

# Geohazard assessment for onshore pipelines in areas with moderate or high seismicity



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## SUMMARY:

Geohazard assessment is one of the most important issues of the engineering design of onshore pipelines. Nevertheless, the geohazard assessment in areas characterized by moderate or high seismicity is more demanding, since many issues are related to the potential earthquakes. The current paper aims to illustrate the main topics of geotechnical earthquake engineering that have to be coped with. After an extensive review of the impact of local site conditions on seismic wave propagation, and consequently on ground surface motion, the paper deals with the quantitative estimation of permanent ground deformations that are regarded as more severe types of pipeline loading than seismic wave propagation. The main seismic norm provisions that are related to onshore pipelines are briefly described, while characteristic case studies in seismic-prone areas are presented to demonstrate the importance of the aforementioned issues.

*Keywords: earthquake, geohazard, seismic design, pipeline*

## 1. INTRODUCTION

Undoubtedly the geohazard assessment comprises one of the most important issues of the engineering design of onshore pipelines, including the interrelated structures, such as compressor stations, tanks, operation and maintenance buildings, etc. Geologists 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, 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 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 seismicity the geohazard assessment requires the identification of all the hazards that are 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 surface 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 slope instabilities) and/or may cause additional geohazards to the pipeline (such as the rupture of active faults or soil liquefaction phenomena). It has to be noted that the *permanent ground deformations (PGDs)* 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 type of loading than *seismic wave loading (SWL)*. Therefore, the *engineering design* of a pipeline should include the *seismic design*, which actually consists of (a) the verification of the pipeline against *SWL* and against *PGDs*, and (b) optimum design of mitigation and/or protection measures (required only in the case that the corresponding verifications are not satisfied). To estimate with relative accuracy both *SWL* and *PGDs* along the pipeline route, a *geotechnical earthquake engineering study* is required that is 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 (slope instabilities, karst phenomena, 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 PGD 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. Note that usually the offset is estimated at the rock outcrop or at the seismic bedrock.
- d) *Seismological study*. The study aims to the deterministic and/or probabilistic estimation of the reference peak ground acceleration at the rock outcrop,  $a_{gR}$ . 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 study / investigation*. 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.

The *geotechnical earthquake engineering study* aims to realistically quantify the aforementioned geohazards, leading to the quantities that are 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 on the ground surface, which actually determines the SWL of the pipeline. The amplification study is based on the findings of the seismological study and of the geotechnical/geophysical survey, taking realistically into account the potential 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.
- 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 expected PGDs 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.

After the completion of the geohazard quantification in the *geotechnical earthquake engineering study*, the pipeline seismic design may follow with all the verifications of the pipeline against SWL and against PGDs. 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 aims to illustrate the main earthquake-related geohazards that have to be coped with for the proper design of pipelines. In the first part of the paper the verification of the pipeline segments against SWL is described in detail after an extensive review of the impact of local site effects (such as soil stratigraphy, valley characteristics, and topography) on the ground surface motion. Emphasis is

given on the second part of the study, which deals with the distress of pipelines due to PGD. Additionally, the paper deals with the corresponding provisions of the seismic standards/norms, such as EN 1998 (Eurocode 8 : Design of structures for earthquake resistance), which officially covers only onshore pipelines, but it seems incapable to cover sufficiently all the aforementioned issues. Utilizing characteristic case studies in earthquake-prone areas, it is shown that the complexity of the specific problems requires advanced numerical modelling and realistic simulations on a case-by-case basis.

## **2. SEISMIC WAVE LOADING (SWL)**

In the seismic analysis and design of important and/or sensitive structures, the *amplification study* and the corresponding *ground response analyses* are regarded as an essential initial step. 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 SWL 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 (ground), 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).

