SEISMIC STRESSES IN BURIED PIPING OF ARBITRARY CONFIGURATION

J. J. DEANS, J. H. K. TANG

Ontario Hydro, 700 University Avenue, Toronto, Ontario M5G 1X6, Canada

The design of new nuclear power plants often requires an accurate evaluation of the stresses induced in underground piping due to a postulated earthquake. Except for very large and rigid tunnels, the near field soil structure interaction rarely is predominant. As a matter of fact, for most pipes and conduits, such as electrical cable ducts, coolant water pipe, generally the soil can be considered stiff compared to the conduit because they are embedded in sufficiently compacted backfill so that the soil-pipe interaction effects would be negligible. Therefore, during an earthquake such a buried structure could be assumed offering no restriction to soil motion. Sakurai and Takahashi (1969) report field observations on the dynamic behaviour of pipe lines during the Matsushiro Earthquakes that confirm the conclusions of this treatment. It has been found that the observed stresses of pipes are related to the observed deformation of the ground and the wave characteristics of the ground. On that basis, the underground piping can be analysed by imposing on it the maximum axial and shear strains and curvatures in soil. A long straight portion of an element, or elements with 90° bends and T-junctions with long sections between bends can be analysed quite easily by expressions developed by Shah and Chu (1974). However, an underground net of seismically classified piping in practice has various configuration which includes bends at arbitrary angles and short elements between bends or junctions. In this case, the computation by the convention method is laborious especially when an iteration process has to be employed to obtain the compatible relative movement between the element and soil. This paper presents a finite element approach to this problem. Thus, the stiffness matrix for a pipe on elastic foundation is developed. The induced loads are calculated based on the imposed free field soil strain or building movements. The seismic strain due to various types of seismic waves and arbitrary angle of incidence are also considered.

In addition, it is recognized that although the analytical techniques have been developed to a high level of sophistication the basic soil parameters such as the "subgrade reactions" and frictional resistance are not very well known. There is a wide spread of values reported in literatures Therefore, it is recommended that a parametric study should always be carried out to investigate the effects of variations in these values with respect to the stresses resulting from the seismic ground motion since this can easily be done by use of the computer program developed.

1. Introduction

Various safety systems in a nuclear power plant are partly comprised of pipes or conduits which are completely buried underground and which interface with auxiliary buildings, pump houses or other structures. Detailed seismic analysis of such piping to demonstrate its structural capability as required by the regulatory agency is essential for licensing. Buried pipes and concrete conduits, such as electrical cable ducts, emergency water supply and collant water pipes are generally embedded in compacted backfill with a sufficient density so that the backfill does not lose its integrity due to liquefaction during seismic disturbances. Except for very large and rigid tunnels, the near field soil structure interaction rarely is predominant. In most cases, the soil can be considered stiff compared to the pipes and conduits. This means that the earthquake deformation of the soil is imposed on the pipes and conduits, which must conform to this deformation. Sakurai and Takahashi (1969) report field observations on the dynamic behaviour of pipe lines during the Matsushiro Earthquakes that confirm the conclusion's of this treatment. It has been found that the observed stresses of pipes are related to the observed deformation of the ground and the wave characteristics of the ground. On that basis, the underground piping can be analysed by imposing on it the maximum axial and shear strains and curvatures in soil. The analytical determination of the strains due to seismic wave propagation in long buried conduits, bent at an angle of 90°, or T-junction has been developed by Shah and Chu (1974). However, a rigorous analysis of buried piping taking into account various configuration of bends and elbows is difficult to accomplish. In addition, seismically induced stresses on non-idealized layout by various waves types from oblique directions cannot be determined easily. This paper presents a finite element approach to this problem. Since the relative displacement between the soil and the pipe is governed by frictional force, the maximum axial force in the pipe and thus deformation are computed by an iteration scheme which can be incorporated into a computer program.

2. Earthquake Action

The fundamental assumption governing the method of analysis of buried piping is that the soil can be considered stiff relative to the pipe. This implies that the soil deformation is imposed on the pipe. Therefore, it is first necessary to evaluate the soil strain during an earthquake. Where a wave motion is propagated in one direction without interference with other waves in other directions, and where the change in shape of the wave from point to point is relatively small, the maximum soil strain can be determined by a fairly simple expression discussed by Newmark (1971):

$$\epsilon_{max} = \pm V_{max}/C \qquad (1)$$

where V_{max} is the maximum ground velocity and C is the velocity of the earthquake wave propagation. Similarly, the maximum curvature is determined as follows:

$$\text{curvature} = a_{max}/C^2 \qquad (2)$$

where a_{max} is the maximum ground acceleration.

