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Strength reduction for upheaval buckling of buried pipes in blocky clay backfill

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ABSTRACT

Offshore pipelines are often buried to protect the pipeline from external loads and upheaval buckling. Models for pipe uplift resistance in clay soils are based predominantly on homogenous backfill conditions. In practice, however, there will be significant soil disturbance during installation. With certain trenching techniques this may produce a backfill more akin to a matrix of lumps of intact soil connected by weaker remoulded interfaces. This research uses centrifuge modeling to assess the resistance provided by a representative lumpy clay backfill that has experienced self-weight consolidation. A model pipe is then uplifted through this model backfill in order to assess the soil uplift resistance. Results show that the uplift resistance in this material is governed strongly by the size of the lumps and, to a lesser extent, by the rate at which displacement occurs. When interpreted in terms of the strength reduction η, that may be used to correct between theoretical and measured uplifts, lower values were derived than those currently used based on intact soils. The value of η is seen to be controlled by a non-dimensional drainage parameter, but may be practically estimated based on an estimate of the size of lumps relative to the pipe diameter.

1. Introduction

Offshore pipelines are often buried to protect the pipeline from external loads or to reduce thermal losses. However, if the depth of soil cover above the pipeline is insufficient then the pipe can buckle upwards to relieve thermal strains. It is therefore important to both ensure adequate depth of cover above a given pipeline and also to be able to reliably evaluate how much resistance this can provide should the pipeline attempt to uplift.

Recent years have seen many studies on prediction of uplift behaviour. These are largely divided between theoretical work e.g. (Maltby and Calladine, 1995; Martin and White, 2012), full-scale testing e.g. (Eiksund et al., 2013; Schaminee et al., 1990; Trautmann et al., 1985) and geotechnical centrifuge testing e.g. (Cheuk et al., 2007; Ng and Springman, 1994; Wang et al., 2009). This accumulated knowledge has fed into design codes such as that published by DNV (2007) which provide a framework for designers to best predict the likely performance of their pipelines. Much of the literature, however, relies on analysis and testing of a pipeline that has been wished-in-place. That is, there is no attempt to model disturbance of the insitu soil or backfill material caused during the installation process.

Installation may be undertaken using jetting where the soil is subjected to localised high water pressures facilitating a downwards settlement of the pipeline under gravity. Depending on the soil condition and the jet configuration, the soil may either be liquefied into a homogenously sedimenting mass of particles as examined by Bransby et al. (2002), or it may be locally cut resulting in a matrix of lumps of relatively intact soil connected by a weaker reconsolidated material. Both of these mechanisms are highly disruptive to the condition of the soil and therefore allowance should be made that the soil conditions above the pipeline will not be the same as those of the in-situ soil in either case.

This research therefore aims to better understand the factors governing behaviour of a pipeline in a material that is more representative of conditions following a disruptive installation process. Of particular interest here is the case when the backfill is not homogenous but consists of intact lumps in a normally consolidated matrix, a "lumpy" or "blocky" backfill. This is achieved through physical model tests of a 1:25 scale small scale model pipe being uplifted through clay seabed of variously disturbed condition. A range of uplift velocities were also tested (0.6 mm/hour, 6 mm/hour, 60 mm/hour, prototype scale). In order to accelerate consolidation and to correctly match the in situ self-weight stresses of the soil to the larger prototype, the tests were performed on a geotechnical centrifuge with a gravitational field of 25*g*.

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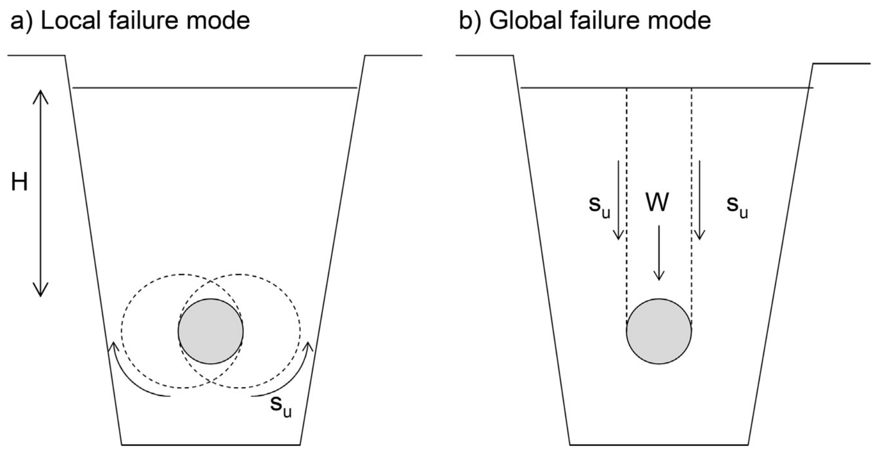

Fig. 1. Schematic illustration of uplift mechanisms a) local failure b) global failure.

