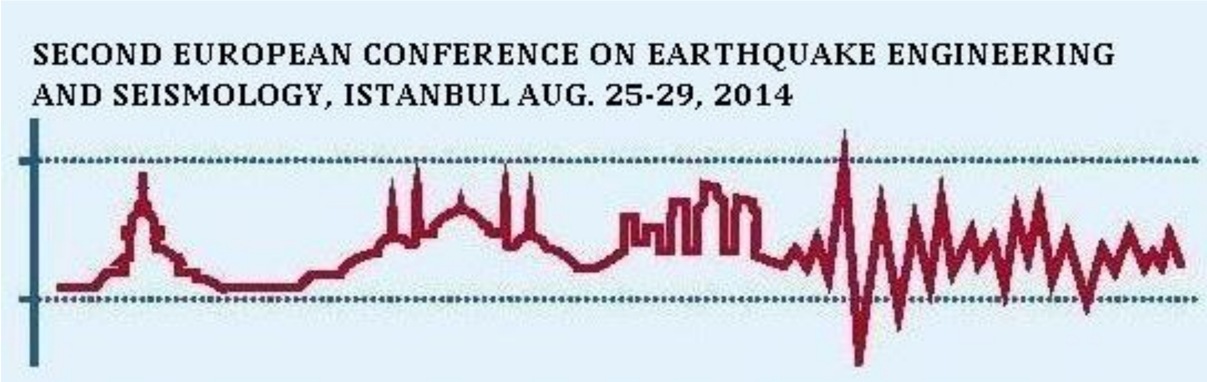


# REAL – TIME EARTHQUAKE DAMAGE ASSESSMENT OF BURIED GAS PIPELINES

## ABSTRACT

Okan İLHAN[1] and A. Can ZÜLFİKAR[2]

It is widely known that recent earthquakes have significantly influenced lifeline structures such an extent that even if there is not any damage in structures or any facilities on ground, the fracture of gas pipelines resulting in gas leakage leads to fires, economic losses and disability of lifeline networks. Therefore, the possibility of encountering with such a great hazard makes inroads into developing a method to assess the performance of available gas pipeline networks and to envisage the way of reinforcing them in the case of considerable damages under earthquake excitation.

In general, researchers have dealt with the quantification of lifeline hazard in that some of them appeal to the equations which are improved after the occurrence of significant earthquake by computing the number of pipeline breakages/leakages on site with respect to distance. (km) This route allows people to constitute fragility curves including relationship between repair per distance and selected ground motion parameter. (usually PGV) However, this method requires the pipeline damage inventory for a certain specific ground motion(s) and prevents one who does not have any data to develop such relationships. Also, it is customary to model numerically the pipeline and surrounding soil to perform nonlinear time history analysis so that computed strains can be compared with the limiting values given with regard to different types of buried gas pipelines. However, this pipeline – soil interaction model does not make it possible for constructing a rapid response system to be implemented immediate after earthquake.

This paper presents a procedure which bears on the system assuming that pipelines behave as a structural beam and the surrounding soil is modelled as elasto – plastic spring element. (Winkler Foundation System) This is the method released in “Recommended Practices for Earthquake Resistant Design of Gas Pipelines (Draft, 2000) – Japan Gas Association” but modified and further developed to provide compatibility with buried gas pipeline network and seismic hazard data of İstanbul. The Japan Code states that the surrounding soil and pipeline act together under the condition of wave passage, thus paving the way for deriving a transfer equation to calculate pipeline strain with the help of the one occurred in soil. For that purpose, İstanbul is geographically partitioned into grid cells having dimensions of 400mX600m. Pipeline inventory (length, diameter, the number of straight pipelines, bends and tees, pipe coating types, radius of curvature values for bends), real – time spectral acceleration values at 0.2 and 1 sec., soil classification data are extracted from each cell. Finally, this modified procedure is coded by means of VBA in Microsoft Excel in order to perform a rapid analysis with respect to given inventory and data. Results are mapped to supply gas company with a chance to perform immediate treatment to risky areas after earthquake.

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# INTRODUCTION

Larger magnitude earthquakes point out that lifeline systems composing of water, sewage and gas pipelines are exposed to significant damages due to either wave passage or permanent displacements resulting from landslides or both of them, thus resulting in economic loss in urban areas. This drawback has forced people to derive or appeal to a great variety of methodologies in that finite element methods can be deployed to model both pipeline and surrounding soil in order to conclude in more detailed and reliable results or some empirical equations can be proposed to reveal the response of lifeline facilities to a certain earthquake. EC FP7 project SYNER – G(2009), developed for Europe, outlines the empirical correlations plotted as repair rate per km vs. PGA/PGV and fragility curves for determined probability of exceedance values. This route provides researchers with attaining reasonable and amenable results due to the fact that relationships are not dependent on generic data.

Also, it is possible to generate Finite – Element Model (FEM) or Finite – Difference Model (FDM) in which the soil medium and pipe material can be modelled more rigorously but this way of observing pipeline damages requires lots of time even for quite small – scale model. This disadvantage of numerical approaches impel people to consult on both simpler and trouble – free models in that pipelines and surrounding soil are devised as structural beam and elasto – plastic spring element, respectively. Nonlinear direct integration time history analysis can be performed on them provided that ascertained convergence criterion should be satisfied during calculations. Such calculations may demonstrate the earthquake response characteristics of site and lifeline characteristics but this type of approach bears on theoretical basis for soil – spring rigidity that has not been justified by real measurements or well – designed lab tests. This means that one should be aware of the deficiency of this model, thus compelling him to do trade – off between time and accuracy constraints.

