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ANALYTICAL MODEL FOR THE SEISMIC BEHAVIOR OF BURIED PIPELINE WHEN SUBJECTED TO GROUND LIQUEFACTION

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SUMMARY

The assessment of seismic performance of buried pipelines of gas and oil networks encounter a fragment of the principal line of natural gas network and of oil's network when subjected to ground liquefaction. For the analytical model of the buried pipelines it is proposed a discrete model, such as the Winkler, and two distinct analysis: in the first one it is imposed a displacement's history and in the second it is considered an acceleration's history. Through these procedures, several conclusions are stated, mainly related to the influence of the different factors studied on the seismic behavior of buried pipelines: earthquake intensity, effective soil mass coefficient, pipeline diameter and width of permanent ground deformation caused by liquefaction phenomenon.

INTRODUCTION

The seismic behavior and safety of buried pipelines has involved a great deal of attention in last few years. The pipeline systems are often referred to as "lifelines" since they carry materials fundamental to the support of life such as water, sewage, oil, and natural gas. Important characteristics of buried pipelines are that they cover large areas and can be subjected to a variety of geotectonic hazards. They can be damaged either by wave propagation effects or by permanent ground deformations (PGD). Typically the damage is a combination of both hazards. Nevertheless, wave propagation hazards occur in much larger areas but with lower damage rates. Permanent ground deformations include lateral spreading and seismic settlement due to liquefaction, land sliding and surface faulting. These phenomena are commonly limit to isolated regions within the pipeline network, however the potential for damage is very high (i.e., higher pipe breaks and leaks per unit length of pipe).

When subjected to permanent ground deformations (PGD) due to the occurrence of an earthquake, the assessment of the performance of buried pipelines may have great importance not only for the design phase of this type of structures but also in a rehabilitation program. While the seismic behavior of pipes when submitted to seismic waves propagation is significantly well known both on the basis of case analysis and of the results obtained from analytic models, the answer to this buried elements when subject to PGD is more complex, mainly due to:

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- Substantial doubt concerning the possible amplitude and direction of PGD, as well as the length and width of the PGD zone;
- Inadequate use of simple and realistic methods to allow extending sensitivity studies;
- The existing empirical relations to characterize the seismic vulnerability of buried pipes were deduced on the basis of a small number of cases, mainly in comparison to the relations obtained to evaluate the effect of seismic wave propagation.

The need to correctly evaluate the seismic behavior of buried pipelines submitted to high values of ground deformations, with the purpose of clearly evaluate the seismic vulnerability of these structures, requires that analytic studies are carried out in order to characterize adequately the level of damage when this type of phenomenon occur. Among the phenomena, which cause permanent ground deformations, the settlement and lateral spreading induced by liquefaction are considered as the main cause of damage in buried structures [1]. Thus, liquefaction has been adopted in this study as the cause of ground rupture and as the origin of the high values of ground deformations considered.

Numerical modeling of buried pipelines is proposed based upon a discrete model (Winkler type) and two different analyses; in analysis A_1 it was imposed a displacement history obtained by corrected integration of accelerograms, while in analysis A_2 it was used a history of accelerations (accelerograms). It is also important to refer that the in analysis A_2 it was necessary to consider the mass of the soil involving the pipe in order to better simulate the movement of the ensemble soil/pipe as a whole. With this purpose the mass evaluated is concentrated at the level of the extremity nodes of the several elements of the model. The programme used in both analyses is SAP2000 [2].

ANALITICAL MODEL FOR THE BURIED PIPELINES SUBJECTED TO SOIL'S LIQUEFACTION

Buried pipelines modeling

As mentioned previously, to simulate the behavior of buried pipelines it is adopted a discrete model, Winkler type. The models adopted do not reproduce the continuous character of pipeline's involving environment neither takes into account existing three-dimensional effects. Furthermore, they are a function of a parameter (reaction coefficient), which depends of several factors (pipe diameter and length of the considered element). In spite of these constraints associated to the empirical parameter of the model, these numerical methods have had big practical applicability, taking into account that they are easy to use [3]. Opposite to this, they may allow to consider easily the non-linear behavior of the system, by taking into account non-linear empirical curves 'p-u' that relate reaction and ground displacement – Figure 1.