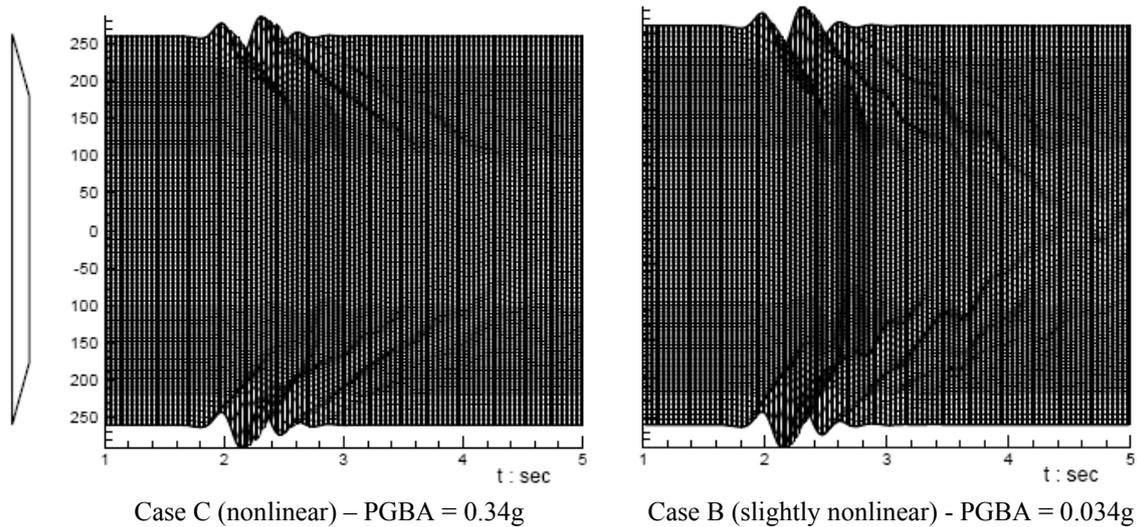
### **Geomorphic (valley) effects**

Records and analyses (e.g. Aki 1988, Finn 1991, Gatmiri & Arson 2008) have shown that, apart from the soil material conditions, the geomorphic conditions tend to alter the amplitude, frequency content, duration, and spatial variability of ground shaking. Hence, their importance in seismic design of sensitive long structures, such as pipelines, is substantial. Note that the seismological studies usually ignore the local site conditions, estimating the acceleration levels only at the “seismic bedrock”, which is generally defined as the interface between the soft soil layers and the underlying hard rock. It is evident that, in the case that soil layers do not exist the seismic bedrock coincides with the ground surface, while in many real cases experience is required to locate the seismic bedrock since the terms “soft” and “hard” are not very strictly defined.

In geotechnical earthquake engineering it is a common practice to estimate the ground seismic response performing 1-D analyses, assuming parallel soil layers of infinite extent, and neglecting thereby the potential impact of geomorphic conditions. On the other hand, objective difficulties in classifying the large variety of geomorphic and topographic features makes it a formidable task to account for these effects in simplistic, code-type prescriptions. To cope with this, 2-D (or even 3-D) site-specific ground response analyses become essential. Such 2-D linear analyses, as well as records from microtremors and small-magnitude earthquakes, have usually shown very substantial “local site effects” (Bard 1994, Yegian et al. 1994). A small number of published nonlinear analyses have seriously questioned such an “aggravation”. For instance Zhang & Papageorgiou (1996), simulating the seismic response of the Marina District Basin during the 1989 Loma Prieta earthquake and utilizing an equivalent-linear method (to account for soil nonlinearity), showed that in the case of strong-motion excitation “three-dimensional (3-D) focusing and lateral interferences, while still present, are not as prominent as in the weak-motion excitation case. Additionally, the abovementioned study underlined the fact that the energy dissipation during strong-motion excitation dampens substantially the surface waves, and thus, the response of the valley is dominated by the nearly vertically propagating waves.

Psarropoulos (2009), trying to capture any significant 2-D valley effects on the amplitude and the variability of ground shaking in an extremely soft alluvial valley in Japan, realized that the linear 2-D numerical analyses can successfully explain the recorded ground shaking. This consistency may be

attributed to the low acceleration levels of the excitation and the high plasticity index of the soil which entails elastic soil behaviour even at high deformation levels. On the other hand, equivalent linear 2-D ground response analyses have shown that a hypothetical increase of the intensity of base shaking and/or a decrease of the plasticity index of the soil may lead to substantially lower valley effects (see Figure 1). The significant energy dissipation that takes place in such a case dampens substantially the laterally propagating Rayleigh waves generated at the valley edges, while the changing with time soil modulus renders any wave resonance of vertically propagating body waves less important of vertical or inclined body waves from multiple reflections at the interfaces.