On the other hand, there exists such logic to evaluate the response of lifeline systems under strong ground motion that soil strains occurred due to the wave passage effect can be converted into those experienced by pipelines with the help of transfer coefficients. This methodology is the most appropriate one for our case since we have been expected to put forth a rapid response routine which allows gas distribution company to perform immediate treatments on site under any hazardous condition. For similar demand in 2000, “Recommended Practices For Earthquake Resistant Design of Gas Pipelines (Draft, 2000) – Japan Gas Association” was proposed to produce pipeline damage investigation system based on soil – pipeline interaction model. Just as done in the approach explained above, pipeline and its surrounding soil are devised as structural beam and spring having elasto – plastic force – deformation relationship, respectively. What is different from classical interaction theory embraced for lifeline facilities is that there is no need for carrying out linear/nonlinear direct integration time – history analysis, instead maximum spectral velocity values of selected earthquake for interested region are sufficient for estimating the degree of earthquake disruptiveness. In other words, these values are employed as an indication of how strong motion is hazardous and enable us to calculate maximum strain of surrounding soil. All these computations are conducted on grid – based İstanbul Map, equipped with the information on pipeline network inventory, soil classification data, spectral acceleration values, etc. What is provided to reader in this article is grid – based real – time earthquake hazard investigation routine of the gas pipeline network in spite of the deficiencies in its ability to cover all phenomena pertaining to behaviour of the network under earthquake excitation. All the details on theory and computations are presented in the Analysis Section.

# ANALYSIS

As stated in Introduction Part, the onset of methodology is to calculate the maximum ground displacement of surface layer(s), which are in essence obtained from the integration of ground strain, occurred in relevant direction. In our case, the ground displacement is contrived as sinusoidal shape in conjunction with the adoptation of quarter – wavelength theory, which yields,

$$U_h = \frac{2}{\pi^2} * T * S_v(T) \cos\left(\frac{\pi z}{2H}\right) \tag{1}$$

Where,
$U_h$, is ground displacement of surface layer (m)
$S_v(T)$, is spectral velocity of interested soil layer(s)
T, is natural period of ground surface layer (sec)
z, is buried depth of pipeline (m) (~1 m)
H, is thickness of interested soil layer(s) (m).

In (1), the spectral – velocity values are computed through using the real – time spectral accelerations at 0.2 and 1 sec. extracted from ELER (Earthquake Loss Estimation Routine), which provides us with an opportunity of achieving those in regard to probability of exceedance of 50%, 40%, 10% and 2%, respectively. At first, all values approximated by ELER are converted into spectral velocity values by means of $S_a = \omega * S_v$, where the natural period of ground surface layer can be obtained as;

$$T = \frac{4H}{V_s} \tag{2}$$

In our situation, H is assumed as 30 m since ELER makes available to use $V_{s30}$ (m / sec) data for each grid. After the calculation of real – time ground displacements, the soil strains for uniform ground condition can be given as;

$$\varepsilon_{G1} = \frac{2\pi * U_h}{L} \tag{3}$$

$$L = V * T \tag{4}$$

Where,
V, apparent propagation velocity of seismic motion (m/sec), which can be obtained through Figure 1.
T, is natural period of ground surface layer(s) (sec)

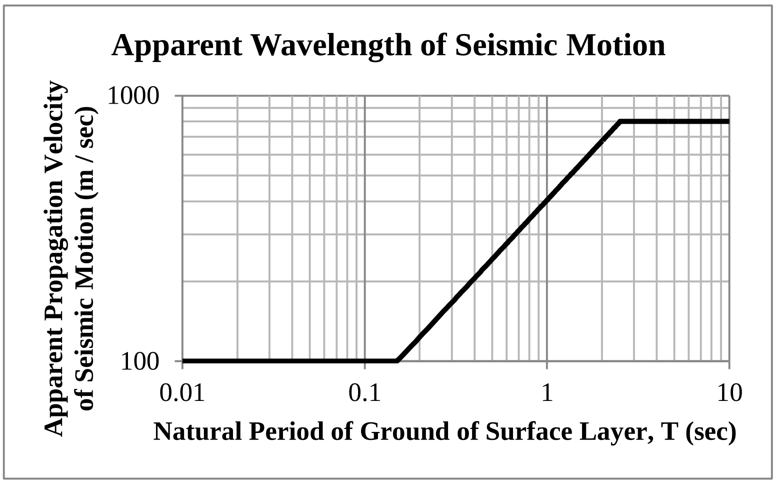


Figure 1. Apparent Wavelength of Seismic Motion

At this point, we should ensure whether our ground surface is uniform or not because it may be required to the modification of soil strain such as;

$$\varepsilon_{G2} = \sqrt{\varepsilon_{G1}^2 + \varepsilon_{G3}^2} \tag{5}$$

Where
$\varepsilon_{G2}$, is ground strain caused in irregular shallow ground
$\varepsilon_{G3}$, is ground strain caused by inclined seismic base rock. (assumed as 0.3% in this study)

At this point, the key parameter "soil strain" is plugged into the analysis carried out for straight pipeline, bend and tee components in that each of these elements has their own transfer coefficients. In the first place, we will deal with the design of straight pipeline, whose damage state is employed as the indication of other two elements.

