and so the embedment of the pipeline (driven by upstream scouring) began to reduce, see the screenshot from the camera inside the model pipeline Figure 3-10 (a).

**Stage 4:** When the embedment of the pipeline reduced to a certain point, a mixture of sand and water broke through from one point on the downstream side of the pipeline and the onset of scour immediately happened. The mixture of sand and water was similar in appearance to that for the flat seabed onset of scour tests (see Figure 3-7 (b) and the screenshot from the camera inside the model pipeline Figure 3-10 (b)).

The variation of pipeline embedment tests O12, O13, O25 and O27 are plotted in Figure 3-11. In this figure, the total embedment of the pipeline was measured from the video recordings outside the O-tube and micro cameras inside the model pipeline to obtain the best estimation of the embedment at the centre of the pipe.

From Figure 3-11, it is evident that at first the total embedment of the pipeline increased (i.e. Stage 1). This increase then continued, but at a faster rate, until the embedment of the pipeline reached to a maximum value (i.e. Stage 2). The embedment then proceeded to reduce when the upstream supply reduced (Stage 3), until the onset of scour occurred (Stage 4).

A second observation from Figure 3-11 is that at a given embedment, for a test with lower flow velocity, the time prior to the onset of scour was longer (eg. Test O25, O27 and Test O12, O13). The breakout of a mixture of sand and water in the final stage of the uneven onset of scour experiments was in agreement with the description of piping in Sumer et al. (2001) (see also Figure 3-7 (b)). This suggests that for the uneven onset of scour experiments the pipeline embedment could vary with time without the onset of
scour until the embedment reduced to some critical embedment depth where piping occurred. Interestingly, based on Figure 3-11, this critical embedment appears to be similar to that defined by Eq.(3-1) despite the fact that Eq.(3-1) is derived from experiments on a flat seabed. In this study, the pressure difference between the upstream and downstream sides of the pipeline was not measured physically. Therefore, in the next section numerical analysis is presented. This analysis was performed to investigate the onset of scour for the uneven onset of scour experiments with limited sediment supply, where the onset of scour was observed when the surrounding seabed was not flat.

3.4 Numerical simulations

The onset of scour underneath a partially buried pipeline in current is dependent on two flow processes, one is the flow around the pipeline and the other one is the seepage-flow within the porous underlying sediment. These two flow processes were simulated separately by FLUENT software in this study to back analyse the experimental results.

3.4.1 Governing equations and numerical method

3.4.1.1 Flow field

The governing equations for the incompressible flow above the seabed are the two-dimensional Reynolds-Averaged Navier-Stokes (RANS) equations, which can be written in Cartesian coordinate form as:

\[
\frac{\partial u_i}{\partial t} + u_j \frac{\partial u_i}{\partial x_j} = - \frac{1}{\rho} \frac{\partial p}{\partial x_i} + \nu \frac{\partial^2 u_i}{\partial x_i^2} + \frac{\partial}{\partial x_j} (-\bar{u}_i \bar{u}_j),
\]

(3-8)

\[
\frac{\partial u_i}{\partial x_j} = 0,
\]

(3-9)

where \( u_i \) is the velocity component, \( u_i \) and \( u_j \) are the fluctuating velocities, \( t \) is the time, \( \rho \) is the density of fluid, \( p \) is the pressure, \( \nu \) is the kinematic viscosity of fluid, \( x_i \) (or \( x_j \)) are the coordinates in horizontal and vertical directions, respectively. The Reynolds stress tensor \(-\bar{u}_i \bar{u}_j\) is computed by

\[
-\bar{u}_i \bar{u}_j = \nu_t \left( \frac{\partial u_i}{\partial x_j} + \frac{\partial u_j}{\partial x_i} \right) - \frac{2}{3} k \delta_{ij}
\]

(3-10)

where \( \nu_t \) is the turbulent viscosity and \( k \) is the turbulent energy. The turbulent viscosity is obtained by solving the SST (shear stress transport) \( k - \omega \) turbulence equation developed by Menter (1994). In practice, the \( k - \varepsilon \) models are generally well behaved in the far field, while the \( k - \omega \) models are more accurate and much more numerically
stable in the near-wall region. Moreover, with the freestream independence of the \( k - \varepsilon \) model in the far field, which makes the SST \( k - \omega \) model more accurate and reliable for a wider class of flows than the standard \( k - \omega \) model. Menter (1994) suggested a blended model, which has a similar form to the standard \( k - \omega \) model:

\[
\frac{\partial k}{\partial t} + u_i \frac{\partial k}{\partial x_i} = \frac{\partial}{\partial x_i} \left( (v + \sigma_k v_t) \frac{\partial k}{\partial x_i} \right) + P_k - \beta^* \omega k, \tag{3-11}
\]

\[
\frac{\partial \omega}{\partial t} + u_i \frac{\partial \omega}{\partial x_i} = \frac{\partial}{\partial x_i} \left( (v + \sigma_\omega v_t) \frac{\partial \omega}{\partial x_i} \right) + \frac{\gamma}{v_t} P_k - \beta \omega^2 \tag{3-12}
\]

\[+ (1 - F_1) \frac{2\sigma_\omega^2}{\omega} \frac{\partial k}{\partial x_i} \frac{\partial \omega}{\partial x_i},\]

where \( \omega \) is the specific dissipation rate, \( P_k \) is the turbulent kinetic energy production, \( v_t \) is the turbulent eddy viscosity, \( \sigma_k \) and \( \sigma_\omega \) are the turbulent Prandtl numbers for \( k \) and \( \omega \) respectively, \( F_1 \) is the blending function, which is defined as

\[
F_1 = \tanh \left\{ \min \left[ \max \left( \sqrt{\frac{k}{\beta^* \omega y}}, \frac{500 v}{y^2 \omega} \right) \frac{4\rho \sigma_\omega^2 k}{C_{D_{k\omega} y^2}} \right]^4 \right\}. \tag{3-13}
\]

where \( y \) is the normal distance to the wall and \( C_{D_{k\omega}} \) is the positive portion of the cross-diffusion term

\[
C_{D_{k\omega}} = \max \left( 2\rho \sigma_\omega^2 \frac{1}{\omega} \frac{\partial k}{\partial x_i} \frac{\partial \omega}{\partial x_i}, 10^{-10} \right). \tag{3-14}
\]

\( F_1 \) is equal to zero away from the surface (\( k - \varepsilon \) model), and switches over to one inside the boundary layer (\( k - \omega \) model).

The turbulent eddy viscosity is defined as follows:

\[
\nu_t = \frac{a_1 k}{\max(a_1 \omega, SF_2)}, \tag{3-15}
\]

where \( S \) is the invariant measure of the strain rate and \( F_2 \) is a second blending function defined by:

\[
F_2 = \tanh \left\{ \max \left( \frac{2\sqrt{k}}{\beta^* \omega y}, \frac{500 v}{y^2 \omega} \right) \right\}. \tag{3-16}
\]

The coefficients \( \sigma_k, \sigma_\omega, \alpha, \beta \) in Eq. (3-11) and Eq. (3-12) are computed with the general form

\[
\phi = F_1 \phi_1 + (1 - F_1) \phi_2, \tag{3-17}
\]

where \( \phi_1 \) corresponds to coefficients from the \( k - \omega \) model and \( \phi_2 \) corresponds to coefficients of the \( k - \varepsilon \) model.

\[
\gamma = \gamma_1 F_1 + (1 - F_1) \gamma_2, \tag{3-18}
\]
The model closure coefficients in Eqs. (3-11)-(3-20) are listed in Table 3-3.

<table>
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<th>$\sigma_{k,1}$</th>
<th>$\sigma_{k,2}$</th>
<th>$\sigma_{w,1}$</th>
<th>$\sigma_{w,2}$</th>
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<th>$\beta^*$</th>
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<th>$\beta_2$</th>
<th>$\kappa$</th>
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</thead>
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<td>2.0</td>
<td>1.168</td>
<td>0.31</td>
<td>0.09</td>
<td>0.075</td>
<td>0.0828</td>
<td>0.41</td>
</tr>
</tbody>
</table>

3.4.1.2 Seepage flow field

Considering the sand bed as a porous medium, the seepage flow through the sand bed is determined by the permeability of the medium. If the sand grain size is smaller than about 1 mm, the seepage flow in the sand bed is laminar and the velocity of seepage flow can be calculated by Darcy’s Law. Alternatively, if the sand grain size is larger than about 1 mm, the seepage flow becomes turbulent, which is governed by the viscosity of water and the kinetic energy of the water. Darcy’s Law is now replaced by the Forchheimer Law (Soulsby, 1997). In this study, the range of sand properties in the numerical simulation is limited to the range of physical experiments, therefore, Darcy’s Law is adopted in this study with the form as

$$U_{Bi} = -\frac{K_p}{\rho \nu} \frac{dp}{dx_i} \quad (3-21)$$

where $U_{Bi}$ is the discharge of water per unit area in $x_i$ direction and $dp/dx_i$ is the pressure gradient, $K_p$ is the specific permeability ($m^2$), which is governed by the properties of the porous medium, such as grain size and porosity. Ergun (1952) proposed an expression for $K_p$:

$$K_p = \frac{d_{50}^3 \varepsilon^3}{150 (1-\varepsilon)^2} \quad (3-22)$$

where $\varepsilon$ is the void fraction.

3.4.2 Validation of the numerical model

The flow-field model is validated against the experimental data (Bearman and Zdravkovich, 1978) of steady current flow past a pipeline resting on an impermeable plane boundary with $Re = 4.5 \times 10^4$. In the experiments of Bearman and Zdravkovich (1978), the boundary layers thickness is 0.8 D at the position of the cylinder. To obtain a
similar boundary layer at the pipeline position in the present numerical simulations, a
long computational domain 200 D in length and 10 D in height was employed without
the pipeline. The location corresponding to 0.8 D thick boundary layer was found to be
around 60 D from the inlet boundary. Therefore, a rectangular computational domain of
120 D in length and 10 D in height was discretised with structured quadrilateral four node
elements to simulate the flow past a cylinder at the centre of the grid. Figure 3-12
illustrates the computational mesh in the vicinity of the cylinder for $Re = 4.5 \times 10^4$. The
cylinder surface is discretised by 210 nodal points, and the minimum element size in the
radial direction next to the pipeline surface was 0.0002 D, which resulted in a maximum
non-dimensional mesh size next to the solid surface, $y^+$, of less than 1 (where $y^+ = u_c^* \Delta / \nu$, with $u_c^*$ being the friction velocity and $\Delta$ the dimensional mesh size). The total
number of nodes employed to discretise the computational domain is 30600. The
convergence of the numerical results with respect to the mesh density is also tested. It is
found that the numerical results change little if the mesh density is increased further. The
mesh densities in the following computation are similar to that shown in Figure 3-12. The
conditions $y^+ \leq 1$ was selected to ensure mesh convergence. Finally, a non-dimensional
time step of $U_m \Delta t / D = 0.0024$ was used in the calculations.

Figure 3-13 shows the calculated distributions of pressure coefficient along the cylinder
surface and on the plane boundary together with the measured data of Bearman and
Zdravkovich (1978) for $e / D = 0.0$ and $e / D = 0.1$. The pressure coefficient $C_p$ in Figure
3-13 is defined as

$$C_p = \frac{p - p_\infty}{0.5 \rho U_c^2} \quad (3.23)$$

where $p$ is the pressure; $p_\infty$ is the pressure at far field; $\rho$ is the fluid density; $U_c$ is
amplitude of steady current at the top and upstream of the pipe. From comparison, it is
seen that the calculated pressure coefficient agrees well with the experiment data on the
pipeline surface as well as the seabed surface. The pressure distribution along the seabed
calculated from the flow field simulation is used to calculate the seepage flow in the soil.

3.4.3 Flow structure and pressure distribution

To investigate the flow structure and pressure distribution along seabed prior to the
onset of scour, four typical seabed profiles were chosen based on the general scour profile
observed for the uneven seabed onset of scour experiments. The computational mesh and
seabed profile in the vicinity of the pipeline are shown in Figure 3-14. Specifically, Case
Chapter 3 Onset of scour below subsea pipelines

N1 represents the initial embedment conditions on a flat sand bed at the start of Stage 1 (for $e/D = 0.2$). Subsequently, Case N2 represents a point at the end of Stage 2, in which the embedment has reached the peak embedment. Case N3 then represents a point during Stage 3 when the embedment has reduced to an embedment equivalent to the initial embedment (i.e. $e/D = 0.2$), but on an uneven seabed. Finally, Case N4 represents the scour profile at a smaller embedment than the initial equivalent embedment (i.e. when $e/D = 0.1$). It should be noted that the number in the bracket indicates the embedment relative to the far field seabed.

The calculated flow field for the different seabed profiles are shown in Figure 3-15. As can be seen in Figure 3-15a, the three major vortices around the pipeline on the flat seabed observed by Mao (1986) are reproduced numerically. It is observed from Figure 3-15b,c,d that the luff vortex always exists at the nose of the pipe, even as the scour hole at the upstream side of the pipeline continually enlarges. At the downstream side of the pipe, the lee wake vortex always exists.

The pressure coefficient distribution along the seabed for the four cases are illustrated in Figure 3-16 (the centre of the pipeline is located at $x = 0$). Figure 3-16 confirms the existence of the pressure drop between the upstream and downstream sides of the pipeline when the pipeline is placed on a flat seabed as well as an uneven seabed. Moreover, it is found from the calculated results that for the four cases the pressure remains almost constant for about 1D length at the upstream side of the pipeline and more than 3D length at the downstream side of the pipe. This characteristic will be used in analysing the pressure gradient below the pipeline. The change in pressure coefficient between the upstream and downstream sides of pipeline is listed in Table 3-4. It is found that in the four cases, the flat seabed case (case N1) has the largest pressure difference, followed by case N4 and case N2. The uneven seabed case with $e/D = 0.2$ has the smallest pressure difference. In other words, the pressure differences for uneven seabed in the four cases are all smaller than the flat seabed case with $e/D = 0.2$.

To further investigate the relationship between the pressure difference and the embedment, several supplementary flat seabed and uneven seabed cases were simulated with $e/D = 0.1, 0.3, 0.41$ respectively. The corresponding calculated results are also listed in Table 3-4. The pressure coefficient along the seabed in the vicinity of the pipeline for the flat seabed cases are also shown in Figure 3-17, which indicates that the pressure difference gradually decreases with the increase of embedment. This trend is not
surprising considering the extreme conditions ($e/D = 1.0$), where no pressure difference will exists due to the pipeline.

Figure 3-18 shows the pressure coefficient distribution along the uneven seabed. For the cases shown in Figure 3-18, the free stream embedment is equal to the pipeline local embedment in an uneven seabed. As for the flat seabed cases, the pressure difference gradually decreases with increase in embedment. Moreover, the pressure distribution is still quite uniform in the front and behind the pipe.

Figure 3-19 shows the pressure drop $\Delta C_p$ versus pipeline embedment $e/D$ for the flat seabed cases as well as the uneven seabed cases. The embedment labelled in the legend is the free stream embedment, and the local embedment is consistent with the X-axis. It again shows the pressure drop is highly dependent on $e/D$ for pipeline placed on a flat plane, moreover, for the uneven seabed cases, the pressure drop is a little more complicated than the flat seabed situations. However, if the far field embedment is equal to the local embedment, the pressure drop of uneven seabed cases (eg. N9, N3, N10, N11) is generally smaller than the flat seabed cases (eg. N1, N6, N7, N8).

### 3.4.4 Seepage flow pressure gradient

For the seepage-field simulation, a computational domain of 120 D in length and 5 D in height was discretised by four-node quadrilateral elements. The top boundary was the seabed and buried part of pipe. The length and mesh node distribution on the top boundary were exactly the same as the bottom boundary on flow-field mesh, and the buried part of the pipeline is defined as a wall. The results for pressure distribution along the seabed calculated from the flow-field simulation were directly transferred to the seepage-field simulation as the pressure inlet boundary condition. The sides and bottom of the seepage-field domain were defined as wall. The seepage flow in the seabed was regarded as laminar flow as mentioned above, therefore, the boundary layer and wall function were no longer applied in this simulation. The specific permeability $K_p$ is calculated by Eq. (3-13), and the seepage flow velocity is calculated by Eq. (3-11). To investigate the pressure gradient and seepage flow path prior to the onset of scour in the uneven seabed onset of scour tests, the seepage flow field was simulated corresponding to the flow field study in Figure 3-14 and the flow simulation listed in Table 3-4.

The onset of scour due to piping is driven by the pressure difference across the pipe. Chiew (1990) outlined that the piping process would occur when the pressure gradient...
\[ \frac{\partial(p/\gamma)}{\partial sp} \] exceeded the floatation gradient \((s - 1)(1 - n)\), in which the pressure gradient is defined as

\[
\frac{\partial}{\partial sp} \left( \frac{p}{\gamma} \right) = \frac{p_{upstream} - p_{downstream}}{\gamma sp},
\]

(3-24)

where \(\gamma\) is the specific weight of water; \(p_{upstream}\) is the pressure at the upstream side of the pipe; \(p_{downstream}\) is the pressure at the downstream side of the pipe; and \(sp\) is seepage flow path. Therefore, the point with maximum pressure gradient along the seabed at the downstream side of the pipeline is most likely point to succumb to piping.

Figure 3-20 illustrates the calculated pressure distribution in the water and in the soil for the four typical seabed morphologies around the pipeline prior to the onset of scour. The numerical labels in the figure indicate the pressure coefficients. The seepage flow velocity streamlines are also drawn in the figures, the arrow on the lines indicate the flow direction. For all four seabeds, the seepage flow pressure at the upstream side of the pipeline were all higher than those at the downstream side of pipe, and this pressure drop further induce seepage flow within the soil underneath the pipe, which is consistent with the results of the flow-field simulation.

For the flat seabed case (Figure 3-20 (a)), the contour of seepage-pressure and seepage streamlines showed that the maximum seepage gradient between the upstream and downstream side of the pipeline is in the direction upwards tangential to the pipeline surface, as in good agreement with the direction and location of the breakout of the sand particles observed in the experiments (see Figure 3-7 (a)) and previous results (Gao and Luo, 2010; Sumer et al., 2001). It also can be seen from Figure 3-20 (a) that, as described in Liang and Cheng (2005a), there are two numerical singular points at the corner in front and behind the pipeline at the water-soil interface. However, the exit seepage gradient at the rear corner point of the pipeline surface can still be obtained by utilizing the grid point next to the singular point (Zang et al., 2009). Moreover, the seepage exit pressure gradient distribution along the seabed at the downstream side of the pipeline are illustrated in Figure 3-21. It can be seen in Figure 3-21 (a), for the pipeline placed on a flat seabed, the seepage exit pressure gradient is continually increasing towards the corner of water-soil interface and pipeline. Therefore, the maximum seepage exit pressure gradient can be calculated based on the average pressure gradient along the buried pipeline surface.

For the uneven seabed situations, such as cases N2-N4 shown in Figure 3-20b,c,d, the singular points still exists at the upstream corner between the pipeline and seabed, however, at the downstream side of the pipe, it can be seen in the figures that the seepage
flow streamlines are perpendicular to the water-soil interface. Furthermore, from Figure 3-21 (b), it can be found that the maximum seepage exit pressure gradient is not at the corner point of seabed and pipe, instead, the pressure gradient increases from the corner point along the seabed until reach a peak point not far from the corner point. On the other side, from observation of physical experiments, when the seabed is uneven due to the lee and luff erosion, the piping breaking point at the downstream of the pipeline was no longer located at the touch point of the pipeline and seabed, but at a certain point of the scour slope, such as Figure 3-7 (b). This result of numerical simulation explains why the breaking point observed in the physical experiments was a small distance downstream of the pipeline. Therefore, the shortest seepage path in the uneven seabed case should be the sum of two parts: (1) a portion of the buried perimeter of the pipe, and (2) a line normal to the seabed slope downstream of the pipe. It can be noted that this seepage length is shorter than the length for a pipeline with an equivalent embedment on a flat bed.

The maximum seepage exit pressure gradients have been calculated and the results are listed in Table 3-4. For the four cases with typical seabed morphology prior to the onset of scour, it can be observed that the pressure gradient decreases as the scour progressed from Case N1 to Case N2, and this was due to an increase in embedment which led to a reduction in pressure gradient. However, after scour developed to the point represented by Case N3, the pressure gradient had again reduced to a value similar to that at the start of stage 1 in Case N1 (and consistent with the discussion in above section). The pressure gradient then continued to increase in case N4, which has the largest pressure gradient in these four cases. This large pressure gradient is expected to easily cause piping and the onset of scour, explaining why the onset of scour occurred under the flow conditions.

<table>
<thead>
<tr>
<th>Case No.</th>
<th>e/D</th>
<th>Free stream e/D</th>
<th>Morphology</th>
<th>∆C_p</th>
<th>SP (D=1)</th>
<th>Maximum C_p gradient</th>
</tr>
</thead>
<tbody>
<tr>
<td>N1</td>
<td>0.2</td>
<td>0.2</td>
<td>Flat</td>
<td>0.91</td>
<td>1.287</td>
<td>0.707</td>
</tr>
<tr>
<td>N2</td>
<td>0.41</td>
<td>0.2</td>
<td>Uneven</td>
<td>0.863</td>
<td>1.747</td>
<td>0.494</td>
</tr>
<tr>
<td>N3</td>
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<td>0.2</td>
<td>Uneven</td>
<td>0.821</td>
<td>1.244</td>
<td>0.660</td>
</tr>
<tr>
<td>N4</td>
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<td>0.2</td>
<td>Uneven</td>
<td>0.871</td>
<td>0.887</td>
<td>0.982</td>
</tr>
<tr>
<td>N5</td>
<td>0.0</td>
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<td>Flat</td>
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<td>--</td>
<td>--</td>
</tr>
<tr>
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<td>0.1</td>
<td>Flat</td>
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<td>0.902</td>
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<td>0.3</td>
<td>Flat</td>
<td>0.834</td>
<td>1.591</td>
<td>0.524</td>
</tr>
<tr>
<td>N8</td>
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<td>Flat</td>
<td>0.8</td>
<td>1.879</td>
<td>0.426</td>
</tr>
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<td>0.1</td>
<td>Uneven</td>
<td>0.87</td>
<td>0.887</td>
<td>0.981</td>
</tr>
<tr>
<td>N10</td>
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<td>0.786</td>
<td>1.510</td>
<td>0.521</td>
</tr>
<tr>
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<td>0.41</td>
<td>Uneven</td>
<td>0.721</td>
<td>1.747</td>
<td>0.413</td>
</tr>
</tbody>
</table>
below the critical velocity corresponding to the initial flat seabed embedment in the uneven seabed onset of scour experiments.

