

DISCUSSION / DISCUSSION

Reply to the discussion by McCarron on “Large-scale modelling of soil–pipe interaction during large amplitude cyclic movements of partially embedded pipelines”¹

C.Y. Cheuk, D.J. White, and M.D. Bolton

The authors are grateful for the discussor’s interest in our work. The discussion raises a number of important points that we are able to clarify and expand on and highlights certain areas of pipe–soil interaction that we agree remain challenging sources of uncertainty. Some of the points raised in the discussion have been the focus of the authors’ more recent work. These recent findings have been published elsewhere, and references are given in this reply.

Dimensionless group $G = s_u/D\gamma'$

The discussor questioned the use of the dimensionless group $G = s_u/D\gamma'$ within expressions for pipe embedment and breakout resistance. Dimensional analysis presented by Verley and Lund (1995) suggests that the lateral breakout resistance of a partially embedded pipeline is governed by the following dimensionless groups:

$$[1] \quad \frac{F_h}{Ds_u} = f\left(\frac{W'_p}{Ds_u}, \frac{s_u}{D\gamma'}, \frac{z}{D}, \frac{a}{D}\right)$$

where a is the amplitude of small oscillations imposed on the pipe during installation (“dynamic lay effects”), and the others symbols are as defined in our original paper.

The expressions of Verley and Lund (1995) for embedment and breakout resistance, given as eqs. [1] and [3] in our original paper, are commonly used in practice (Det Norske Veritas 2007) and thus formed the initial focus of our back-analysis. The empirical parameters required to weight the various dimensionless groups within these expressions were calibrated using a statistical analysis of the available data (Verley and Lund 1995). The authors agree with the discussor that it is preferable to establish a theoretical basis for these expressions, which we aimed to do with our simple upper-bound solution that can then be tested

against available data and modified in a manner guided by the observed behaviour (rather than on a wholly statistical basis).

Of the dimensionless parameters in eq. [1] herein, a/D is not relevant to our paper because dynamic lay effects have been ignored. However, it is recognized that this parameter significantly affects the as-laid embedment of seabed pipelines and thus strongly influences the breakout resistance. Recent centrifuge modelling has explored this behaviour (Cheuk and White 2008).

Of the remaining parameters, the quantity z/D is a function of W'_p/Ds_u . Therefore, the two dimensionless groups governing the breakout behaviour are W'_p/Ds_u and $s_u/D\gamma'$. Since s_u appears in both groups, its contribution to the overall behaviour is not in proportion to that of γ' . Therefore, the two parameters do not have “equal importance” in the solutions of Verley and Lund (1995), as the discussor suggested.

However, the authors agree with the discussor that the influence of soil weight (and hence the dimensionless group $G = s_u/D\gamma'$) is minimal at breakout. We have recently used particle image velocimetry (PIV) coupled with close-range photogrammetry (White et al. 2003) to analyze the images of pipe breakout captured in a centrifuge model test. This has allowed the deformation mechanism during breakout to be identified (Dingle et al. 2008). The results illustrate that the peak resistance during breakout coincides with tensile failure at the rear of the pipe, which is consistent with the pore-water pressure measurements presented in our original paper. The two-sided symmetry of the failure mechanism mobilized immediately prior to this loss of tension (Fig. 1a) implies that there is minimal change in the potential energy of the soil. Breakout resistance is therefore not sensitive to soil weight. The one-sided mechanism present immediately

