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Using Neural Network for Reliability Assessment of Buried Steel Pipeline Networks Subjected to Earthquake Wave Propagation

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Abstract: Several cases of failures in steel buried pipelines under the effect of wave propagation have been reported. Due to seismic waves propagations these pipelines will encounter various axial forces and bending moments which will consequently lead to the local buckling of the pipes and the reduction of the pipes hollow-sectional area. These effects cause overall reduction of efficiency of pipes. Due to the probabilistic nature of soil and earthquake specifications, a deterministic approach for analyzing buried pipeline networks against earthquake excitations is not appropriate. In this study an algorithm for reliability assessment of buried pipeline networks is proposed which is based on nonlinear dynamic analysis and calculation of reliability using Monte Carlo simulation. Due to complexity of numerical analyses of buried pipeline networks, there is no possibility of an explicit calculation for the performance limit state function, so a trained multilayer feed forward neural network was used as an alternative. For this purpose, the obtained results of many deterministic numerical analyses were used for training the neural network and the performance limit state function was replaced by trained neural network. Finally, based on the probability density function, standard deviation and average of probabilistic parameters, reliability of the pipeline network for different performance levels, was determined. By investigating a buried pipeline network in sandy soil as a case study, effectiveness of the proposed algorithm was investigated and by determining the importance measure of probabilistic parameters, sensitivity analysis was performed.

Key words: Buried steel pipe network, earthquake wave propagation, reliability, neural network, local buckling

INTRODUCTION

Buried pipelines are endangered by Permanent Ground Deformations (PGD) and wave propagations. PGDs like landslide, faulting, lateral spread and subsidence are more dangerous for the buried pipes due to more intensive localized effects. Several cases of failures due to the effects of PGDs have been reported by O'Rourke and Liu (1999). The strains created in the pipeline under the effect of PGDs are greater than those created by earthquake waves. However, wave propagations cover a significantly larger area of the pipeline network which imposes a more meaningful failure to the system.

The Mexico City earthquake of 1985 is a good example for this type of failure. Based on the report of Ayala and O'Rourke (1989) no significant PGD in the city was observed. But extensive failures in the water supply due to the wave propagation were reported. Although, these failures were more common in brittle pipes such as asbestos cement or concrete pipes, rather than the more ductile steel or polyethylene pipes. The Study of earthquake effects on buried pipeline networks and the damage induced on them is usually performed by calculating damage functions or fragility curves. These functions or curves present the number of failures per unit area, against the Peak Ground Acceleration (PGA) or Velocity (PGV).

Eguchi *et al*. (1983) separated damages to the pipelines owing to PGD or wave propagations, in order to compute damage functions. O'Rourke and Ayala (1993) presented the damage rate against the PGV for different kinds of concrete pipes, cast iron, asbestos cement and etc., which was based on the available information of four earthquakes in the United States and two earthquakes in Mexico. Chen *et al*. (2002) did a study about the damage to gas and water supply systems on Chi-Chi (Taiwan) earthquake. Through calculating damage functions, they

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concluded that the best input parameter for determining the rate of damage of the gas pipes is PGA. Shih and Chang (2006) surveyed damages to the water pipelines in Taiwan due to the Chi-Chi earthquake. They categorized the causes of the failures in buried pipelines due to the earthquake. In which a Ground vibrating and wave propagation of 48% and a PGDs, liquefaction and other 52% were recorded.

As it is shown, the failure due to wave propagation has a high percentage. Damage functions may define just an overall sense of the damage in a specific network and cannot offer any information about occurred failure levels, location or reduction of the network's performance. To have a better understanding of the damages, researchers proposed using analytical and numerical methods for calculating the pipelines response subjected to wave propagation. Newmark and Rosenblueth (1971) calculated the analytical response of the buried pipeline under earthquake effects, by assuming similarity of pipe and ground deformation. Other methods by Shah and Chung (1974), Takada (1977), O'Rourke *et al*. (1982), O'Rourke and El-Hmadi (1988) and Wang *et al*. (1982) were offered, each of which has employed some more refined assumptions comparing with the initial work by Newmark and Rosenblueth (1971).

In spite of the efficiency of all these analytical methods, in some cases the exact response of the pipelines cannot be calculated. This is due to the complexity of the behavior of buried pipes, alongside their complex geometry and the nonlinear behavior of soil. Therefore, methods based on numerical analysis were given much greater attention by researchers.

Various modeling techniques have been proposed by researchers in recent decades. Wang and O'Rourke (1977) concluded that relative motion between the soil and the pipe are negligible by modeling the pipe as a beam model on an elastic foundation. They assumed linear behavior for both pipe and soil with considering ground deformation as a sinusoidal wave. Muleski *et al*. (1979) used finite cylindrical shell model for buried pipelines and Winker model for pipe and soil interaction. They refrained from dynamic effects and by quasi static method analyzed buried pipes undergoing effects caused by earthquakes. Datta *et al*. (1982, 1984) studied the dynamic behavior of the buried pipes in a state of plane strain. They modeled the buried pipe as an infinite thin, isotropic, homogenous and elastic shell surrounded by a circular trench with the pipe concentrated at the centre.

O'Leary and Datta (1985) presented a method for estimating the maximum stress in buried pipes under the earthquake effect. They proposed a three-dimensional analysis method for obtaining the dynamic response of buried pipelines to incident compressional waves (S and P) travelling along the pipeline, with low frequencies and long wave lengths. Elhmadi and O'Rourke (1990) conducted a study about the effect of wave propagation on segmented buried pipelines. They considered nonlinear behavior for the connections and soil properties and also the connection behavior were assumed to be probabilistic parameters. Then, by using Monte Carlo simulation and quasi static analysis, they obtained axial displacement in each connection of the pipe and developed a diagram of connection tensile failure against ground strain. Datta (1999) conducted a detailed review on various modeling techniques for numerical analysis of buried pipes in soil and had considered the wave propagation as an important source of damage to pipelines.

