THE UPLİFTİNG BEHAVİOR OF SHALLOW BURİED PİPELİNES WİTHİN THE LİQUEfiABLE SOİLS UNDER CYCLİC LOADİNGS

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Keywords: Baseline, Liquefaction, Buried Pipelines, Finite Difference Method, Flac- 2D, Finne Model.

ABSTRACT

The liquefaction of soils under earthquake loadings has always been a main concern for geotechnical engineering practices. As an earthquake causes the ground to liquefy, the effective stress and hence the shear strength of the soil decreases sharply, and large deformations happen in the area. This phenomenon occurs only rarely when the liquefaction occurs at a large depth. However, deformations increase extensively when this layer is located in shallow depths near the ground level. In this case super structures and also underground structures may be severely damaged. Pipelines buried in saturated sand deposits, during earthquake loading could damage from resulting uplift due to excess pore water pressure generation. Especially for previously buried pipelines, in order to set the priority for seismic retrofit, evaluating the risk of floatation in each region could be a concern. In this paper, effects of several parameters including soil dilatancy angle, soil friction angle, density ratio of natural soil, diameter and burial depth of pipe on uplift of pipe by construct an advanced soil- pipe model in Flac- 2D software and Finn behavior model under cyclic loading, have been investigated. Results show the prominent role of friction angle of soil, diameter of pipe and exist an optimum level for burial depth in pipe response reduced floatation.

1. INTRODUCTION

When the ground is subjected to strong shaking during an earthquake, liquefaction and subsequent ground settlement and/or flow failures, which involve extremely large movements of soil masses, may cause serious damage to civil/geotechnical infrastructures. Among those infrastructures, lifeline systems buried underground, such as common utility conduits and sewage systems, are vulnerable to medium to large ground movement. The geotechnical structures buried near the surface have a wide range of applications, from small-scale pipelines for gas transmission, telecommunications, water supply, and sewerage pipelines, to large-scale structures, including tunnels for various transportation system (Koseki J et. al, 1997 and Kang GC et. al, 2014).Moreover, destruction of water pipelines could prevent fire fighter's activities in restraining these fires. The 1989 Loma Prieta earthquake (O'Rourke et. al, 1991), 1994 earthquake of Northridge (Schiff AJ, 1997) and 1995 earthquake of Kobe(Shinozuka M et. al, 1995)were the well-known examples of lifeline failures, which drew more attention towards investigation of circumstances that cause pipeline failures. The underground tunnels that act like large diameter pipes could experience the same problems. Urban subway system of Taipei in 1999 earthquake of Chi-Chi encountered damages, as reported by Chou et al.(Chou HS et. al, 2001).In literature, seismic behavior of buried pipelines under earthquake excitations has been investigated by several researchers(Yong Y, 1997, Data TK, 1992, Karinski and Yankeleysky, 2007and Lee DH et. al, 2009). Moreover, for pipelines crossing active faults, series of centrifuge tests have been performed to evaluate the effects of several soil and pipe parameters on the structural response of buried pipelines (Abdoun TH et. al, 2009 and Vazouras P, 2010) and a remediation technique of using expanded polystyrene geofoam block as a low density backfill to reduce soil restraint and pipeline strains has been proposed(Choo YW et. al, 2007).Also, there are the results of studied effects of wave and soil characteristics and pipe geometry on excess pore pressure generation for seabed installation of pipelines(Maotina et. al, 2009 and Zhang XL et. al, 2009). A rather newly arisen phenomenon for scientists to investigate in the past decade was the floatation of buried pipelines in saturated deposits during intensive earthquakes, which can be defined as follows. Under earthquake loading, granular materials such as sands are susceptible to compaction. In saturated deposits, reduction in volume is prevented by the presence of pore fluids. Lack of drainage due to low permeability and short duration of loading result in a nearly untrained condition. This untrained condition that is accompanied by tendency to reduction in volume of soil skeleton builds up the pore fluid pressure. Consequently, the effective stress and so the shear resistance of these cohesion less soils reduces. By continuing generation of excess pore fluid pressure, gradually the effective stress diminishes, the process in which liquefaction could occur. Generation of excess pore water pressure beneath the pipeline and shear resistance reduction of soil above it, results in floatation of pipeline. Uplift resistance of offshore pipelines buried in liquefied clay was assessed by Bransby et al.(Bransby MFet al, 2002) and Cheuk et al.(Cheuk CY et al, 2007) Studies have been conducted on liquefaction-induced uplift of tunnels and two measures of secondary injection grouting and application of cutoff walls proposed (Chou HS et. al, 2001, Azadi and Mir Mohammad Hosseini, 2010, and Liu and Song , 2006).