## Hazard Analysis for Straight Pipe

As stated above, our model bears on the fact that elasto – plastic soil spring is attached to the pipelines conceived as structural beams at certain points, which are determined as “vertex points” in the study. The first step to be followed is to compute ground spring rigidity in axial direction with regard to formulations presented in ALA (2001);

$$T_u = \pi D \alpha c + \pi D Z \bar{\gamma} \frac{1 + K_0}{2} \tan\delta \tag{6}$$

Where,
D, is pipe outside diameter
c, is soil cohesion representative of the soil backfill (c is in ksf or kPa/100)
Z, is depth to the pipe centreline
$\bar{\gamma}$, is effective unit weight of soil
$K_0$, is coefficient of pressure at rest, $= 1 - \sin\varphi$

$$\alpha = 0.608 - 0.123c - \frac{0.274}{c^2 + 1} + \frac{0.695}{c^3 + 1} \tag{7}$$

$\delta$, is interface angle of friction for pipe and soil, $f\varphi$
$\varphi$, is internal friction angle of soil
f, coating dependent factor relating the internal friction angle of the soil to the friction angle at the soil – pipe interface.

Representative values of f for various types of external pipe coatings are given in Table 1. In addition to this, the displacement values beyond which different types of soil display nonlinear behaviour are presented in Table 2. In the analysis, the soil characteristics in Table 3 and the force – deformation relationship (FDR) in Figure 2 were used. This allows one to calculate the initial rigidity as 8635 kN/m. Also, it is stated that required parameter for pursuing the evaluation of straight pipeline under wave passage effect is the transfer coefficient. To that end, (8) presents the strain transfer coefficient of the straight pipe without sliding taken into account;

Table 1. Friction factor f for Various External Coatings

| Pipe Coating | f |
|---|---|
| Concrete | 1.0 |
| Coal Tar | 0.9 |
| Rough Steel | 0.8 |
| Smooth Steel | 0.7 |
| Fusion Bonded Epoxy | 0.6 |
| Polyethylene | 0.6 |

Table 2 Δt Values for Different Soil Types

| Soil Type | Δt (mm) |
|---|---|
| Dense Sand | 3 |
| Loose Sand | 5 |
| Stiff Clay | 8 |
| Soft Clay | 10 |

Table 3 Soil Properties

| Soil Type | Dense Sand |
|---|---|
| Burial Depth (m) | 1 |
| γ (kN/m3) | 19 |
| $\Phi^0$ | $45^0$ |
| (c) (kPa) | 0 |

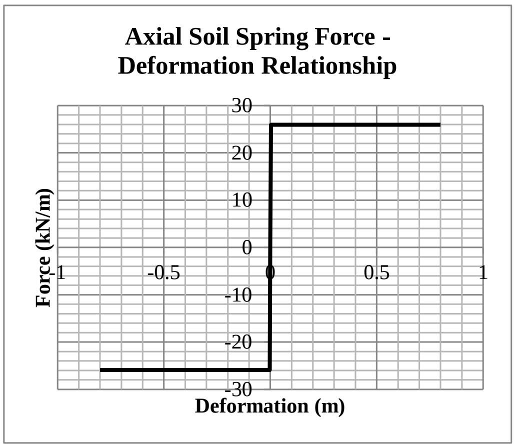

Figure 2 FDR for Surrounding Soil (as representative for 8’’)

$$\alpha_0 = \frac{1}{1+\left(\frac{2\pi}{\lambda_1 * L}\right)^2}, \lambda_1 = \sqrt{\frac{K_1}{E * A}} \tag{8}$$

Where,
E, is elastic modulus of pipe material. ($2 * 10^8$ kN / m$^2$)
A, is cross – sectional area of pipeline.

At this point, $\alpha_0$ is combined with the sliding reduction coefficient, q, whose definition is dependent on the criterion of whether sliding initiation critical shear stress ($T_u$) is greater than shear stress acting on the pipe surface $\tau_G$ or not;

1. If $\tau_G \geq T_u$

$$q = 1 - \cos\xi + \Omega * \left(\frac{\pi}{2} - \xi\right)\sin\xi \tag{9}$$

2. $\tau_G < T_u$

$$q = 1 \tag{10}$$

Where,

$$\tau_G = \frac{2\pi}{L} Et\alpha_0\varepsilon_G \tag{11}$$

$$\xi = \arcsin\left(\frac{\tau_{cr}}{\tau_G}\right) \tag{12}$$

Ω, is correction factor for evaluating q on the conservative side, 1.5.
t, is radial thickness of pipe material
$\varepsilon_G$, is computed soil strain.

After that, the strain transfer coefficient, α, can be attained as,

$$\alpha = q * \alpha_0 \tag{13}$$

The remaining part in straight pipe analysis is to compute the pipe strain provided that it should be assessed whether it lies within the linear range or not since nonlinearity during calculation imposes the alteration of strain values to reach more reasonable results. Therefore, following division is adopted to recover the possibility of nonlinear behavior occurrence throughout analysis as;

1. If $\alpha * \varepsilon_G \leq \varepsilon_y$ (Elastic range)

$$\varepsilon_p = \alpha * \varepsilon_G \tag{14}$$

2. If $\alpha * \varepsilon_G > \varepsilon_y$ (Elastic range)

$$\varepsilon_p = \varepsilon_G \tag{15}$$

Where,
$\varepsilon_y$, is yield strain of pipe material. (for material used by gas company ~ 0.002)
$\varepsilon_p$, is strain of straight pipe caused by earthquake.

