Seismic vulnerability of buried pipelines

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RESUMEN

Existen varios documentos e investigaciones que señalan los daños causados por los fuertes sismos del pasado siglo a las tuberías de agua potable, drenaje y gas. Precisamente los sismos acontecidos en México en septiembre de 1985 corresponden a esta clase de sismos que causaron fuertes daños a varias tuberías, tanto en el Distrito Federal como en el Puerto Industrial Lázaro Cárdenas. Daños semejantes han ocurrido en fechas más recientes, como es el caso de los sismos de Loma Prieta en 1989 (Distrito de Marina), Northridge, 1994, en California, E.U.A., y en Kobe, Japón, en 1995.

En este trabajo se hace ver la problemática del daño que los sismos producen sobre las tuberías enterradas, las consecuencias de su fallamiento y la metodología que hoy en día existe para su análisis al considerar los efectos de las deformaciones permanentes del suelo y él de la propagación de ondas sísmicas. Se presentan igualmente algunos procedimientos simplificados disponibles para tomar en cuenta este efecto en el diseño de tuberías continuas y segmentadas. Finalmente se dan algunas recomendaciones que se deberán tomar en cuenta en el diseño e instalación de tuberías ubicadas en zonas sísmicas.

PALABRAS CLAVE: Riesgo sísmico, tuberías, agua potable.

ABSTRACT

Earthquakes may cause severe damage to buried pipelines. One dramatic example occurred during the Michoacán earthquakes of September 1985. Many aqueducts and pipelines in Mexico City and in Lázaro Cárdenas failed as a result of those earthquakes. Similar damages were also observed more recently in the earthquakes in Loma Prieta (Marina District), in 1989; in Northridge, California in 1994; and in Kobe, Japan, in 1995.

This paper illustrates the kind of problems that take place after the failure of pipelines conducting water, gas or sewage. It also gives the principal causes of those failures, and the methods available today to take into consideration seismic effects for the proper design of buried pipelines. Some simplified procedures for preventing effects due to permanent ground deformation and seismic wave propagation on continuous and segmented pipelines are given, together with some recommendations to be followed when installing buried pipelines in seismic areas.

KEY WORDS: Earthquake damage, pipelines, water supply.

INTRODUCTION

There are several publications and reports from all over the world that discuss the severe damages caused by the failure of water and gas pipelines during or after the occurrence of high-intensity earthquakes. This kind of information has existed since the occurrence of the San Francisco earthquake, in 1906, where extensive damage was produced because of the failure of several water pipelines, which hampered fire fighting efforts. As it is well known, it was fire that destroyed that city located in a highly intensive seismic area. Since that year, there is much more information about the damage observed in many buried pipelines of big cities located in seismic areas. One of those cities is Mexico City, which, as we well know, suffered two large earthquakes in September 1985, and several of the main potable water systems failed as a consequence of the large soil displacements due to seismic wave propagation effects. As result of these failures, more than 4 million people, living in the eastern part of the city, did not have potable water for about three weeks.

Several other more recent earthquakes have struck buried pipelines in urban areas. The 1989 Loma Prieta earthquake caused severe damage in the Marina District due to soil liquefaction. During the 1994 Northridge earthquake, several pipelines and aqueducts were broken due to large permanent ground deformation; and during the 1995 Kobe earthquake, around 2000 repairs had to be done in the water distribution system due to significant ground shaking, ground distortion, and liquefaction in the artificial fills constructed near the bay (Eidinger and Avila, 1999).

This work presents some of the consequences produced when a pipeline system fails, the main seismic causes of this failure, and the methodologies or procedures currently available to take into consideration seismic effects for the design

and behavior of straight continuous and segmented buried pipelines.

Buried pipeline damage due to seismic effects

The damage produced by breakage or disconnection of pipelines is quite variable, and can be related to technical, economical and social aspects. The breakage of gas pipelines, for instance, besides representing a health hazard and fire risk, causes leakage and the repairs in the pipeline represent an important cost. In the case of sewer systems, their failure is linked to potential problems of soil and water contamination, particularly when the water level is close to the ground surface or the water main is close to the sewer pipelines that failed. Illnesses and epidemics may emerge due to this kind of contamination.

The most dramatic damage due to pipeline failure usually occurs when a pipeline carries drinking water. It means the interruption of water supply to several sectors and developments near the site where the failure occurred, and second, there will be no water available for putting out any fires that might arise due to electrical or gas leakage problems. On the other hand, social pressure might be very important if water supply is not promptly restored.

