Lateral cyclic interaction between catenary riser and soft seabed

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Abstract

The cyclic movement of a riser at the touchdown zone (TDZ), which involves complicated pipe–soil interaction, is critical to its long-term safety. Most previous studies have focused on the pipe–soil interaction of the riser in the vertical plane at the TDZ. Therefore, we conducted laboratory tests to investigate the pipe–soil interaction during lateral cyclic pipe movements and the influence of seabed evolution around the TDZ on the following vertical cyclic pipe movements. At the TDZ, the bell mouth shape of the clay bed and ladle-like shape of the pipe were observed during lateral cycling. The movement trajectory of the pipe as well as the gradual penetration process was recorded, which may be mainly attributed to the lateral ploughing and soil softening around the pipe. The test dynamic embedment factor (ratio of the dynamic embedment and static embedment) was 1.7–2.7. The enlargement of the TDZ during cycling was measured, which was mainly related to the movement of the trench surface point. The increase in the undrained strength of the soil at the TDZ after dynamic tests proves that a crust of soil was formed beneath the pipe owing to consolidation during cyclic pipe–soil interaction.

Keywords: SCR, cyclic tests, 3D experiment, lateral interaction, clay seabed

1. Introduction

A steel catenary riser (SCR) is a long steel pipe with a typical diameter of 150–600 mm suspended between the seabed and a floating production system, such as a spar (Fig. 1). An SCR is suspended in a catenary curve towards the seabed. Owing to the flexibility of the mooring system, the spar moves around under various metocean conditions. The movement of the spar stimulated by wave and currents can cause dynamic movements of the riser, leading to dynamic pipe–soil interaction. This interaction significantly affects the response of the riser, such as the embedment (Oliphant and Yun [1], Randolph and White [2]) and stress amplitude distribution, especially at the touchdown zone (TDZ) (Shiri [3]). The as-laid embedment of an offshore pipeline is pivotal to its lateral and axial stability (Westgate *et al.* [4]) and the variation of stress is critical to its lifetime.

Most previous studies have focused on the pipe–soil interaction of the riser in the vertical plane at the TDZ; tests have been performed under 1g conditions (Fontaine *et al.* [5], Langford and Aubeny [6]) and geotechnical centrifuge conditions (Hodder *et al.* [7], Hu *et al.* [8]). However, the lateral movement is also significant in terms of the riser response. Several researchers including Orozco *et al.* [9] and Cheuk and White [10] have performed two-dimensional tests with a short rigid pipe to investigate the horizontal pipe–soil interaction mechanism. Cheuk and White [10] performed a series of centrifuge tests with a unit rigid pipe in kaolin clay and found that only a few cycles of small-amplitude horizontal oscillations could cause significant embedment.

Two-dimensional experimental results can be used to build or calibrate an interaction model, as in Westgate *et al.* [11] and Chatterjee *et al.* [12]. However, the three-dimensional behaviour of the SCR, especially at the TDZ, is of more significance. Wills and West [13] and Bridge *et al.* [14] conducted a full-scale pipe–soil interaction test at Watchet Harbor in the south west of England. This was the first experimental investigation of three-dimensional pipe–soil interaction and it had several limitations. In particular, the soil conditions were uncontrolled because the experiment was

conducted on a natural harbour. The trench formation process and development of pipe embedment could not be measured in real time. Hodder and Byrne [15] performed three-dimensional modelling in a laboratory to study the riser response at a TDZ on a sand seabed, and the results showed good agreement with the pipe–soil interaction model derived from traditional two-dimensional tests. In addition, Elliott *et al.* [16] [17] developed a novel experimental tool for SCR testing in a geotechnical centrifuge that could simulate the riser–fluid–seabed interaction mechanism.

To obtain further insights into the pipe–soil interaction mechanism, on the basis of the pioneering work of Wang *et al.* [18], which simulated the vertical cyclic interaction between an SCR and a clay seabed, we performed a large-scale lateral riser–soil interaction experiment by applying lateral movements of different amplitudes at one end of the model riser. Then, two vertical amplitudes were applied immediately following the lateral cycling. After the dynamic tests, the strength of the seabed soil at different positions along the pipeline was tested and compared with the corresponding intact strength profiles.

2. Experimental

2.1 Experimental system

The large-scale pipe–soil interaction test system comprising three subsystems: the test subject, loading system, and sensor system (Wang *et al.* [18]), was constructed at Zhejiang University, as shown in Fig. 2.

The test flume was 15 m in length, 1.2 m in width, and 1.5 m in depth (Fig. 3). The bottom of the flume was covered with a 100-mm-thick layer of gravel for drainage. A geotextile sheet was placed between the drainage layer and the upper soil layer to avoid the soil grains from running off. The average soil thickness after consolidation was about 640 mm, which is about 4D (D is the pipe outer diameter: 160 mm), which is sufficiently thick to ignore the boundary layer effects at the bottom. Similarly, the boundary effects at the sidewall might be neglected because of the 7.5D width of the flume sidewall. The water level in the flume was maintained at about 650 mm above the bed surface and the model pipe remained submerged throughout the static and cyclic tests.

