Proceedings of the 16th International Conference on Soil Mechanics and Geotechnical Engineering

doi:10.3233/978-1-61499-656-9-1801

Pipeline uplift mechanisms using finite element analysis

Analyse des mécanismes de soulèvement de pipelines par eléments finis

J-F. Vanden Berghe, D. Cathie & J-C. Ballard
Fugro Engineers S.A., Brussels, Belgium

ABSTRACT
The proposed paper presents simulations of the soil behaviour during uplift displacement of a pipeline buried in very loose sand using finite elements.
The 2D finite element software Plaxis is used for this study. Sand densities analyses the range from loose to very loose sand (typically a relative density between 0% and 20%). The strain softening behaviour expected for these densities is investigated using negative values of the dilation angle ψ. This assumption is analysed by comparing simulation of classical undrained triaxial tests with actual test data from the literature.
Mechanisms of failure in drained conditions are investigated for uplift loading in both dilative and contractive soils and compared with a simplified uplift model. The potential for uplift by some inclined mechanism in contractive soils is then investigated.

RÉSUMÉ
Cet article présente une analyse par éléments finis du comportement du sol lors du soulèvement de pipelines enfouis dans des sables très lâches.
Le logiciel Plaxis est utilisé pour cette étude. Des densités très faibles sont analysées – typiquement des sables ayant un densité relative entre 0% et 20%. Des angles de dilatance négatifs sont utilisés pour simuler le comportement fortement contractant de ce type de matériau. La validité de cette méthode est analysée en comparant les résultats de simulations d'essais triaxiaux non drainés avec des résultats expérimentaux.
Les mécanismes de rupture sont analysés en conditions drainées pour des sables aussi bien contractants que dilatants. Les résultats sont comparés avec un modèle actuellement utilisé pour le dimensionnement. Le risque de soulèvement par un mécanisme incliné est aussi investigué dans le cas des sables contractants.

1 INTRODUCTION

Submarine pipelines are often buried for thermal insulation and protection from trawling or scour. When the pipeline transports a hot product at high pressure the compressive forces that result create a tendency to buckle. Buckling may be in the vertical plane (upheaval buckling) or laterally (snaking) according to the restraint provided by the soil and the direction of the initial imperfection.

The problem of upheaval buckling has been treated widely both structurally (Hobbs, 1984) and for soil resistance to pipeline uplift (Schaminee, 1990; Pederson, 1988; White, 2001). Experimental work on pull-out resistance of plate anchors (Row & Davis, 1982) supplements the more limited data available for pipelines. There is very little data on lateral stability of buried pipelines.

Pipeline burial is often achieved by jet trenching in which high pressure water jets are used to fluidise the soil to allow the pipeline to sink. Soil resedimentation around the pipe (in the case of sand) can leave the sand in a loose state (Kvalstad, 1999).

This paper addresses the pipeline-soil response for very loose sand which is contractive when sheared. The problem is treated in 2D by finite element analysis using a hyperbolic stress-dilatancy model and assuming drained behaviour. Mechanisms of failure are investigated for uplift loading in both dilative and contractive soils and compared with a simplified uplift model currently used for design. The potential for uplift by some inclined mechanism is then investigated.

2 PIPELINE UPLIFT MECHANISMS

2.1 *Failure mechanism*

The mechanism of failure of a pipe undergoing uplift displacement has been developed from anchor pull-out studies (Balla, 1961) and applied to pipelines (Schaminee, 1990). Figure 1 shows the notation used in this paper and the simple failure mechanism that is generally observed.

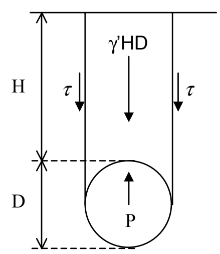

Figure 1 - Notation and wedge uplift mechanism

The wedge mechanism shown in Figure 1 is applicable for drained resistance of medium dense to dense sands, i.e. sands above their critical density (greater than about 23% relative density – Bolton (1986)). More recently, experimental work has shown that a flow-around mechanism occurs under certain conditions even under drained conditions (existence of a gap below the pipe, loose soil, large H/D ratio – White & al, 2001). Under these conditions the wedge mechanism is not appropriate.