In the analysis the pipe type of API 5L Grade B has been used and this enabled us to extract the limiting strain values from IITK – Gsdma Guidelines for Seismic Design of Buried Pipelines Provisions with Commentary and Explanatory Examples (November, 2007), in which an equation for the onset of wrinkling damage is suggested as;

$$\varepsilon_{cr-c} = (0.75 \sim 1.0) * 0.175\frac{t}{R} \tag{16}$$

Where,
t, is thickness of pipe
R, is radius of pipe

The straight pipes used in analysis were all subjected to this routine in which the spectral velocity values were supplied with respect to the probability of exceedance of 50%, 40%, 10% and %2 in 50 years. Pipe types employed during computations and hazard map for 2% probability of exceedance in 50 years (having return period of 2475 years) were provided in Table 4 and Figure 3, respectively. In Figure 3, only two categories as safe and unsafe (in gray and blue colors, respectively) are proposed for classification of calculated strains in that the former one represents the values smaller than limiting strain and the latter indicates the possibility of earthquake damage occurrence at that location.

Table 4. Different Pipe Types Used in the Study

| Pipe Type | Pipe Diameter (mm) | Radial Thickness (mm) |
|---|---|---|
| 2’’ | 60 | 3.9 |
| 3’’ | 89 | 4 |
| 4’’ | 114.3 | 4.37 |
| 6’’ | 168.3 | 4.37 |
| 8’’ | 219.1 | 4.78 |
| 10’’ | 273 | 5.2 |
| 12’’ | 323.8 | 5.56 |
| 16’’ | 406.4 | 6.35 |
| 20’’ | 508.0 | 7.14 |
| 24’’ | 609.6 | 7.92 |
| 28’’ | 711.2 | 9.52 |
| 30’’ | 762.0 | 11.13 |

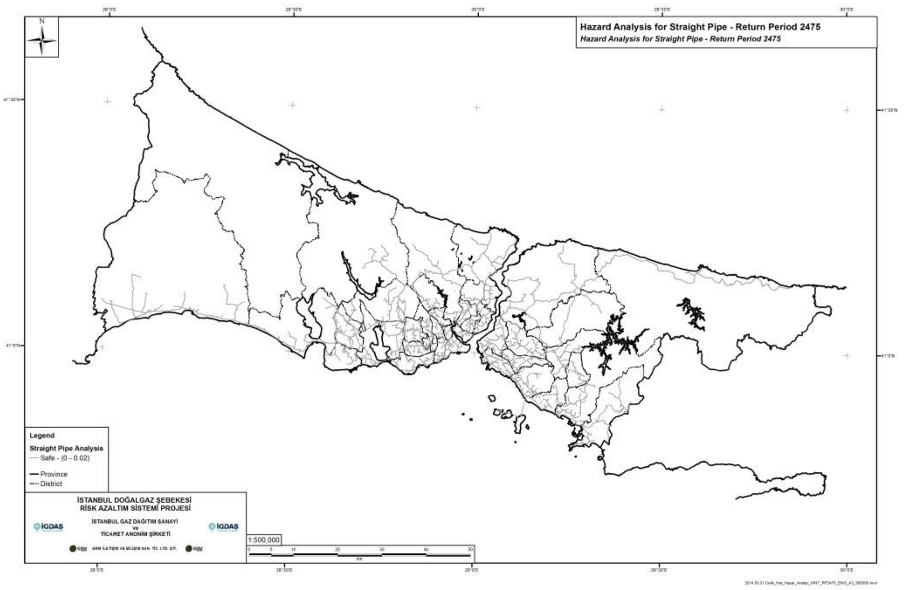


Figure 3. Hazard Map of Straight Pipe for 2% Probability of Exceedance in 50 Years

## Hazard Analysis for Bend and Tee Elements

As stated by Japan Code, bends and tees are the most vulnerable elements due to the stress concentrations during wave passage effect through pipeline network, thus forcing people to quantify the possible detriment on these components. The logic that has been explained until this point is also applicable for bends in that the relative displacement between pipe components which are attached to each other by these elements (see Figure 4) and surrounding ground gains importance. The displacement values (m) for two interested pipes can be computed to employ the greater one in the analysis as ;

$$\Delta = (1 - \alpha^{*}) * U_h \tag{17}$$

Where,

$\alpha^*$, has identical definition to that of (13) but is modified in terms of $q^*$ as,

1. If $\tau_G \geq T_u$

$$q^{*} = \sin\xi * \left(1 + \frac{\pi^2}{8} - \frac{\xi^2}{2}\right) - \xi * \sin\xi \tag{18}$$

2. If $\tau_G < T_u$

$$q^{*} = 1 \tag{19}$$

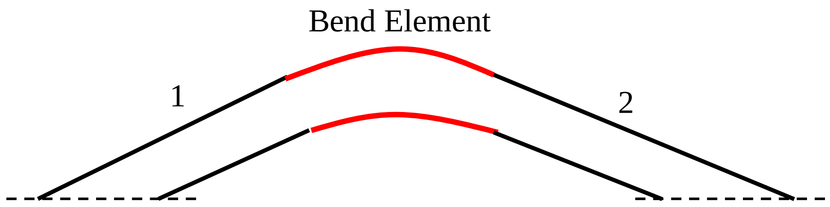


Figure 4. Bend Element Configuration

After that, the same route is tracked in bend strain calculations with a difference that conversion coefficient of bend, which is nearly similar to transfer coefficient, is introduced as;

$$\beta_B = \frac{2 i_B A \bar{\lambda}^2 D \left|(5 + R\bar{\lambda}) b_1\right| + 4\bar{\lambda}^3 I |5(1 + b_2) - b_1|}{10A + 5LI\bar{\lambda}^3 (1 + b_2) + 10Ab_3} \tag{20}$$

