Pipeline Seismic Design and Potential Mitigation Measures

by

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ABSTRACT

During the next decades hundreds of onshore or offshore oil and gas pipelines will be constructed all over the world. It is evident that the stress analysis of these pipelines is one of the most important issues of their design. However, in some areas that are characterized by moderate or high seismicity the design will be much more demanding and challenging, since many issues are directly or indirectly associated to a potential earthquake. The current paper aims to illustrate the main topics of geotechnical earthquake engineering and soil-structure interaction that have to be coped with for the proper design of pipelines. In the first part of the paper the main earthquake-related geohazards are briefly described. Seismic wave loading is the main dynamic loading for a pipeline, while quasi-static permanent deformations caused by an active-fault rupture, seismic slope instabilities, and/or soil liquefaction phenomena may be also detrimental. Emphasis is given on the second part of the paper that deals mainly with the numerical simulation of the static and dynamic interaction between the soil and the pipeline. Finally, the paper deals with the potential mitigation measures that may be adopted in the case of excessive pipeline distress. It has to be emphasized that the provisions of seismic standards, such as EC8, are not capable to cover sufficiently all the aforementioned issues. It is shown that the complexity of the specific problems requires advanced modelling and realistic simulation on a case-by-case basis. Characteristic case studies of pipelines in seismic prone areas are also presented.

1. INTRODUCTION

One of the most important issues of the engineering design of oil and gas pipelines (including the interrelated structures, such as compressor stations, tanks, buildings, etc.) is the assessment of all potential geohazards. Geoscientists and engineers use the term “geohazard” to describe the hazards to the pipeline that may derive from any potential gravity-related geological / geotechnical problem or failure, such as slope instabilities, landslides, rockfalls, ground settlements, etc. It is evident that the safety of any pipeline is directly related to (a) the verification of the pipeline against the identified and quantified geohazards, and (b) the proposal and the design of any mitigation or protection measure in case of excessive pipeline distress.

However, in areas characterized by moderate to high seismicity the geohazard assessment requires the identification of all the hazards that are somehow related to the seismic activity. In the case of a moderate or strong earthquake the varying (both in time and space) seismic motion at the ground may impose additional distress to the pipelines, which is usually described by the term “seismic wave loading”. However, a seismic event may also aggravate the aforementioned gravity-related geohazards by triggering a slope instability (such as a landslide or a rockfall) and/or may cause additional geohazards to the pipeline (such as the rupture of an active fault or soil liquefaction phenomena). It has to be noted that the “permanent ground deformations” that may be caused by fault ruptures, soil liquefaction phenomena, and/or seismic slope instabilities are of great importance in the seismic design of a pipeline since they are regarded in general as a more severe loading than seismic wave loading. Therefore, the engineering design should include the pipeline seismic design, which actually consists of:

a) Verification of the pipeline against the seismic wave loading and all types of permanent ground deformation.

b) Optimum design of mitigation (and/or protection) measures. It is evident that these measures are required only in the case that the corresponding verification is not satisfied.

To estimate with relative accuracy the seismic wave loading and the permanent ground deformations along the pipeline, a Geotechnical Earthquake Engineering Study is required which will be based on the following surveys/studies:

a) Topographic Survey. The survey should be performed in a relatively wide zone along the pipeline route to capture all the topographic features of the area under examination.

b) Geological Mapping / Survey. The survey should include a detailed description of the geological formations and a qualitative identification of the potential geohazards under static and mainly under seismic conditions (karst phenomena, landslides, rockfalls, liquefiable areas, etc).

c) Tectonic (or Seismotectonic) Survey / Study. In earthquake-prone areas, the identification and classification of the active (or the probably active) faults are absolutely essential. As it was mentioned above, an active fault or an active fault zone may imply substantial permanent ground deformations that will cause additional distress on the pipeline. Therefore, the tectonic study should also include a realistic estimation of the potential rupture to quantify the expected drift.

d) Seismological Study. The study aims to the deterministic and/or probabilistic estimation of the reference peak ground acceleration at bedrock, a_{gR}. For a normal pipeline, a_{gR} has to be calculated for various return periods T_R (or equivalently for various probabilities of exceedance P_R), depending on the limit states under consideration. Note that for less important structures the seismological study may be avoided (provided that the seismic zonation maps of the area under examination are regarded as sufficient).

e) Geotechnical and Geophysical Surveys / Investigations. The study should focus on the problematic areas. It will be based mainly on a geotechnical survey / investigation (in-situ and laboratory tests), and secondarily on a geophysical survey (cross-hole or down-hole tests). The aim is to identify the soil profile (thickness of the soil layers, valley morphology, water table level, etc) and to determine the mechanical properties of the various geological formations. The geophysical survey aims to estimate the shear-wave velocity (V_S) of the soil layers.

It is evident that, after the geotechnical and geophysical surveys / investigations, an experienced geotechnical engineer has to evaluate the geotechnical parameters, and to compile the Ground Investigation Report, which according to EN 1997 (Eurocode 7) consists of:

a) the presentation of all available geotechnical information including geological features and relevant data

b) a geotechnical evaluation of the information, stating the assumptions made in the interpretation of the test results.

The Geotechnical Earthquake Engineering Study aims to realistically quantify the aforementioned geohazards, leading to the quantities required for the engineering design (such as safety factors, acceleration levels, permanent displacements, etc.). The study should include at least the following:

a) Amplification study. This study is mainly performed by ground response analyses in one dimension (1-D) or more preferably in two dimensions (2-D). The analyses are required to estimate the design ground acceleration a_g at various locations, which actually determines the seismic wave loading of the pipeline. The amplification study is based on the findings of the seismological study and of the geotechnical and geophysical surveys, taking realistically into account the potentially non-linear dynamic soil behaviour.

b) Estimation of the liquefaction susceptibility. Given the calculated acceleration levels and the geotechnical findings, the liquefaction potential can be quantified. Note that in liquefiable areas with topographic irregularities (such as a river bank, a shore of a lake, or a sea-shore) the phenomenon of lateral spreading may also occur, leading to substantial lateral soil movements.

c) Seismic slope stability assessment. It requires all the prerequisite surveys/studies as well as ground response analyses. Since the pipeline under examination may be capable to withstand a certain level of deformation (axial and/or bending), the permanent deformations of a slope should be calculated with relatively high accuracy. Since many pipelines cross hilly or mountainous areas, slope stability assessment (both static and seismic) is a very important issue of the design. Note that, as it is described in Antoniou et al. (2012), rockfalls is regarded as a special case of instability of rock slopes and require special treatment.

Given the Geotechnical Earthquake Engineering Study, the Pipeline Seismic Design follows with the verifications of the pipeline against seismic wave loading and against the expected permanent ground deformations. These verifications may be performed with (semi-) analytical methods of the literature and/or numerical simulations (e.g. finite elements) with various levels of sophistication. Depending on the circumstances, it is evident that the seismic design should include the proposal and the design of various mitigation and/or protection measures.