2.2 *Simplified design methods*

One of the most widely used models is attributed to Schaminee et al (1990) based on the wedge mechanism:

$$N_{up} = \frac{P}{\gamma' HD} = 1 + f\frac{H}{D} \qquad \text{[Eqn 1]}$$

Where f is the empirical uplift resistance factor. Recommended values of this empirical factor vary considerably (between 0.15 and 0.6 in sand) to encompass the range of data from experimental work.

It has been observed recently that the peak uplift resistance in dilative soils is associated with a shearing mechanism along planes angled at ψ (the dilation angle) to the vertical, and that peak resistance is a function of density (and hence dilatancy). An analytical expression for the uplift factor f based on ϕ and ψ has been proposed to take account of the sand density and dilatancy (White et al, 2001):

$$f = \tan\psi + (\tan\phi - \tan\psi)\left[\frac{(1+K_0)}{2} - \frac{(1-K_0)(\cos 2\psi)}{2}\right] \qquad \text{[Eqn 2]}$$

The envelope of curves predicted by this method encompasses almost all the published data.

3 FINITE ELEMENT MODEL

3.1 *Model description*

The soil around the pipeline has been modelled using the Hardening Soil Model provided in Plaxis (Plaxis, 2002).

Limiting states of stress are described by a friction angle φ and a dilatancy angle ψ with zero cohesion. The friction angle φ is the failure angle at maximum shear stress. Based on the Rowe stress-dilatancy theory, the friction angle φ, the critical state angle φ_{cv} and the dilatancy angle ψ are linked by the following equation:

$$sin(\varphi_{cv}) = \frac{sin(\varphi) - sin(\psi)}{1 - sin(\varphi)sin(\psi)} \qquad \text{[Eqn 3]}$$

The critical state angle φ_{cv} is generally about 30° for wide range of soil types. Therefore, there is a direct relationship between the friction angle φ and the dilatancy angle ψ. For a large range of values of φ and ψ, this relationship can be simplified as follows:

$$\varphi = \psi + 30° \qquad \text{[Eqn 4]}$$

In order to simulate the strain softening behaviour expected for loose sand, negative values of the dilation angle ψ are used.

The pipeline was modelled as linear elastic with a high stiffness. A layer of interface elements is used around the pipeline in order to simulate the soil/structure interface.

3.2 *Analysis procedure*

Uniform soil conditions with no residual excess pore pressure was assumed for the initial state. The initial vertical stress condition was based on the soil weight. The initial horizontal stress was calculated using the earth pressure coefficient at rest K_0.

Soil-pipeline interaction was modelled by applying a prescribed upward displacement to the pipeline and computing the resisting load. During this operation, the soil conditions were considered perfectly drained and so no excess pore pressure could be generated.

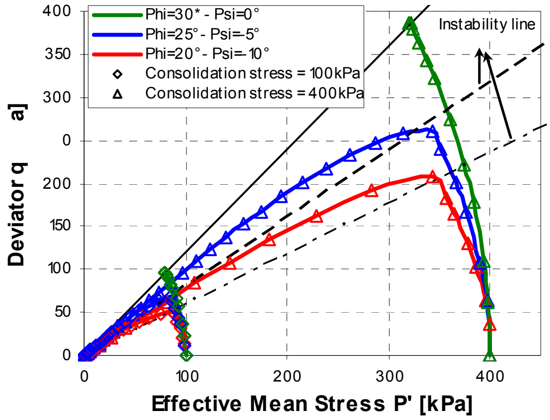

Figure 2 – Undrained triaxial test simulations

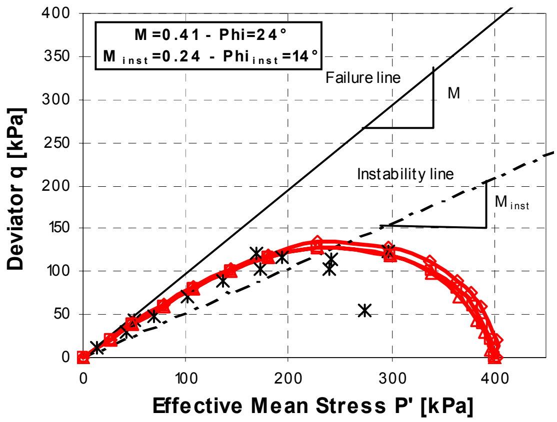

Figure 3 – Laboratory results of undrained triaxial test on very loose sand (Norris et al, 1997)

3.3 *Soil model validation by triaxial tests simulation*

Simulations of undrained triaxial tests were performed for different negative dilatancy angle ψ . The objective was to analyse the model response for negative dilatancy angles and the relationships between ψ and the relative density of the sand. The results of the simulations are presented on Figure 2.