Hosseini and Mogharian (1999) performed a study on the effect of harmonic transverse waves on a buried pipeline made of ST-37 steel, in which the nonlinear behaviors of the soil and pipe were taken into account. They used ERAUL-3 computer program and obtained the minimum amplitude by which yielding starts in the pipe section for any given excitation frequency. Hosseini and Ajideh (2001) conducted a study on the seismic analysis of jointed pipe systems in which the wave propagation was modeled as a multi-node input for both soil and pipe.

Rofooie and Qorbani (2008) performed a parametric study on the seismic behavior of buried pipes under the effect of three dimensional Kobe (Japan) earthquake records. Pipes with different lengths and free and fixed boundary conditions were analyzed. They concluded that for pipes with lengths above 900 m, the axial displacement in the middle of the pipe is independent of the two end conditions. Shi *et al*. (2008) examined the effect of an earthquakes surface axial wave on buried segmented pipes and their connections.

In a recent study by Hosseini *et al*. (2010) the seismic functionally of water distribution networks were evaluated. They conducted a series of nonlinear time history analyses in which the nonlinear behavior of pipe connections had been taken into consideration. They modeled pipe segments as beam elements and the connections as nonlinear springs to evaluate the performance of a sample distribution network. Finally, Hosseini and Roudsari (2010) performed a study on the effects of surface transverse waves on buried steel pipelines by considering the nonlinear behavior of soil and pipes. In their study the possible damages in straight continuous steel buried pipelines subjected to strong surface transverse waves had been investigated. By using numerical modeling they presented the maximum compressive strain corresponding to the start of local buckling in the pipe.

In attention to the probabilistic behavior of soil and earthquake parameters, reliability base analyses can present the seismic response of buried pipeline networks with an acceptable precision. As it is seen in the previous studies, researchers did not focus on the relationship between the reliability of buried pipeline networks and peak ground displacements or other earthquake specifications. The main purpose of this study is to determine the reliability of buried pipeline networks under the effect of earthquake wave propagation.

Due to the high ductility of steel pipes, it is unlikely that the pipe will become ruptured due to earthquake waves. However, large lateral or axial movements can lead to the pipe experiencing plastic deformation and buckling on its walls (Roudsari, 2011). These effects will result in the ovalization of the pipe section and the reduction in the pipe's hollow section area. With the reduction of the pipe hollow section area, the efficiency of the pipelines material transmission encounters difficulty and this causes a reduction in the pipes serviceability level. So, the damage level for each pipeline and approximate ovalization rate of its hollow section area, due to the effect of earthquake wave propagation should be determined based on the maximum strain of the pipeline. Calculation of the maximum strain in each pipeline is performed by the nonlinear dynamic finite element analysis.

By determining the Limit State Function (LSF) of the pipeline network, the Monte Carlo simulation method will be used to calculate the reliability. However, due to the use of finite elements analysis, the LSF of the pipeline network is not explicitly available and should be approximated. Multilayer feed forward Neural Network (NN) as a strong approximator is considered by many researchers.

Hurtado and Alvarez (2001) made a comparison between multilayer NNs and radial basis function network in determining the reliability of a complex system. They showed that the performance of a multilayer feed forward NN in approximating the LSF is better than the radial basis function network. Goh and Kulhawy (2003) used NN to model the LSF for reliability assessment. With different examples they showed the advantage of the NN over the other approximators, such as polynomials. Gomes and Awruch (2004) performed a comparison between the determination of the reliability, based on the response surface method by using NNs and other methods. It was concluded that the response surface method and approximation of the LSF with NN, is more accurate with a less number of input pairs. Schueremans and Van Gemert (2005) presented contents about the benefits of using the Splines and NN in determining the reliability of the structures. Deng *et al*. (2005), Elhewy *et al*. (2006) and Cardoso *et al*. (2008) used numerous linear and nonlinear examples to show the NN efficiency in the calculation of reliability in structures with implicit LSF.

Hence, the performance of multilayer feed forward NN in the approximation of complex functions by different researchers has been approved. Eventually by replacing the trained NN with performance LSF, the reliability of the pipeline network against the peak ground displacement is determined. The main purpose of this study was calculation of reliability of buried pipeline networks against Earthquake wave propagation using Neural Network and Monte Carlo simulations. The other goal of this study was to perform sensitivity analysis and investigate the effect of various probabilistic parameters on the network reliability.

MODELING THE BURIED PIPE NETWORK

With regard to modeling methods of buried pipes, comprehensive information can be found in reference Datta (1999). Definitely, the most perfect model is a three-dimensional model which the soil and pipe are modeled as solid and shell elements, respectively by considering their actual nonlinear behavior. Due to pipelines relatively large length and the high volume of required calculations, the use of this model is not practically possible. The most common model of buried pipelines used in this study is shown in Fig. 1, in which the pipeline is modeled by beam elements.

In this 2- or 3-dimensional model the surrounding soil is modeled by bilinear springs as shown in Fig. 2. The model shown in Fig. 1 is capable in taking into account soil-pipe interaction based on the ASCE Technical Council on Lifeline Earthquake Engineering (TCLEE) model. As shown in Fig. 2 behaviors of longitudinal and transverse-horizontal soil springs in two directions are assumed to be symmetric. The equivalent stiffness of these springs depends on the pipe diameter and soil specifications including density, internal friction angle, cohesion coefficient as well as the burial depth of the pipe.

The time delay to the wave propagation effect was taken into account by giving different start instances of excitation, to different equivalent soil springs along the pipe model based on the shear wave velocity in the soil. However, the variation of frequency content of the excitation along the pipe model has been neglected, as the soil type has been assumed to be the same along the pipe in each case.