In order to specify the damage that buried pipelines may sustain as a consequence of an earthquake, it is worth noting that, in general, there are two types of pipelines: continuous (mainly steel and polyethylene pipes), and segmented (with bell and spigot-type joints, for instance). In continuous pipelines, the most frequent kinds of failures are due to large tensile strains at the welded joints, and local buckling due to large axial compression stresses at the pipe wall; they also might experiment beam buckling similar to Euler buckling of a slender column, and slip joint failure at joints that have a lower strength than that of the pipe material. Segmented pipelines, on the other hand, might have axial pull-out at the joints, crushing at the joints, and round flexural cracks in pipe segments away from the joints.

Seismic hazards to buried pipelines

Seismic hazards have been classified as being either permanent ground deformation hazards or wave propagation hazards (O'Rourke and Liu, 1999).

Permanent ground deformation (PGD) might be a) abrupt, as caused by a geological active fault or a large seismic settlement, or b) spatially distributed as produced by landsliding or lateral soil spreading due to liquefaction. In the case of spatially distributed permanent ground deformation, it can be sub-classified as longitudinal (parallel to the pipe-

line axis), like those occurred along Balboa Blvd. during the 1994 Northridge earthquake, or transverse (perpendicular to the pipeline axis), as during the 1971 San Fernando earthquake over the *Upper Van Norman Reservoir* (O'Rourke and Tawfik, 1983). Figure 1 shows a sketch representing this kind of failure and a model of a pipeline in which the soil has been substituted by springs. Figures 2 and 3 show some scenes of the damages due to permanent soil displacements, observed in the San Fernando Valley, Calif., after the Northridge earthquake in 1994.

The second type of soil displacement hazard is directly related to the propagation of seismic waves that originate in the epicenter and release energy that dissipates along the soil stratum crossed by these waves. Sometimes, however, dis-
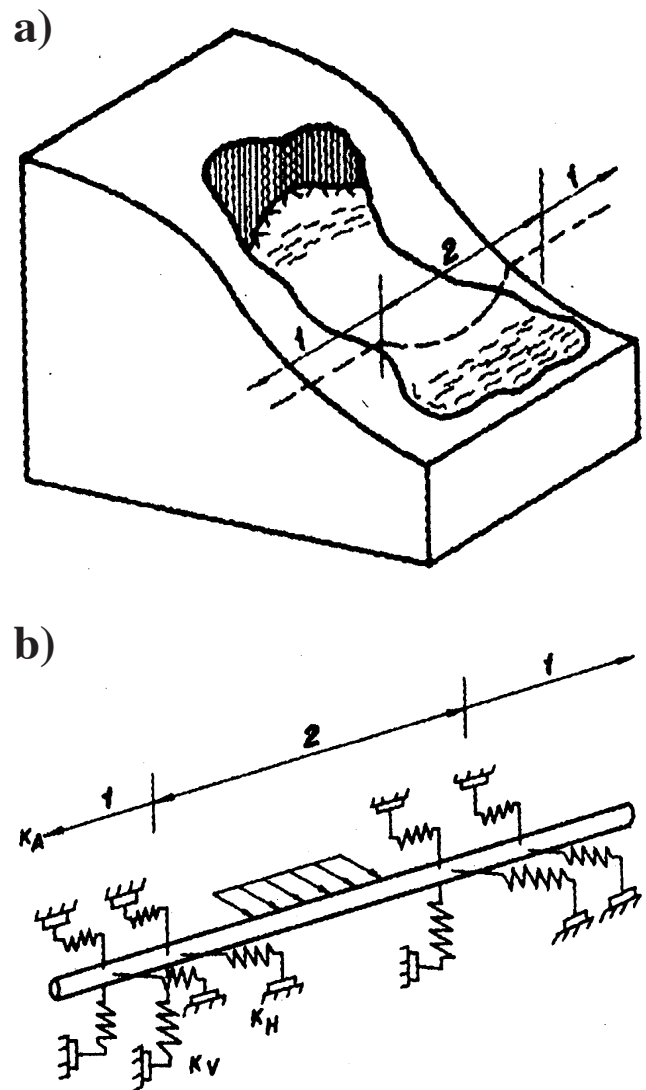
a)

b)

Fig. 1 (a) Pipeline displacement due to a slope stability failure, (b) Spring model representing forces of the pipeline in (a).

Fig. 2. Potable water leakage in the San Fernando valley after the Northridge earthquake, in January, 1994.

Fig. 5. Close view of a joint failure; Mexico City earthquake, 1985.