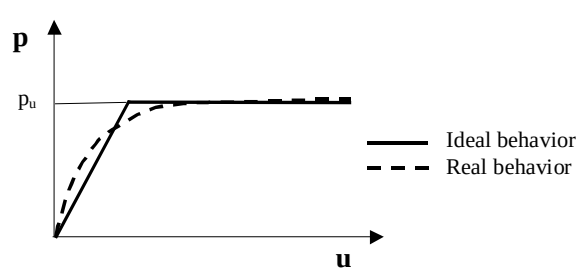

Figure 1. 'P-u' curve based on the elastic perfectly plastic behavior model

Different characteristic analyses are made with the discrete model referred: firstly the seismic action simulated by means of displacements histories, corresponding to the corrected integration of the selected accelerograms, taking into account the effect of the ground damping; and secondly it is considered that the

seismic action is simulated by means of artificially generated or previously chosen accelerograms. In the latter further the damping, both the pipeline mass and the involving ground is considered. After having been conveniently determinate, the involving ground mass must be concentrated at the extremity nodes of each model elements.

The reference model adopted for the analyses performed is a 500 m length pipeline and a liquefaction zone width of 100m. On the basis of these conditions – reference situation – models are studied by varying the pipeline diameter, the thickness of the pipeline wall, the length of the liquefaction zone, the ground rigidity constants so that one could draw some conclusions related to value variation of damage of buried gas and oil pipelines resulting from the occurrence of an earthquake causing ground liquefaction. Figure 2 shows the reference pipeline model where a three distinct zones piping are considered: a central zone (corresponding to the ground liquefaction), a transition zone and the external zones (zones without liquefaction).

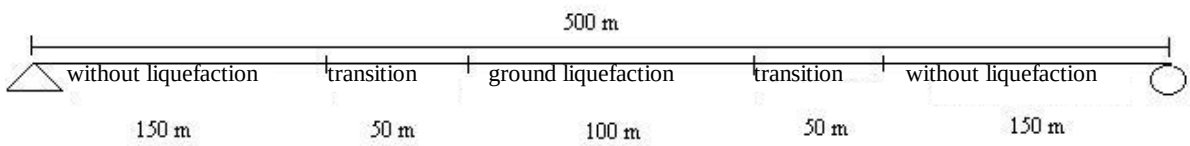

Figure 2. Analyze pipeline zones [4]

Structural model

Though the structure is a pipeline (hollow a cylinder section) it was noticed, based on some sensitivity studies carried out, that formulation based on finite elements such as beams lead too many satisfactory results. The elements' length is presented in Figure 3. For that reason, in all studied models this kind of finite elements are adopted. As referred, the ground is modeled by means of variable rigidity springs placed at the end of each finite element.

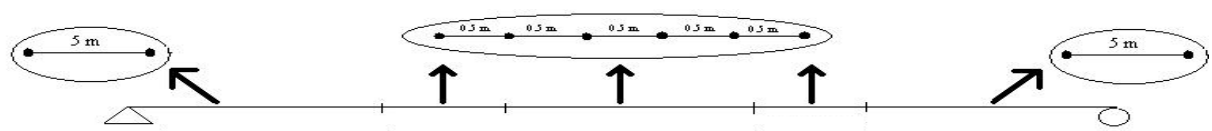

Figure 3. Finite element network: sixty 50m elements and four hundred 0.5m elements [4]

The ground rigidity value initially considered in the example for each zone is determined on the basis of the ground constant used in the calculation of the ground bordering strength in the pipeline's transversal direction [5]. It is considered that this ground constant takes an 11000kN/m^3 value, being designated by k_2. As k_2 corresponds to a reaction coefficient it is necessary to transform it to the corresponding rigidity value. For that purpose, k_2 is multiplied by the corresponding external pipe diameter – ϕ (711.2 mm for gas network and 406.4 mm for oil network). The obtained value, k, is named reaction module and is defined according Equation 1.

$$k = k_2 \, \phi \tag{1}$$

Finally, in order to know the rigidity of each spring (K), placed in each one of the three possible displacement direction at the extremity of each finite element, it is necessary to consider its influence length (Equation 2) To make it easier, it is considered that the rigidity of each spring would take the same value for the three considered directions (x, y and z) in which L corresponds to the influence length of each spring.

$$K = k\,L \tag{2}$$

In the liquefaction zone one assume that the ground, even at the end of the liquefaction process, keeps a residual rigidity value corresponding to about 100 times smaller than the one obtained for elements having the same length in non liquefaction zones. Finally, in transition zones –(between different length elements and between non liquefied/liquefied zones) the spring rigidity is evaluated according to Equation 3. In which k' and k'' correspond to values of the reaction module for each one of the referred zones, being the L' and L'' values of the respective influence lengths.

$$K = k'\,L' + k''\,L'' \tag{3}$$

Having determined the ground rigidity values it has been possible to build the base structural model [2] used in both analyses and shown in the Figure 4.

