ASSESMENT OF SIMPLIFIED PIPE-SOIL INTERACTION MODELS

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Abstract. *In pipeline design it is essential to predict lateral behavior of the pipe-soil interaction due to lateral loads resulting from relative movements or lateral buckling. Current design practice is to substitute the surrounding soil with a Winkler type foundation. The determination of the stiffness of the foundation is generally based on guidelines depending on general soil properties and embedment ratio. This approach has some shortcomings. The computation of force displacement relationship of the springs depends on the accurate determination of the ultimate resistance of the soil as well as associated displacement.The computation of these quantities should take into account the change in collapse mechanism with embedment depth. Moreover Winkler type foundation does not model the actual soil behavior since it neglects the coupling between adjacent springs. The aim of this study is to assess the influence of the above shortcomings by comparing results from simplyfied analysis with three-dimensional models.*

Keywords. *pipe-soil interaction, finite element model, nonlinear analysis*

1. Introduction

Due to the highly nonlinear behavior of soil materials, pipe-soil interaction phenomena and the possibility of pipe distortion, buried pipelines have complex behavior. Two basic methods are used for numerical modeling of buried pipelines: continuum finite element representation of both pipe and soil, and using specialized beam-type finite elements for the pipe and Winkler type representation of surrounding soil.

Current practice is to use the beam on nonlinearly elastic foundation, which is considerably less computationally demanding than full three-dimensional modeling methods (Popescu & Konuk, 2001). Soil continuum is approximated by a series of discrete springs representing independent orthogonal soil deformation components (ASCE, 1984). This simplified model does not capture details of pipe behavior, such as localized bending of the pipe shell, and limitations on shear interaction between different soils zones along the pipe.

The aim of this study is to assess the influence of the above shortcomings by comparing results from simplified analysis with three-dimensional models. Initially two-dimensional FE contact models between a rigid pipe and clayey soil under undrained conditions are used to compute the force-displacement relationships for different embedment ratios. The computed nonlinear soil springs are then compared with the existing guidelines in the literature. Finally full three-dimensional contact models between a deformable cylindrical shell and the soil are analyzed under different loading conditions and compared to the beam on nonlinear Winkler foundation simplification. The commercial finite element code ABAQUS/Standard (HKS, 2001) is used for numerical computations.

2. Force-displacement relationship of soil springs

In order to build a simplified model to analyze a buried pipe it is necessary to determine the constitutive law of the springs that represent the effects of the surrounding soil. In the current practice, the force-displacement relationship of the soil is modeled by standard functions (hyperbolic or bilinear) whose parameters depend on the soil type, soil properties, and embedment ratio (ASCE, 1984). This paper aims to verify the validity of this procedure comparing the prescribed force-displacement curves with the ones obtained by an appropriate two-dimensional FE model of the pipe-soil system. It should be emphasized that the present study deals only with pipes subject to lateral movements (*p-y* springs) and buried in clays under undrained conditions.

2.1. Finite element model

The geometry and the boundary conditions of the two-dimensional FE model representing a typical section of a buried pipeline is depicted in Fig. (1), where D is the pipe outer diameter, H is the distance between the surface and the pipe centerline, and the dimensions B and L are chosen in order to simulate an infinite distance. The pipe cross section was modeled as a rigid body with analytic geometric description (HKS, 2001), while plane strain condition was adopted to describe the soil behavior owing to the large pipeline length. Quadratic (8-node) elements were used in the numerical discretization. A constant diameter ($D = 0.3\ m$) was used in all examples presented in this work.