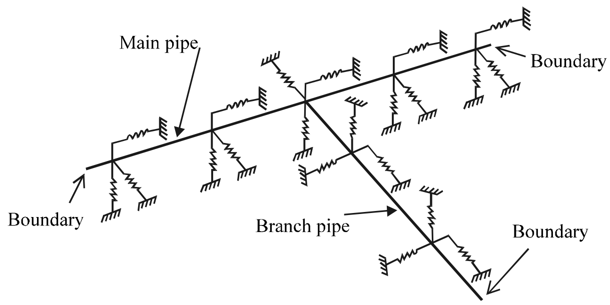

Fig. 1: Structural model of the buried pipeline

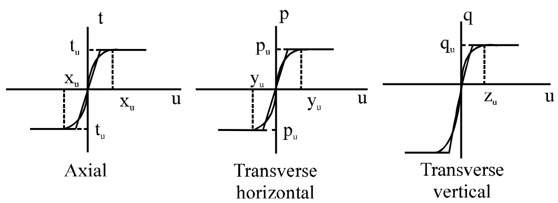

Fig. 2: The actual and idealized longitudinal and lateral behaviors of the equivalent soil springs for modeling the soil and its interaction with buried pipes

EVALUATION OF LOCAL BUCKLING OCCURRENCE AND THE REDUCTION OF PIPE HOLLOW SECTION AREA

Large ground deformations can cause the ductile steel pipes to experience plastic deformation and the reduction of pipe hollow section area. The first indicator of this reduction in the pipes hollow section area is the start of local buckling due to compressive strain resulting from bending and axial compressive forces. Hall and Newmark (1977) proposed that the axial strain, ε, corresponding to the start of local buckling in steel pipes to be in the following range:

$$0.15\frac{t}{R} < \varepsilon < 0.2\frac{t}{R} \tag{1}$$

where, t is the thickness of the pipe and R is the radius. Equation 1 was based on the laboratory test on thin wall cylinders. Based on the numerical analysis Hosseini and Roudsari (2010) confirmed Eq. 1 as a starting point of the local bucking for steel pipes with different diameters and thicknesses. For assessment of the efficiency reduction in the pipeline, the relationship between the reduction rate of the pipe hollow section area and the compressive strain resulting from bending and axial load, should be found. The numerical seismic analysis on pipelines such as water and gas are usually modeled as beam elements rather than shell elements due to their large length. Therefore, it is not possible to calculate the change in pipe hollow section area directly from these models. To measure the reduction in hollow section area, the pipe should be modeled by shell or solid elements.

In order to determine the relationship between the reduction of the pipe hollow section area and the maximum compressive strain, a pipe segment, was modeled by Shell elements. An analysis was performed by fixing one end and applying some rotation to the other end, the middle of the pipe was considered to have a small thickness imperfection, with the highest compressive stress located at this point.

By using this analysis the reduction of pipe hollow section area in the middle section of the pipe against the rotation of its free end was obtained. In the next step, the same pipe without the thickness imperfection was modeled by beam element, under the same loading. The maximum compressive strain values in the middle of the pipe segment against the rotation of its free end were calculated. By repeating these two cases for pipe segments with different diameters and thicknesses (according to Iranian Standards), the variation in A/A_0 value (obtained by Shell elements) against ε value (obtained by Beam elements) was plotted for pipe segments of different dimensions.

Figure 3 shows the relation between reduction of the pipe hollow section area and the maximum axial strain for the pipe with a diameter of 0.6096 m and a thickness of 0.00792 m. After analyzing a buried pipeline network under the effect of earthquake wave propagation and calculating the maximum axial strain in all the pipes, Fig. 3 and similar graphs can be used to calculate network efficiency reduction.

It is noticeable that Eq. 1 shows corresponding strain from the start of local buckling for the unburied pipe. Hosseini and Roudsari (2010) modeled pipes in various

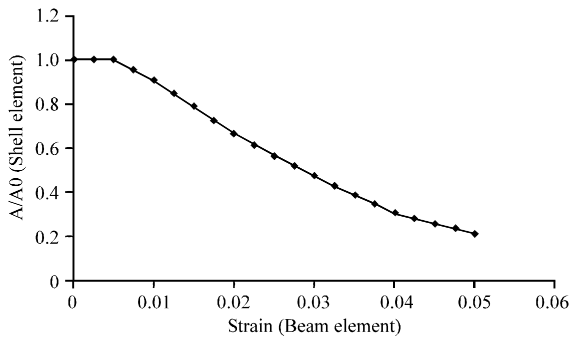

Fig. 3: The reduction of the pipe hollow section area with diameter of 0.6096 m and thickness of 0.00792 m, with respect to maximum axial strain

types of soil and showed that the strain value corresponding to the start of local buckling in the pipe buried in loose sand is less than those of the dense sand. This is due to the loose sand being softer which provides less lateral restraint for the pipe. They concluded that soil properties are influential in the amount of strain corresponding to the start of local buckling in the pipe. Although, this effect has been neglected in the Eq. 1 proposed by Hall and Newmark (1977). They corrected Eq. 1 for different types of soil and offered Eq. 2 and 3 for dense and loose sandy soil, respectively.

$$0.17\frac{t}{R^{1.5}} < \varepsilon < 0.26\frac{t}{R^{1.5}} \tag{2}$$

$$0.15\frac{t}{R^{1.1}} < \varepsilon < 0.34\frac{t}{R^{1.1}} \tag{3}$$

By numerous modeling of pipeline subjected to earthquake wave propagation, they concluded that the straight continuous buried steel pipes are not vulnerable against seismic waves with amplitudes of up to 1.0 m. They reported that damages due to wave propagation in the buried steel pipes in past earthquakes have been mainly due to the complex geometry of the pipe network and existence of several intersections. These factors change the seismic behavior of the buried pipe system. Meanwhile, part of the damages observed in buried pipe networks are due to corrosion in the pipes which is not the focus of this study (Yahaya *et al*., 2011). In the following text, the branch effect on the created strain in pipeline subjected to earthquake wave propagation will be considered.