Fig. 3. Slab displacement due to permanent ground displacement. San Fernando valley, Calif. 1994 (Balboa Blvd).

Fig. 6. Failure of a steel pipeline; Mexico City earthquake, 1985.

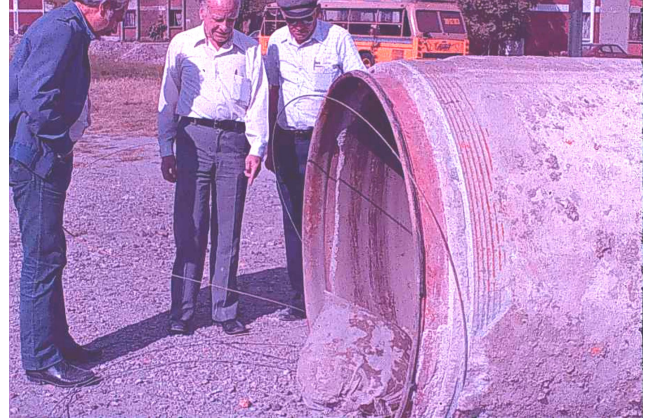Fig. 4. Joint failure of a concrete pipeline after Mexico City earthquake, 1985.

placements are amplified in such a way that the soil movements are larger at the ground surface than at the bedrock. A pipeline either tries to follow the soil movements when the seismic soil movements are relatively small, or it tries to *slip* along the surface contact between the surrounding soil and the pipeline when such movements are very large. These kinds of movements induce strains and stresses in the buried pipelines, and when those stresses and/or strains are greater than allowable, the failure of the pipeline might take place at any time. This is the kind of failure that occurs more often and it is the one that took place in the buried pipelines of Mexico City during the earthquakes of 1985. Figures 4 and 5 show damage observed in concrete pipelines of Mexico City during the 1985 earthquakes; Figure 6 corresponds to a damaged steel pipeline. The reparation of these pipelines took several days, since it was necessary to construct special joints and carry out difficult maneuverings in order to solve those problems.

Soil-pipeline interaction

The soil around a pipeline plays a very important role in relation to its seismic behavior; if it is cohesive soil, the softer it is, the greater differential settlements there will be due to consolidation or higher amplification effects; if it is granular material, the probability of liquefaction becomes higher the looser it is. Nevertheless, when we talk about soil-pipe interaction, it is supposed that the soil will not fail, but the soil displacements will produce friction-like forces at the soil-pipe interface.

An elasto-plastic model is often adopted for the force-deformation behavior at soil-pipeline interface (O'Rourke et al, 1995). This model is fully defined by two parameters: the maximum axial force per unit length at the soil pipe interface $\mathbf{f}_m$ and the relative displacement at which slippage between pipe and soil occurs. The slippage displacement is small and conservatively neglected here.

The maximum axial force per unit of length f_m depends on the type of soil surrounding the pipe and the method of pipe installation (i.e., the compaction control of the backfill). For cohesionless soil, f_m depends on the effective normal stress at the soil-pipe interface, the effective friction angle φ between the soil and the pipe material, and the pipe diameter ϕ_p. Considering that we have plane strain, and that the coefficient of lateral pressure k_o for compacted soil is approximately equal to unity, the effective normal stress σ'_v is simply equal to: $\sigma'_v = \gamma'_m H$, where γ'_m is the effective unit weight of the soil, and H is the depth to the pipe's centerline. Thus, for cohesionless backfill, the friction force per unit length becomes

$$\mathbf{f}_m = \sigma'_v \tan\varphi . \pi\phi_p . \tag{1}$$

For cohesive soil, f_m depends on the undrained shear strength S_u of the soil. For normally consolidated soils, Lambe and Whitman (1969) recommended the use, as adhesion, of the undrained shear strength of an equivalent normally consolidated soil. Therefore, for cohesive soil, $f_m = S_u . \pi\phi_p$. For the most general soil condition, when the soil surrounding the pipe has both friction and cohesive characteristics, f_m will be given by

$$f_m = (c + \sigma'_v \tan\varphi)\pi\phi_p , \tag{2}$$

where c = shear strength of the soil corresponding to zero effective vertical stress on the shear-strength curve.

Methods for seismic analysis of pipelines

Since the effect of the soil displacements on the behavior of pipelines depends on the seismic hazard, it is appropriate to describe separately the existing methods for PGD and those for wave propagation effects. Further more, since the behavior of continuos pipelines are different for segmented pipelines, such difference must be taken into consideration when using a seismic analysis for buried pipelines.