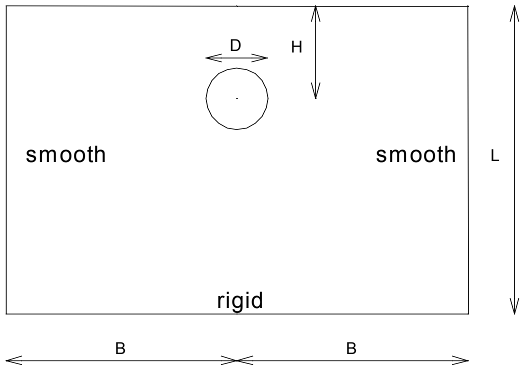

Figure 1. Geometry and boundary condition of the two-dimensional FE model.

In order to obtain an accurate description of the pipe-soil interaction phenomenon, the model includes several sources of nonlinearities. The most obvious is the elasto-plastic behavior of a clayey soil. The undrained condition was adopted, since pipe buckling and ground movements result in fast loadings. The soil properties used here were $c_u = 50\ kPa$, $E_u = 100\ c_u$ and $\nu = 0.48$ (to simulate the incompressibility condition). The relative movement and the friction between the pipe and soil ($\mu = 0.2$ and $\tau_{max} = c_u/3$) is considered using the contact surface approach (HKS, 2001). Finally, geometrically nonlinear effects due to large displacements of the soil close to the pipe are also included.

As in the literature (ASCE, 1984), the embedment ratio is described by the parameter *H/D* and the pipe weight is neglected. The geostatic stresses (vertical and lateral) were not included in the current model, leading to the immediate breakaway of the soil behind the pipe. This procedure was adopted here because the available expressions for the stiffness of clays (ASCE, 1984) do not include the soil weight. It should be noted that this model underestimates the soil stiffness, which is a conservative approach for the analysis of buckling problems. However, for pipelines subjected to differential ground movements the situation is exactly the opposite. Therefore, in the latter case it may be interesting to adopt the condition of no breakaway between the soil and the pipe, which simplifies the model eliminating the contact constraints. Obviously, if the pipe weight and the geostatic stresses are known, they can be easily included in the FE model, leading to a more realistic response.

Applying a prescribed horizontal displacement and computing the corresponding support reaction, the load-displacement curve of the pipe-soil system can be determined. Using this procedure a parametric study was performed to establish the influence of the embedment ratio over the soil reaction, and the computed results from *H/D = 1* to *6* are depicted in Fig. (2). It can be noted that two different loading processes were considered. In the first one the vertical displacement is kept fixed, while in the other the vertical displacement is free. According to this figure, both cases lead to a similar behavior. Thus, for small embedment ratios (shallow cover) it can be seen that the soil reaction presents a well-defined plateau (rupture load), while for large ratios (deep cover) the reaction continue to increase even for very large displacements. Therefore, in the latter case it is necessary to use some additional procedure in order to define a consistent ultimate load. However, the figure clearly shows that the rupture load increases with embedment ratio until *H/D = 4*, and that for greater ratios there are no significant increase in the soil resistance. Finally, the load-displacement curves indicate that transition between shallow and deep cover occurs for *2 < H/D < 4*.

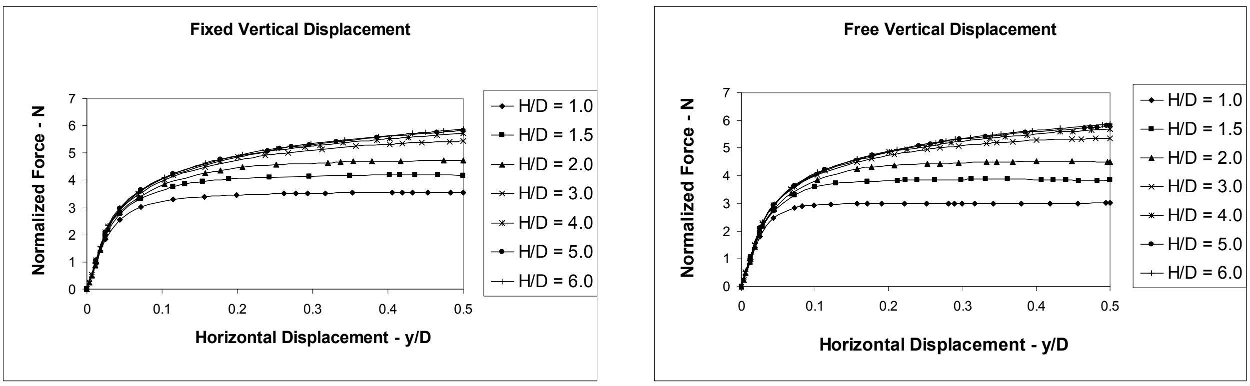

Figure 2. FE computed load-displacement curves.

