50272-101

REPORT DOCUMENTATION PAGE	1. REPORT NO. NSF/RA-780340	2.	3. Recipient's Accession No. PB291412

4. Title and Subtitle
Earthquake Response of Buried Pipelines (Seismic Vulnerability Behavior and Design of Underground Piping Systems, Technical Report No. 4)

5. Report Date
March 1978

6.

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8. Performing Organization Rept. No.
4

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10. Project/Task/Work Unit No.

11. Contract(C) or Grant(G) No.
(C)
(G) ENV7614884

12. Sponsoring Organization Name and Address
Applied Science and Research Applications (ASRA)
National Science Foundation
1800 G Street, N.W.
Washington, DC 20550

13. Type of Report & Period Covered
Technical

14.

15. Supplementary Notes

16. Abstract (Limit: 200 words)

Overall aims of this research are to develop a systematic way of assessing the adequacy and vulnerability of water/sewer distribution systems subjected to seismic loads and also to develop future design methodologies for water/sewer systems. Because of the geographical extent of buried pipelines, analysis and design procedures for buried pipelines are quite different than the standard procedures developed for building type structures. The purpose of this paper is to investigate the following assumptions which form the basis for the presently available seismic design procedures for buried pipelines subjected to ground shaking. (1) It is assumed that there is no relative motion between the pipe and the soil; and (2) It is assumed that the shape of the seismic waves does not change as it traverses the pipeline.

17. Document Analysis a. Descriptors

Design	Water pipes	Piping systems
Earthquakes	Water supply	Subsurface structures
Earthquake resistant structures	Sewer pipes	
Dynamic structural analysis	Hazards	

b. Identifiers/Open-Ended Terms

Earthquake engineering
Buried pipelines

c. COSATI Field/Group

18. Availability Statement	19. Security Class (This Report)	21. No. of Pages
NTIS.	20. Security Class (This Page)	22. Price

(See ANSI-Z39.18) See Instructions on Reverse OPTIONAL FORM 272 (4-77) (Formerly NTIS-35) Department of Commerce

Seismic Vulnerability, Behavior and Design of Underground Piping Systems

Earthquake Response of Buried Pipelines

by

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Sponsored by National Science Foundation

Research Applied to National Needs (RANN)

Grant No. ENV76-14884

Technical Report (SVBDUPS Project) No. 4

March 1978

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ACKNOWLEDGEMENT

This is fourth in a series of reports under the general title of 'Seismic Vulnerability, Behavior and Design of Underground Piping Systems' (SVBDUPS). The report will be presented at the ASCE Geotechnical Division Specialty Conference on Earthquake Engineering and Soil Dynamics June 19-21, 1978, Pasadena, California.

The research has been sponsored by the Earthquake Engineering Program of NSF-RANN under Grant No. ENV76-14884 and Dr. S.C. Liu is the Program Manager of this project. The overall aims of the research are to develop a systematic way of assessing the adequacy and vulnerability of Water/Sewer distributions subjected to seismic loads and to develop future design methodologies.

This report is written by Dr. Michael O'Rourke, Assistant Professor of Civil Engineering and Dr. Leon R.L. Wang, Associate Professor of Civil Engineering. Dr. Leon R.L. Wang is the Principal Investigator of the project.

Appreciation goes to the Advisory Panel which consists of Mr. Holly A. Cornell, President of CH2M Hill, Inc., Corvallis, Oregon; Mr. Warren T. Lavery, Superintendent of Latham Water District, Latham, NY; Dr. Richard Parmelee, Professor of Civil Engineering, Northwestern University, and Drs. Jose Roesset and Robert Whitman, Professors of Civil Engineering, MIT for their helpful comments and suggestions.

The typing and proof-reading of this report by Mrs. Jo Ann Grega are also appreciated.

Please note that although our project is sponsored by the National Science Foundation, any opinions, findings and conclusions or recommendations expressed in this publication are those of the authors and do not necessarily reflect the veiws of NSF.

Earthquake Response of Buried Pipeline

by Michael O'Rourke[1], A.M. and Leon R.L. Wang[2], M.

