Dynamic Response of Buried Gas Pipelines Due to Earthquake Induced Landslides by Nonlinear Numerical Modeling

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LISBOA 2012

SUMMARY:
Landslides, liquefaction, wave propagation and faulting are four major earthquake induced hazards to lifelines. Although, there are a few guidelines for pipeline design against liquefaction; a reliable design code or guidelines considering the effects of all mentioned phenomena are not available and more studies are required. Therefore, many investigations and studies on special subjects like faulting and land sliding are in progress. In this study, potential slides in Tehran and their geometrical, geotechnical and structural specifications are selected. A buried steel gas pipes with 24 inch diameter is considered as structure and its performance against a local slope is numerically analyzed by ABAQUS program. The slope has 30 degree inclination and is composed of sedimentary cemented soils. A 0.5 g amplitude sinusoidal excitation, equivalent to a 8.5 moment magnitude earthquake, was imposed to models. The pipe having three positions in the slope, is taken perpendicular to sliding. Furthermore, a comprehensive nonlinear analysis was taken on boundary conditions of slope. By considering the variation of these parameters, 7 models were numerically analyzed and the obtained numerical results clarify the effect of various parameters. Then, some recommendation and probable modifications are suggested to rehabilitate the buried pipe performance.

Keywords: Buried Pipeline, Landslide, Numerical modelling, Earthquake

1. INTRODUCTION

Vulnerability of lifelines to dynamic excitation is proved in many earthquake events worldwide. However the causes of structural or geotechnical failures are matters of controversy and therefore comprehensive research to reveal the fact have been performed or are now in progress. Lifelines, categorized as energy, water treatment, transportation and information systems are greatly affected by three major causes including ground conditions, seismic scale and intensity, and lifeline features. This paper focuses on gas pipeline systems as a mean to convey energy from natural sources to any consumer. Tehran mega city, the capital of Iran, is the place where the research is concentrated and structural specifications are selected due to API (grade X42) gas pipeline standard. Ground condition which can be divided to two major groups of hard and soft, directly and indirectly affects the pipelines. For example while wave propagation and faulting have direct impact on pipe behaviour; liquefaction and sliding belong to the group having indirect influences. This research reviews the pipeline behaviour under earthquake induced slides and the numerical approach was selected for modelling.

The modes of pipeline deformations are greatly dependant on loading direction, as a case in point, the ground deformation can be parallel or normal to pipe axis. An oblique mode can also be considered. In this case, there are variety combinations of deformation modes. For example, a pipeline can cross the slope in 30, 45, 60 and 75 degrees. R. Brusshi (1995) modelled the slow soil movement for intersection angle of 10, 40 and 70 degrees and concluded that the induced axial force of pipe increases with decreasing pipe-slope angle, however the bending moment has direct relationship with that angle. This paper considers the 90 degree intersection condition to account the pipe deformations.

Permanent Ground Deformation (PGD) perpendicular to pipe axis can be considered spatially distributed or localized abrupt. While the first type is attributed to lateral spreading or some type of slides, the second is allotted to faulting or slope instability. These phenomena are analytically, numerically and physically analyzed. For example Suzuki et al. (1988) and Kobayashi et al. (1989) proposed two spatially distributed deformation pattern shown in Fig 1.1.

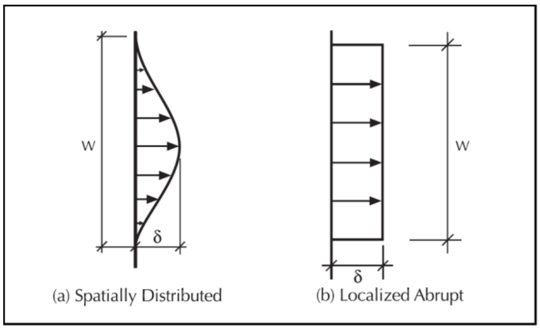


Figure 1.1. Two ground deformation type perpendicular to pipe axis (after M.J.O'Rourke & X.Liu 1999)