The Fig. (3) shows that for deep cover there are no significant differences between the soil reaction for fixed and free vertical displacements. However, the converse occurs for shallow cover, where the condition of fixed vertical displacement leads to a higher ultimate load. Obviously, is up to the engineer to decide which loading case is closest to the field conditions.

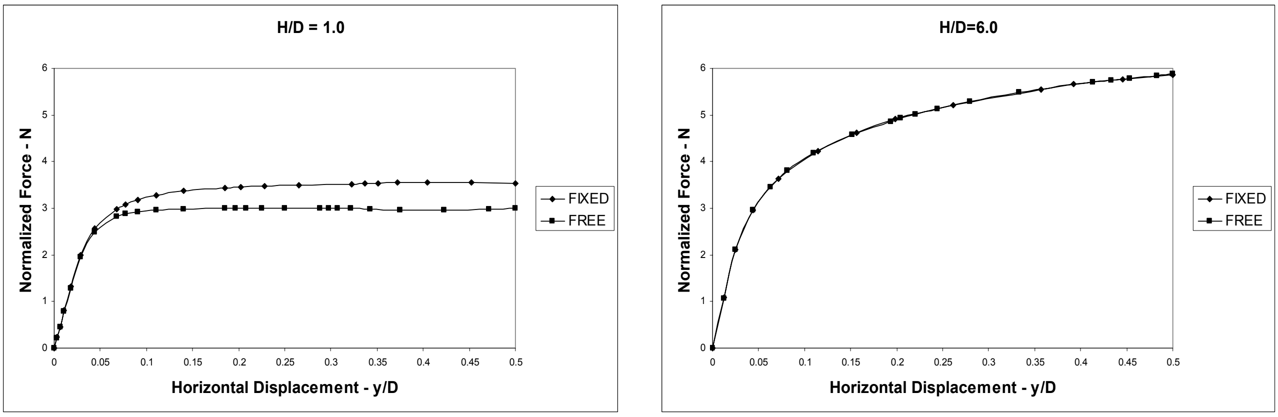

Figure 3. Load-displacement curves for shallow and deep cover.

The differences between the shallow and deep pipe is due to the different failure mechanisms involved in each case. As shown in Fig. (4), the soil failure for a shallow pipe is due the formation of a small plastic zone between the pipe and the ground surface, with the soil rupture occurring when the plastic zone reaches the surface. It can be noted that large vertical displacements occur in the ground surface. On the other hand, for deep pipes the failure is due to the plastification of a large region in front of the pipe, which is a typical example of contained plastic strains leading to ever increasingly loads. For deep pipes no noticeable vertical displacements occur in the ground surface. These different failure mechanisms explain the similarities and differences between the conditions of fixed and free vertical displacements. The same failure mechanisms also occur for vertical anchor plates buried in clays (Rowe & Davis, 1982).