For this purpose, the main pipeline with a T-shaped branch according to Fig. 1, with different diameters and thicknesses in different types of soil was modeled. These models occurred under the effect of three-dimensional components of amplified earthquakes. By calculating the maximum strain in the main and branch pipe by nonlinear dynamic finite element analysis and comparing pipe responses with local buckling threshold values, it is shown that the local buckling can occur in both pipes. The obtained results for one of these models will be reviewed. In this case, the main pipe was assumed to have a length of 1200 m and a diameter of 1.0668 m, with a consistent thickness of 0.0127 m. The branch pipe was also assumed to have a length of 600 m, diameter of 0.6096 m and a thickness of 0.00792 m. The stress-strain behavior of pipe steel material was assumed to be of Ramberg-Osgood model with steel type X-60 and burial depth of 1.5 m for pipes.

Table 1: Soil specifications and peak ground displacements of four selected earthquakes in each direction

Sand type	γ	φ	V_s	Earthquake name	D_A	D_{HT}	D_{VT}
Dense	21000	35	625	Northridge	100	92	60
				Chi Chi	86	95	96
Loose	18000	30	150	Kobe	107	98	45
				Loma Prieta	77	71	66

φ is the internal friction angle of soil in degrees, γ is the soil specific weight in N m^{-3}, V_s is the shear wave velocity in soil in m s^{-1} and D_A, D_{HT} and D_{VT} are, respectively the peak ground displacement in axial, transverse-horizontal and transverse-vertical directions in centimeter

Table 2: Maximum of compressive strain in the main and branch pipes and comparison of them with the corresponding values of starting local buckling

			Maximum axial compressional strain	Strain at start of buckling (Eq. 2 or 3)	Occurring of buckling in pipe
Dense sand	Northridge	Main	0.0085	0.0055	Yes
		Branch	0.0046	0.008	No
	Chi Chi	Main	0.0065	0.0055	Yes
		Branch	0.0036	0.008	No
Loose sand	Kobe	Main	0.0047	0.0038	Yes
		Branch	0.0062	0.0044	Yes
	Loma prieta	Main	0.004	0.0038	Yes
		Branch	0.0029	0.0044	No

Four earthquake records were selected based on their compatibility with the soil conditions, alongside their corresponding shear wave velocities are displayed in Table 1. In each analysis case, three records of the longitudinal, transverse and vertical displacements of the earthquake motions were simultaneously applied.

Table 2 compares the maximum compressive axial strain in the main and branch pipes based on the finite element analysis, with the lower limits of the corresponding strains from the start of local buckling (Eq. 2, 3). It can be seen that local buckling in some cases occurred.

To investigate the occurrence of local buckling at the junction, a short length of each side of this point modeled with the Shell element is shown in Fig. 4. Obtained

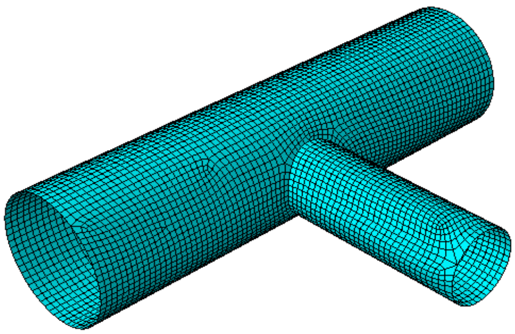

Fig. 4: The sample modeled for assessing the occurrence or non-occurrence of local buckling on the junction

displacements and rotations from dynamic analysis with the beam model were applied to these points identically and the nonlinear dynamic analysis was performed with consideration to the material and geometrical nonlinearity. Finally, the analyses were carried out in various cases and it was observed that although the little points at the junction entered yield phase, there was no observable local buckling or reduction of the pipes hollow section area. Therefore, it can be concluded that damages to the buried pipeline networks under the effect of earthquake wave propagation are local buckling of the pipes with the reduction of their hollow section area.

THE PROPOSED ALGORITHM FOR DETERMINING THE RELIABILITY OF BURIED PIPELINE NETWORKS

In this algorithm for determining the reliability of buried pipeline networks, NN and Monte Carlo simulation will be used. Figure 5 shows the various steps of the algorithm that will be reviewed in the following text with greater detail.

Selection of the probabilistic parameters that are effective on the pipelines network reliability: At the first step, all the effective parameters on the reliability of the pipeline network including soil characteristics, geometry and specifications of the pipeline network and ground motion specifications should be determined. It can be claimed that in an ideal case, all of these parameters are probabilistic and there are no deterministic parameters in the system. Since the explicit calculation of LSF in complex systems such as buried pipeline network are not possible, the NN as a powerful approximator will be used. For training the NN, a large number of nonlinear dynamic analyses should be conducted and the minimum numbers of required inputs are related with the numbers of probabilistic parameters, exponentially, as considered in the next section. On the other hand the nonlinear dynamic analyses of large pipeline networks under the effect of earthquake wave propagations are very time consuming. Therefore, the number of probabilistic parameters of the system should be reduced if possible.