Besides the soil movement, boundary conditions are also a major subject in numerical modelling. Winkler spring, a routine way of surrounding soil modelling, has been widely used in previous studies. To take into account the pipe-soil interaction, M.J. O'Rourke et al (1995) used this method (elasto-plastic spring) along with Ramber-Osgood model for steel pipe. By considering some simplifying assumptions, T. O'Rourke (1988) and Suzuki et al. (1988), Liu & M. O'Rourke (1997) and M. Maugeri (2004) presented several analytic formulas to predict PGD with elastic-plastic Winkler soil model. In these particular approaches, the influence of shear stress between two adjacent springs was not considered. As a result, these assumptions could not actually calculate the induced dynamic forces caused by seismic slope failure or soil settlement. To compensate the shortcomings of this approach, finite element modelling which considers the slope as a continuum media, has been increasingly applied for soil-structure interaction modelling. This paper uses FE analysis to evaluate the pipe seismic performance buried in a potential slope.

1.1. Failure Modes of Pipelines

Although the three major modes of failure in straight pipelines are tension rupture, local buckling duo to compression and general buckling, more than 80 percent of pipeline failures occur because of joint ruptures. This paper focuses on straight pipe behaviour in landslides and therefore the joint ruptures are not modelled.

While Newmark & Hall (1975) considered tensile strain of 4% as a limit of pipe failure, ASCE regulation suggests 2 to 5 % strain as an upper and lower case. Pipe wall deformation and cracks, the representatives of local buckling, occurs in 1.3 to 1.4 of 0.6 t/R (t is pipe thickness and R is pipe radius). Although pipe wall deformation is considered as a buckling failure mode, beam buckling similar to column buckling is another cause of pipeline failures. In conditions which the burial depth of pipe is less than 1 meter and loose material let the pipe to deform, the continuous pipe tend to protrude the overburden material and general buckling will occur.

2. NUMERICAL MODEL CHARACTERISTICS

Use of finite element to model the landslides under both static and dynamic failure conditions have greatly increased during last decade. The finite element program ABAQUS v6.10 has been used to model the slope. In order to gain the realistic results which can be a base for predicting PGD, the model specifications should be selected in a way close to natural conditions. However in any numerical modelling, some divergences between model and prototype are inevitable, the loading condition, surrounding soil and pipe characteristics and boundary condition are selected as close as an actual continuous pipeline projecting a slope. These condition and the modelling methods for each one are described in following sections.

2.1. Numerical Analysis

Finite element analysis for every structure comprised with two base steps. At the first stage the gravity forces from different materials density is calculated and the model will be in equilibrium state. As the study is focused on the dynamic instability of the slope, it should be stable in static condition. By imposing the dynamic excitation, second step of the analysis will be started. In this step the implicit algorithm is selected to solve the dynamic equilibrium equations.

2.2. Loading Characteristics

As the paper focuses on Tehran area, we have used the seismic parameters regarding to National Iranian Seismic Regulation No. 2800 and seismic accredited reports. Although this standard proposes 0.35g pick acceleration for this area, some seismic reports suggests the designers to consider higher amplitude for worse conditions. In this regard, a sinusoidal acceleration with 0.5g amplitude and 1.2 Hz frequency excitation has been induced to model base for 15 seconds.

2.3. Surrounding Soil and Pipe Characteristics

Many studies have been performed to determine the Tehran cemented soil characteristics. Haeri & Rastgu (2008) used large scale direct shear and triaxial test results on disturbed and undisturbed soil sample from North East of Tehran. They proposed sample values for soil parameters.
Ghanbari et al. (2009) considered more than 100 geotechnical results in Tehran South alluvium and suggested several relationships for elastic module calculation. Khanlari & Alipour (2009) performed several pressure-meter and in situ shear test on Tehran north cemented soil and suggested the variation ranges of soil cohesion, internal friction angle and elastic module values. Although, all above studies considered actual soil conditions and used natural soil samples for tests, many researches were performed on artificial cemented sand, modelling Tehran soil conditions with similar gradations and cement content. As an illustration, Haeri & Asghari (2004) used lime to make artificial cemented soil with 45% gravel, 49% sand and 6% fine materials and performed triaxial compression tests for calculating shear strength parameters for different cement content.
Regarding to above mentioned studies; we have selected two layer soil slopes to model Tehran landslides. Table 2.1 summarized the selected parameters for soil conditions.