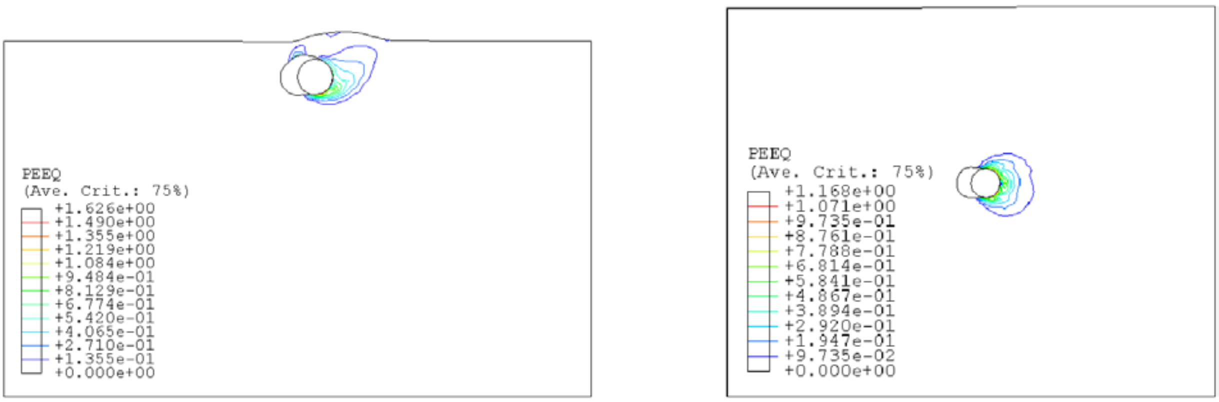

Figure 4. Plastic strains for $H/D = 1$ and $H/D = 6$.

2.2. Comparison with current practice

The current design practice is based on the use of simple formulas to compute the load-displacement (p-y) curves of the soil springs. According the recommendations of ASCE (1984), a hyperbolic relationship should be used:

$$p = \frac{y}{A + B\,y}; \quad A = \frac{0.15\,y_u}{p_u}; \quad B = \frac{0.85}{p_u} \tag{1}$$

In the expression above, p_u is the ultimate load (per unit pipe length) and y_u is the displacement where the ultimate load occurs. Therefore, $p = p_u$ for y greater than y_u. The ultimate load can be computed from the expression:

$$p_u = c_u\, N_{ch}\, D \tag{2}$$

where c_u is the undrained shear strength and N_{ch} is the dimensionless bearing capacity factor. The ASCE procedure adopts the bearing capacity factors determined for vertical piles horizontally loaded (Hansen, 1961). Finally, this procedure recommends using a y_u value between 3% to 5% of $(H + D/2)$.

Table (1) shows the bearing capacity factor (N_{ch}) given by the ASCE procedure and the factor computed by the FE model presented in this work. It can be seen that the ASCE recommendations strongly overestimate the soil résistance.

This occurs because an overestimation of the soil stiffness is the conservative approach for the design of pipes subject to ground movements, which is the main objective of the ASCE procedure. In order to obtain a better estimate of the ultimate load, Popesku and Konuk (2001) suggest to use the bearing capacities presented by Rowe & Davis (1982) to vertical anchor plates. The feasibility of this procedure is confirmed by the data presented in Tab. (1), which shows good agreement between the Rowe & Davis and present results.

Table 1. N_{ch} for shallow cover.

H/D	ASCE(Hansen)	Rowe & Davis	Present Work
1.0	5.0	3.5	3.54
1.5	5.5	4.3	4.18
2.0	6.0	4.8	4.70

The ultimate displacement controls the shape of the load-displacement curve of the soil. As consequence, it controls the stiffness of the soil springs, with smaller values corresponding to stiffer soils. However, to estimate this parameter is much more difficult than to estimate the ultimate load, since experimental data shows a large scatter and the recommended range is very wide. This fact is clearly shown in Fig. (5), where it can be seen that the load-displacement curve computed in this work is outside the curves generated using the recommended upper and lower bounds, and only using a much larger ultimate displacement, a good agreement is obtained. On the other hand, the present procedure does not show this drawback, since the only parameter defining the soil stiffness is the undrained Young's modulus (E_u) of the soil, which can be determined by undrained triaxial tests (Wood, 1990). Therefore, the present methodology is a useful tool for the analysis of buckling problems, where good estimates of initial stiffness are essential to obtain accurate results. It can also be used to obtain not too conservative estimates for the case of pipes subjected to differential ground movements.