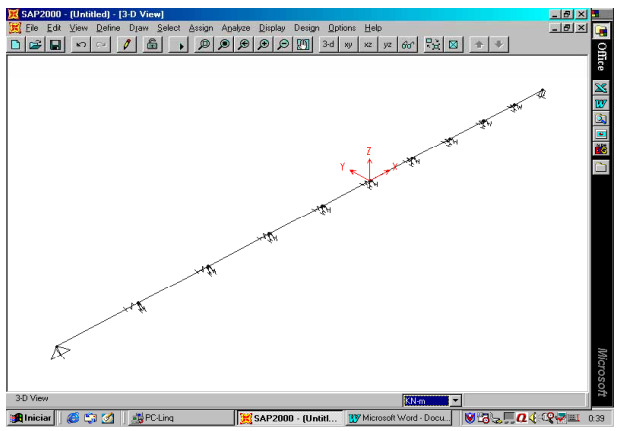

Figure 4. Standard pipeline for the base model [4]

The geometry characteristics adopted in the model are the same for both analyses, A_1 and A_2. Besides, in both situations it has been necessary to consider the damping of the ensemble ground/pipe. Although the pipelines are in steel, it is considered a damping coefficient value of 2% as well as 0% and 20%. The different values considered for the damping are justified as a way to try to evaluate their influence in each analysis. For analysis A_2 it is still necessary to determine the mass of ground which is assumed to vibrate together with the pipeline, so that the displacement value of the buried structure is the same as the displacement value of the surrounding soil. In determining the value of the vibrating masses for each element it is important to consider a given volume of involving soil – necessary to determinate the effective mass. For the surrounding soil it is considered a length of 20ϕ (approximately 15m for the gas network and 8m for the oil network) and a height of 3ϕ (respectively 2m and 1.2m for the two analyzed networks).

Table 1 presents the masses defined for the different elements, belonging to each different zone and placed at the extremity of each of the mentioned elements. In this table *ZWL* stands for *Zone Without Liquefaction* and *LZ* for *Liquefied Zone*.

Table 1. Effective mass for all pipelines zones [4]

Network	ϕ mm	Mass ton				
		ZWL(5m)	ZWL(5/0,5)	ZWL(0,5m)	ZWL/LZ	LZ(0,5m)
Transgás	711.2	266.4	146.5	26.6	28.1	29.6
CLC	406.4	87.8	48.3	8.8	9.3	9.8

In the extremity nodes of the model it is assumed a mass of about half of the obtained value for the elements of 5m width, corresponding to the zone without liquefaction because only half of the element contributes for the mass of the referred nodes.

Seismic action modeling

In this work time analyses are carried out through a step-by-step integration, being the seismic action defined by histories of ground displacements and accelerations along time. As a way of simulating the seismic action through time it is possible to use deterministic and stochastic methods. The referred accelerograms are generated [6] on the basis of spectral density functions of potency presented in RSA [7] for type 1 and type 2 actions which represent respectively earthquakes of moderate magnitude to a small focal distance where high frequencies are predominant as well as earthquakes of a higher magnitude to a higher focal distance where smaller frequencies are observed. Also considered are the types of the soil: type I and III, including rocks and stiff coherent soils (type I) and soft, very soft cohesive soils and also loose cohesion less soils (type III) [7]. The reason for this choice is that is intended to determine the maximum and minimum displacements observed at the ground level when a certain seismic action occurs.

For both seismic actions and both considered soils, 10 accelerograms are generated corresponding to the horizontal component of acceleration. When generating the accelerograms, potency spectra referring to the horizontal component of ground acceleration indicated in RSA are considered. Selection of artificially generated accelerograms is made in order that response spectra corresponding to those accelerograms would present pacing similar to those observed for code response spectra purpose by RSA. After having defined the artificially accelerograms, the displacement histories were evaluated by means of corrected integration. In Figure 5a it is presented the code response spectrum considered the mean response spectrum obtained from the artificially accelerograms generated and, as example, three response spectra defined from three artificially accelerograms. Figure 5b shows three displacement histories used in the parametric studies presented in this work.