ABSTRACT

The earthquake response of buried water and sewer lines is receiving attention because of the impact of these lifelines upon the health and safety of the people served by these systems. Because of the geographical extent of buried pipelines, analysis and design procedures for buried pipelines are quite different than the standard procedures developed for building type structures. Seismic design procedures for buried pipelines are based upon two assumptions dealing with the relative motion between the pipe and the soil and also with the character of the seismic waves. Specifically, it is assumed that there is no relative motion between the pipe and the soil and that the shape of the seismic waves does not change as it traverses the pipeline. The purpose of this paper is to investigate these assumptions which form the basis for the presently available seismic design procedures for buried pipelines subjected to ground shaking.

INTRODUCTION

Lifeline earthquake engineering is becoming a major concern of the engineering profession because of the importance of lifelines vis a vis the health and safety of the populus during and after an earthquake. Transportation and communication facilities as well as buried water and sewer lines are examples of such lifeline systems. Buried water lines, which are the subject of this paper, impact the health and safety of the population through the possible contamination of water supply and/or reduction of the fire fighting capabilities after an earthquake. The majority of the 700 deaths and $400,000,000 property damage in the 1906 San Francisco earthquake are attributed to fire which went unchecked because all but one of the water mains were severed during the quake. Similarly, 35% of the city of Tokyo was destroyed by fire after the 1923 earthquake. Although such catastrophic fires have not occurred after recent earthquakes, outbreaks of typhoid due to contaminated water were reported after the 1976 Guatemala Earthquake.

There are three major causes for the failure of buried pipeline due to seismic excitation; general failure of the pipeline and soil in areas of poor soil properties due to landsliding or soil liquefaction, shear type failure where a pipeline crosses a fault, and axial or flexural failure of the pipeline due to ground shaking.

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Newmark and Hall (14) have developed procedures for pipeline design to resist large fault displacement. This procedure involves insuring that the maximum pipe strain is less than the rupture strain. Other fail-safe procedures (4,6) have also been proposed for a pipeline crossing a fault. A summary of the fail-safe procedures can be found in a recent paper (21) by the authors.

Procedures also exist for analysis and design of pipelines for ground shaking or wave propagation effects. The purpose of this paper is to investigate in detail the assumption underlying these procedures for the effects of ground shaking on buried pipeline.

PIPELINE RESPONSE TO SHAKING

The analysis of the response of pipelines, which by their nature have large geographical extent, is different than the response analysis for a building structure. Since the width of most buildings is very small compared to the wavelength of seismic waves, the building is subjected to essentially point excitation. That is, the three components of ground motion at one edge of the building are essentially the same as the three components of ground motion at the other edge of the building. Hence, either set of acceleration time histories can be assumed to represent the motion of the foundation during a particular earthquake and standard response spectrum techniques may be applied. For pipelines, this assumption is not valid. Consider two points on a pipeline, A and B, at different distances from the earthquake epicenter. The ground motion under these points will not be the same. First of all there will be some change in the wave shape due to variations in the soil properties along the pipeline and, to a lesser extent, changes in the wave shape due to attentuation. Secondly, the two ground motions will be out of phase due to the different seismic wave travel times from the epicenter. This phase difference is a function of the velocity of the seismic waves as well as the difference in epicentural distance. Even though pipelines have not received the detailed attention which building type structures have received, procedures exist (1,8,11,13,15,19,22) for the analysis and design of underground pipes for seismic shaking or wave propagation effects. Two basic assumptions underlie these procedures. The first assumption deals with the seismic waves impinging on the pipeline. It is assumed that the wave shape remains constant while traversing the pipeline. That is, it is assumed that the acceleration, velocity and displacement time histories of the ground motions at points A and B are the same except for a time lag. As mentioned previously, this time lag is a function of the wave velocity and the separation between the points.

The second assumption deals with the dynamic response of the pipeline to traveling seismic waves. Any pipeline motion can be decomposed into two parts; the motion of the soil and the motion of the pipeline with respect to the soil. In the presently available procedures, it is assumed that the motion of the pipeline with respect to the soil is negligible. That is, it is assumed that the soil is much stiffer than the pipeline and the soils displaced configuration, at any point in time, is imposed upon the pipeline. Hence, the design of the pipeline involves supplying sufficient flexibility or ductility so that the pipeline can follow the soil.