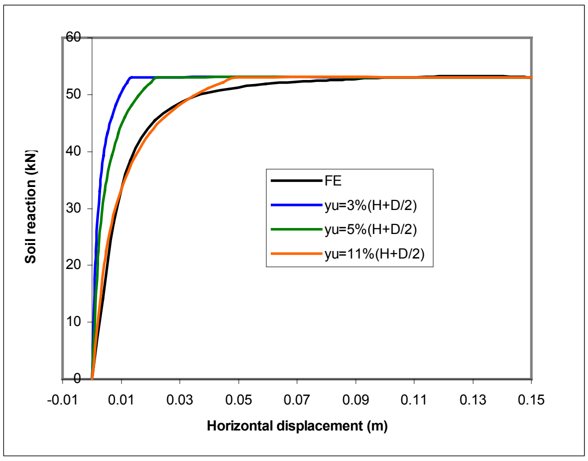

Figure 5. FE x ASCE load-displacement curves for *H/D = 1*.

3. Analysis of buried pipe

In order to asses the accuracy of the traditional beam on nonlinear Winkler foundation model an example of a buried pipeline subjected to concentrated lateral loads is analyzed using a three-dimensional finite element model. A beam model is then constructed using the spring force/displacement relationships developed in the previous section. The obtained displacements and stresses are then compared.

3.1 Problem description

Consider an infinitely long pipe shown in Fig. (6) subjected to concentrated horizontal forces F=420kN spaced at 12m. The pipe has an outside diameter of 300mm and shell thickness of 9.5mm. Concentrated horizontal forces are applied through ring beam stiffeners with outside diameter of 600mm, thickness of 19mm, in order to avoid localized bending of the pipe shell. The material is linearly elastic with $E=2.1\times10^5$ MPa and $\nu = 0.3$. The surrounding soil is a clay with c_u = 50 kPa, E = 100c_u.

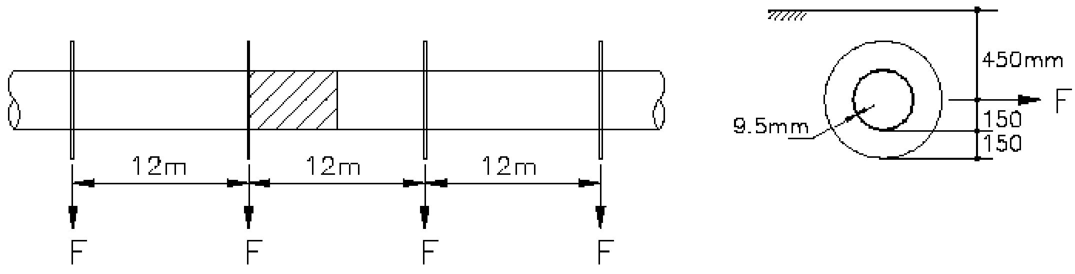

Figure 6. Buried pipeline.