The loading system was a computer-controlled bidirectional actuator that can separately apply corresponding displacement paths at the pipe end in the vertical and horizontal directions.

Fig. 4 shows the sensor system comprising strain gauges, pore pressure transducers, and displacement sensors. Ten groups of strain gauges (denoted as BM) were used to measure the bending moment, and four pore pressure transducers (denoted as PT) were used to record the excess pore water pressure at the pipe bottom. In addition, fifteen displacement sensors (denoted as DW) along the pipe were used to measure the pipe section trajectory. The detailed installation method of the strain gauges is described in Wang *et al.* [18]. According to El-Sherbiny [19], a coordinate system was established (Fig. 3) with O as the origin and P_0 as the initial location of the pipe in order to measure the pipe displacement. The locations of the pulleys (Points A and B) were fixed. The distance between the pipe location and the points A and B (initial values are l_1 and l_2) can be measured at the start of the test. When the pipe moves from P_0 to a new location P' , the distance between the pipe location and points A and B will be l_1' and l_2' (the differences between l_1' and l_1 and l_2' and l_2 can be determined through DW3 and DW5, respectively). Then, the coordinates of P' can be easily calculated. The same principle was also applied to the displacement sensors

group: DW4–DW8, DW6–DW10 and DW7–DW9. DW1 and DW2 were equipped to directly measure the horizontal and vertical displacements of the pipe end.

2.2 Model pipe

The model polyethylene pipe was 14 m long with an outer diameter of 160 mm and a wall thickness of 8 mm. The Young's modulus of the pipe material was 0.4 GPa, which was recalibrated after ~18 months storage since the last test (Wang *et al.* [18]). Iron bars were placed inside the pipe to increase its submerged weight. During the entire test, the pipe was filled with water and fully submerged in water with its two ends sealed; the submerged weight of the pipe was 223.7 N/m.

2.3 Soil properties

Zijiang clay was used in this experiment; the soil properties are listed in Table 1. The soil in the flume was prepared as in Wang *et al.* [18]. The fully remoulded soil was consolidated under its self-weight for about five months while the drain valves at the bottom of the flume were kept open. During consolidation, the water layer was maintained above the upper soil surface to keep the soil saturated all the time.

A set of T-bar tests were conducted before the pipe–soil interaction tests to investigate the soil strength and homogeneity. The T-bar used in this study was a steel cylinder bar of length 160 mm and diameter 25 mm with a steel rod of diameter 20 mm attached perpendicular to its end. The undrained shear strength profile of the soil was obtained with a T-bar bearing factor $N_{T\text{-bar}}$ of 10.5 according to Stewart & Randolph [20]. The penetration and extraction speed was about 1 mm/s, which approximated the undrained condition. Fig. 5 shows the positions of the T-bar penetrating locations; the positions are at a sufficient distance from the pipe to avoid influencing the results of the pipe–soil interaction tests.

The strength profiles at different T-bar locations (denoted as TPs) confirmed the general homogeneity of the soil (Fig. 6). Cyclic tests of the T-bar were conducted at two depths (1D and 2D) of the soil bed to survey the strength variation during soil remoulding. Fig. 7 shows a sharp reduction in $q_{T\text{bar}}/q_{T\text{bar},\text{ini}}$ after the initial cycle, followed by a more gradual reduction towards a stable value of approximately 0.5, which means that the soil sensitivity is approximately 2.

3. Static test

A static test that simulates the installation of the SCR was performed as a benchmark experiment after the T-bar tests. The installation process is very complex and it also includes lateral movement. This experiment investigated the influence of lateral loading on the response of the pipe at the TDZ. To focus on the main points without redundancies, the installation process was simplified by lowering the pipe slowly onto the seabed.

In this test, the pipe was connected to the actuator through a pinned joint at one end, and the longitudinal guiding bearings were relaxed. Therefore, the connecting end of the pipe was free to rotate in the plane of the pipe and to move axially. The pipe was initially suspended with its bottom just above the free surface of the bed with four chain rings at lifting points (Fig. 5) as well as the actuator at the end. Then, the displacement sensors and the pore pressure transducers were set at zero. The supporting platform was adjusted to a pipe bottom elevation of 250 mm above the bed surface at the actuator, and the pipe was carefully lowered onto the seabed by simultaneously pulling the hanging chains while the far end of the pipe was set freely on the bed.

1 The pipe was kept static on the seabed for about 18 h until the embedment became stable.

2 Fig. 8 shows the change in pipe embedment at DW7 ($x=4\text{m}$). The solid line is the development
3 of embedment in 18h and the dashed line describes the change in the initial 0.5h. The pipe quickly
4 penetrated the soil bed the moment it was lowered onto the soil surface, and the pipe embedment
5 increased to about 0.3D in the first 0.25 h. Then the pipe penetrated much slower. The embedment
6 grew by only 0.009D in the last 12 h of static test (from 6h to 18h). Therefore, it could be deemed
7 that the pipe penetration was stable.