Determining LSF of the pipeline network: In order to determine the reliability of the pipeline network, the failure level in the network should be determined as a function of the probabilistic parameters. Failure rate in each pipeline is calculated based on the occurrence of local buckling and the reduction of hollow section area as a function of the maximum axial compressive strain on the pipe. This is in accordance to Eq. 2, 3 and Fig. 3. So, the LSF should be able to calculate the maximum axial strain (ε_{max}) in each pipeline as a function of the probabilistic parameters. If the probabilistic parameters of the pipeline network are assumed to be x_1, x_2 and ... x_n, for the ith pipeline of a network, the performance LSF is as follows:

$$\varepsilon_{max,i} = LSF_i\,(x_1, x_2,..., x_n) \tag{4}$$

The relationship between probabilistic parameters such as x_1, x_2 and ... x_n and the maximum compressive axial strain in each pipeline is established by nonlinear finite element dynamic analysis. NN is used as an alternative because there is no possibility of calculating an explicit expression for the LSF. Thus, some deterministic analyses is performed and the obtained results are applied for training the NN; then the trained NN (i.e., Emulator) is used for interpolation. The selection of the number of inputs and their distribution in the space of probabilistic parameters are very important for the NN training process and predicting results. With increasing ground motion amplitudes, plastic deformation will occur in the buried pipeline network, as a result LSF will be more complex and NN requires more inputs for the training process.

In practice, upper and lower limits of the parameters x_1, x_2 and ... x_n should be determined and for each probabilistic parameter, some values in the range of its variation must be chosen. Then, by generating different possible combinations, many deterministic analyses should be performed. Yun and Bahng (2000) proposed the Hypercube method for determining the number of inputs for training the NN. According to their opinion, for each variable only the maximum, minimum and mean values are selected and the total number of inputs for training the NN is suggested 2^n+2n+1 that n is the number of probabilistic parameters. Thus, according to the

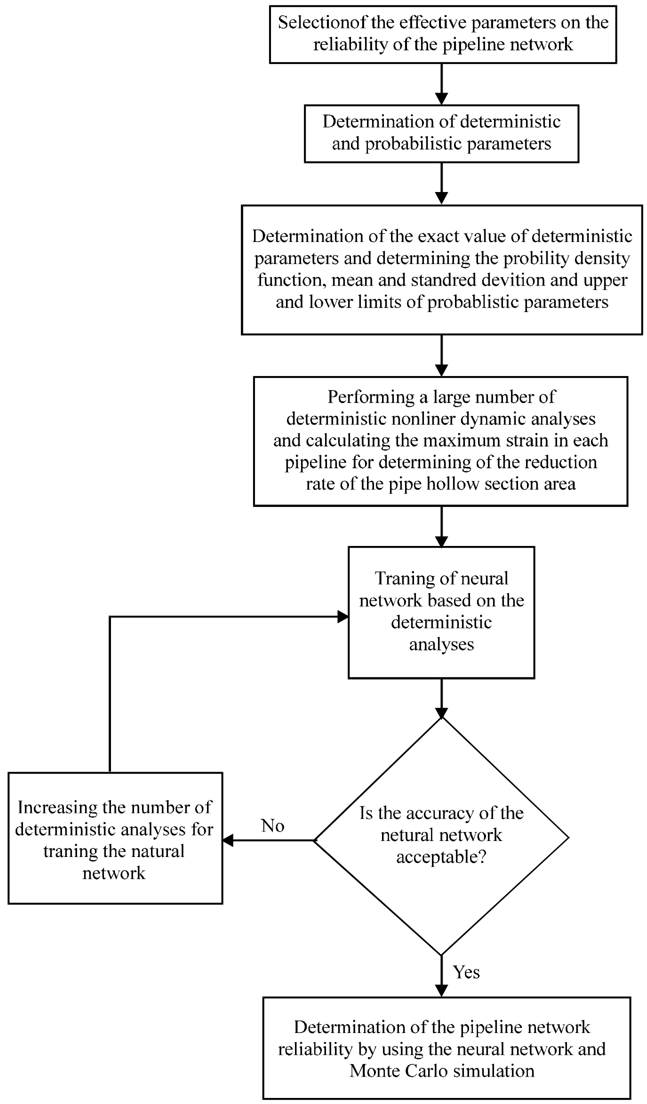

Fig. 5: Proposed algorithm for reliability assessment of buried pipeline networks under the effect of earthquake wave propagation

hypercube method, the number of inputs for training the NN is related to the number of probabilistic parameters exponentially.

In the proposed algorithm, number of input pairs for training the NN is determined based on the hypercube method. After training the NN, its accuracy will be controlled based on a series of pairs that were not used in the training process. If the network accuracy is not acceptable, by performing a new deterministic analyses and generating new input pairs, the training process of NN will be repeated. Thus, determining the number of inputs for training the NN is based on trial and error.

Determining the reliability of the pipeline network: Monte Carlo simulation is used to calculate the reliability of pipeline networks. By assuming that all the probabilistic parameters and their probability density functions, mean and standard deviation are known, a string of random numbers are generated for each parameter. Then for each combination, by using the trained NN, the maximum axial compressive strain for each pipeline is calculated. Therefore, in the case of local buckling occurring, a reduction of pipeline hollow section area can be determined. The level of the pipeline network efficiency is related to the pipelines hollow section area and after the occurrence of local buckling with some reduction in the pipes hollow section area, the network efficiency is reduced. By calculating the network efficiency level, for all the generated strings, the reliability can be obtained.

For a further detailed explanation, Fig. 6 is considered, showing a buried pipeline network. Assume that the reliability of the network for the minimum efficiency level of 80% should be found against the peak ground displacement with amplitude D. Suppose that N different combinations of probabilistic parameters have been produced and for all of these cases the efficiency level of the pipelines have been determined. The pipeline network is functioning when the outputs of nodes 1 and 2 are at least 80%. So it can be said that the network is functioning when the reduction in pipe hollow section area in all the pipes L1 to L4 is less than 20%. If the pipeline network in n cases of N combinations is functioning, the reliability of the network is equal to:

$$R = \frac{n}{N} \tag{5}$$

There are different methods for calculating the pipeline network reliability. For example by determining the reliability of nodes 1 and 2 separately, the reliability of the pipeline network will be obtained as a combination of their numerical weights, as follows:

$$R = \frac{\sum_{i=1}^{N} w_i . R_i}{\sum_{j=1}^{N} w_j} \tag{6}$$

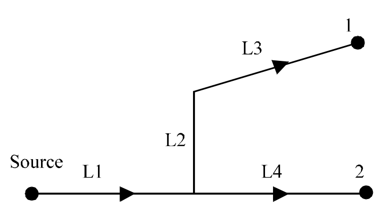

Fig. 6: The sample pipeline network for determining the reliability

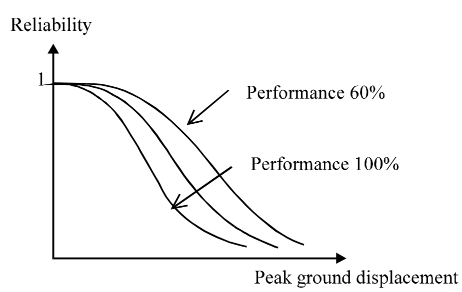

Fig. 7: The qualitative results for reliability of the buried pipeline network against earthquake wave propagation

In this equation w_i is the assigned weight and R_i is the reliability of the ith node. These calculations should be repeated for different values of D to be able to obtain the pipeline network reliability for different levels of network efficiency and the maximum earthquake displacements. As the final output of the proposed algorithm, Fig. 7 shows qualitative results for the reliability of the pipeline network against the earthquake wave propagation.

INVESTIGATION OF PROPOSED ALGORITHM IN A SAMPLE PIPELINE NETWORK

Figure 8 shows the sample pipeline network. Here, the reliability of the sample pipeline network was calculated against the maximum earthquake displacement, based on the proposed algorithm. Northridge (USA) earthquake records with variable peak ground displacements and constant frequency content were used, as shown in Fig. 9. It is considered that the pipeline network is in dense sandy soil, with a main pipe diameter of 1.0668 m and thickness 0.0127 m; with the branches having a diameter of 0.6096 m and thickness 0.00792 m. The stress-strain behavior of the steel pipe material was assumed to be of Ramberg-Osgood model with steel type X-60, with a burial depth of 1.5 m and hinged supports for all the free ends of the pipes. The axial and transverse-horizontal records of the earthquake affected are displayed in Fig. 8 with the transverse-vertical record being perpendicular to both of the axial and transverse-horizontal records. Wave propagation direction was assumed to be along the branch pipes.

The probabilistic parameters and their upper and lower limits are observed in Table 3. As shown in Table 3, five probabilistic parameters were selected and the total numbers of $2^5+2\times5+1 = 43$ different combinations were considered as the input of the NN, based on the hypercube method. Meanwhile ten random combinations were chosen for checking the accuracy of the NN after the training process, without any contributing to the training cycles (Table 4).

Table 3: Probabilistic parameters and their range of variations for the sample pipelines network

Parameters	Min	Max
$\Phi°$	25	45
γ (N m^{-3})	16000	22000
V_s (m sec^{-1})	300	900
D_v (cm)	10	140
D_h (cm)	10	140

D_v and D_h are the maximum vertical and horizontal displacements of the earthquake records, respectively. D_h is the resultant of the axial and transverse-horizontal displacements of the earthquake

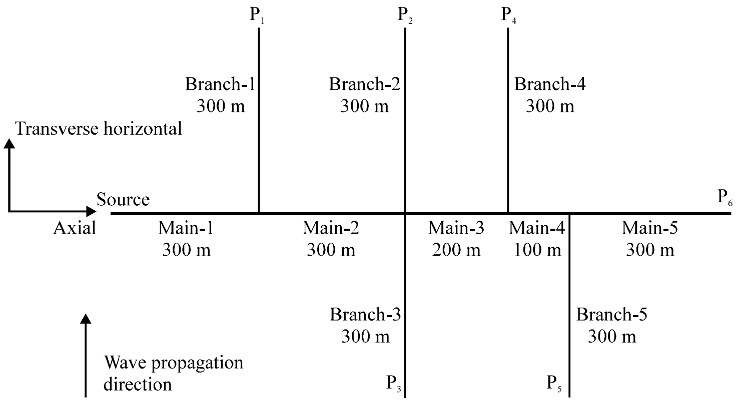

Fig. 8: Sample pipeline network for determining the reliability, based on the proposed algorithm

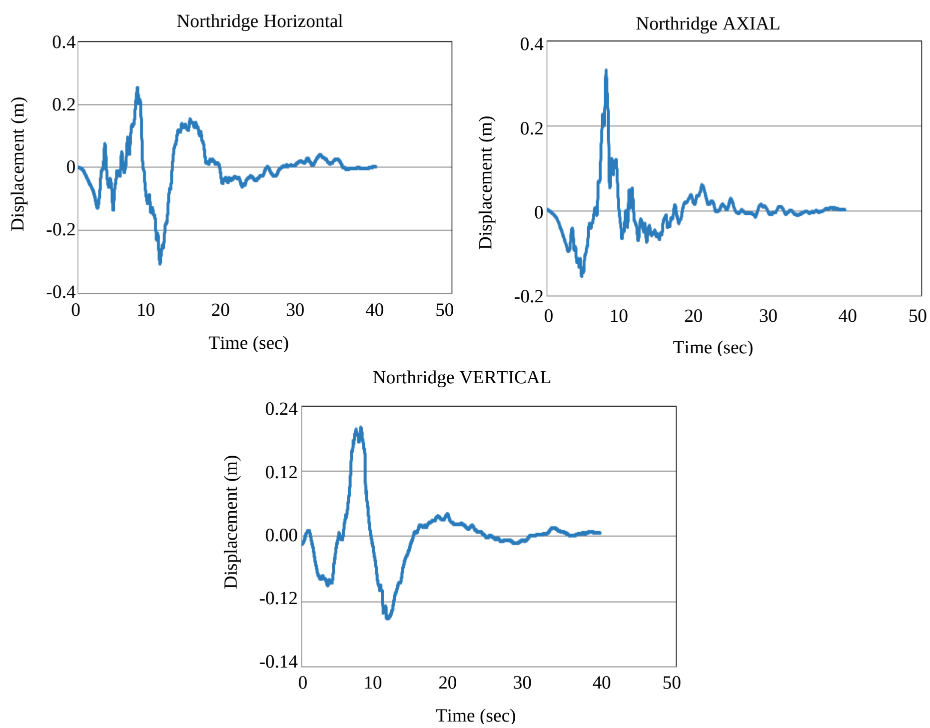

Fig. 9: Displacement records of the Northridge earthquake used in this section

Table 4: Some combinations for training the NN according to the hypercube method

No.	φ°	γ (N m^{-3})	V (m s^{-1})	D_h (cm)	D_v (cm)
1	25	16000	300	10	10
16	25	22000	900	140	140
20	45	16000	300	140	140
35	35	16000	600	75	75