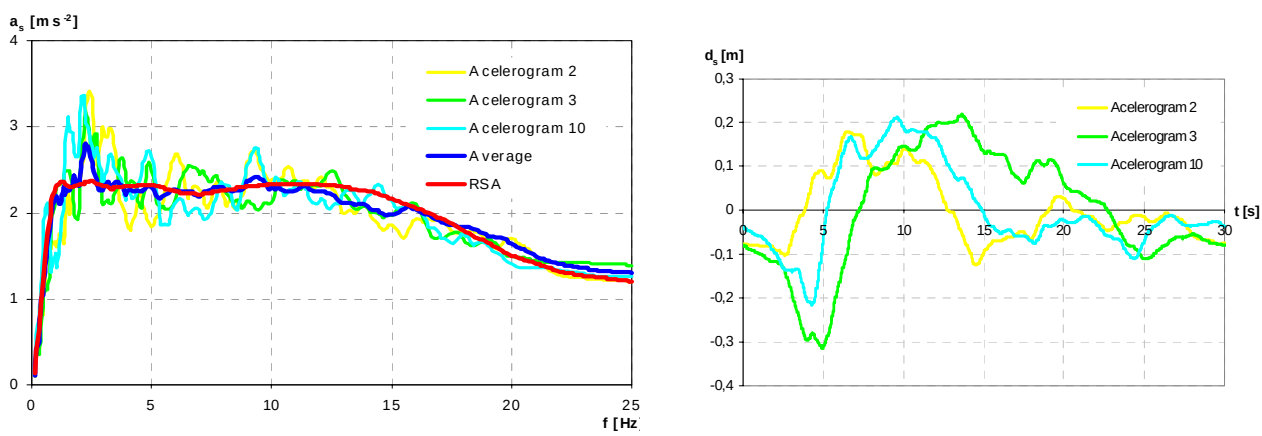

Figure 5. Earthquake type 2 and soil type III: (a) Response spectra and (b) displacement histories

PARAMETRIC STUDY

It was chosen as reference the gas pipeline model with a diameter of 711.2mm, 500m length and 12.7mm wall thickness (TRANSGÁS network), in which the ground reaction coefficient is 11000kN/m^3 and the liquefaction zone spread along a 100m width. The CLC network is also modeled because it is the only oil network existing in the zone under study, besides the fact that it presents a diameter of about half (406.4mm) of the gas network modeled (711.2mm). Also implemented is the model for pipelines having different wall thickness (8.7mm, 11.1mm and 15mm) from the considered one (12.7mm) either for the base model of the mentioned network and for the oil pipelines. It is also carried out a study intending to analyze the influence of soil's rigidity variation on results. For that it is used the reference model considering k_2 values of about 5000kN/m^3 and 50000kN/m^3. Table 2 shows the values obtained for the soil's rigidity K for all pipeline's zones.

Table 2. Soil's rigidity constants for all pipeline's zones [4]

Network	ϕ mm	k_2 kN/m^3	K KN/m				
			ZWL(5m)	ZWL(5/0.5)	ZWL(0.5m)	ZWL/LZ	LZ(0.5m)
Transgás	711.2	5000	17780	9779.0	1778.0	897.9	17.8
		11000	39116	21513.8	3911.6	1975.3	39.1
		50000	177800	97790.0	17780.0	8978.9	177.8
CLC	406.4	11000	22352	12293.6	2235.2	1128.8	22.4

In addition, it is also studied the influence of liquefaction width. As the width considered in the base model is 100m, it is adopted two different situations corresponding to 10m and 50m liquefaction width. In the following it is presented graphically the results obtained with the analyses A_1 and A_2, performed for the various parametric studies adopted. Firstly, the variation of the external diameter of the pipeline is studied, assuming as constants the wall thickness (12.7mm), the soil reaction coefficient of 11000kN/m^3 and the 100m length of the liquefied zone – Figure 6.

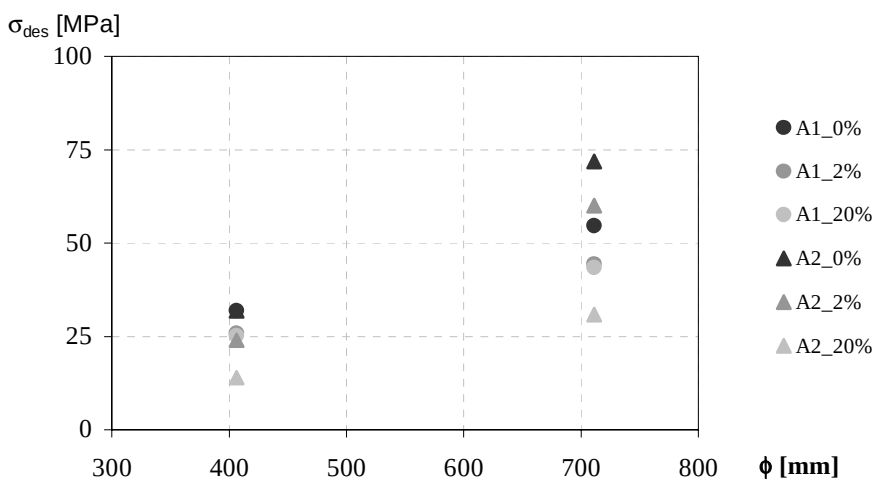

Figure 6. Design stresses as function of the external diameter of the pipeline [4]

Regarding the values obtained in the two analyses it is possible to conclude that, both for smaller external diameter values (406.4mm) and higher values (711.2mm), the analysis that leads to higher design stresses is analysis A_2 for a damping coefficient value of 0%. On the analyses under study, the one that always leads to smaller strength values, and therefore less probable to induce breaks or leaks in pipelines, is analysis A_2 in which a damping of 20% is assumed. Also to be mentioned is an almost coincidence

between design strength obtained for a diameter of 406.4mm in analysis A_1_2%, A_1_20% and A_2_2%. In what concerns the largest studied diameter (711.2mm) it is observed similar results in analysis A_1_2% and A_1_20%. Although the studied diameters of pipelines are the most significant in the area defined for the study of the gas and oil networks, more reliable results could be obtained if it has been studied intermediate diameters.