8 Fig. 9 presents the variation in the excess pore water pressure at PT3 ($x=4\text{m}$). The excess pore
9 water pressure grew rapidly at first and reached its peak (1.5kPa) in a short time (about 40s), and it
10 began to drop quickly to 0.3kPa in about 1.6 h. Then, the excess pore water pressure gradually
11 decreased, and it completely dissipated to about zero at the end of the static test.

12 The development of bending moment at BM3 ($x= 3.5\text{m}$) is illustrated in Fig. 10. Its positive
13 value indicates the tension of the pipe invert line. The bending moment increased rapidly to
14 $\sim 90\text{Nm}$ in 80s and grew slowly to about 112Nm in 1.5 h. Then, it became steadier, equal to
15 $\sim 116\text{Nm}$, which indicated that the stress of the pipe was stable.

16 Numerous studies have used advanced numerical models to describe the global loading response
17 of a pipeline interacting with a seabed. In particular, Shiri [21] used the hysteretic model to
18 simulate the vertical interaction and demonstrated that the hysteretic model can capture the cycling
19 loading response of a pipeline at the TDZ and hence it can predict the SCR fatigue performance
20 well [21]. Yu and Konuk [22] performed large-deformation analyses using the arbitrary
21 Lagrangian-Eulerian method to simulate an as-laid pipe undergoing cycles of lateral displacement
22 on a 3D bed of soil. This model could successfully determine the complex behaviour of the pipe
23 during the lateral buckling process, which cannot be achieved by simple models. In addition,
24 Martin *et al.* [23] used the coupled Eulerian-Lagrangian methodology in Abaqus with a 3D
25 structural model of the pipe interacting with a series of 2D soil slices. They successfully modelled
26 the large plastic soil deformation, which is not available in a simple spring model, and found that a
27 2D continuum model is more efficient than a full 3D analysis.

28 However, only the static test results must be simulated and a simple linear spring numerical
29 model is sufficient here. This study used the Timoshenko beam element (element type B21) in
30 ABAQUS/Explicit to model the pipe, and the soil was modelled as a series of tension-free linear
31 elastic springs. The relationship between the static vertical load and pipe embedment is given
32 as follows (Randolph and White *et al.* [2], Merifield *et al.* [24] and Aubeny *et al.* [25]):

$$\frac{V}{D s_u} = a \left(\frac{w}{D} \right)^b + f_b \frac{A_s}{D^2} \frac{\gamma' D}{s_u} \quad (1)$$

34 where V is the vertical load (per unit length); D is the pipe diameter; s_u is the soil shear strength at
35 the pipe bottom; w is the pipe embedment (in reference to the initial soil surface); a and b are
36 fitted values of the power law, which depend on the relative roughness of the pipe–soil interface;
37 γ' is the effective soil weight; A_s is the nominal area of the pipe section penetrating below the
38 mudline; and f_b is a buoyancy factor. Kong [26] proposed an explicit prediction model to quantify
39 the buoyancy factor f_b accounting for the influence of soil unit weight and shear strength gradient.
40 According to Merifield *et al.* [24], f_b is taken as 1.5 for simplicity considering the soil heave around
41 the pipe. According to Westgate *et al.* [27], for a relatively smooth pipe–soil interface with
42 a combination of soil softening around the pipe and the down-drag of softer surface soils, a and

1 bare taken as 4.97 and 0.23 for $w/D \leq 0.5$, and 4.88 and 0.21 for $w/D > 0.5$, respectively (Aubeny *et al.* [25]). The variation of s_u within 1D is shown in Fig. 11, which was used in equation (1). The resulting V - w curve is presented in Fig. 12. The stiffness per unit length k in the finite element model was considered constant and evaluated as the slope of the V - w curve, which is 4.12 kPa. Fig. 13 compares the numerical simulation and experimental results as well as the results of Wang *et al.* [18], which show good agreement. Fig. 13(a) shows that the pipe embedment was slightly lesser than that of Wang *et al.* [18] in the TDZ owing to the higher strength of the prepared soil in the present test, which also caused a slight difference in the bending moments in Fig. 13(b).

9 4. Cyclic tests

10 4.1 Lateral cyclic tests

11 Lateral cyclic tests were performed immediately after the static test. Three amplitudes were consecutively applied to the end of the riser during lateral loading (Fig. 14), while vertical displacement was not allowed. The cycle period was 5s, and each loading phase lasted for 600 cycles.

12 The development of lateral and vertical displacements at $x=2m$ is shown in Fig. 15. It can be seen that when an input amplitude of 0.125D was applied, the lateral amplitude at $x=2m$ was only $\sim 0.02D$ in the first 20 cycles and it rapidly increased to 0.025D in 40 cycles. Then, the lateral amplitude gradually grew to $\sim 0.03D$ in 100 cycles and became stable until the end of phase 1. The static embedment was 0.05D at $x=2m$. When the lateral loading began, only 10 cycles with an input of 0.125D led to a sharp increase in embedment to $\sim 0.09D$. Then, the increase slowed down to an ultimate embedment of $\sim 0.26D$ at the end of phase 1. The increase in lateral amplitude and vertical displacement can be attributed to the progressive soil softening around the pipe. Fig. 15(c) shows the trajectory of the pipe section during lateral cycling. It can be seen that the pipe penetrated into the soil gradually as it oscillated in the lateral direction, which might be attributed to the lateral ploughing and soil softening.