Seismic analysis for PGD

As mentioned earlier, there are different types of PGD (abrupt, spatially distributed, etc.). Because of space limitations, this article discusses only some of the current methods used for longitudinal PGD, considering both, continuous and segmented pipelines. For other types of PGDs, most of the existing methods of analysis are given by O'Rourke and Liu (1999).

Continuous pipelines subjected to PGD

Flores Berrones and O'Rourke (1992) use a block pattern of PGD (see Figure 7). The strain in an elastic pipe is given by

$$\varepsilon = \frac{\alpha L}{2L_{em}} \qquad for\ L < 4L_{em} \tag{2a}$$

$$\varepsilon = \frac{\alpha L}{\sqrt{LL_{em}}} \qquad for\ L > 4L_{em}, \tag{2b}$$

where $L_{em} = \alpha EA/f_m$ = length over which the constant slippage force f_m must act to induce a pipe strain ε equal to the ground strain α, and L is the length of the PGD zone. Characterizing the uniaxial stress-strain behavior by a Ramber-Osgood model, O'Rourke *et al*. (1995) arrived at the following expressions for computing the peak axial strain:

$$\varepsilon_{max} = \frac{\beta_p L}{2E}\left[1 + \left(\frac{n}{1+n}\right)\left(\frac{\beta_p L}{2\sigma_y}\right)^r\right], \tag{3}$$

where E = modulus of elasticity for the pipe material (=200,000Mpa for steel), n and r are Ramber-Osgood parameters; σ_y= effective yield stress, and β_p is the pipe burial parameter defined as

$$\beta_p = \frac{f_m}{\pi\phi_p t} = \frac{\tan\varphi\gamma_m H}{t} \quad for\ granular\ soils \tag{4}$$

$$\beta_p = \frac{\alpha_o Su}{t} \quad for\ cohesive\ soils, \tag{5}$$

where α_o is an empirical coefficient depending on S_u.

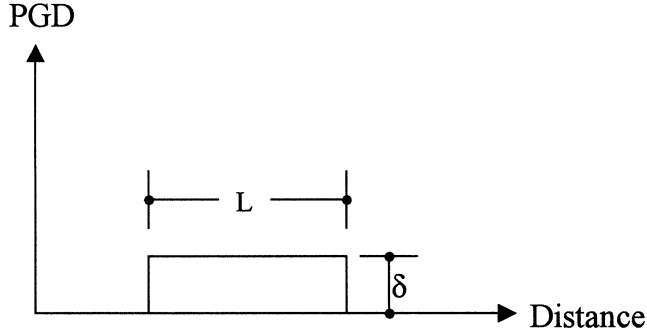Fig. 7. Block pattern in longitudinal permanent ground deformation.

c) Round flexural cracks in the pipe segment away from the joints

As demonstrated by O'Rourke and Liu (1999), if the longitudinal PGD has a block pattern (see Figure 7), one would expect that a few pipe joints near the head and toe of the zone would have to accommodate essentially all the abrupt differential ground movements. On the other hand, if the ground movement varies within the PGD zone (i.e., an idealized ridge pattern as the one shown in Figure 8), the rate of change along the segment pipeline leads to an equivalent ground strain. Therefore, it is expected that all joints within the PGD zone will experience relative axial displacement. For the particular case of longitudinal PGD in segmented pipelines, damages will typically occur at pipe joints, since the strength of the joints is generally less than the strength of the pipe itself. Whether the joints will fail or not, depends on the strength and deformation capacity of the joints, as well as on the characteristics of the PGD. If the PGD occurs along the whole length of the pipeline, like the one observed in the ridge pattern (Figure 8), the ground strain is

From laboratory tests on thin wall cylinders, Hall and Newmark (1977) suggested that compressional wrinkling in a pipe normally begins at a strain of one-third to one-fourth of the theoretical value $\varepsilon_{theory}=0.6$ t/R, where t = pipe wall thickness and R = pipe radius. Therefore the midpoint of the range suggested by Hall and Newmark will be

$$\varepsilon_w = 0.175 \ t/R. \tag{6}$$

Substituting this critical local buckling strain into equation (3), the critical length L_{cr} of PGD is obtained. This value can be used to calculate the critical ground movement δ_{cr} that might cause buckling; this displacement is obtained from

$$\delta_{cr} = \frac{\beta_p L_{cr}^2}{E} \left\{ 1 + \frac{2}{2+r} \frac{n}{1+r} \left(\frac{\beta_p L_{cr}}{\sigma_y} \right)^r \right\}. \tag{7}$$

Using the Ramber-Osgood model, O'Rourke *et al.* (1995) developed critical values for δ and L, which result in wrinkling of the pipe wall in compression. When these formulas were applied to the longitudinal PGD at Balboa Blvd during the 1994 Northridge earthquake, O'Rourke and Liu (1999) obtained excellent agreement with the experimental observations.

