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A PARAMETRIC STUDY ON THE COMPREHENSIVE ANALYSIS OF PIPELINES UNDER GENERALIZED ACTIONS

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SUMMARY

The present work on the seismic analysis of pipelines outlines major aspects of a parametric study of steel pipelines under generalized actions. Design recommendations are synthesized for the seismic design of aboveground and buried pipeline systems. Some relevant parametric results are obtained characterizing the behavior and performance of aboveground and buried pipeline systems under potential seismic actions.

For an aboveground pipeline supported by concrete saddles and subjected to seismic actions of two types, a comparative study was performed for a series of critical values of pipeline system responses. The numerical results presented herein were obtained by a spectral analysis with the design spectrum, the time history analyses with eight accelerograms based on the design power spectrum, and by another spectral analysis with the mean response spectrum obtained for the eight accelerograms. Mean spectra are compared with the design spectra, from which it is concluded that the generalized forces obtained with the spectral analysis are similar to the generalized mean forces obtained by time history analysis. Some discrepancies obtained for seismic action of second type were thoroughly justified.

For the buried pipeline, the inertia forces resulting from pipeline-soil interaction are much smaller than the forces associated with soil deformation, which leads to a static model for seismic design. A parametric study of the soil-pipeline interaction, for a guillotine-type imposed settlement, considers variability of several pipeline design parameters. It was concluded that the pipeline bending moment is higher for pipeline of larger diameter, due to its higher stiffness. The soil friction angle influence is much smaller than the influence of pipeline diameter, even though relevant. The vertical uplift soil forces are greater for larger diameter pipelines, since they have a better capability to adapt and recover from imposed deformations, resulting in larger lengths of yielded soil around the fault. An increase in the imposed guillotine settlement also increases the pipeline internal generalized forces. For bigger settlements, displacements and forces along the pipeline and soil forces will be also significant in greater extensions.

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INTRODUCTION

Lifelines are all public, semi-public and private engineering infrastructures that support given standards of quality of life of society and civilization. Their good performance is of paramount importance, because of social economic and political implications. Among lifeline structures, tanks and pipelines must be carefully designed, since their damage or destruction may have serious human, social, economic and environmental consequences.

The seismic analyses of aboveground steel storage tanks by two distinct finite element method approaches have been developed at FEUP (Faculdade de Engenharia da Universidade do Porto) by Barros [1, 2], Barros et al. [3], permitting to draw some conclusions and design considerations with respect to avoiding the elephant foot bulge deformation. Barros [4] also addressed some analytical aspects on the vibration control in pipelines. The present work on the seismic analysis of pipelines is based upon a parametric study of steel pipelines under generalized actions, partially published recently by Pereira et al [5, 6], under which considerations for the seismic design of pipelines were also established. Herein design examples are considered, in order to illustrate the seismic considerations used in the present analyses.

ABOVEGROUND PIPELINES

In this section it will be addressed the dynamic analysis required in earthquake resistant design of piping systems laid above the ground surface. Extensive dynamic studies have been done and reported internationally with respect to the piping system for nuclear power stations; however the present work details an analysis of a piping system within a refinery complex. In general, the piping system for nuclear power stations is designed in such way that the natural frequency of the low damping piping system is higher than that of the support structures, avoiding resonance of piping vibrations with building vibrations. The vibration displacements, the loads on pipe nozzles and the damage to equipment and pipes are reduced.

The piping systems are classified according to their importance and are designed for the most severe seismic load combinations using appropriate methods, in accordance with specified regulations and norms. In order to adopt common legislation for all European countries, the European Committee on Standardization developed Eurocode 8 (EC8) for the design of structures under seismic actions [7]. Part 4 of this document specifies design rules for tanks, silos and pipelines.

Seismic design based on Eurocode-8

Referring to pipelines EC8 indicates values for the importance coefficients, which depend mainly on the kind of use for the system and the class of risk in which it is included. These values should multiply the peak ground acceleration. In terms of requirements it is referred that a pipeline system should be designed in order to assure its supply capabilities, even after major earthquakes, and that the level of protection against explosions or fire should be equated based on the exposed population, on the site characteristics and on the risk of environment impacts.

Serviceability limit state should be verified limiting the deflections (as specified in EC3) and ultimate limit state should be verified by limiting compressive and tensile strains to (0,4 t/R = 0,05). In addition the forces at joints, the global equilibrium and stability against overturning as well as soil stresses should also be checked. In order to quantify the seismic action, the National Application Document of EC8 [8] specifies the peak ground acceleration on rock a_{gr} for each seismic zone and type of action. This document presents the expressions or graphs for the elastic response spectra, and the corrective factors for the type of soil and for the damping coefficient.

The energy dissipation through non-linear structural deformations is also taken into account by the consideration of a behaviour factor, which should be taken equal to 1 in pipeline systems (EC8 – Part 4). Besides the variability of ground motion associated with each of two possible spectral analyses, land sliding, liquefaction and permanent ground deformations should also be taken into account.