13 Fig. 16 shows the variation in the vertical displacement and Fig. 17 illustrates the variation in the dynamic embedment factor during lateral loading along the pipe. The dynamic embedment factor, f_{dyn} , is defined as follows:

$$f_{dyn} = \frac{w_N}{w_{static}} \quad (2)$$

14 where w_N is the maximum embedment during one cycle after a certain number of cycles and w_{static} is the static embedment at the corresponding location along the pipe. f_{dyn} at $x=3m$ increased to ~ 1.5 in the first 200 cycles of phase 1 and increased gradually to ~ 1.7 at the end of phase 1. f_{dyn} grew slowly at $x=3m$ and $x=4m$ during the 0.125D input amplitude loading. In the following phases 2 and 3, f_{dyn} grew more obviously at $x=3m$ and $x=4m$ owing to the larger amplitude inputs. The dynamic embedment factor increased to 2.7, 2.1, and 1.7 at $x=3m$, 4m, and 5m, respectively, at the end of phase 3. It can be observed that the closer the position to the actuator, the greater the value of f_{dyn} . The embedment at $x=3m$ was much smaller than that at $x=4m$ before the cyclic test, but the embedment at the two positions were almost the same after 1800 cycles. However, different initial embedments occurred at different locations along the pipe. Therefore, the initial

1 embedment at different positions should not be excluded in the discussion of the dynamic
2 amplification effect when considering the riser rather than the pipe section.

3 Fig. 18 shows the side and plan views of the pipe shape profiles after certain numbers of lateral
4 cycles. The vertical displacement of the pipe invert illustrated here is the maximum value during
5 one cycle. A smooth curve is used to fit the shape of the riser in terms of the measured
6 displacement at discrete points along the pipe. The obvious ladle-like shapes can be seen in Figs.
7 18 (a), (c) and (e), which are consistent with the results of Wang *et al.* [18] as well as Bridge and
8 Howells [28]. Figs. 18(b), (d) and (f) show the bell mouth shapes, which are similar to
9 those observed by Bridge and Howells [28]. When the lateral motion just began, the pipe
10 immediately moved in the lateral direction with the bell mouth shape quickly formed and the bell
11 mouth shape opened wider at the moment the larger amplitudes were applied (phases 2 and 3) and
12 propagated forward. The bell mouth shape became wider rapidly at the beginning of every loading
13 phase and stabilized quickly.

14 When the pipe shape profile was almost stable under the 0.125D input amplitude loading
15 condition, the movement amplitudes of the pipe section were $\sim 0.09D$ ($x=1m$), $\sim 0.03D$ ($x=2m$),
16 $\sim 0.01D$ ($x=3m$) and $\sim 0.001D$ ($x=4m$), as shown in Fig. 18(b). The closer the pipe section was
17 to the actuator, the more significantly it was influenced. This consequence is similar to the results
18 of the vertical cyclic test (Wang *et al.* [18]). When the applied amplitude doubled in phase 2, the
19 pipe section at different positions moved at amplitudes of $\sim 0.18D$ ($x=1m$), $\sim 0.1D$ ($x=2m$),
20 $\sim 0.03D$ ($x=3m$) and $\sim 0.02D$ ($x=4m$). It can be seen that the motion amplitude of the pipe section
21 was related to the input amplitude and its distance from the actuator. The amplitude at $x=1m$ was
22 doubled just as the loading end did and it was nearly tripled at $x=2m$ and $x=3m$. At $x=4m$, the
23 motion amplitude increased much more because the pipe section here began to move obviously. In
24 phase 3, the corresponding amplitudes at $x=1-4m$ were $\sim 0.4D$, $\sim 0.32D$, $\sim 0.16D$, and $\sim 0.06D$,
25 respectively. From phases 2 to 3, when the end amplitude doubled, the amplitude in the following
26 positions along the pipe increased by more than one time.

27 The embedment of the pipe section at $x=4m$ increased by 0.07D, 0.17D, and 0.18D during
28 phase 1, phase 2 and phase 3 and the corresponding stable value of lateral movement amplitudes
29 were 0.001D, 0.02D, and 0.06D, respectively. Comparing the relationship of motion amplitude
30 and development of embedment in phases 2 and 3 showed that the tripled amplitude (from 0.02D
31 to 0.06D) only led to a similar embedment growth (0.18D, nearly equal to 0.17D). This indicated
32 that the increased soil strength along the depth produced larger resistance of soil, which
33 precluded the pipe from penetrating much deeper as expected according to the increased amplitude.
34 However, it can also be concluded that even small lateral movements could indeed multiply the
35 pipe embedment.