The current paper, being an extension of Psarropoulos et al. (2012), aims to illustrate the main topics of geotechnical earthquake engineering and soil-structure interaction that have to be coped with for the proper seismic design of pipelines. In the first part of the paper the main earthquake-related geohazards are briefly described. Seismic wave loading is the main dynamic loading, while quasi-static permanent deformations caused by active-fault ruptures, seismic slope instabilities soil, and/or liquefaction phenomena may be detrimental for a pipeline. Emphasis is given on the second part of the paper that deals mainly with the numerical simulation of the interaction between the soil and the pipeline. Finally, the paper deals with the potential mitigation measures that should be adopted in the case of excessive pipeline distress. It has to be emphasized that the provisions of seismic standards, such as EC8, are not capable to cover sufficiently all the aforementioned issues. It is shown that the complexity of the specific problems requires advanced modelling and realistic simulation on a case-by-case basis. Characteristic case studies of pipelines in seismic prone areas are also presented.

2. EARTHQUAKE-RELATED GEOHAZARD ASSESSMENT

2.1. General

As mentioned previously, the main earthquake-related geohazards can be categorized to (a) seismic wave loading, and (b) permanent ground deformations. Seismic wave loading is actually a dynamic loading caused by the seismic ground motion. Permanent ground deformations are regarded in general as a type of loading more severe than the seismic wave loading since the strains induced to the pipeline by permanent ground deformations may become fairly large, leading thus to a failure (either due to tension or due to buckling). These deformations may be induced by faulting, slope instabilities, and/or ground displacements induced by soil liquefaction phenomena.

2.2. Seismic wave loading

Records and analyses in the past have shown that, apart from the soil stratigraphy, the geomorphic and the topographic conditions of an area tend to alter the amplitude, frequency content, duration, and spatial variability of the seismic ground motion, and consequently of the seismic wave loading of any structure.

In the seismic analysis and design of important and/or sensitive structures, an amplification study and the corresponding ground response analyses are regarded as an essential initial step for the assessment of the seismic wave loading. Especially in the case of long structures, such as pipelines or bridges (which usually cross valleys and/or topographic irregularities), the success in calculating the seismic distress depends primarily on the ability of the geotechnical earthquake engineer to estimate realistically the level of the seismic wave loading on the surrounding soil under free-field conditions (i.e. without the existence of the structure). The dynamic stress field developed in the soil is a function of the characteristics of excitation at the base of the soil deposit and the local site conditions. In general the term “local site conditions” is being used to describe both material (soil), geomorphic, and topographic conditions.

The amplification study and the ground response analyses shall be based on the available geological / geotechnical studies / surveys (definition of seismic bedrock, soil profile – classification, and soil properties), and the seismological data at seismic bedrock (peak ground motion parameters, response spectra, and accelerograms).

2.3. Fault rupture

Seismic distress may be imposed on engineering structures and infrastructures not only due to seismic wave loading, but also due to a fault movement. The vulnerability of various engineering structures to permanent displacements resulting from fault movement has been observed during several earthquakes.

Given the pipeline route, the procedure of allocation, identification and quantification of the potentially active faults includes the following stages:

a) The first stage includes the allocation via remote-sensing and analysis of topographic data (terrain analysis). The results of the aforementioned procedures will be again cross-checked against the available seismological data.

b) The second stage includes the so called “ground-truth” process, which will be conducted in combination with the geological mapping of the pipeline zone.

The aforementioned procedure, usually described as Tectonic (or Seismotectonic) Survey / Study, provides qualitative and quantitative data for the characterization, in terms of activity, geometry, displacement and kinematics of the allocated fault zones. More specifically, the main data are the location, the size of the area affected (fault zone), the type and the estimated cumulative offset (measure) of the fault displacement. The anticipated per-event surficial displacement is a value that can be obtained by empirical formulas such as Wells & Coppersmith (1994) and Ambraseys & Jackson (1998).

2.4. Seismic slope instability

Since the pipelines are long structures, their route is expected to cross regions of high risk of landsliding. It is evident that in earthquake-prone areas the risk is increased as a seismic event may increase the driving forces, triggering thus a potential landslide. Consequently, after the identification of these regions in the geological survey, the geotechnical engineer has (a) to evaluate the slope stability under static conditions, and (b) to assess realistically the seismic slope stability.

Seismic slope stability assessment is performed with the application of methods which are grouped according to the adopted mathematical model in three main categories: (a) pseudostatic, (b) permanent deformation or sliding block, and (c) finite element or stress deformation. The simplified methods have been prevailing in the current practice partly because of the increasing complexity of more elaborate finite element models, which require the definition of stress – strain soil response under cyclic loading. However, the application of these methods is based on major underlying assumptions.

The main issue raised in the pseudostatic method is the selection of the so called seismic coefficient. The latter is defined as the ratio of the constant seismic force acting on the potential failure surface divided by the weight of the failure wedge. The approximation of a constant seismic coefficient becomes an erroneous selection since: (a) near the slopes the role of topography effects is predominant, hence the magnitude and the frequency content of the acceleration response time history varies throughout the potential failure surface, and (b) the time-varying nature of the dynamic response indicates that severe loading lasts only instantly. The conservatism of the method arising from the negligence of both spatial and time variation of the inertia forces was early recognized and seismic coefficients calibrated to acceptable level of displacements were proposed for dam design. Modern guidelines for the evaluation of seismic induced landslides, such as the Guidelines for evaluating and mitigating seismic hazards in California (CGS-2008), propose the dependence of the seismic coefficient on the peak ground acceleration at the bedrock, the distance from the seismic source and the acceptable seismic displacements.

The permanent deformation methods are pertinent modifications of the Newmark's sliding block approach. This approach is based on the fundamental assumption that stability may be established according to a simple model, which consists of a rigid block on an inclined plane, and therefore displacements are obtained by double integration of the relative acceleration. Relative acceleration is the difference between the applied and the critical (or yield) acceleration, where the latter refers to the value of the acceleration required to approach incipient sliding state i.e. factor of safety equal to unity. The most influential assumption of this method is the negligence of the flexibility of the sliding mass. Ever since Newmark's pioneering study, two different approaches have been proposed to overcome this limitation: the decoupled procedure where the dynamic response of the examined failure surface is calculated separately from the induced displacements, and the coupled procedure where the dynamic response is considered simultaneously to the permanent displacement development by the direct solution of the governing differential equations.

It has to be emphasized that, although in the static slope stability analyses safety factors SF lower than 1,0 are unacceptable (since they correspond to total slope failure in a limit-equilibrium analysis), in the seismic slope stability assessment values of dynamic safety factor SF_d lower than 1,0 may be accepted since in most of the cases they do not necessarily imply total failure, but accumulated permanent ground deformations. These deformations may be accepted or not, depending on the circumstances (type of structure, specifications, etc). It is evident that the less deformation accepted, the more conservative the design should be. Therefore, performance-based design could be applied (in combination with techno-economic analysis) to achieve cost-effective solution. Obviously, if zero permanent ground deformations are required for any reason ($SF_d > 1$), the design is expected to be extremely conservative, leading thus to very expensive mitigation (stabilization) measures.