Specimens with negative dilatancy angle ψ exhibit contractive behaviour. During undrained simulations, the contractive behaviour induces an increase of pore pressure and a decrease of shear resistance until zero shear resistance is reached. The rapidity of reduction in shear stress depends strongly on the dilation angle: the lower the dilatancy angle, the faster the degradation. The instability lines defined as the point where the strain softening is triggered are also plotted on Figure 2 for different values of dilative angle ψ. As illustrated on the figure, the position of the instability line is not dependent on the consolidation stress. The instability line in the model could be characterised by an instability angle φ_{inst} defined as the mobilised friction angle at the instability point. The obtained angles are 30°, 20° and 15° for ψ of 0°, -5° and -10° respectively.

The angles obtained are comparable with experimental data provided by Norris (1997) for example. Norris correlated the instability angle and the relative density of the soil and obtained instability angles of 14° and 21° for relative densities of –16% (density lower than minimum density) and 16%.

The main limitation of the model presented above is how the dilatancy angle is used. Dilatancy is only activated when the instability line is reached. At the instability line, the dilatancy is used until a dilatancy cut-off is reached. In case of negative dilatancy in undrained conditions, excess pore pressures are generated inducing the strain softening of the soil. In reality, however, the dilatancy angle is not a constitutive parameter and should vary during the shearing. It could change sign and equal zero at the critical state. This difference between the Plaxis model and the actual soil behaviour explains the difference in the shape of the stress paths presented on Figure 2 and 3. The simulation underestimates the dilatancy during the first part of the simulation and overestimates it during the strain softening. Nevertheless, the model captures well the general soil behaviour and is therefore useful for estimating the overall behaviour of the problem.

4 FINITE ELEMENT RESULTS

Different aspects have been investigated using the finite element model described above. The results are described in the subsequent sections.

FE analysis can be used as a tool to validate analytical methods. FE results are compared with an analytical method recently developed by White et al. (2001).

FE analysis can also illuminate aspects of behaviour that cannot be addressed by simplified methods. Firstly, the failure mechanism in extremely loose material has been investigated. Secondly, the oblique and lateral resistance has been compared to the vertical uplift resistance.

4.1 *Effect of the normalisation*

Normalised or dimensionless parameters are used to generalise results of experimental or numerical analysis, and to group parameters that control behaviour. Experimental data for pipe uplift are often presented in a plot N_{up} vs. H/D. N_{up} is normalised as shown in [Eqn 1].

FE analysis has been used to confirm the validity of this normalisation. Simulations using different pipe diameters and different cover heights have been compared and demonstrate the validity of the normalisation.

4.2 *Uplift resistance in dilative sand – comparison with White analytical method*

White et al (2001) proposed a shearing mechanism along planes angled at ψ (the dilation angle) to the vertical. FE analysis confirms this shearing mechanism along inclined planes (Figure 4). However, the observed angles are slightly greater than ψ, especially for high embedment ratios.

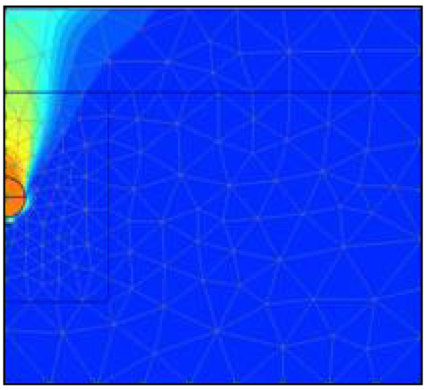

Figure 4 – Uplift failure mechanism in dense sand (H/D=4)