Similar to what is made for the external diameter variation, several analyses are compared when varying the pipeline wall thickness (Figure 7). The external diameter corresponding to this parametric study is 711.2mm (gas network), the soil's reaction coefficient is 11000kN/m^3 and the width of the liquefaction zone is 100m.

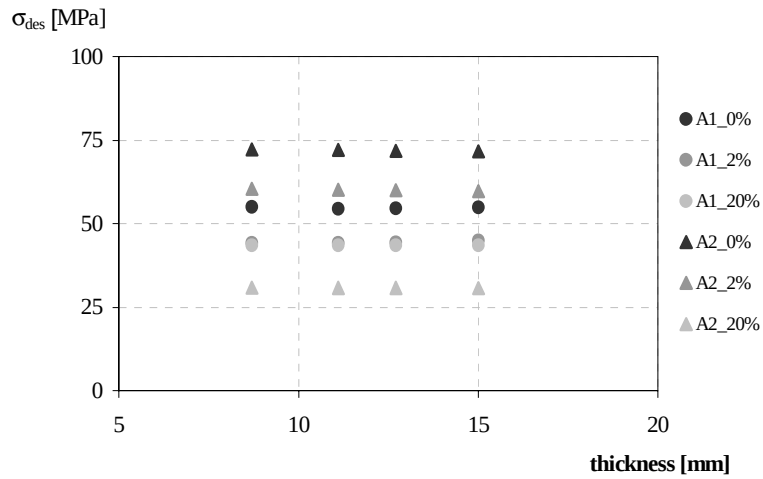

Figure 7. Design stresses as function of pipeline wall thickness [4]

From Figure 7 it is possible to verified that analysis A_2_0% is the on, which reaches higher, and therefore more serious, values of design stress, what ever is the thickness of the pipeline the analyzed value. On the other hand, it is verified that the analysis that gives more reduced stress values is A_2_20%. This difference within the same analysis may be justified by the fact the mass considered for the ensemble soil/pipeline is responsible for the increase of inertia and consequently for the increase of stress in the pipeline in a non damped situation, whereas considering a higher value for the damping coefficient of the ensemble. An almost perfect coincidence between A_1_2% and A_1_20% is also noticed. Therefore it is possible to conclude that the damping values doesn't influence in a significant way the analysis results when the action is considered as a displacement history. In general, design stress values obtained in analysis A_1 and A_2 are constant for any variation of the pipeline's wall thickness.

For the situation in which the soil's reaction coefficient and consequently the soil's rigidity varied, being constant the external diameter of the pipeline (711.2mm), the thickness of the pipeline wall (12.7mm) and the length of the liquefaction zone (100m), the results obtained are presented in Figure 8.

Generally it is noticed that the design stress values decrease when the soil's reaction coefficient, k_2, increases. Thus, for increasing k_2 values, the soil presents a higher rigidity, and so the correspondent internal forces tend to be to be smaller as the k_2 coefficient increases. A possible explanation for this tendency is that relative deformation between the liquefaction zone and the remaining zones reduces, therefore leading to stress values smaller. The analysis performed lead to higher design stress values, both for the more reduced reaction coefficient values (5000kN/m^3) as well as for the highest one is A_1_0%, while the dominant analysis for intermediate values is A_2_0%. For low damping the values of the design

strength obtained in the analysis A_2 can result from an excessive value of dynamic amplification resulting from resonance phenomena that might have appeared as a result of the alteration of the vibration frequencies induced by the variation of soil's reaction coefficient. For more reliable conclusions it would be necessary to make a more exhausting parametric study for intermediate values of damping and k_2. In what concerns results obtained in A_2_20%, it can be stated that, in any circumstance, they are the ones that raise more reduced design stress values and thus generate lower probabilities for leaks and breaks to occur in buried pipelines.

