NUMERICAL MODELING OF UPLIFT RESISTANCE OF BURIED CONCRETE DUCTS & PIPES

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ABSTRACT
This paper presents simulation of the pipe and soil system behavior during uplift displacement of pipelines in dense and loose sand by a 2D Finite Element modelling. As the first part of an ongoing research, this study focuses on the type of the soil failure mechanism which occurs in saturated condition at different burial depths and soil densities for constant pipe diameter. Then, at this stage, pipe is considered as a linear elastic but very stiff material (compare to the soil). Using conventional continuum elements for the soil, its material behavior is modelled by the Drucker-Prager criterion. The numerical results compared to the laboratory observations and results as well as theoretical aspects. Different laboratory failure mechanisms in loose and dense soils and also load-displacement curves impressively reproduced by the numerical modellings. Even, development of a gap below the pipeline in dense sand well captured by the FEM results. The essential effect of the soil density on the resistance force against uplift displacement is also well illustrated by the numerical results. To obtain a normalized upheaval load-displacement curve for this phenomenon, displacements were normalised by the pipe diameter and also burial depth, however, another more suitable length parameter is under investigation.

Keywords: buried pipeline, uplift, sand, numerical modeling, FEM

1. INTRODUCTION
Lifeline pipes and ducts are commonly buried to provide environmental stability, thermal isolation and mechanical protection. In shallow trenches of saturated sand soils, buried pipelines and ducts are under threaten of progressive upheaval creep failure, during long term operation. This phenomenon called as upheaval buckling. This type of failure can cause the pipe to resurface at its bed-line and causes probable fracturing and damages, specially on the concrete ducts. This presents considerable operational problems and would have significant costs. The backfill soil in the trench and the pipe weight contribute to prevent the upheaval buckling load imposed to the pipeline. However, the resistance to the upheaval load provided by the soil is difficult to calculate. A number of theoretical models has been developed to predict the resistance to upward movement provided by the pipeline/soil system. Generally, the models
have considered the backfill soil without considering imperfections of the material during pipe placement. Also, no roughness effect of the pipe-soil contact surface was considered. It is widely accepted that the uplift resistance is complex and it is related directly to the geotechnical properties of the soil.

Most theoretical analyses assume some failure surfaces extending through the soil above the pipe. The simplest of these, reported by Matyas and Davies (1983) [1], is to assume a vertical slip surface extending above the pipe, Figure 1(a). The uplift resistance per unit length derived from the weight of soil above the pipe, the weight of soil displaced by the upper half of the pipe and the shearing resistance along the vertical slip surface, is given by the expression (1):

\[ F_v = (1 - \frac{\pi D}{8H} + K \tan \phi \frac{H}{D}) \gamma H \]

(1)

\[
F_v = \left(1 + \frac{2H}{D} \frac{H}{D} \frac{H}{H} K \tan \phi \frac{H}{D} \right) \gamma H
\]

(2)

in which \( \gamma \) = effective soil unit weight, \( H \) = depth to centre of pipe, \( D \) = pipe diameter, \( \phi_{ps} \) = angle of soil friction in plain strain, and \( K \) = lateral earth pressure coefficient. The value of \( K \) is often taken as \( K_0 \), the at rest coefficient for loose sand, but its value in case of the dense sand is difficult to assess and can often be greater the one. Trautmann et al (1986) found [3], this theory has good agreement with the experimental data with \( K \) values of 0.5, 0.65 and 0.75 for pipes in loose, medium and dense sand, respectively. However, rupture surface above pipes are generally curved in broad agreement with the pyramidal shaped geometry analyzed by Meyerhof and Adams (1968) [2], shown in Figure 1(b). For shallow embedment, they ignored the second term in Eq. 1, and assumed \( K=0.95 \), while at greater depths the Eq. 2 was proposed:
where \( H_c \) = vertical extent of the rupture surface and depends on \( \varphi_{ps} \) and \( D \).

The uplift force, \( N_u \) can be non-dimensionalised and re-expressed as uplift factor, \( f_u \), as suggested by Schaminee et al (1990) [4]. This calculated using:

\[
f_u = \frac{N_u}{(H/D)}
\]

in which:

\[
N_u = \frac{F_u}{\gamma H D L}
\]  

(3)

where \( H \) is the instantaneous embedment depth measured to the crown of the pipe.