For each 53 cases, nonlinear finite element dynamic analysis was performed and the maximum axial strain in each of the ten main and branch pipelines were determined; which the results for some of these combinations are presented in Table 5. It is noteworthy that the location and instance of maximum strain occurrence during earthquake, in each pipeline and in each 53 combinations were different. Therefore, the LSF of the pipeline network becomes very complex and the training of the NN becomes difficult.

The testing results of trained NN based on the 43 primary inputs were not acceptable, so 32 new

Table 5: The maximum axial strain in 10 pipelines for some of the 53 combinations based on the nonlinear finite element analysis (*1E-6)

No.	M1	M2	M3	M4	M5	B1	B2	B3	B4	B5
1	184	299	303	286	287	342	357	338	338	303
16	6150	10321	5274	5656	6532	7432	6423	11967	8284	11067
20	7408	5413	8046	6650	4556	11715	12487	8624	13768	9691
35	2360	3363	2961	3087	2835	3860	4429	2022	5095	2022

combinations of inputs were generated and the training and testing process was performed again based on 75 and 10 combinations, respectively. In this step, the accuracy of the trained NN was desirable as a substitute for the performance LSF in the sample buried pipeline network. Figure 10 presents strains from the finite element analyses in comparison with trained NN for 10 random combinations in pipelines M1 to M 5. The linear correlation coefficient which is approximately 99% can be found from Fig. 10. Therefore, the NN can be replaced by implicit LSF, confidently.

To calculate the reliability by Monte Carlo simulation, the probability density function of probabilistic parameters should be determined. Many researchers in previous works considered the soils probabilistic parameters as normal distribution (Liang *et al*., 1999; Malkawi *et al*., 2000; Al-Homoud and Tahtamoni, 2000; Heidari and Roudsari, 2009). On this basis, the probability density function of probabilistic parameters such as sand internal friction angle, soil specific weight and shear wave velocity was assumed to have normal distribution. So the mean values for probabilistic parameters based on dense sand specifications (As shown in Table 1) are selected and a standard deviation of 10% and normal distribution are assumed.

The reliability was calculated separately, for various performance levels of 100, 95, 90 and 80%. For each of these performance levels, after determining the reliability of points P1 to P 6, by using Eq. 6 with equal weights, the reliability of the pipeline network is presented. In the case of 100% performance, the occurrence of local buckling in the pipes is assumed as failure. However, in the case of 80% performance, a more than 20% reduction of the pipes' hollow section area is considered as failure. The obtained results are observed in Fig. 11.

From these graphs, it can be concluded that:

- For D_v, $D_h \leq 60$ cm the amount of strain in the pipes does not reach the amount required for the start of local buckling and local buckling does not occur at any point
- For D_v, $D_h \leq 90$ cm created damage in the network is very little and its performance is above 95%
- For D_v, $D_h \leq 110$ cm the network performance is above 90%
- For D_v, $D_h \leq 140$ cm the network performance is above 80%

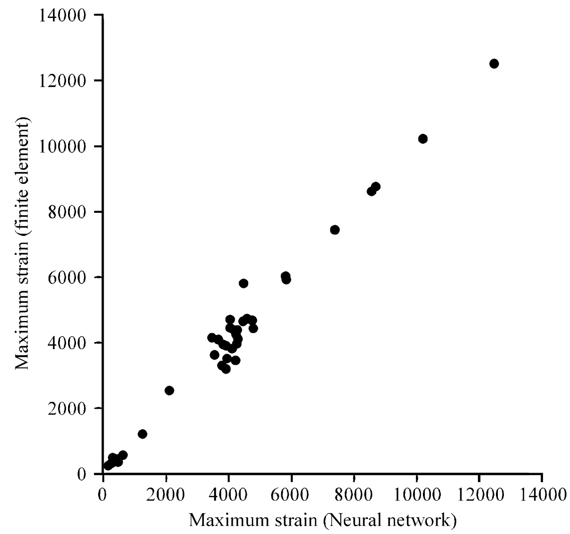

Fig. 10: Maximum strains from finite element analysis in comparison with trained NN outputs for 10 random combinations in pipelines M1 to M5 (*1E-6)

Effect of the vertical component of the earthquake displacement on the reliability of the sample pipeline network is more than horizontal component.

For determining the contribution of probabilistic parameters on the reliability of the sample pipeline network, sensitivity analysis was performed. Importance measure of a probabilistic parameter which defines as derivative of network reliability related to that parameter, represents the effect of that parameter on the network reliability (Levitin and Lisniansky, 1999). For the sample pipeline network, shown in Fig. 8, importance measure of the probabilistic parameters was determined. These probabilistic parameters include angle of the soil internal friction, soil specific weight and the shear wave velocity. In determining the importance measure, simultaneous effect of horizontal and vertical displacements with amplitude of up to 100 cm was also considered.