36 Fig. 19 illustrates the pipe profile at the TDZ (here the penetration of the pipe invert is the
37 maximum value in one cycle), and Fig. 20 shows the offset of the trench surface point (TSP),
38 trench bottom point (TBP) and touchdown point (TDP) during lateral cycling. The definitions of
39 TSP, TBP, and TDP are similar to those in Shiri [29]. TDP is where the riser reaches the nominal
40 level of the seabed, TBP is where the trench reaches its maximum depth, and TSP indicates where
41 the SCR reaches the zero gradient towards the far end of the riser. TDZ is defined as the region
42 between TDP and TSP. In Fig. 20, L_{z-surf} stands for the length of the TDZ, and L_{z-max} is the
43 distance between TDP and TBP. Fig. 21 shows the development of L_{z-surf} and the variation in the
44 ratio of L_{z-surf} and L_{z-max} , denoted as r .

The development of the TDZ range can be described by the change in the locations of TDP, TBP and TSP. In phase 1, the TDP moved rapidly from $x = 1.86\text{m}$ to $x = 1.60\text{m}$ towards the actuator in the first 200 cycles and then shifted much more slowly (Fig. 19). The lateral cycling increased embedment. The TSP kept moving towards the far end of the riser, which was also indicated in Shiri [3] who simulated the evolution of TSP under a cyclic vertical input on top of the riser. Its shifting rate decreased during each loading phase (Fig. 20), which was exactly the same as that of TDP. The combined movements of TDP and TSP led to the extension of the TDZ range. The range of the TDZ was $\sim 6.6\text{m}$ at the beginning of the cycling and increased by $\sim 1\text{m}$, $\sim 2.1\text{m}$, and 1m after phases 1, 2 and 3, respectively. In the process of TDZ extension, the contribution of the movement of TSP accounts for 70%, 92.4% and 88.4% during phases 1–3, respectively, while the effect of TDP was slight. It can be concluded that the development of TDZ was mainly reflected in the shift of TSP with development of the embedment.

The shape of the TDZ is mainly related to the relationships among the locations of TDP, TBP and TSP. During phase 1, although the shifting rate decreased as the cycling continued, the TBP was always moving towards the actuator, which is similar to the results of Nakhaee [30] and Shiri ([3] [29]) who simulated the effects of the vertical input. Actually, the location where the embedment was the largest was almost invariable during entire phase 2 and shifted slightly away from the actuator during phase 3. The ratio between $L_{z\text{-surf}}$ and $L_{z\text{-max}}$, r , which showed the relative change among the locations of TDP, TBP, and TSP, indicated the trench shape profile to some extent. Here, r can be considered a shape parameter of the trench profile. The value of r was ~ 2.7 and then increased to ~ 4.7 after phase 1, as shown in Fig. 21. As the embedment of the pipe developed during phases 2 and 3, the value of r fluctuated and became stable at about 4.8, which is close to that deduced from a quadratic exponential formula (Shiri [29]).

The development of excess pore water pressure during lateral testing at different positions is shown in Fig. 22. Figs. 22 (a) and (b) show that the excess pore water pressure oscillated about the zero line with certain steady amplitudes at $x = 2\text{m}$ and $x = 3\text{m}$ in phases 1 and 2 with no distinct accumulation. In phase 3, an accumulation of excess pore water pressure occurred at $x = 2\text{m}$ and $x = 3\text{m}$. Positive and negative pore pressures occurred alternately around the accumulation value. Fig. 22(c) shows the excess pore water pressure at $x = 4\text{m}$ during phases 1–3, which indicated that the excess pore water pressure at a farther location from the actuator was not influenced much by the applied loading. The generation of positive or negative excess pore water pressure implies that the soil was under cyclic loading as the pipe moved back and forth in the lateral direction. When the applied amplitude was small (phase 1, phase 2), although the soil around the pipe was disturbed, the excess pore water pressure dissipated instantly with cycling. However, when the amplitude was larger (phase 3), excess pore water pressure accumulated, which decreased the effective stress inside the soil. Therefore, the accumulation of excess pore water pressure at $x = 2\text{m}$ and $x = 3\text{m}$ demonstrated soil softening around the pipe.

The internal stress of SCR is the combination of bending, axial tension, and torsion. Bending plays an important role among these actions. Fig. 23 shows the development of bending moment during lateral cycling at different locations. It can be seen that the variation of moment in the vertical plane was very small as the pipe moved back and forth in the lateral direction. The bending moments at $x = 1.5\text{m}$, $x = 2.5\text{m}$, and $x = 3.5\text{m}$ were growing slightly as the cycles went through phases 1–3. The fact that the decreasing trend of the bending moment at $x = 4.5\text{m}$ transformed to gradual growth was related to the variation in the pipe shape profile around this position, where the

change in curvature occurred.