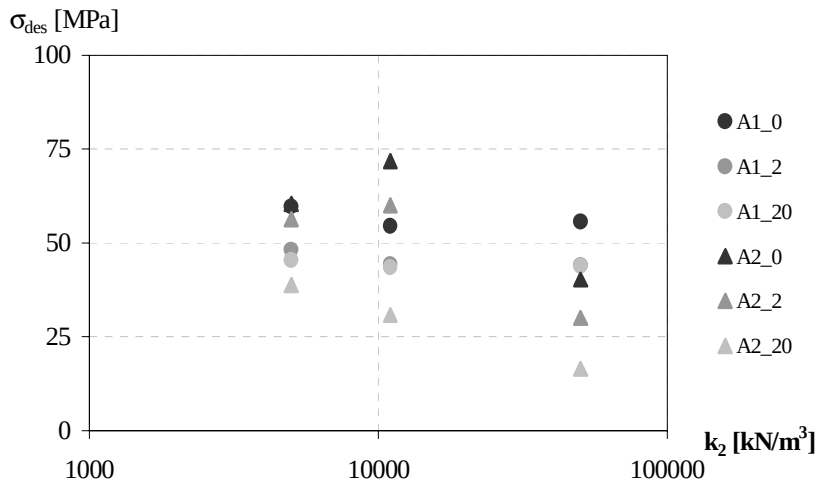

Figure 8. Design strength as function of the soil's reaction coefficient [4]

Different analyses were also performed making vary the width of the zone where liquefaction occurs (L_{liq}) and considering the external diameter (711.2mm), the thickness of the pipe wall (12.7mm) and the reaction coefficient constant (11000kN/mm^3). The results obtained are presented in Figure 9.

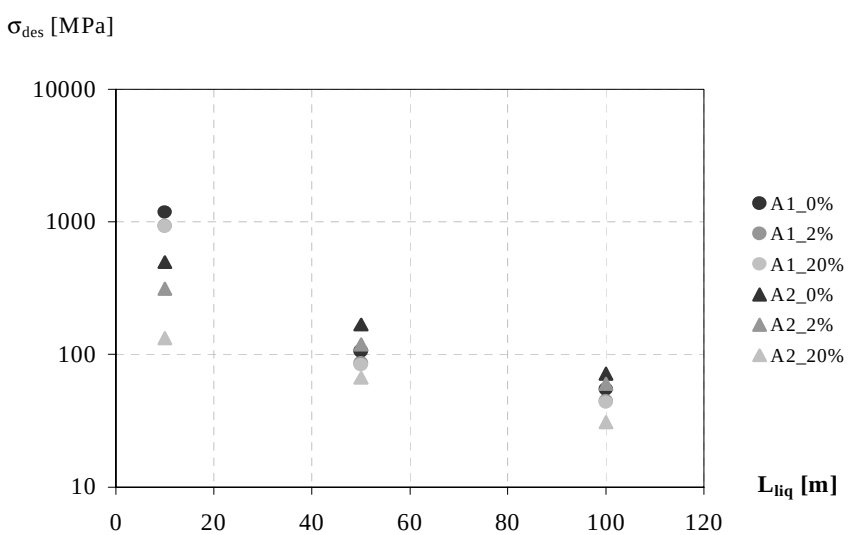

Figure 9. Design stresses as function of ground liquefaction length [4]

In what concerns the analyses made, stress values decrease as the width of the liquefaction zone increases, whatever the damping coefficient considered. A close proximity is noticed in what concerns the design stress obtained in all analyses where a 100 m width of liquefied zone is considered, although in analysis A_2_20% a considerable deviation is observed. As it can be verified, the values obtained for design stress

only reach the values of the yielding stress of the material in analysis A_1 (for all the damping values) and A_2_0% with a 10m width of liquefaction's zone.

Yielding stress of materials used in TRANSGÁS and CLC pipelines corresponds to 482MPa and 448MPa respectively. In this study a very simplified hypothesis is adopted regarding the definition of pipelines ruptures. It is considered that pipeline' breaks only occur when the maximum design stress reaches the yielding stress of the pipeline material. In this way, when the yielding stress of the material is not reached it is admitted that ruptures in pipeline's sections do not occur.

Intending to know for which values of soil's displacement the buried pipelines break it was made an additional parametric study by means of analysis A_1. As the displacement's history obtained initially, from the integration of the acceleration's history, leads to a maximum value of displacement of 0.32m (RSA 2, Soil type III), it was decided to adopt multiplication factors of 3, 5 and 7 leading to maximum soil's displacement of 0.96m, 1.5m and 2.24m. In this last parametric study it was considered a fragment of the principal line of natural gas network (TRANSGÁS) of about 500m with a nominal diameter of 711.2 mm, thickness of 12.7 mm, soil's reaction coefficient of 11000 kN/m^3 and the three previous hypotheses for the ground liquefaction length: 10 m, 50 m and 100 m.

The following figures (Figure 10 to 12) schematize the graphic presentations obtained from the additional parametric study performed for the TRANSGÁS buried pipelines. The results obtained are defined in terms of design strength, as allows a comparison with the yielding stress of the material used in the TRANSGAS buried pipelines (482MPa) represented as a dashed line in Figures 10, 11 and 12.