Where,

$$b_1 = -\frac{1 + 2R\bar{\lambda} + (\pi - 2)nR^2\bar{\lambda}^2}{(1 + R\bar{\lambda})\{2 + \pi nR\bar{\lambda} + (4 - \pi)nR^2\bar{\lambda}^2\}} \tag{21}$$

$$b_2 = -\frac{1 - 2nR^2\bar{\lambda}^2 - (4 - \pi)nR^3\bar{\lambda}^3}{(1 + R\bar{\lambda})\{2 + \pi nR\bar{\lambda} + (4 - \pi)nR^2\bar{\lambda}^2\}} \tag{22}$$

$$b_3 = nR^3\bar{\lambda}^3 \left\{\frac{\pi}{2} + \frac{\pi I}{2nAR^2} + \left(1 - \frac{I}{nAR^2}\right) b_1 + \left(\frac{2}{R\bar{\lambda}} + \frac{\pi}{2} + \frac{\pi I}{2nAR^2}\right) b_2\right\} \tag{23}$$

$i_B$, is stress index for the bending load of the bend, obtained from the following formula;

$$i_B = \frac{1.95}{\left(\frac{4tR}{D^2}\right)^{2/3}} \text{ or } 1.5, \text{whichever is larger} \tag{24}$$

n, is flexibility factor of the bend, obtained from the following formula;

$$n = \frac{1.65}{\frac{4tR}{D^2}} \tag{25}$$

A, is sectional area of pipe ($m^2$)
R, is radius of curvature (m)
I, is moment of inertia ($m^4$)
D, is outside of diameter of pipe (m)
L, is apparent wavelength of seismic motion (m)

$$\bar{\lambda} = \sqrt[4]{\frac{K_2}{4EI}} \tag{26}$$

$K_2$, is ground spring constant in the transverse direction of the pipe (kN / m) and is computed by recommendations of ALA (2001) with respect to Table 3 as 15681 kN/m;

$$P_u = N_{ch} cD + N_{qh} \bar{\gamma} ZD \tag{27}$$

Where
$N_{ch}$, is horizontal bearing capacity factor for clay (0 for c = 0);

$$N_{ch} = a + bx + \frac{c}{(x+1)^2} + \frac{d}{(x+1)^3} \leq 9 \tag{28}$$

$N_{qh}$, is horizontal bearing capacity factors for sand (0 for $\varphi = 0^0$)

$$N_{qh} = a + b(x) + c(x^2) + d(x^3) + e(x^4) \tag{29}$$

Each coefficient can be taken (or interpolated by) from using Table 5 and the FDR for transversal soil spring is presented in Table 5 and Figure 5, respectively as,

Table 5. Values for Parameters in (27)

| Factor | $\varphi^0$ | x | a | b | c | d | e |
|---|---|---|---|---|---|---|---|
| $N_{ch}$ | 0 | H/D | 6.752 | 0.065 | -11.063 | 7.119 | 0 |
| $N_{qh}$ | 20 | H/D | 2.399 | 0.439 | -0.03 | 0.001059 | -0.00001754 |
| $N_{qh}$ | 25 | H/D | 3.332 | 0.839 | -0.09 | 0.005606 | -0.0001319 |
| $N_{qh}$ | 30 | H/D | 4.565 | 1.234 | -0.089 | 0.004275 | -0.00009159 |
| $N_{qh}$ | 35 | H/D | 6.816 | 2.019 | -0.146 | 0.007651 | -0.0001683 |
| $N_{qh}$ | 40 | H/D | 10.959 | 1.783 | 0.045 | -0.005425 | -0.0001153 |
| $N_{qh}$ | 45 | H/D | 17.658 | 3.309 | 0.048 | -0.006443 | -0.0001299 |

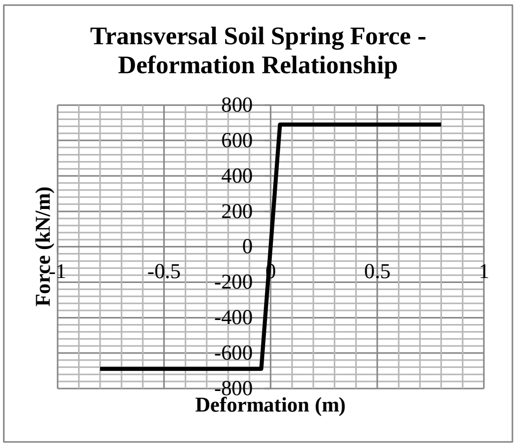


Figure 5. FDR for Transversal Soil Spring (as representative for 8'')

In addition to this, the displacement value beyond which different types of soil displays nonlinear behaviour are presented as;

$$\Delta_p = 0.04\left(Z + \frac{D}{2}\right) \leq 0.1D \text{ to } 0.15D \tag{30}$$

The differentiation between linearity and nonlinearity is also embraced in the implementation of damage evaluation routine for bends such as;