2.5. Soil liquefaction phenomena

Soil liquefaction is a phenomenon in which cohesionless soil deposits below the groundwater table may lose a substantial amount of strength due to strong ground motion, potentially resulting in reduced foundation-bearing capacity, lateral spreading, settlements, and other adverse effects. Liquefaction and related phenomena have been responsible for tremendous amounts of damage in many earthquakes around the world.

Liquefaction occurs in cohesionless saturated soils, that is, soils in which the space between individual particles is completely filled with water. This water exerts a pressure on the soil particles that influences how tightly the particles themselves are pressed together. Prior to an earthquake, the water pressure is relatively low. However, earthquake shaking can cause the water pressure to increase to the point where the soil particles can readily move with respect to each other.

Given that liquefaction is likely at a particular location, of most importance from an engineering perspective is to predict the amount of horizontal and vertical permanent ground deformations associated with the liquefaction. More details on the estimation of liquefaction-induced permanent ground deformations can be found in O' Rourke & Liu (1999).

3. PIPELINE VERIFICATIONS

3.1. General

In the case of a buried pipeline, the pipeline behaviour should be analyzed as a typical soil-structure interaction (SSI) problem. The term “structure” is used to describe the pipeline itself, while “soil” represents either the native soil or the backfill, depending on the circumstances. The following sections describe the basic issues of the aforementioned interaction and the required verifications of the pipeline integrity. The first section is mainly devoted to the calculation of the soil spring values, while the rest describe the following verifications:

a) Verification against seismic wave loading which includes the estimation of the distress along the pipeline due to the strong ground motion.

b) Verification against fault rupture that includes the estimation of the pipeline distress (stresses, strains, etc.) due to the potential rupture of an identified active fault.

c) Verification against slope instability that includes the estimation of the pipeline distress due to permanent ground deformations caused by seismic slope instabilities.

d) Verification against soil liquefaction phenomena.

Note that the verification of the pipeline should take into consideration the combinations of each of the aforementioned earthquake-induced loadings and the operational loading (due to gravity, internal pressure, and temperature difference).

3.2. Estimation of soil spring values

Typically, the soil compliance around the pipeline is usually represented by four translational bilinear soil springs at all directions (see Figures 1 and 2). More specifically:

a) Axial soil springs

b) Lateral soil springs

c) Vertical uplift soil springs

d) Vertical bearing soil springs

Note that soil spring forces should generally be based on the native soil properties, besides the axial springs for which soil properties representative of the backfill should be used to compute the corresponding forces.

Given the available geological and the geotechnical surveys/studies, the soil springs can be categorized in various groups along the pipeline route. Based on the data of these studies, soil spring forces F and the corresponding mobilizing soil displacements δ can be calculated according to ALA (2002) for the four soil springs.

Figure 1: The four springs around the pipeline representing the soil compliance

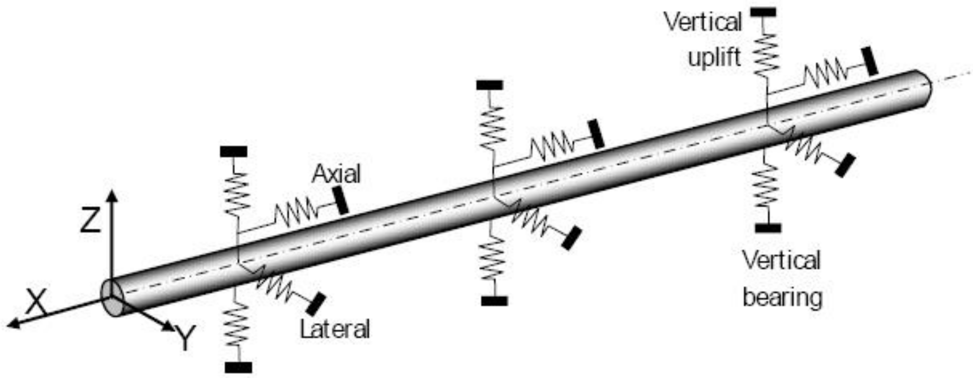
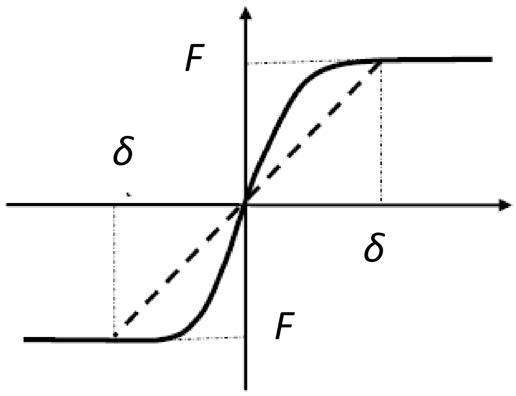


Figure 2: Idealized representation of the bi-linear soil springs



3.3. Verification against seismic wave loading

The verification of the pipeline against seismic wave loading should be based on the estimation of the maximum developed strains for both the pipeline straight sections and the pipeline bends.

According to the current state of practice, the strains induced due to seismic wave propagation on the pipeline could be calculated utilizing the analytical methods described in the corresponding design guidelines / provisions (ALA-2002 and JGA-2000) as well as in the relative literature.

Straight pipeline sections

According to ALA (2002), wave propagation provisions are presented in terms of longitudinal axial strain, that is, strain parallel to the pipeline axis induced by ground strain. Flexural strains are neglected since they are small for typical pipeline diameters.

The axial strain, ε_α, induced in a buried pipeline by seismic wave is approximated using the following equation:

$$\varepsilon_\alpha = V/C \quad (1)$$

where:

V is the peak ground velocity caused by ground shaking, and
C is the apparent propagation velocity of the seismic waves.

In general, the peak ground velocity (PGV) should be estimated realistically through the amplification analyses of the geotechnical earthquake engineering study in order to incorporate the local site effects (due to stratigraphy, geomorphologic and topographic conditions).

When such a study is not available, the seismological data and the provisions of EC8 can be used for the approximate assessment of PGV. In case of rock-outcrop the PGV values proposed by the seismological study may be used as representative. To take into account the local site conditions, EC8 proposes for the estimation of PGV values at the ground surface a soil factor S and a topographic amplification factor S_T.

Since pipelines are typically buried horizontally few meters below the surface, both body and surface seismic waves are of interest. In the following paragraphs, the several methodologies, available in the literature for the estimation of the apparent propagation velocity, C, are described.

According to the provisions of ALA (2002), regardless of the wave type, the apparent wave propagation velocity C is conservatively considered equal to 2000 m/s. An additional reduction factor of 2 is used at the denominator of Eq. 1 when shear waves (S-waves) are considered.

