Numerical analysis of soil-pipeline interaction phenomena in unstable slopes

Etude des phénomènes d'interaction entre conduites et sol eu pentes instables

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ABSTRACT: This paper deals with the numerical aspects of an extensive research programme aimed at the study of interaction phenomena occurring between a pipeline and the surrounding soil when the pipeline crosses slowly deforming slopes. A numerical analysis of the soil-pipe interaction problem is performed using a finite element discretisation of the pipe and a non linear spring-type model for representing soil response. The geometry of the problem is treated as fully 3D and the interaction forces arising at the soil-pipe interface are set to satisfy the pipe-soil interaction patterns as obtained from past research work involving field and laboratory testing. By imposing a slope deformation field as boundary condition, the model gives the stress distribution in the pipe as it results from the complete solution of the soil-pipe interaction problem. The stress distributions obtained through the model are compared with those inferred by monitoring strains along the pipe.

RÉSUMÉ: Cet article traite des aspects numériques dans un vaste programme de recherche adressé à l'étude des phénomènes d'interaction entre conduites et sol, lorsque celles ci traversent des pentes argillaires, qui se déforment lentement. L'analyse numérique du problème d'interaction se développe par une représentation du tuyau aux éléments finis et du terrain avec des ressorts à comportement non-linéaire. La géométrie du problème est traitée en 3D et les forces d'interaction tuyau-sol réfléchissent les résultats d'expérience sur le comportement de l'interface. En imposant l'état de déformation de la pente comme condition, le modèle présente la distribution des efforts dans le tuyau. Celles-ci sont comparées avec les distributions obtenues par mesures de déformation exécutées directement sur la conduite.

1 INTRODUCTION

Pipelines for the transportation of gas, oil, liquid fuel, etc. must sometimes cross unstable areas, where slow slope movements are taking place. Soil displacements will produce strains and therefore stresses in the pipeline; these will depend on various factors, such as relative position with respect to the direction of movement, dimension and materials used for pipeline and coating, backfill material, etc.. Correct design requires prediction of the stresses in the pipeline, in order to assess the risk of breakage and plan monitoring activities.

Existing guidelines for calculations draw from an analogy with pile skin resistance evaluation (CGL, 1984). However, several important differences can be pointed out: axial symmetry does not exist for pipelines, while actual behaviour can be very much influenced by the installation method adopted; pipelines usually lie near to the ground surface, where the soil has very different properties from that surrounding a driven pile; on the other hand, in the case of pipelines, the use of more accurate models is possible since the geometry and material properties are much better known.

Prediction of stress transmitted to pipelines is closely related to correct quantification of the shear strength at the pipe-soil interface. This has been usually done in the past by means of the so-called α method, which is a total stress approach. It is mainly an empirical method, in which the shear resistance at the interface is expressed as:

$$f_s = \alpha c_u \tag{1}$$

where c_u is the undrained strength and α is a parameter which takes into account various physical factors and disturbances. The method has been mostly developed for clayey saturated soils, and has proven quite satisfactory for evaluating skin resistance of driven piles. Good results depend upon a correct evaluation of c_u, and the choice of an appropriate value for α; these are usually obtained by tables existing in the literature, where α values are determined through full-scale field tests on experimental piles. The basic assumption is that ultimate conditions are reached in undrained conditions, which is normally the case for piles driven at great depths.

Some problems arise however when attempting to apply the same method to pipelines. In fact, the assumption of undrained failure is questionable near the ground surface, and though it may be valid for some types of loadings which influence pipelines, such as for example those resulting from an earthquake, it is certainly not true with respect to slow slope movements, where drained conditions are almost certainly reached. In any case, correct evaluation of c_u is much more difficult in the case of pipelines, due to the types of soil involved and the disturbance from positioning. Furthermore, choosing an adequate value of α becomes very uncertain, since existing tables report empirical values which have been determined for piles, and therefore in completely different circumstances.

It is clear therefore that for pipelines it is necessary to use a model based on effective stresses; such is the so-called β method (Burland, 1973), also developed initially for pile skin-resistance evaluation. In this method the general expression of shear strength at any depth is:

$$f_s = c_a + \sigma'_n \,\mathrm{tg}\delta \tag{2}$$

where c_a is a cohesion term, σ'_n is the normal effective stress and δ is the interface friction angle; in the case of piles the normal effective stress is equal to the horizontal effective stress at the same depth σ'_h , which is commonly expressed as a function of the vertical effective stress σ'_v, through the parameter k:

$$\sigma'_n = k \cdot \sigma'_v \ . \tag{3}$$

In this method, neglecting the term c_a, under the assumption that the friction contribution to strength is much greater than any cohesion mechanism, values of $\beta = k \cdot \mathrm{tg}\delta$ are obtained empirically, through back-analysis of the behaviour of instrumented piles.