4.2 Vertical cyclic tests with disturbed seabed

Pipe-soil interaction is actually very complex as it is a combination of both horizontal and vertical movements. Yuan *et al.* [31] demonstrated the importance of horizontal movements for pipe embedment when horizontal and vertical movements occur simultaneously. In this test, we attempted to determine the influence of horizontal cycles on the consequential vertical ones. The vertical cyclic tests started 90s after the end of the lateral tests. Two amplitudes were consecutively applied at the end of the riser and each amplitude lasted for 600 cycles, which are called phases 4 and 5. The displacement input curves are shown in Fig. 24. When the vertical motion was applied, the lateral position of the loading end was fixed.

Fig. 25 presents the development of the pipe profile at the equilibrium position after certain numbers of cycles during phases 4 and 5, which shows that there was nearly no increase in the embedment along the pipe. Fig. 26 illustrates the variation in excess pore water pressure during the vertical cycling. The initial values of accumulation at $x = 2\text{m}$ and $x = 3\text{m}$ were $\sim 0.1\text{ kPa}$ and 0.2 kPa , respectively, which were much smaller than the values immediately after the end of the lateral tests as the excess pore water pressure dissipated before the vertical cyclic tests. The dissipation continued during the vertical cycling and the accumulation of excess pore water pressure gradually decreased to zero. As shown in Fig. 27, the upper soil in the TDZ was pushed aside by the lateral motion of the riser, which created shorter paths for the dissipation of pore water in the shallow layer soil just beneath the pipe. During the vertical pipe-soil interaction, the soil at the bottom of the pipe was drained quickly and the excess pore water pressure decreased.

Fig. 28 shows the variation in the bending moments during the vertical tests. As the pipe moved up and down, the difference between the maximum and minimum bending moment was almost constant in one cycle, and the values at equilibrium were also stable. Fig. 29 gives the equilibrium distribution of the bending moment along the riser during the vertical tests, which shows that the bending moments along the length of the pipe did not change with the vertical cycling.

5. Soil strength tests after cycling

Two sets of ball penetration tests were conducted to investigate the soil strength after the pipe-soil interaction tests. The ball penetrometer was a steel ball of diameter 50mm with a 25mm diameter steel rod attached perpendicular to the end of the bar (Randolph and White *et al.* [32]). The ball bearing factor N_{Ball} was taken as 11.5 (Low and Randolph [33]). Fig. 30 shows the positions of the two groups of ball penetrating points. BP1–3 lie on three different positions (1.5m, 3.5m and 5.5m, respectively, from the actuator along the pipe) just on the centreline of the soil bed (the centreline of the trench), and BP4–6 are 0.4m from the centreline. To eliminate the influence of the previous T-bar test (section 2.3), the centres of BP4–6 were 150mm from the centres of the corresponding initial TPs (Fig. 30). However, in the following discussion, the results of BP4–6 are considered equivalent to those at the same positions as TP1–3. Fig. 31 compares the results of the soil strength after the cyclic tests with the initial strength profiles (TP1–3, shown in Fig. 6). Zero depth is defined as the elevation of the soil surface before the static test. It can be seen that the soil strength profiles of BP4–6 were very close to the initial strength curves (TP1–3), which indicates that the soil near the sidewall of the flume was intact. However, for BP1–3, the strength

profiles were different. The sharp increase in strength in the shallow soil surface of the trench bottom implies that a crust was formed beneath the pipe after lateral cycling and vertical cycling. The crust formation can be attributed to phenomena. One is the compaction of the pipe on the soil under its dead weight during lateral cycling and the other is the punning of the pipe during vertical cycling. The lateral motion swept the surface soil away and created shorter drainage paths; therefore, the soil could consolidate under the compaction and punning. Both phenomena can lead to soil consolidation, resulting in an increase in strength. The depth of the crusts at different positions could be up to $\sim 1.3D$, $\sim 1.1D$, and $\sim 1.3D$ at $x = 1.5\text{m}$, $x = 3.5\text{m}$, and $x = 5.5\text{m}$, respectively. The strength increment decreased with increasing depth, even forming an ‘inflection point’, which finally coincided with the initial strength profile (Fig. 31).

6. Conclusions

This study conducted a two-stage pipe–soil interaction experiment, which included lateral cyclic tests followed by a series of vertical cyclic tests. The pipe embedment development and the pipe shape profile were observed during the cyclic interaction. The development of TDZ caused by lateral interaction was measured, including its depth and length, which was related to the locations of TDP, TBP, and TSP. The soil strength after the vertical cyclic tests was also measured. The following conclusions were drawn from the experiment.