2. PREVIOUS MODELLINGS’ RESULTS

Over the last decade, the geotechnical aspects associated with upheaval resistance of the buried pipes have received considerable attention by the researchers. The focus of this attention has been aimed at the mechanism of soil failure and measurement of uplift load for various soil types. Because of practical difficulties and the high cost of conducting full scale field tests, the majority of these works has been done at small scale, or using geotechnical centrifuge modelling to simulate full scale conditions [5]. Barnsby et al (2001) [6] undertook a combined study using numerical FE analysis and scaled physical model testing to investigate soil resistance to upwards pipeline movement. Rezaee et al (2005) [7] carried out full scaled laboratory tests by improving the conditions of the experiments done by the Trautmann et al (1986) [3], which the FEM modelings of these experiments are presented in here.

3. EXPERIMENTAL OBSERVATION

The laboratory model built by Rezaee et al (2005) [7],[8], was including four parts: test box, coarse sub-grade, transducers & gauges, and water intake system to apply loading.

Rigid boundaries have to be located remote from the pipe so as not to interfere with failure mechanisms or effect on the effective stresses in deforming zone. According to the Trautmann suggestion [3], the width and height of the model should be chosen at least five times and its length nine times of the pipe diameter. Then, the test box dimensions- in toughened glass material of 1 cm thickness- considered 69cm of width and height, and 177cm of length, comparing to 11cm of the P.V.C pipe diameter. The test box was well braced not to deform during the loading. As it was necessary to increase the water surface uniformly, a coarse graded layer in 10 cm thickness spread over the test box floor. This layer also prevented cavitations and water worn effect on the soil. In Trautmann’s test load was applied mechanically to the pipe, while in here, real uplift force of the water is applied to.

According to the tests done on the local area soil which used in this study, it classified as a poor aggregated sand (SP) with 2.66 of the solid density and 0.00225 cm/sec of the permeability rate. Its minimum and maximum specific weight were 1.38 and 1.78gr/cm³, respectively. The internal friction angle for its loose case was equal to 32.9° and in dense case was 37.6°.
3.1. Laboratory Failure Mechanisms

In initial tests [7] using dense sand ($\gamma = 1.78 \text{ gr/cm}^3$), pipe was placed in depth equal to its diameter, i.e. 11 cm- from its center level to the soil surface. During the loading, two mechanisms of failure was observed; first, an angled sliding block mechanism, when the soil resistance reached to its utmost strength against upheaval forces, an inclined slip surface (about 20° diversion from the normal direction) created. This failure mechanism happened under small upheaval displacement (about 2mm)- as shown in Figure 2(a)- and this followed by a quick reduction in resistance upheaval force. By increasing the upheaval displacement, uplift resistance more decreased, and a second mechanism called circulation mechanism, observed around the pipe. At this stage, two gaps below the pipe formed [Figure 2(b)] and by progressing the upward moving of the pipe, these gaps were filled by flow dropping of the upper soils [Figure 2(c)]. As the curve in Figure 2 depicts after 10mm of displacement, uplift resistance force reaches to a constant amount, called as the residual force, with breaking of the inter-locking between the soil particles and make the pipe buoyant, and a considerable uplift displacement.

![Figure 2. Load-displ. diagram of dense soil on the top of the buried pipe under upheaval force [7]](image-url)
For the loose sand, only a circular mechanism observed as the failure mechanism of the soil. In this case, there was no sign of the sliding surfaces (trivial interlocking) and from the beginning, only the two gaps created under the pipe, and further on, they were filled by flow dropping of the upper soil. Figure 3, illustrates the variation curves of the resistance load against upheaval displacement for different burial depths as well as sand densities.

As the curves depict, for embedment ratio of 0.7 and 1, the residual force in dense sand is less compare to the loose one. Its reason can be stated by this fact when the angled sliding block happened in dense soil, the soil reminds above the pipe is less than the amount in case of loose sand. As the resistance shear force depends on the surcharge loads, then the experimental results is justified. Also, as the curves show the effect of the density is more than the burial depth ratio in uplift load carrying capacity of the soil.