**Figure 1.** Wavefields of acceleration calculated along the surface of the valley for two (of the three) cases of nonlinearity examined (after Psarropoulos et al. 2007).

The main conclusions of the aforementioned studies are the following:

- a) Records in the literature (mainly from small earthquakes) and numerical analyses show that local site effects may have a substantial impact on the ground surface motion in the case of elastic soil behaviour. That happens when the acceleration levels of the applied excitation are low and/or when the plasticity index of the soil is high.
- b) Nonlinear ground response analyses have shown that an increase of soil nonlinearity may lead to substantially lower site effects, and consequently to lower levels of aggravation. However, since the aggravation diminishes rapidly due to the soil nonlinearity, under certain circumstances the ground surface motion of a valley during a moderate earthquake may unexpectedly be more intense than the corresponding ground surface motion during a strong earthquake.

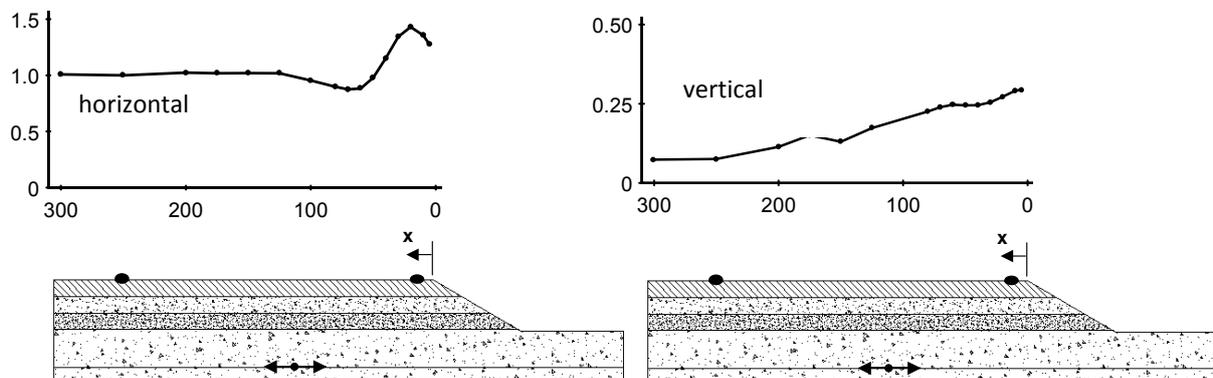
Finally, one should note that another phenomenon that may be of extreme importance for long structures, such as pipelines, and cannot be evaluated with 1-D analyses, is the *spatial variability* of ground shaking. That variability may be substantial even for relatively short distances, but is restricted only near the inclined boundaries of the valleys.

### Topographic effects

As it was mentioned before, the ground surface motion may be altered by the topographic irregularities as well. The seismic effects of “unusual” topography (meaning: non-plane ground surface, as in the case of canyons, hills, ridges, and cliffs) have been repeatedly shown to be detrimental to structures. Concentration of heavy damage near the crest of cliffs and ridges or near the top of hills and canyons has been observed in numerous earthquakes: in Miyagiken-oki 1978, Chile 1985, Germany 1978, Whittier Narrows 1987, Irpinia 1980, and Eje Cafetero–Colombia 1999. Instrumental evidence of topographic amplification is also abundant in weak seismic environments, but rather limited from strong and destructive seismic shaking. Among the few examples: the Pacoima

Dam Abutment record of the 1971 San Fernando Earthquake, two records in the Nahanni 1985 earthquake, and the astonishing records in Tarzana Hill Nursery during the Whittier Narrows 1987 and Northridge 1994 earthquakes. A large number of analytical and numerical studies have provided supporting evidence of the significance of topographic effects; methods of analysis and review summaries can be found in: Wong & Trifunac (1974), Bard (1994), Bard & Tucker (1985), Aki (1988), Sanchez-Sesma & Campillo (1991), Sanchez-Sesma (1991), Faccioli (1991), Finn (1991). However, as shown by Geli et al (1988), the simultaneous effect of heterogeneities in subsurface (soil and rock) shear-wave velocities may also be significant, although this is not so well documented (see also Paolucci et al 1999).