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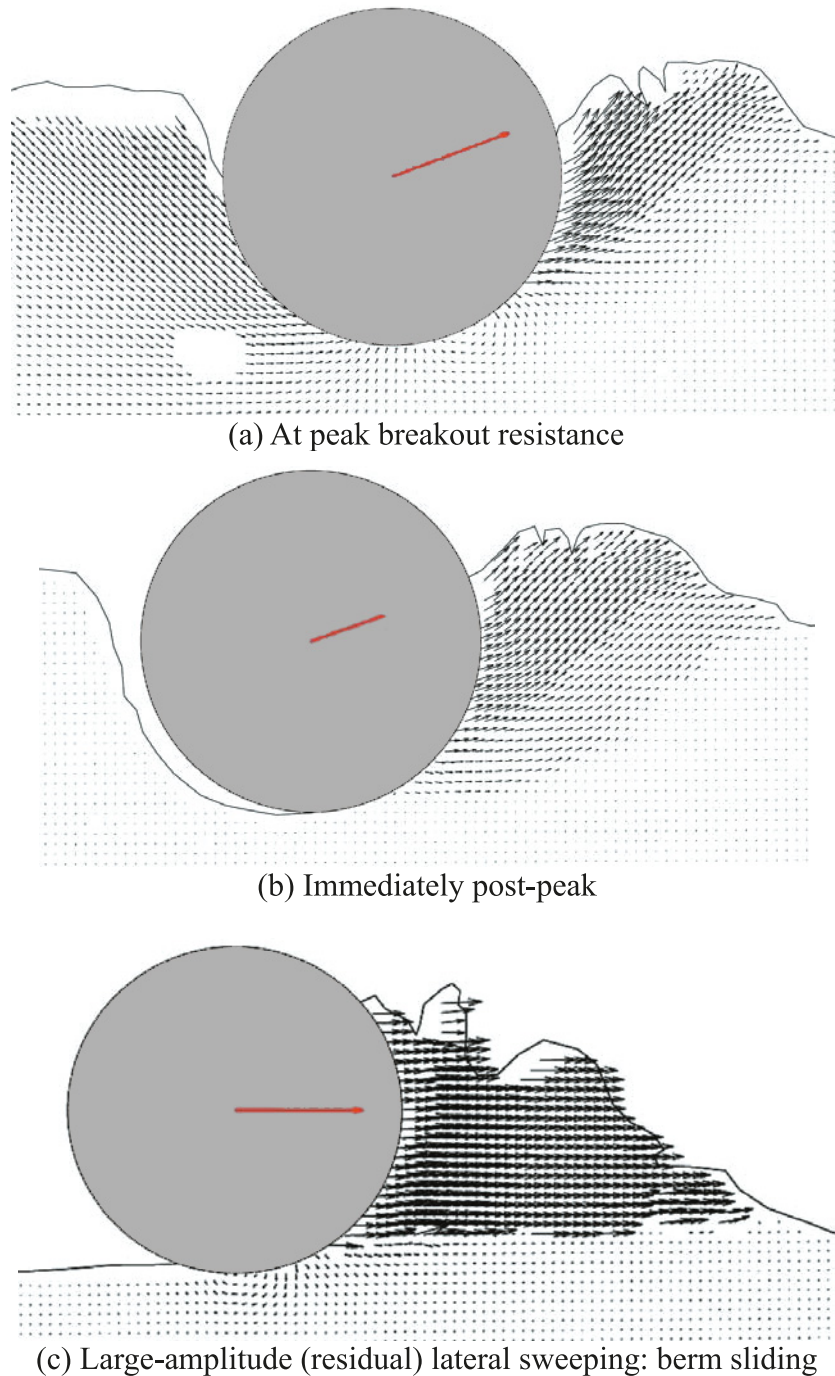
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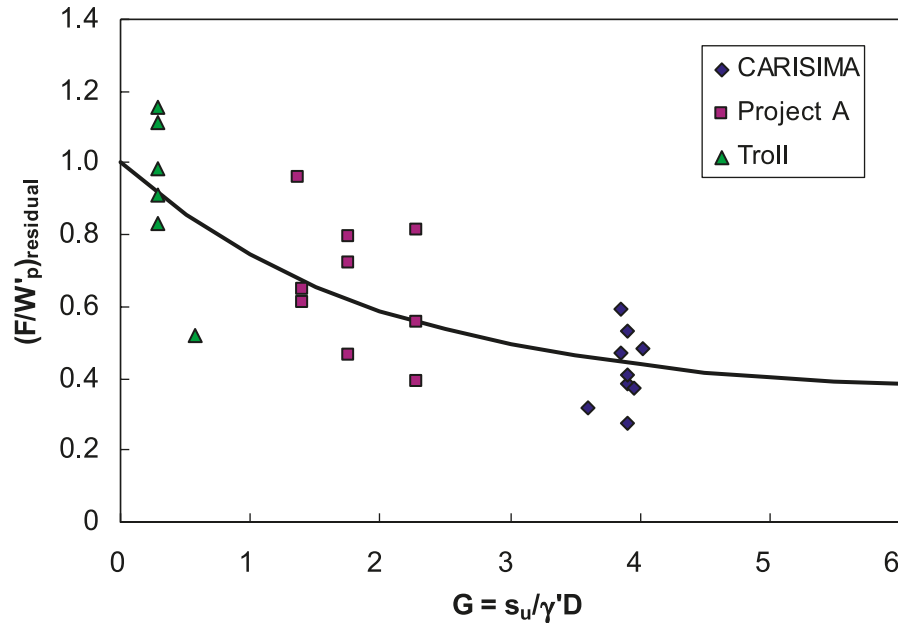
Fig. 1. Observed deformation mechanisms during lateral pipe movement (Dingle et al. 2008).