Combining these two assumptions, the motion of both the pipeline and the soil, v(x,t) becomes

$$v(x,t) = f(x-ct) \quad (1)$$

where x is the space coordinate, c is the velocity of this particular wave propagation and t is time. If the direction of the ground motion is along the axis of the pipeline, the maximum axial strain, ε_{max}, in both the pipe and the soil becomes

$$\varepsilon_{max} = V_{max}/c_p \quad (2)$$

where V_{max} is the maximum ground velocity and c_p is the velocity of the pressure waves with respect to the pipeline. If the direction of the ground motion is perpendicular to the pipe, the maximum curvature in the pipe becomes

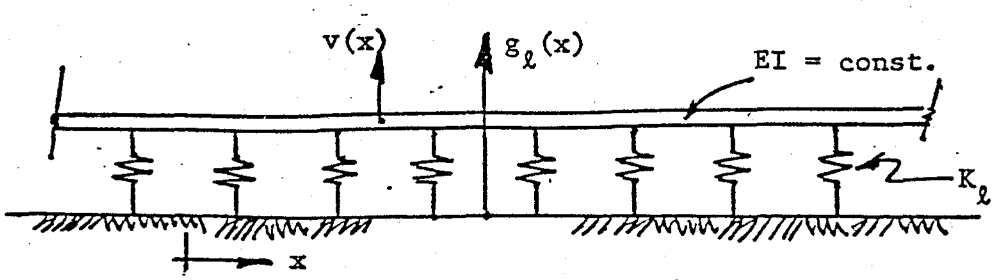
$$(\text{curvature})_{max} = A_{max}/c_s^2 \quad (3)$$

where A_{max} is the maximum ground acceleration and c_s is the velocity of the shear waves with respect to the pipeline. The relationships shown in Eqs. 2 and 3 have been presented and/or used by a number of authors (1,8,11,13,15,19,22). In some cases, the problem of a pipeline at an oblique angle to the direction of propagation has also been addressed. As stated previously, the purpose of this paper is to investigate the assumptions which underlie these relationships.

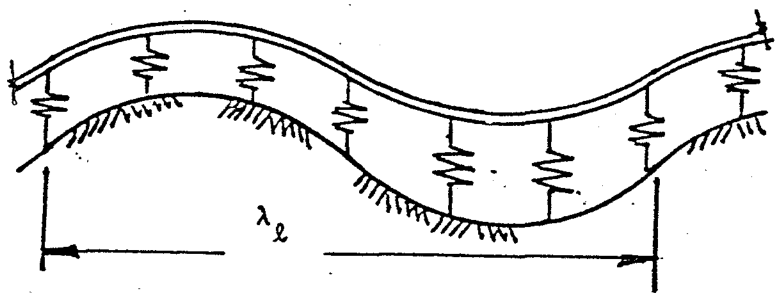
RELATIVE MOTION BETWEEN PIPE AND SOIL

The assumption regarding the negligible relative displacement between the pipe and the soil can be investigated experimentally as well as analytically. A number of Japanese researchers (9,12,18) have experimentally measured the relative motion between pipelines and soil. The field investigation of the seismic response of buried pipelines by Sakorai and Takahasi (18) is typical. Circular pipe having diameters of 267 mm and 89 mm and weighing 42.4 kilometer per meter and 8.7 kilograms per meter respectively were instrumented with strain gauges while soil strain gauges were used for the soil motion. Sakurai and Takahasi note that there was no observable difference between the recorded ground and pipeline deformation during earthquakes. Whether this observation holds for both the axial and lateral motion of all sizes of pipe in various soils may be determined through the use of a static analytical model.