Seismic design example of an aboveground pipeline system, by the finite element method

In this example it is pretended to understand the behaviour and performance of an aboveground steel pipeline located in an important ecological area, evaluating the generalized forces and displacements inflicted to the pipeline system by the design earthquake.

The pipeline is constituted by 80 cm diameter tube with 6 mm thickness, which assures the water transportation between two tanks (Figure 1) located in seismic zone A. The aboveground pipeline is supported, on soil type I, by concrete saddles which contain steel rings that prevent pipe overturning. A ‘U-type’ curve is considered, in order to account for longitudinal deformations under thermal action and under strong longitudinal ground motion. The masses involved in the analyses are calculated with the pipeline specific weight (γ=77 kN/m^3) and water specific weight (γ=9,81 kN/m^3) considering the pipeline flow at full section, therefore neglecting eventual hydrodynamic effects.

Portuguese design standards in RSA [9] for seismic actions of types 1 and 2 (AS-1, AS-2) are also used, since the application of EC8 has not been completely enforced. Conceptually, seismic action of type 1 (AS-1) corresponds to a moderate earthquake at short focal distance with 10 seconds duration, while seismic action of type 2 (AS-2) corresponds to a strong earthquake at longer focal distance with 30 seconds duration.

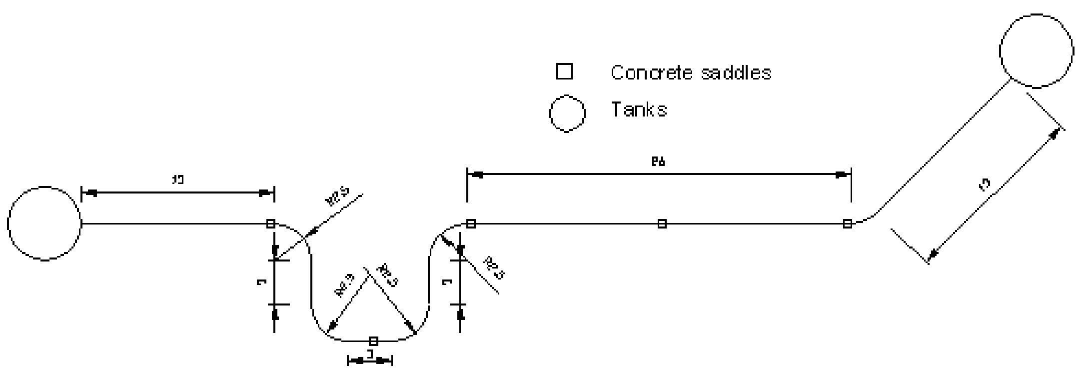

Figure 1: Aboveground pipeline

A comparative study is presented herein between the results obtained by a spectral analysis with the design spectrum, the time history analyses with eight accelerograms based on the design power spectrum, and by another spectral analysis with the mean response spectrum obtained for the eight accelerograms. The vertical seismic action is considered to be 2/3 of the horizontal action.

All kinds of analyses are based on the multimode decomposition method (modal superposition technique) with 25 vibration modes. It is observed that the sum of participation masses of all modes, in the three directions, is accurate enough for good precision in calculating forces and displacements. The frequencies vary between 5,15 Hz and 34,77 Hz. The modes with greater participation masses in each direction are mode 1 with 42,6% (longitudinal direction; 5,15 Hz), mode 12 with 48,5% (vertical direction; 12,16 Hz) and mode 13 with 32,67% (transversal direction; 15,21 Hz). Figures 2 and 3 present the configurations of some of these modes.

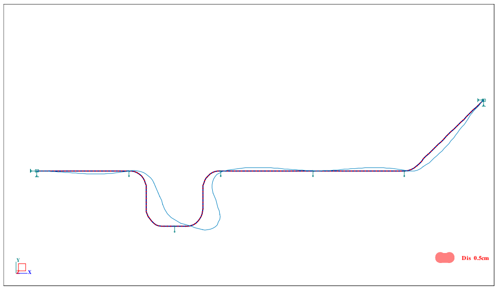

Figure 2: Vibration mode 1

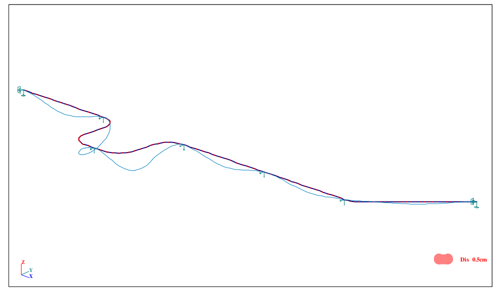

Figura 3: Vibration mode 12

The sophisticated space-time random field theory associated with the strong motion software SIMQKE [10] is used herein to produce conditioned or unconditioned earthquake ground motions, matching the design power spectrum on average. This improved methodology is applied to generate artificial synthetic earthquakes that induce specific instantaneous pipeline responses in distinct generalized variables; these earthquakes are constituted either by 1000 points with total duration of 10 seconds (for AS-1) or by 3000 points with total duration of 30 seconds (for AS-2). One such accelerogram is presented in Figure 4.