afterwards (Fig. 1*b*) is slightly influenced by soil weight because there is a net gain in potential energy. This conclusion has been quantified in the upper-bound analyses presented in Cheuk et al. (2008) and the finite element analyses presented in Merifield et al.³ The new analytical solutions presented in these recent publications show broad agreement with the available experimental data and thus provide a more rigorous basis for calculating breakout resistance than eq. [3] in our original paper.

The failure mechanism at large pipe displacements involves basal sliding of a berm pushed ahead of the pipe (Fig. 1*c*). As this berm grows, soil must be lifted from the soil surface into the berm, leading to some dependency on soil weight. Figure 2 adapted from Bruton et al. (2006) shows experimental data of the residual (large amplitude) lateral resistance from a variety of large-scale pipe model tests. The trend in Fig. 2 illustrates the dependency of residual lateral resistance on the value of G . This link is in agreement with the

³R. Merifield, D.J. White, and M.F. Randolph, M.F. The effect of surface heave on the response of partially-embedded pipelines on clay. Manuscript in preparation.

Fig. 2. Variation of residual lateral resistance with $G = s_u/\gamma'D$ (after Bruton et al. 2006).



upper-bound analyses presented in Fig. 16 of our original paper. However, the scatter within each dataset shown in Fig. 2 indicates that this simple relationship does not capture all of the governing behaviour. In particular, the initial size of the soil berm, which is dependent on the initial embedment, is not accounted for by the fitted curve shown in Fig. 2.

The authors agree that it is not meaningful to apply the empirical expressions for the penetration and breakout resistance (eqs. [1] and [3] in our original paper) at extreme values of G . The comparison presented in the paper aimed to highlight the deficiency of these equations by warning that they fail to capture the influence of G outside the range, over which these equations were originally calibrated. In contrast, our derivation of the effect of G via upper-bound analyses is a robust theoretical approach to incorporate the contribution of soil weight.

An alternative way to express the influence of soil weight is to add a surcharge term to the penetration and breakout expressions following the form of the conventional bearing capacity equation as follows:

$$[2] \quad \frac{W'_p}{D} = N_{c,v}s_u + N_{sw,v}\gamma'z$$

$$[3] \quad \frac{F_h}{D} = N_{c,h}s_u + N_{sw,h}\gamma'z$$

where $N_{c,v}$, $N_{c,h}$, $N_{sw,v}$, and $N_{sw,h}$ are bearing capacity factors that capture the influence of soil strength and weight and vary with z/D . A numerical assessment of the parameters for this approach is described in Merifield et al. 2008 and in Merifield et al. in preparation³ and is summarized in Randolph and White (2008b).