This study continued through centrifuge tests by Ling et al.(Ling HI et. al, 2003) to assess the effectiveness of surface gravel drains in uplift reduction. Application of these countermeasures reduced the uplift of pipe to about 10%. They made a numerical analysis of the aforementioned centrifuge test in the finite element code of DIANA-SWANDYNE II. They presented and compared the results of numerical modeling with experiment up to the onset of liquefaction. Although they did not take the soil–pipe interaction into account. The results showed a good agreement with experiment(Ling HI et. al, 2003). Zou et al.(Zou et. al, 2006)and Saeedzadeh and Hataf (Saeedzadeh and Hataf, 2011) presented the results of studies conducted to compare the performance of different shapes of surface gravel drains.

All the above mentioned researches focused on understanding of pipe uplift and the effectiveness of several mitigating techniques for improving the design of new pipelines and seismic retrofit of previously buried pipelines. But due to large amount of previously buried lifelines, evaluating the vulnerability of eachone in a region in order to set the priority for seismic retrofit could be a concern. One that seems to be a necessary action to predict system fragilities under a range of ground deformation conditions for high intensity earthquakes, specifically after the Tohoku Earthquake of Japan in 2011(Ashford S rt. Al, 2011).

Various parameters affect the uplift of pipelines that should pobe inspected. Azadi and Mir Mohammad Hosseini (Azadi and Mir Mohammad Hosseini, 2010) evaluated the effects of several factors in uplifting behavior of shallow tunnels within the liquefiable soils, pore pressure changes in surrounding soil and bending moments and axial forces in tunnel lining. They examined the friction and dilatancy angles and damping of soil and the embedment depth and diameter of tunnel in addition to the effect of non-liquefied layer in contact with liquefied layer. They concluded that by increasing the damping ratio, friction and dilatancy angles of soil, tunnel embedment depth and reducing the tunnel diameter, the displacement of tunnel decreases.

Saeedzadeh and Hataf (Saeedzadeh and Hataf, 2011) investigated the influence of dilatancy angle in different density ratios of soil, diameter and burial depth of pipe as well as water table and the thickness of saturated soil layer in pipelinefloatation, too. They concluded that by increasing the dilatancy angles of soil in comparative high density ratios (Dr>70%) displacement of buried pipeline always decreases. Also by reducing the Pipe diameter, drop down the water table and increase buried depth uplift of pipelines decreases .In this paper, the influence of dilatancy angle in constant density ratio of soil, densiry ratio, friction angle, diameter and burial depth of pipe in pipeline floatation have been investigated.

2. MODELING STEPS

The simulation model takes place in two steps. First, the primary state is defined. This step has two phases: the initial phase, which includes static steady state in the drained position, and the second phase, in which the pipe modeling is implemented by define the pipe thickness boundary and removing the soil element. In next step, a dynamic analysis is performed. In the seismic loading model is subjected to a harmonic acceleration at the bottom of the model. Also In this step, untrained analysis is carried out for liquefaction modeling. Pipe deformation along the pipeline axis is much less than that in the other directions. On the other hand, pipe shape and its loading are symmetrical. Thus, the 2D plane strain model is an adequate one and can be used in this condition.