Consider first the case of lateral motion of the pipeline in which the pipe-soil system is modeled as a beam on elastic foundation. The relative lateral displacement between the pipe and the soil is a function of the flexural stiffness of the pipe and the lateral stiffness of the soil. This relative displacement may be gauged by giving the base of the soil springs a sinusoidal displaced configuration as shown in Figure 1.



a – initial configuration



b – displaced configuration

Fig. 1 Lateral Displacement Model

In Fig. 1, $v(x)$ is the lateral displacement of the pipeline, $g_\ell(x)$ is the lateral ground displacement

$$g_\ell(x) = A \sin (2\pi x/\lambda_\ell) \tag{4}$$

K_ℓ is the stiffness of the lateral soil springs and λ_ℓ is the wave length of the ground displacement. The static displacement $v(x)$ of the pipeline is the solution to the fourth order linear differential equation

$$\frac{d^4 v}{dx^4} + 4\,\alpha^4\,v = 4\,\alpha^4\,A \sin (\beta_\ell\,x) \tag{5}$$

where

$$\alpha^4 = K_\ell/4\,EI$$

$$\beta_\ell = 2\pi/\lambda_\ell$$

After satisfying the boundary conditions that the pipeline displacement is finite at infinity, the pipeline displacement becomes

$$v(x) = \gamma_1\,A\,[\sin(\beta_\ell x)] \tag{6}$$

where

$$\gamma_1 = 4\,\alpha^4/(\beta_\ell^4 + 4\,\alpha^4)$$

Now the relative displacement between the pipe and soil can be quantified by a ratio R_ℓ of the pipe displacement to the soil displacement

$$R_\ell = \frac{v(x)}{g_\ell(x)} = \frac{d^2v/dx^2}{d^2g_\ell/dx^2} = \frac{4\alpha^4}{\beta_\ell^4+4\alpha^4}$$

If R_ℓ is exactly equal to one, there is no relative lateral movement between the pipe and the soil. If, on the other hand, the ratio R_ℓ is much lower than one, the assumption that there is no relative lateral motion between the pipe and soil is inaccurate. It should also be noted that R_ℓ is also a ratio of pipe and soil curvatures.

In order to determine a lower bound for the ratio R_ℓ, an upper bound for β_ℓ and a lower b ound for α should be used. A lower bound for α corresponds to the case of a stiff pipe in a weak soil. For the purposes of this analysis, a 36 in. diameter concrete pipe is assumed for which the flexural rigidity (EI) is 253×10^6 K in^2. The soil is assumed to have a shear wave velocity, Vs, of 500 fps. The equivalent soil spring stiffness, K_ℓ, for the lateral case is an average of values developed by Parmelee (17) and Blaney (2,3).

$$K_\ell = 3.0\ G_{eff} = 0.30\ G$$

where G_{eff} is the effective shear modulus at high strain, taken as one tenth of the shear modulus, G, at small strain. This yields a value for K_ℓ of 1630 psi.

A reasonable upper bound for β_ℓ or conversely a reasonable lower bound for λ_ℓ can be estimated indirectly from displacement time histories. If τ is the predominate period of the displacement time history records, then the wave length of the lateral ground displacement λ_ℓ becomes

$$\lambda_\ell = V_s \cdot \tau$$

where V_s is the shear wave velocity. For typical earthquake displacement time histories, τ ranges in value from 2.0 to 6.0 sec. Using a value of 2.0 sec for τ and, again, 500 fps for the shear wave velocity, V_s, yields

$$\lambda_s = 12000 \text{ in.}$$

For this worst case analysis of the pipe-soil system (i.e., $\lambda_\ell = 12000$ in, $K_\ell = 1.63$ ksi and $EI = 253 \times 10^6$ K - in^2) the value of R_ℓ becomes

$$R_\ell = 0.99998$$

This value of R_ℓ is very close to one. Hence, for this static analysis the relative lateral motion between the pipe and soil is negligible.