From the Table 3-4, for a given embedment it can be seen that the pressure difference in the flat seabed case is larger than for the uneven seabed (as can also be seen in Figure 3-19), however, the length of seepage path for flat seabed is longer. Consequently both effects tend to balance each other out and so, in part, these results confirm why the embedment at which scour occurred in Figure 3-11 is similar to Eq.(3-1).

3.5 Conclusions

The conclusions derived from the physical experiments are summarized as follows:

1) The onset of scour can happen earlier than the critical velocity defined by Sumer et al. (2001) if the upstream sediment supply is restricted.

2) If the onset of scour happens due to limited sediment supply it will take a relatively longer time (compared with velocities above the critical velocity) for the onset of scour to finally occur.

3) When the onset of scour happens under a limited upstream sediment supply condition, the sand bed topography just before onset is quite different from the initial profile. This is due to luff and lee wake erosion. It should be noted that conclusions that both luff and lee wake erosion affect the potential for piping and tunnel scour is consistent with comments in Sumer et al. (2001), in which they point out that the presence of vortices in front of the pipeline and in the lee-wake may contribute to the onset of scour.

4) Observations from micro cameras within a transparent pipeline confirmed that onset of scour occurs due to piping for the uneven seabed onset of scour experiments with limited sediment supply.

From the numerical simulations, the following conclusions can be summarized:

1) The luff vortex still exists in the scour hole at the upstream side of the pipeline prior to the onset of scour on an uneven seabed, and the lee wake vortex is maintained the sediment slope at the downstream side of pipe.

2) The pressure difference between the upstream and downstream sides of the pipeline on an uneven seabed is smaller than on a flat seabed with similar embedment, if the far field embedment is equal to the local embedment for a flat seabed.
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3) Even if significant local scour alters the sea bed profile around a pipeline, the pressure gradient is not significantly different to that for a pipeline with equivalent embedment on a flat seabed. This is because the scoured profile leads to a reduced pressure gradient but also a shorter seepage path.
References


Mao, Y., 1987. The interaction between a pipeline and an erodible bed. SERIES PAPER TECHNICAL UNIVERSITY OF DENMARK(39).


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Figure 3-1 Experiment setup: (a) Seabed model 1 (standard experiment); (b) Seabed model 2 (limited sediment supply experiment).

Figure 3-2 Example flow velocity profile above the false floor.
Figure 3-3 Particle size distribution of the model sand in the MOT: (a) A sand; (b) B sand; (c) C sand.
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Figure 3-4 Schematic of camera pipe.

Figure 3-5 Experiments on the onset of scour. Test results from Sumer et al. (2001) and results from this investigation (O7 and O10) are shown with Eq. (3-1).
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Figure 3-6 Flat and uneven seabed onset of scour tests.

Figure 3-7 The seabed profiles when the onset of scour which happened in two experiments: (a) Test O15; (b) Test O9.
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Figure 3-8 Sediment transport around pipeline (based on the figure in the reference, Mao (1987)): (a) Flat seabed; (b) Uneven seabed.

Figure 3-9 The definition of embedment. (a) flat sand bed; (b) scoured sand bed.
Figure 3-10 Screenshot from the camera inside the pipe: (a) Before piping happens; (b) Piping happens.
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Figure 3-11 Change in embedment with time for various experiments reported in Table 3-2.

Figure 3-12 Model validation computational mesh around the pipeline: (a) $e/D = 0.0$; (b) $e/D = 0.1$. 
Chapter 3 Onset of scour below subsea pipelines

Figure 3-13 Pressure coefficient distribution \((Re = 4.5 \times 10^4)\): (a) \(e/D = 0.0\), (b) \(e/D = 0.1\).
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Figure 3-14 pipeline geometries analysis. Free stream embedment is given in brackets for each case: (a) case 1 \( \left( \frac{e}{D} = 0.2(0.2) \right) \); (b) case 2 \( \left( \frac{e}{D} = 0.41(0.2) \right) \); (c) case 3 \( \left( \frac{e}{D} = 0.2(0.2) \right) \); (d) case 4 \( \left( \frac{e}{D} = 0.1(0.2) \right) \).
Figure 3-15 Streamlines around the pipeline, $Re = 4.5 \times 10^4$, and $\delta/D = 0.8$ : (a) $e/D = 0.2$ flat seabed; (b) $e/D = 0.41(0.2)$ uneven seabed; (c) $e/D = 0.2(0.2)$ uneven seabed; (d) $e/D = 0.1(0.2)$ uneven seabed.
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Figure 3-16 Pressure coefficient along the seabed, for four typical scour profiles prior to the onset of scour. \((Re = 4.5 \times 10^4)\).

Figure 3-17 Pressure coefficient along the flat seabed for different pipeline embedment. \((Re = 4.5 \times 10^4)\).

Figure 3-18 Pressure coefficient along the uneven seabed for different pipeline embedment. \((Re = 4.5 \times 10^4)\).
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Figure 3-19 Variance of the pressure drop coefficient with $e/D$ ($Re = 4.5 \times 10^4$). The embedment labelled in the legend is the far field embedment. The X-axis embedment is the local embedment.
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Figure 3-20 Pressure distribution in the water and in the soil, seepage flow streamlines in the soil: (a) N1 ($e/D = 0.2(0.2)$); (b) N2 ($e/D = 0.41(0.2)$); (c) N3 ($e/D = 0.2(0.2)$); (d) N4 ($e/D = 0.1(0.2)$).
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Figure 3-21 Contour of seepage-pressure underneath the pipeline (left column); Seepage-flow streamlines (right column): (a) N1 (e/D = 0.2(0.2)); (b) N3 (e/D = 0.2(0.2)).
4. PIPELINE EQUILIBRIUM SCOUR DEPTH IN CURRENT/WAVES AND COMBINED WAVES AND CURRENT

This Chapter presents the results of an experimental investigation on scour around fixed subsea pipelines placed on a flat sandy bed in current, waves, and combined waves and current conditions using both MOT and LOT facilities together with two model pipelines with diameter of 50 mm and 196 mm. Comparisons of equilibrium scour depth and scour width have been done between the present experimental data and existing empirical formulae and data in all three flow conditions. The empirical formula for equilibrium scour depth in steady current has been updated based on the existing data. A new method of predicting the equilibrium scour depth in combined waves and current is proposed based on the current component of equilibrium scour depth and the wave component of equilibrium scour depth. Moreover, the bed features (e.g. sand ripples) which occur in the model tests are discussed in detail using three dimensional scour profile measurements. These results show that different regimes of bed forms occur, but the average scour hole shape is generally not affected by bed forms. Secondly, the wall boundary effect between the pipeline model and o-tube side walls is discussed and quantified using experimental results and three dimensional numerical simulations. Collectively, the investigations of sand ripples effect and wall effect provide a better understanding of the physical experiments and a quantified view of the laboratory data.

4.1 Introduction

Local scour below offshore pipelines has been the topic of a number of research projects over the past four decades due to its engineering significance. Once the onset of scour happens under a pipeline, the seabed material under the pipeline will be eroded and leads to a growing scour hole under the pipe. The equilibrium scour depth will be reached
corresponding to the fully-developed stage, which is critical for pipeline engineers to evaluate scour potential and self-burial in pipeline design.

The scour depth in different flow conditions has been studied extensively. The first empirical formula for equilibrium scour depth in steady current was established by Kjeldsen et al. (1973), which linked the equilibrium scour depth to the mean flow velocity and pipe diameter. Several years later, Bijker and Leeuwestein (1984) updated the previous empirical formula by adding the influence of grain size. After that, other researchers were also involved in this area (Lucassen, 1984; Mao, 1986; Sumer and Fredsøe, 1990). Sumer and Fredsøe (2002) stated that the mean value of the normalized scour depth was 0.6 with ± 0.2 standard deviation. Although many other physical investigations have been conducted in the last 30 years (Lucassen, 1984; Mao, 1986; Sumer and Fredsøe, 1996), these results have not been combined to provide an improved predictive formula.

Sumer and Fredsøe (1990) indicated that the equilibrium scour depth in waves depends on the $KC$ number, and proposed the relationship using experimental data. Sumer and Fredsøe (1996) worked out a series of empirical expressions to link the equilibrium scour depth in combined waves and current with the scour depth in current alone $S_c$, $KC$ number and flow ratio $m = U_c/(U_c + U_w)$.

The width of the scour hole around the pipeline is a representation of the seabed area affected by local scour. It was demonstrated that the tunnel scour and lee-wake scour play key roles in the scour process around the pipeline. Sumer and Fredsøe (2002) suggested for the steady current alone cases, the width of scour hole at the upstream side of the pipeline is approximately 2 $D$ and 4 $D$ at the downstream side of the pipeline. For the wave alone cases, they indicated that the lee-wake erosion on both sides of the pipeline is dependent on the $KC$ number, and then presented an empirical formula to calculate the scour width in waves as a function of $KC$ number. They also indicated that the scour width in combined waves and current is in the area between current alone case ($m \to 1$) and wave alone case ($m \to 0$).

In this study, the previous findings related to the scour depth and scour width are revisited using the previous data and present physical experimental results with a relatively large model pipeline (196 mm in diameter). Moreover, the sand ripples effect and wall effect are also analysed in this chapter.
4.2 **Experiment setup**

4.2.1 **O-tube**

The O-tubes are fully enclosed circulating water channels with a rectangular test section and a propeller-type pump driven by a motor. Two different scale O-tubes were employed in this study, namely the Large O-tube (LOT) and Mini O-tube (MOT) respectively. A brief description (and sketch figures) of these two facilities can be found in chapter 1, and will not be repeated here. A model pipeline with a diameter of 196 mm was used in the LOT tests, which was rigidly fixed at the centre of the test section and extended across the full width of test section. The sediment utilized in the LOT experiments was silica sand with median grain size is $d_{50} = 0.24$ mm and a geometric standard deviation of $\sigma_g = \sqrt{d_{84}/d_{16}} = 1.37$. The particle size distribution is shown in Figure 4-1 (a). The sediment was filled in the middle of the test section, covering a length of 15.7 m in length and a depth of between 0.3 m and 0.7 m (the deeper section covering about 1 m in length either side of the model pipe).

Supplementary experiments were conducted in the MOT facility at the University of Western Australia, which is approximately 5 times smaller than the LOT. The bottom 0.11 m of the working section was filled with sediment. Two types of sediment were used in the tests, one identical to that used in LOT with $d_{50} = 0.24$ mm and the other with $d_{50} = 0.6$ mm and geometric standard deviation of $\sigma_g = \sqrt{d_{84}/d_{16}} = 1.24$. The particle size distribution of the latter sediment is shown in Figure 4-1 (b). A model pipeline with diameter of $D = 50$ mm was implemented in the tests. The model pipeline was rigidly fixed and the surface of the model pipeline was smooth. In this chapter, the LOT experiments are designated as large-scale pipeline experiments, and the latter experiments are designated as small-scale pipeline experiments.

4.2.2 **Velocity measurements**

A SonTek Acoustic Doppler velocimeter (ADV) was used to measure velocity at a sampling rate of 50 Hz. In the case of current alone, the velocity profile across the depth was measured at 2 m (10 D) upstream of the model pipeline in the LOT and 0.4 m (8 D) upstream of the model pipeline in the MOT. The velocity at each measurement point was obtained by averaging measured data over six minutes. The measured velocity profiles agree well with a logarithmic distribution.
In the cases of wave alone and combined waves and current, the ADV was located centrally 2 m (10 D) upstream of the model pipeline and 0.1 m above the initially flat seabed in the LOT. In the MOT, it was located centrally 0.4 m (8 D) upstream of the model pipeline and 0.025 m above the seabed. The ADV monitored the flow for at least 50 cycles during each experiment, and the period-averaged orbital velocity was estimated using the phase ensemble averaging method (Jensen, 1988)

\[ \bar{u}(\omega t) = \frac{1}{N} \sum_{i=1}^{N} u(\omega(t + (i - 1)T_w)) \]  

where \( N \) is the number of cycles of the oscillating flow, \( T_w \) is the oscillatory flow period, \( \omega \) is the angular frequency of the oscillating flow and \( t \) is the time. In this study, velocity time series were generated in the form of:

\[ U_{wc} = U_w \sin(\omega t + \varphi) + U_c \]  

where \( \varphi \) is phase angle, the oscillatory flow component \( U_w \) and current component \( U_c \) quoted herein have been calculated based on the least square difference between Eq. (4-2) and the period-averaged velocity from Eq. (4-1). Figure 4-2 (a) presents a sample of measured velocity; whilst Figure 4-2 (b) shows the phase ensemble average of 50 wave cycles in the combined wave and current test Owc 7. The \( KC \) number and the flow ratio (defined later) were calculated based on these fitted velocities.

4.2.3 Scour depth and scour profile

To monitor the scour depth in the LOT a 2 mm thin steel probe with marked scales was fixed on the bottom of the model pipeline extending from the base of the sand basin to the bottom of the cylinder surface. The evolution of the scour depth during the scour process could be read directly from outside the glass wall. The probe was fixed at 250 mm away from the glass side wall to achieve a clear view of the scale on the probe and to avoid the wall effect. A way to determine the area affected by the wall is discussed later in this chapter.

The final scour profile of each experiment in the LOT was scanned using a handheld three-dimensional (3D) scanner based on the Microsoft Kinect. The typical scanned images are shown in Figure 4-12, Figure 4-13 and Figure 4-14.

This scanner allows us to obtain metric information about the 3D morphological feature of the bed. The 3D profile is captured by an infrared (IR) emitter and an IR depth sensor equipped in the scanner. The emitter emits infrared light beams and the depth sensor reads
Chapter 4 Equilibrium scour depth in current/waves and combined waves and current

the IR beams reflected back to the sensor. The reflected beams are converted into depth information measuring the distance between an object and the sensor. The resolution of the 3D depth image is up to $640 \times 480$ and the frame rate is up to 30 frames per second. The resolution of the infrared camera is determined by the distance between sensor and seabed. In this study, the maximum distance between the 3D scanner and the seabed is below 1 m. According to Khoshelham and Elberink (2012)’s investigation, the precision of 3D scour profile data is around 2 mm. The 3D handheld scanner was calibrated by know scale (length, width and height) reference object, and then the precision was confirmed.

Scour measurements in the MOT were made using video and the final scour profile was measured using a SICK ranger 3D camera with ± 0.1 mm accuracy. The typical scanned images are shown in Figure 4-16, Figure 4-17 and Figure 4-18. Using laser triangulation, the 3D camera is able to measure the three-dimensional shape of the scanned area, and sends the measurement results to a PC for further processing. Before the tests, the 3D laser camera was calibrated by a known scale reference object.

4.2.4 Test performed

The test procedure in the LOT was typically as follows:

1. Level off the bed, position the model pipeline (noting to install the thin steel scour measurement probe underneath the pipeline before tests) and the ADV, turn on video cameras around the test section;

2. Turn the motor on instantaneously to the maximum designed speed, and then start to record the velocity (generally it took between 2 seconds for the velocity to reach the peak required velocity; which was a short time compared with the time scale of the scour process);

3. Run the test until little change in scour depth is observed within a substantial period of time (typically 30 minutes), which is considered as equilibrium stage. It should be noted that in some tests, fluctuations in the scour depth could still be observed even after a very long period of testing. The fluctuations were attributed to the passage of bed ripples through the observation area. The time dependent scour depth was measured continuously during the experiment by reviewing the recorded video;

4. Stop the test, then scan the final scour profile with the hand held 3D scanner in the LOT.
The test procedure in the MOT was as follows:
1. Level off the bed, position the pipeline and the ADV, turn on camera around the test section;
2. Turn the motor on instantaneously to the maximum designed speed, then start to record the velocity;
3. Stop the tests regularly, remove the model pipe, then scan the seabed profile around pipeline with the 3D camera, after that, reposition the model pipeline to its original position, continue the tests;
4. Continue the test till the scour process reached its equilibrium (similar criterion with LOT tests), and then scans the final scour profile with 3D camera.

4.2.5 Test conditions

The test conditions and results are summarized in Table 4-1, Table 4-2 and Table 4-3.

In Table 4-1, $U_c$ is the velocity measured at the level of the pipeline centre distance above the bed in both LOT and MOT tests. The pipeline Reynolds number in current $Re_c$ is defined as follow:

$$Re_c = \frac{U_c D}{\nu}, \quad (4-3)$$

here, the viscosity of water is taken to be $1 \times 10^{-6} \text{ m}^2/\text{s}$ throughout. The equilibrium scour depth in current is obtained from the least square fit with Eq. (5-7) in Chapter 5, by measuring the scour depth underneath the centre of pipe, as shown in Table 4-1. The similar way is also applied for scour depth in waves and combined waves and current. The equilibrium scour width in steady current for upstream side of the pipeline $W_{c1}$ and downstream side of the pipeline $W_{c2}$ are defined as shown in Figure 4-4 (a).

In Table 4-2, the $KC$ number is defined as

$$KC = \frac{U_w T_w}{D} \quad (4-4)$$

where $U_w$ is the maximum orbital velocity (referenced at the level of the pipeline centre above the bed), and the pipeline Reynolds number in waves is defined as

$$Re_w = \frac{U_w D}{\nu}, \quad (4-5)$$

Due to the symmetry scour profile in waves, the scour width in waves $W_w$ is defined as shown in Figure 4-4 (b).

In Table 4-3, the flow ratio $m$ is defined as
where, $U_c$ is the current velocity (referenced at one pipe diameter distance above the bed).

The equilibrium scour width in combined waves and current for upstream side of the pipeline $W_{wc1}$ and downstream side of the pipeline $W_{wc2}$ are defined as shown in Figure 4-4 (a).

Lucassen (1984) carried out a series of tests in combined waves and current with $3 < KC < 15$, $0.45 < m < 0.7$ and $50 \text{ mm} \leq D \leq 180 \text{ mm}$. In the combined waves and current tests of Sumer and Fredsøe (1996), the test range is $6 < KC < 52$, $0.25 < m < 0.62$ and $15 \text{ mm} \leq D \leq 70 \text{ mm}$. The tests with relatively large model pipe diameter $D > 70$ is insufficiency for relatively large $KC$ number ($KC > 15$). Therefore, in this study total 23 experiments which conducted, spreading relatively uniformly across the $KC - m$ parameter space and covering a range of Shields parameter. For convenience, both $KC$ and $m$ are plotted for each experiment in Figure 4-3 together with the parameter values for existing published experimental data (Lucassen, 1984; Sumer and Fredsøe, 1996).
## Table 4-1 Tests and test results under current only conditions.

<table>
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<tr>
<th>Test No.</th>
<th>Pipe Diameter</th>
<th>Sand size</th>
<th>Undisturbed velocity near the bed</th>
<th>Pipe Reynolds number</th>
<th>Curve fit dimensionless equilibrium scour depth</th>
<th>Measured dimensionless equilibrium scour width</th>
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</thead>
<tbody>
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<td>Oc1</td>
<td>196</td>
<td>0.24</td>
<td>0.53</td>
<td>1.0 × 10^5</td>
<td>0.60</td>
<td>1.8</td>
</tr>
<tr>
<td>Oc2</td>
<td>196</td>
<td>0.24</td>
<td>0.97</td>
<td>1.9 × 10^5</td>
<td>0.85</td>
<td>2.6</td>
</tr>
<tr>
<td>Oc3</td>
<td>50</td>
<td>0.6</td>
<td>0.59</td>
<td>2.9 × 10^4</td>
<td>0.80</td>
<td>1.8</td>
</tr>
<tr>
<td>Oc4</td>
<td>50</td>
<td>0.24</td>
<td>0.59</td>
<td>3.0 × 10^4</td>
<td>0.76</td>
<td>1.6</td>
</tr>
<tr>
<td>Oc5</td>
<td>150</td>
<td>0.24</td>
<td>0.65</td>
<td>9.7 × 10^4</td>
<td>0.51</td>
<td>1.5</td>
</tr>
</tbody>
</table>

## Table 4-2 Scour in waves.

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Pipe Diameter</th>
<th>Sand size</th>
<th>Time period</th>
<th>Maximum orbital velocity near the bed</th>
<th>Keulegan carpenter number</th>
<th>Pipe Reynolds number</th>
<th>Curve fit dimensionless equilibrium scour depth</th>
<th>Measured dimensionless equilibrium scour width</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ow1</td>
<td>196</td>
<td>0.24</td>
<td>8.5</td>
<td>0.72</td>
<td>31</td>
<td>1.4 × 10^5</td>
<td>0.63</td>
<td>3.5</td>
</tr>
<tr>
<td>Ow2</td>
<td>196</td>
<td>0.24</td>
<td>12.9</td>
<td>0.48</td>
<td>31</td>
<td>9.3 × 10^4</td>
<td>0.43</td>
<td>1.2</td>
</tr>
<tr>
<td>Ow3</td>
<td>196</td>
<td>0.24</td>
<td>21.4</td>
<td>0.53</td>
<td>57</td>
<td>1.0 × 10^5</td>
<td>0.63</td>
<td>1.7</td>
</tr>
<tr>
<td>Ow4</td>
<td>196</td>
<td>0.24</td>
<td>15.1</td>
<td>0.79</td>
<td>61</td>
<td>1.5 × 10^5</td>
<td>0.98</td>
<td>5.7</td>
</tr>
<tr>
<td>Ow5</td>
<td>50</td>
<td>0.6</td>
<td>3.5</td>
<td>0.61</td>
<td>43</td>
<td>3.1 × 10^4</td>
<td>0.64</td>
<td>3.3</td>
</tr>
<tr>
<td>Ow6</td>
<td>50</td>
<td>0.24</td>
<td>3.5</td>
<td>0.39</td>
<td>27</td>
<td>2.0 × 10^4</td>
<td>0.54</td>
<td>2.4</td>
</tr>
<tr>
<td>Ow7</td>
<td>50</td>
<td>0.24</td>
<td>3.5</td>
<td>0.63</td>
<td>44</td>
<td>3.2 × 10^4</td>
<td>1.12</td>
<td>4.6</td>
</tr>
</tbody>
</table>
### Chapter 4 Equilibrium scour depth in current/waves and combined waves and current

Table 4-3 Scour in combined waves and current.