Although an initial compressive failure in steel pipes subjected to longitudinal PGD is more likely than an initial tensile rupture, and the critical failure strain in compression is less, it is always useful to determine the condition for initial tensile rupture by substituting a tensile rupture strain into equation (3). Using this equation, it is possible to obtain the critical length of the PGD zone, which combined with equation (7) gives the critical ground displacement.

Segmented pipelines subjected to PGD

There are three failure modes for segmented pipelines:

a) Axial pullout at joints
b) Crushing of the bell and spigot joints

$$\varepsilon_g = 2\delta/L . \tag{8}$$

If the length of each segment is L_o, the average relative displacement at the joints is

$$U_{av} = 2\delta L_o/L . \tag{9}$$

If this value is larger than half the total joint depth, the pipeline is expected to fail, unless there are special joints having a large contraction/expansion capacity and/or anti-pullout restraints. If the joint displacements can be larger than tolerable, it is better to substitute the segmented pipeline by a continuous one, or place special joints at critical points.

In case of abrupt longitudinal deformations, like those observed in the block pattern (Figure 7), the analysis of the segmented pipeline can be made following the procedure recommended by O'Rourke *et al.* (1995) for continuous pipelines and synthesized here in the previous headline.
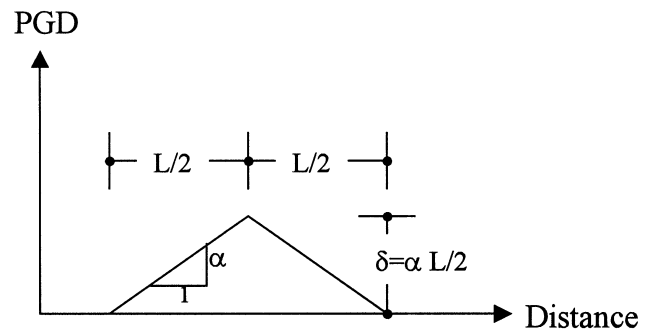

Fig. 8. Ridge pattern (Flores Berrones and O´Rourke, 1992).

Response of continuous pipelines to wave propagation

The wave propagation effect on soils causes axial and bending stresses in the pipelines due to the interaction effect between the soil and the external surface of the pipeline. While most of the damage due to this effect occurs a the joints in segmented pipelines, in the case of continuous pipelines, the failure mechanism is due to buckling.

It is usually assumed that pipeline displacements are the same as those of the ground; however, for very large deformations, some slippage occurs at the pipe-soil interface, resulting in a pipe strain somewhat lesser than the ground strain.

There are several methods for computing pipe strain due to the wave propagation effect; summarized here are only three of these methods:

1) Newmark (1967) suggested the simplest one, and it consists of assuming that pipeline strain is equal to the ground strain parallel to the pipe axis. For a pipeline subjected to S wave propagation in a vertical plane, having an angle of incidence γ_s with respect to the vertical, the ground strain is

$$\varepsilon_g = \frac{V_m \cos\gamma_s}{C} = \frac{V_m}{C_s}\sin\gamma_s \cos\gamma_s , \qquad (10)$$

where $C = C_s/\sin\gamma_s$ = apparent wave velocity, and $V_m \cos\gamma_s$ = ground velocity parallel to the pipe axis.

If the pipeline is subjected to Rayleigh waves, the ground strain parallel to the pipe axis is

$$\varepsilon_g = V_m/C_{ph} , \qquad (11)$$

where C_{ph} is the Rayleigh phase velocity.

2) Shinozuka and Koike (1979) use the following equilibrium equation for a pipe segment

$$\rho\frac{\partial^2 u}{\partial t^2} - E\frac{\partial^2 u}{\partial z^2} = \frac{\tau_s}{t} , \qquad (12)$$

where u is the pipeline displacement in the z direction (direction of wave propagation), ρ is the mass density of pipe material, τ_s is the shear force at the pipe soil interface per unit of length, and t is the pipe wall thickness. To obtain the pipe strain, they recommend the following conversion factor for the ground strain

$$\beta_o = \frac{1}{1+\left(\frac{2\pi}{\lambda}\right)^2\frac{AE}{k_g}} , \qquad (13)$$