However, several researchers have shown that the strains induced by Rayleigh waves (R-waves) are much higher and should be considered (see O' Rourke & Liu 1999 and Hashash et al. 2001). Love waves (L-waves) are in general neglected as they generate bending strains on the pipeline which are significantly less than the axial strains induced by R-waves. The effective (phase) velocity C_R of R-wave is estimated according to the methodology proposed in O' Rourke & Liu (1999).

According to this methodology, for the case illustrated in Figure 3(a) of a single soil layer over a half-space, the phase velocity of Rayleigh waves C_R is determined utilizing a normalized dispersion curve. The dispersion curve is described by the following expressions:

$$C_R = \begin{cases} 0.875V_{S,2}\ , & for\ \dfrac{H_1 f}{V_{S,1}} \le 0.25 \\ 0.875V_{S,2} - \dfrac{0.875V_{S,2} - V_{S,1}}{0.25}\left(\dfrac{H_1}{V_{S,1}} - 0.255\right), & for\ 0.25 \le \dfrac{H_1 f}{V_{S,1}} \le 0.50 \\ V_{S,1}\ , & for\ \dfrac{H_1 f}{V_{S,1}} \ge 0.50 \end{cases}$$

where

$V_{s,1}$ and H_1 are the wave velocity and the thickness of the surface soil layer, respectively,
$V_{s,2}$ is the wave velocity of the half-space, and
f is the frequency in Hz.

The methodology has been also extended to multiple soil layers. Two cases are examined:

a) the surface layer lies over a half-space with the properties of the second soil layer, and
b) both the surface and the underlying layer are considered as one equivalent layer lying over the "rock" half-space (Figure 3(b)).

Note that the implementation of the aforementioned analytical methodologies should be performed both for the Damage Limit State (DLS) and Ultimate Limit State (ULS).

According to the current design recommendations (e.g. ALA-2002), the maximum strain on the pipeline induced by the seismic wave cannot exceed the ultimate strain $\varepsilon_{\alpha,ult}$ induced on the pipeline by the soil-pipeline interface friction which is given by the following expression:

$$\varepsilon_{a,ult} = \frac{\tau_u\, L_a}{4\mathrm{EA}} \qquad (2)$$

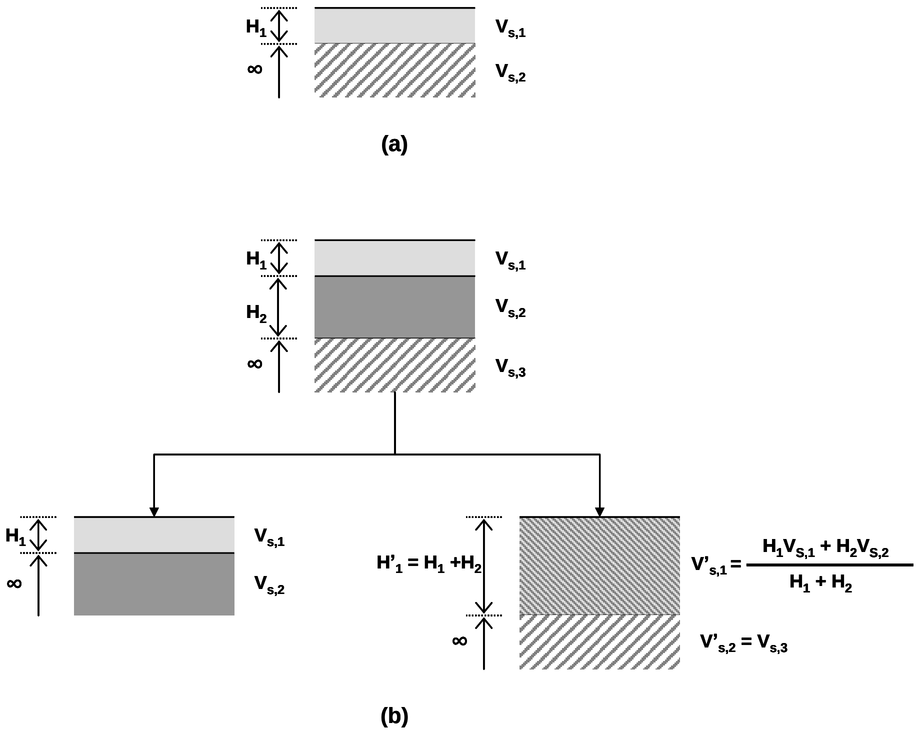
where:

τ_u is the maximum frictional force per unit length at soil-pipe interface,
L_a is the apparent wavelength of seismic wave (taken as 1 km, according to ALA-2002),
E is the modulus of elasticity of steel, and
A is the cross-sectional area of the pipeline.

However, since for seismic wave loading it is regarded risky to rely on the friction developed on the soil-pipeline interface, the values of ε_α (for DLS and ULS) should be conservatively adopted at the verification process of the straight pipeline segments.

The differences between the aforementioned analytical methodologies are attributed mainly to the wave type considered (S waves or R waves) and secondarily to the wave propagation velocity adopted.

Figure 3 Idealization of complex soil profiles for estimation of the Rayleigh wave phase velocity according to O' Rourke & Liu (1999).



Pipeline bends

The strains on the pipeline bends could be estimated through various methodologies. An indicative methodology has been proposed by Ogawa & Koike (2001) and recommended by the JGA (2000) for the earthquake-resistant design of pipelines.

According to this methodology, the maximum bending strain ε_b on a pipeline bend is given by the following expression:

$$\varepsilon_b = \beta_b \, \Delta \qquad (3)$$

where:

β_b is a conversion factor from the relative displacement to the structural strain of the bend, and
Δ is the soil-pipe relative displacement due to seismic wave loading.

The relative displacement Δ is a function of the anticipated seismic ground displacement U_h which is related to the ground strain ε through the expression:

$$U_h = \varepsilon \frac{L_a}{2\pi}$$

where L_a is the apparent wavelength of seismic wave.

Two approaches could be followed for the calculation of the apparent wavelength. According to ALA (2002), the ground strains ε are calculated for S-waves and an apparent wavelength of 1000m is assumed, while Ref. [11] proposes that the ground strains ε are calculated for R-waves and the wavelength is estimated as $L_\alpha = C_R / f$, where C_R is the wave propagation velocity and f the seismic wave frequency. For the typical frequencies of seismic excitations (up to 10 Hz), it is proven that the maximum strains appear for frequency f = 5 Hz.

It has to be underlined that, given the radius of curvature R of the bend, the pipeline flexibility factor n and the stress intensification factor i could be calculated according to ASME (2007).

3.4. Verification against fault rupture

In case that any active faults are identified along the pipeline route, representative numerical models should be developed, taking into account the characteristics of fault rupture and the geological / geotechnical data. The verification of the pipeline against fault rupture should be performed utilizing a finite-element tool. For this purpose, two-dimensional (2-D) or three-dimensional (3-D) models are recommended to be developed, considering the soil – pipeline interaction. Note that in the case of a surface fault scarp not clearly identified, the soil shall be modelled as a continuum, aiming to determine the fault rupture propagation path and the distribution of the deformation at the pipeline.