<table>
<thead>
<tr>
<th>Test</th>
<th>Pipe Diameter</th>
<th>Sand size</th>
<th>Time period</th>
<th>Max near bed oscillatory velocity</th>
<th>Near bed current velocity</th>
<th>Kunlegan-Carpenter number</th>
<th>Pipe Reynolds number based on $U_c$</th>
<th>Pipe Reynolds number base on $U_m$</th>
<th>Curve fit scour depth</th>
<th>Measured equilibrium scour width</th>
</tr>
</thead>
<tbody>
<tr>
<td>Owc1</td>
<td>196</td>
<td>0.24</td>
<td>8.4</td>
<td>0.34</td>
<td>0.09</td>
<td>15</td>
<td>0.20</td>
<td>$1.7 \times 10^4$</td>
<td>6.7 $\times 10^4$</td>
<td>0.25</td>
</tr>
<tr>
<td>Owc2</td>
<td>196</td>
<td>0.24</td>
<td>8.7</td>
<td>0.34</td>
<td>0.24</td>
<td>15</td>
<td>0.42</td>
<td>$4.8 \times 10^4$</td>
<td>6.6 $\times 10^4$</td>
<td>0.47</td>
</tr>
<tr>
<td>Owc3</td>
<td>196</td>
<td>0.24</td>
<td>8.6</td>
<td>0.36</td>
<td>0.97</td>
<td>16</td>
<td>0.73</td>
<td>$1.9 \times 10^5$</td>
<td>7.1 $\times 10^4$</td>
<td>0.90</td>
</tr>
<tr>
<td>Owc4</td>
<td>196</td>
<td>0.24</td>
<td>10.7</td>
<td>0.51</td>
<td>0.13</td>
<td>28</td>
<td>0.20</td>
<td>$2.5 \times 10^5$</td>
<td>$1.0 \times 10^5$</td>
<td>0.46</td>
</tr>
<tr>
<td>Owc5</td>
<td>196</td>
<td>0.24</td>
<td>10.7</td>
<td>0.53</td>
<td>0.47</td>
<td>29</td>
<td>0.47</td>
<td>$9.2 \times 10^4$</td>
<td>$1.0 \times 10^5$</td>
<td>0.85</td>
</tr>
<tr>
<td>Owc6</td>
<td>196</td>
<td>0.24</td>
<td>12.8</td>
<td>0.52</td>
<td>1.10</td>
<td>34</td>
<td>0.68</td>
<td>$2.1 \times 10^5$</td>
<td>$1.0 \times 10^5$</td>
<td>0.79</td>
</tr>
<tr>
<td>Owc7</td>
<td>196</td>
<td>0.24</td>
<td>11.7</td>
<td>0.77</td>
<td>0.17</td>
<td>46</td>
<td>0.18</td>
<td>$3.3 \times 10^4$</td>
<td>$1.5 \times 10^5$</td>
<td>0.69</td>
</tr>
<tr>
<td>Owc8</td>
<td>196</td>
<td>0.24</td>
<td>11.4</td>
<td>0.80</td>
<td>0.69</td>
<td>46</td>
<td>0.46</td>
<td>$1.4 \times 10^5$</td>
<td>$1.6 \times 10^5$</td>
<td>0.90</td>
</tr>
<tr>
<td>Owc9</td>
<td>196</td>
<td>0.24</td>
<td>29.4</td>
<td>0.30</td>
<td>0.97</td>
<td>45</td>
<td>0.76</td>
<td>$1.9 \times 10^5$</td>
<td>$5.8 \times 10^4$</td>
<td>0.77</td>
</tr>
<tr>
<td>Owc10</td>
<td>50</td>
<td>0.6</td>
<td>3.5</td>
<td>0.22</td>
<td>0.39</td>
<td>16</td>
<td>0.64</td>
<td>$2.0 \times 10^4$</td>
<td>$1.1 \times 10^3$</td>
<td>0.76</td>
</tr>
<tr>
<td>Owc11</td>
<td>50</td>
<td>0.24</td>
<td>3.5</td>
<td>0.35</td>
<td>0.32</td>
<td>24</td>
<td>0.48</td>
<td>$1.6 \times 10^4$</td>
<td>$1.7 \times 10^4$</td>
<td>0.68</td>
</tr>
</tbody>
</table>
Chapter 4 Equilibrium scour depth in current/wave and combined wave and current

4.3 **Test results and comments**

4.3.1 **Equilibrium scour depth**

4.3.1.1 **Equilibrium scour depth in steady current**

For the steady current cases, the equilibrium scour depth obtained from the least square fit in each of the experiments, normalized by the pipe diameter, is shown in Table 4-1. The dimensionless scour depth is plotted against the pipeline Reynolds number in Figure 4-5 (originally plotted by Sumer and Fredsøe (1990)). Across all the data, there appears to be a weak decreasing trend of scour depth with the increase in Reynolds number. This may be attributed to either scaling effects or experimental errors related to insufficient testing time for equilibrium scour at larger Reynolds number. No further attempts were made in the present study to resolve this issue.

Kjeldsen et al. (1973) established an empirical relation between the equilibrium scour depth, $S_e$, the pipe diameter, $D$, and the mean flow velocity, $V$

$$S_e = 0.972 \left( \frac{V^2}{2g} \right)^{0.2} D^{0.8} = 0.972 \left( \frac{V^2}{2gD} \right)^{0.2} D$$  (4-7)

Bijker and Leeuwestein (1984) stated that the scour depth depends on the depth-averaged flow velocity, pipe diameter, flow depth, height of the pipeline above bed level and grain size. Using results from a series of model tests, including those conducted by (Kjeldsen et al., 1973), they proposed a slightly different empirical equation for computing scour depth below submarine pipelines:

$$S_e = 0.929 \left( \frac{V^2}{2g} \right)^{0.26} D^{0.78} d_{50}^{-0.04} = 0.929 \left( \frac{V^2}{2gD} \right)^{0.26} D \left( \frac{D}{d_{50}} \right)^{0.04}$$  (4-8)

The two empirical formulae are drawn in Figure 4-6 as several curves in terms of $S_e/D$ against $V^2/2gD$. The results of present current tests are also plotted in the figure. It is shown that general agreement is achieved, however, the present tests results are slightly below the empirical formulae curves. This may be attributed to the velocity value adopted for the present tests being the level at the pipeline centre above the seabed, which is generally lower than the mean flow velocity.
Table 4-4 Summary of previous investigations.

<table>
<thead>
<tr>
<th>Name</th>
<th>Water depth</th>
<th>Pipe Diameter</th>
<th>Sand size</th>
<th>velocity</th>
<th>Pipe Reynolds number</th>
<th>Froude numbers</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>(m)</td>
<td>(mm)</td>
<td>(mm)</td>
<td>(m/s)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Kjeldsen et al.</td>
<td>0.43</td>
<td>1.43</td>
<td>60</td>
<td>0.074</td>
<td>0.2-0.52</td>
<td>1.0×10⁴</td>
</tr>
<tr>
<td>(1974)</td>
<td></td>
<td>110</td>
<td>0.2</td>
<td></td>
<td>2.1×10⁵</td>
<td>0.1-0.57</td>
</tr>
<tr>
<td></td>
<td></td>
<td>225</td>
<td>0.2</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>500</td>
<td>0.2</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Lucassen (1984)</td>
<td>0.25</td>
<td>0.66</td>
<td>25</td>
<td>0.1</td>
<td>0.2-0.8</td>
<td>1.9×10³</td>
</tr>
<tr>
<td></td>
<td>0.66</td>
<td>40</td>
<td>0.1</td>
<td></td>
<td>1.1×10⁵</td>
<td>0.15-0.56</td>
</tr>
<tr>
<td></td>
<td></td>
<td>50</td>
<td>0.2</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>63</td>
<td>0.2</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>70</td>
<td>0.2</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Mao (1986)</td>
<td>0.25</td>
<td>0.35</td>
<td>50</td>
<td>0.36</td>
<td>0.35-0.989</td>
<td>2.8×10⁴</td>
</tr>
<tr>
<td></td>
<td>0.25</td>
<td>100</td>
<td>0.36</td>
<td>0.35</td>
<td>4.7×10⁴</td>
<td>0.34-1.2</td>
</tr>
<tr>
<td></td>
<td>0.35</td>
<td>100</td>
<td>0.36</td>
<td>0.989</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sumer and Fredsøe (1996)</td>
<td>0.39</td>
<td>15</td>
<td>0.195</td>
<td>0.255</td>
<td>3.9×10³</td>
<td>0.31-0.68</td>
</tr>
<tr>
<td></td>
<td>0.39</td>
<td>70</td>
<td>0.37</td>
<td>0.37</td>
<td>2.6×10⁴</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

The above two empirical formulae have been proposed for more than 30 years. In the last three decades, scour depth in steady current has been investigated extensively (Lucassen, 1984; Mao, 1986; Sumer and Fredsøe, 1996). More tests with different pipe diameter, sediment and flow conditions have been presented by now. Some physical experiments about the scour around a fixed pipeline placed on the flat sandy bed are listed in Table 4-4. Using the results of model tests in Table 4-4 as well as the present steady current tests, the previous empirical formulae can be corrected. At first, the current flow velocity in the different references are uniformed to the undisturbed current flow velocity at the level of the pipeline centre based on the original information presented in the references. Then, the relationship between the scour Euler number $IE_S = U_c/\sqrt{2gS}$ and the pipeline Froude number $IF_D = U_c/\sqrt{gD}$ is found based on the following least-square fit with the available data

$$IE_S = 0.7515IF_D^{0.7513}$$ (4-9)

Figure 4-7 illustrates the curve of Eq.(4-9) with the existing data, and it is shown that the curve of Eq.(4-9) collapses well the experimental data. The equilibrium scour depth in steady current is obtained by solving Eq. (4-9)

$$S_c = 1.052\left(\frac{U_c^2}{2g}\right)^{0.25}D^{0.75}$$ (4-10)
Chapter 4 Equilibrium scour depth in current/wave and combined wave and current

It is found that the updated Eq.(4-10) is very similar with the previous empirical formula Eq.(4-7), except the mean flow velocity has been replaced with the undistributed current velocity at the level of centre of pipeline which is more direct related to the scour around subsea pipelines. This empirical formula is valid for pipeline Reynolds number in the range of

$$1.9 \times 10^3 < Re < 2.1 \times 10^5$$  \hspace{1cm} (4-11)

and pipeline Froude number in the range of

$$0.1 < IF_D < 1.2$$  \hspace{1cm} (4-12)

The effect of grain size on the equilibrium scour depth is also examined in this study, the results show that the grain size has very little influence on the scour depth with the order of $$d_{50}^{0.0158}$$, therefore, it is ignored in this study.

4.3.1.2 Equilibrium scour depth in waves

In contrast to the steady current results, the fitted equilibrium scour depths obtained from the present LOT/MOT experiments in wave only conditions, together with existing data extracted from Sumer and Fredsøe (1990), have a clearer trend as shown in Figure 4-8. Using the previous data, the equilibrium scour depth and the $$KC$$ number has a strong relationship in waves, which reads as:

$$\frac{S_w}{D} = 0.1\sqrt{KC}$$  \hspace{1cm} (4-13)

here $$S_w$$ is the equilibrium scour depth in waves. This equation was suggested by Sumer and Fredsøe (1990) and was found to be applicable from $$KC$$ numbers around 2 to 1000. Figure 4-8 indicates that the present results agree well with the existing data, indicating that the present experimental setup used in the LOT/MOT provides consistent results compared with the earlier data. However, it should be noted that the size of the data points in Figure 4-8 are proportional to the model pipe diameter used in the experimental testing. At $$KC$$ numbers greater than ~ 40, the present results are obtained with a pipeline at least one order of magnitude larger in diameter than earlier results. This provides improved confidence in using the empirical relationship Eq. (4-13) for predicting scour of pipelines with larger diameters that are more representative of field conditions.

4.3.1.3 Equilibrium scour depth in combined waves and current

In the case of combined waves and current, the equilibrium scour depths normalized by the pipe diameter in the present study are listed in Table 4-3 and plotted with existing experimental data presented by Sumer and Fredsøe (1996) in Figure 4-9. Generally, the
present experimental data trends consistently with the existing data. For instance, the scour depth increases with flow ratio \( m = U_c / (U_c + U_w) \) when the KC number is fixed. It also increases with KC number when the current number is fixed.

Sumer and Fredsøe (1996) proposed a series of empirical formulae to predicted the equilibrium scour depth in combined waves and current flow conditions as a function of scour depth in the current alone \( (S_c) \), KC and flow ratio \( m \). To provide a more objective comparison, Figure 4-10 (a) compares predictive formulae due to Sumer and Fredsøe (1996) with previous data and the present LOT/MOT experimental results. It can be seen that the predicted method generally underpredicts the experiments for deep scour depths (>0.06 m). To evaluate the goodness of fit for the predicted formulae, \( R^2 \) is calculated and the results is 0.7539.

The relation between the equilibrium scour depth around pipeline and its dependent parameters can be written

\[
S_c = f \left[ \text{flow}(U_c, U_w, T_w, \rho, v, g) \right] \text{Seabed sediment} \left( d_{s0}, \sigma_g, \rho_s \right) \text{Pipe Geometry}(D), \quad (4-14)
\]

Under combined waves and current flow conditions, there is a non-linear interaction between the wave and current boundary layers, which will affect the equilibrium scour depth underneath the pipeline. Based on previous experimental data and the present experimental results, the following simple non-linear equation for equilibrium scour depth in combined waves and current \( S_{wc} \) is considered in this study

\[
S_{wc} = (S_c^2 + S_w^2)^{0.5} \quad (4-15)
\]

Here \( S_c \) represents the equilibrium scour depth corresponding to the current component at the level of pipeline centre in the combined waves and current flow conditions, which can be obtained by Eq. (4-10); meanwhile, \( S_w \) is the equilibrium scour depth corresponding to the wave component, which can be calculated using Eq.(4-13). The complete form of Eq. (4-15) can be written as follows:

\[
S_{wc} = \left( \frac{1.107U_c}{\sqrt{2g}} D^{1.5} + 0.01U_wT_wD \right)^{0.5} \quad (4-16)
\]

Here \( U_c \) is the current velocity at the level of the pipeline centre, \( U_w \) is the maximum wave velocity at the level of the pipeline centre, and \( T_w \). From Eq. (4-16), it is seen that the equilibrium scour depth in combined waves and current is dependent on the current velocity component \( U_c \), wave velocity component \( U_w \), wave time period \( T_w \) and the pipe diameter \( D \). Due to the nonlinear interaction, the flow ratio \( m \) is reflected by the
combination of current velocity component and wave velocity component. Figure 4-10 (b) compares the predicted results based on Eq.(4-16) and the measured data. It is shown that the present method gives generally good agreement with the experimental data for the relatively deep scour depth \(( > 0.05 \text{ m})\), and reasonable agreement for sallow scour depths \(( < 0.05 \text{ m})\). The \(R^2\) for present predicted method is 0.8344.

4.3.2 Equilibrium width of scour hole

For steady current, the scour profile in the vicinity of pipeline generally formed a steep slope at the upstream side of the pipeline which is governed by the potential flow, and a more gentle slope at the downstream side of the pipeline which is controlled by vortex shedding due to flow separation. Sumer and Fredsøe (2002) suggested the width of scour hole at the upstream side of the pipeline is \(2D\) and \(4D\) at the downstream of the pipeline in steady current, which is roughly agreement with present data.

For the wave alone case, Jensen et al. (1989) indicated that the length of the lee-wake increases linearly with \(KC\) number. The width of the scour hole in waves is determined by the strong part of the lee-wake causing erosion to occur on both sides of the pipe. Therefore, Sumer and Fredsøe (2002) presented the following empirical formula to calculate the width of the scour hole in waves

\[
\frac{W_w}{D} = 0.35KC^{0.65}. \tag{4-17}
\]

Here \(W_w\) is the width measured from the center of the pipeline to the end of the scour hole (see Figure 4-4 (b)). The existing data as well as the present tests results are drawn in Figure 4-11. Although the above empirical formula is based on only three data points (data with empty square symbol), Eq.(4-17) provides reasonable agreement with the data.

For the four special designed cases drawn in Figure 4-11, the wave shields parameter is considered, which reveals the lee wake strength for some extent. It is clearly shown that with similar \(KC\) number, the larger wave Shields parameter gives larger scour width. Meanwhile, with similar wave Shields parameter, the larger \(KC\) number leads to larger scour width. It is indicated that the scour width is not only affected by the length of the lee-wake, but also the strength of the lee-wake.

Figure 4-12 illustrates the variation of the width of scour hole with respect to the flow ratio \(m\), in which \(W_{wc1}\) was the width measured from the centre of the pipeline to the upstream side end of the scour hole, and \(W_{wc2}\) is the width measured from the centre of the pipeline to the downstream side end of the scour hole (see Figure 4-5 (a)). The data
in Figure 4-12 (a) and Figure 4-12 (b) indicate that the scour width in combined wave and current also generally depends on the values of $K_C$ and $U_{wc}$. For $m < 0.4$, the $W_{wc1}$ and $W_{wc2}$ increase with the increasing $K_C$ number under similar $m$. Furthermore, it appears that the scour widths are close to constant values for $m$ larger than 0.7, namely, $W_{wc1}/D \rightarrow 2$ and $W_{wc2}/D \rightarrow 5$. The above results are consistent with the previous findings (Sumer and Fredsøe, 1996). In Figure 4-12 (c) it is shown that a similar tendency can be found in $W_{wc1} + W_{wc2}$, while the ratio of $W_{wc2} / W_{wc1}$ generally increases with the growth of flow ratio $m$, which is shown in Figure 4-12 (d).

### 4.3.3 Sand ripple effect

When the flow conditions exceeds the threshold of motion ($\theta > \theta_{cr}$), sediment transport will occur over the entire bed and various types of bed features will form ranging in size from small ripples up to major sandbanks (Soulsby, 1997). These bed features will have an influence on the frictional characteristics near seabed and they can migrate close to the pipeline. In the MOT/LOT experiments, all tests were performed in live bed scour conditions ($\theta > \theta_{cr}$), and although the ripples are expected to wash out when the shields parameter is larger than 0.8–1 (sheet flow) ripples were present in most of the present laboratory experiments. Given that, the ripple size to model diameter is larger than in the prototype, the effect of sand ripples was evaluated and discussed in this section.

The video record obtained during the scour process and the three dimensional scour profile obtained in the LOT/MOT tests are used to investigate sand ripple formed during the tests. Firstly, when the flow conditions was relatively weak, it was observed that sand ripples with relatively small, irregular and strongly three-dimensional patterns. These features can be observed from the final 3D scan profile data in some of LOT tests, such as steady current test Oc1 in Figure 4-13 (a), wave test Ow2 and Ow3 in Figure 4-14 (b) (c) and combined waves and current tests Owc1 and Owc2 in Figure 4-15 (a) (b) (c). From the observation of the scour process, it is found that such bed scour patterns were only observed after a relatively long period of time. At the initial stage of the test, the sand ripples on the seabed formed a 2D pattern parallel to the cross section. As the time elapsed, these 2D sand ripples gradually transformed to the 3D irregular sand ripples. Typically, in the LOT tests, the height of the crest to trough of such sand ripples was around 50 mm, and the wavelength was approximately three times longer than the height. No significant evidence of sand ripples observed at the edges of the pipeline scour hole on both sides of the pipe. This may be because of the enhanced flow to the present of pipe.
When the far field Shields parameter was smaller than 0.2 in the present tests, the sand ripple was not completely washed out but become unremarkable. In general, because of the small size of the ripple itself, it cannot exert much influence on the scour around the pipeline.

Secondly, when the flow intensity became stronger, instead of small 3D ripples, 2D regular sand waves emerged with much bigger height and wavelength. In the current alone cases and current dominated combined waves and current cases, the sand ripple with the form of progressive wave slowly moved towards the current flow direction. Normally it took 1 to 2 hours depending on the flow conditions for the sand wave to arrive at the vicinity of the model pipe. For many of the LOT tests, the equilibrium scour depth was reached in one hour, therefore, in these tests the sand wave did not get chance to affect the area around the pipe, since the equilibrium stage was already achieved. Figure 4-15 (f) shows the nose of sand wave just arriving at the vicinity of the pipeline (4 D away from the pipeline centre at the upstream side).

For the MOT tests, a relatively large flow conditions was adopted, therefore no 3D small sand ripples were observed during the tests, however, the sand wave with maximum 20 mm height merged during some of tests. Figure 4-16 and Figure 4-18 illustrates the regularly scanned scour profile during the scour process for steady current case Oc4 and combined waves and current case Owc11 in the MOT tests. The forepart of such progressive sand waves can be observed from $t = 15$ mins in Figure 4-16 (g) and $t = 3$ mins in Figure 4-18 (e) respectively, and then continually pass through the model pipe. During the period of the sand wave passing underneath the pipe, the seabed surface underneath the pipeline remained smooth and the sand wave cannot be observed. Meanwhile, the shape of scour profile remained similar in different time after the scour hole formed, and sand waves only led to fluctuations in scour depth underneath the pipe. The possible reason was that the amplification of velocity and shear stress around the pipeline was strong enough to wash out the sand wave and other small ripples close to the pipeline and keep the shape of the scour hole, but not strong enough to maintain the scour depth due to the large amount of sediment. That was transported to the vicinity of pipeline in the form of sand wave. Therefore, the equilibrium scour depth underneath the pipeline fluctuated randomly around a mean value (see the measured scour depth in Figure 4-19), which was also a key difference character between the clear water scour and live-bed scour. In this study, the equilibrium scour depth was acquired from a curve-fitting method of the scour depth time series data, rather than the final scour depth (see Figure 4-19).
In the strong wave alone cases and wave dominated combined wave and current cases, the sand ripples were symmetrical about the crest in cross-section with relatively sharp crest (see Figure 4-14 and Figure 4-17). Because the flow for wave tests in the O-tube is oscillatory flow, little sediment in the far field can be transported to the vicinity of the pipe. Therefore, the sand ripples or sand waves emerged in the far field were not considered in this study. In the vicinity of the pipe, Sumer and Fredsøe (1990) demonstrated that the effect of sand ripples was not essential in the scour process under waves. They compared their equilibrium scour depth results in waves with Lucassen (1984)’s results, and found that for different pipe diameters, the relation between scour depth and KC-number are the same. Due to the rate of ripples dimensions to the pipe diameter were changed considerably in their comparison tests, if the sand ripples have effect on the scour process, the constant relation between the scour depth and KC number would not be found. The present wave tests results have good agreement with the previous data (see Figure 4-8), in this way, the effect of sand ripples in wave can also be ignored.

4.3.4 Wall effect

In the experiments, the flow field is complex at the ends of the model pipe, adjacent to side wall of the flume and the seabed. It was observed in some experiments that the scour depth closed to the side walls of the O-tube working section was larger than the scour depth in the middle of the pipeline. This increased scour depth towards the ends of the pipeline is considered herein to be a wall effect. Although the rate of model pipeline length to pipe diameter is about 5 in the LOT, to get a clear view of the scour depth a needle was placed underneath the pipe close to the glass wall. To make sure that the scale needle is placed outside the area of wall effect is important in this study to obtain accurate test results.