where k_g is the linear soil stiffness per unit of length and A is the cross section area of pipe. Therefore, the pipe strain is

$$\varepsilon_p = \beta_o\varepsilon_g . \qquad (14)$$

Considering that the shear strain at the pipe-soil interface γ_o is

$$\gamma_o = \frac{2\pi}{\lambda}\frac{Et}{G}\varepsilon_g\beta_o \qquad (15)$$

and the critical shear strain is

$$\gamma_{cr} = \frac{tu}{\pi DG} \quad (\approx 1x10^{-3}) , \qquad (16)$$

where G is the shear modulus of soil (Shinozuka and Koike, 1979), if $\gamma_o > \gamma_{cr}$, slippage will occur and the conversion factor is

$$\beta_c = \frac{\gamma_{cr}}{\gamma_o}q\beta_o , \qquad (17)$$

where q is a slippage factor that varies between 1 and $\pi/2$.

3) O'Rourke and El Hmadi (1988) present another method which takes into account the slippage at the pipe-soil interface. In this case, the pipe strain is

$$\varepsilon_p = tuL_s/AE , \qquad (18)$$

where $L_s = \lambda/4$.

O'Rourke and Liu (1999) applied these three methods to a pipeline subjected to three different Rayleigh wave frequencies and made a comparison of the three approaches. The pipe strains resulting from Shinozuka-Koike's method and from O'Rourke and El Hmadi were practically the same, while those obtained using Newmark's method were much larger, due to the fact that this method does not consider any slippage at the pipe-soil interface.

Response of segmented pipeline to wave propagation

As already mentioned, segmented pipelines are subjected to three failure modes: a) pull-out at joints due to tension strains, b) crushing of bell-spigot joints due to compression, and c) circumferential cracks due to bending.

Under tension ground strains, the maximum joint displacement Δu is given by

$$\Delta u = \varepsilon_{max}.L_o , \qquad (19)$$

where ε_{max} is the maximum ground strain parallel to the pipe axis, and L_o is the pipe segment length. The maximum relative rotation at pipe joints is

$$\Delta\theta = k_g L_o , \qquad (20)$$

where k_g is the maximum ground curvature $= A_g/C^2$; A_g is the maximum ground acceleration, and C is the propagation velocity of the seismic wave.

As a function of the pipe stiffness, Wang (1979) deduced the maximum relative joint displacements (joint openings) and the average of the maximum pipe strains for different types of pipelines (reinforced concrete pipes, cast iron steel and ductile iron pipes, etc.). He observed that the maximum relative joint displacements decrease when the joint stiffness increases, and that the maximum average strains in the joints increase when the joint stiffness increases. He also observed that joint displacements increase for large ground strains, but these displacements vary somehow from segment to segment; the average joint displacement, however, is approximately equal to the product of the ground strain times the pipe segment length.

When a pipeline is subjected to compressive ground strains, a kind of crushing occurs at its joints. Based upon a series of laboratory tests on reinforced concrete cylinder pipelines with rubber gasket joints, Bouabid (1995) obtained the graph shown in Figure 9, in which the joint displacements are plotted for different compressive forces. This experimental graph shows that there is a force after which the joint "locks up", and the joint compressive displacement Δu_{ult} occurs; according to O'Rourke and Liu (1999), Δu_{ult} typically ranges from 0.32-0.95 cm, with corresponding loads of 16 to 20 kN.
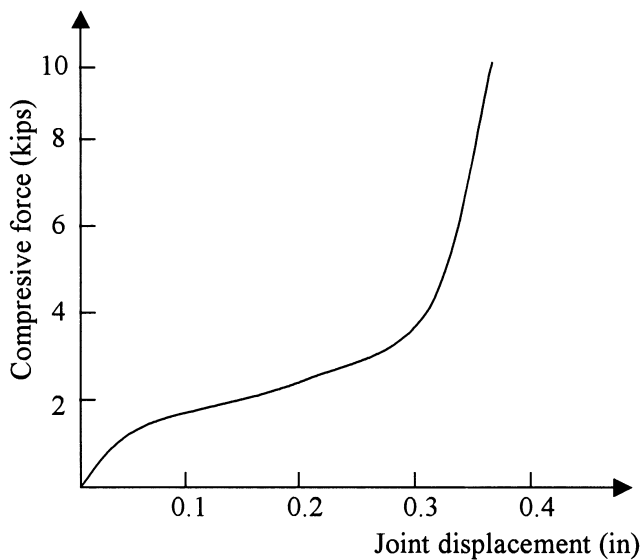
When n joints are fully closed in sequence, and the ground strain is uniform, according to O'Rourke and Liu (1999), the pipe segment compressive strain is

$$\varepsilon_p = \varepsilon_g - \frac{n}{1+n}\frac{\Delta u_{ult}}{L_o} . \qquad (21)$$

The upper bound of this value corresponds to n = 1, and the lower bound to n = ∞.