3. MODEL SPECIFICATIONS

The purpose of the article is to present the results of the study of the dynamic behavior of the buried pipe in the liquefied soil. For this purpose, a pipe with the selected mesh shown in Fig.(1) is considered in the area. According to this figure, the pipe is considered with 3 m interior diameter. The center of which is located 3 m below the ground surface. Pipes geometry is symmetry and soil properties are homogeneous so in this research for increasing of analysis rate, half of experimental model is simulated.

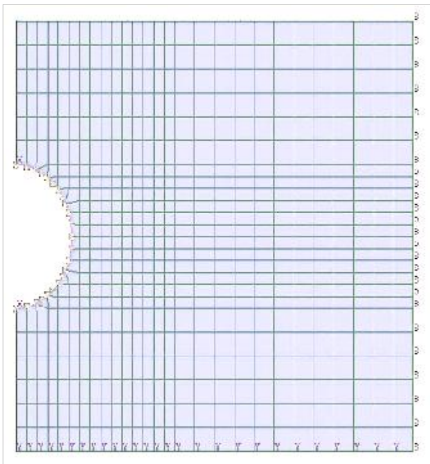

Figure 1. Display of mesh and pipe position in the base model

Regarding the assessment of liquefaction effects, the soil behavior modeling should be used to consider the excess pore pressure during the liquefaction. The Finn model is capable of considering variations of the volumetric strain and shows the increase of excess pore pressure under underlain condition. The aforementioned model has been introduced by Martin in 1975 and defines the relationship between variations of volumetric strain Increment $\left(\Delta £_{vd}\right)$ and cyclic shearing strain $\left(¥\right)$ as follows:

$$\Delta £_{vd} = C_1\left(¥ - C_2 £_{vd}\right) + \frac{C_3 £^2_{vd}}{¥ + C_4 £_{vd}} \tag{1}$$

C_1, C_2, $C_3$and $C_4$are coefficients that can be obtained from cyclictriaxial tests and are constant in the equation. Based on Pashangpishe' studies (Pashangpishe Y, 2004), 0.79, 0.52, 0.2 and 0.5 are values selected respectively, for above coefficients. In FLAC software, Finn model is correlated with plastic Mohr- Columb modelby Eq (1).

In this research use of Nevada sand specifications with four density ratios of 40%, 50%, 60% and 70%, from Arulmoli et. al(Arulmoli et.al, 1992) report of VELACS project are summarized in Table 1. Recommendations of software are used for other parameters. It has to be noted that because of shallow burial of pipelines, soil specifications of isotropic ally consolidated untrained compression tests in low confining pressures have been used.

In this study, the same Rayleigh damping coefficients as Ling et al.(Ling HI et. al, 2008)have been used, $\sim = 56.548$ and $= 1.768 \times 10^{-4}$.

The finite difference grid (square elements) spans the physical domain being analyzed. Most problems, however, are defined by grids that consist of hundreds or thousands of zones. A grid is defined by specifying the number of zones desired in the horizontal (x) direction, and the number of zones in the vertical (y) direction. The grid is organized in a row-and-column fashion. Beam elements are two-dimensional elements with three degrees of freedom (x-translation, y-translation, and rotation) at each end node. Beam elements can be joined together with one another and/or the grid. Beam elements are used to represent a structural member in which bending resistance and limited bending moments are important.

Table 1. Nevada sand specifications

D_r(%)	e_0	B- Value	Φ(deg)	ψ(deg)
40	6.85	0.98	32.5	1.9
50	5.20	1	33.9	3.7
60	4.90	0.98	36.4	4.2
70	4.30	1	38.3	4.4

4. ANALYSES AND RESULTS

To evaluate the influencing parameters on pipeline floatation, five variables including soil’s dilatancy angle (ψ), density ratio (Dr), friction angle (Φ), pipe’s diameter (D) and burial depth (BD) have been assessed. Naghan and Tabas earthquake records and a sinusoidal acceleration of 0.6 g and frequency of 3 Hz for 10 s have beenused. The Naghan and Tabas earthquake recordis presented in Appendix A. First, the finite difference model should be validated. Thus, this model results have been compared against results of a centrifuge test and also a numerical modeling of Ling et al.(Ling HI et. al, 2003 and Ling HI et. al, 2008) .