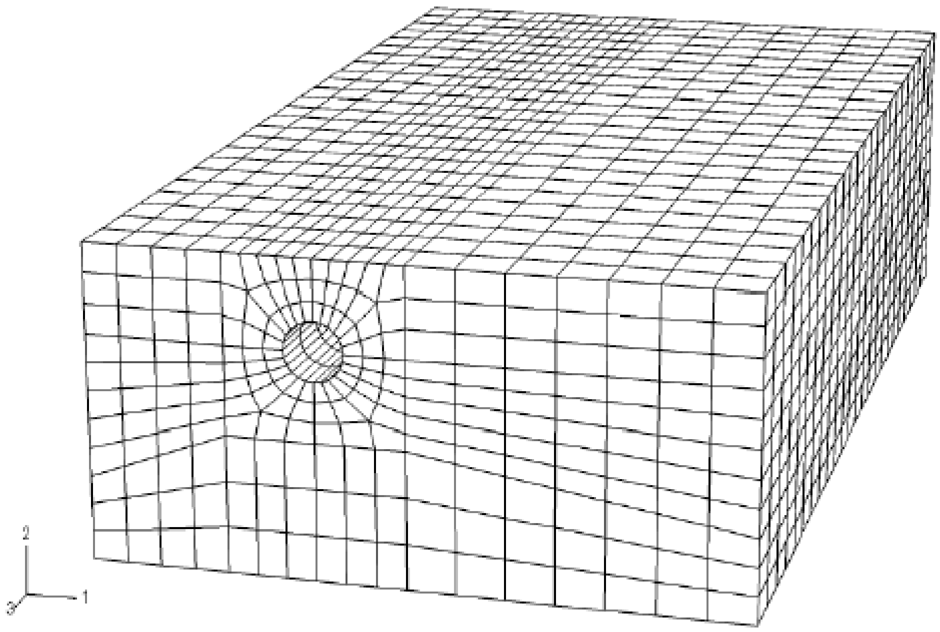

Figure 7. Discretization mesh.

3.2 Three-dimensional model

Because of symmetry only the 6m long shaded segment shown in Fig. (6) needs to be analyzed. The pipeline is modeled as a thin cylindrical shell. Linear quadrilateral reduced integration shell elements, ABAQUS (HKS, 2001) S4R5 elements, are used throughout.

An elastic-plastic material is used to model the soil as a clay under undrained conditions. To simulate near incompressible material behavior Poisson module is taken as 0.48. Soil medium is represented by a parallelepiped 1.6m in depth and 3.3m wide. The distance from pipe axis to edge on the loaded side is 2.15m. Fixed boundary conditions are used except on symmetry planes and soil surface. It is modeled with fully integrated linear hexahedron elements, ABAQUS C3D8 elements. Attempts to use reduced integration met with hourglass singular modes, even with the ABAQUS provided control. Quadratic elements may also cause problems due to possible directional inconsistencies between applied pressures and nodal forces. A view of the mesh used to discretize to soil medium is shown in Fig. (7).

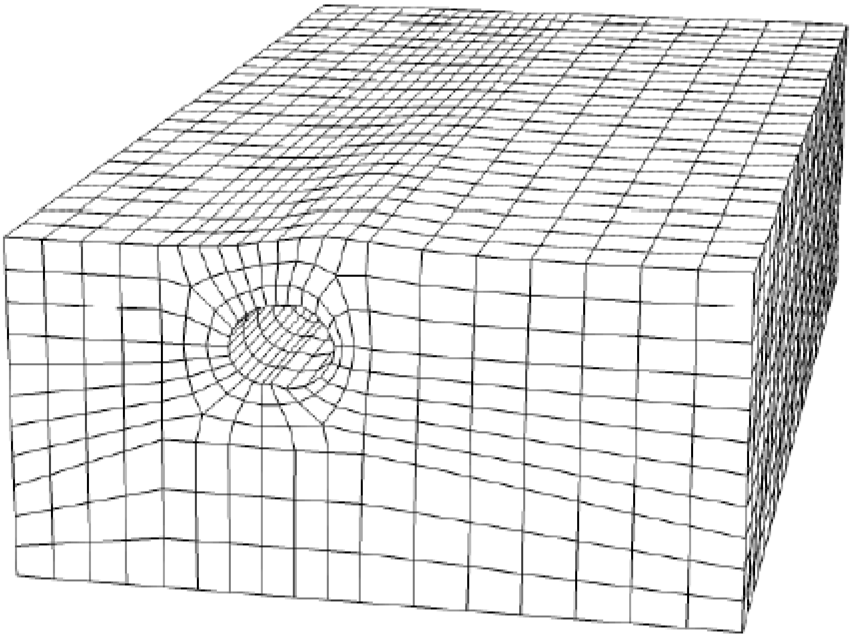

Figure 8. Deformed shape of soil medium.