1. $\beta_B \Delta \leq 1.27\varepsilon_y$ (Elastic range)

$$\varepsilon_B = \beta_B \Delta \tag{31}$$

2. $\beta_B \Delta > 1.27\varepsilon_y$ (Plastic range)

$$\varepsilon_B = C_B \beta_B \Delta \tag{32}$$

$C_B$, is correction factor for the strain of the bend in the full plastic range,

$C_B = 2$ (Below 24'')

$C_B = 1$ (Above 24'' including 24'')

At the end of calculations, same limiting strain definition in (16) is adopted to generate hazard map for bend elements as;

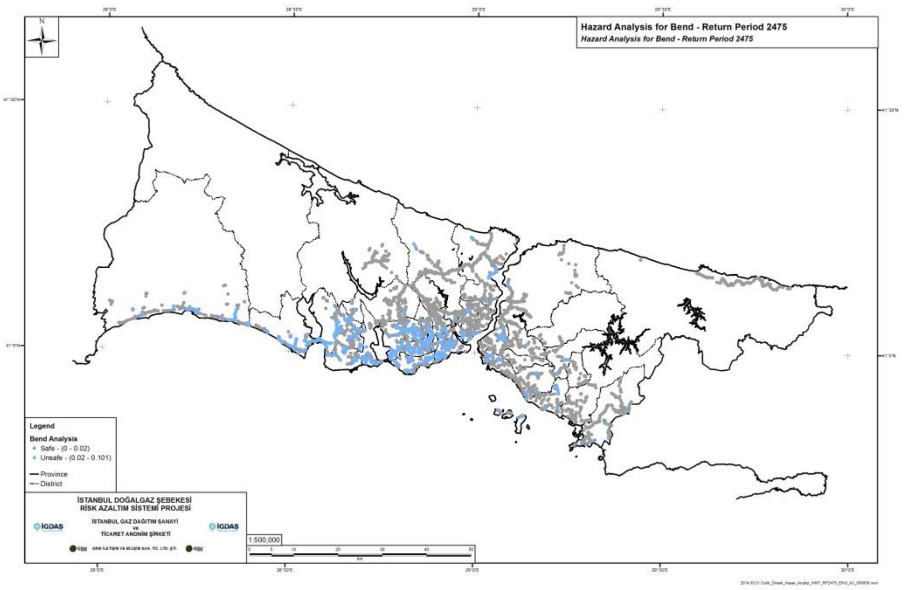


Figure 6. Hazard Map of Bend Elements for 2% Probability of Exceedance in 50 Years

The rationale behind assessing the earthquake performance of tee components is in essence identical to those mentioned above. In this case, a branch pipe is added to the joint between two pipes at which a tee element is located in order to reduce the diameter of main pipeline as equal to that of branch one. (see Figure 7) This component is also quite prone to stress concentrations which may be likely to result in metal fatigue and forced researchers to cope with the likelihood of damage occurrence in those elements.

At first, the conversion coefficient of tee is computed like performed in bend as;

$$\beta_T = \frac{4\bar{\lambda}_1^2 D_1 A_2 (C-1)}{4A_2 + L I_1 \bar{\lambda}_1^3 C} \tag{33}$$

Where,

The subscripts for D, A, I and $\lambda$ express the following;

Subscript 1: Branch Pipe Side

Subscript 2: Main Pipe Side (In computations, the main pipe portion exposed to larger displacement value is assigned as one possessing subscript 1.)

$$C = \frac{1 + 4\left(\frac{\bar{\lambda}_1}{\bar{\lambda}_2}\right)^3 \left(\frac{D_2}{D_1}\right)}{1 + 2\left(\frac{\bar{\lambda}_1}{\bar{\lambda}_2}\right)^3 \left(\frac{D_2}{D_1}\right)} \tag{34}$$

The separation between nonlinear and linear behavior of tee element is also imposed in calculations in which it appears as;

1. If $\beta_T \Delta \leq 1.27 \varepsilon_y$

$$\varepsilon_T = \beta_T \Delta \tag{35}$$

2. If $\beta_T \Delta > 1.27 \varepsilon_y$

$$\varepsilon_T = 2 \beta_T \Delta \tag{36}$$

At the end of calculations, same limiting strain definition in (16) is adopted to generate hazard map for tee elements in Figure 8 as;