Table 2.1. Tehran soil parameters used for analysis

Soil layer type	Layer thickness (m)	Cohesion (kPa)	Friction angle (deg)	E_{max} (MPa)
1	20	80	30	150
2	20	100	35	367

Mohr-Coulomb behaviour model was selected to account the stresses and strains in soils. Since the model is excited by dynamic loading, the nonlinear behaviour of soil has great influence on slope instability. Consequently, USDFLD ABAQUS subroutine written by FORTRAN compiler was used to consider the soil decreasing strength with increasing dynamic strains. K.H. Stokoe (2010) and T.H. Tika (2010) suggested some correlations to determine G/Go reduction curve, used to model the soil nonlinearities. Table 2.2 summarizes the mentioned values for soil shear strength parameters.

Table 2.2. G/G_0 value for different strains

γ value	**G/G_0**	**γ value**	**G/G_0**	**γ value**	**G/G_0**	**γ value**	**G/G_0**	**γ value**	**G/G_0**
0	1	6.00E-06	0.961	3.00E-05	0.802	1.00E-04	0.454	6.00E-04	0.193
1.00E-06	0.996	1.00E-05	0.915	3.70E-05	0.697	2.00E-04	0.327	1.00E-03	0.100
2.00E-06	0.989	2.00E-05	0.869	8.00E-05	0.521	3.50E-04	0.228	1.00E-02	0.050

Material damping, as another important parameter attenuating the internal energy generated from seismic loading is introduced to model for tow basic frequencies. The Rayleigh damping coefficients (□ and □) are calculated for dynamic loading frequency (1.2 Hz) and natural frequency of the slope. This natural frequency was calculated through a frequency linear perturbation step (for detailed information refer to ABAQUS manual documents). For this analysis the damping ratio of 5 % (from geotechnical reports of Tehran soil) was introduced to the program.
Regarding to Iranian National Gas Company, polyethylene and steel pipe can be used, however due to its more usage; the steel pipe is selected for the analysis. The pipe grade was API-5L-X42 steel with 200 GPa elastic modulus and had poison ratio of 0.3.

2.4. Boundary Conditions and Model Geometry

From the studies performed on the potential slopes, a typical slope dimension was selected for the analysis. Figure 2.1 shows the slope characteristics. The steel pipe had 24” outer diameter and 0.312” wall thickness.

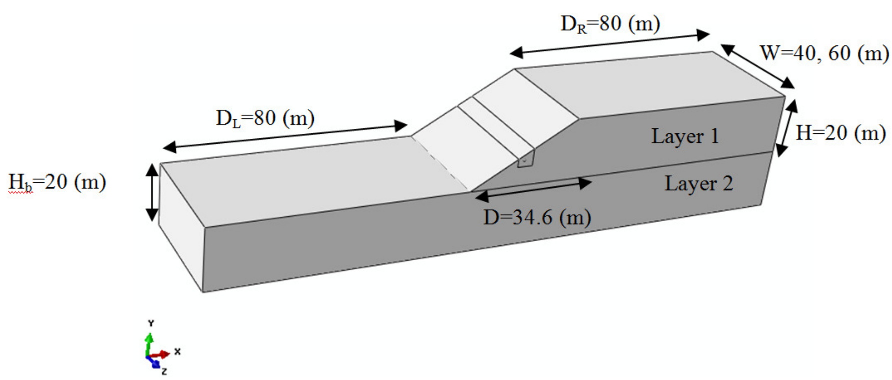


Figure 2.1. Typical slope dimension for the dynamic analysis

The 3 dimensional slope geometry has 5 boundary planes; each has its own interaction with adjacent medium. These specifications are summarized in Table 2.3.