- (1) The 3D experiment can simulate the TDZ of an actual SCR interacting with a soft seabed. It can show the response at different locations along the pipe, including pipe embedment, pore pressure, trench profile, and development of the TDZ, which cannot be obtained in 2D tests.
- (2) A few cycles of small-amplitude lateral oscillations can multiply the pipe embedment and the test dynamic embedment factor can be 1.7–2.7 under an input displacement of $0.125D$ to $0.5D$. The penetration as the pipe oscillated in the lateral direction is attributed to the lateral ploughing and softening of the soil around the pipe. The phenomenon that the lateral movement amplitude initially increases during each lateral loading phase is attributed to the lateral sweeping motion of the pipe, which shows the bell mouth shape.
- (3) The development of TDZ is a combination of the movements of TDP and TSP. TDP kept moving towards the actuator and TSP was always found to move towards the far end of the pipe during lateral cycling. The extension of TDZ was mainly attributed to the movement of TSP as the embedment develops. The TBP of SCR moved towards the actuator first, but it was observed to move towards the far end as the embedment developed (in phases 2 and 3). The trench shape profile at TDZ tended to be stable as the pipe embedment grew, and the shape parameter $r = L_{z\text{-surf}}/L_{z\text{-max}}$ was stable at ~ 4.8 .
- (4) A soil crust beneath the pipe was found and it was attributed to the compaction of pipe under its dead weight during lateral interaction and vertical punning. The depth of crusts can be up to $\sim 1.3D$. The lateral motion swept the surface soil away and created shorter drainage paths so that the soil consolidated under the compaction during lateral cycling and punning during vertical interaction.

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Reference

- [1] J. Oliphant&G.J. Yun,Pipeline embedment prediction using as-laid data, ASME 2011 30th International Conference on Ocean, Offshore and Arctic Engineering. (2011) 839-845.
- [2] M.F. Randolph&D.J. White, Pipeline embedment in deep water: processes and quantitative assessment. In: Proc. Offshore Technology Conf., Houston, OTC 19128,(2008).
- [3] H. Shiri, Response of steel catenary risers on hysteretic non-linear seabed, Applied Ocean Research, 44(2014) 20-28.
- [4] Z.J. Westgate, M.F. Randolph, D.J. White & S. Li,The influence of sea state on as-laid pipeline embedment: a case study, Applied Ocean Research, 32(2010) 321-331.
- [5] E. Fontaine, J.F. Nauroy. P. Foray, A. Roux & H. Gueveneux, Pipe-soil interaction in soft kaolinite: vertical stiffness and damping. In: Proc. 14thInt. Offshore and Polar Engineering Conf., ISOPE, (2004).
- [6] T. Langford & C.P. Aubeny, Model tests for steel catenary riser in marine clay, Proc. Offshore Technology Conf., OTC 19495, (2008).
- [7] M.S. Hodder, D.J. White &M.J. Cassidy, Centrifuge modelling of riser–soil stiffness degradation in the touchdown zone of a steel catenary riser, Proc. of the ASME 27th Int. Conf. on Offshore Mechanics and Arctic Engineering, OMAE 57302, (2008).
- [8] H.J.E. Hu, C.F. Leung, Y.K. Chow &A.C. Palmer, Centrifuge modeling of SCR vertical motion at touchdown zone, Ocean Engineering, 38(2011) 888-899.
- [9] M. Orozco, P. Foray&J.F. Nauroy, Pipe-soil horizontal dynamic stiffness in soft soils. In: Proc. 17thInt. Offshore and Polar Engineering Conf., ISOPE,(2007).
- [10] C.Y. Cheuk&D.J. White, Modelling the dynamic embedment of seabed pipelines, Géotechnique, 61(2011) 39–57.
- [11] Z.J. Westgate, D.J. White& M.F. Randolph, Modelling the embedment process during offshore pipe-laying on fine-grained soils, Canadian Geotechnical Journal, 50(2012) 15-27.
- [12] S. Chatterjee, D.J. White& M.F. Randolph, Coupled consolidation analysis of pipe–soil interactions, Canadian Geotechnical Journal, 50(2013) 609-619.
- [13] N.R.T. Wills &P.T.J. West, Interaction between deepwater catenary risers and a soft seabed: Large scale sea trials, Proc. Offshore Technology Conf., OTC 13113, (2001).
- [14] C.D. Bridge, H.A. Howells, N. Toy, G.A.R. Parke& R. Woods. Full-scale model tests of a steel catenary riser. In: International Conference on Fluid Structure Interaction,(2003).
- [15] M.S. Hodder&B.W. Byrne.3D experiments investigating the interaction of a model SCR with the seabed, Applied Ocean Research, 32(2010) 146-157.
- [16] B.J. Elliott, A. Zakeri, A. Macneill, R. Phillips, EC Clukey& G. Li,Centrifuge modeling of steel catenary risers at touchdown zone part I: Development of novel centrifuge experimental apparatus, Ocean Engineering, 60(2013) 200-207.
- [17] B.J. Elliott, A. Zakeri, J. Barrett, B. Hawlader, G. Li & EC Clukey, Centrifuge modeling of steel catenary risers at touchdown zone part II: Assessment of centrifuge test results using kaolin clay, Ocean Engineering, 60(2013) 208-218.
- [18] L.Z. Wang, J. Zhang, F. Yuan&K. Li, Interaction between catenary riser and soft seabed: large-scale indoor