According to ALA (2002), the approach to evaluate pipeline response to a fault rupture requires finite element analyses that account for non-linear soil and pipeline behaviour. It is noted that this specific approach is similar for all cases of imposed permanent ground deformations PGD (e.g. faulting, slope instabilities, etc.).

The analysis of the pipeline could be performed with two three-dimensional (3-D) numerical models:

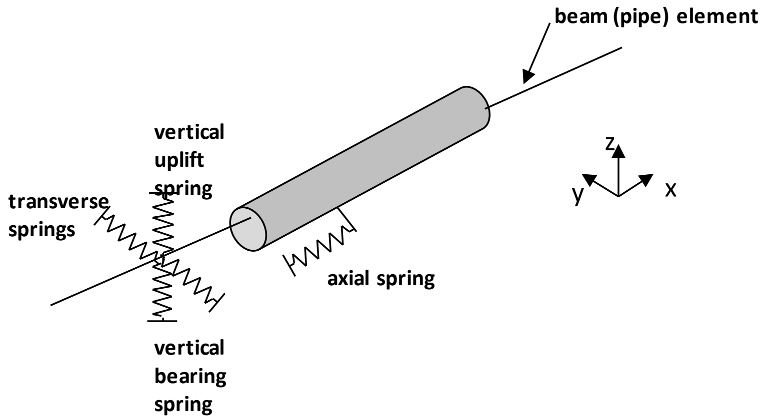
a) a finite-element model in which the pipeline is simulated with beam elements, and

b) a finite-element model in which the pipeline is simulated with shell elements.

Beam-element model

A 3-D beam-on-nonlinear-Winkler-foundation finite-element model (BNWF) can be utilized for the estimation of the pipeline response to permanent ground deformation. In this model the pipeline can be simulated through beam elements resting on springs which represent the soil surrounding the pipe. A sketch of the specific model is presented in the following figure.

Figure 4: Sketch of the beam element model.

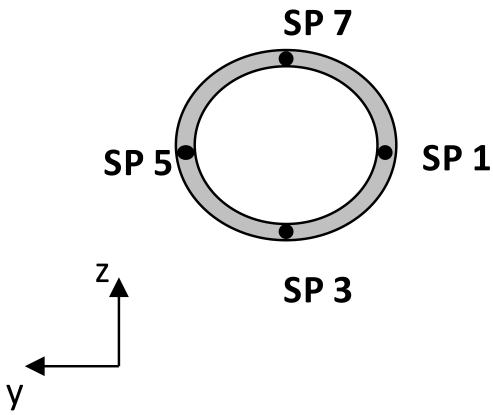


The nonlinear response of the soil (axial and transversal) is simulated through the four bilinear springs assessed in Section 3.2. (axial, transverse, vertical-uplift, and vertical-bearing). Figure 5 depicts a close-up of a pipeline beam-element model.

Figure 5: Close-up of a pipeline beam-element model. The springs simulating the soil around the pipeline are discerned. Note that the axial spring is not visible as it was simulated using a vertical oriented spring acting though in the horizontal (axial) direction.

The pipeline can be simulated through pipe elements which can incorporate the effects of stressing due to internal pressure and calculate the corresponding hoop stresses and strains. In addition to the stresses and strains being calculated for the whole section through section integration, values are provided also for the section integration points shown in Figure 6.

Figure 6: Pipe section points where the stresses and strains should be calculated.



In this way, it is possible to estimate simultaneously both tensile and compressive stress/strain at every cross-section along the pipeline. The nonlinear stress-strain relationship of the pipe material should be considered through a plasticity model, while large displacement effects should also be taken into account.

While analyzing the pipeline for permanent ground deformation (PGD), it is assumed that the development of ground deformation is gradual. Hence, pseudo-static analysis is applied for pipelines subjected to PGD. The ground deformation (in this case due to faulting) is assigned at the fixed ends of the soil springs of the hanging wall. The damping and inertia effect can be ignored in this analysis.

It is emphasized that the analysis should be conducted assuming that the pipe is fully operational (i.e. internal pressure and temperature difference). That means that the calculated maximum axial strain is attributed not only to the fault rupture, but to the operational loads as well.

Shell-element model

In the second model the pipeline consists of shell elements in order to capture stress and strain concentrations in a more accurate way. The total length of the model should extensively cover the fault area. Similarly to the beam-model, the surrounding soil can also be simulated with the bilinear springs described in Section 3.2. (see Figures 7 and 8).

The pipeline section can be discretized along the periphery, while springs are attached at all nodes in all directions. The values of the springs are assumed to be a function of the projected area of the cross section in the corresponding direction. Internal pressure should be modelled as a uniformly distributed load on the internal face of all shell elements, while the fault movement should be applied as an imposed displacement at the free ends of the soil springs in half of the model.

Figure 7: Detail of the 3D shell-element model and the surrounding soil springs.

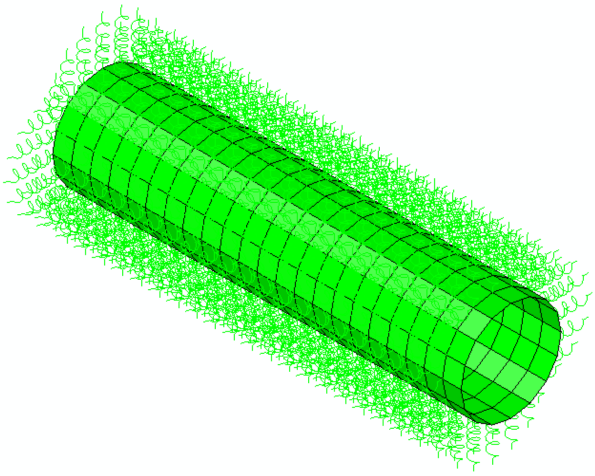
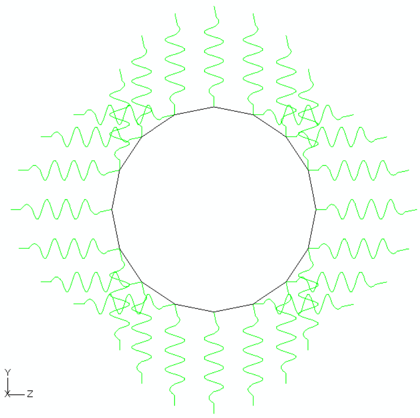


Figure 8: Cross section of the 3D shell-element model and the surrounding soil springs.



The following two figures show representative results of a three-dimensional finite-element simulation conducted recently by the authors in order to estimate the kinematic loading of a gas pipeline crossing an active fault with a rupture of the order of 0.6 m. Figure 9 shows the deformed shape and contours of vertical displacements of the pipeline in the area of the fault, while the axial strains of the pipeline are presented in Figure 10.

Note that in this case the estimated pipeline distress was excessive (according to the seismic norm EN 1998), and thus mitigation measures have been proposed. More details on the potential mitigation measures can be found in Section 4.3.