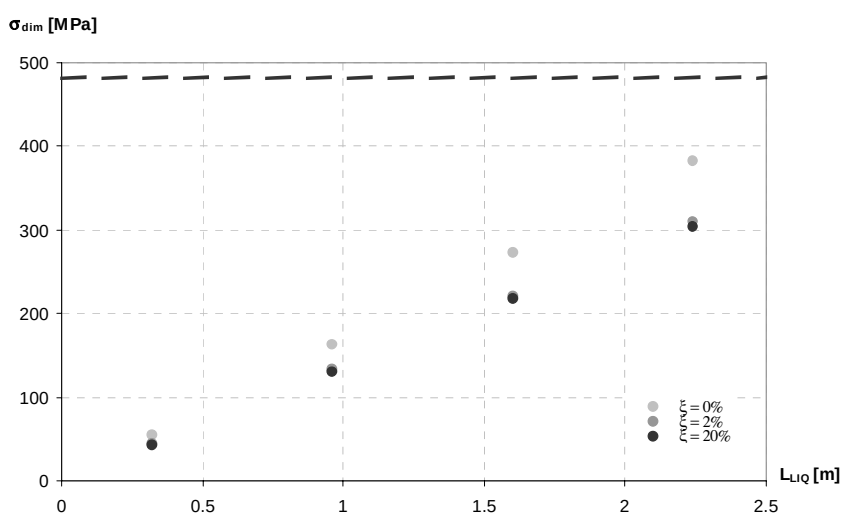

Figure 10 - Design strength *vs* displacement's history, for 100m ground liquefaction length of [4]

For the chosen values of the maximum soil's displacement and for the ground liquefaction length of 100m (corresponding to 20% of the analyzed buried pipeline), it is noticed that yielding strength of the material, that constitutes the TRANSGAS pipeline, it is not reached in any circumstance. In fact, the worst value obtained for the design strength corresponds to about 60% of the referred yielding strength of the material. So, in this situation, the pipelines are very aware from the situation of local rupture and subsequent collapse of the network's operations.

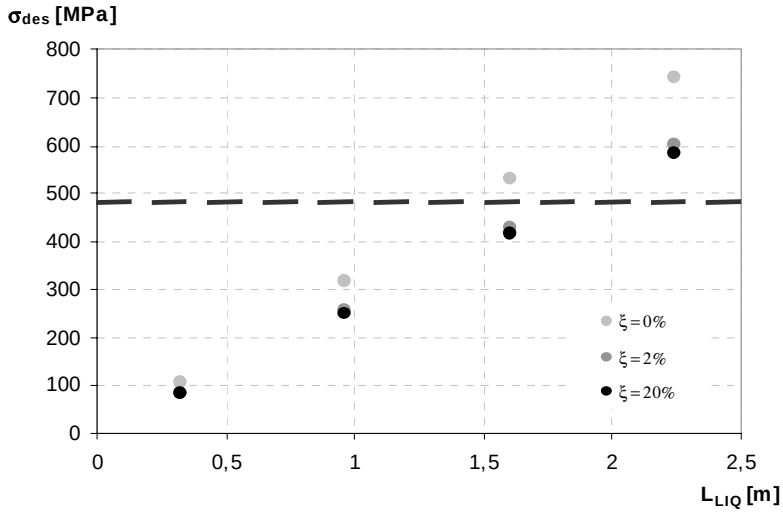

Figure 11 - Design strength *vs* displacement's history, for a ground liquefaction length of 50m [4]

If the liquefaction phenomena occurs in a length of 50m (corresponding to 10% of the analyzed buried pipeline), the yield strength of the material's pipeline is not reached for any damping value if the maximum soil's displacement is less than about 1.8m. Considering an analysis without damping that value decreases to approximately 1.5m. For higher values of maximum soil's displacement the yielding of the buried pipeline is reached, with consequent rupture and subsequent collapse of the network's operations. For the most unfavorable analyzed situation the design strength exceeded in about 50% the yield strength.

If it considered that the ground liquefaction occurs in a small ground liquefaction length, for instance 2% (10m) of the analyzed pipeline – Figure 12, the rupture is reached for small values of maximum soil's displacement. As it is represented in Figure 12, for a ground liquefaction length of 10 m the design stress is always higher than the yield stress for all the damping coefficient values considered and for any imposed displacement's history. In this situation, the rupture of the section occurs leading to, at least, local consequences in the performance of the network.