When a vertical circular pile is placed on a plane wall, a horseshoe vortex will be formed in front of the pile (Sumer and Fredsøe, 2002). If another plane wall is added into the area of the horseshoe vortex and placed perpendicular to the previous wall, the horseshoe vortex will be interrupted by the second plane bed, and then transformed to a half horseshoe vortex (Sumer et al., 2001), which can enhance the scour effect.

From the 3D scan profile images (see Figure 4-13 to Figure 4-18) it is observed that the scour profile around the pipeline is enlarged closed to vertical glass wall. For steady current cases, the approximate area with deeper scour is about 150 mm from the wall for the 196 mm diameter model pipeline in the LOT and 30 mm from the wall for the 50 mm
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diameter model pipeline in the MOT respectively, meanwhile the rest scour profile along the pipeline was relatively uniform.

For waves cases and combined waves and current cases, the area with deeper scour is smaller than for steady current, and this may be explained by the fact that the boundary layer thickness is thinner than for steady current cases.

To better understand the flow field around the pipe, wall and seabed, several numerical simulations have been conducted under steady current flow conditions. The Reynolds-Averaged Navier-Stokes (RANS) equation and Shear-Stress Transport (SST) k-ω turbulence model were adopted in this study, in a similar manner to the flow-field simulations reported in Chapter 3. The detail of the numerical model is similar with previous numerical study but in 3D manner, and will not be listed here.

4.3.4.1 numerical model setup

In the LOT tests, model pipeline length (1 m) to pipe diameter (196 mm) is about 5. To simulate the situation more close to the experimental setup and optimize the mesh node number, a half symmetry computational domain 120 D in length, 5 D in height and 2.5 D in width is discretized by eight-node hexahedral elements. The boundary conditions are illustrated in Figure 4-20. Four different gap ratio between pipeline and plane wall \( G/D \) were simulated in this study, which were 0.0, 0.1, 0.4, 0.72 respectively and the pipe Reynolds number is \( 4.5 \times 10^4 \). Three of these gap ratios \( G/D = 0.0, 0.1 \) and 0.4 coincided with experiments performed by Bearman and Zdravkovich (1978) with a no-slip boundary conditions imposed on one side wall. Hence validation of the numerical model as well as the investigation of the wall effect is possible. The pressure coefficient around the cylinder and along the seabed in the 3D numerical simulation is compared with the experimental results for different \( G/D \), good agreement was generally observed on the pipeline far away from the vertical wall.

One case \( e/D = 0.72 \) with a scoured seabed was also simulated numerically to represent the situation when the equilibrium scour depth was reached The scour profile for this case was a digitized reproduction from Jensen et al. (1990) which is a well-known steady current test with solid scoured seabed profile. The two-dimensional mesh layout in the XY-plane for all cases is shown in Figure 4-21.

To obtain the similar boundary layer thickness at the pipeline position with Bearman and Zdravkovich (1978), an empty flow field without a pipeline was simulated initially, from which it was concluded that the seabed boundary layer thickness developed to 0.8
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D at the length of 60 D from the upstream boundary (where the pipeline was to be placed). For Case 1 ($G/D=0.0$), in order to maintain a good quality of computational mesh beneath the pipeline, the pipeline is deliberately placed above the plane boundary (seabed) by 0.01 D in the simulation. However, the gap between the pipeline and seabed is blocked computationally under the pipeline.

The total number of nodal points in the computational meshes were 772800 for $e/D=0.0$, 779200 for $e/D=0.1$, 974000 for $e/D=0.4$ and 807200 for $e/D=0.72$. The convergence of the numerical results with respect to the mesh density is also tested for each case, and it was found that the numerical results changed little if the mesh density was increased further. For the four cases, a high density of nodal points were distributed around the solid boundaries to ensure the accuracy of the solution. The pipeline perimeter was discretized by 160 nodal points and the minimum element size in the radial direction next to the pipeline surface was 0.0001 D, while for the Z-direction, the number was 0.0002 D. The maximum dimensionless mesh size next to the solid surface $y^+$ (defined by $y^+=u^*_c\Delta/u$, with $u^*_c$ being the friction velocity and $\Delta$ the dimensional mesh size) was less than 2.

4.3.4.2 Numerical results

The wall shear stress is directly related to the bed-load sediment transport. Figure 4-22 shows the wall shear stress distribution on the seabed around the pipeline for each case. For case 1 ($G/D=0.0$), the results show that the wall shear stress increases in front of the pipeline due to the vortex in front of the pipe. For the rest flat seabed case 2 and case 3, it is shown that there is a shear stress increase just at the corner of the wall and seabed underneath the pipeline, which is believed that caused by the wall effect. Moreover, the length of the region with increased shear stress extends about 0.5 D along the pipeline. Beyond that, in the remaining areas along the pipe, the wall shear stress remained constant. For the uneven seabed case 4, the length of the region with increased shear stress on the seabed along the vertical wall is longer than flat seabed cases, but the length along the pipe is still less than 0.5 D.

Figure 4-23 shows the pressure coefficient around the pipelines for each case. The wall effect influences the pressure coefficient at the corner of the pipeline and wall in all the cases. However, the maximum influenced region along the pipeline was no more than 1 D.
To capture the head of the half horse-shoe vortex mentioned in Sumer et al. (2001), a slice was extracted from XY-plane at the center of pipeline for each case and the XZ-velocity streamtraces are drawn in Figure 4-24. In all the cases, the vortex was around 1 D to 2 D away from the pipeline at the upstream side of the pipeline. The affected area was limited to 1 D in length along the pipe, and the flow remained uniformed along the rest part of pipeline.

All in all, from the simple numerical simulation investigation, the wall effect was confirmed to exist, and moreover, the affect area was below 1 D, which is consistent with the observations of the physical experiments. This analysis increases the confidence about the accuracy of the scour depth data from the marked probe underneath the pipeline which was placed greater than 1 D from the edge of the pipeline.

4.4 Conclusions

1. The previous empirical equilibrium scour depth formula in steady current is updated using the past 40 years of available data and present test results. The mean flow velocity in the previous empirical formula is replaced with the undistributed flow velocity at the level of pipeline centre, which is more convenient for use with subsea pipeline.

2. For equilibrium scour depth in waves, the previous empirical formula is shown to be valid for relatively large pipeline 196 mm, when the KC number is around 30 to 60.

3. A new method to predict the equilibrium scour depth in combined waves and current is proposed based on the current component equilibrium scour depth and the wave component equilibrium scour depth. Comparisons of the new method with the available data gives good agreement.

4. The equilibrium scour width measured in this study collapses quite well with the previous empirical results. Using the present experimental data, development of an empirical formula for scour width under combined wave and current conditions may be possible in the future.

5. Two types of bed features have been investigated in the present experiments: (i) small 3D ripples in the free field and within the scour hole; (ii) 2D plane sand waves migrating through the scour hole. The presence of bed features was found to have limited influence on the average scour hole depth and width underneath the pipeline.
6. The wall effect in the laboratory test has been investigated using final scour profile from the experiments and numerical simulations. The area of seabed with elevated shear stress due to the presence of wall is shown to be confirm to a maximum length along the pipeline of 1 D.

Reference


Figure 4-1 Particle size distribution of the model sand in the Large O-tube: (a) sediment in both LOT and MOT ($d_{50} = 0.24$ mm); (b) sediment in the MOT ($d_{50} = 0.6$ mm).
Figure 4-2 Test Owc7 of large pipeline test in combined wave and current: (a) Measured velocity time series; (b) Phase ensemble averaging.
Figure 4-3 Test range.

Figure 4-4 Sketch for scour below pipeline: (a) Steady current/combined waves and current; (b) waves.
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Figure 4-5 Data for equilibrium scour depth in current only. Live bed ($\theta > \theta_{cr}$), based on Sumer and Fredsøe (1990).

Figure 4-6 The present experimental results compared with Eq.(4-7) and Eq.(4-8).
Figure 4-7 Euler-Froude relation.

\[ IF_D = \frac{U_c}{\sqrt{gD}} \]

Figure 4-8 Equilibrium scour depth in waves. Live bed \( (\theta > \theta_{cr}) \), based on Sumer and Fredsøe (1990). Size of symbol scale with the diameter of model pipe.
Figure 4-9 Equilibrium scour depth in combined wave and current, live bed ($\theta > \theta_{cr}$), based on Sumer and Fredsoe (1996). The size of the symbols are proportional to the model pipe diameter used in the experiments.
Figure 4-10 Measured versus calculated scour depths in combined waves and current. (a) through the method of Sumer and Fredsøe (1996); (b) through present procedure with Eq.(4-15).
Figure 4-11 Equilibrium scour width of the scour hole in waves conditions. Live bed \((\theta > \theta_{cr})\)
Figure 4-12 Equilibrium scour width of the scour hole in combined waves and current. Additional data from Sumer and Fredsøe (1996): (a) Equilibrium scour width at the upstream side of the pipe; (b) Equilibrium scour width at the downstream side of the pipe; (c) Equilibrium scour width $W_1 + W_2$; (d) Equilibrium scour width of $W_2/W_1$.

Figure 4-13 LOT current tests: (a) Test Oc1; (b) Test Oc2
Figure 4-14 LOT waves tests: (a) Test Ow1; (b) Test Ow2; (c) Test Ow3; (d) Test Ow4.
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(a) Owc1

(b) Test Owc2 upstream

(c) Test Owc2 downstream

(d) Test Owc6 upstream

(e) Test Owc6 downstream
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(f) Test Owc7 upstream

(g) Test Owc7 downstream

(h) Test Owc8 upstream

(i) Test Owc8 downstream

(j) Test Owc9

Figure 4-15 LOT combined wave and current tests.
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(a) $t = 1$ min
(b) $t = 2$ mins
(c) $t = 3$ mins
(d) $t = 4$ mins
(e) $t = 5$ mins
(f) $t = 10$ mins
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(g) t = 15 mins  (h) t = 20 mins

(i) t = 40 mins  (j) t = 60 mins

Figure 4-16 Current test Oc4 in the MOT.
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(a) $t = 0.5$ min

(b) $t = 1$ min

(c) $t = 2$ mins

(d) $t = 3$ mins

(e) $t = 4$ mins

(f) $t = 5$ mins
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(g) $t = 10$ mins  
(h) $t = 20$ mins

Figure 4-17 wave test Ow7 in the MOT.
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(a) Before the test t = 0 min
(b) t = 0.5 min
(c) t = 1 min
(d) t = 2 mins
(e) t = 3 mins
(f) t = 4 mins
Figure 4-18 Combined waves and current test Owc11 in the MOT.
Figure 4-19 Time development of scour depth. (a) Steady current case Oc3. (b) Combined waves and current case OwC11.

Figure 4-20 3D numerical simulation boundary condition.
Figure 4-21 Computational mesh around the pipeline. (a) $e/D = 0.0$; (b) $e/D = 0.1$; (c) $e/D = 0.4$; (d) $e/D = 0.72$. 

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Figure 4-22 Seabed wall shear stress contours. (a) \( e/D = 0.0 \); (b) \( e/D = 0.1 \); (c) \( e/D = 0.4 \); (d) \( e/D = 0.72 \).
Figure 4-23 Cylinder surface pressure coefficient contours: (a) $e/D=0.0$; (b) $e/D=0.1$; (c) $e/D=0.4$; (d) $e/D=0.72$. 
Figure 4-24 x-z plane velocity streamtraces. (a) $e/D=0.0$; (b) $e/D=0.1$; (c) $e/D=0.4$; (d) $e/D=0.72$. 
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5. TIME SCALE OF LOCAL SCOUR BELOW PIPELINES IN CURRENT / WAVES AND COMBINED WAVES AND CURRENT

In this Chapter the time scale of local scour below pipeline is discussed in current only, waves only and combined waves and current conditions based on the physical model experiments reported in the previous chapter and existing data available in the literature by using a two-step evaluating procedure. An improved empirical formula for predicting the time development of scour in current only, wave only and in combined waves and current conditions is presented and compared with the existing formulae. After comparison with various types of Shields parameter, the non-dimensional time scale of the scour process is found to be mainly dependent on the maximum Shields parameter in all three flow conditions, leading to a universal predictive formula. Collectively, the experimental results concerning time scale in this chapter and equilibrium scour depth in the previous chapter, allow for the first time, for predications of pipeline scour to be made in any combination waves and current conditions directed perpendicular to a pipeline.

5.1 Introduction

Over the last four decades the significance of local scour below offshore pipelines has attracted a large number of investigations. These studies have revealed many features of scour in steady current, waves and in combined waves and current. Due to the complexity of the scouring process, the majority of literature has concentrated on determining the maximum equilibrium scour depth for given flow conditions and sediment. Comparatively fewer researchers have investigated the dependence of this process on time.

Shen (1965) was one of the early researchers to investigate the scour around piers and concluded that the Froude number was the most significant parameter in determining the scour depth. They also provided empirical curves of scour depth as a function of time.
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For a pipeline originally placed on the seabed, the time scale is the transitional time period required for the scour depth to develop towards the fully developed stage. In practice, it is important to understand time histories of scour development. Fredsøe et al. (1992) suggested that the time development of scour beneath a pipeline (leading to the scour hole shown in Figure 5-1) could be approximated by the formula:

\[ S(t) = S_{ef} \left( 1 - \exp \left( -\frac{t}{T_f} \right) \right), \]  

where \( S(t) \) is the time-varying scour depth, \( t \) is time, \( S_{ef} \) is the equilibrium scour depth and \( T_f \) is the characteristic time-scale of the scour process (so that in this case, the scour depth is about 0.63 \( S_{ef} \) after scour has occurred for a duration equal to the time scale).

Using continuity arguments and an assumption that the scour hole scales with pipe diameter, Fredsøe et al. (1992) reasoned that the time scale could be normalized according to

\[ T_f^* = \frac{(g(s - 1)d^3)^{1/2}}{D^2} T_f, \]  

where \( D \) is the diameter of the pipeline, \( s \) is the relative density of the sediment, \( d \) is the mean sediment grains size, and \( g \) is the acceleration due to gravity. Based on an empirical fit to limited experimental data available in the literature at time in both current only and waves only cases. Fredsøe et al. (1992) found that Eq. (5-1) could explain the scour process reasonably well when the non-dimensional time scale across all of the experiments was calculated according to:

\[ T_f^* = \frac{1}{50} \theta^{-5/3}, \]  

where \( \theta \) is the normalized free field shear stress, known as the Shields parameter, and is given by

\[ \theta = \frac{\tau}{\rho g(s - 1)d'} \]  

where \( \rho \) is the density of water and \( \tau \) is the shear stress. In current only conditions \( \theta \equiv \theta_c \) is calculated using the current shear stress \( \tau_c \), and in waves only conditions \( \theta \equiv \theta_w \) which is calculated using the maximum wave shear stress \( \tau_w \) (Fredsøe et al., 1992).

Whitehouse (1998) suggested a more general expression than Eq. (5-1) by adding a fitting coefficient for pipeline as well as pile (see Eq.(5-5)). Briaud et al. (1999) proposed a hyperbola formula to predict the scour depth development with time for cylindrical bridge pier on cohesive sediment (see Eq.(5-12)).
Collectively, the significant amount of research on equilibrium scour depth (in particular the results of Sumer and Fredsøe (1990) and the work cited in Chapter 4), together with the time scale analysis in Fredsøe et al. (1992) based on Eq. (5-1), allow for the evolution of scour to be predicted for fixed pipelines subjected to perpendicular current only and waves only conditions. However, the evolution of scour beneath a pipeline in any combination of perpendicular combined waves and current conditions cannot yet be predicted confidently because of a lack of systematic study concerning the time scale in combined waves and current conditions. Because of this, the engineering model is incomplete to predict the scour history around pipeline for a long time period (Harris et al., 2010).

In the present study, a two-step evaluating procedure is established to investigate the time scale of scour around subsea pipeline. A series of physical experiments have been reported in Chapter 4 are analysed.

In the first step, the previous three forms of scour depth time history expressions proposed by Sumer et al. (1992), Whitehouse (1998) and Briaud et al. (1999) are compared with the experimental results and existing data.

Despite the lack of investigations in combined waves and current flow conditions, Whitehouse (1998) has hypothesised that Eq. (5-3) may be applicable in combined waves and current conditions if $\theta$ is calculated as the combined waves and current shear stress; however questions remain as to the appropriateness of this hypothesis and to whether the mean combined Shields parameter, the maximum combined Shields parameter or a different formulation (such as the linear combination $\theta_w (1 - m) + \theta_c$, which has been recently adopted by Cheng et al. (2014) with $m$ defined as $U_c/(U_c + U_w)$) is most appropriate to use. In the second step, the above three types of Shields parameter are compared using the available experimental data.

### 5.2 Experimental setup

This chapter is based on the physical experiments described in the Chapter 4. The present test conditions and results are shown in Table 5-1 for steady current, Table 5-3 for waves alone and Table 5-5 for combined waves and current flow conditions. The previous test conditions and results are shown in Table 5-2 for tests in steady current conducted by Mao (1986) and Table 5-4 for test in waves conducted by Fredsøe et al. (1992). In these tables, the characteristic parameter such as flow velocity, time period, pipe diameter, grain size, $KC$ number and flow ratio $m$ are consistent with the values in
the previous chapter. The calculation method and definition of equilibrium scour depth and time scale will be illustrated in the following sections.

5.3 Scour depth development with time

The time scale of the scour process may be described in several ways. Apart from Eq.(5-1) proposed by Fredsøe et al. (1992), Whitehouse (1998) proposed a general form based on previous investigations related to the time development of scour at piles and pipelines. This expression was given as:

$$S(t) = S_{eo} \left(1 - \exp \left(- \left(\frac{t}{T_p}\right)^p \right) \right), \quad (5-5)$$

where \(p\) is an empirical coefficient and \(T_p\) is the time scale associated with this exponent. (Note there is an error in the original formulation given in Whitehouse (1998), where \(-t/T_p\) is incorrectly raised to the exponent \(p\).) It was suggested that the value of \(p\) may be independent on types of flows around the structure. The particular value of \(p\) alters the shape of the \(S(t)\) curve. Eq.(5-5) is consistent with Fredsøe et al. (1992) when \(p = 1\).

A further examination of the previous experimental data (Fredsøe et al., 1992; Mao, 1986) and the present data suggest that an improved correlation with the experimental data may be achieved by optimizing the value of \(p\) in Eq.(5-5). A total of 28 experimental results of scour depth development time history incorporating current only, waves only and combined waves and current conditions, (including all LOT and MOT experiments and cases marked with an ‘*’ in for which published time series scour data was also available) have been collected and analysed in this study. The R-square is adopted to evaluate the goodness of fit between the predicted curve and measured data.

The correlation coefficient \(R^2\) measures how successful the fit is in explaining the variation of the data, which is determined by

$$R^2 = 1 - \frac{\sum_{i=1}^{N}(f_i - y_i)^2}{\sum_{i=1}^{N}(y_i - \bar{y})^2}, \quad (5-6)$$

in where, \(f_i\) is the predicted data and \(y_i\) is the measured data, \(\bar{y}\) is the mean of the measured data. R-square can take on any value between 0 and 1, the closer the value is to 1, the better the curve fits data. For a specific value of \(p\) the correlation coefficient, \(R^2\) was calculated for each of the experimental data sets, and then the mean correlation coefficient \(\bar{R}^2\) across all 28 experiments was calculated. This mean value is plotted in Figure 5-2 (b) and shows clearly that the mean R-square is followed a parabola type track,
and the maximum $\bar{R}^2$ is 0.9799 when $p = 0.58$. The Figure 5-2 in other way to prove that the goodness of fit is highly dependent on the value of the fit coefficient $p$. In comparison, $\bar{R}^2$ is 0.9351 when $p = 1$ (equal to that suggested by Fredsøe et al. (1992)). For convenience, R-square is regarded as the default statistical parameter to evaluate the goodness of fit in this study.

Based on the analysis summarised in Figure 5-2, the correlation coefficient indicates that the optimized fit is reached when the fit coefficient $p$ is closed to 0.6. It is therefore apparent that in current only conditions, wave only conditions and combined waves and current, the time development of scour seems to be approximated best according to:

$$S(t) = S_{ep} \left(1 - \exp \left(-\left(\frac{t}{T_p}\right)^{0.6}\right)\right), \quad (5-7)$$

where $S_{ep}$ and, more importantly, the time scale $T_p$ are best fit parameters evaluated when $p = 0.6$. It should be noted that the time scale $T_p$ is distinct from $T_f$ obtained by Fredsøe et al. (1992) which is valid for $p = 1$.

The dimensionless equilibrium scour depth and the time scale obtained by curve fitting based on Eq. (5-1) and Eq. (5-7) for present experiments are listed in Table 5-1, Table 5-3 and Table 5-5 in the form of $S_{ef}, T_f, T_f^*$ and $S_{ep}, T_{ep}, T_{ep}^*$ respectively. For the existing tests results, the scour depth development time history and the curve fitting results are listed as ‘*’ marked cases in Table 5-2 and Table 5-4.

In the previous investigation (Fredsøe et al., 1992), the time scale $T_f$ was predicted either by calculating the slope of the line to the curve of Eq.(5-1) ($p = 1$) at $t = 0$ (see Figure 5-3 (a)) or by integrating $S - S(t)$ over time based on Eq.(5-1) ($p = 1$) (see Figure 5-3 (b)). Comparatively, for the Eq. (5-7), the time scale $T_p$ cannot be predicted by the tangent line to the scour-depth-versus-time curve at $t = 0$, due to the fact that $\left.\frac{dS}{dt}\right|_{t=0} = \infty$. However, the time scale $T_p$ can still be predicted from the depth difference integration, therefore the time scale $T_f$ in the previous investigation predicted from the area is converted by $T_p$ in the present study. Time scale $T_f$ predicted from the area of the scour-depth-versus-time information in the previous study is obtained as follow:

$$\int_0^\infty \left[\frac{S - S(t)}{S}\right] dt = \int_0^\infty e^{-\left(\frac{t}{T_f}\right)} dt \approx T_f, \quad (5-8)$$

whilst, time scale $T_p$ predicted from the area for Eq. (5-7) can be calculated by
\[
\int_0^\infty \frac{[S - S(t)]}{S} \, dt = \int_0^\infty e^{-\frac{t}{T_p}} \, dt \approx 1.5 T_p, \quad (5-9)
\]

Hence, it follows that:

\[
T_p \approx 0.66 T_f. \quad (5-10)
\]

This relationship has been used to convert the previous tests without scour depth development time history, and the results are listed as no marked cases in Table 5-2 and Table 5-4.