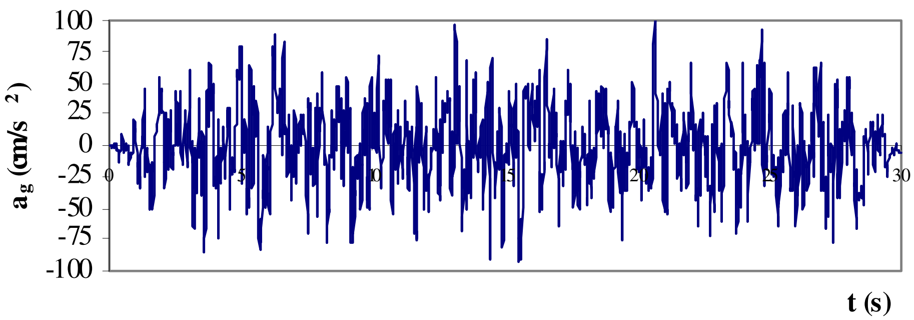

Figure 4: Acceleration signal of an artificial synthetic earthquake (AS-2)

A numeric integration process computes the response of each vibration mode to the imposed excitations. After summing all modal contributions (considered), tables and graphs are created that allow the visualization of force and displacement variations in time. For example, Figure 5 represents the bending moment time history (for a given earthquake) at the connection between the pipeline and the left tank shown in the aboveground pipeline layout.

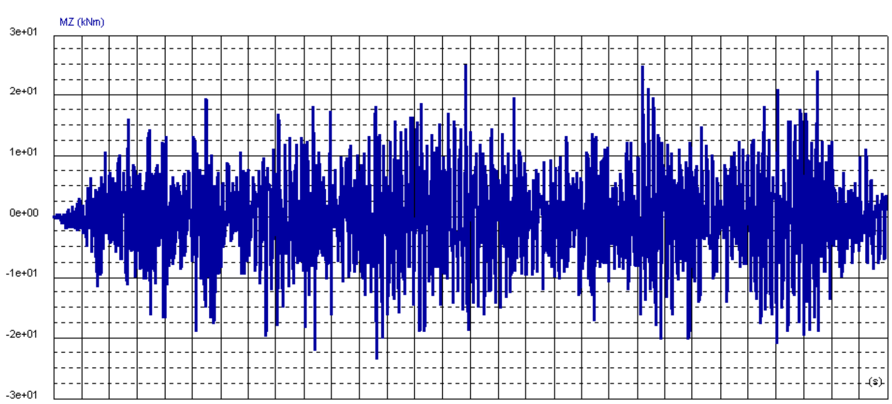

Figure 5: Bending moment at the pipeline connection with the left tank

Limiting the maximum ground accelerations at the pipeline system site on soil type I by the spectral density of accelerations specified in Portuguese standards [9] for zone A (with higher seismic risk) under seismic actions AS-1 and AS-2, a numerical study was performed for a series of critical values of pipeline system responses.

For all eight accelerograms of seismic actions AS-1 and AS-2, the maximum acceleration has been determined for different frequencies of single-degree of freedom oscillators using the program DUHAMEL. This program determines the response of single-degree of freedom oscillators using Duhamel's integral and provides displacements, velocities and accelerations at desired time steps. By registering the maximum acceleration of each oscillator with different frequency, the response spectrum for each accelerogram under seismic actions AS-1 and AS-2 is obtained (Figures 6-7).