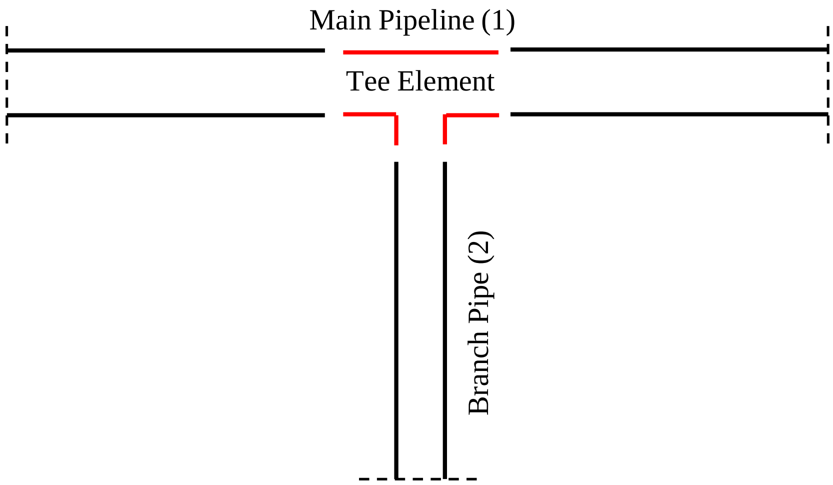


Figure 7. Tee Element Configuration

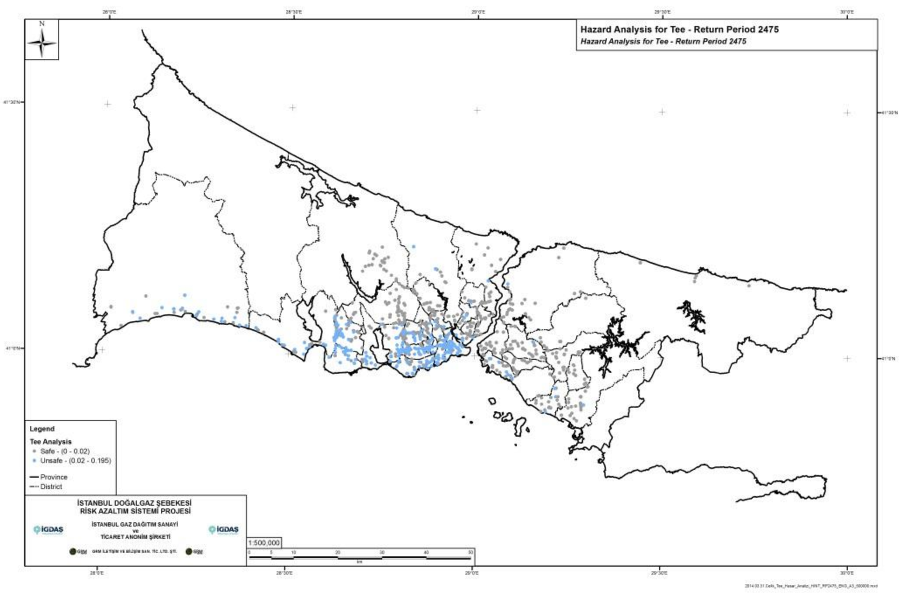


Figure 8. Hazard Map of Tee Elements for 2% Probability of Exceedance in 50 Years

The flow chart for our routine, which is coded and named as PipeHazard, is delineated in Figure 9 as;