In contrast to the exponential form given by Eq.(5-1) and Eq. (5-7), it is noted that Briaud et al. (1999) employed a hyperbolic equation to predict the time development of scour around a vertical pile in cohesive sediments. This equation is given as:

\[
S(t) = \frac{t}{Z_i + \frac{t}{S_{eb}}}, \quad (5-11)
\]

where \(Z_i\) is defined as the initial rate of increase in \(S(t)\) (i.e. \(Z_i = \frac{dS}{dt}_{t=0} = \frac{S_{eb}}{T_b}\), where \(T_b\) is a new time scale) and the equilibrium scour depth \(S_{eb}\) is the ordinate of the asymptote. Therefore Eq. (5-11) can be rewritten as

\[
S(t) = \frac{t}{T_b \frac{S_{eb}}{S_{eb}} + \frac{t}{S_{eb}}} = \frac{S_{eb} t}{T_b + t}, \quad (5-12)
\]

Since the hyperbolic equation works very well in representing the temporal development of scour around piles (Briaud et al., 1999) comparison has also been made with the data analysed for pipelines in this chapter. Since \(\frac{dS}{dt}_{t=0} = \frac{S_{eb}}{T_b}\), the time scale predicted from the slope of the line to \(S(t)\) curve at \(t = 0\) in Fredsøe et al. (1992) can be adopted in the analysis of Eq. (5-12). The time scale \(T_p\) cannot be predicted by integrating \(S_{eb} - S(t)\), due to the fact that \(\int_0^\infty \frac{[S_{eb} - S(t)]}{S_{eb}} \, dt = \infty\).

To better illustrate how well the fitted relationship for these three scour depth development time histories equations match with the experimental data. Figure 5-3 compares both Eq.(5-1), Eq. (5-7) and Eq.(5-12) with the measured experimental data for current only, waves only and combined waves and current conditions, respectively. In these figures dimensionless scour depth (given by \(S/S_{ef}, S/S_{eb}, S/S_{ep}\)) and time (given by \(t/T_f, t/T_b\) and \(t/T_p\)) are used so that all experiments can be shown on a single figure for each equation. In general, it is seen that all three equations follow the trend of increasing scour depth with time. From the view of mean R-square which is given from
Chapter 5 Time scale in current/wave and combined wave and current

averaging the R-square for each case, it is demonstrated that the empirical formula in Eq.(5-7) and Eq.(5-12) provides a consistently better fit than Eq.(5-1).

After the onset of scour occurred underneath the pipe, the scour depth developed rapidly at the very beginning of the scour process. Therefore, it is important to determine the time scale as well as the equilibrium scour depth. To better review the scour depth change at the rapid development stage of scour process, a ‘zoom’ in figure is plotted followed by the overall figure for each equation. For Eq.(5-1) in Figure 5-3 (a-2) the predicted equation underestimates the scour depth at the initial stage of scour and overpredicts the scour depth when the scour rate start to decrease. For Eq.(5-7) in Figure 5-3 (b-2) although the predicted curve overpredicts the scour depth for few cases at the initial scour stage, the experimental results generally collapse fair well with predicted curve. For Eq.(5-12) in Figure 5-3 (c-2), the predicted curve works better than the Eq.(5-1) for most of cases, however, generally overpredicts the scour depth for some of the cases in combined waves and current flow conditions.
<table>
<thead>
<tr>
<th>Test No.</th>
<th>Pipe Diameter</th>
<th>Sand size</th>
<th>Undisturbed velocity near the bed</th>
<th>Shields parameter</th>
<th>Dimensionless equilibrium scour depth</th>
<th>Time scale</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>D (mm)</td>
<td>d50 (mm)</td>
<td>Uc (m/s)</td>
<td>θc</td>
<td>S_ef/D</td>
<td>S_ep/D</td>
</tr>
<tr>
<td>Oc1*</td>
<td>196</td>
<td>0.24</td>
<td>0.53</td>
<td>0.18</td>
<td>0.54</td>
<td>0.60</td>
</tr>
<tr>
<td>Oc2*</td>
<td>196</td>
<td>0.24</td>
<td>0.97</td>
<td>0.56</td>
<td>0.79</td>
<td>0.85</td>
</tr>
<tr>
<td>Oc3*</td>
<td>50</td>
<td>0.6</td>
<td>0.59</td>
<td>0.15</td>
<td>0.79</td>
<td>0.80</td>
</tr>
<tr>
<td>Oc4*</td>
<td>50</td>
<td>0.24</td>
<td>0.59</td>
<td>0.27</td>
<td>0.73</td>
<td>0.76</td>
</tr>
<tr>
<td>Oc5*</td>
<td>150</td>
<td>0.24</td>
<td>0.65</td>
<td>0.25</td>
<td>0.49</td>
<td>0.51</td>
</tr>
</tbody>
</table>

Table 5-2 Experimental data on time scale of scour process in steady current due to Mao (1986).

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Pipe Diameter</th>
<th>Sand size</th>
<th>Flow velocity</th>
<th>Shields parameter</th>
<th>Time scale From the area (^1)</th>
<th>Time scale From the area (^2)</th>
<th>From curve fit</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>D (mm)</td>
<td>d50 (mm)</td>
<td>Uc (m/s)</td>
<td>θc</td>
<td>T</td>
<td>Tf</td>
<td>Tp</td>
</tr>
<tr>
<td>M1*</td>
<td>100</td>
<td>0.36</td>
<td>0.35</td>
<td>0.048</td>
<td>1140</td>
<td>3.1</td>
<td>752</td>
</tr>
<tr>
<td>M2*</td>
<td>100</td>
<td>0.36</td>
<td>0.40</td>
<td>0.065</td>
<td>1360</td>
<td>3.7</td>
<td>898</td>
</tr>
<tr>
<td>M3*</td>
<td>100</td>
<td>0.36</td>
<td>0.50</td>
<td>0.098</td>
<td>504</td>
<td>1.4</td>
<td>333</td>
</tr>
<tr>
<td>M4</td>
<td>50</td>
<td>0.36</td>
<td>0.64</td>
<td>0.18</td>
<td>19</td>
<td>0.2</td>
<td>13</td>
</tr>
<tr>
<td>M5</td>
<td>50</td>
<td>0.36</td>
<td>0.76</td>
<td>0.25</td>
<td>18</td>
<td>0.2</td>
<td>12</td>
</tr>
<tr>
<td>M6*</td>
<td>50</td>
<td>0.36</td>
<td>0.87</td>
<td>0.33</td>
<td>10</td>
<td>0.11</td>
<td>7</td>
</tr>
<tr>
<td>M7</td>
<td>50</td>
<td>0.36</td>
<td>0.99</td>
<td>0.43</td>
<td>10</td>
<td>0.11</td>
<td>7</td>
</tr>
</tbody>
</table>

Note: 1. Based on Eq.(5-1), data from (Fredsoe et al., 1992); 2. Based on Eq.(5-10).
### Table 5-3 Scour in waves (present experiments).

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Pipe Diameter</th>
<th>Sand size</th>
<th>Time period</th>
<th>Maximum orbital velocity near the bed</th>
<th>Wave Shields parameter</th>
<th>Keulegan Carpenter number</th>
<th>Dimensionless equilibrium scour depth</th>
<th>Time scale</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ow1*</td>
<td>196</td>
<td>0.24</td>
<td>8.5</td>
<td>0.72</td>
<td>0.28</td>
<td>31</td>
<td>0.58</td>
<td>0.63</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Eq.(5-1)</td>
<td>Eq.(5-7)</td>
</tr>
<tr>
<td>Ow2*</td>
<td>196</td>
<td>0.24</td>
<td>12.9</td>
<td>0.48</td>
<td>0.13</td>
<td>31</td>
<td>0.40</td>
<td>0.43</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Eq.(5-1)</td>
<td>Eq.(5-7)</td>
</tr>
<tr>
<td>Ow3*</td>
<td>196</td>
<td>0.24</td>
<td>21.4</td>
<td>0.53</td>
<td>0.14</td>
<td>57</td>
<td>0.57</td>
<td>0.63</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Eq.(5-12)</td>
<td>Eq.(5-7)</td>
</tr>
<tr>
<td>Ow4*</td>
<td>196</td>
<td>0.24</td>
<td>15.1</td>
<td>0.79</td>
<td>0.30</td>
<td>61</td>
<td>0.92</td>
<td>0.98</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
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<td></td>
<td></td>
<td></td>
<td>Eq.(5-12)</td>
<td>Eq.(5-12)</td>
</tr>
<tr>
<td>Ow5*</td>
<td>50</td>
<td>0.6</td>
<td>3.5</td>
<td>0.61</td>
<td>0.20</td>
<td>43</td>
<td>0.63</td>
<td>0.64</td>
</tr>
<tr>
<td></td>
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<td></td>
<td></td>
<td></td>
<td>Eq.(5-7)</td>
<td>Eq.(5-7)</td>
</tr>
<tr>
<td>Ow6*</td>
<td>50</td>
<td>0.24</td>
<td>3.5</td>
<td>0.39</td>
<td>0.12</td>
<td>27</td>
<td>0.49</td>
<td>0.54</td>
</tr>
<tr>
<td></td>
<td></td>
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<td></td>
<td></td>
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<td>Eq.(5-12)</td>
<td>Eq.(5-12)</td>
</tr>
<tr>
<td>Ow7*</td>
<td>50</td>
<td>0.24</td>
<td>3.5</td>
<td>0.63</td>
<td>0.26</td>
<td>44</td>
<td>1.05</td>
<td>1.12</td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td>Eq.(5-7)</td>
<td>Eq.(5-7)</td>
</tr>
</tbody>
</table>

**Notes:**
- *Ow1*: 196
- *Ow2*: 196
- *Ow3*: 196
- *Ow4*: 196
- *Ow5*: 50
- *Ow6*: 50
- *Ow7*: 50

**Equations:**
- Eq.(5-1)
- Eq.(5-7)
- Eq.(5-12)
Table 5-4 Experimental data on time scale of scour process in waves due to Fredsøe et al. (1992).

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Pipe diameter (mm)</th>
<th>Wave frequency ( f_w ) (s(^{-1} ))</th>
<th>Maximum flow velocity ( U_w ) (m/s)</th>
<th>Sand size ( d_{50} ) (mm)</th>
<th>Wave Shields parameter ( \theta_w )</th>
<th>Keulegan Carpenter number (KC)</th>
<th>From slope ( T_f^* ) (s)</th>
<th>From the area(^1) ( T_f^* ) (s)</th>
<th>From the area(^2) ( T_f^* ) (s)</th>
<th>From curve fit ( T_b^* ) (s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>F1</td>
<td>50</td>
<td>0.84</td>
<td>0.25</td>
<td>0.58</td>
<td>0.13</td>
<td>6</td>
<td>21</td>
<td>0.47</td>
<td>17</td>
<td>0.39</td>
</tr>
<tr>
<td>F2</td>
<td>30</td>
<td>0.42</td>
<td>0.14</td>
<td>0.58</td>
<td>0.035</td>
<td>11</td>
<td>45</td>
<td>2.8</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>F3</td>
<td>30</td>
<td>0.81</td>
<td>0.27</td>
<td>0.58</td>
<td>0.14</td>
<td>11</td>
<td>12</td>
<td>0.75</td>
<td>10</td>
<td>0.64</td>
</tr>
<tr>
<td>F4</td>
<td>30</td>
<td>0.82</td>
<td>0.24</td>
<td>0.18</td>
<td>0.19</td>
<td>11</td>
<td>24</td>
<td>0.25</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>F5</td>
<td>30</td>
<td>0.55</td>
<td>0.22</td>
<td>0.58</td>
<td>0.08</td>
<td>13</td>
<td>24</td>
<td>1.5</td>
<td>56</td>
<td>3.5</td>
</tr>
<tr>
<td>F6</td>
<td>30</td>
<td>0.40</td>
<td>0.18</td>
<td>0.18</td>
<td>0.1</td>
<td>15</td>
<td>138</td>
<td>1.5</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>F7</td>
<td>30</td>
<td>0.37</td>
<td>0.21</td>
<td>0.58</td>
<td>0.06</td>
<td>19</td>
<td>33</td>
<td>2.1</td>
<td>49</td>
<td>3</td>
</tr>
<tr>
<td>F8</td>
<td>20</td>
<td>0.67</td>
<td>0.27</td>
<td>0.18</td>
<td>0.18</td>
<td>20</td>
<td>18</td>
<td>0.44</td>
<td>25</td>
<td>0.6</td>
</tr>
<tr>
<td>F9</td>
<td>30</td>
<td>0.27</td>
<td>0.18</td>
<td>0.58</td>
<td>0.03</td>
<td>23</td>
<td>45</td>
<td>2.8</td>
<td>121</td>
<td>7.6</td>
</tr>
<tr>
<td>F10</td>
<td>30</td>
<td>0.50</td>
<td>0.35</td>
<td>0.18</td>
<td>0.19</td>
<td>23</td>
<td>18</td>
<td>0.19</td>
<td>14</td>
<td>0.15</td>
</tr>
<tr>
<td>F11*</td>
<td>30</td>
<td>0.32</td>
<td>0.26</td>
<td>0.58</td>
<td>0.035</td>
<td>27</td>
<td>48</td>
<td>3</td>
<td>156</td>
<td>9.8</td>
</tr>
<tr>
<td>F12</td>
<td>30</td>
<td>0.37</td>
<td>0.39</td>
<td>0.18</td>
<td>0.19</td>
<td>35</td>
<td>21</td>
<td>0.22</td>
<td>32</td>
<td>0.35</td>
</tr>
<tr>
<td>F13</td>
<td>30</td>
<td>0.22</td>
<td>0.24</td>
<td>0.58</td>
<td>0.03</td>
<td>36</td>
<td>57</td>
<td>3.6</td>
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<td>5.9</td>
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<td>10</td>
<td>0.35</td>
<td>0.20</td>
<td>0.58</td>
<td>0.05</td>
<td>56</td>
<td>8</td>
<td>4.2</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

Note: 1. Based on Eq.(5-1), data from (Fredsøe et al., 1992); 2. Based on Eq.(5-10),
Table 5-5 Scour in combined waves and current (present experiments).
5.4 Time scale in steady currents

Figure 5-5 presents the three dimensionless time scales $T_f^*$, $T_b^*$ and $T_p^*$ together with the corresponding available current Shields parameter data under steady current flow conditions. The current Shields parameter in this figure has been computed according to

$$\theta = \frac{u_c^2}{g(s-1)d_{50}}$$  \hspace{1cm} (5-13)

where $u_c$ is current friction velocity and has been calculated assuming a logarithmic velocity profile:

$$U_c(z) = \frac{u_c}{\kappa} \ln \left( \frac{z}{z_0} \right),$$  \hspace{1cm} (5-14)

where $z_0$ is bed roughness length and $\kappa$ is von Karman’s constant ($= 0.41$). Following Soulsby (1997), the bed roughness length $z_0$ has been estimated using the expression of Christoffersen and Jonsson (1985):

$$z_0 = \frac{k_s}{30} \left[ 1 - \exp\left( \frac{-u_c k_s}{27v} \right) \right] + \frac{v}{9u_c},$$  \hspace{1cm} (5-15)

where $v$ is the kinematic viscosity of water, the Nikuradse roughness $k_s$ (taken to be 2.5$d_{50}$ (Soulsby, 1997)). It should be noted that this method of calculating the Shields parameter differs by less than 2% compared with the method reported by Fredsøe et al. (1992). For this reason, the current Shields parameter data in Table 5-2 and some of the cases in Figure 5-5 takes directly from the previous reference to maintain consistency.

A clear observation from Figure 5-5 is that all three normalized time scales correlate very well with the current Shields parameter. The clear trend is that the time scale decreases with increases in Shields parameter, as demonstrated by Fredsøe et al. (1992).

5.5 Time scale in waves

Figure 5-6 presents the three dimensionless time scales $T_f^*$, $T_p^*$ and $T_b^*$, plotted against the wave Shields parameter $\theta_w$. The wave Shields parameter $\theta_w$ has been calculated for each experiment according to

$$\theta_w = \frac{u_{w}^2}{g(s-1)d_{50}},$$  \hspace{1cm} (5-16)

in which the undisturbed friction velocity $u_{w}$ is calculated from
Chapter 5 Time scale in current/wave and combined wave and current

\[ u_{*w} = \sqrt{\frac{f_w}{2} U_w}, \]

(5-17)

here, \( f_w \) is the friction coefficient and is estimated using the following equations (Fredsøe and Deigaard, 1992).

In the case of a hydraulically rough bed \( (d_{50} u_{*w}/v > 10) \), the friction factor \( f_w \) is defined as

\[
f_w = \begin{cases} 
0.04 \left( \frac{a}{k_s} \right)^{-0.25}, & \frac{a}{k_s} > 50 \\
0.04 \left( \frac{a}{k_s} \right)^{-0.75}, & \frac{a}{k_s} < 50 
\end{cases}
\]

(5-18)

where \( a \) is the free stream amplitude, given by

\[ a = \frac{U_w T_w}{2\pi}. \]

(5-19)

In the case of a hydraulically smooth bed \( (d_{50} u_{*w}/v < 10) \) the variation in the friction factor is approximated by

\[ f_w = 0.035 RE^{-0.16}, \]

(5-20)

where \( RE \) is the amplitude Reynolds number, calculated by

\[ RE = \frac{U_w a}{v}. \]

(5-21)

The previous investigation (Fredsøe et al., 1992) indicated that the final scour depth mainly depends on \( KC \) number, whilst the normalized time scale is governed by the wave Shields parameter only. In Figure 5-6 all three methods show that larger wave Shields parameter generally leads to shorter time to reach the equilibrium stage, and that there is a clear correlation between wave Shields parameter and the measurement.

5.6 Time scale in combined waves and current

From the above discussion, it is clear shown that the dimensionless time scale is mainly dependent on maximum Shields parameter under current only and waves only flow conditions. Although a non-linear interaction exists under combined waves and current flow conditions, it is reasonable to believe that the dimensionless time scale in combined waves and current is also affected by the Shields parameter.

As noted in the introduction, for the case of perpendicular combined waves and current conditions there are at least three different Shields parameters which may be considered: the maximum Shields parameter \( \theta_{max} \), the mean Shields parameter \( \theta_m \), and the effective
Chapter 5 Time scale in current/wave and combined wave and current

Shields parameter $\theta_{eff} = \theta_w (1 - m) + \theta_c$ adopted by Cheng et al. (2014). In this study, the maximum Shield parameter has been calculated according to

$$\theta_{max} = \frac{\tau_{max}}{g(\rho_s - \rho)d_{50}},$$

(5-22)

where the maximum shear stress $\tau_{max}$ in combined waves and current can be estimated according to Soulsby (1997):

$$\tau_{max} = \tau_m + \tau_w.$$  
(5-23)

The mean shear stress, $\tau_m$, which is dependent on the non-linear wave and current interaction has been estimated following Soulsby (1997)

$$\frac{\tau_m}{\tau_c} = 1 + 1.2 \left( \frac{\tau_w}{\tau_c + \tau_w} \right)^{3.2},$$

(5-24)

where $\tau_c$ and $\tau_w$ corresponds the shear stresses due to the current alone and wave alone conditions respectively.

To investigate which definition is most appropriate, the mean, effective and maximum Shields parameters are plotted against the dimensionless time scales in Figure 5-7. A clear trend is shown for all three methods, with reduction the dimensionless time scale associated with an increasing in Shields parameter. This result is reasonable because if implies that larger sediment transport rates around the pipe lead to faster scour.

Comparing the three types of Shields parameter, it is clear that the time scale appears to correlate best with the effective Shields parameter and the maximum Shields parameter. A possible explanation for this is that the maximum Shields parameter determines the entrainment rate of sediments while the mean Shields parameter is responsible for sediment diffusion. Furthermore, due to the shear stress being amplified beneath the pipeline, the maximum shear stress becomes many times larger than the mean shear stress, such that the maximum shear stress (represented by the maximum Shields parameter) controls the erosion process beneath the pipeline.

5.7 Time scale in any combination of waves and current

Since the dimensionless time scale in current only, wave only and combined waves and current are all dependent on the maximum Shields parameter, it is possible to produce one universal formula based on the available data which can be applied for all three flow conditions.

To do that, the time scale $T_f^*, T_p^*$ and $T_b^*$ obtained based on Eq.(5-1), Eq.(5-7) and Eq.(5-12) are plotted against the mean, effective and maximum Shields parameters in all...
Chapter 5 Time scale in current/wave and combined wave and current

flow conditions in Figure 5-8. To evaluate how well the dimensionless time scales collapse with Shields parameter, a power function trendline is fitted for all available data in each sub figure. Comparing the value of $R^2$, it is demonstrated that for each dimensionless time scale method, as discussion above, the data calculated from the mean Shields parameter (Figure 5-8 (a-1) (b-1) (c-1)) has much lower $R^2$ value than the data calculated from the effective Shields parameter (Figure 5-8 (a-2) (b-2) (c-2)) and maximum Shields parameter. Furthermore, the data with maximum Shields parameter shows the best fit with the effective Shields parameter (Figure 5-8 (a-3) (b-3) (c-3)).

Among the three dimensionless time scales, the dimensionless time scale obtained from Eq.(5-1) (Figure 5-8 (a-3)) and Eq.(5-12) (Figure 5-8 (c-3) have similar goodness of fit with the data, however, the best fit is found when the dimensionless time scale calculated by Eq.(5-7) and the maximum Shields parameter is used (Figure 5-8 (b-3)). A general empirical fit between the maximum Shields parameter and dimensionless time scale is now developed.

From the view of sediment transport, based simply on continuity arguments, the time scale of scour should be inversely proportional with the sediment transport rate $q$ underneath the pipeline, so that $T_p \propto 1/q$ (or $T_p^* \propto 1/\Phi$, where $\Phi$ is the dimensionless transport rate). Although the transport rate in combined waves and current is difficult to determine, in current only and wave only conditions it is widely accepted that when the shear stress significantly exceeds the critical shear stress, as is likely to be the case in live bed conditions under the pipeline at the start of scour, dimensionless analysis and many empirical equations for transport rate suggest that (see, for example, in currents: (Bagnold, 1963; Madsen, 1991; Meyer-Peter and Müller, 1948; Wilson, 1966; Yalin, 1963) ; and in waves: (Nielsen, 1992; Sleath, 1978; Soulsby, 1997):

$$\Phi \propto \theta^{1.5}. \quad (5-25)$$

Hence, based on continuity and Eq. (5-25) it is reasonable to expect that the empirical formula for dimensionless time scale in all three flow conditions should be of the form $T_p^* = A\theta_{max}^{-1.5}$. From Figure 5-8 (c-3), it shown that the exponent of Shields parameter in the best data fit formula is -1.557, which is very close to -1.5. Therefore, in this study, the exponent of Shields parameter is set to -1.5 and the fitted equation is given by:

$$T_p^* = 0.017\theta_{max}^{1.5}, \quad (5-26)$$

the Eq. (5-26) is shown in Figure 5-9 and appears to explain the experimental data very well.
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The previous empirical formula proposed by Fredsøe et al. (1992) for waves only and currently only conditions is also plotted in Figure 5-9. This equation is similar to Eq. (5-26), however it correlates less well with the data, especially for $\theta_{max} > 0.1$ where live bed conditions are expected and therefore Eq.(5-26) is more likely. The scatter in the data shown in Figure 5-9 may be attributed to the potential influence of $KC$ number and flow ratio $m$, as well as experimental errors, however it is clear that the maximum Shields parameter explains the majority of the trend.