However, for this static result to be applicable to seismic excitation, the natural period of the pipe-soil system must be much smaller than the expected earthquake periods. Neglecting the flexural stiffness of the pipe, an upper bound for the lateral natural period of the pipe becomes

$$\tau_\ell = 2\pi\sqrt{M/K_\ell}$$

where M is the mass per unit length of the pipe. For the pipe and soil parameters used previously

$$\tau_\ell = 0.04 \text{ sec.}$$

This very low natural period falls in the region on earthquake response spectra where the maximum relative displacement are very low and the maximum pseudo acceleration is the same as the maximum ground acceleration. Hence, for the case of lateral motion, the assumption that there is no relative displacement between the pipe and the soil appears justified.

A similar analysis can be performed for the axial motion of the pipe. Consider the pipe-soil system shown below in Fig. 2 in which the soil is given a longitudinal sinusoidal displacement

$$g_a(x) = A \sin(2\pi x/\lambda_a)$$

where λ_a is the wavelength of the ground displacements along the pipe axis.

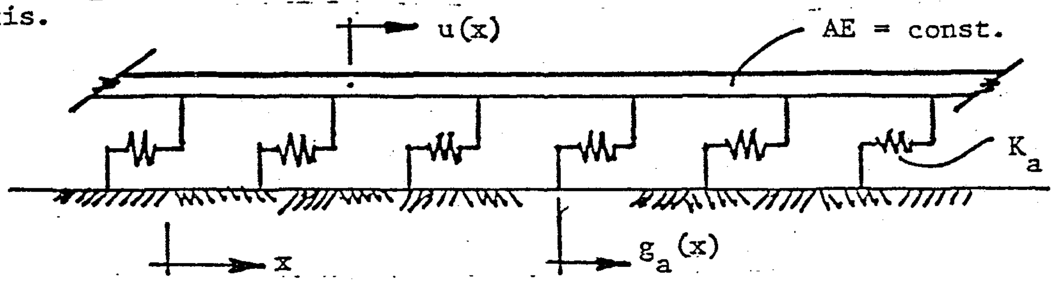


Fig. 2 Axial Displacement Model

The axial displacement of the pipe, u(x), is the solution to a second order linear differential equation

$$\frac{d^2u}{dx^2} - \rho^2 u = -\rho^2 A \sin(\beta_a x) \tag{7}$$

where

$$\rho = \sqrt{K_a/EA}$$

$$\beta_a = 2\pi/\lambda_a$$

In Eqn. (7) AE is the axial rigidity of the pipe while K_a is the soil stiffness resisting axial motion of the pipe. After insuring that the pipe displacement and strain are finite at infinity, the axial displacement of the pipe becomes

$$u(x) = \frac{\rho^2 A}{\beta_a^2 + \rho^2} [\sin(\beta_a x)]$$

The maximum relative axial displacement between the pipe and soil can be quantified by a ratio R_a

$$R_a = \frac{u(x)}{g_a(x)} = \frac{\rho^2}{\beta_a^2 + \rho^2}$$

If R_a is close to one, there is little relative axial motion between the pipe and the soil. If R_a is not close to one, this standard assumption is not applicable for axial motion. Again, as with the lateral motion case, an upper bound for β_a will be used along with a lower bound for ρ. Since pressure waves travel approximately 1.7 times faster, depending upon Poisson's Ratio, than shear waves, the previously used value of 12000 inches will be increased to 21000 inches for λ_a. For the 36 inch diameter concrete pipe, the axial rigidity, EA, is 12.8×10^8 lbs. Novak and Beredugo's results (16) and unpublished results from a dynamic finite element program for a pile in soil were used to quantify the axial soil stiffness, K_a.

$$K_a = 2\, G_{eff} = \frac{2G}{10}$$

Using a value of 500 fps for V_s, the corresponding value of R_a becomes

$$R_a = 0.967$$

Again R_a is very close to one for this static analysis. Also, the natural period for axial motion of the soil-pipe system, neglecting the axial stiffness of the pipe is

$$\tau_a = 2\pi \sqrt{M/K_a} = 0.03 \text{ sec}$$

This natural period is again in the range on response spectra where the maximum relative displacements are very small.