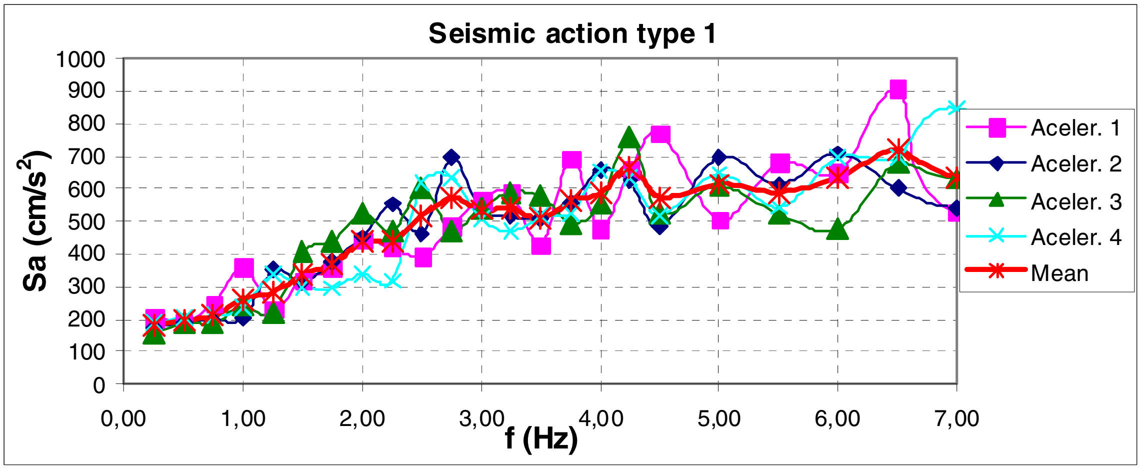

Figure 6: Response spectra for seismic action AS-1

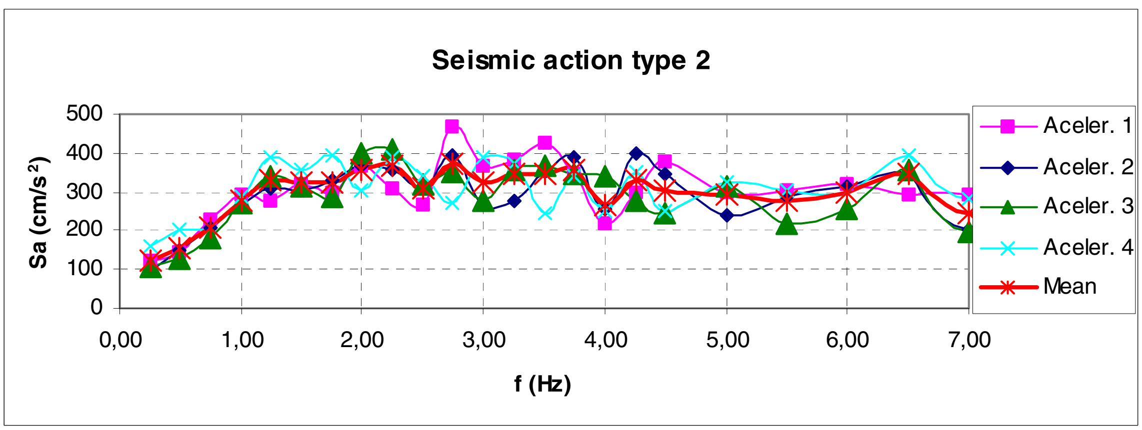

Figure 7: Response spectra for seismic action AS-2

In these figures are represented the mean spectra obtained from the eight accelerograms of each type of seismic actions and (for comprehension) only four response spectra. In Figures 8-9 the mean spectra are compared with the design spectra. It should be noted that for each seismic action type the eight response spectra are very similar to the design spectrum, which reflects the good quality of the accelerograms.

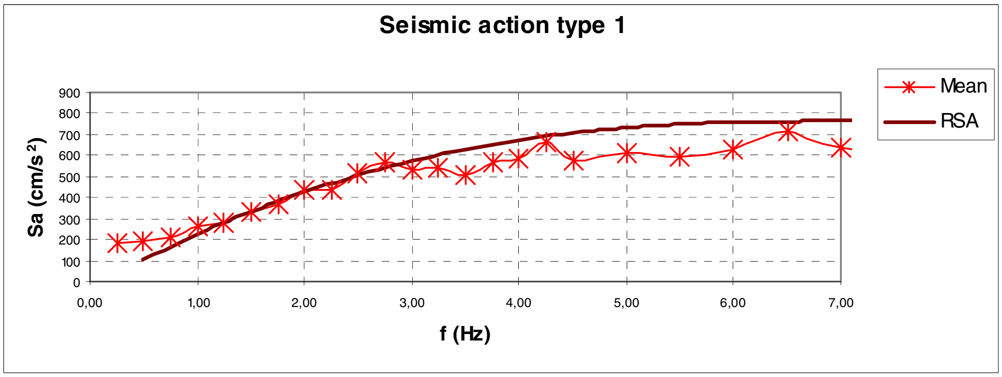

Figure 8: Comparison between Mean and Design response spectra, for seismic action AS-1

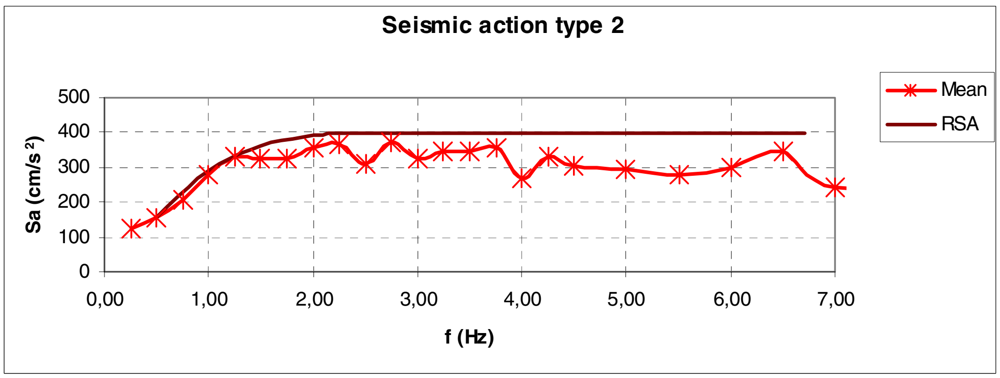

Figure 9: Comparison between Mean and Design response spectra, for seismic action AS-2

The maximum forces and displacements at each point are computed with the complete quadratic combination (CQC). In Tables 1-3 and 4-6, the maximum axial force and maximum bending moment at the connection with the left tank, and the maximum shear force at the middle section of the U-curve are presented, for all eight accelerograms representative of seismic actions of types 1 and 2 (AS-1, AS-2).