Several numerical models try to reproduce the conditions of pipelines subjected to earthquakes. Figure 10 represents a sector of a soil - pipe – joint system that uses dashpots, springs and rigid elements; this model has been extensively used in Japan and is known as "Multiple Response Particle Method" (Kubota, 1981). A more flexible model for representing segmented pipelines is the one based in the Finite Element Method (FEM). In Figure 11, three types of finite elements are used: a two-dimensional isoparametric quadrilateral element (for the soil), a two-noded beam element for pipes, and a spring element for boundaries. The soil surrounding the pipes is represented by 4-noded quadrilateral finite elements, and the soil-pipe interaction takes place by an elastic connection at the finite element nodes between the beam finite elements (pipes) and the plane elements (soil). In an isotropic case, the soil is characterized by its modulus of elasticity E_s (Young's modulus) and Poisson's ratio ν. If the soil is stratified and its mechanical properties are different in two mutually perpendicular directions, the characteristics of the soil include the modulus of elasticity and Poisson's ratio in two isotropic planes. Using this FEM model, it is possible to determine the axial and bending stresses of the pipeline, the relative soil and pipeline displacements, as well as the expansion – contraction of each joint (Flores-Berrones and Vassilev, 1999).

Some recommendations for seismic damage mitigation

In case of pipelines buried in soil strata that are susceptible to liquefaction, it is recommended to do either a probabilistic or a deterministic analysis in order to define that susceptibility, and if necessary, to do whatever is necessary to avoid such phenomenon to take place, or at least to reduce the potential for liquefaction. For instance, it might be necessary to apply a dynamic or vibratory compaction, or any other procedure to improve the *in situ* density of the granular material; one could also replace liquefaction soils in the vicinity of the pipe with non-luiquefiable materials, such as coarse gravel. Regarding soil liquefaction analysis, there are many publications available that present different empirical and analytical procedures together with field and laboratory tests, to predict the occurrence of the liquefaction phenomenon (see for instance Ostadan, 1991). Some other alternatives to solve the problem of soil liquefaction are: 1) to relocate the pipeline in better soil conditions; 2) shallow burial
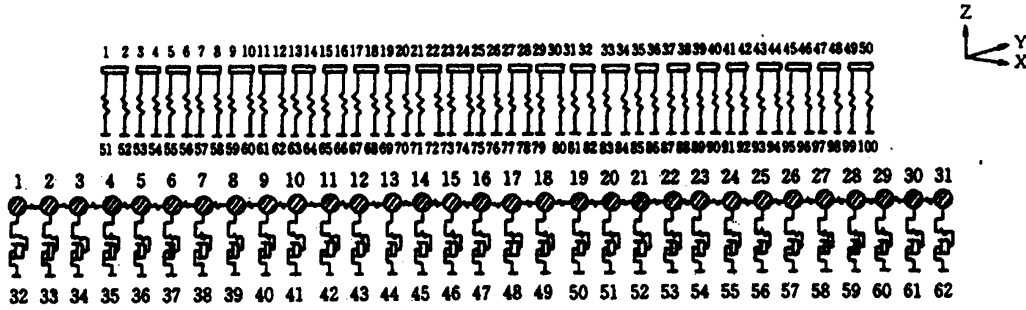
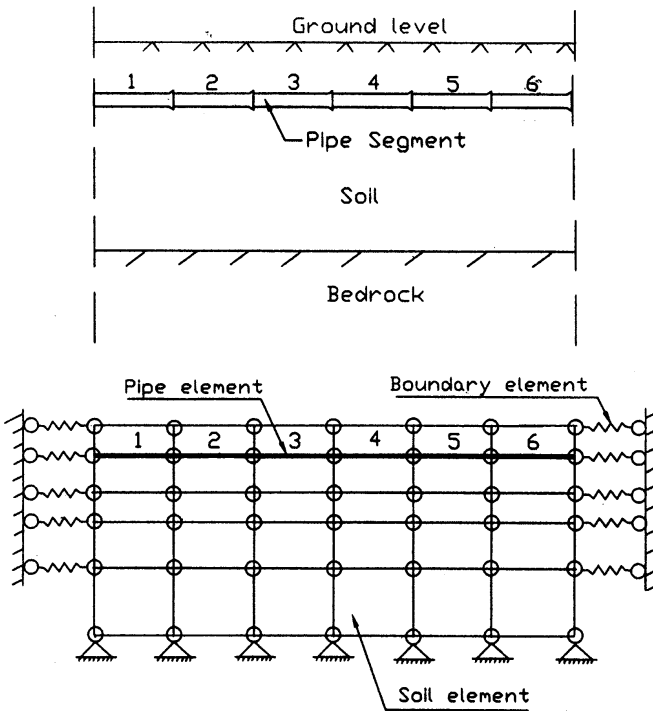Fig. 9. Force displacement relationship for reinforced concrete pipe joints (Bouabid, 1995).