4.1. FINITE DIFFERENCE MODEL VALIDATION

Due to unspecified soil–pipe interaction characteristic, the interface streng thre duction factor (R_{inter}) has been changed to gain the best conformity with experimental results. Five different amounts for this factor were supposed and the uplift at the top and bottom of the pipe was compared with the experimental results. It was shown that the best match is achieved for reduction factor of 0.38. Uplift of pipe at the top and bottom of the pipe are indicated in Fig. (2) and Fig. (3), respectively. After 2–3 s of shaking, the analyzed and measured results show good agreements, while for excess pore water pressures, this agreement is achieved much sooner.

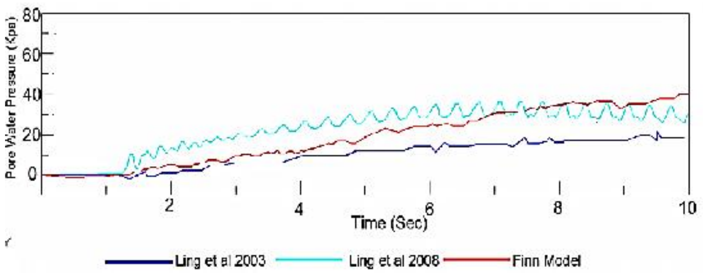

Figure 2. Experimental andtwo numerical resultsofexcessporewaterpressure,top of pipe.

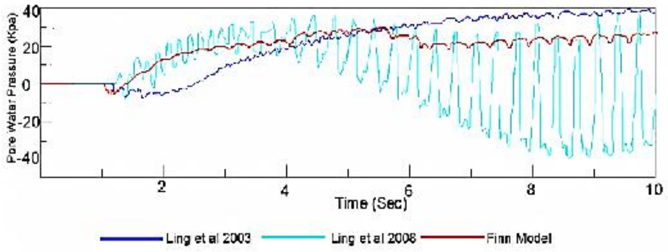

Figure 3. Experimental andtwo numerical resultsof excessporewaterpressure,bottom of pipe.

4.2. EFFECTS OF DILATANCY ANGLE OF SOIL

To evaluate the effect of dilatancy angle of soil on pipe uplift, 3 dynamic analyses, consisting of 3different angles, for a pipe of 3 m in diameter and burial depth of 1.5 m have been performed, Table 2

Table 2. Analyses to evaluate soil dilatancy angle effect

(·) (Deg)	Uplift (cm)
1	6
2.3	10.05
3.7	9.1

The seismic loading these analyses was the sine wave acceleration of 0.6g and frequency of 3Hz for 10s. The maximum uplift at the end of shakinghave been plotted against dilatancy angle for 40% density ratios, Fig. (4). As can be seen, in a soil with constant relative density,increase the uplift of pipe with increase the dilatancy angle and reach peakarises,then decrease slowly. Results show that variations of uplift depending on specific range of dilatancy angle. For example increase of this factor for soils with dilatancy angle more than 2.3 degree,cause to decrease uplift of pipe.

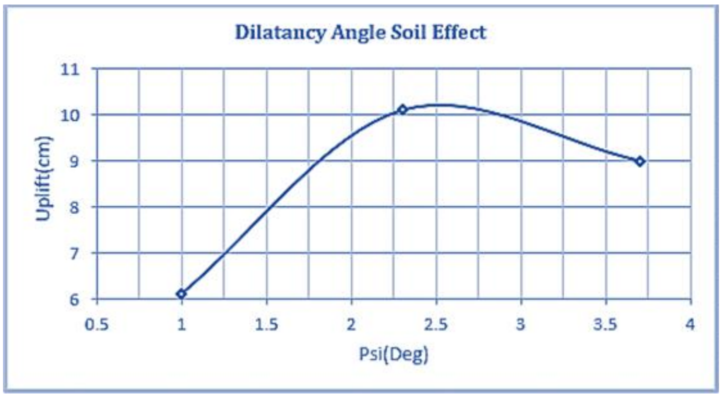

Figure 4. Effect ofdilatancyangleofsoilonpipeuplift

4.3. EFFECTS OF RELATIVE DENSITY OF SOIL

A 3m diameter pipe at five different density ratios of soil, at burial depth of 1.5 m under, Tabas earthquake record analyzed. Graphs of maximum uplift of pipe versus density ratio of these 5 analysis are presented in Fig. (5). Results show, by increasing the density ratio of soil, pipe floatation decreases.