The maximum axial force induced on the pipe is the product of the soil strain, the Young's modulus and the area of cross-section of the pipe. The maximum bending moment is the product of the curvature and flexural rigidity of the pipe. However, if the resultant axial force exceeds the frictional resistance, slippage will occur at the soil-pipe interface and the deformation of the soil is no longer imposed on the pipe. At pipe junctions or elbow the resultant axial forces and the effective slippage lengths are dependent unknowns and are usually evaluated by trial and error.

3. Method of Analysis

When transverse and rotational displacements occur, the buried structure can be assumed to be a semi-infinite element supported on an elastic foundation (see Figure 1). According to the theory of beams on elastic foundation, the basic differential equation of equilibrium is given by: (see Figure 2).

$$EI \frac{d^4y}{dx^4} = -k\, y \qquad (3)$$

where y is the deflection of the beam, k is the foundation spring constant, E is the modulus of elasticity and I is the moment of inertia.

The homogeneous solution of eq. (3) is

$$y = C_1 e^{\lambda x}\text{Cos}\lambda x + C_2\, e^{\lambda x}\text{Sin}\,\lambda x + C_3 e^{-\lambda x}\text{Cos}\,\lambda x + C_4 e^{-\lambda x}\,\text{Sin}\,\lambda x \qquad (4)$$

where $\lambda = \left[k/4EI\right]^{\frac{1}{4}}$

By differentiating eq. (4), expressions for the shear and moment as a function of x can be obtained as

$$M(x) = -2\lambda^2\, EI\left\{- e^{\lambda x}\, C_1\,\text{Sin}\lambda x + e^{\lambda x}C_2\,\text{Cos}\lambda x + e^{-\lambda x}C_3\text{Sin}\lambda x - C_4 e^{-\lambda x}\text{Cos}\lambda x\right\} \qquad (5)$$

$$Q(x) = -2\lambda^3\;EI\left\{- e^{\lambda x}\left[C_1(\text{Cos}\,\lambda x + \text{Sin}\lambda x) - C_2(\text{Cos}\,\lambda x - \text{Sin}\,\lambda x)\right] + e^{-\lambda x}\left[C_3(\text{Cos}\,\lambda x - \text{Sin}\lambda x) + C_4(\text{Cos}\,\lambda x + \text{Sin}\,\lambda x)\right]\right\} \qquad (6)$$

The member stiffnesses for the restrained member shown in Figure 3 are the actions exerted on the member by the restraints when unit displacements (transverse translation and rotations) are imposed at each end of the member. The unit displacements are considered to be induced one at a time while all other end displacements are retained at zero. The values of these restrained actions may then be obtained after the four coeffecients C_1, C_2, C_3 and C_4 are determined in each case. For example with the four boundary conditions as (1) $x=0$, $y=1$ (2) $x=0$, $\frac{dy}{dx}=0$ (3) $x=L$, $y=0$ and (4) $x=L$, $\frac{dy}{dx}=0$ and by substituting into eq. (4) and its derivative the following equations are obtained:

$$C_1 + C_3 = 1 \quad (7a)$$

$$\lambda C_1 + \lambda C_2 - \lambda C_3 + \lambda C_4 = 0 \quad (7b)$$

$$e^{\lambda L}\cos\lambda L C_1 + e^{\lambda L}\sin\lambda L C_2 + e^{-\lambda L}\cos\lambda L\, C_3 + e^{-\lambda L}\sin\lambda L C_4 = 0 \quad (7c)$$

$$\lambda e^{\lambda L}(\cos\lambda L - \sin\lambda L)\, C_1 + \lambda e^{\lambda L}(\cos\lambda L + \sin\lambda L)\, C_2 - \lambda e^{-\lambda L}(\cos\lambda L + \sin\lambda L) C_3 + \lambda e^{-\lambda L}(\cos\lambda L - \sin\lambda L)\, C_4 = 0 \quad (7d)$$

The four coefficients can then be determined. Repeating with other sets of boundary conditions by replacing the right hand side of eq. (7) with vectors $\{0,1,0,0\}$, $\{0,0,1,0\}$ and $\{0,0,0,1\}$ respectively, other sets of four coefficients are computed.