Table 2.3. Boundary condition for slope planes

Plane name	**Ux**	**Uy**	**Uz**
X=0, $X=D_r+D+D_l$	Not constrained	Not constrained	Not constrained
Y=0	Not constrained	constrained	constrained
Z=0, Z=w	Not constrained - constrained	Not constrained	constrained

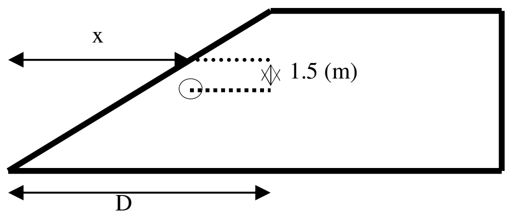
To avoid the box effect in boundary planes with regard to propagating wave, viscous absorbent boundary element, using dashpots, proposed by Lysmer & Kuhlemeyer (1969) has been used. In addition to overcome the redundant permanent displacement at low frequencies, normal and tangential springs developed by Kellezi (2000) have been applied to unconstrained planes. However some shortcomings together with these boundary conditions at constrained planes are inevitable, in order to make the sliding style more similar to natural condition, that method was used.
Soil and pipe discretization has been performed by 8-node linear brick elements; however the soil elements have reduced integration point and hourglass control property. According to Lysmer studies the elements dimensions were less than 1/8 to 1/10 shear wave length.

2.5. Soil-Pipe Interaction

Pipe-soil tangential contact can be defined in three ways, frictional, frictionless and full contacts. This study used penalty frictional contact (Coulomb frictional formulation) with 0.493 friction coefficient. On the contrary, normal behavior was defined by hard contact with allowing separation of surfaces option. Since the pipe outer surface was harder than the soil, in contact area, this surface was introduced master. As a result, penetration of slave surface (soil) in master surface (pipe) was controlled to be minimum.

2.6. Modelling Characteristics

The numerical modeling was divided to two major approaches. The first one focuses on the relative geometry of pipe in slope. Figure 2.2 describes this situation with parameter "A", defining the relative burial place of pipe in the slope. For the analysis, three quantity of 0.2, 0.5 and 0.8 was selected. Dynamic analysis was performed for each model and the deformation of slope and pipe was compared in each case.



$$A = {}^{x}/_{D} \rightarrow \begin{cases} 0 < A < \frac{1}{3} \\ \frac{1}{3} < A < \frac{2}{3} \\ \frac{2}{3} < A < 1 \end{cases}$$

Figure 2.2. Description of parameter "A"(Not to Scale)

The second approach focuses on the boundary conditions in Z=0 and Z=w planes. To account for the influence of boundary conditions, two constraint types (Table 3) were introduced to the program and the results of dynamic analysis were compared. Also, in the last analysis the width of the model was expanded to 60 m and the sliding pattern of the slope is studied.

3. NUMERICAL ANALYSIS RESULTS

According to the aforementioned details of the geometry and boundary conditions, 7 different models were analyzed by ABAQUS program. The models were divided into 2 basic types. By introducing "A" parameter (see Fig. 2.2), the first type determines the influence of relative geometry of the pipe in the slope and compares the plastic strains and displacement of the pipe. Figures 3.1 and 3.2 show the plot contours of the plastic strains and resultant displacement of the typical slope without a pipe in z=w/2 plane; respectively. The sliding plane of slope can be obviously seen from plastic strain pattern. Although the strains in the shallow depth of the slope were limited to 1.14×10^{-2}, in deeper depths plastic stains increase to 1.37×10^{-1}.

Figures 3.3(a) and (b) describe the horizontal and vertical displacement vector for the pipe with A=0.2, 0.5 & 0.8. These three conditions will be named A1, A2 and A3 hereafter. Maximum horizontal and vertical displacements which reflect the displacement of three part of the slope for A1, A2 and A3 types were summarized in Table 4; respectively. In lower part (A=0.2), maximum horizontal displacement occurs in $0.15<Z<0.35$, meaning that mid-section of the slope ($z=w/3$ to $z=2w/3$) had lower displacement than the adjacent parts, see Figure 3.3 (b). This phenomenon which shows that the middle part of the slope has moved toward the inner part, proved the rotational movement of the slope material around the center of the sliding surface.