Table 3. Analysis toevaluatesoildensity ratioeffect

D_r (%)	Uplift (cm)
40	6.85
45	5.95
50	5.20
55	4.90
60	4.30

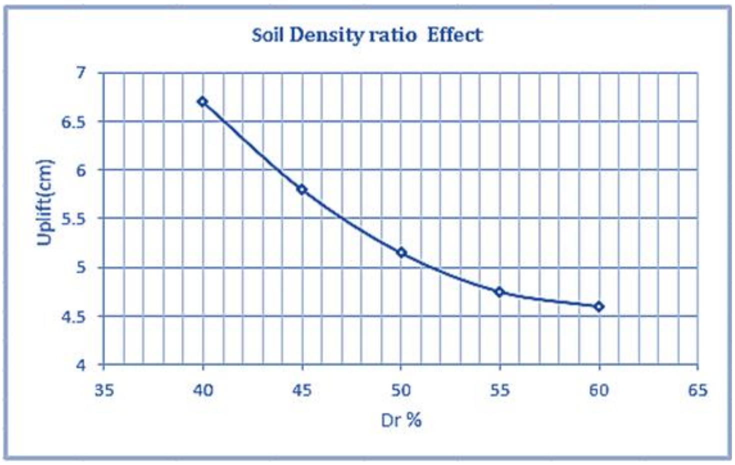

Figure 5. Effect ofsoildensity ratioonpipeuplift

The Fig (5) shows that reductionrate of uplift less than 55 percent is noticeable, but this variation for density ratiomore than 55 percent is negligible.

4.4. EFFECTS OF FRICTION ANGLE OF SOIL

Another parameters influencing the soil strength against liquefaction is the soil friction angle. Uplift of the Pipe is evaluated by variations of the soil friction angle. Fig (6)Illustrates variations of the pipe uplift versus changing the soil friction angle.

According to the figure, increasing the friction angle causes structural uplift to reduce. The reduction is the consequence of decreasing the soil liquefaction effect resulting from higher interparticleforces between grains in this condition. Based on the analysis results, in a soil with a friction angle of 15 degree,the vertical displacement is estimated to about 40 cm, while the structural uplift reduces to about 6 cm in the soil, having a friction angle of 35degree. Thus, the soil friction angle has a significant effect on the damages caused by uplifting due to the soil liquefaction. According to Fig (6), it is seen thatwith increasing of friction angle, uplift force decreases.This result is in agreement with the findings by Azadi and Mir Mohammad Hosseini (Azadi and Mir Mohammad Hosseini, 2010)and Joshi. et. al.(Joshi S et. al, 2011) for the soil friction angle.