Psarropoulos (2001) and Gazetas et al. (2002) have been involved with the small community of Adámes during the  $M_S$  5.9 Athens earthquake in 1999, where large concentration of damage to residential and industrial buildings occurred in regions near the banks of the Kifisos river canyon. One such region that experienced unexpectedly heavy damage was Adámes, which borders the canyon at its deepest point. To explore whether the particular topographic relief and/or the actual soil profile have contributed to the observed concentration and non-uniform distribution of damage within a 300 m zone from the edge of the canyon cliff, wave propagation analyses were conducted in one and two dimensions. Finite elements (ABAQUS) and spectral elements (AHNSE) were used to this end. To avoid spurious wave reflections at the boundaries, two-dimensional (2-D) finite-element analyses utilize Bielak's *effective seismic excitation method*. Soil layering and stiffnesses were determined from ten SPT-boreholes and four crosshole tests. Ricker wavelets and six realistic accelerograms were used as excitation; two of the latter are selected from the literature and four were obtained on the basis of the four strongest motions of the earthquake, recorded in central Athens. As shown in Figure 2, the 2-D topography effects were substantial only within 50 meters from the canyon ridge, but these effects materialized only in the presence of the relatively soft soil layers that exist in the profile at a shallow depth. The so-called *Topographic Aggravation Factor* (defined as the 2-D / 1-D Fourier spectral ratio) varies from 1.5 to 2 over a broad frequency band which covers the significant excitation frequencies. At the location of four collapsed buildings, about 250 m from the edge, 2-D (topography) effects are negligible, but the specific soil profiles amplify one-dimensionally all six ground base excitations to spectral acceleration levels that correlate well with the observed intensity of damage.



**Figure 2.** Distribution of the normalized horizontal surface acceleration and the normalized “parasitic” vertical surface acceleration in the case of a Ricker pulse excitation (after Psarropoulos 2001).

### 3. PERMANENT GROUND DEFORMATIONS (PGDs)

As it was mentioned above, PGD are of great importance in the pipeline seismic design since they are regarded in general as a type of loading more severe than the seismic wave propagation. This is because the axial and bending strains induced to the pipeline by PGD may become fairly large and lead to rupture, 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.

## **Fault rupture**

Many pipelines worldwide are expected to cross areas of significant active tectonic deformation. That deformation is manifested through the current seismicity which includes several moderate or strong earthquakes recorded both instrumentally and in historical records and reports. Earthquakes may occur on distinct zones of active deformation. Nevertheless, seismicity can also be sporadic and the determination of the causative active fault(s) is not always straightforward. Another parameter that adds to the difficulty of delineating active fault zones in such areas is the fact that earthquake recurrence intervals may be long, and/or unequal. This is usually the case where an area appears to be inactive (aseismic) for several hundreds of years, but eventually is hit by earthquake clusters that may last even a few decades, before the accumulated stress is relaxed. The procedure of allocation, characterization of active faults and their correlation with the pipeline route includes two sub-stages:

- a) The first stage includes the *allocation of active faults* 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. The main data that should be included in the tectonic survey / study 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).

However, many researchers, such as Zania (2009), have shown that the pattern of permanent ground deformations at the surface depends on the fault rupture type, the induced offset at the bedrock, but mainly on the geometrical and mechanical properties of the top soil layers.

## **Soil liquefaction & lateral spreading**

Potential of liquefaction development shall be estimated at every location where the geologic and geotechnical surveys indicate that the soil formations are susceptible to liquefaction (loose saturated sandy soil). Liquefaction causes the soil to lose its strength which in turn results in the flow or lateral soil movement (lateral spreading). For the pipeline performance, two situations shall be reported in relation to lateral spreading induced permanent deformations: (a) the top of the liquefied soil layer is at ground surface and the pipeline is subjected to the horizontal force due to soil flow as well as to the uplift and buoyancy forces, and (b) the top of the soil layer is below the bottom of the pipeline and the pipeline is subjected only to the horizontal forces due to the lateral spreading.