Fig. 10 Multi-Particle Response Model (MPRM).

Fig. 11. Finite element discretization of a pipeline-soil system.

reported practically without any damage in Mexico City during the 1985 Michoacán earthquake, the authorities have been substituting concrete pipelines with polyethylene ones.

For segmented pipelines, the use of flexible joints can help absorb large amounts of pipe displacements. Isenberg and Richardson (1989) present several joint types with different expected deformation capacity that can be use for pipelines subjected to PGD hazard. Figures 12 and 13 represent two types of especial joints than can be used to absorb estimated displacements.
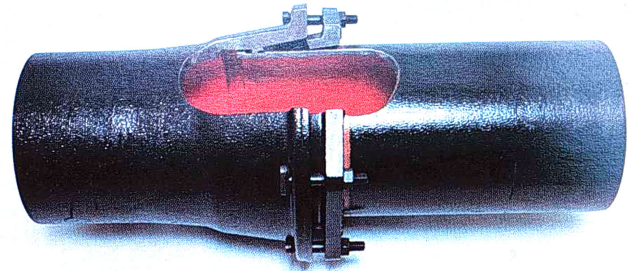

Fig. 12. Special contraction – expansion joint for ductile iron pipe.

in order to minimize soil stresses and to make simple repairs whenever necessary.

For continuous pipelines, the use of stronger materials and a thicker wall will improve seismic performance results. Another alternative to mitigate the strains in continuous pipelines is to reduce the applied load by reducing the burial depth H, and use a low density back fill material in order to reduce the burial parameter defined by equations (4) and (5).

The use of pipelines with more flexible materials can also help to avoid damage due to seismic effects. For instance, it is preferable to use steel and high density polyethylene pipes than those made of more brittle materials, such as asbestos cement, cast iron, concrete, and polyvinyl chloride (PVC). Based on the excellent behavior of polyethylene pipelines
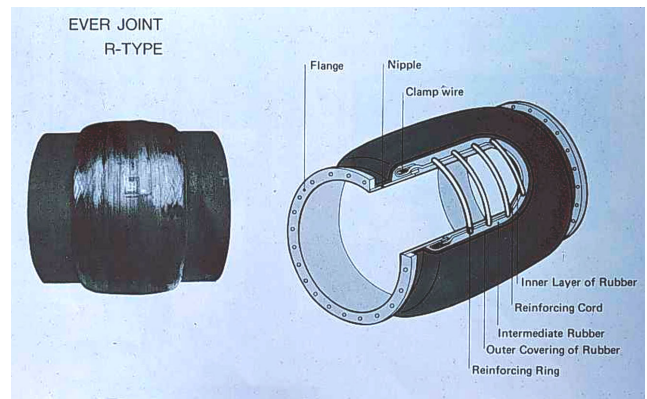

Fig. 13. Flexible joint for large pipeline deformations.

CONCLUSIONS

The vulnerability of buried pipelines in seismic zones can be very serious and it is necessary to take preventive measures that eliminate, or at least decrease, that vulnerability.

The breaking of buried pipelines can have several causes. They are well identified and have been studied in such a way that they can be avoided or taken into consideration when defining which kind of pipeline should be used in seismic areas.

There are important research works and numerical models through which it is possible to estimate the maximum strains, stresses and displacements to which a buried pipeline might be subjected due to seismic effects in the soil. Seismic effects should be taken into consideration when designing and constructing pipelines located in seismic areas. Moreover, aqueducts and pipelines that are the main source of water distribution for important cities in seismic zones should be investigated and analyzed in terms of vulnerability to earthquakes.

Institutions and authorities responsible for the design, construction and operation of buried pipelines located in seismic zones should demand that the seismic effects are correctly taken into consideration in order to assure the good behavior of such pipelines during their working life.

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