Table 4. Analyses to evaluate soil friction angle effect

(%)	Uplift (cm)
15	40
20	32
25	21
30	11
35	6

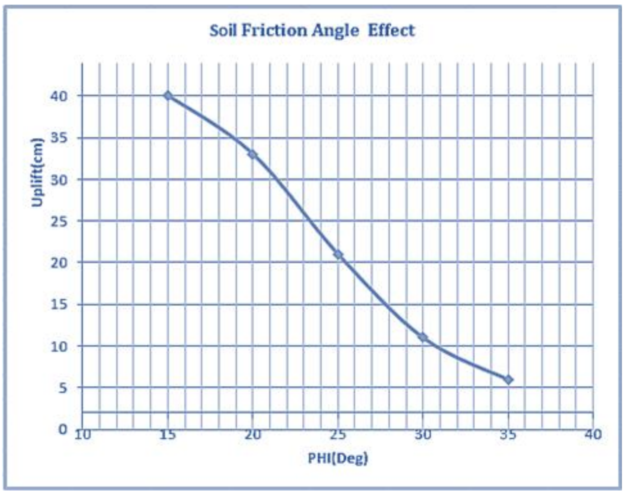

Figure 6. Effect ofsoil friction angle onpipeuplift

4.5. EFFECTS OF DIAMETER OF PIPE

Investigation to determine the effect of pipe diameter on floatation has been made through four analyses on pipes with varying diameters, buried in Nevada sand of 40% density ratio, at the depth of 3 m under previously mentioned sine wave acceleration. According to Fig.(7), with increasing the pipe diameter ratio (pipe diameter normalized by pipe diameter in base model), due topipe weight andconsequentlybuoyancy force increment, uplift of pipeincreased,but it seems that increasing rate of uplift for diameter ratio more than 1.16 negligible.This result is in agreement with the findings by Saeedzadeh and Hataf(Saeedzadeh and Hataf, 2011).

Table 5. Analyses to evaluate diameter of pipe effect

Diameter ratio	Uplift (cm)
0.58	6.10
0.80	8.70
1.00	10.35
1.16	11.20
1.25	11.39

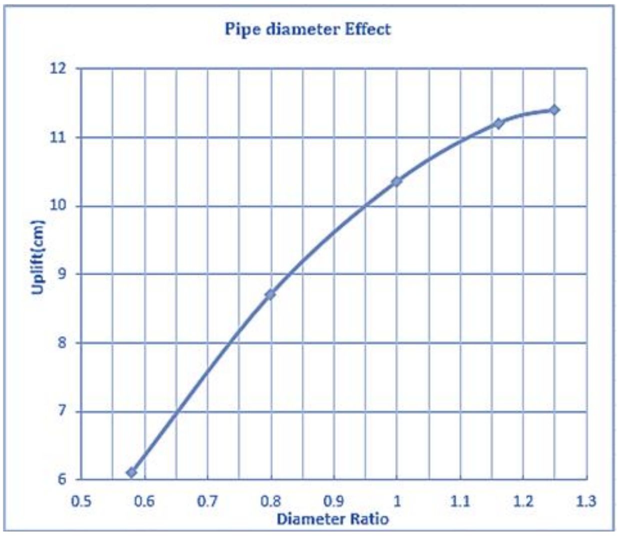

Figure 7. Effect ofdiameter of pipe onpipeuplift

4.6. EFFECTS OF PIPE BURIAL DEPTH

Another varying parameter in this study was the burial depth of pipe. Then five analysis on a 1 m in diameter pipe, buried in Nevada sand of 40% density ratio, atdifferent depths Tab.(6), under acceleration record of Naghan earthquake have been conducted. Normalized uplift of pipe by burial depth versus burial depth plottedin Fig (8). Thefigureindicatesthatthe uplift ofpipedecreasesasthe burial depthandconsequentlytheoverburdenpressureincreases. Moreover, therateofthisreductionbypassingtheburial depth equal to pipediameter(BD=D=1 m) decreases.Theseresultsareinaccordance with the findings by AzadiandMirMohammad Hosseini(Azadi and MirMohammad Hosseini, 2010),Saeedzadeh and Hataf (Saeedzadeh and Hataf, 2011) and findings by Baziar et.(Baziar M H et. al, 2014)in study of interaction between reverse faulting and tunnel.

Table 6. Analyses to evaluate burial depth of pipe effect

burial depth (m)	(Uplift / burial depth) (cm)
0.40	0.127
0.50	0.118
1.00	0.05
1.20	0.04
1.50	0.033