Again, the method cannot be directly applied to pipelines, since in this case the mean value $0.5(\sigma'_v+\sigma'_h) = 0.5(1+k)\sigma'_v$ must be attributed to the normal stress σ'_n, so that the parameter β, apart from having been determined in different situations, also has a different physical meaning.

In conclusion, it can be seen that the methods developed for piles for skin resistance evaluation offer guidelines which are certainly useful, but cannot be simply transferred to pipelines without modification. Such direct applications have been done in the past, and in fact a great number of existing pipelines have been thus designed, but it is clear that only a very approximate estimate of strength is obtained.

The Authors are involved in a research project in which the aim is to develop specific methods for pipelines, regarding determination of longitudinal shear strength and evaluation of interaction mechanisms. Extensive experimental surveys have been carried out, including both laboratory and on site pull-out tests, through which insight has been acquired into the behaviour of soil-pipeline interaction at the interface and mechanisms of stress transfer. In particular, a scheme for calculating longitudinal strength was arrived at by means of experimental procedures, in which the test results are interpreted in the light of an effective stress general model (Scarpelli et al. 1995, Cappelletto et al. 1998, Scarpelli et al. 1999).

In this paper a numerical model is proposed, comprising a discrete spring model for soil-pipeline interaction and finite element analysis for the pipeline itself. The model is applied to a real situation, and the results are here presented.

2 THE NUMERICAL MODEL

In the numerical model here implemented the pipeline is discretised into linear elastic beam elements, while the surrounding soil is represented by a 3D series of mutually orthogonal springs.

The model operates by assigning a known displacement field to the surrounding soil, and hence to the nodes of the springs. By means of assigned interaction laws, some part of this kinematics is transferred through the spring onto the pipeline nodes, by means of the mobilised strength:

$$f_s = k_s \cdot s \tag{4}$$

where s is the relative displacement and k_s is a transfer coefficient; finite element analysis is then employed to compute stress and deformation fields in the pipeline. The transfer coefficient k_s is assigned separately in each direction, and can be a general non-linear function. In this work two types of transfer function have been considered: a hyperbolic law for the transverse direction and an exponential law for the longitudinal direction. Particular attention was given to the latter, considering the greater importance of longitudinal interaction. An exponential law appears to give better results since it can correctly reproduce non-linear behaviour in a wide range of strain values (Furlani 1999). The expression adopted is:

$$f_s(s) = f_{s\,\lim}\left(1-\exp\left(-\frac{k_{s\,\max}}{f_{s\,\lim}}\cdot s\right)\right); \tag{5}$$

it requires two parameters, ultimate strength on the interface $f_{s\,\lim}$ and initial stiffness $k_{s\,\max}$. Ultimate strength is given by:

$$f_{s\,\lim} = 0.5\left(1+k\right)\gamma' H \tan\delta \tag{6}$$

where k is assumed to coincide with the "at rest" value $k_0 = 1 - \sin\varphi'$, while initial stiffness $k_{s\,\max}$ can be expressed in terms of known elastic parameters of the medium at small strains; in particular (e.g. Scott 1981):

$$k_{s\,\max} = \frac{G_{s\,\max}}{4\left(1-\nu\right)\dfrac{D}{2}} \tag{7}$$

where G_s is the soil shear modulus at small strains, ν is Poisson's ratio and D is the pipe diameter.

Regarding transverse interaction, the hyperbolic law and the parameter values used have been chosen on the basis of known solutions which can be found in the literature (e.g. Trautmann & O'Rourke 1985).

The solution is obtained through an explicit iterative step-by-step procedure, in which the global stiffness matrix and the geometry of the pipeline are adjusted at each time-step, thus taking into account large deformations.

The governing equations in incremental form, derived through 3D equilibrium conditions, are the following:

$$[\mathbf{K}_s]\cdot\{\delta u\}+\lfloor\mathbf{K}_g\rfloor\cdot\{\delta u-\delta u_g\}=\{0\} \tag{8}$$

where $\mathbf{K}_s$ and $\mathbf{K}_g$ are the stiffness matrices of the pipeline and soil respectively, and δu and δu_g are arrays of global displacements of the pipeline nodes and the surrounding soil respectively. The difference $\{\delta u-\delta u_g\}$ therefore denotes the soil-pipe relative displacement vector.