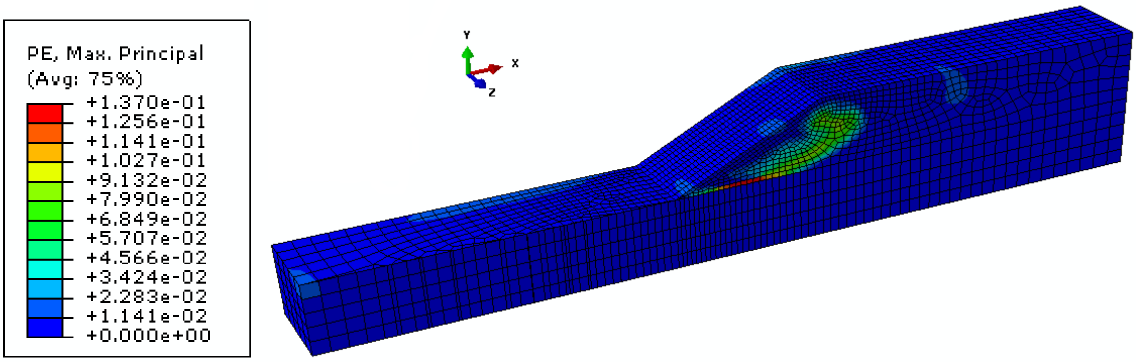


Figure 3.1. 3D plot contours of plastic strains (value) in the slope

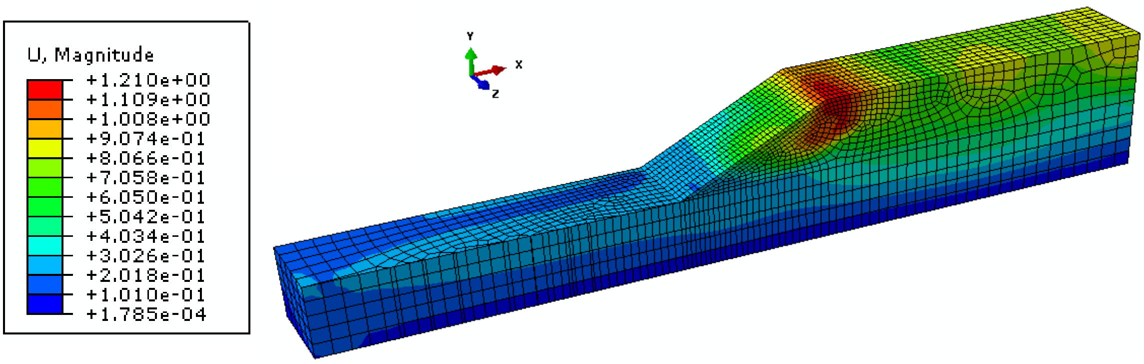


Figure 3.2. 3D plot contours of resultant displacement (m) in the slope

Table 3.1. Maximum horizontal and vertical displacement of the pipe with variation of parameter “A”

A	U_{1max}(cm)	Z for $U_{1\ max}$	U_{2max}(cm)
0.2	4.9	$0.15<Z<0.35$	26.6
0.5	36.9	$0.35<Z<0.5$	35.5
0.8	64.6		63.5

The calculated resultant pipe displacements show that A1 type has the lowest deformations and consequently it is suggested that for a pipe crossing a slope with rotational sliding pattern, the passage route should be as near as the slope toe.
The second analysis type focuses on boundary conditions of the slope in z=0, w planes. Three analysis groups were designed and analyzed by ABAQUS. Table 3.2 summarizes the boundary conditions for these three models.
In models 1 and 2, two types of B.C were considered with w=40 m (see Table 3.1). In order to show the

effect of boundary condition type and to be more close to natural circumstances, a w=60 m model was analyzed. Similar to model 1, this model had constrained boundaries in x direction for z=0, 60 planes. The horizontal displacement of z=w/2 plane for these three analysis case were compared in Table 3.3.

