Seismic Vulnerability, Behavior and Design

of Underground Piping Systems

Seismic Vulnerability, Behavior and

Design of Buried Pipelines

by

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Sponsored by The National Science Foundation

Directorate for Applied Science and

Research Application (ASRA)

Grant No. PFR76-14884
(Formerly ENV76-14884)

Technical Report (SVBDUPS Project) No. 9

(Final Report of Phase I Study)

March 1979

Department of Civil Engineering
Rensselaer Polytechnic Institute
Troy, New York 12181

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50272-101

REPORT DOCUMENTATION PAGE	1. REPORT NO. NSF/RA-790093	2.	3. Recipient's Accession No. PB298269

4. Title and Subtitle
Seismic Vulnerability, Behavior and Design of Buried Pipelines, Final Report of Phase I Study, Technical Report

5. Report Date
March 1979

6.

7. Author(s)
L.R. Wang, M.J. O'Rourke, R.R. Pikul

8. Performing Organization Rept. No.
Technical Report #9

9. Performing Organization Name and Address
Rensselaer Polytechnic Institute
Department of Civil Engineering
Troy, New York 12181

10. Project/Task/Work Unit No.

11. Contract(C) or Grant(G) No.
(C) PFR7614884
(G) (formerly ENV7614884)

12. Sponsoring Organization Name and Address
Engineering and Applied Science (EAS)
National Science Foundation
1800 G Street, N.W.
Washington, D.C. 20550

13. Type of Report & Period Covered
Final

14.

15. Supplementary Notes
Part of Project: "Seismic Vulnerability, Behavior, and Design of Underground Piping Systems".

16. Abstract (Limit: 200 words)
The seismic damage and response behavior of general buried pipelines is presented and vulnerability evaluation and design criterion of buried simple piping systems are described. The investigation focuses on the "Simplified Analysis" and "Quasi-static Analysis" approaches for determining the axial strains and relative joint displacements/rotations due to seismic shaking. This is justified by observations that the axial strains are predominant and the effects of pipeline inertia forces are negligible. To verify the assumptions and limitations in the analyses, the ground motion characteristics of the San Fernando earthquake were studied in detail. To fulfill the analysis requirements, the related soil parameters are discussed. To evaluate the seismic vulnerability/design of simple buried piping systems, a seismic risk analysis using data for Albany, New York is performed, and a failure criterion for buried water pipes is proposed. Finally, a case study is performed for the Latham Water Distribution System using these procedures. Based on a parametric study, the seismic responses of buried piping systems were found to be influenced by the physical properties of pipes and joints, geotechnical properties at the site, and the seismological parameters of the geographical region.

17. Document Analysis a. Descriptors
Earthquakes
Earthquake resistant structures
Dynamic structural analysis
Pipelines
Seismology
Design
Piping systems

b. Identifiers/Open-Ended Terms
Earthquake engineering
Buried pipelines
San Fernando Earthquake
Seismic damage
Seismic shaking
Seismic risk analysis
Albany, New York

c. COSATI Field/Group

18. Availability Statement	19. Security Class (This Report)	21.
NTIS	20. Security Class (This Page)	22. Price PC A07 MF A01

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

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6110 EXECUTIVE BOULEVARD
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ROCKVILLE, MARYLAND 20852

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ACKNOWLEDGEMENT

This is the ninth in a series of technical reports under the general title of 'Seismic Vulnerability, Behavior and Design of Underground Piping Systems' (SVBDUPS). This also serves as the final report under the research grant No. PFR76-14884 A03 (formerly ENV76-14884) sponsored by the Earthquake Engineering Program of the Directorate for Applied Science and Research Applications (ASRA) (formerly RANN) of the National Science Foundation for the period from September 1978 to December 1978. Dr. S.C. Liu was the Program Manager for the first half of the research project whereas Dr. W. Hakala was the Program Manager for the latter half of the project. Dr. Leon Ru-Liang Wang is the Principal Investigator.

The research has been carried out by Professors Leon R.L. Wang and Michael J. O'Rourke, a doctoral student, Richard R. Pikul and a number of graduate students and research assistants, G. Davis (1976-77), E. Solla (1976-77), K.M. Cheng (1976-78), N. Centola (1977-78), R. Fung (1977-79), E. Fok (1978-79). Graduate Assistant F. Tobin and Graduate Student M. Carroll worked in the 1978 Summer through Fall terms with special task projects. In addition, several undergraduate students, B. Conner, J. Getaz, J. Gotschall, A. Gottlieb, P. Koch, W. Koerner, H. Konopka and D. Tanous have participated in some aspects of the research.

Before the initiation of the project, a graduate student, N. Luk did some background studies in 1975-76.