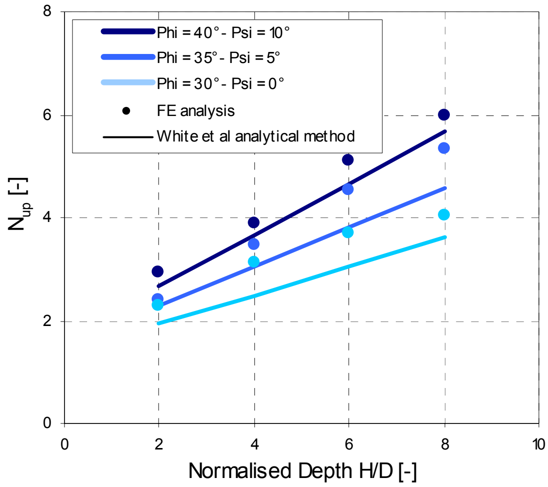

Figure 5 – Comparison between FE results and White method

Figure 5 compares the peak uplift resistance (presented as N_{up}) predicted with FE analyses and the White method. The trend is similar and the FE analysis confirms the strong density dependency. However, slightly higher peak resistances are obtained from the FE method, probably due to the greater inclination of the shear planes.

4.3 *Extension to contractive soils*

Offshore pipelines can be mechanically backfilled or jet-trenched. The process of mechanical backfilling usually leads to a loose to medium dense backfill. The White method is applicable to calculate the uplift factor.

On the other hand, sand backfill after jet trenching is likely to be very loose and contractive material. Uplift in such material is associated with a different failure mechanism and the shear plane failure mechanism is no longer appropriate.

This extremely loose soil is simulated in Plaxis using a negative dilation angle. A circular flow-around mechanism is observed (Figure 6) as also seen experimentally (White et al, 2001).

To trigger the flow-around mechanism, the pipe needs to be displaced sufficiently for a gap to open up beneath the pipe. In loose sand, as the pipe moves up, the sand above contracts. This allows uplift without forming a wedge failure to the surface (a "local" mechanism). When the gap is suitably large, the flow mechanism kicks-in.

However, in dense sand, as observed with the FE simulations, the peak uplift resistance is associated with the wedge failure. The "easier" flow-around mechanism is only triggered after the peak resistance occurs at larger pipe displacements.

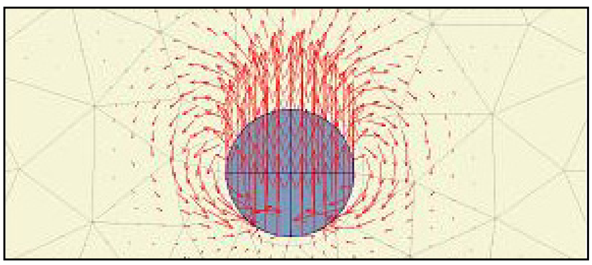

Figure 6 – Flow-around mechanism

Figure 7 shows the uplift bearing capacity factor for extremely loose sand (typically between 0% and 20% relative density) . There is a lack of experimental data at these densities. Nup is much lower than in dilative sand (relative density greater than about 23% (Bolton, 1986)) and could be even lower if undrained failure occurs. Indeed, the collapse of the soil structure can lead to static liquefaction if soil permeability is sufficiently low. It should be noted that the increase of uplift resistance with burial depth is reduced.

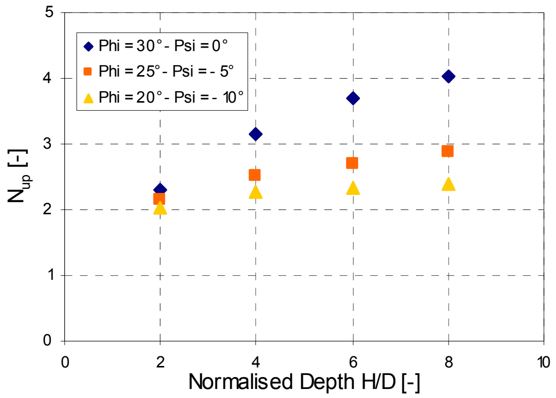

Figure 7 – Uplift bearing capacity factors for contractive sands

4.4 *Oblique and lateral resistance in contractive soils*

Powerful jet trenchers can erode and fluidise a large zone to bury the pipe. This leads to a large zone of very loose material around the pipe. It has been observed that pipelines can buckle obliquely and eventually protrude above the seabed.

Oblique and lateral resistances have been compared with the vertical resistance using the FE method. Figure 8 shows that the increase in resistance with the displacement direction (with respect to the vertical) is small between 0 and 30°, especially for the loosest soil. However, at larger inclinations, the difference between horizontal and vertical resistances increases.