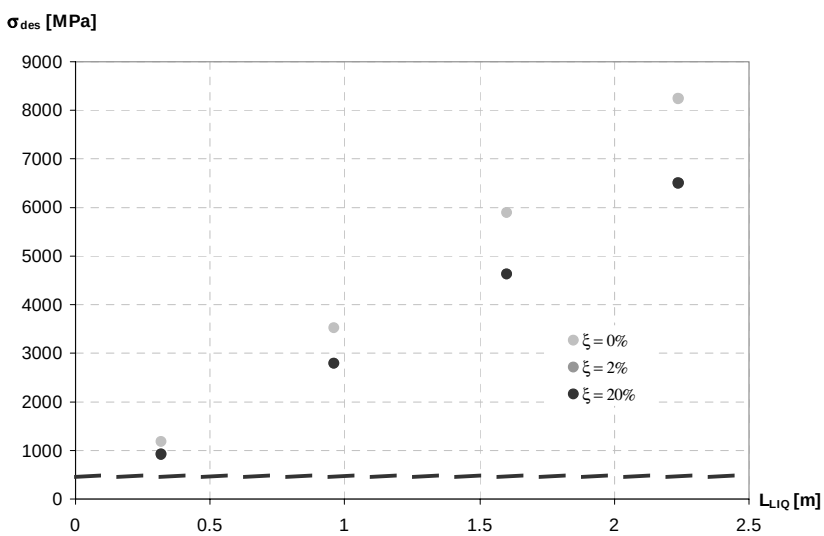

Figure 12 - Design strength *vs* displacement's history, for a ground liquefaction length of 10m [4]

From the results presented and shown in previous Figures, it is possible to conclude that there is a strong tendency to reach and even overtake the yield strength of the material of the TRANSGAS buried pipelines, as the ground liquefaction length decreases. This tendency has been shown to be completely independent of the value of damping - Figures 10 to 12. For more general conclusions it should have been made exhausting analyses, considering intermediate values of maximum soil's displacement, and so it would be possible to obtain more precisely the real evolution of the results.

CONCLUSIONS

In this work the seismic vulnerability of buried pipelines is evaluated, based on analytical simulations. The vulnerability definition can be due to the transient seismic wave propagation or permanent ground deformations (PGD). These effects must be considered separately.

As it has been verified that PGD are systematically the cause of the most severe observed damages, it is adopted in this study only the effect of permanent ground deformation due to liquefaction phenomena. The results obtained with the different parametric studies performed allow reaching the following conclusions:

- The analyses A_1, corresponding to impose displacement histories, leads to more coherent outcomes;
- The differences observed between A_1 and A_2 are mainly because of the input is displacement and acceleration, respectively. Besides that is important not to forget the value of damping considered in each analysis;
- The influence of variation of the liquefaction zone width has a great importance in the final results. The change from a 10 m width of liquefaction zone to an 100 m width leads to a considerable decrease of the design stress in the pipelines, belonging to both analyzed networks in the present study;
- Not identical to the methodology proposed by HAZUS99 [8], analytic modeling of buried pipelines could account for (as is the case in this study) factors such as the variation of the external diameter and wall thickness of pipeline, the soil's rigidity and the width of the area where liquefaction occurs. All these must not, in any case, be neglected in a model of a given buried pipeline in a given continuous environment, like the soil. The pipeline vulnerability is necessarily influenced by more factors than those indicated in HAZUS99;
- With the analytical model proposed in this work is very easy to include the non-linear behavior of the soil. This phenomenon can be modeled by means of imposing a non-linear constitutive law for the spring elements.

REFERENCES

1. Lim, Y.M., Kim, M.K., Kim, T.W., Park, A.S.W. "Seismic behavior of pipelines during transverse liquefaction considering effects of ground deformation". 12WCEE Proceedings of the 12th World Conference of Earthquake Engineering, 2000. Auckland, New Zealand.
2. SAP2000. "Integrate finite element analysis and design of structures". Computers and Structures Inc., 2000. Berkeley. California, USA.
3. Santos, J. 1999. "Caracterização de solos através de ensaios dinâmicos e cíclicos de torção – Aplicação ao estudo de estacas sob acções horizontais estáticas e dinâmicas". Tese de Doutoramento. Instituto Superior Técnico, Universidade Técnica de Lisboa (in portuguese).
4. Falcão Silva, M.J. 2002. "Vulnerabilidade sísmica de redes de gás e combustível", Tese de Mestrado, Instituto Superior Técnico, Universidade Técnica de Lisboa (in portuguese).

5. ERDCJ – Earthquake Resistant Design Codes in Japan. JSCE, 2000.
6. Guerreiro, L. “Geração de séries de acelerações”. DTC Report nº ½. ICIST Instituto de Engenharia de Estruturas, Território e Construção, 2002, Lisbon (in portuguese).
7. RSA – Regulamento de Segurança e Acções, 1983, Decreto Lei Nº 235/83, Lisbon.
8. HAZUS99. “Earthquake loss estimation methodology”. Federal Emergency Management Agency, 1999, Washington, D.C.