5.8 Discussion

In this study it has been shown that existing and new experimental data for scour development beneath a fixed pipeline can be explained well by a formula of the type suggested by Whitehouse (1998) when $p =0.6$. This exponent is slightly different to the value of $p =1$ adopted by Fredsøe et al. (1992), but it appears to explain the data consistently better. The equilibrium scour depth of present experiments are all obtained from the curve of Eq.(5-7) fit with the measured scour depth development data, which are all presented in Chapter 4 for the equilibrium scour depth investigation.

In this Chapter is has also been shown, for the first time, that the non-dimensional time scale can be computed for any combination of wave and current conditions using Eq.(5-26). This confirms the Whitehouse (1998) hypothesis, but only if the maximum combined waves and current shear stress is used to define the Shields parameter in combined waves and current flow conditions.

In summary, the present work is important in that it now gives, for the first time, a method to predict local pipeline scour in any combination of perpendicular waves and current conditions. This result may be used to develop scour predictions in engineering models and update more complicated models of 3D scour processes along pipelines, such as that investigated by Cheng et al. (2014).

5.9 Conclusion

The following conclusions can be drawn:

1. Among the three methods compared in this study, Eq.(5-7) and Eq.(5-12) appears to provide a better empirical fit for scour development beneath a fixed pipeline than previously quoted formula such as that given in Eq.(5-1) based on new and all available experimental measurements,
2. The non-dimensional time scale of the scour process below a pipeline in combined waves and current is governed by the maximum Shields parameter.

3. A universal equation can be written for the non-dimensional time scale in any combination of wave and current conditions. After comparison, the best fit with data is found when the time scale is obtained from Eq.(5-7). And the time scale in current only, waves only and combined waves and current flow conditions can be calculated based on Eq.(5-26).
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Reference


Cheng, L., Yeow, K., Zang, Z. and Li, F., 2014. 3d scour below pipelines under waves and combined waves and currents. Coastal Engineering, 83(0): 137-149.


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Figure 5-1 Definition sketch of scour beneath a pipeline. Scour depth is illustrated.

Figure 5-2 The mean R-square changing with $p$ from 0.01 to 2.
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(a-1) $R^2 = 0.9351$

(a-2) $R^2 = 0.9796$

(b-1) $R^2 = 0.9351$
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Figure 5-3 Time development of scour depth in all conditions, experimental data compared with the curve defined by: (a) Eq.(5-1); (b) Eq.(5-7); (c) Eq.(5-12).
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Figure 5-4 The time scale predicted from the scour-depth-versus-time information in the previous study (Fredsoe et al., 1992): (a) From the slope; (b) From the area.

Figure 5-5 Dimensionless plot of time scale against Shields parameter in steady current, dimensionless time scale obtained from: (a) Eq.(5-1); (b) Eq.(5-7); (c) Eq. (5-12).

Figure 5-6 Dimensionless plot of time scale against Shields parameter in waves, dimensionless time scale obtained from: (a) Eq.(5-1); (b) Eq.(5-7); (c) Eq.(5-12).
Figure 5-7 Dimensionless plot of time scale against Shields parameter in combined waves and current, dimensionless time scale obtained from: (a) Eq.(5-1); (b) Eq.(5-7); (c) Eq.(5-12).
Figure 5-8 Dimensionless plot of time scale in all conditions, dimensionless time scale obtained from: (a) Eq.(5-1); (b) Eq.(5-7); (c) Eq.(5-12).
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Figure 5-9 Dimensionless plot of time scale against Shields parameter. All data (steady-current, waves and combined waves and current).
6. TIME SCALE OF LOCAL SCOUR BELOW PIPELINE IN CHANGING FLOW CLIMATE

In this chapter, the investigation of time scale of local scour below a pipeline is extended to consider multiple consecutive flow conditions and ramp-up flow conditions. The algorithms to accumulate the effects of different flow velocities are based on those derived from the last chapter and previous investigations. Several experiments were conducted and used as case histories to develop predictive formula, which assumes that the scour process is quasi-steady. The results show that the present formula gives a good fit with the measured data.

6.1 Introduction

In practice, the scour process under time variable flow conditions is important to understand in offshore pipeline design. A few researchers have considered time varying flow conditions. For instance, Fredsøe et al. (1992) conducted several waves only conditions experiments to investigate the scour process following a change in waves. They concluded that the equilibrium scour depth is always determined by the final KC number in a transitional situation where the waves change from one climate to another. Whitehouse (1998) suggested a time stepping approach to predict the scour development in time varying flow. Briaud et al. (2001) proposed a method to predict the scour depth at a cylindrical bridge pier for a random velocity-time history and multilayer soil stratigraphy. Harris et al. (2010) described the development of an engineering model to predict the development of scour evolution through time around an offshore structure under current, waves and combined wave-current flow. Draper et al. (2015) investigated the stability of subsea pipelines during large storm flow conditions theoretically and experimentally.

Since new formulae to predict the scour depth development and time scale were proposed in the last chapter for current only, waves only and combined waves and current flow conditions that do not change in time. It is necessary to extend these new formulae to multiple independent flow conditions and continuously changing (ramp-up) flow
Chapter 6 Time scale in changing flow climate

conditions. This chapter develops a predictive model to achieve this. The predictive model are compared and validated against experimental data.

6.2 Experiment setup

Scour experiments were conducted in the LOT and MOT. The details of the facilities can be found in Chapter 1. The measurement of velocity and scour depth were consistent with the methods mentioned in Chapter 4. The scour profiles during the LOT experiments were obtained from video images, while the final scour profile in the LOT and MOT were obtained from the three-dimensional image devices.

The LOT tests conditions and results are listed in Table 6-1. Each test in the LOT was divided into two stages. The first stage test was started from a flat seabed and the gap between the pipeline and seabed was nil. The pipeline was fixed at the original position and the test was run to the equilibrium stage. The first stage tests were also employed in the discussion in previous chapter. For the second stage of the test, the pipeline was still fixed in the original position, based on the previous stage scour profile, then, a different flow conditions was introduced until a new equilibrium stage was reached corresponding to the new flow condition. At the end of each stage, the scour profile was scanned by the handheld three-dimensional camera.

The ramp-up tests were conducted in the MOT facility. The conditions and results are listed in Table 6-2. Each ramp test contained two flow conditions stages, a ramp-up stage and a constant flow stage, respectively. The test was started from the ramp-up stage with a flat seabed and the gap between the pipeline and seabed was nil. Then, the pipeline was maintained at the original position, the peak flow remained constant in the second stage.
## Chapter 6 Time scale in changing flow climate

Table 6-1 Two stages scour experiments performed in the LOT.

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Time period</th>
<th>Max velocity near bed</th>
<th>Undisturbed velocity near the bed</th>
<th>Shields parameter in current alone</th>
<th>Max Shields parameter in wave</th>
<th>Max Shields parameter</th>
<th>Keulegan-Carpenter number</th>
<th>$\frac{U_c}{U_c + U_w}$</th>
<th>Pipe Reynolds number based on $U_c$</th>
<th>Pipe Reynolds number base on $U_w$</th>
<th>Scour depth</th>
<th>Time scale</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$T_w$</td>
<td>$U_m$ m/s</td>
<td>$U_c$ m/s</td>
<td>$\theta_c$</td>
<td>$\theta_w$</td>
<td>$\theta_{max}$</td>
<td>$KC$</td>
<td>$m$</td>
<td>$Re_c$</td>
<td>$Re_w$</td>
<td>$\frac{S}{D}$</td>
<td>$T_p$ s</td>
</tr>
<tr>
<td>Owc1</td>
<td>8.24</td>
<td>0.33</td>
<td>0.12</td>
<td>0.01</td>
<td>0.08</td>
<td>0.09</td>
<td>14</td>
<td>0.26</td>
<td>$1.7 \times 10^4$</td>
<td>4.8 $\times 10^4$</td>
<td>0.25</td>
<td>352</td>
</tr>
<tr>
<td>Owc1-2</td>
<td>--</td>
<td>--</td>
<td>0.62</td>
<td>0.17</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td>$8.9 \times 10^4$</td>
<td>--</td>
<td>0.72</td>
<td>639</td>
</tr>
<tr>
<td>Owc2</td>
<td>8.66</td>
<td>0.29</td>
<td>0.31</td>
<td>0.04</td>
<td>0.06</td>
<td>0.11</td>
<td>13</td>
<td>0.52</td>
<td>$4.4 \times 10^4$</td>
<td>4.1 $\times 10^4$</td>
<td>0.44</td>
<td>436</td>
</tr>
<tr>
<td>Owc2-2</td>
<td>8.6</td>
<td>0.33</td>
<td>0.83</td>
<td>0.30</td>
<td>0.08</td>
<td>0.38</td>
<td>15</td>
<td>0.71</td>
<td>$1.2 \times 10^5$</td>
<td>4.8 $\times 10^4$</td>
<td>1.13</td>
<td>179</td>
</tr>
<tr>
<td>Owc4</td>
<td>10.66</td>
<td>0.50</td>
<td>0.18</td>
<td>0.02</td>
<td>0.15</td>
<td>0.18</td>
<td>27</td>
<td>0.27</td>
<td>$2.6 \times 10^4$</td>
<td>7.2 $\times 10^4$</td>
<td>0.46</td>
<td>184</td>
</tr>
<tr>
<td>Owc4-2</td>
<td>12.28</td>
<td>0.79</td>
<td>--</td>
<td>0.31</td>
<td>--</td>
<td>50</td>
<td>--</td>
<td>--</td>
<td>$1.1 \times 10^5$</td>
<td>--</td>
<td>0.77</td>
<td>247</td>
</tr>
<tr>
<td>Owc6</td>
<td>12.76</td>
<td>0.46</td>
<td>1.18</td>
<td>0.62</td>
<td>0.12</td>
<td>0.75</td>
<td>30</td>
<td>0.72</td>
<td>$1.7 \times 10^5$</td>
<td>6.6 $\times 10^4$</td>
<td>0.79</td>
<td>78</td>
</tr>
<tr>
<td>Owc6-2</td>
<td>22.48</td>
<td>1.05</td>
<td>--</td>
<td>0.46</td>
<td>--</td>
<td>120</td>
<td>--</td>
<td>--</td>
<td>$1.4 \times 10^5$</td>
<td>--</td>
<td>1.3</td>
<td>132</td>
</tr>
<tr>
<td>Owc3</td>
<td>8.64</td>
<td>0.35</td>
<td>0.96</td>
<td>0.41</td>
<td>0.09</td>
<td>0.50</td>
<td>16</td>
<td>0.73</td>
<td>$1.4 \times 10^5$</td>
<td>5.1 $\times 10^4$</td>
<td>0.90</td>
<td>152</td>
</tr>
<tr>
<td>Owc3-2</td>
<td>8.54</td>
<td>0.33</td>
<td>0.32</td>
<td>0.05</td>
<td>0.08</td>
<td>0.14</td>
<td>14</td>
<td>0.50</td>
<td>$4.7 \times 10^4$</td>
<td>4.8 $\times 10^4$</td>
<td>0.81</td>
<td>869</td>
</tr>
<tr>
<td>Owc5</td>
<td>10.7</td>
<td>0.54</td>
<td>0.53</td>
<td>0.12</td>
<td>0.17</td>
<td>0.32</td>
<td>29</td>
<td>0.50</td>
<td>$7.6 \times 10^4$</td>
<td>7.7 $\times 10^4$</td>
<td>0.85</td>
<td>371</td>
</tr>
<tr>
<td>Owc5-2</td>
<td>--</td>
<td>--</td>
<td>-0.63</td>
<td>0.17</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td>$9.0 \times 10^4$</td>
<td>--</td>
<td>0.81</td>
</tr>
</tbody>
</table>

Note: The diameter of the model pipe in LOT is 196 mm, the $d_{50}$ of test sediment is 0.24 mm.
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Table 6-2 Experiments performed in the MOT.

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Time period</th>
<th>Max velocity near the bed</th>
<th>Undisturbed velocity near the bed</th>
<th>Shields parameter in current alone</th>
<th>Max Shields parameter in wave</th>
<th>Max Shields parameter</th>
<th>Keulegan-Carpenter number</th>
<th>$\frac{U_c}{U_c + U_w}$</th>
<th>Max Pipe Reynolds number based on $U_c$</th>
<th>Max Pipe Reynolds number Base on $U_w$</th>
<th>Curve fit scour depth</th>
<th>Rate</th>
</tr>
</thead>
<tbody>
<tr>
<td>OCR-1</td>
<td>--</td>
<td>--</td>
<td>0-0.67</td>
<td>0.29</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td>$2.4 \times 10^4$</td>
<td>--</td>
<td>$3.7 \times 10^{-4}$</td>
<td>0.8</td>
</tr>
<tr>
<td>OCR-2</td>
<td>--</td>
<td>--</td>
<td>0.67</td>
<td>0.29</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td>$2.4 \times 10^4$</td>
<td>--</td>
<td>$3.7 \times 10^{-4}$</td>
<td>0.8</td>
</tr>
<tr>
<td>OCR-3</td>
<td>3.5</td>
<td>--</td>
<td>0.67</td>
<td>0.29</td>
<td>--</td>
<td>26</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td>$1.4 \times 10^4$</td>
<td>0.5</td>
<td>2.1</td>
</tr>
<tr>
<td>OWR-1</td>
<td>3.5</td>
<td>0-0.37</td>
<td>--</td>
<td>--</td>
<td>0.11</td>
<td>26</td>
<td>--</td>
<td>--</td>
<td>$1.4 \times 10^4$</td>
<td>--</td>
<td>0.5</td>
<td>2.1</td>
</tr>
<tr>
<td>OWR-2</td>
<td>3.5</td>
<td>0.37</td>
<td>--</td>
<td>--</td>
<td>0.11</td>
<td>26</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td>$1.4 \times 10^4$</td>
<td>0.5</td>
<td>2.1</td>
</tr>
<tr>
<td>OWR-3</td>
<td>3.5</td>
<td>0.37-0</td>
<td>--</td>
<td>--</td>
<td>0.11</td>
<td>26</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td>$1.4 \times 10^4$</td>
<td>0.5</td>
<td>2.1</td>
</tr>
<tr>
<td>OWCR-1</td>
<td>3.5</td>
<td>0-0.31</td>
<td>0.33</td>
<td>0.07</td>
<td>0.08</td>
<td>0.17</td>
<td>22</td>
<td>0.5</td>
<td>$1.2 \times 10^4$</td>
<td>$1.2 \times 10^4$</td>
<td>0.65</td>
<td>2.1</td>
</tr>
<tr>
<td>OWCR-2</td>
<td>3.5</td>
<td>0.31</td>
<td>0.33</td>
<td>0.07</td>
<td>0.08</td>
<td>0.17</td>
<td>22</td>
<td>0.5</td>
<td>$1.2 \times 10^4$</td>
<td>$1.2 \times 10^4$</td>
<td>0.65</td>
<td>2.1</td>
</tr>
<tr>
<td>OWCR-3</td>
<td>3.5</td>
<td>0.31-0</td>
<td>0.33</td>
<td>0.07</td>
<td>0.08</td>
<td>0.17</td>
<td>22</td>
<td>0.5</td>
<td>$1.2 \times 10^4$</td>
<td>$1.2 \times 10^4$</td>
<td>0.65</td>
<td>2.1</td>
</tr>
</tbody>
</table>

Note: The pipe diameter is 50 mm, the $d_{50}$ of test sediment is 0.24 mm.
6.3 **Continued scour process under unsteady flow condition**

The velocity time history experienced by a pipeline installed on a seabed over many years is quite complex, and can include tidal current, solitons and storm with different return periods. Different flow conditions lead to different scour profiles around the pipe, such as symmetric scour profiles (see Figure 6-1 (a)) in waves only flow conditions and antisymmetric scour profiles (see Figure 6-1 (b)) in current only and combined waves and current flow conditions. Moreover, even in the same type of flow, different flow intensities generate various scales of scour profile. Different flow types and flow intensities will reflect on the evolution of the scour depth underneath the pipe. The increase of the approaching flow intensity normally causes the sediment transport capacity of the flow underneath the pipe to increase. Consequently the scour depth underneath the pipeline may become deeper, and such transitional process may be called continued scour process.

6.3.1 **Two consecutive flow conditions**

In order to investigate the influence of the increase of approach flow intensity, the case of a sequence of two different, yet constant, flow conditions is considered first. This resembles a step change in flow conditions.

Considering the equilibrium stage has reached under each of these two steps, therefore, the scour depth development with time for the initial flow conditions can be expressed as

\[
S_1(t) = S_1 \left( 1 - \exp \left( -\frac{t}{T_1} \right)^p \right),
\]

\[(6-1)\]

where, \(S_1\) and \(T_1\) are the equilibrium scour depth and time scale corresponding to the first flow condition.

The scour due to the higher velocity second flow develops from the equilibrium scour profile after the first step flow, which can be expressed by

\[
S_2(t) = S_2 \left( 1 - \exp \left( -\frac{t + t_i}{T_2} \right)^p \right),
\]

\[(6-2)\]

where, \(S_2\) and \(T_2\) are the equilibrium scour depth and time scale corresponding to the second flow condition. \(t_i\) is the time elapse for second flow conditions to achieve the scour depth of first flow condition, which is calculated as

\[
t_i = T_2 \left[ -\ln \left( \frac{S_2 - S_1}{S_2} \right) \right]^{1/p}.
\]

\[(6-3)\]
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For \( p = 1 \), following Fredsøe et al. (1992), the time elapse \( t_{fi} \) is

\[
t_{fi} = -T_{f2} \ln \left( \frac{S_2 - S_1}{S_2} \right).
\]

(6-4)

For \( p = 0.6 \), as was obtained in the previous chapter, the time elapse \( t_{pi} \) is

\[
t_{pi} = T_{p2} \left[ -\ln \left( \frac{S_2 - S_1}{S_2} \right) \right]^{5/3}.
\]

(6-5)

For the LOT tests Owc1, Owc2, Owc4 and Owc6 in Table 6-1, after the equilibrium scour stage was reached in the first stage, a stronger flow conditions was applied until a new equilibrium scour stage reached. All these four tests were started from a combined waves and current flow climate, then, followed by a steady current (Owc1-2), combined waves and current (Owc2-2) and waves only (Owc4-2 and Owc6-2) respectively.

The scour depth time history during the whole process including the scour process of the first stage and the continuous scour process of the second stage are plotted in Figure 6-2. In each sub-figure, the predicted curves in the first stage were based on Eq. (6-1) with \( p = 1 \) (the dash line) and \( p = 0.6 \) (the solid line), likewise the predicted curves in the second stage were based on the Eq.(6-2). It should be note that in the scour process of the first stage, the equilibrium scour depth and the time scale were gain from the curve fitting exercise to the experimental data as \( p = 1 \) and \( p = 0.6 \) respectively, which was consistent with the previous chapter. While the equilibrium scour depth in the second stage scour process was determined by the final scour depth.

For \( p = 1 \), the non-dimensional time scale \( T_f^* \) was calculated by the empirical formula represented by previous authors (Fredsøe et al., 1992)

\[
T_f^* = \frac{1}{50} \theta^{-5/3},
\]

(6-6)

where \( \theta \) is the Shields parameter. And it is replaced by the current Shields parameter \( \theta_c \) in steady current; while in waves only it is replaced by the maximum waves Shields parameter \( \theta_{w} \); in combined waves and current it is replaced by the maximum Shields parameter \( \theta_{max} \).

The time elapse for the second flow to reach the equilibrium scour depth in the second flow was calculated by Eq.(6-4), and the predicted curve in the second stage was calculated by Eq.(6-2) with \( p = 1 \).

For \( p = 0.6 \) the non-dimensional time scale was calculated by the empirical formula provided in the previous chapter

\[
T_p^* = 0.017 \theta^{-1.5}
\]

(6-7)
The time elapse for the second flow to reach the equilibrium scour depth in the second flow was calculated by Eq.(6-5), meanwhile, the predicted curve in the second stage was calculated by Eq.(6-2) with \( p = 0.6 \). Although Briaud et al. (1999) proposed a hyperbola equation to predict the scour depth development for pier on cohesive sediment, no corresponding non-dimensional time scale formula was proposed based on the hyperbola equation, therefore, the hyperbola equation was not included in this comparison.

It is shown in the Figure 6-2 that the two predicted curves both collapse fairly well with the experimental data, and the curve with \( p = 0.6 \) was a slightly better fit than the curve with \( p = 1 \).

For completeness, Figure 6-3 illustrates the development of bed profile in the second stage with time for these four cases. It can be seen that the bed profile changes dramatically from the first stage equilibrium scour profile to the final equilibrium scour profile. Furthermore, at the initial stage of scour, the scour profile transforms quite fast, not only the scour depth underneath the pipeline increased fast, but also the rudiment of the second flow scour profile was formed quickly. And then with the time elapsed, the scour rate around the pipeline gradually slowed down, meanwhile, the scour depth underneath the pipeline increased slowly based on the characteristic scour profile around the pipeline corresponding to the second flow type. Finally, after sufficient time, the equilibrium stage was reached, which was shown in the Figure 6-4.

### 6.3.2 Continuously Ramp-up flow conditions

The second type of the continues scour process is analysed a continuous increasing flow climate, namely ramp-up flow. This scenario is important to understand because, in practice, it can be used to simulate the ramp-up phase of a storm induced flow condition. The whole period of storm normally contains three stages, the ramp-up stage, the peak and the ramp-down stage. In this section, the first two stages during a whole period of storm are considered.

Whitehouse (1998) reported a methodology for predicting the scour development in unsteady flows based on the assumption that the quasi-steady flow exists if the time-step is small enough relative to the period of the non-steady fluctuation.

\[
\Delta S = \frac{dS}{dt} \Delta t, \tag{6-8}
\]

Assuming the scour depth development with time can be defined by
\[ S(t) = S \left( 1 - \exp \left( -\left( \frac{t}{T_p} \right)^p \right) \right). \] (6-9)

This implies that the scour depth increment during any time step can be calculated by

\[ \frac{dS}{dt} = \frac{p(S_n - S_i)}{T_n \left[ -\ln \left( \frac{S_n - S_i}{S_n} \right) \right]^{\frac{1}{p-1}}}, \] (6-10)

where \( S_n \) is the equilibrium scour depth corresponding to the increased flow conditions for each increment, \( S_i \) is the scour depth which has already occurred and \( T_n \) is the time scale corresponding to \( S_n \). It should be noted that the \( S_n \) is regarded as the ultimate final scour depth in the book (Whitehouse, 1998).