Appreciation also goes to the Advisory Panel which consists of Mr. Holly A. Cornell, Board Chairman of CH2M Hill, Inc., Corvallis, Oregon; Mr. Warren T. Lavery, Superintendent of the Latham Water District, Albany, N.Y.; Dr. Richard Parmelee, Professor of Civil Engineering, Northwestern University; Dr. Jose Roesset, Professor of Civil Engineering, University of Texas at

Austin; Dr. Robert Whitman, Professor of Civil Engineering, Massachusetts Institute of Technology; and the faculty consultants: Profs. Dimitrios A-Grivas and Ricardo Dobry for their constructive advice and suggestions throughout the course of this investigation. In addition, Dr. Cetin Soydemir participated in the research discussions during the 1976-77 Academic year when he was visiting RPI.

The typing and proofreading of this report by Mrs. Jo Ann Grega and Mrs. Bonnie Hoag are also appreciated.

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

ABSTRACT

This is the final report of the first phase of a research project entitled "Seismic Vulnerability, Behavior and Design of Underground Piping Systems (SVBDUPS)" sponsored by the Earthquake Engineering Program, Directorate of Applied Science and Research Applications (ASRA) of the National Science Foundation. The report presents the seismic damage and response behavior of general buried pipelines, describes vulnerability evaluation and design criterion of buried simple piping systems and proposes recommendations for further studies.

The investigation is centered on the "Simplified Analysis" and "Quasi-static Analysis" approaches for determining the axial strains and relative joint displacements/rotations due to seismic shaking. This is justified by observations that the axial strains are predominant and the effects of pipeline inertia forces are negligible. To verify the assumptions and limitations in the analyses, the ground motion characteristics of the San Fernando Earthquake were studied in detail. To fulfill the analysis requirements, the related soil parameters are discussed. To evaluate the seismic vulnerability/design of simple buried piping systems, a seismic risk analysis using data for Albany, New York is performed, and a failure criterion for buried water pipes is proposed. Finally, a case study is performed for the Latham Water Distribution System using the procedures outlined above.

Based on a parametric study, the seismic responses of buried piping systems were found to be influenced by the physical properties of pipes and joints, geotechnical properties at the site and the seismological parameters of the geographical region. The following general conclusions can be made:

1. An upper bound for axial strain in the pipe is the maximum ground strain. This is based upon the assumptions that the pipeline inertia effects are negligible and the pipeline has negligible axial stiffness relative to the ground.

2. The upper bound on relative joint displacement is the product of the maximum ground strain and the pipe segment length. This is based upon the assumptions that inertia terms are negligible but the pipe segments between joints are infinitely rigid relative to the ground.

3. The maximum axial strains and maximum relative joint displacements of an actual pipeline in a seismic environment are within these two bounds. Their magnitudes depend on the relative stiffnesses of pipes and joints and the surrounding medium. The axial strain will be the greatest for continuous pipelines in which relative joint displacements are absent.

4. The pipe strains and relative joint displacements are higher for pipelines surrounded by relatively "soft" soils.

5. The seismic responses of buried simple pipelines are critically influenced by the slope of the ground displacement time history (i.e., the maximum ground velocity).

Since this report applies primarily to simple piping systems under simple geological environments, several recommendations to extend the study to general piping systmms with junctions/intersections under general geological environments are outlined as a final portion of this report.

CHAPTER I INTRODUCTION

I.1 Objective

This report is the Phase I final report of the research project under the general title of "Seismic Vulnerability, Behavior and Design of Underground Piping Systems (SVBDUPS)" sponsored by the Earthquake Engineering Program, Directorate of Applied Science and Research Applications (ASRA), formerly Research Applied to National Needs (RANN) of the National Science Foundation. The overall aims of the research are to develop a systematic way of assessing the adequacy and vulnerability of water/sewer distribution systems subjected to seismic loads and to develop design methodologies and criteria.

The specific purpose of this report is the study of the seismic vulnerability, behavior and design of simple buried piping systems. The report summarizes the research results from the task technical reports[5,6,7,8,10,13,14,16,17]*, technical memoranda[11,12,15] and student projects and theses[1,2,3,4,9] written in the two years of the investigation beginning in September 1976.

I.2 Assumptions and Limitations

This phase of the study deals with a simple buried piping system which is defined for the purposes of this report as a buried pipeline with the following restrictions:

- The pipeline is straight, segmental or continuous, but without branches or junctions as shown in Fig. I.1;

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* References are listed at the end of each Chapter.

- The pipeline is buried at a site of idealized soil and geological conditions and that the seismic excitation does not change dramatically over the region under investigation.
- The pipeline is subjected to seismic shaking or wave propagation effects only.
- There is no failure of soil surrounding the pipe such as landslide or liquefaction.
- Pipe material strength may be different in tension and compression but is linearly elastic.
- Pipeline inertia forces are neglected in the analysis.

Note that more specific assumptions will be made later in the report, when applicable.

I.3 Scope

Based on References 16 and 17, Chapter II will present the state of the art of buried lifeline earthquake engineering as the background for this report. The framework of the research is then laid out in a manner consistent with the background material.

Two methods of analysis and design for buried pipelines are discussed in this report; one is called the "simplified" approach and the other is called the "quasi-static" approach.