Table 1: Maximum axial force at connection with left tank, for seismic action type 1 (AS-1)

		F_x (kN)	Mean value (kN)	Relative error (%)	Standard deviation (kN)	Relative error (%)
Accelerogram	1	45.52	45.52	0.00	0.000	0.00
	2	51.56	48.54	6.22	4.271	100.00
	3	40.20	45.76	6.08	5.684	24.86
	4	55.77	48.26	5.19	6.825	16.73
	5	45.90	47.79	0.99	6.005	13.67
	6	52.01	48.49	1.45	5.640	6.46
	7	44.87	47.98	1.08	5.328	5.86
	8	37.80	46.70	2.72	6.105	12.73
Spectrum	Mean	48.30		3.42		
	Design	75.86		62.43		

Table 2: Maximum bending moment at connection with left tank, for seismic action type 1 (AS-1)

		M_z (kN m)	Mean value (kN m)	Relative error (%)	Standard deviation (kN m)	Relative error (%)
Accelerogram	1	51.28	51.28	0.00	0.000	0.00
	2	59.39	55.34	7.33	5.735	100.00
	3	40.18	50.28	10.05	9.644	40.53
	4	51.37	50.56	0.54	7.893	22.18
	5	69.62	54.37	7.01	10.928	27.77
	6	65.62	56.24	3.33	10.800	1.19
	7	44.89	54.62	2.97	10.752	0.44
	8	53.87	54.53	0.17	9.958	7.97
Spectrum	Mean	45.57		16.43		
	Design	65.55		20.21		

Table 3: Maximum shear force at middle section of U-curve, for seismic action type 1 (AS-1)

		F_y (kN)	Mean value (kN)	Relative error (%)	Standard deviation (kN)	Relative error (%)
Accelerogram	1	27.88	27.88	0.00	0.000	0.00
	2	34.62	31.25	10.78	4.766	100.00
	3	23.06	28.52	9.57	5.807	17.92
	4	28.84	28.60	0.28	4.744	22.40
	5	33.14	29.51	3.08	4.583	3.52
	6	33.23	30.13	2.06	4.371	4.83
	7	30.19	30.14	0.03	3.991	9.54
	8	26.85	29.73	1.38	3.873	3.03
Spectrum	Mean	32.28		8.59		
	Design	40.46		36.11		

The tables compare exhaustively, for successive higher number of generated artificial earthquakes, the average results in each variable and their standard variation; additionally the relative error in determining these quantities is also given. This kind of analysis is recommended by Eurocode 8 and has been used successfully by Barros [1, 2] in the seismic analysis of bottom-supported tanks, with convergence achieved after the consideration of 12-15 artificial synthetic earthquakes satisfying the spectral density of accelerations of AS-1 and AS-2. For comparison, the results for spectral analysis are also presented.

Table 4: Maximum axial force at connection with left tank, for seismic action type 1 (AS-2)

		F_y (kN)	Mean value (kN)	Relative error (%)	Standard deviation (kN)	Relative error (%)
Accelerogram	1	15.17	15.17	0.00	0.000	0.00
	2	13.80	14.49	4.73	0.969	100.00
	3	17.05	15.34	5.57	1.632	40.63
	4	13.71	14.93	2.73	1.562	4.48
	5	14.87	14.92	0.08	1.353	15.45
	6	14.78	14.90	0.16	1.211	11.68
	7	13.69	14.72	1.17	1.196	1.27
	8	15.11	14.77	0.33	1.116	7.20
Spectrum	Mean	15.06		1.95		
	Design	21.54		45.81		

Table 5: Maximum bending moment at connection with left tank, for seismic action type 1 (AS-2)

		M_z (kN m)	Mean value (kN m)	Relative error (%)	Standard deviation (kN m)	Relative error (%)
Accelerogram	1	24.77	24.77	0.00	0.000	0.00
	2	24.50	24.64	0.55	0.191	100.00
	3	22.36	23.88	3.18	1.320	85.54
	4	23.83	23.87	0.05	1.078	22.45
	5	23.16	23.72	0.59	0.986	9.40
	6	20.05	23.11	2.65	1.740	43.35
	7	21.56	22.89	0.97	1.693	2.76
	8	24.19	23.05	0.70	1.633	3.65
Spectrum	Mean	19.73		14.41		
	Design	34.47		49.53		