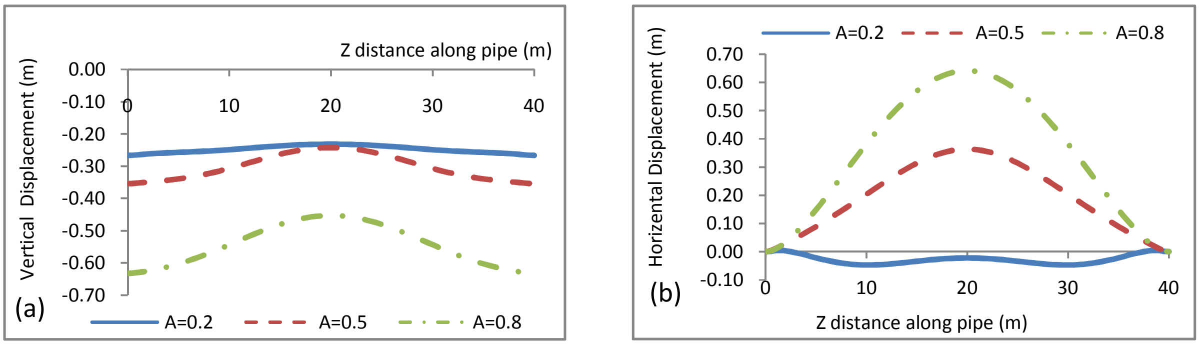


Figure 3.3. (a) Vertical and (b) Horizontal displacement along pipe length (m) for Parameter “A”

Table 3.2. Boundary Condition Types in z=0,w Planes

Model No.	**W**	**B.C**
1	40	constrained
2	40	unconstrained
3	60	constrained

Table 3.3. Maximum Calculated Horizontal Displacement for Mid-Section of the Slope

Model No.	**Z (m)**	**$U1_{max}$(cm)**
1	40	65
2	40	21
3	60	56
3	50-equevalent to z=40 in other models	20 (For Section Z=50 m)

Models 1 and 2, considered as two extreme boundary conditions of the planes, imposed minimum and maximum displacement in mid section of the slope. In model 3, which z=10, 50 is equivalent to z=0,40 in two other models, it is expected that the displacement be between the measures of model 1 and 2. This expectation was right from the numerical analysis results presented in Figures 3.4 to 3.7.
Nevertheless, it is more accurate to extend the width of the model beyond 60 m to gain more logical values, but the model discritization and F.E. calculation need more effort and time for the analysis. So it is inferred that the model 3 approximates the displacement closer to a natural slope.
The authors suggest that the geotechnical situation should oblige the designer choose the right boundary condition and model dimensions.
Figure 3.8 compares the maximum horizontal and vertical displacements and total strains (logarithmic ABAQUS strain) in longitudinal direction of the pipe. Since the strain values were symmetrically distributed along pipe length, the presented values are for mid-length of the pipe. However, maximum horizontal pipe displacement for all three models were calculated for mid-length of the pipe, the obvious similarities shown for displacement distribution pattern of models 1 and 3 proved that free boundary conditions assumed for model 2 cannot correctly predict the pipe deformations. Furthermore, when the boundary conditions are left to be free, the whole model displacement follows a rigid mass pattern and consequently calculated deformations values are less than the other conditions.