Pipe trajectory during large-amplitude lateral sweeps

The discussor commented on the trajectory of the model

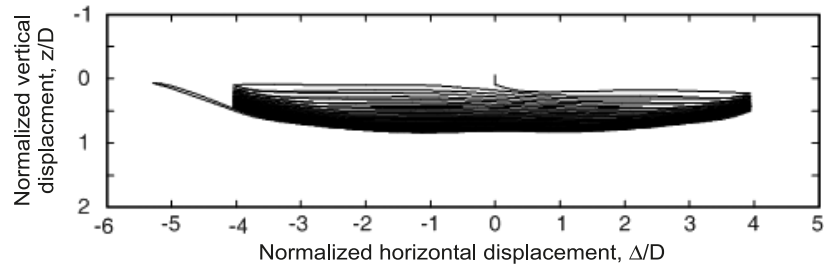
pipe and, in particular, the lack of vertical movement when the pipe approaches the dormant soil berms. This lack of vertical movement leads to the potential for unwanted internal friction within the test mechanism, which (as we acknowledged in our paper) may have affected the vertical settlement of the pipe during lateral sweeping.

However, since the pipe trajectory is predominantly horizontal throughout these tests, the contribution from the vertical load (multiplied by the vertical component of velocity) to the (rate of) energy dissipation within the soil is minimal. Therefore, the conclusions from our comparison between the measured horizontal resistance and the resistance calculated using the upper-bound solution (based on the measured vertical embedment) would not be significantly affected if unwanted friction was present in the apparatus. The authors do concede, though, that the observed accumulation of pipe embedment with cycles should not be relied upon because of the possible influence of the apparatus on the pipe trajectory.

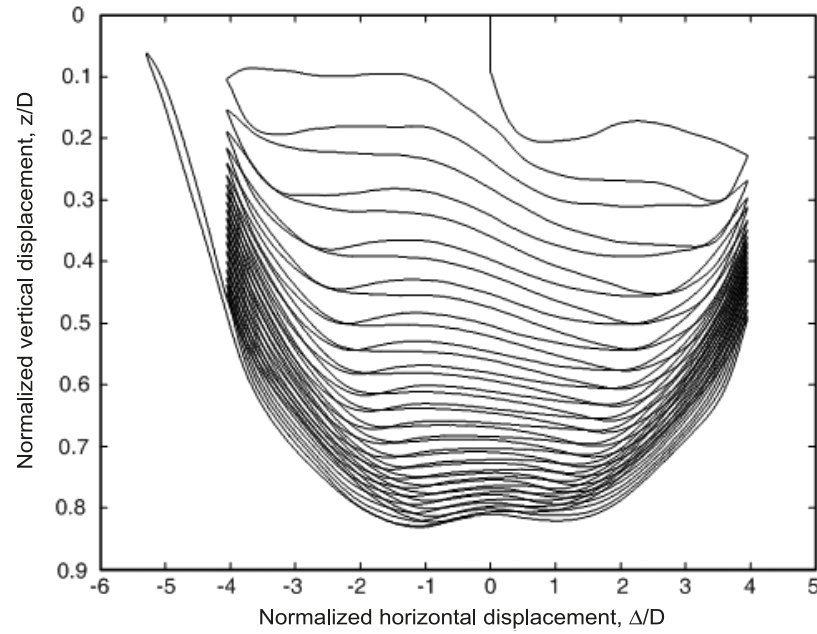
The discussor asked how the vertical pipe movement reported in our paper compared with the behaviour observed in the centrifuge modelling. The authors present the following results from our recent modelling that support our observations described previously regarding the horizontal resistance but confirm the discussor's comment that a vertically-free pipe will rise when a dormant berm is approached.