Figure 9: Deformed shape and contours of vertical displacements of a gas pipeline in the area of a fault (with deformation scale factor = 10).

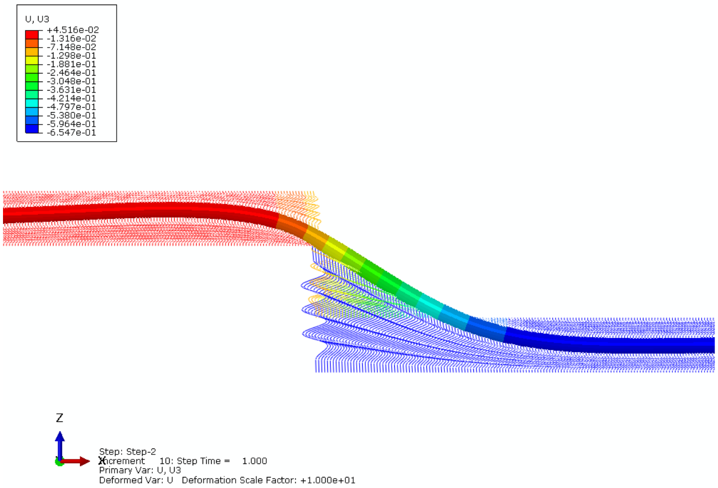
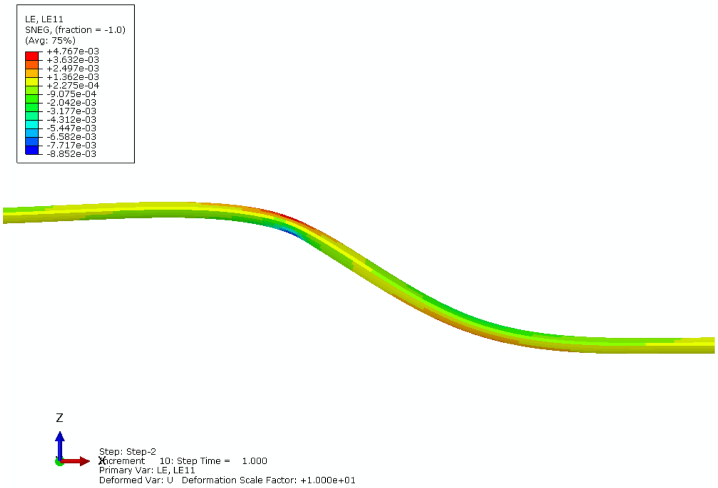


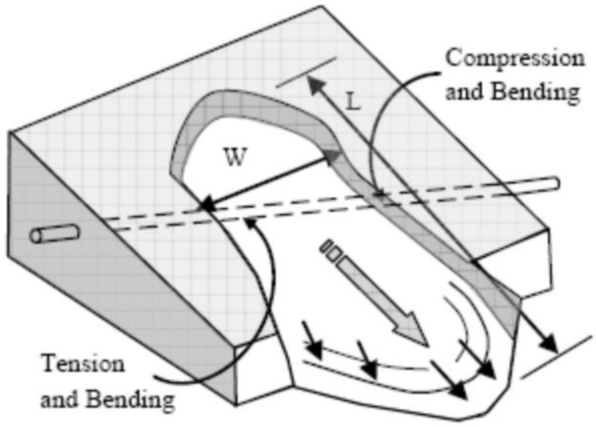
Figure 10: Deformed shape and contours of axial strains of a pipeline in the area of a fault (with deformation scale factor = 10). The maximum compressive strain is depicted in blue, while the maximum tensile strain is depicted in red.



3.5. Verification against slope instability

In case of a slope instability, there are many patterns of permanent ground deformation which depend on the local geological / geotechnical conditions. As depicted in Figure 11, a pipeline may cross the permanent ground deformation zone in any arbitrary direction. However, verifying the pipeline against slope instability, the engineer has to examine separately the parallel crossing and the perpendicular crossing. The parallel crossing will lead to tension at the upper part of the zone and compression at the lower part of the zone, while the perpendicular is expected to cause bending.

Figure 11: Pipeline crossing a landslide area at arbitrary angle (IITK-GSDMA 2007).



As mentioned before, in the Geotechnical Earthquake Engineering Study the permanent ground deformations under the expected seismic excitation will be estimated for the two limit states examined (DLS and ULS). In case of nonzero soil deformations due to seismic slope instability, the pipeline shall be verified against the imposed seismic deformations.

The verification of the pipeline shall be performed utilizing a finite-element tool. For this purpose, three – dimensional (3-D) models are recommended to be developed, considering the soil – pipeline interaction. For the evaluation of the pipeline distress due to these slope instabilities, the beam-element model developed for the fault rupture simulation (and described in the previous Section) could be adopted here as well. The anticipated PGDs are applied on the ends of the soil springs located on the unstable soil mass that is defined by the corresponding failure mechanism. It is emphasized that the analysis should be conducted assuming that the pipe is fully operational (i.e. internal pressure and temperature difference). That means that the calculated maximum axial strains are attributed not only to the slope instability, but to the internal operational loads as well.

If the calculated stresses on the pipeline are excessive, the engineers should propose various mitigation measures. More details regarding the potential mitigation measures are given in Section 4 of the current paper.

3.6. Verification against liquefaction

As it was mentioned, the phenomena of soil liquefaction and of lateral spreading cause horizontal and/or vertical permanent ground deformations. The verification of the pipeline could be based on the numerical models described previously applying the estimated permanent ground deformations. However, it is evident that in the case of liquefaction where a pore-pressure build-up takes place, the soil spring values adopted in the numerical models should be reduced accordingly.

4. MITIGATION & PROTECTION MEASURES

4.1. General

In areas where the pipeline distress due to the examined geohazards will be unacceptably excessive, the relocation of the pipeline to avoid the problematic area(s) would be an option. However, since the pipeline relocation may be impractical or even impossible for various reasons, mitigation and/or protection measures should be adopted aiming to eliminate or reduce the imposed pipeline distress to acceptable levels. It is evident that the final geometrical and mechanical properties of any adopted measure, along with its impact on the pipeline distress, should be verified by detailed geotechnical investigation and simulations on a case-by-case basis. Given the special characteristics of the problematic area, the selection of any mitigation or protection measure should take into consideration various parameters, such as environmental impact, constructability, accessibility, cost, etc.

4.2. Seismic wave loading

Despite the fact that for buried pipelines seismic wave loading has been proven to be less severe than permanent ground deformations, the verification of the pipeline against seismic wave loading should be performed. If the axial strain of the pipeline (either in tension or in compression) is excessive, and since it is impossible to diminish seismic wave loading, various mitigation measures may be adopted for the pipeline along a specific spatial extend. One approach to protect the pipeline is to increase locally the pipe wall thickness, while an alternative is to increase the pipeline flexibility utilizing joints.