For \( p = 1 \), Eq.(6-10) can be simplified to

\[ \frac{dS}{dt} = \frac{S_n - S_i}{T_n}. \] (6-11)

For \( p = 0.6 \), Eq.(6-10) can be written as

\[ \frac{dS}{dt} = \frac{0.6(S_n - S_i)}{T_n \left[ -\ln \left( \frac{S_n - S_i}{S_n} \right) \right]^{\frac{2}{3}}} \] (6-12)

Taking a ramp-up current case as an example, if the current velocity increases at a constant rate of \( a_s \) (m/s\(^2\)), the velocity time history can be divided into many time steps. If the step is small enough, the current flow in this section can be roughly regarded as quasi-steady flow. The equilibrium scour depth in each time step can be estimated using the empirical equation presented in chapter 4, givelly

\[ S_c = 1.052 \left( \frac{U^2}{2g} \right)^{0.25} D^{0.75}. \] (6-13)

The time scale for each time step is dependent on the current Shields parameter corresponding to the current velocity in each time step. For \( p = 1 \), the dimensionless time scale \( T_f^* \) was calculated via Eq.(6-6), and the scour depth increment in each time step was calculated via Eq.(6-11). Meanwhile, for \( p = 0.6 \), the dimensionless time scale \( T_p^* \) was calculated via Eq.(6-7), and the scour depth increment in each time step was calculated via Eq.(6-12). Figure 6-5 (a) shows the measured velocity time history for a ramp-up current (OCR-1), followed by a steady current (OCR-2). Figure 6-5 (b) shows the measured scour depth varying with time under such velocity time history and the predicted curves using the above method. The predicted curve with \( p = 0.6 \) has a quite good agreement with the experimental data at the initial stage of the ramp-up current.
history, then starts to overestimate the scour rate after the current velocity exceeds 0.45 m/s.

Figure 6-6 (a) shows the measured velocity for waves only ramp-up case (OWR-1) and following peak waves only case (OWR-2). The method used to predict scour depth development with time for this case was similar to the current ramp-up case. However, there were two modifications: (i) the time interval, (ii) the equilibrium scour depth. For the waves ramp-up flow, the quasi-steady flow was assumed to exist in at least one whole waves time period. Therefore, the time interval was set at one waves time period or several waves time periods. The equilibrium scour depth in each time step was calculated in terms of the $KC$ number using the empirical formula (Sumer and Fredsøe, 1990).

$$S_w = 0.1 \sqrt{KC}.$$  

(6-14)

The $KC$ number corresponded to the quasi-steady waves flow in each time step. Figure 6-6 (b) shows the measured scour depth development time history corresponding to the measured velocity in Figure 6-6 (a). The two predicted curves show a similar trend with the experimental data. Moreover, the predicted curve with $p = 0.6$ has a better fit with the measured data than the curve with $p = 1$.

For the combined waves and current ramp-up case, the time interval was also set as the wave time period or several wave time periods. The equilibrium scour depth in each time step was estimated by the empirical formulae proposed in the chapter 4

$$S_{wc} = (S_c^2 + S_w^2)^{0.5},$$  

(6-15)

here, $S_c$ represented the equilibrium scour depth corresponding to the current component at the level of pipeline centre in the combined waves and current flow conditions, which can be obtained by Eq.(6-13); meanwhile $S_w$ is the equilibrium scour depth corresponding to the waves component, which can be calculated by Eq.(6-14).

Figure 6-7 (a) shows the measured velocity for ramp-up combined waves and current case (OWCR-1) and the steady peak combined waves and current case (OWCR-2). The corresponding measured scour depth varying with time and the predicted curves based on the above method are shown in Figure 6-7 (b). Both predicted curves have a good agreement with experimental data during the ramp-up stage, however, they underestimate the final equilibrium scour depth. Comparing two predicted curves, the curve with $p = 0.6$ collapses slightly better than the curve with $p = 1$ with the experimental data.

Overall, the scour depth development time history has been predicted in the present study reasonable well. Moreover, the method with $p = 0.6$ indicates a better fit results
than the method with \( p = 1 \). It should be noted that only three experimental data were conducted to assess the feasibility of the above method. More experiments need to be done to assess the reliability of the predictive model.

### 6.4 Reducing flow conditions and backfill

The previous two sections demonstrated that the equilibrium scour depth changes with flow conditions. However if a scour hole generated by one flow conditions and the second flow has a lower intensity, the scour hole formed in the first step may be backfilled. This is because the decrease in the approach flow intensity causes the sediment transport capacity of the flow underneath the pipeline to drop. Consequently the sediment underneath the pipeline may be accumulated and the scour depth underneath the pipeline may become shallower. Such a transitional process is so-called the backfilling process.

For a substantial amount of backfill to develop, a certain amount of time must elapse. This time is called the time scale of backfilling process. To quantify this time scale following definition presented by Sumer et al. (2013) will be adopted in the present study:

\[
S(t) = S_{final} + (S_{initial} - S_{final}) \exp \left( -\frac{t}{T_{bf}} \right)^p, \tag{6-16}
\]

where \( T_{bf} \) is the time scale of the backfilling process, \( S_{initial} \) is the initial scour depth corresponding to the initial flow condition, and \( S_{final} \) is the final scour depth corresponding to the final flow condition. It is noted that \( p = 1 \) is adopted in the original form of the Eq. (6-16) in Sumer et al. (2013).

Two tests related to the backfilling process were conducted in the present study to observe the backfilling process and access the analysis method, these tests included Owc3-2 and Owc5-2, respectively.

The final equilibrium scour depth and the time scale of the backfilling process was obtained by curve fitting to the experimental data. Due to the limited test data, it is impossible to demonstrate the best fit coefficient \( p \) in the present study. Therefore, \( p = 1 \) and \( p = 0.6 \) were chosen as a reference values to predict the scour depth time history. Figure 6-8 shows the measured scour depth time history of the backfilling process for test Owc3-2 and Owc5-2, and corresponding initial scour process (Owc3 and Owc5). For the initial scour process (Owc3 and Owc5), the predicted curves with \( p = 1 \) and \( p = 0.6 \) were consistent with the curve fitting results in chapter 5. In the backfilling process, before the scour depth gradually decreased, there was a transitional period for the scour
profile around the pipeline to shift. The transitional period was longer in test Ow5-2 than test Ow3-2.

Figure 6-9 shows the bed profile change with time for the two cases. For test Ow3-2 (Figure 6-9 (a)) the initial scour profile was generated by a combined waves and current flow (test Ow3) and the scour hole was backfilled by a second weaker combined waves and current flow (test Ow3-2) in the same flow travel direction. The backfilling process started based on the previous bed profile. The bed profile at the upstream side of the pipeline roughly retained its morphology during the whole process, while the bed profile underneath the pipeline and at the downstream side of the pipeline gradually reduced in depth until the new equilibrium scour profile was reached. The whole backfilling process took more than 3 hours to accomplish. For completeness, final 3D scanned scour profiles for Ow3-2 are illustrated in Figure 6-10 (a) and (b).

For test Ow5-2 (Figure 6-9 (b)), the initial equilibrium scour profile was also generated by a combined waves and current flow, then, the second stage of scour process was under a steady current. Furthermore, the flow travel direction of the second stage steady current was in the opposite direction to the first stage. In the scour process of the second stage, the bed profile at the upstream and downstream sides of pipeline altered dramatically, one side raised and the other reduced. A new bed morphology corresponding to the opposite direction steady current was formed step by step. During the bed profile shift, the scour depth underneath the pipeline was unstable. However, after the bed profile shift was finished, the scour depth underneath the pipeline started to decrease steadily. The final 3D scour profile for Ow5-2 is plotted in Figure 6-10 (c). The 3D pattern sand ripples implied that the current flow is very weak, however, after sufficient time, the equilibrium scour profile with current flow inducted character is formed eventually.

Based on the present experiments it appears that the scour profile always transforms from the scour profile related to the initial flow conditions to the scour profile related to the final flow condition. This transformation happens simultaneous with the scour depth increase (continue scour process) or decrease (backfilling process), however, if the dramatic scour profile evolution needs to be achieved such as test Ow5-2, the scour profile transformation may take long time before the scour depth becomes stable.
6.5 Conclusion

In this chapter, the investigation of time scale of local scour below a pipeline is extended to consecutive flow and continues ramp-up flow conditions based on the present physical experiment. The following conclusions can be drawn:

1. Continued scour beneath a pipeline in the live-bed regime occurs when the flow climate changes from a relatively weak flow to a relatively strong flow. Furthermore, the final scour profile is determined by the final flow condition.

2. The scour depth development time history in continued scour processes can be predicted by accumulating the time and the equilibrium scour depth corresponding to different stage flow conditions.

3. The scour depth development predicted curve and the dimensionless time scale empirical formulae obtained in the previous chapters can be extended to multi-flow conditions and ramping up flow conditions. The empirical formulae proposed in the previous chapters for equilibrium scour depth and time scale appear to give good agreement with experiments.

4. Backfilling of a scour hole beneath a pipeline in the live-bed regime occurs when the flow climate changes from a relatively strong flow to a relatively weak flow. Furthermore, the final scour profile is determined by the final flow.

5. The time scale of the backfilling process is completely different from the scour process. The time scale of backfilling is much larger than the scour process in the present tests results.

6. There is a transformation period in both the continued scour process and the backfilling process. If the overall scour morphology around the pipeline changes due to the second flow (i.e., the width and depth change), the transformation period can take much more time to accomplished.
Chapter 6 Time scale in changing flow climate

Reference


Figure 6-1 Sketch of scour below pipeline. (a) Waves; (b) Steady current/combined waves and current.
Chapter 6 Time scale in changing flow climate

(a)  
\[ \text{Owc1} \]

(b)  
\[ \text{Owc2} \]

(c)  
\[ \text{Owc4} \]
Figure 6-2 Scour depth development with time: (a) Test Owc1 and Owc1-2; (b) Test Owc2 and Owc2-2; (c) Test Owc4 and Owc4-2; (d) Test Owc6 and Owc6-2.
Chapter 6 Time scale in changing flow climate

(a) Test Owc1-2

(b) Test Owc2-2

(c) Test Owc4-2
Chapter 6 Time scale in changing flow climate

Figure 6-3 Development of bed profile with time for continue scour process: (a) Test Owc1-2; (b) Test Owc2-2; (c) Test Owc4-2; (d) Test Owc6-2.

Figure 6-4 Final equilibrium scour profile for continuing scour process: (a) Owc1-2 upstream; (b) Owc1-2 downstream; (c) Owc4-2 upstream; (d) Owc4-2 downstream.
Figure 6-5 Ramp-up current velocity time series (Test OCR-1 and OCR-2): (a) velocity time histories; (b) scour depth development.
Figure 6-6 Ramp-up waves velocity time series (Test OWR-1 and OWR-2): (a) velocity time histories; (b) scour depth development.
Figure 6-7 Ramp-up combined waves and current velocity time series (Test OWCR-1 and OWCR-2): (a) velocity time histories; (b) scour depth development.
Figure 6-8 Flow climate change in two stages: (a) Test Owc3 and Owc3-2; (b) Test Owc5 and Owc5-2.
Figure 6-9 Development of bed profile with time for backfilling process: (a) Test Owc3-2; (b) Test Owc5-2.
Figure 6-10 Final equilibrium scour profile for backfilling process.
7. SCOUR AROUND TWO TANDEM PIPELINES

In this chapter, a series of physical experiments are reported which investigate local scour around two identical pipelines in a tandem arrangement. The effects of gap ratio \((G/D)\) on the scour depth and time scale under both live bed and clear water conditions is investigated. It is found that for small the gap ratio \((G/D)\), the interaction between two tandem pipelines is larger. Furthermore, the equilibrium scour depth of downstream pipeline is slightly larger than the upstream pipeline when \(0 < G/D < 3\); whilst the equilibrium scour depth of downstream pipeline is slightly smaller than the upstream pipeline when \(G/D \geq 3\). The time scale of scour below two tandem pipelines has also been studied for each individual pipe. It is found that the scour depth development time history predictive formula proposed in Chapter 5 gives a good fit with the measured data for two pipeline systems. The wall effect as well as the sand ripples effect are also discussed.

7.1 Introduction

With increasing complexity of subsea oil and gas systems, including producing wells, injection wells and processing systems, multiple pipelines are inevitably placed. Although many studies have been conducted to investigate scour below a single pipeline, the scour behaviour around two pipelines is not properly understood. Only a few studies have focused on the scour process below two pipelines. Jensen et al. (1990) found that vortex shedding from the pipelines occurs after scour reaches a certain depth. Zhao and Cheng (2008) studied scour below a piggyback pipeline (comprised of two pipelines of different diameters) in steady current and found that the existence of a small pipeline on top of the big one increases the scour depth.

For two close parallel pipelines the local scour process is affected by the spacing between two pipelines. J.H. Westerhorstmann et al. (1992) investigated scour below two and three-pipelines in tandem arrangements with 0.5 D and 1 D spacing under steady current and combined waves and current conditions. It was found that for two pipeline systems, spacing between pipelines of 0.5 D resulted in less scour depth than 1 D spacing, and less scour occurs under combined waves and current flow conditions than steady
current flow conditions. Harichandan and Roy (2012) performed numerical investigations for circular cylinders in tandem close to a wall at Reynolds number $Re = 100$ and $Re = 200$ for gap to pipe diameter ratio equal to 1 and 4. Rao et al. (2013) numerically studied the dynamics and stability of the flow past two cylinders sliding along a wall in a tandem configuration for Reynold numbers between 20 and 200, and gap to pipe diameter ratio from 0.1 to 10. Zhao et al. (2014) were the first to investigate local scour around two subsea pipelines in tandem numerically. The $k - \omega$ SST model was used to solve the flow past two pipelines and the conservation of sediment mass was used to solve for predicting the evolution of the sea bed profile. The numerical model was validated against the experimental data under both clear water and live bed scour conditions. Then, the effects of the gap between two pipelines on the scour were studied by extensive numerical simulations with the gap to diameter ratio between two pipelines ranging from 0.5 to 5.

The present experimental study focuses on understanding the scour behaviour of two tandem pipelines with different pipeline spacings. The effects of the gap ratio ($G/D$) on scour depth and scour time scale under both live bed and clear water conditions is investigated. The definition of gap ratio and scour depth are illustrated in Figure 7-1. In live bed current conditions, the gap ratios between two pipelines ($G/D$) ranges from 0 to 6, where $G$ is the gap between two pipelines. In clear water conditions, the gap ratios ($G/D$) range from 0.5 to 2. One wave conditions case was also conducted with $G/D = 3$.

### 7.2 Experiment setup

All experiments were conducted in the Large O-Tube facility. The model sediment utilized in this study is the same as that used in the previous LOT experiments, with the grain size of the sediment equal to $d_{50} = 0.24$ mm, with a geometric standard deviation of $\sigma_g = \sqrt{d_{94}/d_{16}} = 1.37$. The specific gravity of the sediment is 2.65. Two smooth PVC model pipelines with $D = 150$ mm were used in the tests. In order to fix two pipelines in the test section and to adjust $G/D$, each model pipeline was supported by a 10mm steel rod from each end of the model pipelines. The top part of each rob was threaded through the pipeline and ended at the top of the model pipe. The bottom part of each rod was inserted into the sediment and reached the bottom of the O-tube. At the bottom, each rob was fixed to a heavy plastic plate to stabilize it. Both pipelines were installed in the deep area of the pit section and the depth of the sediment below each pipeline was 400 mm (2.67 D). The velocity was measured at a point located 2 m...
upstream of the first model pipeline and 0.15 m above the initially flat seabed. After each scour test was finished, the water in the O-tube was drained out and the final scour profile was measured using the Laser scanner which has been described in Chapter 4. To record the time history of the scour profiles along the stream wise direction, a row of 1.5 mm diameter steel needles with gaps of 5 cm were positioned vertically in the sand parallel to the glass side walls of the O-tube. The needles are marked with scales to read scour depths during the tests. To ensure the clarity of the reading, the needles are located 30 cm from one glass side wall of the O-tube instead of at the centre. The evolution of the scour profile was read from these needles in the videos recorded from outside the glass walls. The differences between the bed profiles read from the needles and the profiles scanned by the laser profiler were found to be less than 4 mm (2.7% D) for all tests. The test conditions and results for the live bed, the clear water and the wave conditions are listed in Table 7-1, Table 7-2 and Table 7-3, respectively.
### Chapter 7 Scour around two tandem pipelines

#### Table 7-1 Test conditions and results of current experiments ($\theta > \theta_{cr}$).

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Undisturbed velocity near the bed $U_c$ m/s</th>
<th>Shields parameter in current alone $\theta_c$</th>
<th>Pipe Reynolds number based on $U_c$ $Re_c$</th>
<th>Pipe number</th>
<th>G/D</th>
<th>Fitting scour depth $S_f$</th>
<th>Fitting time scale $T_f$</th>
<th>Fitting scour depth $S_p$</th>
<th>Fitting time scale $T_p$</th>
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<td>CL1</td>
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<td>$\infty$</td>
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<td>0.51</td>
<td>0.10</td>
<td>75</td>
<td>0.05</td>
<td>87</td>
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<td>0.51</td>
<td>0.10</td>
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<td>367</td>
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<tr>
<td>CL4</td>
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<td>0.53</td>
<td>0.10</td>
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<td>327</td>
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<td>156</td>
<td>0.10</td>
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Note: The diameter of the model pipeline in LOT is 150mm, the $d_{50}$ of test sediment is 0.24 mm.

#### Table 7-2 Test conditions and results of current experiments ($\theta < \theta_{cr}$).

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Undisturbed velocity near the bed $U_c$ m/s</th>
<th>Shields parameter in current alone $\theta_c$</th>
<th>Pipe Reynolds number based on $U_c$ $Re_c$</th>
<th>Pipe number</th>
<th>G/D</th>
<th>Fitting scour depth $S_f$</th>
<th>Fitting time scale $T_f$</th>
<th>Fitting scour depth $S_p$</th>
<th>Fitting time scale $T_p$</th>
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<td>0.10</td>
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<td>9179</td>
<td>6.10</td>
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</tbody>
</table>

Note: The diameter of the model pipeline in LOT is 150mm, the $d_{50}$ of test sediment is 0.24 mm.
### Table 7-3 Test conditions and results of wave experiments ($\theta > \theta_{cr}$).

| Test No. | Time Period | Undisturbed velocity near the bed $U_c$ (m/s) | Shields parameter $\theta_w$ | Pipe Reynolds number $Re_w$ based on $U_c$ (m/s) | KC | Pipe number $G/D$ | Pipe number $G/D$ | Fitting scour depth $S_{f1}/D$ | Fitting time scale $T_{f1}$ (s) | Fitting scour depth $S_{f2}/D$ | Fitting time scale $T_{f2}$ (s) | Fitting scour depth $S_{p1}/D$ | Fitting time scale $T_{p1}$ (s) | Fitting scour depth $S_{p2}/D$ | Fitting time scale $T_{p2}$ (s) |
|----------|-------------|---------------------------------------------|-----------------------------|-------------------------------------------------|-----|-----------------|-----------------|-------------------------------|------------------|-------------------------------|------------------|-------------------------------|------------------|-------------------------------|------------------|-------------------------------|
| WL1      | 13.1        | 0.7                                         | 0.24                        | $1.1 \times 10^5$                               | 61  | 2               | 3               | 0.62                          | 0.69             | 0.71                          | 0.76             | 0.71                          | 0.76             | 0.71                          | 0.76             | 0.71                          |

Note: The diameter of the model pipe in LOT is 150 mm, the $d_{s0}$ of test sediment is 0.24 mm.
7.3 Scour development

For a single pipeline under live-bed conditions (see Figure 7-2 (a)) and (Mao, 1986; Sumer and Fredsøe, 2002; Sumer et al., 1988), the tunnel erosion initially throws sediment to the downstream side of the pipeline as the scour hole develops. The lee wake vortex generated at the downstream of the pipeline pushes accumulated sand further downstream. After a rapid increase in the initial scour depth, the scour depth below the pipeline fluctuates around a mean value due to the approach sand ripples. Moreover, due to the lee wake vortex at the rear of pipe, the maximum scour depth occurs at the downstream side of the pipeline when the equilibrium stage is reached.

For two tandem pipelines, the flow structure and its interaction with the mobile bed are more complex than that of a single pipeline. When the gap between two pipelines is large enough, the vortices from the upstream pipeline are convected downstream and interact with the rear pipe. The interaction of wake vortices affects the approach velocity of the downstream pipe.

7.3.1 Live-bed conditions

The experimental results for live bed conditions are considered firstly. Based on the observed scour process and the measured equilibrium scour profile around two tandem pipelines, the live-bed cases can be divided into three main categories.

1. For the $G/D = 0$, because two pipelines are placed next to each other, it can be seen from Figure 7-2 (b) that two tandem pipelines act as a single object and only one scour hole is formed underneath them during the whole scour process, which is wider and also deeper than at single pipe. Due to tunnel erosion underneath two joint pipelines, the sand bed between the centre of the two pipelines remains roughly flat.

2. For $G/D = 0.5$ and 1 (see Figure 7-2 (c) and (d)), different from that for $/D = 0$. During the initial stages of scour two scour holes are formed underneath each pipeline due to tunnel erosion, and the upstream one is larger than the downstream one. Following this, the sand berm between the two pipelines is gradually washed away and two individual scour holes ultimately merge into one large scour hole underneath the two tandem pipelines. The maximum scour depth is located at a central point between two pipelines. For $G/D = 0.5$ and 1, two tandem pipelines eventually act as a single object and the scour between the two pipelines is mainly governed by tunnel erosion.
3. For $G/D = 2 - 6$ (see Figure 7-2e, f, g, i), the initial stage of the scour process are similar to $G/D = 0.5$ and 1. For example, two scour holes are formed underneath the two pipelines because of tunnel erosion. Because the gap between pipelines and sand bed has reached a certain value, the vortex shedding behind each pipeline occurs. Therefore, the sand berm between the two scour holes moves downstream, and the height of the sand berm decreases slowly but still exist until the equilibrium stage. It is shown that with the increase of the gap ratio ($G/D$), the vortex shedding behind the upstream pipeline becomes weak before they reaches the sand bed underneath the downstream pipeline and two scour holes are eventually remained at the equilibrium stage. For $G/D = 2 - 6$, the scour between two tandem pipelines are firstly controlled by tunnel erosion, then governed by the lee wake erosion. In general, in this range of gap ratio, two tandem pipelines tend to act as two separate pipelines, considering the extreme conditions ($G/D \rightarrow \infty$), each pipeline will act as single pipeline. Likewise, local scour occurs around each individual pipe, global scour occurs as the general lowering of the sand bed between two pipelines. In order to verify whether two scour holes will finally merge into one, test CL7 was conducted for a relatively long time (3 to 5 times longer than rest tests). For this test, two scour holes developed fairly quickly, and the scour profile then remained the same for the duration of the test, only fluctuating due to the approached sand waves.