Rearranging Equation 8 we obtain:

$$\lfloor\mathbf{K}_s+\mathbf{K}_g\rfloor\cdot\{\delta u\}=\lfloor\mathbf{K}_g\rfloor\cdot\{\delta u_g\} \tag{9}$$

in which the vector $\{\delta u_g\}$ contains known input values for soil displacements, while solution of the system gives $\{\delta u\}$, vector of pipeline nodal displacements.

Input values for each time-step are supplied in the form of velocity values at the spring nodes. These velocity fields are derived from monitoring data, and it is the correct definition of these values which mostly determines the validity of the analysis. It is therefore essential to have access to monitoring data, that should include measured values of displacements at various depths (in particular at the depth of the pipeline) and their variation in time, such as those that can be obtained through inclinometer profiles.

Another use of the proposed model is to estimate the effects of mitigation remedial works, such as stress-relief excavations or, in more severe cases, cut and re-connection of the pipeline. These are modelled by setting stress components transmitted by the soil equal to zero in the case of an excavation, and also internal moment, shear and normal stress when the pipeline is cut.

Output consists of nodal displacements along the pipeline, through which internal stress and strain fields can be calculated. Reference to monitoring data can be made by comparing the calculated values for internal axial force N with readings of extensometer data.

3 APPLICATION TO A REAL SITUATION

The model has been applied to the case of a real unstable slope, located in the vicinity of the town of Pesaro in Central Italy; it is an interesting case for study, as it has been monitored for many years since the pipeline burst in 1987. A plan of the site is shown in Figure 1; the pipeline follows the slope from North to South down to the Storena stream, and then uphill on the other side. It was originally installed in 1971, and was not monitored until 1987, when it broke close to the stream. Remedial works were then performed, the pipeline was excavated almost completely and instruments installed; this can reasonably be assumed as a new zero for stresses in the pipe. In January 1997 some sections were once more excavated, on the basis of monitoring data which indicated some excessive stress. In the present work data of one year of monitoring have been used, namely between September 1996 and October 1997.

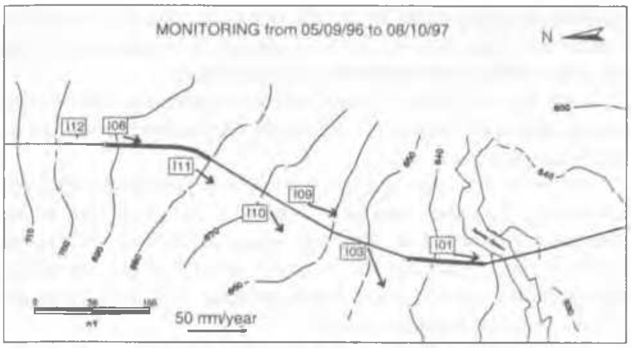

Figure 1. Plan of site and mean values of horizontal displacements.

3.1 *Monitoring and instrumentation*

Existing data consist of inclinometer readings at points near the pipeline, and extensometer readings at various sections from which data regarding strain and stress fields in the pipeline can be derived (Fig. 2). It is clearly seen that inclinometer I12 at the top of the slope is not moving; this gives a geometrical upper boundary condition for the pipeline, while the lower boundary of the unstable area is the Storena stream at the foot of the slope. Another view of inclinometer data is given in Figure 1 where the mean values of horizontal displacements throughout the year are plotted; here it is clear that, although the pipeline runs parallel to the maximum slope gradient, displacement vectors do not follow at each point, so that there is a perpendicular component loading the pipeline. Data of three extensometers is also shown in Figure 2 in terms of normal stress; the jump in all readings corresponds to the stress relief excavation remedial works performed in January 1997. It should be noted that the jump is substantial (in the order of 100 MPa), while after the excavation the trend seems to be that of returning to the previous compressed state.