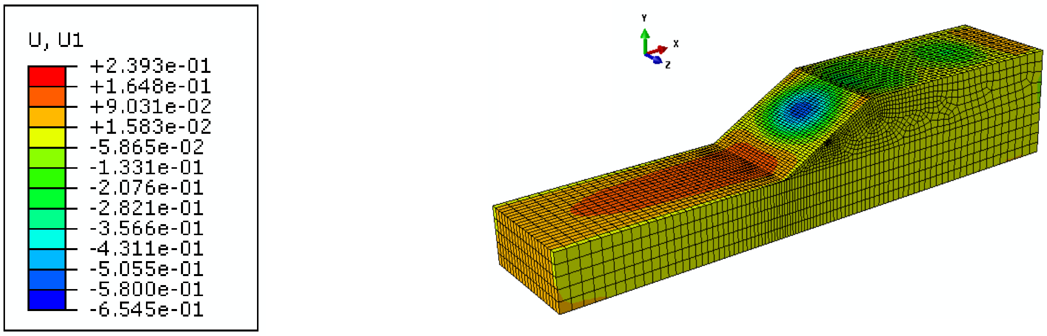


Figure 3.4. Horizontal Displacement (m) of the Slope for Model 1

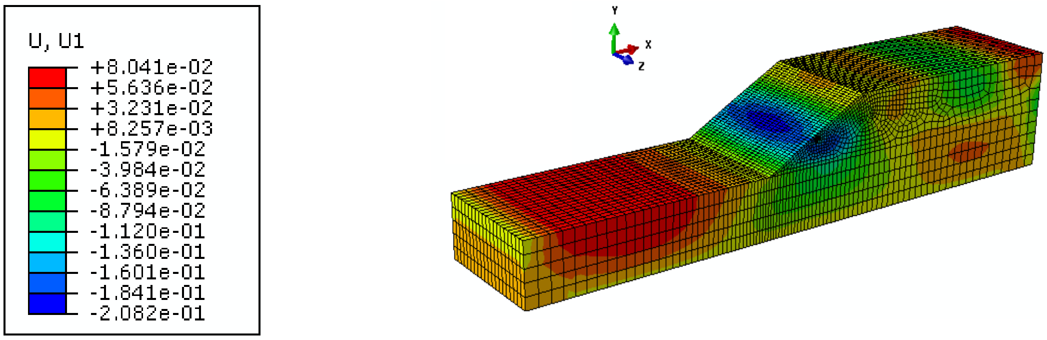


Figure 3.5. Horizontal displacement (m) of the slope for model 2

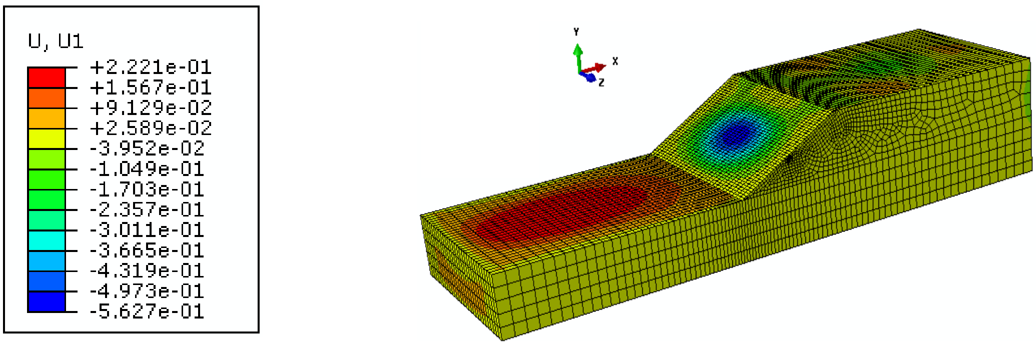


Figure 3.6. Horizontal displacement (m) of the slope for model 3

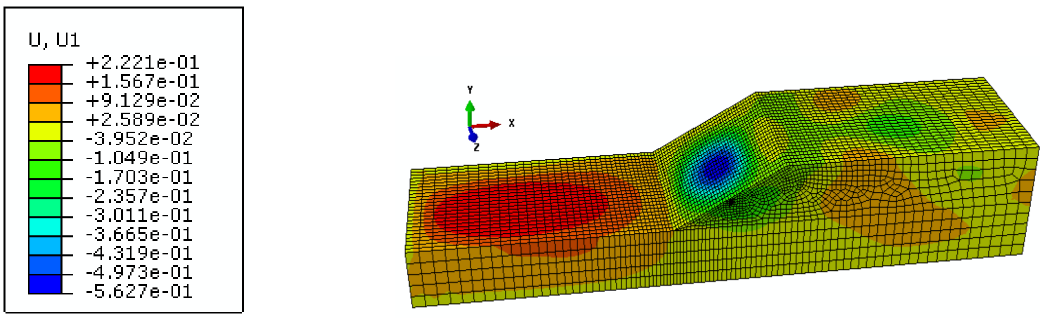


Figure 3.7. Horizontal displacement (m) of the slope for model 3 in z=50 m
(Equivalent to other models boundary planes)