Figure 3. Upheaval load-displ. curves for different burial depths of loose and dense sand [8]

4. FINITE ELEMENT MODELLING

2D finite element plane-strain analyses was carried out to investigate the uplift behavior of circular pipeline to identify the effect of the most important parameters- i.e. the soil density and the embedment depth ratio- for further studies as well as to understand the mechanics of the soil around the pipeline in more details. Uplift capacities and soil failure mechanisms were found for different soil conditions and burial depth ratios. The influence of the mesh size, soil stiffness, ratio between the soil permeability and the loading rate as well as the soil/pipe contact surface effect has not been considered at this instance.

4.1. Soil & Pipe Characteristics

Conventional 8-noded quadrilateral serendipity element used to model the pipe and
soil. Each node has two degrees of freedom of displacement. A non-associated Drucker-Prager elastic-perfectly plastic criterion was chosen to model soil material. This has a constant friction angle $\phi$, a non-associated dilatancy angle $\psi$, and cohesion stress $c$, apart from its elastic parameters $E$, $G$ and $\nu$. For this example, some of the parameters have been speculated to fit suitably the behavior of the soil/pipe system. This can be supported by the facts which are explained in below:

1. Experimental data in the literature are not enough to provide all values of parameters needed in the numerical modeling. Then some of the values (listed in Table 1) have been picked from other sources. This would be the main source of discrepancy in the FEM results compared with the experiments, as the predictions are sensitive to in situ stresses and soil stiffness.

2. It should be pointed out that all analyses were run under plane-strain formulation which results more stiffer and higher load-carrying capacity structure compared to the plane-stress one, which is the more realistic option in 2D analyses for the tested buried pipe.

3. No potential cracking and fracture is modeled or allowed in the soil elements by loading progress. This also results a stiffer response of the numerical model.

<table>
<thead>
<tr>
<th>Material characteristics</th>
<th>Loose</th>
<th>Dense</th>
<th>pipe</th>
</tr>
</thead>
<tbody>
<tr>
<td>$E$</td>
<td>3-4 MPa</td>
<td>8-10 MPa</td>
<td>$2.6 \times 10^7$ MPa</td>
</tr>
<tr>
<td>$\gamma$</td>
<td>1.45 gr/cm$^3$</td>
<td>1.78 gr/cm$^3$</td>
<td>-----</td>
</tr>
<tr>
<td>$\nu$</td>
<td>0.33</td>
<td>0.33</td>
<td>0.3</td>
</tr>
<tr>
<td>$\phi$</td>
<td>32.9°</td>
<td>37.6°</td>
<td>-----</td>
</tr>
<tr>
<td>$c$</td>
<td>2 KPa</td>
<td>5-10 KPa</td>
<td>-----</td>
</tr>
<tr>
<td>$\psi$</td>
<td>0°</td>
<td>0°</td>
<td>-----</td>
</tr>
</tbody>
</table>

4. Dilatancy angle, measured by $\tan \psi$, was constant and set to zero. This means that soil material can slide over the other without producing any vertical displacement and this should affect only marginally the results. However, this is reasonable for the loose soil, but for the dense one, more explanation is presented. As laboratory tests show, dilatancy angle decreases to zero with increasing plastic shear slipping or increasing normal confining pressure. These phenomena occur often combined, particularly in confined structures (for instance deep soil, in here), because shear slip with dilatancy necessarily induces normal compressive stresses. For the concerned case in here, the combined action of these two factors will also produce a faster degradation of the dilatancy angle under increasing confining pressure on the soil.

5. NUMERICAL RESULTS

5.1. Dense Sand

Figure 4-left gives upheaval load-displacement curve of FEM analysis for the dense soil with embedment depth ratio ($H/D$) equal to 1, which well correlates with
the experimental results, with about 10 percent difference in peak load. The numerical results show a steeper initial slope of the curve compared to that obtained from the experimental results. This may be due to a much higher value of the stiffness parameter $E$, which has not been provided in the reference text [7]. The higher initial stiffness caused an increase in the resistance of the soil against more shear deformation by resulting delay in forming the sliding surface, and increasing the stiffness of the soil. Also, Figure 5-left depicts contours of $\varepsilon_{yy}$ on deformed mesh and Figure 5-right shows contours of the upheaval displacement, both at 0.4 cm of vertical applied displacement. These pictures well demonstrate the sliding surface in dense sand which complies with the experiment.

Figure 4. (left) Comparing numerical & experimental load-displacement curves for $H/D=1$ and; (right) comparing the same curves for different ratios of $H/D$, all for dense soil

More over, it is clear that none of the experimental and numerical results of the failure mechanism in dense soil, obeyed the vertical sliding surface theory.