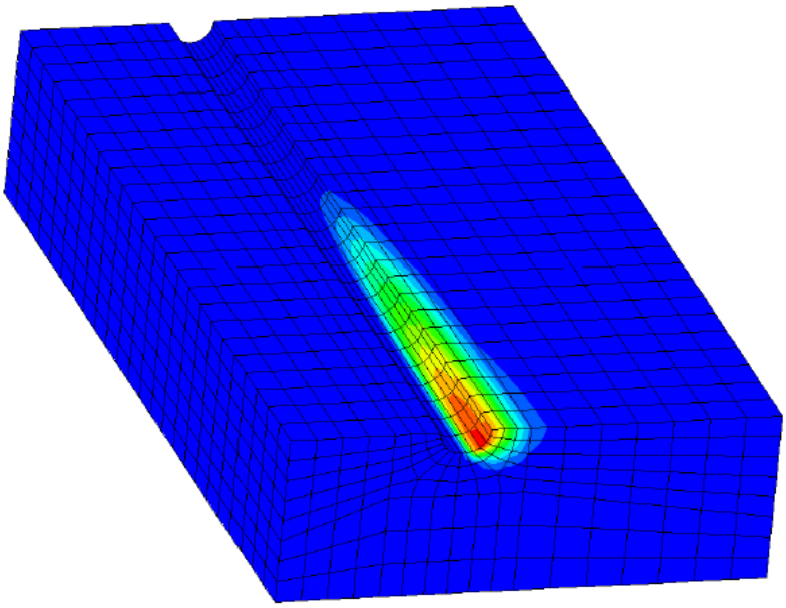

Figure 9. Soil plastic strains.

Pipe-soil interaction is modeled using a frictional contact model with an equivalent shear stress limit. The shear stress limit at the pipe-soil interface is $\tau_{max} = c_u/3$. Geometrically nonlinear effects are taken into account. The analysis is performed in two steps. The first step is used to induce contact between tube and soil. A small force F=1kN is applied while restraining rotational degree of freedom about pipe axis. At the second step additional force of 419kN is applied and the initial rotational restraint about pipe axis is removed. The first step is necessary in order to avoid unrestrained rigid body rotation. Total applied force at 12m intervals is then F=420kN.

Deformed shape of soil medium is shown in Fig. (8). It can be seen that a noticeable hump is formed on the soil surface in front of the pipe. This is characteristic of shallow problems. From the deformed shape of the cylindrical shell it can be concluded that the vertical component displacement of displacements is small. This is due to long embedment of the pipe in soil. Plastic deformations propagate about 2.5m to both sides of the applied load, as shown in Fig. (9). Maximum horizontal displacement of 4.831×10^{-2}m occurs at the stiffened section.

Maximum longitudinal stress in the pipe shell is 505.4MPa Maximum membrane longitudinal section force is 4410kN/m. If uniform longitudinal normal stress through shell thickness is assumed one would get a value of 464.2MPa. The difference is due to shell bending that results from contact soil pressures.

3.3 Beam model

It is customary to model pipe/soil interaction using beam elements to represent the pipe and spring elements to represent the soil (ASCE, 1984). This is a very convenient model with respect to computational time which is a very small fraction of that required by the full three-dimensional model. The shortcomings arise from the inability of the Winkler type foundation to model the coupling between adjacent springs.

In the present case ABAQUS PIPE31 beam elements are used to model the pipeline. Lateral restraint of the soil is modeled using PIPE-SOIL interaction elements whose stiffness are derived from the two-dimensional force/displacement relationships of the previous main section. Since in this case the vertical displacements of the pipe is small the vertically fixed spring stiffness is used. The model used nodal spacing of 0.30m, the pipe diameter.