1.1. Interpretation framework

In this work, the undrained soil behaviour framework is considered, in line with conventional treatment of clay soils. The literature listed above (and others) provides largely similar interpretations of uplift resistance, with the version provided by recommended practice document DNV-RP-F110 (DNV, 2007) used as the basis for this work. This is because this document provides some design commentary on incorporating installation effects into the design. In DNV-RP-F110, the upheaval behaviour of a pipe requires failure of the soil in either a local (Fig. 1a) or a global failure mode (Fig. 1b). As indicated by the governing equations presented in the DNV, the soil resistance to shearing in a global mode is less when the depth of cover H is low, so this is sometimes termed a "shallow" mechanism. However, as the global mechanism requires uplift of overlying soil mass plus shearing along a surface dependent on depth of cover then this becomes inefficient at greater depths, whereas the local mechanism is relatively insensitive to parameter H when considering undrained strengths. Of particular interest in this analysis is the local mechanism, because the formulation of the resistance of such pipes includes an empirical factor η that is intended to correct between measured data and theoretical estimates. The expression for peak uplift resistance R is given in Eq. (1), where N_c is an analytically-derived bearing capacity factor, D is pipe diameter, s_u is the undrained strength of soil measured at the level of the pipe's centre and η is the empirical strength reduction factor.

$$R = N_c D s_u \eta \tag{1}$$

As the pipe is unable to achieve a complete "flow-around" type mechanism until cover depths in excess of 4.5 D are achieved, the bearing capacity factor N_c is provided as in Eq. (2), where parameter r is a roughness factor ranging from 0 in the perfectly smooth case to 1 for a perfectly rough pipe.

$$N_c = 2\pi\left[1 + \frac{1}{3}\tan^{-1}\left(\frac{H + D/2}{D}\right)(1 + r)\right] \tag{2}$$

The commentary on trenching methods in DNV (2007) suggests that jetting causes the entire soil mass to go into suspension. Strength may therefore be modelled by assuming a reconsolidated (i.e. effective stress dependent) shear strength exists throughout if an adequate period of reconsolidation is allowed for strength regain and adequate reduction made to H to consider the reduced depth of cover due to consolidation. However, anecdotal evidence suggests that some soils (and some jetting and ploughing strategies) may not produce a homogenous backfill but a lumpy or blocky backfill as described above. Therefore the backfill becomes a matrix consisting of lumps of relatively intact soil connected by the relatively weaker reconsolidated soil, and the use of a very soft reconsolidated strength based on a very low effective stress of a homogenous soil mass may be unrepresentative.

Parameter η in Eq. (1) accounts for differences between the design undrained shear strength s_u and the back-calculated value following testing in remoulded clays according to DNV. Such differences are described in this source as being due to rate and viscous effects, and progressive failure regime, but the factor could also be usable to examine apparent strength reductions due to trenching disturbance. Data presented in this work will therefore use the strength reduction factor η as a means for identifying the relative effect of having a lumpy material as backfill, rather than a completely uniform soil at either intact or remoulded strength. The rationale will be to complement the existing industry standard formulations rather than create new ones, as well as better explore the meaning of this parameter.

2. Centrifuge testing

2.1. Equipment

As the blocky backfill material relies on both a coherent intact soil and an interstitial soil consolidated under correct effective stresses, small scale models tested without the additional gravity are unable to reach representative strengths. Therefore physical models for this study were tested on the 3 m radius beam centrifuge at the University of Dundee. Spinning the 1:25 scale model such that the normal acceleration field equals 25 times earth's gravity g makes the small soil mass weigh the same as one that was 25 times deeper, matching the effective stress fields in model and prototype. This has the benefit of enabling consolidation to occur at the correct stresses as well as at accelerated timescales. Further discussion of centrifuge scaling laws may be found elsewhere, e.g. (Schofield, 1980); all data presented here will be in prototype or normalized scales with comments on scaling where required.

The model pipe, as shown in Fig. 2a, was 25.4 mm diameter (D), corresponding to 635 mm at prototype scale. The length was 234 mm, slightly less than the width of the model container in order to create plane strain conditions without friction at each end of the pipe. The model container width was 115 mm, leaving space for 1.8D of soil either side of the pipe which is sufficient to enable the formation of a displacement mechanism (Fig. 1). Uplift at the desired speed was provided by a screw jack driven by a stepper motor, which pulled the pipe through a hanger connected to the two thin rods as shown in Fig. 2a. To permit settlement of the pipe during the consolidation phase and eliminate potential hang-up of consolidating soil above a rigidly fixed pipe, the rods were able to move vertically downwards relative to the hanger. Load was measured above the hanger, with the buoyant weight of the pipe/hanger subtracted from measurements so that only

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