Figure 3 shows the trajectory and force–displacement response during a test simulating the large-amplitude lateral motion of a 0.8 m diameter pipe weighing 1.44 kN/m resting on soft clay ($s_u = 0.75 + 1.6z$ (kPa), where z is in m). This test, conducted in the geotechnical beam centrifuge at the University of Western Australia, Perth, Australia, used the experimental arrangement described by Cheuk and White (2008). The pipe was rigidly fixed to the actuator, and load cells measured the applied vertical and horizontal forces. The constant simulated pipe weight was maintained

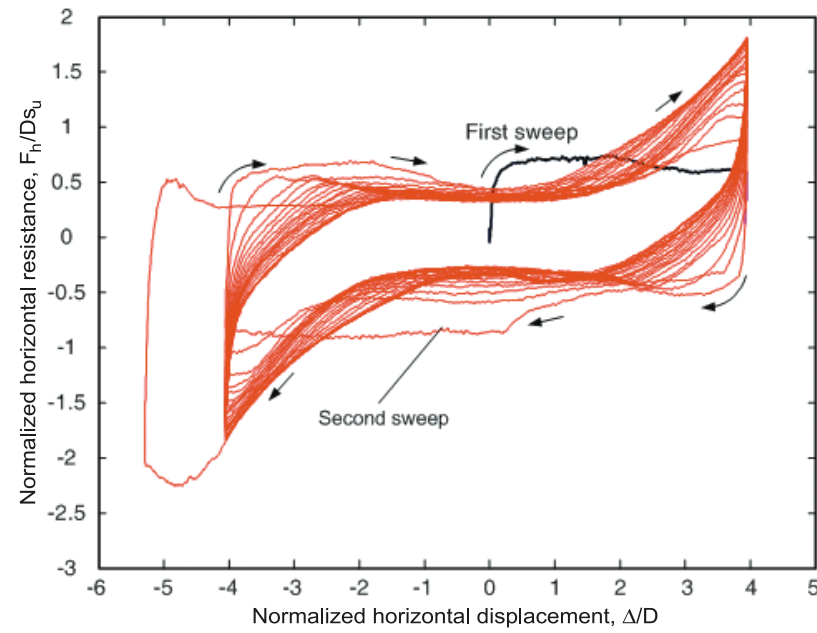
Fig. 3. Centrifuge modelling of cyclic lateral pipe–soil resistance.



(a) Pipe invert trajectory (equal axis scales)



(b) Pipe invert trajectory (axes scaled for clarity)



(c) Normalized horizontal resistance

using software feedback control rather than a “freely-moving” mechanical device. This approach provides verifiable control of the pipe weight, eliminating the concerns associated with our earlier large-scale tests.

During the test shown in Fig. 3, the normalized pipe weight based on the s_u measured at the pipe invert decreased from $W'_p/Ds_u = 1.9$ to 1 as the pipe descended to stronger soil. These results show the following:

- (1) A vertically-free pipe rises as a dormant berm is approached (as stated by the discussor).
- (2) The lateral resistance mobilized at a dormant berm increases as the berm grows in size through the addition of material during each sweep. The dormant berm is reached earlier in each sweep, reflecting the increasing lateral extent. However, the limiting berm resistance is not fully mobilized during fixed amplitude sweeps (as discussed in our original paper).
- (3) The first sweep residual lateral resistance exceeds the residual resistance in subsequent sweeps. This is due to the larger active berm created during the first sweep by the heaved soil that was displaced during the initial penetration of the pipe (as evident in our original tests; see Fig. 8a in our original paper).

It is notable that the initial breakout resistance of $F_h/Ds_u = 0.75$ (for an initial embedment of $z/D = 0.1$) fits on the trend line identified by the discussor for our previous test data (Fig. 1 in the discussion). This confirms the consistency of the horizontal breakout data from the large and centrifuge-scale tests.

It should be noted that the example test presented in Fig. 3 represents a “light” pipe ($W'_p/Ds_u < \sim 2$). Tests modelling a “heavy” pipe ($W'_p/Ds_u > \sim 2$) indicate diving behaviour during the first lateral sweep rather than steady horizontal motion close to the soil surface. This contrast fits with the predictions of pipe motion at breakout that derive from plasticity theory (Cheuk et al. 2008; Randolph and White 2008a). Experimental results relevant to heavy pipe behaviour are described in more detail in Bruton et al. (2008).