When axial displacement occurs, the frictional force provided by the soil comes into effect and the stiffnesses are computed by AE/Li and AE/Lj where Li and Lj are the slippage lengths (see Figure 4).

Finally, the element stiffness matrix is given by

$$\begin{bmatrix} AE/Li & 0 & 0 & 0 & 0 & 0 \\ 0 & -Q(0)_2 & -Q(0)_3 & 0 & -Q(0)_5 & -Q(0)_6 \\ 0 & M(0)_2 & M(0)_3 & 0 & M(0)_5 & M(0)_6 \\ 0 & 0 & 0 & AE/Lj & 0 & 0 \\ 0 & Q(L)_2 & Q(L)_3 & 0 & Q(L)_5 & Q(L)_6 \\ 0 & -M(L)_2 & -M(L)_3 & 0 & -M(L)_5 & -M(L)_6 \end{bmatrix} \quad (8)$$

The equivalent joint loads $\{A_E\}$ is computed by evaluating the member end-actions due to the imposed soil strains corresponding to wave propagation direction as discussed by Yeh [4] (see Figure 5 and 6) and the frictional force is applied as joint loads $\{A\}$. The unknown joint displacement $\{D\}$ can then be solved

$$\{D\} = \{S\}^{-1} (\{A_E\} - \{A\}) \quad (9)$$

Since $\left\{ A \right\}$ is dependent on the member end-actions which are in term dependent on the joint displacements, eq. (9) can only be solved by iteration.

4. Example

A part of the emergency water supply system as shown in Figure 7 is to be analysed for stresses resulting from earthquake to demonstrate the use of this method. The stresses of the 20" dia. pipe at C is obtained as follows:

Wave direction N-S	$\sigma_c =$	4,100 psi
Wave direction E-W	$\sigma_c =$	2,100 psi
Wave direction N45°E	$\sigma_c =$	2,600 psi

5. Conclusion

This paper has presented a technique to analyse underground piping. The versatility of this method can easily be demonstrated by applying the technique to complex geometry of bends and elbows. Non-uniform soil parameters and suitable combination of various waves effects can also be incorporated if necessary.

However, it is important to recognize that although the analytical techniques have been developed to a high level of sophistication, the basic soil parameters such as the "subgrade reactions" and frictional resistance are not very well known. There is a wide spread of values reported in literatures. Therefore, it is recommended that a parametric study should always be carried out to investigate the effects of variations in these values with respect to the stresses resulting from the seismic ground motion since this can easily be done by use of the computer program developed.

REFERENCES

1 Newmark, N.W., "Earthquake Response Analysis of Reactor Structures", Proceedings of First International Conference on Structural Mechanics in Reactor Technology, Berlin, Germany, September, 1971.

2 Kuesel, T.R., "Earthquake Design Criteria for Subways," Journal of Structural Division, ASCE, Proc. Vol. 95, No. ST6, June, 1969.

3 Sakuri A., and Takahashi, T., "Dynamic Stresses of Underground Pipe Lines during Earthquakes", Proc. Fourth World Conference Earthquake Engineering, Santiago, Chile, B-4, 1969.

4 Yeh, G.C.K., "Seismic Analysis of Slender Buried Beams," Bulletin of Seismological Society of America, Vol. 64, Issue 5, October 1974.

5 Shah, H.H., and Chu, S.L., "Seismic Analysis of Underground Structural Elements," Journal of the Power Division, ASCE, Proc. Vol. 100, No. PO1, July, 1974.

6 Igbal, M.A., and Goodling, E.C., "Seismic Design of Buried Piping," Second ASCE Speciality Conference on Structural Design of Nuclear Plant Facilities at New Orleans, Louisiana, December 8 - 10, 1975.