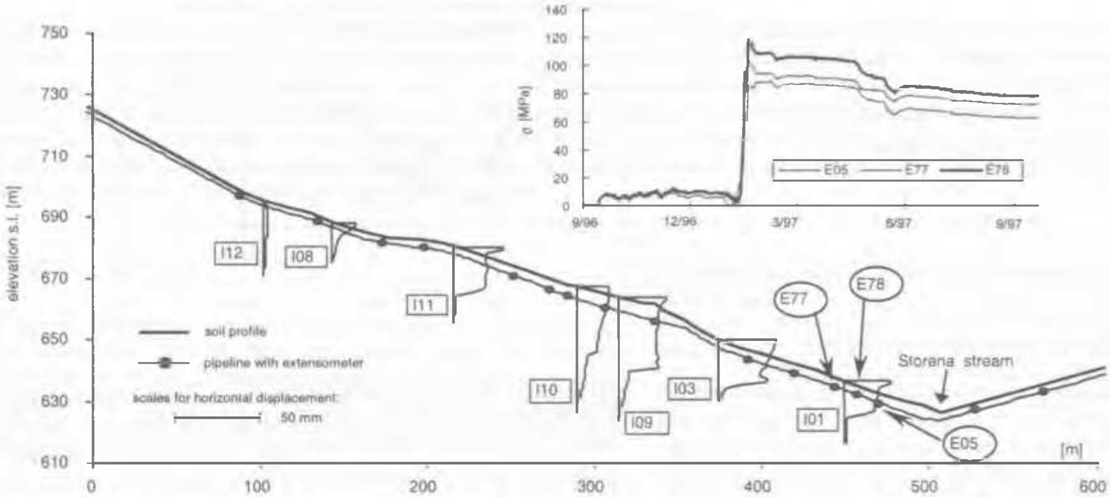

Figure 2. Inclinometer and extensometer data.

3.2 *Modelling and results obtained*

The pipeline, which was modelled for a length of 820 m, was divided into 360 finite elements; these are about 1.50 – 2.0 metres long in the unstable region. The section embedded in the stable part of the slope is 200 m long; it has been seen that this length is sufficient for providing a sound restraint. The pipeline cross-section is of 66 cm diameter and 11.13 mm thickness; Young's modulus is E=205000 MPa. Regarding the surrounding soil, the following parameter values were assumed: unit weight $\gamma = 19$ kN/m^3, friction angle $\varphi' = 30°$, maximum shear modulus $G_{s\,max} = 10$ MPa. The pipeline coating is tar, so, following results of full-scale pull-out tests and laboratory tests (Furlani 1999), the friction coefficient for longitudinal interaction has been assumed equal to tan(0.8).

Input data, consisting of soil velocity values, are obtained from readings of seven inclinometers (cf. Fig. 2) as follows: for each inclinometer, the reading at the appropriate depth was assigned to the closest spring node; for the remaining spring nodes, input values were calculated through linear interpolation between the two nearest inclinometers.

Timesteps of variable duration were used; 10 days is a typical value for analysis, while larger steps, increasing in linear progression, were adopted to cover longer time intervals.

To simulate the excavation, the stresses produced by the soil on the pipeline are removed gradually, in 40 increments.

It is important to note that at the beginning of the period for which input data is available (September 1996) the pipeline had already undergone stress-deformation after the total stress relief operations in 1987. To account for this, and to calculate an initial stress state at September 1996, the mean displacement vectors obtained in one year of monitoring were applied each year from 1987 to 1996, and the resulting stress-strain states were calculated by means of the time-stepping scheme.

Output values consist of pipeline displacements, mobilised shear strength, axial stress, and transverse bending moments. Of these, the axial stress is the most significant, since it can be easily compared with actual extensometer data.

In Figure 3 the calculated mobilised shear strength at a node contained in the segment which was dug out is shown; correct interpretation of applied loading and unloading can be noted.

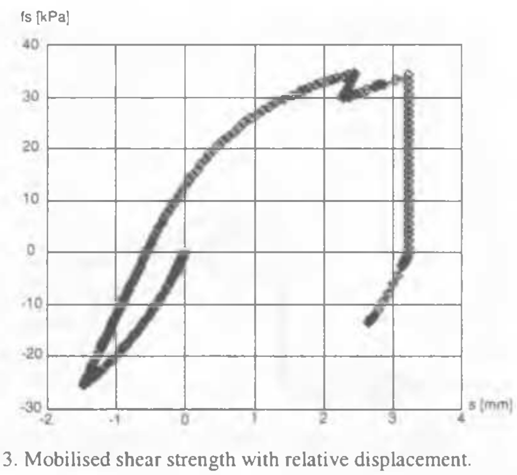

Figure 3. Mobilised shear strength with relative displacement.

Stresses and other output of the model seem realistic, and do qualitatively reproduce the real situation. In Figure 4 three plots of the axial force along the pipeline at various times are shown; the effect of the excavation can be clearly seen: axial stress drops, and an evident subsequent trend towards values before the excavation is clear. The calculated stress state before the excavation correctly indicates the most critical section of the pipeline.

Comparison with extensometer data is shown in Figure 5. Qualitative agreement is satisfactory, and in particular the actual stress trend to return to previous values after the excavation is correctly captured.