The estimation of the liquefaction potential shall be based on empirical analytical relations that can be found in the literature (see Kramer, 1996; Towhata 2008).

## **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 to evaluate the static slope stability, and 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 (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. For more details on the seismic slope stability assessment one may refer to Zania et al. (2011).

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.

A very interesting case study of landslide triggered during a strong earthquake was the Nikawa landslide that took place during the 1995 Kobe earthquake in Japan. Nikawa landslide was one of the most devastating landslides directly related to the earthquake. With a landslide volume in the order of 110,000 m<sup>3</sup>, moving in just a few seconds over a distance of more than 100 m, it destroyed 11 residential buildings causing 35 fatalities. In addition of course to strong seismic shaking perhaps accentuated by topographic amplification, several deeper causes, such as "sliding-surface liquefaction" and water-"film" generation, have been proposed to explain rapid runoff of the slide.

Kallou (1999) examined numerically Nikawa landslide. Since Sassa et al. (1996) had proven experimentally the degradation of the shear resistance of the soil layers during the few seconds of shaking, the calculated accumulated displacements in the work of Kallou were ranging from 1 m up to 5 m, depending on the various values of friction angle of the soil.

#### 4. SEISMIC NORM PROVISIONS

Part 1 of EC8 recognizes that the seismic motion at the ground surface is strongly influenced by the underlying soil conditions. The ground conditions are categorized in 5 general 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  $V_{S,30} > 800\text{m/s}$  (ground type A) to thick alluvium layers over stiffer materials (ground type E). The *design ground acceleration*  $a_g$  (on the surface of type A ground) can be calculated utilizing the following expression:

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

$a_{gR}$  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).

$\gamma_I$  is the *importance factor* which is used to take into account reliability differentiation. The recommended range of  $\gamma_I$  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.

Part 4 of EC8 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* that implies structural failure (it corresponds to the no-collapse requirement of EC8), and (b) the *damage limitation state* that assures the structural integrity and a minimum operating level (it corresponds to the damage limitation requirement of EN 1998). In the ultimate limit state, Part 4 proposes the following expression for the calculation of the *design seismic action*,  $A_{Ed}$ :

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

$\gamma_I$  is the importance factor. Four importance classes are defined, ranging from low risk ( $\gamma_I = 0,8$ ) till exceptional risk ( $\gamma_I = 1,6$ )

$A_{Ek}$  is the *reference seismic action*

In damage limitation state, 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 Part 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 the 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.

Regarding *faulting*, 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. For long structures, such as pipelines, 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.

Annex B of 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  $N_1(60)$ .

As far as the *seismic slope stability assessment* is concerned, EC8 allows the design engineer to select among the different mathematical models when abrupt irregularities in topography and soil stratigraphy are not present, and mechanical behaviour 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 (such as Newmark’s sliding block model), 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. CONCLUSIONS

Since earthquakes and the related geohazards are a major threat to pipelines, current paper deals with the issue of quantification of earthquake-related geohazards. The main conclusions are the following:

- a) Seismological and tectonic studies estimate respectively the levels of ground shaking and the offset of the potentially active faults. Nevertheless, both of them usually refer to the rock outcrop or to the seismic bedrock, no matter what the local site conditions are. That means that in cases where local site conditions are predominant, if seismic design is based only on the seismological and tectonic studies, the impact of local site conditions on the seismic wave loading and on the permanent ground deformations of the ground surface may be ignored. Additionally, geological surveys usually include the identification of the potential slope instabilities, but they rarely include a quantitative estimation of the permanent deformations, especially under seismic conditions.
- b) Seismic norms are based on seismic zonation maps and their simplistic soil / topography factors are insufficient to capture local site effects. Regarding permanent ground deformations caused by slope instabilities, the simplistic norm provisions are incapable to cover realistically all the issues.
- c) The complexity of the specific problems requires advanced modelling and realistic simulations on a case-by-case basis (provided that reliable data exist). That means that a *geotechnical earthquake engineering study* (for the realistic estimation of seismic wave loading and of permanent ground deformations on the ground surface) is a prerequisite for a reliable pipeline seismic design.

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