Table 6: Maximum shear force at middle section of U-curve, for seismic action type 1 (AS-2)

		F_y (kN)	Mean value (kN)	Relative error (%)	Standard deviation (kN)	Relative error (%)
Accelerogram	1	15.17	15.17	0.00	0.000	0.00
	2	13.80	14.49	4.73	0.969	100.00
	3	17.05	15.34	5.57	1.632	40.63
	4	13.71	14.93	2.73	1.562	4.48
	5	14.87	14.92	0.08	1.353	15.45
	6	14.78	14.90	0.16	1.211	11.68
	7	13.69	14.72	1.17	1.196	1.27
	8	15.11	14.77	0.33	1.116	7.20
Spectrum	Mean	15.06		1.95		
	Design	21.54		45.81		

Discussion of results of the aboveground pipeline system, under seismic actions of types 1 and 2

The convergence of the mean values and standard deviations is observed by increasing the number of accelerograms. For the eight accelerograms used, the relative error of the mean value (less than 3% for AS-1; less than 1% for AS-2) is considerably smaller than the relative error of the standard deviation. The convergence of the standard deviation is only achieved by using a greater number of accelerograms, as in the before mentioned case of the seismic analysis of tanks. It was also observed that the relative error in force related values is smaller than the relative error in displacement related values.

The generalized forces obtained with the spectral analysis (mean spectrum) are very similar to the generalized mean forces obtained by time history analysis. In terms of generalized displacements a greater discrepancy between results was observed. The values obtained by using the design spectrum are always greater than the other values, which is probably due to the constant spectral acceleration (Figure 10) for higher frequencies (without decay) inherent to the design standard [9]. Similar conclusions have been observed for moments, shear forces, axial forces and displacements at several points along the pipeline system layout.

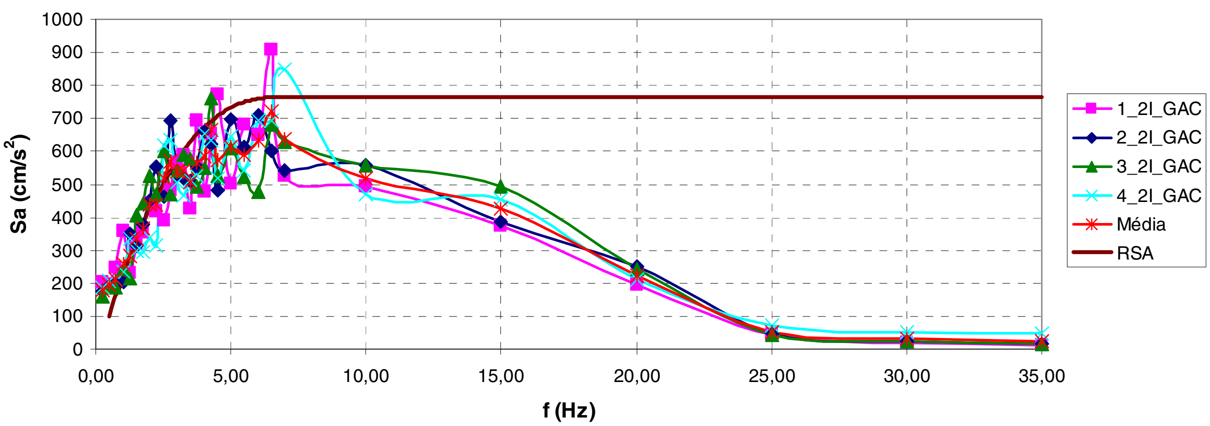

Figure 10: More complete comparison between Mean and Design response spectra (AS-2)

It has been concluded that the spectral analyses with the design spectrum always provide safer values, with a small computation effort. The time history analysis provides smaller values whose precision increases with the number of accelerograms. This kind of analysis demands a high computational effort and is usually difficult to apply at the design stage. Nevertheless it may be very useful in long pipeline layouts (longer than 100 m), since it allows the consideration of the seismic action variability in space.

BURIED PIPELINES

Seismic design based on Eurocode 8

With respect to fundamental provisions, the importance factors and limit states to consider for buried pipelines are the same as for aboveground pipelines. Nevertheless, besides earthquake ground movement, Eurocode EC8 refers two additional situations to be considered in the design of buried below ground pipelines: fault crossing and liquefaction. In the case of liquefaction risk special measures could be taken to prevent damage, like increasing the pipe's depth that could implicate adopting stiffer pipelines (increasing wall thickness), or like using aboveground pipeline supported on piles.

If fault crossing is inevitable, the pipeline should be carefully designed around that area, taking into account the costs of the design solution, the fault activity, and the human social economic and environmental consequences of a potential pipeline rupture.