4 CONCLUSIONS

Soil-pipeline interaction is an extremely complex phenomenon, for which some solution is necessary in order to estimate stress parameters for efficient pipe design. In this paper the particular case of buried pipelines crossing unstable slopes is analysed; soil displacements can deform, and hence stress, the pipeline, and only soil-structure interaction analysis can give a means for prediction of those stresses. Methods currently in use are based on an analogy with pile skin resistance evaluation; these, though useful for giving some indication of the stress levels reached, are not satisfactory.

In the present paper a model for soil-pipeline interaction is proposed, in which the pipeline is discretised into linear elastic beam elements, while the surrounding soil is represented by a 3D series of mutually orthogonal springs. Particular features of the model include the possibility to consider any pipe geometry and non-linear interaction laws for soil-pipe interaction, allowing for general non-monotonic loadings; in the application here presented a hyperbolic law was considered for the directions perpendicular to the pipeline, and an exponential law for the longitudinal direction. A time-stepping procedure has been implemented, thus enabling to follow the successive states of the pipeline as these evolve, due for example to progression of the slope movement, remedial works performed, etc.

Input for the model is provided by monitoring data of the slope under observation, in particular inclinometer readings at various depths.

The model has been applied to a real case for which sufficient monitoring data was available. Output of the model can so be compared with real data. The results are satisfactory, in that the model correctly predicts the location of critical pipe sections, and computed internal stress values compare well with those inferred from extensometer readings.

The model here presented can be put to use either to interpret further data from monitoring activities in terms of stress increase at the pipeline critical sections, or to predict the effects on the pipeline of slow slope movements.

The results obtained obviously apply to the specific case only but the proposed approach can be followed in principle in other, similar, situations. What is needed for different applications of the numerical model is knowledge of the pipe-coating characteristics, the nature of the soil fill around the pipe, the pipe geometry and, finally, an estimate of the slope kinematics in terms of displacements and displacement rates.

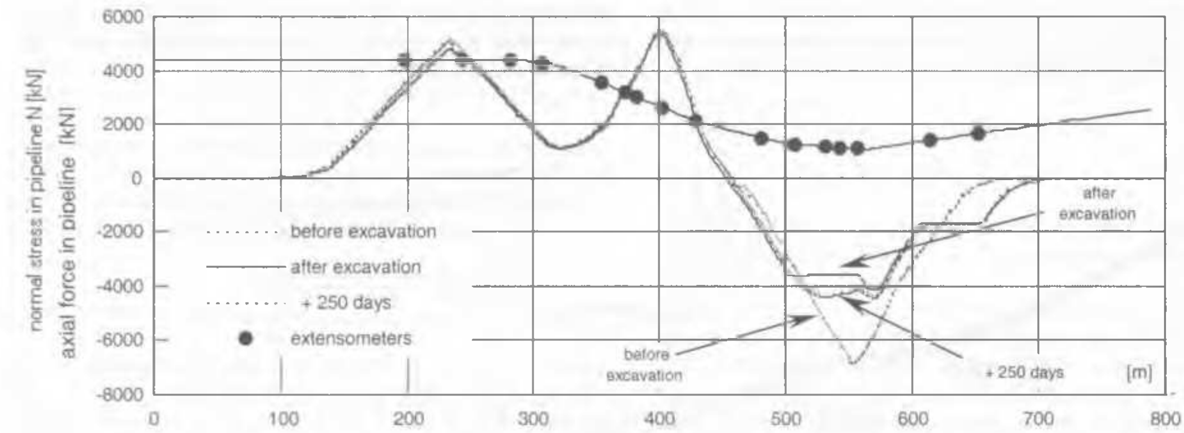

Figure 4. Axial force along the pipeline at various times.

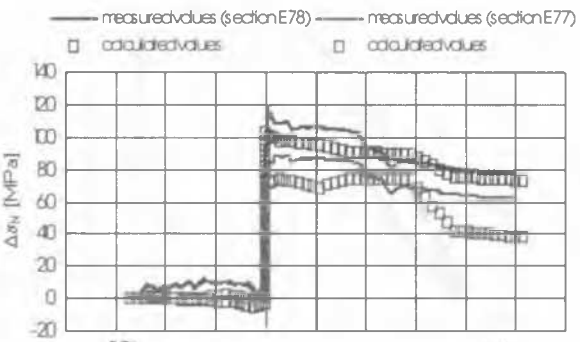

Figure 5. Normal stress change in pipeline: comparison between measured and calculated values.

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