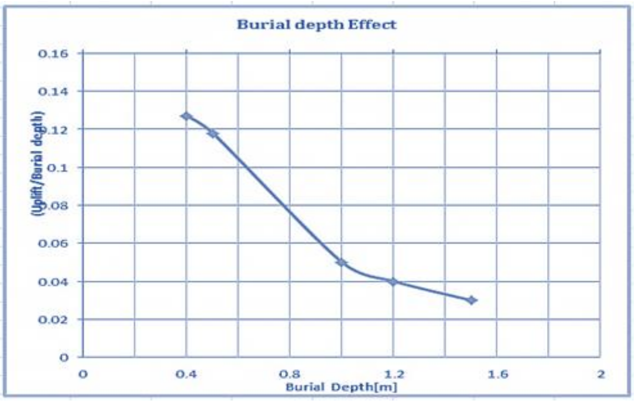

Figure 8. Effect ofburial depth of pipe onpipeuplift

5. SUMMARY AND CONCLUSIONS

The present analysis have been performed to develop the understanding of effective parameters in pipeline uplift and the following conclusions have been made:

1. In this paper, the buried pipelins uplift effects caused by soil liquefaction are assessed by using FLAC 2D software in Finn behavior model and Byrne formulation tomade a advanced model under cyclic loading for soil liquefaction. In these assessments, soil elements are considered as square elements and pipe elements as beam element types. At first a reference model is considered to deal with the issue in which pipe located in Nevada sand and the surrounding soil has been totally liquefied. Thus, different parameters are evaluated by the assessment of liquefaction effects in the model. In this case, a model proposed by Ling et. al(Ling HI et. al, 2006) which describes the pressure dependency of sand behavior for different initial densities and drainage conditions under cyclic loading and is unified over a wide range of confining pressures in the sense of a single set of parameters, could be a suitable choice.
2. For every constant density ratio, there is a specific rang of dilatancy angle that by increasing this parameter can decrease uplift of pipeline in earthquake loading. In this paper for density ratio equal to 40%, dilatancy angle morethan 2.3 degree, effectiveness.

3. Increasing the density ratio of sands lessens the maximumuplift of pipe. As the sand become denser, decrease the rate of uplift.
4. Increasing the friction angle of sands lessens the uplift significantly and this parameter independent from density ratio of soil.
4. Larger diameter pipes undergo more uplift and the rate of this increase decreases as the pipe become larger. It reveals that, although the contact width of larger pipes increases both for the overburden soil and the generated pore water pressure beneath the pipe, the water pressure overcomes the resistant soil. It shows the double effect of excess pore water pressure generation, which simultaneously increases the buoyancy force and decreases the effective stress of soil.
5. Despite the uplift decreasing in deeper buried pipes, bypassing burial depth of pipe diameter lessens the rate of this descent. Then couldassign this depth as optimum depth for buried pipes.

Appendix A

See Figs. A- 1 and A- 2.

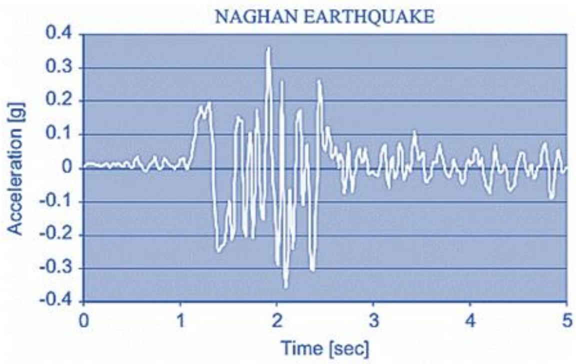

Figure A- 1

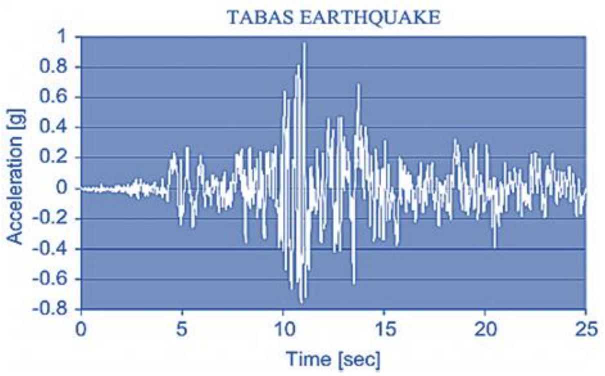

Figure A- 2

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