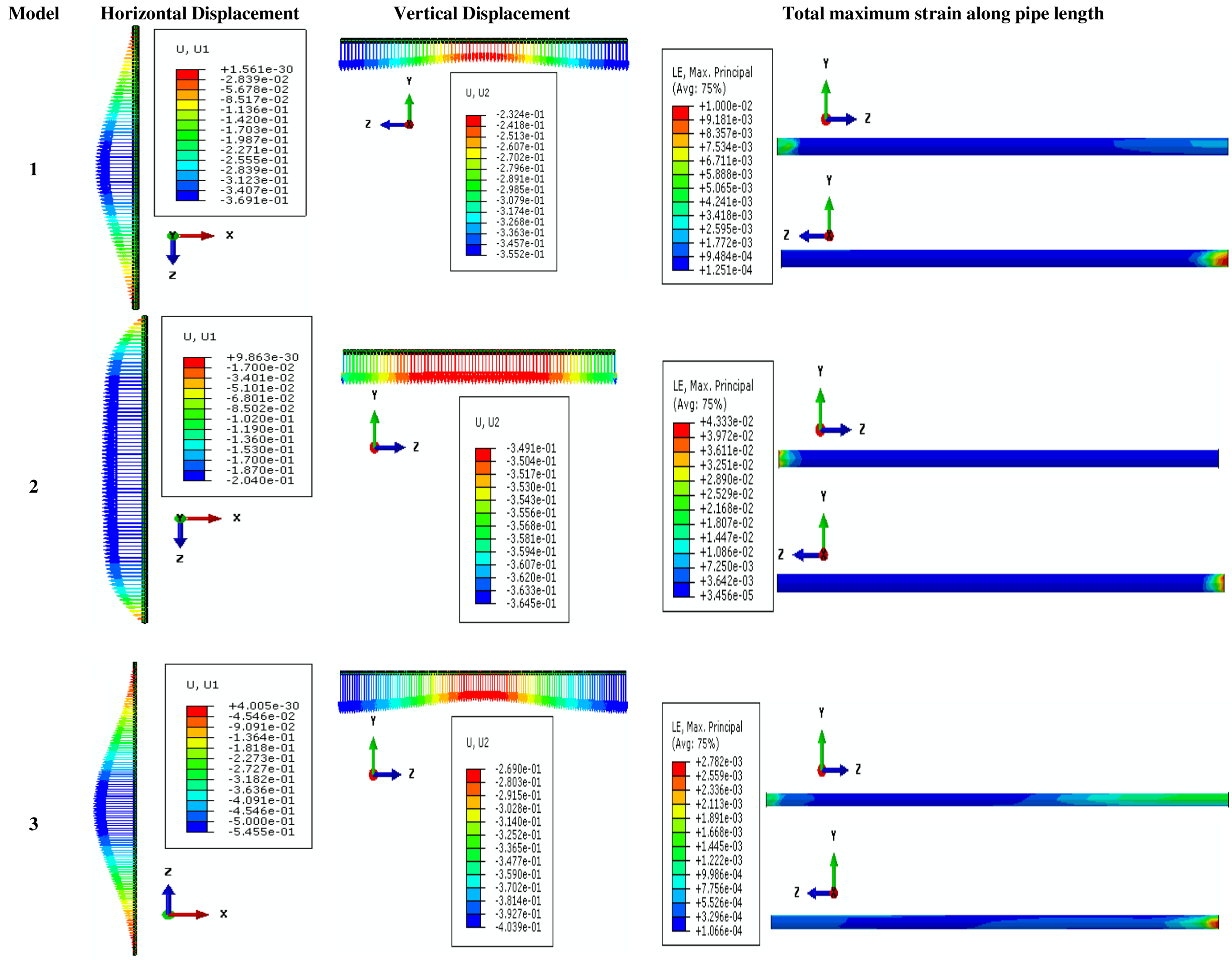


Figure 3.8. Pipe deformation magnitude and pattern and distribution of total strain along pipe

4. CONCLUSION

Behaviour of buried pipelines located in nonlinear cemented slopes and excited by dynamic loading of probable earthquakes in North Tehran area, was numerically modelled. To discover the influence of two parameters on slope deformation pattern and buried pipe strains, 7 numerical analyses were conducted by ABAQUS program. These two parameters were relative geometry of pipe in the slope and boundary conditions of edge planes. It is inferred that placing pipe in the slope toe, produces lower strains. As a result, it is suggested that the pipe passage be in lower parts of the slope (0<A<0.3) for more safe conditions. Besides the first model type, three boundary conditions including constrained, free and wider slope width were analysed and it is inferred that the 3rd model can more logically calculate the slope deformations and pipe strains. Also, free boundary condition of edge planes will predict erroneous results, as the displacements follow a rigid mass pattern. Also, to obtain more realistic results for calculated displacements, deformations and pipe strains, it is suggested that the slope model to have wider width.

AKCNOWLEDGEMENT
The presented results in this paper are part of a research project conducted in Sharif University of Technology supported by Tehran Gas Company as client, which is greatly appreciated.

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