WAVE SHAPE

The accuracy of the assumption that the wave shape remains constant while traversing the pipeline can be guaged by a comparison of actual ground motion recorded at a number of different locations in the same vicinity. Crouse (5) has compared recorded motions at nearby locations. Working in the frequency domain, Crouse noted the good agreement between the Fourier Amplitude Spectrum of nearby seismograms in the low frequency range of 0.1 to 1.0 cps. Working in the time domain Hanks (7) has compared displacement time histories recorded during the 1971 San Fernando earthquake. The displacement time histories were obtained from double integration of the strong motion accelerograms using base line adjustments and filtering techniques. Hanks studied four arrays of recording devices. Shown in Figure 3 (Figure 2 in Hanks paper) is one such array consisting of seven recording devices all located within 1 kilometer of the intersection of Figueroa Street and Olympia Boulevard.

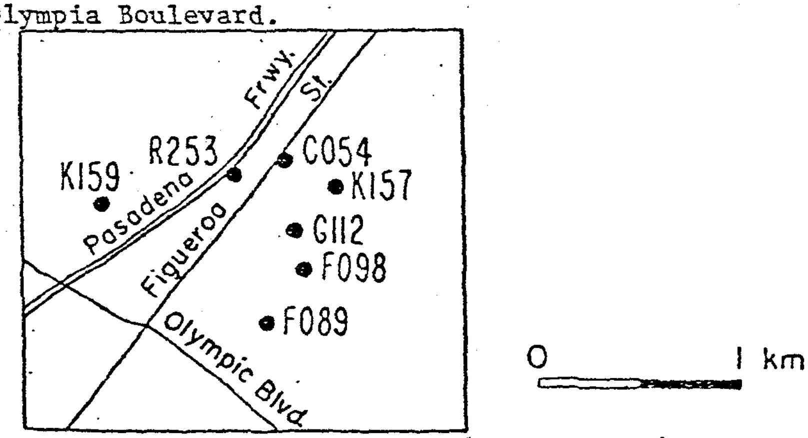


Fig. 3. Location of Recording Devices (After Hanks)

This array is close to the edge of the Los Angeles Basin and is approximately 43 kilometers south-southeast of the 1971 San Fernando epicenter. The individual stations are identified by their California Institute of Technology Earthquake Engineering Research Center reference numbers. All of the instruments were located at or below the ground level except for K 157 and K 159 for which the instruments were located on the second floor. The displacement records of the seven sites were shifted such that the shear wave arrival on each record occurred at 5.9 seconds. Figure 4 (Figure 5 in Hanks paper) presents the displacement time histories (2 horizontal and 1 vertical) for each of the seven recording devices. Figure 5 superimposes the southwest horizontal displacement time histories for F 089, K 157 and K 159 while Figure 6 superimposes the other horizontal component. Note that the amplitude, frequency content and arrival time of the main energy groups agree well. The coherence among the vertical components is not as strong as among the horizontal components but the amplitudes of the vertical motion are less than the horizontal motion.