4.3. Fault rupture

In case of a crossing with a potentially active fault, the following measures are proposed to cope with the expected fault rupture:

a) An increase in pipe wall thickness. This measure will increase the capacity of the pipeline to withstand fault movement at a given level of maximum tensile strain. On each side of the fault relatively thick-walled pipe should be used.

b) Reduction of the angle of interface friction between the pipeline and the soil. This reduction increases the capacity of the pipeline to withstand fault movement at a given level of maximum strain. The angle of interface friction can be reduced through a hard, smooth coating.

c) Close control should be exercised over the backfill surrounding the pipeline over a certain distance on each side of the fault. In general, a loose to medium granular soil without cobbles or boulders will be a suitable backfill material. If the existing soil differs substantially from this, oversized trenches should be excavated for a specific distance on each side of the fault. Figures 12(a) and 12(b) show sketches of the measure of oversized trenches. This mitigation measure was proposed by the authors in the case of a gas pipeline crossing an active fault.

If a slight relocation of the pipeline is possible, then the pipeline crossing a fault should be oriented in such a way as to place the pipeline in tension (and not in compression). Additionally, in fault zones the depth at which the pipeline is buried should be minimized in order to reduce soil restraint on the pipeline during fault movement.

Figure 12(a): Sketch of the plan view of the mitigation measure proposed for the fault crossing.

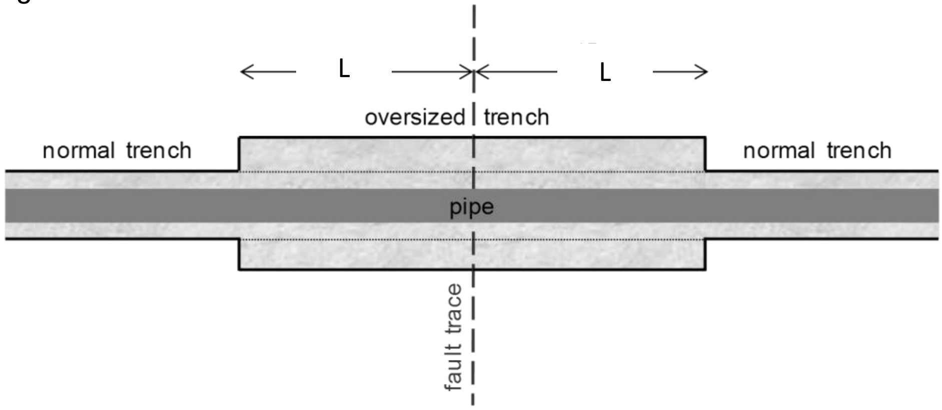
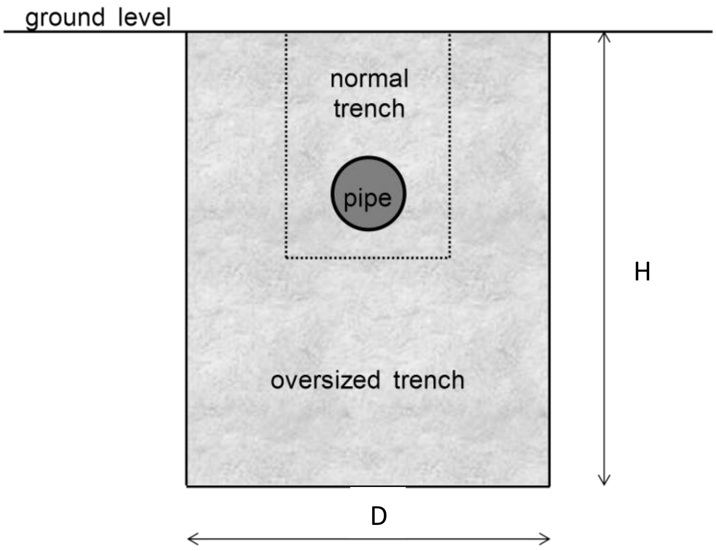


Figure 12(b): Sketch of the cross section of the mitigation measure proposed for the fault crossing.



4.4. Slope instabilities

Since in pseudo-static (and static) analyses the factor of safety (FS) is defined as the ratio of the resistance against instability over the cause of instability, the stabilization of the slope may be achieved by:

a) increasing the resistance using an embankment at the toe of the slope, or a retaining structure (such as a sheet pile wall or a gravity wall),

b) reducing the cause of the instability (by changing the slope inclination or lowering the ground water level)

c) soil improvement (usually performed by soil reinforcement)

As it was aforementioned, besides the pseudostatic analyses, a geotechnical engineer could alternatively (a) estimate the expected permanent ground deformations at the slopes, and (b) verify the pipeline against these deformations. If the pipeline distress is excessive, there exist two ways to proceed. The first is to stabilize the slope adopting one or more of the aforementioned stabilization measures. The second is to change the characteristics of the pipeline either by increasing the pipe wall thickness or increasing the pipeline flexibility.

It has to be emphasized that since any pipeline is capable to withstand a certain level of permanent ground deformations, the adoption of stabilization measures based on the pseudostatic concept (that ignores the permanent deformations) is a-priori a conservative approach, increasing thus the overall construction cost.

Regarding rockfalls (which is a special case of instability), apart from the stabilization of the potentially unstable rock masses above the pipeline with various methods, one could adopt:

(a) an active method of pipeline protection with stoppers, barriers and/or wire fences to prevent any impact of the rockfall on the pipeline, or

(b) a passive method of protection with an increased overburden or an overburden made of synthetic smooth material (such as corpuscles of expanded polystyrene) to protect the pipeline in case of an impact.

In any case, analyses are required to design the optimum mitigation measure, depending on the circumstances. Note that in order to perform any analysis and to propose any mitigation/protection measure against rockfalls, a special geological study of the expected rockfalls is required. The study should include estimation of potential rockfall volume, rock mass properties, dip and dip orientation of joints, wedge or planar failures, etc.

4.5. Soil liquefaction & lateral spreading

In liquefiable areas there exist two potential ways to protect the pipeline from excessive distress:

a) reduction of ground deformations due to the liquefaction phenomena, and

b) isolation of the pipeline from the damaging ground deformations.

Since soil liquefaction (and lateral spreading) is more likely to occur in loose to moderately saturated granular soils with poor drainage, the reduction of the liquefaction-induced permanent ground deformations may be achieved by (a) increasing the density and strength of sandy materials, (b) lowering the ground water level, and (c) increasing the dissipation of pore-water pressure. Finally to reduce the potential for liquefaction, one could replace liquefiable soils in the vicinity of the pipeline with non-liquefiable materials (such as gravels). Nevertheless, these mitigation measures are practical and cost effective only when the liquefiable area is limited and the liquefiable soil layer is relatively close to the ground surface.

In some cases the isolation of the pipeline from the damaging liquefaction-induced ground movements is a more practical measure. The most common way to isolate the pipeline from potential damage is to locate it below the hazardous area. This is usually achieved by directional drilling technology, which is particularly attractive at river crossings that may be susceptible to liquefaction-induced permanent ground deformations of the bank.