The obtained results for the soil internal friction angle can be observed in Fig. 12. This figure shows that the soil internal friction angle does not have an equal effect on the network's reliability. For example, by varying the friction angle from 31 to 32 degrees, the network reliability will increase. However, changing it from 35 to 36 degrees, will result in a decrease in the network reliability. The results show that the soil internal friction coefficient

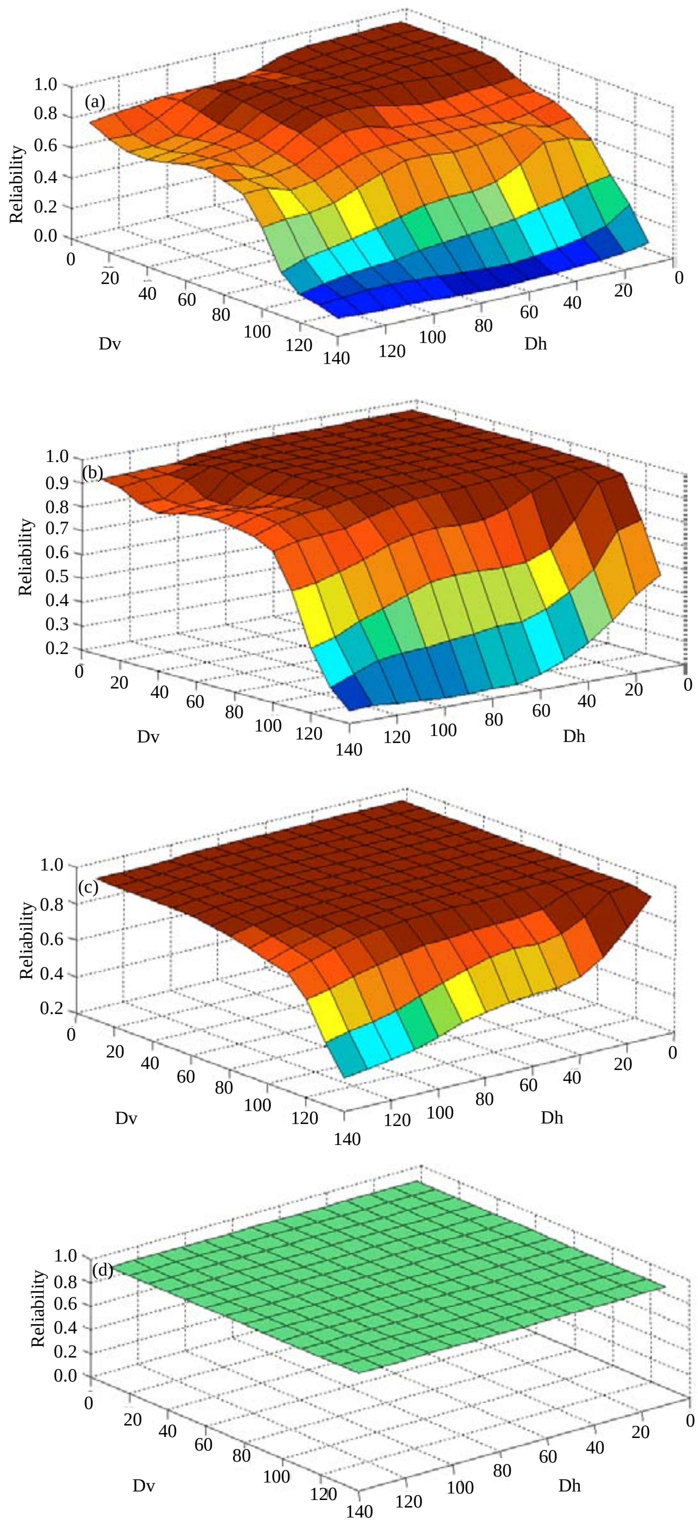

Fig. 11 (a-d): The reliability of the sample pipeline network against vertical and horizontal peak ground displacements (cm) for a: 100% performance, b: 95% performance, c: 90% performance and d: 80% performance

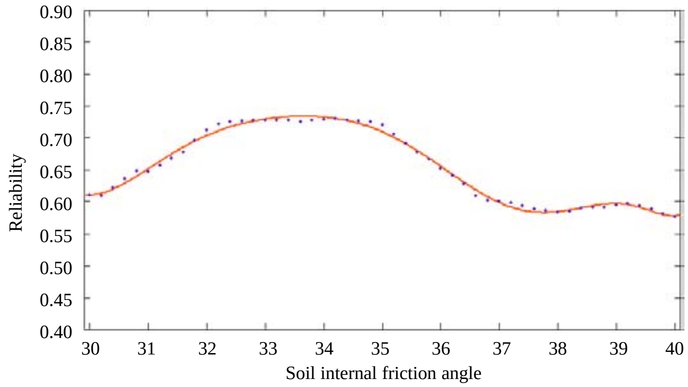

Fig. 12: Variations of the selected network reliability against the internal friction angle of soil

has a stronger effect on the network reliability compared to the soil specific weight and the shear wave velocity.

CONCLUSION

Based on the calculations it can be claimed that the occurrence of local buckling and the reduction of the pipe's hollow section area in the main and branch pipe is possible but is unlikely to happen at junctions; although few points at the junction may experience yielding. By investigating a sample network according to the proposed algorithm, its reliability against the Northridge earthquake records was calculated. It is observed that the effect of the vertical component of the earthquake on the reliability of the sample pipeline network was more than the horizontal component. For simultaneous horizontal and vertical displacements of low to medium amplitudes (around 60 cm), local buckling did not occur at any point of the pipeline. However, for larger amplitudes of about 140 cm, local buckling occurred which reduced the pipeline network efficiency to 80%. Therefore, under the effect of earthquake wave propagation with relatively large amplitudes, limited damages can occur in the steel buried pipeline network. Sensitivity analyses with respect to different probabilistic parameters showed that the soil internal friction angle had greater effect on the sample pipeline reliability, rather than the soil specific weight or the shear wave velocity.

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