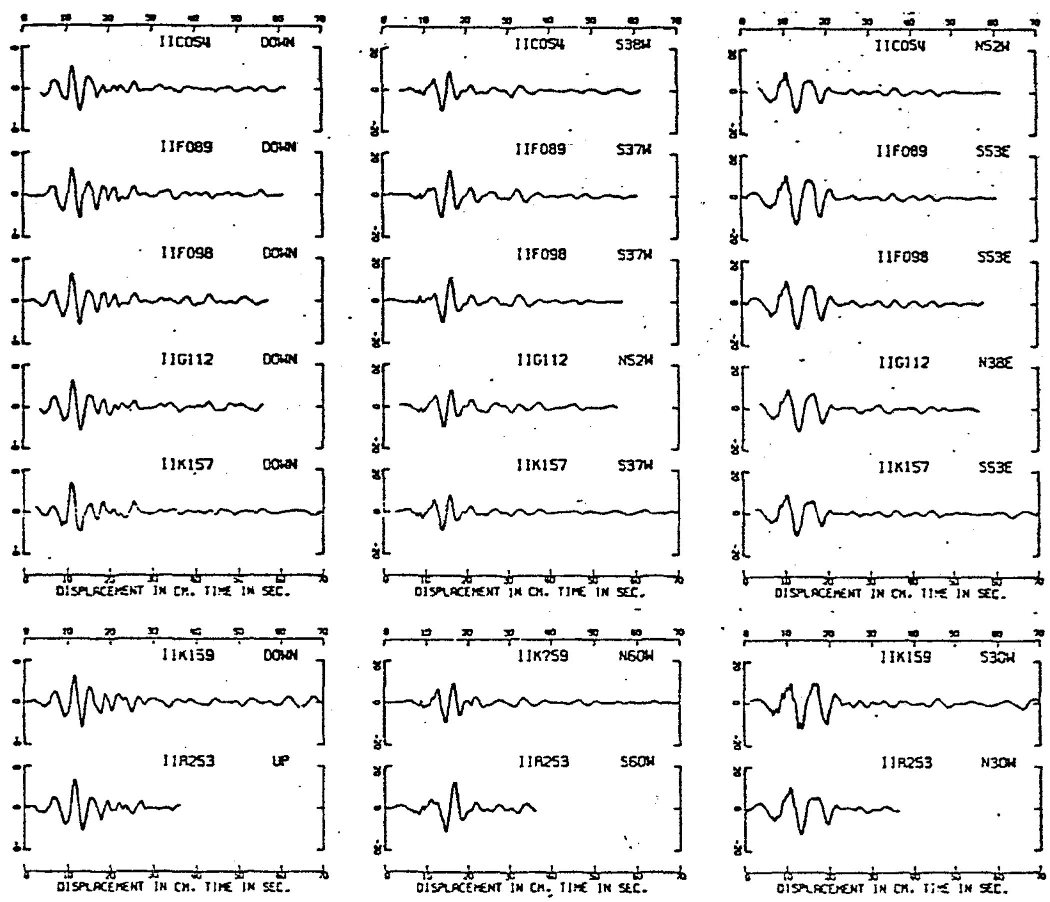


Fig. 4 Displacement Time Histories
For Site in Fig. 3 (After Hanks)

It should be noted that the soil properties for the sites in Figure 3 have been classified as soft by Trifunac and Brady (20). Hence, for relatively uniform soil properties along the pipeline, the wave shape appears to remain relatively constant which would be expected. However, if the soil properties change appreciably along the pipeline, the amplitude and frequency content and hence the shape of the seismic waves would also change along the pipeline.