5. SEISMIC NORM PROVISIONS

5.1. Seismic wave loading

The first part of EN 1998 (EC8) recognizes that the seismic motion at the ground surface is strongly influenced by the underlying soil conditions. The ground conditions are categorized in five general ground types and two special ground types according to the shear-wave velocity in the top 30m, $v_{S,30}$, and/or indicative values for the number of blows evaluated with the standard penetration test, N_{SPT}, and the undrained cohesive resistance, c_u. The general ground types range from rock with $v_{S,30}$ > 800m/s (ground type A) to thick alluvium layers over stiffer materials (ground type E), while in the case of the two problematic ground types (S_1 and S_2) special amplification studies for the definition of the seismic action are required.

The design ground acceleration a_g (on the surface of type A ground) can be calculated utilizing the following expression:

$a_g = a_{gR} \cdot \gamma_I$, where:

a_{gR} is the reference peak ground acceleration on type A ground (rock). It is specified in the seismic zonation maps of each country and corresponds to the reference return period for the no-collapse requirement, T_{NCR} (which has a recommended value of 475 years).

γ_I is the importance factor which is used to take into account reliability differentiation. The recommended range of γ_I is between 0,8 to 1,4, depending on the seismic hazard conditions and on the public safety considerations.

According to EC8, the ground type influences directly or indirectly both the shape of the *elastic response spectra* S_e and the *peak ground acceleration* which coincides with the spectral acceleration S_e in the case of a completely rigid structure (T = 0 s).

Peak ground acceleration is equal to $a_g \cdot S$, where S is the *soil factor* that depends on the ground type and the type of the seismic action. As it was expected, soil factor S ranges from 1,0 in the case of rock up to 1,8 in the case of soft soil layers.

The fourth part of EN 1998 refers to the seismic design of silos, tanks and pipelines. Similarly to Part 1, Part 4 defines two separate limit states:

a) The ultimate limit state (ULS) that implies structural failure (it corresponds to the no-collapse requirement of EN 1998), and

b) The damage limitation state (DLS) that assures the structural integrity and a minimum operating level (it corresponds to the damage limitation requirement of EN 1998).

In ULS, EN 1998 – Part 4 proposes the following expression for the calculation of the design seismic action, A_{Ed}:

$A_{Ed} = \gamma_I \cdot A_{Ek}$, where:

γ_I is the importance factor. Four importance classes are defined:

- Class I (low risk) : $\gamma_I = 0,8$
- Class II (medium risk) : $\gamma_I = 1,0$
- Class III (high risk) : $\gamma_I = 1,2$
- Class IV (exceptional risk) : $\gamma_I = 1,6$

A_{Ek} is the *reference seismic action*

In DLS, a reduction factor v may be used. The factor v is equal to 0,5 for important classes I and II, and equal to 0,4 for classes III and IV.

It has to be underlined that, although EC8 takes into account the soil stratigraphy, it has no specific provisions for the potential geomorphic (valley) effects. On the contrary, for important structures ($\gamma_I > 1,0$) the topographic features of the area under examination should be taken into account by introducing the topographic amplification factor S_T which should be applied near the top of embankments and cliffs. S_T is defined in Annex A of EN 1998-5 and ranges between 1,0 and 1,4 depending on the inclination, the geometry, and the soil conditions.

According to EC8, an alternative representation of seismic action, essentially for nonlinear analysis purposes, could be a set of artificial, recorded or simulated accelerograms, provided that they are scaled to the peak ground acceleration and match the elastic response spectrum for 5% damping.

Note that in the case of sensitive structures, such as long bridges or pipelines, the design ground acceleration a_g (or the design seismic action A_{Ed}) and the corresponding spectral values should be evaluated for various hazard levels (return periods) by the performance of a detailed seismological study, while the impact of the local site conditions on the seismic motion of the ground surface has to be estimated by an amplification study that will take into account not only the soil stratigraphy, but the geomorphology and the topography of the area under examination as well. In any case it is recommended to compare the acceleration levels derived from the amplification studies with the corresponding values proposed by EC8 and seismic zonation of the National Annexes. If the amplification studies lead to lower acceleration levels than those proposed by EC8, it is recommended the EC8 provisions to be applied in the pipeline seismic design.

5.2. Soil liquefaction

Annex B of EN 1998 – Part 5 provides empirical charts for simplified evaluation of liquefaction potential. The charts refer to clean sands and silty sands and they are based on the standard penetration test (SPT) blowcount value normalized for overburden effects and for energy ratio $N_1(60)$.

5.3. Slope instabilities

As far as the seismic slope stability assessment is concerned, EN 1998 allows the design engineer to select among the different mathematical models when abrupt irregularities in topography and soil stratigraphy are not present, and mechanical behavior of soil is not sensitive to cyclic loading (strength degradation or pore pressure built up). Moreover, EC8 proceeds to suggestions with respect to the limitations of each one of the aforementioned simplified methods. Regarding the selection of the seismic coefficient, it is stated to be assigned at the "least safe potential slip surface", while it principally corresponds to "the ultimate limit state beyond which unacceptably large permanent displacements of the ground mass takes place". Hence even though the definition of the unacceptable displacements is not clearly stated, the horizontal seismic coefficient is set to be equal to 50% of peak acceleration at slope surface irrespectively of the depth of the failure surface. Moreover, the serviceability limit state is suggested to be checked after permanent deformation analyses of rigid block models, with the application of recorded earthquake time histories at the ground surface. The frequency content of the seismic motion is essentially accounted for, but not the interaction of the dynamic response and the slip displacement accumulation. Note that neglecting the dynamic response of the failure surface has been proven to be risky.

5.4. Fault rupture

Regarding *faulting*, EN 1998 – Part 5 prohibits the construction of buildings in the immediate vicinity of tectonic faults recognized as being seismically active in official documents issued by competent national authorities. For urban planning purposes and for important structures to be constructed near potentially active faults in areas of high seismicity, special geological investigations should be carried out in order to determine the ensuing hazard in terms of ground rupture and the severity of ground shaking. According EN 1998, in long structures (such as pipelines and bridges) crossing potentially active tectonic faults, the probable discontinuity of the ground displacement should be estimated and accommodated either by adequate flexibility of the structure or by provision of suitable movement joints. However, EN 1998 – Part 4, that provides principles and application rules for the seismic design and for the evaluation of the earthquake resistance of buried pipeline systems, proposes the mitigation measures against fault rupture that have described in Section 4.5 of the current paper.

6. CONCLUSIONS

In areas characterized by moderate or high seismicity the seismic design of any structure or infrastructure is definitely required. In the case of oil and gas pipelines that are long, sensitive and in many cases critical structures, their seismic design and the potentially required mitigation measures against earthquakes are prerequisites for their safety and integrity. The current paper describes briefly all the issues related to the seismic design of a pipeline, such as the identification and quantification of the earthquake-related geohazards, the verification of the pipeline against these geohazards, and the mitigation measures required to avoid the excessive distress and the consequent failure of the pipeline.

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