7 Hetenyi, M., "Beams on Elastic Foundation," The University of Michigan Press, Ann Arbor, Michigan, 1946.

8 Ewing, W.M., Jardetzky, W.S., and Press, F., "Elastic Waves in Layered Media," McGraw-Hill Book Co., Inc., N.Y. 1957.

9 Gere, J.M. and Weaver, W.", Analysis of Framed Structures", Van Nostrand Reinhold Company, N.Y. 1965.

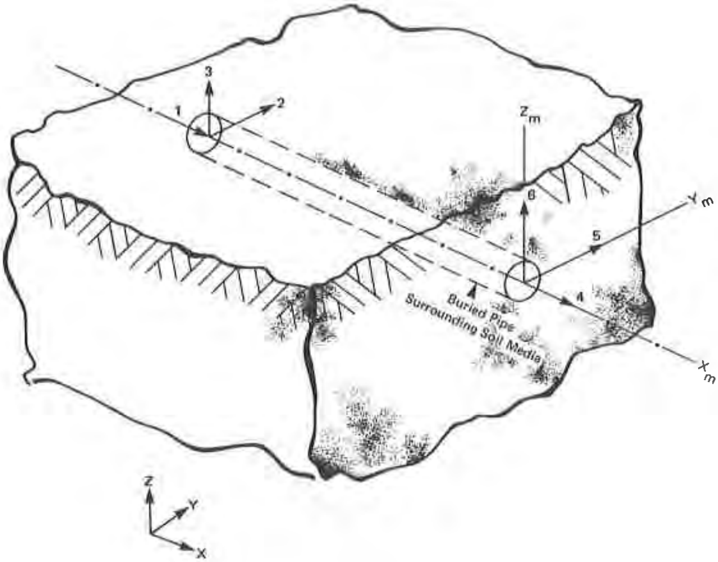

Figure 1
Typical Section of Buried Pipe

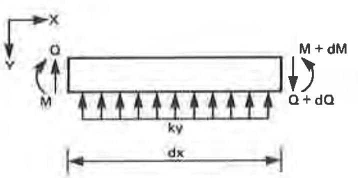

Figure 2
Forces on Element

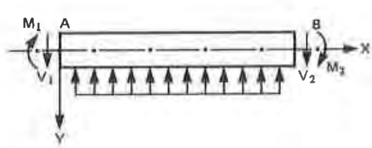

Figure 3
Idealized Element

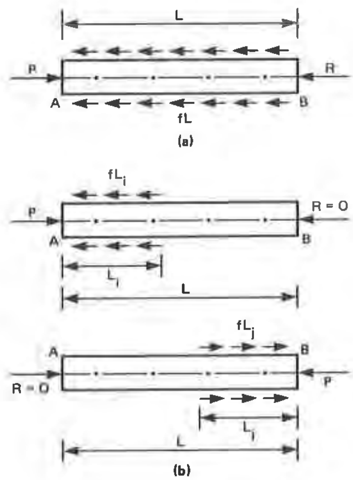

Figure 4
Axial Slippage Length

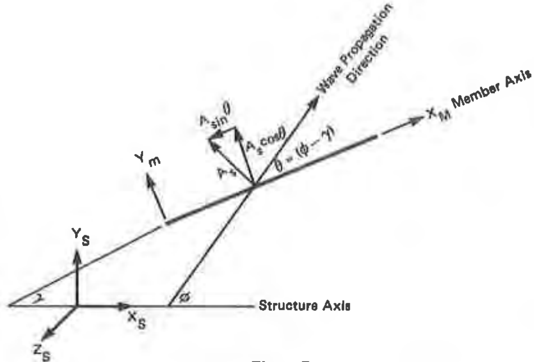

Figure 5
Direction of Shear Wave
Propagation Relative to Pipe

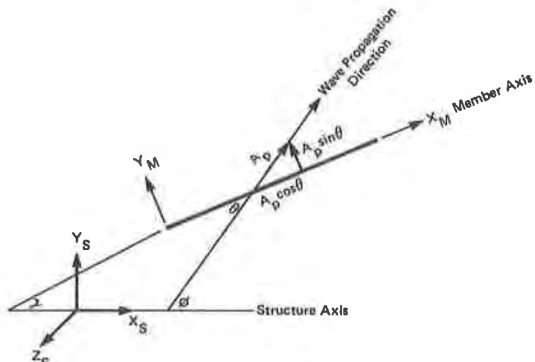

Figure 6
Direction of Compression Wave
Propagation Relative to Pipe

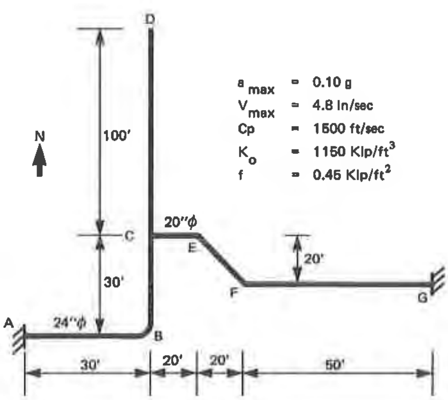

Figure 7
Example Buried Pipe