A maximum displacement of 5.034x10-2m is obtained. This represents a 4.2% difference when compared with the three-dimensional model. In fact, deformed shapes of the pipe in the two models are very similar, as shown in Fig. (10). Local bending of the pipe shell, neglect of longitudinal frictional contact as well as spring decoupling may be listed as some of the factors that account for the small difference.

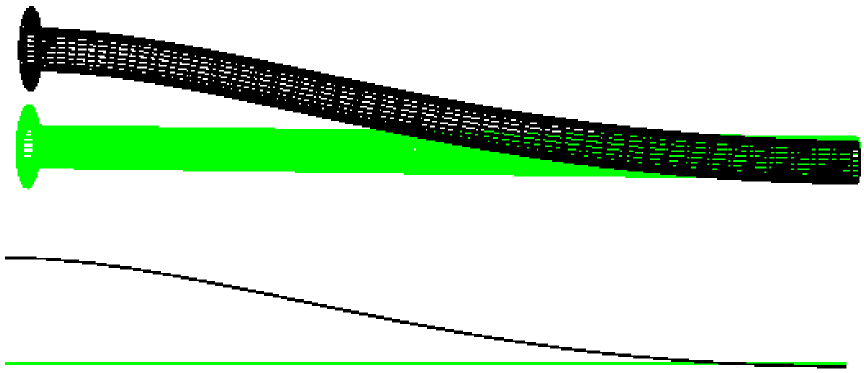

Figure 10. Pipeline deformed shape for both models.

Maximum longitudinal membrane force is 4035kN/m which compares well with the three-dimensional model, representing a difference of 9.3%. Maximum longitudinal normal stress of 424.7MPa is 16% lower than in the three-dimensional model, which is mainly due to localized bending of the pipe shell.

4. Conclusions

In pipeline design it is essential to predict lateral behavior of the pipe-soil interaction due to lateral loads resulting from relative movements or lateral buckling. Current design practice is to substitute the surrounding soil with a Winkler type foundation. The determination of the stiffness of the foundation is generally based on guidelines depending on general soil properties and embedment ratio.

In order to assess current guidelines for computing soil lateral spring load/displacement curves, two-dimensional finite element models including soil plasticity, pipe-soil frictional contact and large displacements were analyzed. It is concluded that current guidelines overestimate the soil stiffness and ultimate load. This is easily explained because the guidelines are primarily intended for seismic problems that involve differential ground movements. In this case it is conservative to use stiffer soil springs. The converse is true for buckling problems. It is recommended to use bearing capacity factors due to Rowe and Davis (1982) instead of the ASCE (1984), which results in close agreement with the present study. However as the ultimate soil displacement is difficult to estimate it is best to run a two-dimensional finite element analysis that provides the load/displacement curve directly. It was found that it is also important to differentiate between two type of behavior: shallow versus deep pipe. For shallow pipes, $H/D \leq 2$, there is a significant difference between force/displacement curves corresponding to fixed or free vertical displacements. In that case the curve to use is problem dependent.

As the simplified model uses beam on Winkler type foundation it is important to assess the significance of pipe distortion and uncoupling between adjacent springs. For that purpose an example of a buried pipe subjected to concentrated lateral loads was analyzed using a beam and a three-dimensional model. In the three-dimensional model solid elements are used to discretize the soil and shell elements are used to discretize the pipe. In the beam model lateral spring stiffness are based on force/displacement relationships obtained form two-dimensional models. Comparing obtain results it can be concluded that the two models have close agreement in displacements, with maximum difference of 4.2%. Stress resultants are also in good agreement with a maximum difference of 9.3% in longitudinal membrane force. Stress values are not so close with a maximum difference of 16% owing to localized bending of the shell due to contact pressures.

It can be therefore concluded that as long as the soil spring stiffness are accurately computed from two-dimensional finite element analyses the beam model yields very good estimates of displacements and reasonable stress values, which are sufficiently accurate for practical pipeline design.

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