For the "simplified" approach which is presented in Chapter III, the pipe is assumed to follow the ground during seismic wave propagation. For the analysis/design of buried pipelines, the upper bounds for strain and curvature in a continuous pipeline due to seismic wave propagation will be the maximum ground strains and curvatures. These maximum ground strains and curvatures are related to the maximum velocity and acceleration at a site which are obtained from a seismic risk analysis. By comparing the ground strains/curvatures to the pipeline reserve strains/curvatures, one can determine the adequacy of a buried pipeline.

Based on References 3, 12, and 13, Chapter IV discusses the quasi-static formulation and analysis of the seismic response of buried pipelines. The formulation includes such important parameters as seismic wave velocity, soil resistant spring constants, pipe and joint stiffnesses and boundary conditions. Parametric behavior of buried pipelines was investigated and several field observations were confirmed.

The ground motion characteristics due to seismic waves[1] are discussed in Chapter V. The ground strains and ground curvatures are of major importance in developing the response behavior of buried pipelines. The data used in this study are the San Fernando Earthquake records processed by the California Institute of Technology.

Chapter VI discusses the seismic risk analysis[5,9]. It presents a methodology for obtaining the design maximum acceleration at a site for a given design life and probability of exceedance with an example applied to the Latham area, Albany, New York.

To analyze the seismic response of buried pipes, the soil resistant spring constants are necessary to study the soil interaction phenomenon. In the literature, there is data on lateral soil resistant characteristics. However, there is very little information on the soil resistance to movement of pipes in the longitudinal direction. Chapter VII presents data available from pipe models used to study the basic soil mechanisms that provide resistance to longitudinal pipe motion.

Chapter VIII presents seismic failure criteria for buried pipelines[4,14]. The failure criteria are based on the reserve strengths/strains of a buried pipeline that are available for seismic resistance beyond the normal loading conditions. For specific applications, this report studies such common water/sewer pipe materials as cast iron, ductile iron, steel and reinforced concrete.

As a case study[8], Chapter IX investigates the vulnerability of the Latham Water Distribution System. Maximum ground acceleration from the seismic risk analysis[5,9] in Chapter VI is used as input for the case study. Using the simplified approach, the maximum ground strains and ground curvatures are taken as the upper bounds for the strains and curvatures[8] of the pipes. By comparing these maximum pipe strains and curvatures with the reserve strains/curvatures[4,14], the safety of the pipeline is determined. This case study illustrates a general "simplified" methodology to evaluate the seismic vulnerability of any water distribution system.

Chapter X presents the conclusions of this Phase I study and presents recommendations for future investigations.

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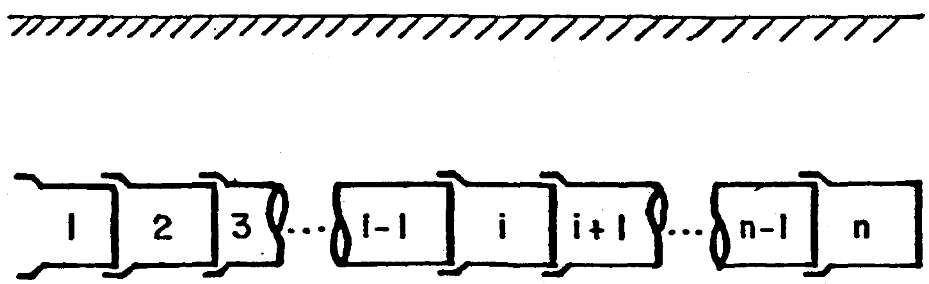

FIG. I.1 SCHEMATIC OF A BURIED PIPELINE

CHAPTER II BACKGROUND

II.1 Preface

This chapter presents state-of-the-art information on the behavior and design of buried lifelines such as submerged tunnels, gas, water and sewer distribution lines subjected to earthquakes. Specifically, a survey of pipeline damage due to past earthquakes as well as current design practices, analysis procedures, code provisions and the latest published research are discussed.

Recent studies[5,7,10,16,22,23,38,47] have shown that buried gas, water/sewer pipelines have been damaged heavily by earthquakes. Because of the importance of these systems to the health and safety of the populace, lifeline earthquake engineering is now beginning to draw the attention of the engineering profession[11]. Recent papers published in the subject may be grouped into six main areas as: (1) state of the art papers [3,20,25,69,70]; (2) observations of earthquake damage and response behavior[17,21,32,33,42,56,60,61,63]; (3) seismic risk analysis[43,53] and ground motion characteristics[45,52,75]; (4) analyses of response due to seismic wave propagation[1,9,14,15,19,24,27,39,40,44,45,49,51,55,56,62,64,66,71,72]; (5) studies of influential parameters[4,5,13,18,29,46,50,55,65,67] and (6) design considerations and design criteria[2,8,12,26,30,36,48,57,58,59,68].

It should be noted that all the above papers deal primarily with a single long pipeline or tunnel. Except for a systems approach to lifeline risk[73], very little discussion has been found on seismic response behavior or design of an entire buried lifeline network.

Presently, there are no codified provisions in the United States for the design of lifelines to resist seismic loads.