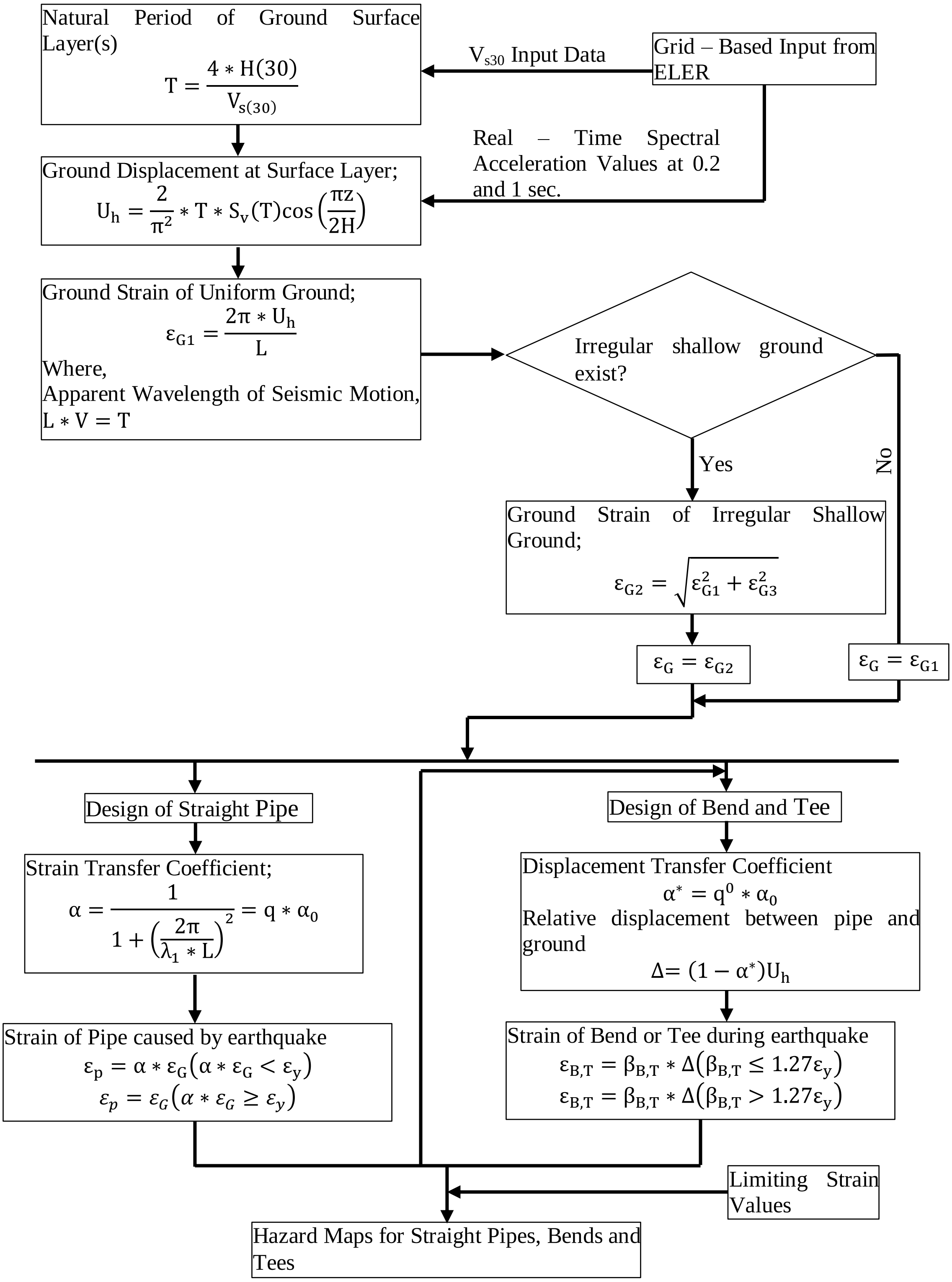


Figure 9. Flow Chart for PipeHazard Routine

# CONCLUSIONS

All computations and the theory lying behind this article are intended for deriving a satisfying answer to the question: “Is it possible to mitigate the risk at lifeline systems during earthquake?”. To that end, we have been trying to fulfill this gap by a routine, which presented in Japan Gas Association (2000) Recommended Practices for Earthquake Resistant Design of Gas Pipelines (Draft) has been modified and also partly developed to be implemented as compatible with data for İstanbul. Being able to extract required data from ELER, we noticed at the onset of project that the calculations could be performed at the real – time scale in order to supply gas company with a chance to generate maps containing possible hazardous locations on site, thus sending them to the responsible workers.

Our modified routine is dependent on a rather simple assumption; "There is a correlation between soil strains during earthquake and those occurred in pipe elements". This valuable phenomenon raises a theory allowing one to devise a transfer coefficient that establishes a bond between surrounding soil and pipeline network. The most critical issue after calculations by means of this presumption is to make decision on how damage criterion is set forth to display the earthquake performance of lifeline facilities in İstanbul. The formula given in document published by Indian Institute of Technology Kanpur (2007) is appropriate for our case since pipe type employed in the study is involved in pipe categories employed to derive this equation.

It is well – known that soils exhibit nonlinear behaviour after certain strains which are usually quite small values thus the differentiation between these phenomena should be placed. In this routine, computed strains for straight pipes, bend and tee elements are modified to encompass nonlinearity which is also compensated for by coefficients to reserve the simplicity of algorithm. At the end, generated maps demonstrated that south side of İstanbul is quite vulnerable to earthquake – induced damage and damage susceptibility gradually reduces towards to regions located in north of İstanbul.

On the other hand, our modified and partly improved algorithm is still primitive and now may not permit us to produce more reliable results. To present more details on this issue, the fact that all pipe portions of the network are modelled as having free – end conditions means that the anchor points which represent the fixed – end situations are ignored in analysis although it may be likely to encounter with great deviations in computations if included in algorithm. Also, equations defined for FDR of soil springs merely based on theoretical approach provide such larger values that they should be modified, developed or renewed as compatible with real measurements or data obtained from lab applications.

In conclusion, this routine is capable of producing reasonable results that researchers can come up with reliable inferences for earthquake hazard characteristics of interested lifeline network in spite of its deficiencies partly explained above. The code, PipeHazard, is still under development to recover possible unexpected outputs and to expand its current capability of representing site and lifeline conditions.

## REFERENCES


American Lifelines Alliance (2001) Guideline for the Design of Buried Steel Pipe, July

Committee on Gas and Liquid Fuel Lifelines of the ASCE Technical Council on Lifeline Earthquake Engineering (1984) Guidelines for the Seismic Design of Oil and Gas Pipeline Systems, October

Japan Gas Association (2000) Recommended Practices for Earthquake Resistant Design of Gas Pipelines (Draft)

Indian Institute of Technology Kanpur (2007) IITK – Gsdma Guidelines for Seismic Design of Buried Pipelines Provisions with Commentary and Explanatory Examples, November

Lee DH, Kim BH, Lee H, Kong JS (2009) "Seismic Behavior of a Buried Gas Pipeline under Earthquake Excitations", *Engineering Structures*, 1011 – 1023

McLaughlin P O' Rourke M (2009) "Strain in Pipe Elbows Due to Wave Propagation Hazards", *TCLEE 2009: Lifeline Earthquake Engineering in a Multihazard Environment, 2009 ASCE*

O' Rourke MJ, Liu X (1999) Response of Buried Pipelines Subjected to Earthquake Effects, Multidisciplinary Center for Earthquake Engineering Research, University of Buffalo

O' Rourke MJ, Symans MD, Masek JP (2008) "Wave Propagation Effects on Buried Pipe at Treatment Plants", *Earthquake Spectra, Volume 24, Pages 725 – 749*

Systematic Seismic Vulnerability and Risk Analysis for Buildings, Lifeline Networks and Infrastructures Safety Gain (SYNER – G) (2009) Fragility functions for gas and oil system networks, November

Takada S, Tanabe K (1987) "Three – Dimensional Seismic Response Analysis of Buried Continious or Jointed Pipelines", *ASME*

Takada S, Horinouchi N, Tsubakimoto T, Tsuchiya M, Ogawa Y (1988) "Earthquake Resistance Evaluation of Service Junctions in a Small – Diameter Steel Pipeline", *Proceedings of Ninth World Conference on Earthquake Engineering, August 2 – 9, Tokyo – Kyoto, JAPAN*

Xie X, Symans MD, O' Rourke MJ, Abdoun TH, O'Rourke TD, Palmer MC, Stewart HE (2013) "Numerical Modeling of Buried HDPE Pipelines Subjected to Normal Faulting: A Case Study", *Earthquake Spectra, May, Vol. 29, No. 2, pp 609 – 632*