- tests, *Applied Ocean Research*, 45(2014) 10-21.
- [19] R.M. El-Sherbiny, Performance of suction anchor anchors in normally consolidated Clay, Ph.D.Dissertation, The University of Texas at Austin, (2005).
- [20] D.P. Stewart & M.F. Randolph, A new site investigation tool for the centrifuge, *Proc. Int. Conf. Centrifuge*, the Netherlands, (1991) 531-538.
- [21] H. Shiri, Influence of seabed response on fatigue performance of steel catenary risers in touchdown zone, PhD thesis. The University of Western Australia, (2010).
- [22] S. Yu & I. Konuk, Continuum FE modelling of lateral buckling. In: *Offshore Technology Conference*, Houston, (2007).
- [23] C. M. Martin, D. Kong & B. W. Byrne, 3D analysis of transverse pipe-soil interaction using 2D soil slices, *Géotechnique Letters*, 3(2013) 119–123.
- [24] R.S. Merifield, D.J. White & M.F. Randolph, Effect of surface heave on response of partially embedded pipelines on clay, *ASCE Journal of Geotechnical and Geoenvironmental Engineering* (2009) 819-829.
- [25] C.P. Aubeny, H. Shi & J.D. Murff, Collapse loads for a cylinder embedded in trench in cohesive soil, *International Journal of Geomechanics*, ASCE 5(2005) 320-5.
- [26] D. Kong, Large displacement numerical analysis of offshore pipe-soil interaction on clay, DPhil thesis, University of Oxford, (2015).
- [27] Z.J. Westgate, D.J. White & M.F. Randolph, Field observations of as-laid pipeline embedment in carbonate sediments, *Géotechnique*, 62(2012) 787-798.
- [28] C.D. Bridge & H.A. Howells, Observations and modeling of steel catenary riser trenches, *The Seventeenth International Offshore and Polar Engineering Conference, ISOPE*, (2007).
- [29] H. Shiri, Influence of seabed trench formation on fatigue performance of steel catenary risers in touchdown zone, *Marine Structures*, 36(2014) 1-20.
- [30] A. Nakhaee & J. Zhang, Trenching effects on dynamic behavior of a steel catenary riser, *Ocean Engineering*, 37(2010) 277-288.
- [31] F. Yuan, D. J. White & C. D. O'Loughlin, The evolution of seabed stiffness during cyclic movement in a riser touchdown zone on soft clay. (2016) 1-11.
- [32] M.F. Randolph, D.J. White, N. Boukpeti & H.E. Low, Strength of fine-grained soils at the solid–fluid transition, *Géotechnique*, 62(2012) 213-226.
- [33] H.E. Low & M.F. Randolph. Strength measurement for near-seabed surface soft soil using manually operated miniature full-flow penetrometer, *Journal of Geotechnical and Geoenvironmental engineering*, 136(2010) 1565-1573.

1

2

Table 1 sSoil index properties

Property	Value
Plastic limit (%)	25
Liquid limit (%)	48
Plastic index (%)	23
Specific gravity	2.72
Average effective unit weight (kN/m ³)	5.27
Clay-size content (<0.005 mm) (%)	35
permeability coefficient (m/s)	4.6×10^{-9}

3

4

Fig. 1 Schematic of typical SCR configuration

Fig. 10 Typical variation of bending moment during the static test (BM3)

Fig. 11 Undrained shear strength profiles before cyclic tests within 1D

Fig. 12 Load-embedment curve within 1D embedment

Fig. 13 Comparison of ABAQUS and static test: (a) shape profile and (b) bending moment

Fig. 14 Displacement amplitude curves

(a)

(b)

(c)

Fig. 15 Variation of displacement during lateral cycling ($x=2m$)

Fig. 16 Variation of vertical displacement at different locations

Fig. 17 Variation of dynamic embedment factor during lateral tests

(a)

(b)

(c)

(d)

(e)

(f)

Fig. 18 Pipe shape profiles during lateral tests: (a) (c) (e) for side view and (b) (d) (f) for plan view

Fig. 19 Pipe shape profile at the TDZ

Fig. 2 Overall view of the large scale 3D soil/structure interaction test system

Fig. 20 Variation of locations of TSP, TBP and TDP

Fig. 21 Development of TDZ during lateral cycling

(a)

(b)

(c)

Fig. 22 Development of excess pore water pressure during lateral cycling

Fig. 23 Variations of bending moments during lateral cycling

Fig. 24 Displacement amplitude curves

Fig. 25 Pipe shape profiles at equilibrium position during vertical tests

(a)

(b)

Fig. 26 Variation of excess pore water pressure during vertical cycling

Fig. 27 Touchdown zone after vertical tests

Fig. 28 Variations of bending moments during vertical cycling

Fig. 29 Bending moment profiles during the vertical testing at equilibrium position

Fig. 3 The Schematic diagram of flume cross section

Fig. 30 Positions of Ball points

(a)

(b)

(c)

Fig. 31 Undrained shear strength profiles after cyclic tests

Fig. 4 Arrangement of transducers along the pipe

Fig. 5 Positions of T-bar points

Fig. 6 Undrained shear strength profiles before testing

Fig. 7 T-bar penetration resistance degradation with cycling

Fig. 8 Typical variation of embedment during the static test (DW7)

Fig. 9 Typical variation of excess pore water pressure with embedment during the static test (PT3)