II.2 Observation of Pipeline Damage

Most of the existing literature concerning buried pipeline damage due to earthquakes, gives a qualitative rather than quantitative description of the damage. In general, it was observed that pipelines with rigid (cement or lead caulked) joints failed more than those with flexible (rubber gasket) joints.

After reviewing damage data from the 1964 Alaska earthquake, the 1971 San Fernando earthquake and the 1968 Mechering Earthquake in Western Australia, Kachadoorian[21] concluded that the geologic environment under the buried pipeline influenced the intensity and frequency of the pipeline damage. Qualitatively, the damage occurred least in bedrock, moderately in coarse-grained soil and the most frequently in fine-grained soils such as clay or silt.

Using damage data from earthquakes in Japan, authors[22,25] have correlated pipeline damage to pipe size and concluded that smaller pipes are more liable to break. However, others have taken an opposite view. Using 1971 San Fernando earthquake data[38], the damage statistics show no definite trend with respect to pipe size as shown in Reference 69.

Recently, Kubo et al[25] observed that the damage was highest in regions of transition from one type of soil to another. Also, damage statistics from the Fukui Earthquake showed that pipes parallel to the direction of wave propagation were more heavily damaged than pipelines normal to the direction of propagation. As one might expect, it was also observed in all investigations that the damage in main lines is proportional to the damage in service lines.

In summary, the following general conclusions may be drawn:

- Pipelines with flexible joints experience less damage during an earthquake than pipelines with rigid joints.
- Pipelines in regions of transition from one soil type to another experience the most damage during an earthquake. Otherwise pipelines in soft soil experience more damage than those in firm soil.

II.3 Response Behavior to Seismic Wave Propagation

Several recent investigations including both field observations and model tests in Japan have reported on the seismic response behavior of buried pipelines[24,27,30,33,49,63] and submerged tunnels[27,32,42,56,60,61]. General conclusions from these investigations are summarized as follows:

1. Most field data have indicated that buried pipelines[24,33,49] and submerged tunnels[42,56,60] move closely with the ground in both longitudinal and lateral directions during seismic wave propagation. There were no appreciable differences in displacements between these buried structures and the ground. Nakayama et al[32] observed that for earthquakes originating far from the site, the long period components were predominant in ground motion and the tunnel had almost the same behavior as the ground. However, where the epicenters are located near the site, the ground motion was governed by the short period components and the behavior of the tunnel did differ slightly from that of the ground.

2. The inertia force generated by motion of the buried lifeline was found to have very little effect upon the response of the structure

itself. Thus, the response behavior (stresses or displacements) of buried pipelines during earthquakes depends largely on the ground displacement characteristics along the route. The ground displacement characteristics are not affected to any significant degree by the existence of the buried lines.

3. Both axial and bending strains of submerged tunnels[32,59] and buried pipelines[33,49] were observed during earthquakes. The axial strains were found to predominate over the bending strains in all cases. The flexural strains at the bends were of the same order of magnitude as in the straight sections.

In discussing axial pipeline strains, Nasu et al[33] observed that the pipeline moved with the ground as long as the adhesion/friction between the pipeline and surrounding soil was not lost.

From the above discussion, it is concluded that the behavior of buried lifelines is governed by the relative displacements of the ground along the route. Ductility is the most important factor for the seismic design of such structures.

II.4 Analysis Procedures

A survey of most of the recently published literature in the areas of earthquake engineering and structural dynamics indicated that there is no single complete analytical model which is capable of predicting the behavior of an underground lifeline system under the attack of an earthquake. Standard text books[37,41,74] offer only brief discussions. There are however, quite a few articles which, after making simplifying assumptions, provide models for analyzing the underground pipelines for particular types of earthquake damage.

In a recent paper, Newmark and Hall[36] present a method which can be used to analyze and design buried pipelines which cross a known active fault. In this paper, the ductility (plasticity) of the material is used to allow for the large deformation.

Treating the soil supports as elastic springs, above ground pipelines[1] and underground structures[14,37,54,64] have been studied. In a recent paper, Luscher et al[28] discussed briefly the design of the buried portion of the Trans-Alaska pipeline.

As for failure behavior, Cheney[9] has presented solutions for the buckling of underground tubes.

Recently, a group of researchers at Weidlinger Associates have suggested the use of an"Interference Response Spectra"[34,35,72] while the authors of this report propose the use of the "Simplified or Quasi-static" approach[66,67] to analyze/design buried pipelines subjected to earthquake excitation. Ariman et al[31] at the University of Notre Dame, are presently studying buried pipes using a shell model. Novak and his students[15,39,40] at the University of Western Ontario have also independently studied a similar problem.