Lateral resistance is associated with an active and passive wedge failure at the depth studied (H/D=2) whereas the oblique resistance is associated with a mix between the flow-around and wedge failure mechanisms (Figure 9).

The practical conclusions from this analysis are that pipeline uplift in loose soils is likely to occur in the quadrant ±30° from the vertical. Moreover, a wider surface of rockdump may be required than considered if movement is to be mitigated.

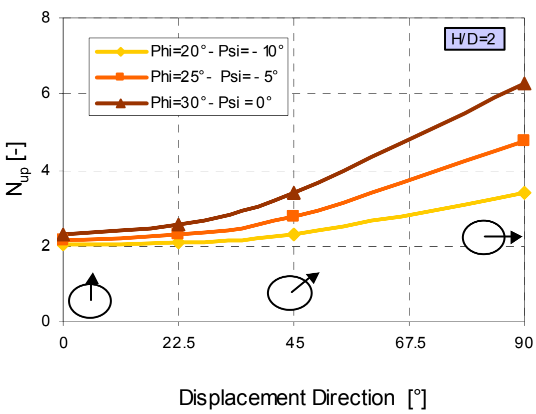

Figure 8 – Comparison between vertical, oblique and lateral resistance in contractive soils

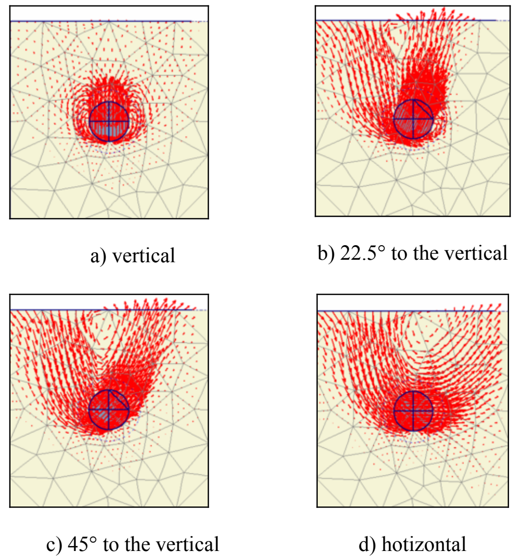

Figure 9 - Failure mechanisms for different displacement directions

5 CONCLUSIONS

Pipeline uplift in very loose sand is dominated by "local" failure and "flow around" mechanism. Simplified methods based on a wedge failure are inappropriate. Uplift resistance is very similar for movements within about 30° to the vertical. This implies that upheaval buckling may well occur on inclined planes.

REFERENCES

Balla, A. (1961), The Resistance to Breaking out of Mushroom Foundations for Pylons, Proc. 5 th International Conference on Soil Mechanics and Foundation Engineering, Paris, 1, 569-576.

Bolton, M.D. (1986), The strength and dilatancy of sands, Geotechnique, Vol. 36, No.1, pp. 65-78

Hobbs, R.E., 1984, In-Service Buckling of Heated Pipeline, Journal of Transportation Engineering, V.110, N.2, March 1984, pp.175-189.

Kvalstad, T.J. (1999), Soil resistance against pipelines in jetted trenches, Proceedings of the Twelfth European Conference on Soil Mechanics and Geotechnical Engineering, Netherlands, Vol. 2, pp. 891-898

Plaxis B.V. (1998), Plaxis finite element code for soil and rock analysesly, Version 8, Balkema.

Plaxis B.V. (2002), Course for experienced Plaxis users, Proceedings, Stichting Postacademisch Onderwijs, Civiele Techniek en Bouwtechniek, March 2002.

Rowe, R.K. and Davis, E.H. (1982), The behaviour of anchor plates in sand, Geotechnique, Vol. 32, No. 1, pp. 25-41

Schaminee, P.E.L., Zorn, N.F. and Schotman, G.J.M (1990), Soil response for pipeline upheaval buckling analyses: full-scale laboratory tests and modelling, Offshore Technology Conference, No. 6486, pp. 563-572

Thales (2003a), Finite element simulations of pipeline uplift, Internal Report

Thales (2003b), Analysis of Plaxis models to simulate sand behaviour, Internal Report

White, D.J., Barefoot, A.J. & Bolton, M.D. (2001), Centrifuge modelling of upheaval buckling in sand, International Journal of Physical Modelling in Geotechnics, Vol. 1, No. 2, pp. 19-28