Interpretation and selection of s_u

The discussor pointed out that the spatial variation in shear strength of ± 0.5 kPa represents a significant fraction of the near-surface soil strength, hampering the back-analysis of these data. Nevertheless, the variability was evenly distributed across the test bins. No systematic variation originating from the sample preparation method was observed. The use of an average strength profile in the interpretation of the test results is therefore justified.

The effective unit weight (γ') was calculated from the soil moisture content measured at various locations, which was found to have a much lower spatial variability of $< 2\%$. The change in γ' with depth is $< 6\%$ within the upper 300 mm for both types of soil. In the back-analysis calculations (including eqs. [2] and [3] in the original paper), the values of G or soil unit weight were calculated from the average γ' of the upper 300 mm of soil, that is, a constant value for each test bin was assumed. This is appropriate because the soil weight term should comprise the entire soil mass involved

in the mechanism instead of the value at a particular depth. For shear strength, the average s_u value at the pipe invert level was adopted, which is the conventional approach. The soil failure mechanisms during vertical penetration extend above and below the pipe invert by approximately equal distances, supporting the use of this value (White and Randolph 2007; Cheuk et al. 2008).

The discussor suggested that the solution for lateral resistance given in our paper is a limit equilibrium solution and therefore cannot be considered as a rigorous upper bound in the terminology of limit plasticity. However, the mechanism described in our paper consists of a single rigid soil block that slides relative to the seabed. There is no energy dissipation within the block. With the assumption of a smooth pipe–soil interface, the equation derived from limiting (moment) equilibrium (eq. [2] in the original paper) is also an energy balance equation, which provides an upper-bound solution. Therefore, for this simple mechanism, the limit equilibrium solution is also an upper-bound solution. The solution is described in more detail by Cheuk et al. (2008).

Prediction models for pipe penetration

As highlighted by the discussor, the prediction of as-laid pipe penetration is critical to many aspects of pipeline design. The embedment influences both the lateral pipe–soil resistance and the lateral hydrodynamic loading and other design considerations such as the thermal insulation of the pipeline by the surrounding soil and the exposure to submarine slides. It was, therefore, perplexing that both the plasticity (Murff et al. 1989) and empirical (Verley and Lund 1995) models for pipe penetration initially gave unsatisfactory predictions for the particular model tests presented in the paper. This discrepancy prompted the use of the geometric mean of the strengths measured during insertion and extraction of a T-bar penetrometer in an attempt to devise a correlation that agreed well with the data for the two clays (which have differing sensitivity).

One of the reasons for the poor correlation is related to the additional penetration caused by other factors, such as consolidation. Recent model tests, including further work within the SAFEBUCK JIP and confidential industry studies, indicate that the plasticity solutions coupled with the intact undrained strength (i.e., measured during insertion of a T-bar) give good agreement with the load–penetration response (Bruton et al. 2008).

The original plasticity solutions of Murff et al. (1989) for vertical pipe penetration have since been refined (Randolph and White 2008a) and compared with experimental (Dingle et al. 2008) and numerical (Merifield et al. 2008) results. Simplified expressions in the form given by eqs. [2] and [3] herein have been derived for routine use.³

The authors suggest that these new solutions should provide the basis for calculating pipe embedment augmented by assessments of dynamic lay effects and consolidation. A recent review of techniques for the assessment of pipe embedment (including dynamic lay effects, catenary stress concentrations, and soil heave) is presented in Randolph and White (2008b). Further work on the subject of pipe–soil interaction within the SAFEBUCK JIP is presented in Bruton et al. (2008).

The authors thank the discussor for his keen interest in and constructive criticism of our work. Since the tests described in our paper were conducted in 2002, there have been significant advances in the experimental methods used to investigate pipe–soil interaction and the theoretical understanding of this behaviour. We are grateful that his discussion has given us the opportunity to describe some continuing advances in this area.

Acknowledgements

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