II.5 Current Design Practice

The conventional structural design of buried water and sewer pipes is based on a static analysis. In this country, no formal provision has been set by code organizations to design buried lifelines to resist earthquakes. However, passive physical design techniques[12] are occasionally used to avoid damage due to seismic effects. The following is a list of common practices and considerations:

- The pipeline should be located as far from fault lines as possible and, as a minimum should not be parallel to the fault line. For locations where the pipeline must cross an active fault, locating the pipeline at an oblique angle to the fault tends to reduce the shear in the pipeline[12].
- Pipeline construction on steep hillsides should be avoided when feasible due to the danger of landslides.
- Redundancy in the distribution system is desirable.
- Installation of blow-off valves near the fault line where higher seismic activity is anticipated should be considered.
- Ductile pipe material such as steel, ductile iron, copper or plastic, should be considered to allow for larger pipeline deformations.
- Flexible joints using rubber gaskets and ball-socket-type connections should be considered in areas of potentially strong seismic activity. Extra long restraining sleeves can be provided for sliding pipe connections.

As to an overall system design approach, Duke and Moran[11] and Whitman et al[73] have suggested the use of a reliability/damage level approach to the design of lifeline systems to resist various intensities of ground motion.

Okamoto[41] has suggested the seismic accelerations of 0.1g to 0.3g for the design of buried pipelines in Japan. The coefficients depend upon soil conditions, the softer the soil the larger the value.

There are a number of additional papers proposing criteria for the seismic design of buried pipelines[30,48] and submerged tunnels[57,58,59].

The authors have proposed the use of a seismic reserve strength/ ductility concept to design water distribution pipelines[68].

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CHAPTER III SIMPLIFIED APPROACH

III.1 Preface

A simplified approach has been developed to analyze a pipeline subjected to earthquake wave effects. For a continuous pipeline, assuming no relative motion between the pipe and the soil and negligible axial stiffness relative to the soil, an upper bound for pipe axial strains and curvatures can be obtained. For such an idealized situation, the maximum pipe strain can be approximated by superimposing the axial and flexural strains of the soil surrounding the pipeline. Maximum soil strains and curvatures can be determined if the maximum ground acceleration and maximum ground velocity for a particular site are available as well as the propagation velocity of the seismic waves. On the other hand, an upper bound for relative joint displacement and rotation of a segmented pipeline can be obtained by assuming that there is negligible axial force and bending moment transferred through the joints and that the pipe segments are rigid relative to the soil.

This simplified approach results in maximum pipe strain criteria and maximum joint displacement/rotation criteria. These two sets of upper bounds are developed by assuming either that the pipe segments are very rigid or the pipeline is very flexible with respect to soil. Actual pipe systems lie somewhere between these two extremes and have a non-zero value for joint stiffness representing some degree of physical restraint. Hence, if a pipe system can meet both sets of upper bound criteria, it will be adequate for behavior which includes some pipe strain and joint movement to accommodate the imposed ground displacement. Note that any relative motion between the pipe and the soil will also tend to mitigate pipeline strain/curvature and joint displacements. Since actual behavior is difficult to predict, except in idealized situations, the upper bound criteria provides a simple and conservative technique for the evaluation of underground pipelines subjected to seismic wave effects.

Using this simplified approach, a continuous pipeline would be required to satisfy the strain and curvature criteria in both tensile and compressive modes. A segmented pipeline with flexible joints would be required to satisfy the same strain criteria as well as the joint displacement/rotation criteria.

III.2 Pipe Strain/Curvature Upper Bounds

For the maximum strain and curvature in the pipeline, an upper bound is obtained by assuming that the pipeline is very flexible with respect to the soil, and hence is equal to the maximum ground strain and curvature. Assuming that the seismic wave shape remains constant while traversing the pipeline, the maximum ground strain, or in this case, the maximum axial strain, ε_a^*, in the pipeline is given by[1,2]

$$\varepsilon_a = V_{max}/C_p \tag{III.1}$$

where V_{max} is the maximum ground velocity and C_p is the propagation velocity of seismic longitudinal waves in the surrounding soil relative to the pipeline axis.

The maximum pipe and soil curvature χ is:

$$\chi_{max} = A_{max}/C_s^2 \tag{III.2}$$

where A_{max} is the maximum ground acceleration and C_s is the transverse wave velocity in the controlling medium. The pipe flexural strain, ε_f, is then

---

* Symbols are defined when they first appear and rearranged in the Appendix-Notations.

obtained by multiplying the curvature by the pipe outer radius, R. Thus,

$$\varepsilon_f = R\chi = \frac{RA_{max}}{C_s^2} \tag{III.3}$$

The combined total pipe strain, ε_t, is conservatively:

$$\varepsilon_t = \varepsilon_a + \varepsilon_f \tag{III.4}$$

This combined strain is conservative since the maximum values of acceleration and velocity would not occur simultaneously.

III.3 Joint Displacement/Rotation Upper Bounds

For the maximum displacement and rotation at joints, an upper bound is given by assuming that the pipeline is rigid with respect to the soil. For this model, a pipe segment moves as a rigid body.

Consider the model shown in Figure III.1, composed of two rigid segments of length L. The soil stiffness is idealized by linearly elastic soil springs, K_a. If the base of the soil springs is given a displacement $g_a(x)$, the final displacement of the rigid bodies, u_1 and u_2, can be determined by equilibrium. As noted in Figure III.2, if there are no significant joint forces on the rigid pipe segment, the sum of the spring forces is equal to zero.