In the case of fault crossing: every adopted measure in the event of fault movement should guarantee that only tensile axial forces act on the pipe (or that the pipeline is the least compressed); the adopted pipeline depth should also be small enough next to fault, in order to assure sufficient pipe flexibility; the pipe wall thickness should be increased in a 300 m distance of the fault; and the soil friction angle should be sufficient small to assure better force distribution around the pipeline.

The inertia forces resulting from pipeline-soil interaction are much smaller than the forces associated with soil deformation, which leads to a static model for seismic design: the pipeline deforms and distorts due to the travelling seismic waves, without considering any dynamic effects.

The seismic waves that propagate deep in the soil during an earthquake (volumetric waves) can be compressive waves and shear waves; for shallower depths also occur the surface waves: Rayleigh waves and Love waves. Since every kind of wave has different wave velocities and different types of movement, the type of wave considered in every circumstance is the one most unfavorable for the pipeline performance. In such case the forces on the pipeline should be obtained by a time-domain analysis, in which time transports the wave through the pipeline structurally connected to the soil by longitudinal and radial spring supports.

According to Pereira et al. [5, 6] the movement may be represented by the following sinusoidal function:

$$u(x,t) = d \cdot \sin\left[\omega\left(t - \frac{x}{c}\right)\right] \tag{1}$$

where *u(x,t)* represents the soil particle displacements at point *x* and instant *t*, *d* the maximum displacement, ω the seismic wave angular frequency and *c* its apparent velocity. The waves may have the pipe's direction (compressive waves) or may be perpendicular to it (shear waves). The soil and pipe strains due to the longitudinal wave propagation may be calculated by equation (2). Designating V as the soil peak velocity ($=\omega d$), the maximum of equation (2) may be determined by equation (3).

$$\varepsilon = \frac{\partial u}{\partial x} = -\frac{\omega d}{c}\cos\left[\omega\left(t - \frac{x}{c}\right)\right] \tag{2}$$

$$\varepsilon_{\max} = \frac{V}{c} \tag{3}$$

The soil and pipe curvature χ due to transversal motion may be determined by equation (4). If *a* is the soil peak acceleration ($=\omega d^2$), the maximum of (4) is given by (5).

$$\chi = \frac{\partial^2 u}{\partial x^2} = -\frac{\omega^2 d}{c^2}\sin\left[\omega\left(t - \frac{x}{c}\right)\right] \tag{4}$$

$$\chi_{\max} = \frac{a}{c^2} \tag{5}$$

Parametric study of the soil-pipeline interaction, for a guillotine-type imposed settlement

Frequently the design engineers only consider the generalized forces induced in the pipelines due to the travelling seismic waves, neglecting other aspects that can sometimes be more severe to the pipeline performance behaviour and stability.

Examples of these situations are the permanent soil deformations, soil liquefaction and landslides. The case to be analysed in this section is a guillotine-type imposed permanent settlement, resulting from fault activity (Figure 11).

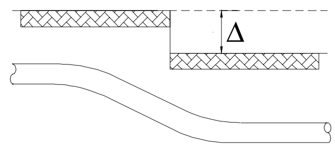

Figure 11: Imposed guillotine settlement

The key point in this analysis is pipe and soil modelling, since its bad quality may produce imprecise results. A finite element program with non-linear analysis capability is required.

The soil is modelled with non-linear elastoplastic supports with 1 meter spacing, whose p-y curves are determined experimentally by geotechnical studies. Each support has different behaviour laws at longitudinal (soil friction) and vertical direction (considering uplift resistance different than bearing resistance). The pipeline is modelled with Timoshenko beam finite elements, considering the deformation due to shear forces, along the total length of 200 m (100 m for each side of the fault, in order to obtain negligible forces and displacements at large distances from the fault). Material and geometrical non-linearity of the steel pipeline is not being considered herein, just because of difficulties to achieve numerical convergence in a personal computer.

The considered variable parameters in this parametric study are the pipe diameter D (24 in , 36 in , 48 in), soil friction angle ϕ (30º , 35º , 40º), pipeline depth H (1 m , 5 m) and settlement magnitude Δ (40 cm , 80 cm) resulting in 36 different parametric cases. For all the cases a 1,17 cm pipe wall thickness was adopted, for a pipeline layout in a soil with a specific weight of 18 kN/m^3. A 50º C uniform thermal expansion is applied in order to also access the pipeline longitudinal behaviour. Notice that there is no axial force due to the imposed settlement, because geometric non-linear behaviour has been neglected by Pereira et al. [5, 6] just for accomplishing these analyses in a personal computer.
Figure 12 presents the results of the parametric study of the guillotine-type settlement, for the diameter and soil friction angle variability, in the case of a 1 meter pipe depth and a 40 cm fault settlement magnitude.