7.3.2 Clear water conditions

In clear-water conditions, the seabed shear stress induced by the approach flow velocity is so weak that the sand berm cannot been completely washed away even for a very small gap ratio of $G/D = 0.5$ (see Figure 7-3 (a)). However, the height of sand berm for $G/D = 0.5$ is lower than for $G/D = 2$. This indicates that the sediment entertainment capacity is still high for small gap ratios in clear water conditions.

7.3.3 Live bed waves condition

For live-bed conditions in waves, although only one test was conducted with $G/D = 2.9$ (see Figure 7-4), it can still be found that the scour profile remains symmetric and the sand berm still exists in the equilibrium stage.
7.4 Equilibrium scour depth

Figure 7-5 shows the variation of the equilibrium scour depth beneath each individual pipeline with the gap ratio for all current cases. For the live bed condition, the equilibrium scour depth underneath the upstream pipeline is always larger than that of a single pipe. The increase in the scour depth is mainly due to the effect of global scour. For \( G/D = 0.5, 1 \) and 2, the equilibrium scour depth underneath the downstream pipeline is slightly larger than that underneath the upstream pipe. Considering a single pipe, the maximum scour depth usually occurs at the downstream side of the pipeline centre. Since two tandem pipelines are acting as one bigger object, the maximum scour depth should occur close to the downstream pipe. For \( G/D = 2 \), the lee wake vortex of the upstream pipeline still retains its intensity before it reaches the downstream pipe, which becomes the main component of the approach flow for the downstream pipe. According to the measurement of near-bed velocity behind the single pipeline on a flat bed from Sumer et al. (1988), the maximum variation of r.m.s value of fluctuating component of near bed velocity occurs around 2.5D away from the pipeline centre, which just reaches the downstream pipeline in two tandem pipeline arrangement. Sumer et al. (1988) indicates that the undisturbed Shields parameter can easily be raised up to 3 to even 4 times momentarily. Therefore, it is understandable that the equilibrium scour depth underneath the downstream pipeline is larger than upstream of the pipeline. For \( 3 \leq G/D < 5.9 \), the equilibrium scour depth of the downstream pipeline is slightly smaller than the upstream pipe, which indicates that the gap ratio, the vortex shedding becomes relatively weak in the relatively long gap.

In the clear-water current conditions, for the \( 0 < G/D \leq 2 \), the equilibrium scour depth of upstream pipeline is larger than the downstream pipe.

In the live-bed wave conditions, due to the scour profile is asymmetry, the equilibrium scour depth of upstream pipeline and downstream pipeline are quite similar.

7.5 Time scale

To investigate the time scale of the scour process, Fredsøe et al. (1992) suggested that scour beneath a single pipeline can be well approximated by the formula:

\[
S(t) = S_{ef} \left( 1 - \exp \left( -\frac{t}{T_f} \right) \right),
\]

where \( S(t) \) is the time-dependent scour depth, \( t \) is time, \( S_{ef} \) is the equilibrium scour depth and \( T_f \) is the characteristic time-scale of the scour process.
A more general expression was suggested by Whitehouse (1998)

\[
S(t) = S_{ep} \left( 1 - \exp \left( - \left( \frac{t}{T_p} \right)^p \right) \right),
\]

(7-2)

where \( p \) is an empirical coefficient and \( T_p \) is the time scale associated with this exponent. Eq.(7-1) can be considered as a special case of Eq.(7-2) with \( p = 1.0 \). In chapter 5, the optimized value of \( p \) for a single pipeline is obtained based on the statistical analysis of the existing experimental data which was found to be \( p = 0.6 \). It is reasonable that an improved correlation with the experimental data for each pipeline in the two tandem pipelines arrangement may be achieved by utilising a similar fitting method.

A total of 11 physical experiments of scour depth development time history were collected for two tandem pipelines and then analysed to determine the value of \( p \) in Eq.(7-2). The data set includes the test results for live-bed steady current only, clear-water steady current only and waves only tests. The single pipeline test (CL1) was included in the data set to represent \( G/D \rightarrow \infty \). As in the investigation of single pipeline in Chapter 5, the least square method was used to determine the scour depth \( S \) and time scale \( T \) based on Eq.(7-2) by changing the fit coefficient \( p \) from 0.01 to 2. The correlation coefficient \( R^2 \) was used to evaluate the goodness of fit for different coefficient \( p \) in each case. For a specific value of \( p \), the R-square value for each case in the data set has been calculated. The mean R-square \( \overline{R^2} \) for all cases is plotted in Figure 7-6 (a) for the upstream pipeline and (b) for the downstream pipeline respectively. Therefore, the mean \( R^2 \) for all the tests in the data set corresponding to the fit coefficient \( p \) from 0.01 to 1.00 for the individual pipeline can be found in Figure 7-6 (a)(b). For the upstream pipe, the maximum mean R-square \( \overline{R^2}_{\text{max}} \) is found to be \( p = 0.55 \). Meanwhile, for the downstream pipe, the maximum mean R-square \( \overline{R^2}_{\text{max}} \) is found \( p = 0.68 \). This may indicate that scour happens faster for the upstream pipeline than the downstream pipeline.

Based on the statistical analysis, it is obvious that even for two tandem pipelines, the optimized fit coefficient \( p \) for each pipeline is fairly close to 0.6. The offset may come from the limited sample number in the data set. Therefore, the time development of scour for individual pipeline in two tandem pipelines arrangement can be defined by the following formula:

\[
S(t) = S_e \left( 1 - e^{-\left( \frac{t}{T_p} \right)^{0.6}} \right)
\]

(7-3)
Figure 7-7, Figure 7-8 and Figure 7-9 present a comparison of the time histories of the scour depths for upstream pipeline and downstream pipeline respectively predicted using Eq.(7-1) and Eq. (7-3) and the test data. Figure 7-7 is for live-bed current conditions, Figure 7-8 is for clear-water current conditions, and Figure 7-9 is for live-bed wave condition. The corresponding test conditions and the results for these three types of cases are summarized in Table 7-1, Table 7-2 and Table 7-3 respectively. It is observed that Eq.(7-1) and Eq.(7-3) are not only able to predict the scour depth development time histories for single pipe, but also provide a good prediction for individual pipeline in two tandem pipelines arrangement. Furthermore, it is also observed that the predictions with Eq.(7-3) generally agree with test data and are better than those with Eq.(7-1) in most of the cases. It should be noted that for most of cases, the scour processes underneath the upstream pipeline as well as the downstream pipeline does not happen simultaneously. The onset of scour process for the downstream pipeline usually occurs later than the upstream pipe. However, for the live-bed condition, such “delay” is relatively short (only 1 or 2 minutes), which can be easily observed in the clear water conditions (see Figure 7-8). It again indicates that the scour process of the downstream pipeline is influenced by the presence of the upstream pipe.

The dimensionless time scale based on Eq.(7-3) against the gap ratio \( \frac{G}{D} \) for all the cases is shown in Figure 7-10. Under live-bed steady current conditions the figure indicates that the downstream pipeline always takes longer time than the upstream pipeline to reach the equilibrium stage when the gap ratio is larger than nil. Moreover, the dimensionless time scale of the upstream pipeline frustrates around the dimensionless time scale of single pipeline (the dash line in the figure). It may be because the scour depth development for the upstream pipeline mainly depends on the steady approach flow. However, for the downstream pipe, the approach flow is affected by the upstream pipeline and the gap space between two pipelines.

### 7.6 Three dimensional final scour profile

In this study, the three dimensional scour profile around two tandem pipelines was measured by the handheld 3D scanner. The 3D contour figures based on the data are drawn in Figure 7-11 for live-bed conditions, Figure 7-12 for clear water conditions and Figure 7-13 for live-bed wave conditions respectively. From the 3D information, the morphology of global scour and local scour can be seen, wall effect, sand ripples and sandwaves can also been observed.
Chapter 7 Scour around two tandem pipelines

Firstly, under live bed flow conditions, compared to the scour profile of single pipe, the presence of two pipelines enlarges the area of sediment erosion in the vicinity of pipelines, which can be identified as the global scour.

Secondly, for the single pipeline (Figure 7-11 (a)), as the numerical simulation in chapter 4, an incomplete horseshoe vortex is expected to form at the junction between the pipeline and the side wall due to the separation of the boundary layer on the side wall, followed by the lee wake vortices at the lee side of the pipeline near the wall, which is similar with the flow around a vertical pile. The horseshoe vortex increases the scour hole near the side wall. Lee-wake vortices near the side wall also affect the scour profile along the side wall behind the pipe. A joint enlarged scour hole can be observed underneath the pipeline near the side wall when $G/D \leq 2$, meanwhile, when $G/D > 2$, the enhanced scour hole can be observed underneath each individual pipeline in the two tandem pipeline arrangement. Like the horseshoe vortex at the junction between the pipeline and the side wall, the lee-wake vortex not only affects the lee-side of the downstream pipe, but also the seabed between the two pipelines. Furthermore, from observation, the maximum effect area is about 150 mm from the wall for the 150 mm diameter model pipe. Meanwhile, the rest scour profile along the pipeline is relatively uniform. The scaled needle used to obtain the scour depth development was fixed underneath each pipe, 200 mm away from the side glass window of flume. For the clear-water cases, no much wall effect was observed during the tests.

Finally, as discussed in Chapter 4, for flows which exceed the threshold of motion, an initially flatbed may deform into various types of bed feature, ranging in size from small ripples up to major sandbanks (Soulsby, 1997). Since most of the tests were conducted under live-bed flow conditions, the sand ripples and sand waves (or sand dunes) appears in the vicinity of the pipelines as well as the whole sediment seabed. Figure 7-14 (a) is typical example of the three-dimensional vortex sand ripples formed at the upstream side of two tandem pipelines, and no evident is shown that such bed form is affected by the presence of pipelines. It should be noted that this bed was obtained after a relatively long period of time. At the start of the test, a two-dimensional rippled bed is formed, however, after two or three hours, the two-dimensional profile is gradually lost and three-dimensional effects become more evident and form an irregular, strongly three-dimensional scour pattern. When viewed from above, the size of such sand ripples is relatively small, normally 40 mm in height from crest to trough and 170 mm in length of the crests of two individual sand ripples. The sand ripples are washed away underneath
the pipelines due to the high velocity and only appear at the dune between two pipelines \((G/D \geq 2)\), and at the downstream side of two pipelines for all the live-bed steady current cases. Apart from the seabed surface scour pattern, the three-dimensional vortex sand ripples have no other effect on the scour profile around two tandem pipelines due to their small size. Meanwhile, in each live-bed steady current cases, the sand waves usually emerge after 1 hour of tests, and moves towards the flow direction, the size of the sand waves are relatively large, with 100 mm in height and 3-4 metres in length. In the Figure 7-11 (e), the head of a sand wave appears at the upstream side of two pipelines. When the sand waves pass through the vicinity of two pipelines, due to the superfluous sediment supply, the whole level of the scour profile around two tandem pipelines increases, however, the shape of the scour profile remains constant. After the crest of the sand waves pass through, the whole level of the scour profile not only drop to the previous level, but also drop further due to the limited sediment supply caused by the trough of the sand waves. Therefore, the whole level of the scour profile basically fluctuates around a mean level during the scour process.

### 7.7 Conclusion

1. The scour around two tandem pipelines is different from that around a single pipe.  
2. The smaller the gap ratio \((G/D)\), the larger the space effect between two tandem pipe. For the \(G/D = 0\), two tandem pipelines behaves as a single body; for the \(0 < G/D \leq 1\), the sand berm formed between two tandem pipelines disappears at the equilibrium stage; for the \(G/D > 1\), the sand berm always exists.  
3. For \(0 < G/D < 3\), the equilibrium scour depth of downstream pipeline is slightly larger than the upstream pipe; for \(G/D > 3\), the equilibrium scour depth of downstream pipeline is slightly smaller than the upstream pipe;  
4. For the time scale of two tandem pipelines, the prediction curves from the previous single pipeline investigations can be extended to two pipelines systems. Moreover, Eq.(7-3) has a better fit with the measured experimental data.  
5. From the observation of scour processes and the final 3D scour profile, it is found that the presence of sand ripples has no significant influence on the scour around two tandem pipelines, moreover, the sandwaves frustrate the entire sand level in the vicinity of two tandem pipeline around a mean level.
6. In the live-bed scour conditions, the wall effect in the tandem two pipelines system is enhanced compared with the single pipeline system.
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Reference


Figure 7-1 Sketch of scour below two pipelines.
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1) Test CL1

2) Test CL2

3) Test CL3

4) Test CL4
Figure 7-2 Scour profile change with time in steady current ($\theta > \theta_{cr}$).
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Figure 7-3 Scour profile change with time in steady current. ($\theta < \theta_{cr}$).

Figure 7-4 Scour profile change with time in waves (Test WL1).
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Figure 7-5 Variation of the equilibrium scour depth with the gap ratio \((G/D)\) for all cases.

Figure 7-6 Mean correlation coefficient with the value of \(p\) from 0.01 to 2: (a) upstream pipe; (b) downstream pipe.
Chapter 7 Scour around two tandem pipelines

(a) Test CL1

(b) Test CL2
Chapter 7 Scour around two tandem pipelines

(c) Test CL3

(d) Test CL4
Chapter 7 Scour around two tandem pipelines

(e) Test CL5

(f) Test CL6
Figure 7-7 Time development of scour depth for two pipelines tests under steady current ($\theta > \theta_{cr}$).
Figure 7-8 Time development of scour depths for two pipelines tests under steady current ($\theta < \theta_{cr}$).
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Figure 7-9 Time development of scour depths for two pipelines tests under wave (Test WL1).

Figure 7-10 Dimensionless plot of time scale against the gap ratio G/D for all existing tests.
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a) Test CL1

b) Test CL2

c) Test CL3
d) Test CL4

e) Test CL5
f) Test CL6
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g) Test CL7

Figure 7-11 3D final scour profile under steady current ($\theta > \theta_{cr}$).

i) Test CL8

Figure 7-12 3D final scour profile under steady current ($\theta < \theta_{cr}$): (a) Test CC1; (b) Test CC2.

Figure 7-13 3D final scour profile under waves (Test WL1).
Figure 7-14 Sand ripples in the vicinity of two tandem pipelines (Test CL7): (a) sand ripple at upstream sides of pipelines; (b) sand ripples at downstream side of pipelines.
8. CONCLUSION AND RECOMMENDATION FOR FUTURE STUDY

8.1 Conclusion

8.1.1 Onset of scour below subsea pipelines

The conclusions derived from the physical experiments are summarized as follows:

1. The onset of scour could happen earlier than the critical velocity defined by Sumer et al. (2001) if the upstream sediment supply is restricted.

2. If onset of scour happens due to limited sediment supply it will take a relatively longer time (compared with velocities above the critical velocity) for the onset of scour to finally occur.

3. When the onset of scour happens under a limited upstream sediment supply condition, the sand bed topography just before onset is quite different from the initial profile. This is due to luff and lee wake erosion. It should be noted that conclusions that both luff and lee wake erosion affect the potential for piping and tunnel scour is consistent with comments in Sumer et al. (2001), in which they pointed out that the presence of vortices in front of the pipeline and in the lee-wake may contribute to the onset of scour.

4. Observations from micro cameras within a transparent pipeline confirmed that the onset of scour occurs due to piping for the uneven seabed onset of scour experiments with limited sediment supply.

Based on the numerical simulation results, the following conclusions can be drawn:

1. The luff vortex still exists in the scour hole at the upstream side of pipeline prior to the onset of scour on an uneven seabed, and the lee wake vortex maintained the sediment slope at the downstream side of pipe.

2. The pressure difference between the upstream and downstream sides of pipeline on an uneven seabed is smaller than on a flat seabed with similar embedment, if the far field embedment is equal to the local embedment.
3. Even if the significant local scour alters the sea bed profile around a pipeline, the pressure gradient is not significantly different to that for a pipeline with equivalent embedment on a flat seabed. This is because the scoured profile leads to a reduced pressure gradient but a shorter seepage path.

8.1.2 Equilibrium scour depth underneath the pipeline in current/wave and combined wave and current

1. The previous empirical equilibrium scour depth formula in steady current is updated using the past 40 years available data and present test results. The mean flow velocity in the previous empirical formula is replaced with the undistributed flow velocity at the level of pipeline centre.

2. For equilibrium scour depth in waves, the previous empirical formula is proved to be valid for relatively large pipeline (196 mm), when the KC number is around 30 to 60.

3. A new method to predict the equilibrium scour depth in combined waves and current is proposed based on the current component equilibrium scour depth and the wave component equilibrium scour depth. Compared the new method with the available data, a general good agreement is obtained.

4. The equilibrium scour width measured in this study collapses quite well with the previous empirical results. Using the present experimental data development of an empirical formula for scour width under combined wave and current conditions may be possible in the future.

5. Two types of bed features have been investigated in the present experiments: (i) small 3D ripples in the free field and within the scour hole; (ii) 2D plane sand waves migrating through the scour hole. The presence of bed features was found to have limited influence on the average scour hole depth and width underneath the pipeline.

6. The wall effect in the laboratory test is investigated using final scour profile and numerical simulation, the wall effect area in this study is confirmed in the area of 1 D in length along the pipe.

8.1.3 Time scale of local scour below pipelines in current/wave and combined wave and current

The following conclusions can be drawn:
1. Among the three methods compared in this study, Eq.(5-6) and Eq.(5-13) appear to provide a better empirical fit for scour development beneath a fixed pipeline than previously quoted formula such as that given in Eq.(5-1) based on new and all available experimental measurements,

2. The non-dimensional time scale of the scour process below a pipeline in combined waves and current is governed by the maximum Shields parameter.

3. An universal equation can be written for the non-dimensional time scale in any combination of wave and current conditions. After comparison, the best data collapse is found when the time scale is obtained from Eq.(5-6). And the time scale in current only, waves only and combined waves and current flow conditions can be calculated based on Eq.(5-27).

8.1.4 Time scale of local scour below pipeline in changing flow climate

1. Continued scour around pipeline in the live-bed regime occurs when the flow climate changes from a relatively weak flow to a relatively strong flow. Furthermore, the final scour profile is determined by the final type of flow.

2. The scour depth development time history in continued scour process can be predicted by accumulating the time and the equilibrium scour depth corresponding to different flow conditions.

3. The scour depth development predicted curve and the dimensionless time scale empirical formulae obtained in the previous chapters can be extended to the multi-flow conditions and ramping up flow conditions, and which have a slightly better fit with the measured data than the previous formulae based on the present test results. The empirical formulae proposed in the previous chapter for equilibrium scour depth in current only and combined waves and current flow conditions are also applied in this study.

4. Backfilling of a scour hole around a pipeline in the live-bed regime occurs when the flow climate changes from a relatively strong flow to a relatively weak flow. Furthermore, the final scour profile is determined by the final type of flow.

5. The time scale of the backfilling process is completely different from the scour process. The time scale of backfilling is much larger than the scour process in the present tests results.

6. There is a transform period in both the continued scour process and the backfilling process. If the overall scour morphology around the pipeline changed due to the
Chapter 8 Conclusion and recommendation for future study

type of the final flow, the transform period takes much time to accomplished, otherwise, such period of time is very short.

8.1.5 Scour around two tandem pipelines

1. The scour around two tandem pipelines is different from that around a single pipe.
2. The smaller the gap ratio \((G/D)\), the larger the interaction between two tandem pipelines. For the \(G/D = 0\), two tandem pipelines behaves as a single body; for the \(0 < G/D \leq 1\), the sand berm formed between two tandem pipelines disappears at the equilibrium stage; for the \(G/D > 1\), the sand berm always exists.
3. About the equilibrium scour depths, for \(0 < G/D < 3\), the equilibrium scour depth of downstream pipeline is slightly larger than the upstream pipe; for \(G/D > 3\), the equilibrium scour depth of downstream pipeline is slightly smaller than the upstream pipe;
4. For the time scale of two tandem pipelines, the prediction curves from the previous single pipeline investigation can be extended to two pipelines systems. Moreover, \(S(t) = S_e(1 - e^{-\left(\frac{t}{T_p}\right)^{0.6}})\) has the best fit with the measured experimental data.
5. From the observation of scour process and the final 3D scour profile, it is found that the presence of sand ripples has no significant influence on the scour around two tandem pipelines, moreover, the sandwaves frustrate the global sand level around two tandem pipelines around a mean level.
6. In the live-bed scour conditions, the wall effect in the tandem two pipelines system is enhanced compared with the single pipeline system.

8.2 Recommendations for future research

1. Three dimensional flow structures and scour profiles were observed in the onset of scour experiments when the seabed profile adjacent to the pipeline changed significantly during the test. Since the secondary flows associated with this three dimensional flow structure had an effect on the scour profile near to the pipeline it is possible that they also had an effect on the onset of scour for embedded pipelines. Because of this we have referred to the results in this paper as ‘preliminary’ in the abstract. Better understanding of the three-dimensional flow structure around pipelines, particularly for small to medium values of flume width to pipe diameter, is currently the focus of future work.
Chapter 8 Conclusion and recommendation for future study

2. With the increase data of physical experiments in combined waves and current, an empirical formula for the equilibrium scour width may be gain in the future.

3. The present work is important in that it now gives, for the first time, a means to predict local pipeline scour in any combination of collinear and perpendicular wave and current conditions. The principle use of this model is that it can be used to update more complicated models of 3D scour processes along pipelines, such as that investigated by Cheng et al. (2014).

4. Scour around pipelines placed on silty calcareous sediment should be studied in the future in order to understand and quantify the mechanisms of scour in soil conditions relevant to the North West Shelf of Australia.

5. Local scour around tandem pipelines in waves only conditions and combined waves and current conditions can be investigated in the future based on the present physical experiments.

6. In the present study, the scour around pipeline is limited in the conditions of fixed pipe, the pipeline drop and following self-buried could be investigated in the future.
Chapter 8 Conclusion and recommendation for future study

Reference

Cheng, L., Yeow, K., Zang, Z. and Li, F., 2014. 3d scour below pipelines under waves and combined waves and currents. Coastal Engineering, 83(0): 137-149.