This may be represented as:

$$\int_0^L K_a \, [u_2 - g_a(x)]dx = 0 \tag{III.5}$$

$$\int_0^L K_a \, [u_1 - g_a(x)]dx = 0 \tag{III.6}$$

where x is a variable along the axis of the pipe.

Simplification of the above leads to:

$$u_2 = \frac{1}{L}\int_0^L g_a(x)dx \tag{III.7}$$

and,

$$u_1 = \frac{1}{L}\int_{-L}^0 g_a(x)dx \tag{III.8}$$

The relative displacement between the two rigid segments, U, becomes

$$U = u_2 - u_1 = \frac{1}{L}\int_0^L g_a(x)dx - \frac{1}{L}\int_{-L}^0 g_a(x)dx \tag{III.9}$$

If we assume that the variation of the ground displacement $g_a(x)$ over the two rigid segments can be modeled by a straight line, the ground variation can be represented as:

$$g_a(x) = \beta + \alpha x \tag{III.10}$$

From equation (III.10)

$$U = \frac{1}{L}\int_0^L (\beta + \alpha x)dx - \frac{1}{L}\int_{-L}^0 (\beta + \alpha x)dx \tag{III.11}$$

which can be simplified to:

$$U = \alpha L \tag{III.12}$$

where αL represents the relative joint displacement. Since α is the slope of the displacement versus distance function:

$$\alpha_{max} = [\frac{d}{dx}\ g_a(x)]_{max} \tag{III.13}$$

For a traveling seismic wave, $g_a(x) = f(x-C_p t)$ which leads to:

$$[\frac{d(g_a)}{dx}]_{max} = \frac{1}{C_p}\ [\frac{\partial(g_a)}{\partial t}]_{max} = \frac{V_{max}}{C_p} \tag{III.14}$$

Hence, an upper bound for the relative displacement, U_{max}, can be obtained from the expression

$$U_{max} = \frac{V_{max} L}{C_p} \tag{III.15}$$

Although the above result assumed that the soil spring was linear, it can be shown that the same result is obtained regardless of the shape of the soil spring function (elastic-plastic, non-linear) as long as the rigid body movement occurs with no joint restraining force.

Maximum relative joint rotation will occur in regions where the rate of change of slope, the curvature, is a maximum. Assuming that the wave length is very long as compared to the pipe segment length, rigid segments will follow the curve as a series of tangent sections with rotation or slope change concentrated at the joint. Figure III.3 illustrates such a condition in a region where the curvature, χ, is assumed constant. For small angles, $\tan(\theta/2) = \frac{\theta}{2} = \frac{L\chi}{2}$, therefore, θ_{max}, the maximum relative joint rotation can be represented as:

$$\theta_{max} = L(\chi)_{max} = \frac{LA_{max}}{C_s^2} \tag{III.16}$$

Thus, equations (III.15) and (III.16) provide a simple and conservative technique for calculating maximum joint displacement and rotations in a rigid segmented pipe system if the earthquake accelerations and velocities can be predicted and if the propagation velocities with respect to the pipeline can be estimated.

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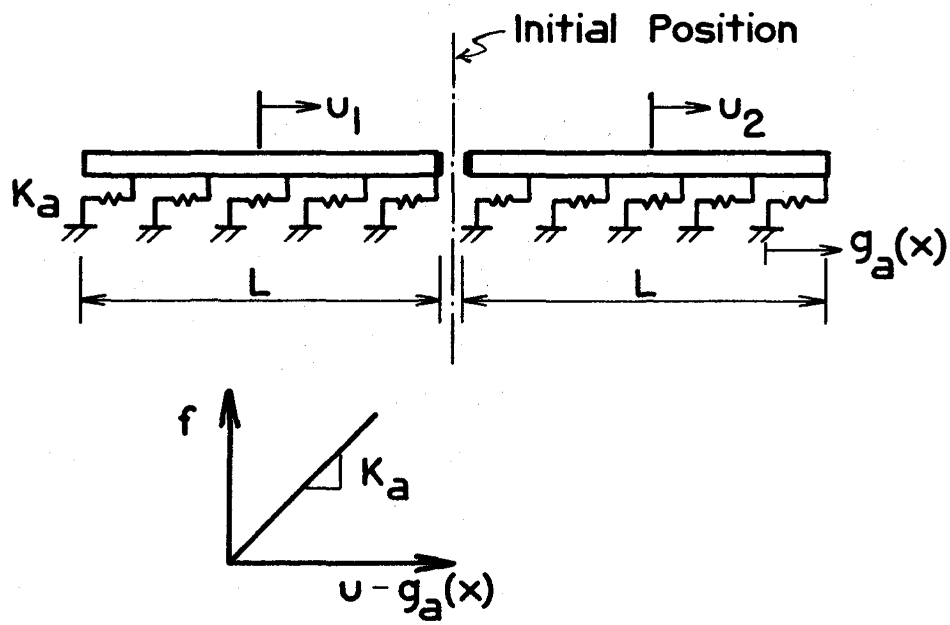