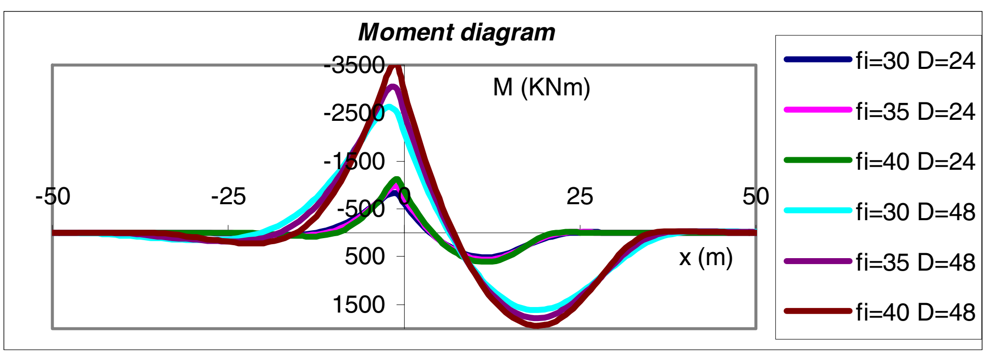

Figure 12: Variation of bending moment with pipeline diameter and soil friction angle

Figure 13 presents the results of the parametric study of the guillotine-type settlement, for the pipeline depth and soil friction angle variability, in the case of a 36 inches pipe diameter and a 40cm fault settlement magnitude.

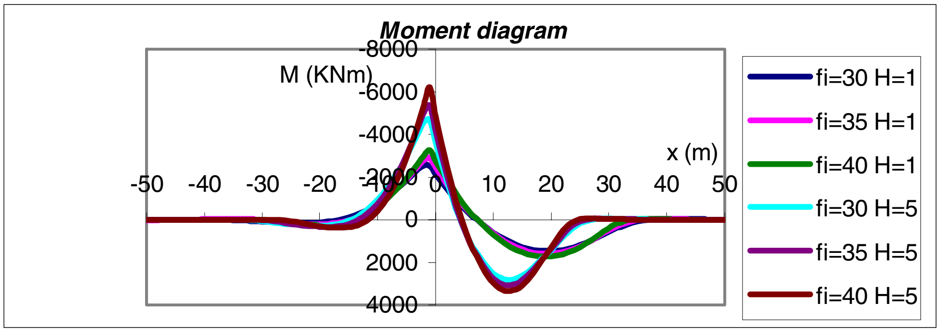

Figure 13: Variation of bending moment with pipeline depth and soil friction angle

Discussion of results of the parametric study for a guillotine settlement in a buried pipeline

The pipeline bending moment is higher for pipeline of larger diameter, due to its higher stiffness. On the other hand the lateral extension of the diagram indicates that larger pipelines deform to larger distances from the fault, which can be explained by their better capability to adapt and recover from imposed deformations. The soil friction angle influence is much smaller than the influence of pipeline diameter, even though relevant. Since soils with bigger soil friction angle are stiffer, it means that the greater this parameter is the lesser the pipeline adapts and recovers from the imposed deformations, resulting in higher internal forces.

The vertical uplift soil forces are greater for larger diameter pipelines, since they have a better capability to adapt and recover from imposed deformations, resulting in larger lengths of yielded soil around the fault. The soil bearing resistance forces are greater for larger diameters and higher soil friction angles since the global stiffness is bigger in this case.

The larger moments for higher depths indicate that the more the soil is stiffer (higher depths) the larger are the internal forces, since the pipeline will have lower capabilities to adapt and recover from the imposed settlement. As for the diameter, the depth influence on the internal forces is much higher than the influence of soil friction angles.

Although it seems that the depth influence on the internal forces is of the same order of importance as the diameter influence, that fact is not entirely correct. For example, unlike what occurs for the diameter, an increase in depth leads to a deformation in closer areas to the fault settlement location. Although for higher depths the moments are bigger, these occur closer to the fault location than for lower depths. The most relevant difference in increasing the pipeline diameter or its depth, is that in the first case the increase in stiffness occurs on the pipe, and in the second it occurs on the soil, although in both cases the moments are bigger. The soil bearing resistance forces also increase with the pipeline depth, but bearing capacity was never reached at any point along the pipeline. Due to the soil greater flexibility for lower depths of the pipeline layout, the soil would yield along larger areas.

An increase in the imposed guillotine settlement also increases the pipeline internal generalized forces. Moreover, displacements and forces along the pipeline and soil forces will be significant along greater extensions for bigger settlements. The axial forces due to thermal expansion are bigger for soils that can mobilize greater longitudinal resistance, that is for soils with higher soil friction angles, pipe depths and pipe diameters.

CONCLUSIONS

Design recommendations were synthesized for the seismic design of aboveground and buried pipeline systems. Two computational examples were included for each of the distinct pipeline situations, each of which was used as the source of comprehensive parametric studies with respect to some design parameters associated with the individual pipeline layouts. The approach used in the analyses and the applied methodologies appear to be efficient for the design of these special structures in a personal computer. Some relevant parametric results were obtained characterizing the behaviour and performance of aboveground and buried pipelines under potential seismic actions along their layout.

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