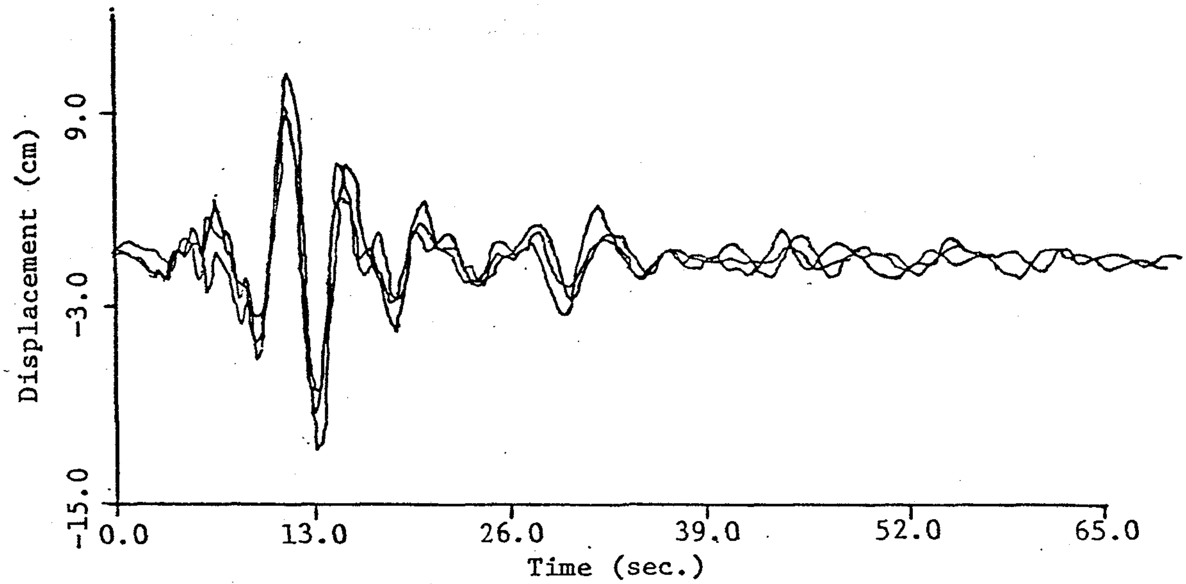


Fig. 5 Southwest Component of F 089, K 157 and K 159 Displacement Time Histories

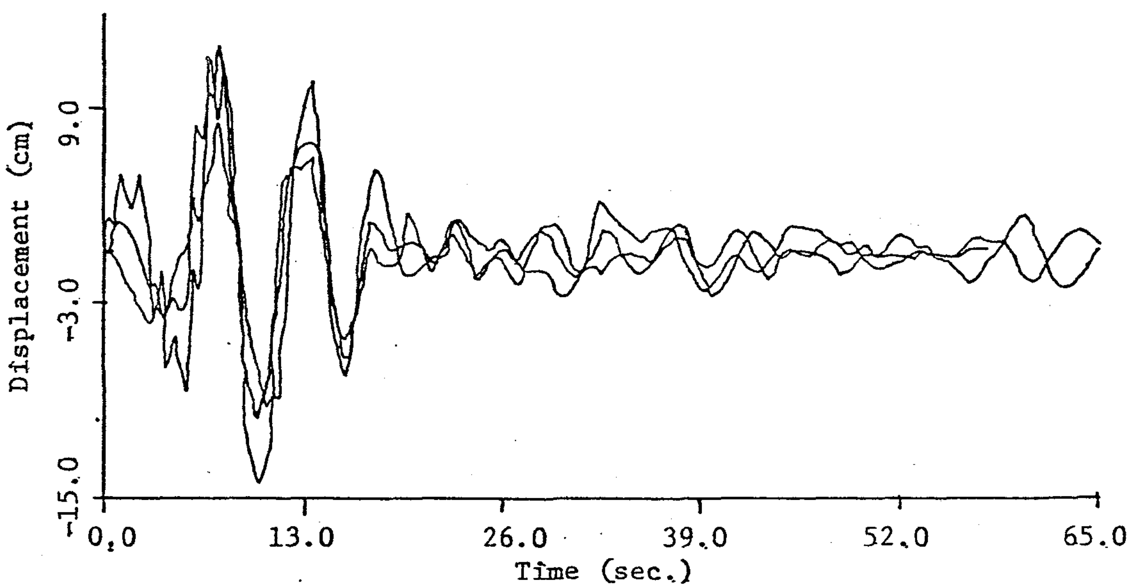


Fig. 6 Southeast Component of F 089, K 157 and K 159 Displacement Time Histories

Kubo (10) has mentioned that during the 1923 Kanto Earthquake the greatest pipeline damage was associated with changing soil properties along the pipeline. As opposed to wooden houses which were most heavily damaged in areas of soft alluvial ground, pipeline damage was highest in the transition zone from loam to alluvial soils. Hence, the presently available design procedures for underground pipelines may not be applicable for areas where heavy pipeline damage occurs, that is, in areas where the soil properties change appreciably along the pipeline.

Conclusions

The two assumptions which underlie the presently available design procedures for the effects of ground shaking on buried pipelines have been investigated. The first assumption that the relative motion between the pipelines and the soil is negligible is shown to hold for the worst case of a strong pipe in a weak soil. The second assumption that the seismic wave shape remains constant while traversing the pipeline is strictly applicable only for a pipeline with uniform soil properties along its length. It is finally noted that heavy pipeline damage has been associated with pipelines for which the soil properties are not uniform along its length, that is, in areas of transition from one soil type to another.

Acknowledgement

The work described in this paper is being supported by the National Science Foundation/RANN Program under grant number AEN-7614884. The authors gratefully acknowledge this support. However, the conclusions are those of the authors and do not necessarily reflect the views of the National Science Foundation.

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