FIG. III.1 BURIED RIGID PIPE MODEL

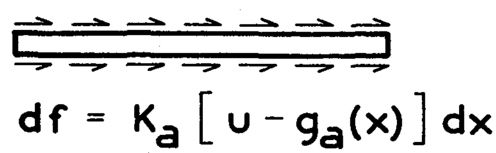

FIG. III.2 SOIL RESISTANCE TO PIPE MOTION

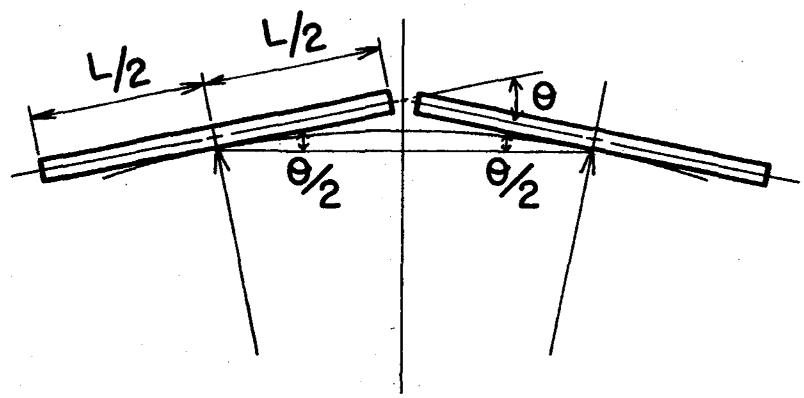

FIG. III.3 RELATIVE JOINT ROTATION OF RIGID PIPE MODEL

CHAPTER IV QUASI-STATIC ANALYSIS APPROACH

IV.1 Preface

As indicated in Chapter II - "Background", pipeline damage caused by earthquakes in the longitudinal direction has been found to be a major mode of failure[4,5,6]. During seismic shaking, the response behavior of buried pipelines depends mainly on the ground displacement characteristics along the pipeline route, and to a much lesser extent, the inertia of the pipeline itself[7 to 11]. Therefore, this investigation is limited to the axial response due to imposed ground displacement time history without dynamic terms.

In Chapter III, upper bounds for pipe strains and relative joint displacements are obtained by assuming that the pipeline is continuous and very flexible (upper bound for strains) or the pipeline consists of isolated rigid segments (upper bound for relative joint displacements).

Actually, a buried pipeline reacts to the seismic wave propagation through the media of the surrounding soil. Thus, the response behavior of the buried pipeline will be influenced by a number of physical, geotechnical and seismological parameters. The physical parameters are the geometrical and mechanical properties such as pipe diameter, thickness, segment length, and Young's modulus. The geotechnical parameters are the soil-structure interaction resistant constant, its variation along the pipeline and the wave propagation velocity. The seismological parameters are the form, amplitude and the slope of the ground displacement time history.

In earlier investigations[2,13], a simple quasi-static model consisting of rigid pipe segments connected by elastic joint springs was used to study conservatively the relative joint motions of segmented pipelines due to seismic shaking. Based on the general formulation[12], the purpose of this study was to develop a more rigorous quasi-static analysis model for the response of actual buried pipelines, segmented or continuous, subjected to earthquake motion in the axial direction. By comparing the pipe strains from the analysis with the seismic design criteria[14], discussed in Chapter VIII, the safety of buried pipelines subjected to a given set of earthquake loadings may be evaluated.

Since the effects of pipeline inertia terms on the response behavior of buried pipelines[7,8,10,11] have been found to be negligible, the inertia and damping terms in the dynamic equations of motion will be dropped. Because the input ground motion is a function of time, the response will also be a function of time. Thus, the analysis is called "Quasi-static".

IV.2 Formulation For General Buried Pipelines

The detailed derivations for the quasi-static analysis of buried pipeline are given in Ref. 12. The formulation for the soil-structure interaction system is based on the variational principle of energies neglecting dynamic (inertia) terms. This section briefly describes the development.

A long buried piping system consisting of n-segments is shown in Fig. IV.1. The pipe segment has axial stiffness (Ea/L) and a node at each end. The joints are represented by linearly elastic springs. The resistance forces that develop between the soil and the pipe segments are represented by linearly elastic soil springs.

The equations of static equilibrium, obtained from the variation of the total strain energy[1] in the soil-structure interaction system, are found to be:

$$\underset{2n \times 2n}{[K_{system}]} \quad \underset{2n\times 1}{\{X\}} = \underset{2n \times 2n}{[K_{soil}]} \quad \underset{2n\times 1}{\{X_G\}} \tag{IV.1}$$

where $[K_{system}]$ and $[K_{soil}]$ are symmetrical tridiagonal structural system and soil resistance matrices respectively, $\{X\}$ is the nodal axial displacement vector and $\{X_G\}$ is the ground displacement vector which varies with time. Note that this general model has 2n degrees of freedom.