Figure 5. (left) Contours of the vertical strain and, (right) contours of the upheaval displacement at 0.4 cm of displacement in dense sand for $H/D=1$

As it was explained before, for dense soil, uplift resistance force reaches to its peak amount in small upheaval deflections. At this stage, the soil above the pipe has failed and this causes a progressive reduction in uplift force (the descending branch of the curve), to get its residual value in large displacements. Figure 4-right
compares load-upheaval displacement curves for other ratios of H/D, obtained from the numerical analyses. These curves show the amount of deduction in resistance force varies with the ratio of H/D. Numerical results give lesser reduction in the peak resistance load compare to the experimental ones.

5.2. Loose Sand
Figure 6 draws load-displacement curves obtained numerically for the loose soil. All show by increasing the burial depth, upheaval load capacity has increased, but upheaval displacement corresponding to the peak load has decreased. However, numerical results in Figure 6-left gives a stiffer model compare to the experimental one with a steeper initial slope of the curve as well as a higher resistance peak load, but its failure mechanism has been well captured by the FEM results. As Figure 7.right shows, in loose sand, a larger region of deformed soil under circulation mechanism of the soil flow underneath the pipe has created (from beginning of the failure procedure). Figure 6-right compares load-upheaval displacement curves for other ratios of H/D, obtained by the numerical analyses which have the same trend explained for the dense sand. Figure 7-left also pictures deformed mesh at 0.4 cm of upheaval displacement for H/D = 1.3 (deflections magnified).

Figure 6. (left) Comparing numerical & experimental load-displacement curves for H/D=1 and; (right) comparing the same curves for different ratios of H/D, all for loose soil

Figure 7. (left) Mesh deformation and, (right) contour of upheaval displacement at 0.4 cm of displacement in loose soil for the H/D=1.6
5.3. Further Studies
Figure 8-left depicts non-dimensional curves of the upheaval load-displacement variations for different ratios of the burial depth. The load-displacement results are re-plotted normalized by peak load and embedment depth, respectively. There is an excellent agreement between the results for the different embedment ratios, with all showing similar normalized stiffness and that d/H equal to 1% for dense sand and 1.5% for the loose sand. However, there is a bit concern for the correlation of the curves after peak load, specially in dense sand, which is, of course, of some shortcomings data, explained earlier. Also, there is less good agreement between the normalized curves when the results are normalized by pipe diameter (not presented here).

Figure 9. Normalized load-displacement curves (left) for dense sand and; (right) for loose sand

Figure 9. Normalized uplift peak load for various H/D ratios of dense and loose sand

Also, according to Eq. 3, normalized uplift peak load (N_u) has drawn as a function of H/D, in Figure 9, for dense and loose sand. As this figure shows, normalized peak load is increased by increasing the burial depth and it is more in dense sand compare to loose one for an equal depth ratio.

6. CONCLUSION
A series of Finite Element analyses have been described to examine the soil density and the embedment depth ratio on the uplift capacity and corresponding displacement on the failure of a circular buried pipeline subjected to vertical uplift. Finite Element model gave a very good approximation to the system behavior and
was able to reproduce the complete deformation pattern of the system up to and beyond the peak load until total degradation of strength, without major numerical difficulties. The following conclusions have been drawn from this study:

1. Type and mechanism of failure differs in dense and loose sand. In dense sand, only sliding block mechanism with angled surfaces is formed, in small upheaval deformations, while in loose sand, deformation of wide range of the soil above the pipe is happening.

2. Corresponding displacement for the uplift peak load is much more in loose sand compared to the dense one, in the same H/D ratio.

3. Effect of the increase in soil density on uplift capacity is more than the effect of increase in burial depth ratio, which is very important from an economical view.

4. Deduction in peak resistance force capacity to reach the residual force in dense sand is more than the loose one, as a result of different failure mechanisms.

5. Normalized $N_u$ is increased with the embedment ratio, and its peak amount is more in the dense sand compared to loose one, for an equal H/D.

6. FEM results suggest that displacement should be normalized by embedment depth, H, rather than pipe diameter, D.

7. Obtaining some crucial parameters carefully and some more refinement, FEM has potential for use in defining advanced design parameters and rules.

REFERENCES