IV.3 Models For Special Cases

To save computation time, two special case models have also been developed (one for the buried continuous piping system and the other for the rigid segmental piping system).

For a continuous buried pipeline system, there are no joint springs. In this case, $X_{2i} = X_{2i+1}$. In an n-segment pipeline system, there will be n+1 degrees of freedom while for a rigid pipe segment system, there will be n degrees of freedom.

Note that in the absence of relative joint displacements, the continuous system will approach the upper bound values for pipe strains. The rigid segmental system, on the other hand, will give the upper bound of relative joint displacements for a given seismic input.

IV.4 Ground Motion Input

The solution for pipe motion $\{X\}$ given in Eqn. (IV.1) depends on the inputs of the ground motion $\{X_G\}$. Since $\{X_G\}$ is a function of time, the solution of $\{X\}$ is also a function of time.

Assuming that the wave form of the traveling seismic excitation remains constant over the entire length of the pipeline which consists of n-segments, the inputs of the time-space varying ground motions starting from the first support are:

$$X_{G0} = \begin{cases} 0 & t < 0 \\ \Delta_{max}\, h(t) & t \geq 0 \end{cases}$$

$$X_{G1} = \begin{cases} 0 & t-\eta_1 < 0 \\ \Delta_{max} h(t-\eta_1) & t-\eta_1 \geq 0 \end{cases} \qquad \text{(IV.2)}$$

$$X_{Gi} = \begin{cases} 0 & t-\eta_i < 0 \\ \Delta_{max} h(t-\eta_i) & t-\eta_i \geq 0 \end{cases}$$

where Δ_{max} is maximum ground displacement in an earthquake record; h(t) is the displacement time function; η_i is the delay time of seismic wave traveling from the first support to the end face of i^{th} pipe segment

$$\eta_i = \sum_{j=1}^{i} L_j/C_j \qquad \text{(IV.3)}$$

and C_j is the propagation velocity of seismic wave with respect to the pipe segment j.

IV.5 Results

IV.5.1 General

The system of governing equations requires the input of ground displacement at an instant of time. The response of nodal displacements, X_i, are calculated at each time step for the entire time-history of the earthquake input record. The resulting pipeline nodal displacements, X's are used to determine three parameters:

$$\varepsilon_i = (X_{2i} - X_{2i-1})/L_i \qquad \text{(IV.4)}$$

$$U_i = X_{2i+1} - X_{2i} \qquad \text{(IV.5)}$$

$$Y_i = X_{2i} - X_{Gi} \qquad \text{(IV.6)}$$

where

ε_i = average strain of i^{th} pipe segment

U_i = relative displacement of i^{th} joint spring between two adjacent pipe segments

Y_i = relative displacement between pipe and the ground

By comparing these parameters within the earthquake time domain, the maximum values of average pipe strains, ε_{max}, relative joint displacement, U_{max} and relative displacement between the ground and the pipe, Y_{max} and their corresponding time and location are determined.

Note that a computer program for the general quasi-static analysis and parametric studies of buried pipelines has been written by Fok[3], this Chapter presents only the important results and conclusions without details.

IV.5.2 Reference Pipeline Conditions

To establish a basis for determining the effect of various parameters upon the seismic response, the following conditions are arbitrarily set as the "reference" conditions.

Pipeline Parameters

Material (Cast Iron)	E_s = 14000 Ksi (96500 MPa)
Outer Diameter	D = 18 in (45.72 cm)
Wall Thickness	t_o = 0.54 in (1.37 cm)
Number of Segments	n = 20
Free End Conditions	$\bar{K}_0$, $\bar{K}_{n+1}$ = 0,0
Joint Spring Constant	$\bar{K}_i = 10^{-1} - 10^{6}$ kips/in ($1.75 \times 10^{-1} - 1.75 \times 10^{6}$ kN/cm)

Soil Parameters

Wave Propagation Velocity C_S = 800 ft/sec. (240 m/sec.)

Resistant Spring Constant K_a = 3400 lbs/in/in (2340 N/cm/cm)

Seismic Parameters

El Centro May 18, 1940 S90W Component (Fig. IV.2)

Maximum Ground Displacement Δ_{max} = 7.79 in (19.79 cm)

Duration T = 50 sec.

Maximum Ground Velocity V_{max} = 14.5 in/sec (36.9 cm/sec)

For evaluating the effects of a particular parameter upon the response, only that parameter will be varied from the above mentioned "reference" conditions.

IV.5.3 Effect of Pipe Materials and Joint Stiffnesses

The results of maximum average pipe strains, ε_a, and maximum relative displacement, U_{max} for four 18 in. (45.72 cm) diameter water pipes of different materials (cast iron, ductile iron, reinforced concrete, and steel) are shown in Fig IV.3 and IV.4 for various joint stiffnesses. One can see from these figures, that when the joint stiffness is large (approaching a continuous pipe) the strain will be larger and the relative joint displacement becomes smaller. However, the differences in strains or relative joint displacements for three of the different materials are negligible.

IV.5.4 Effect of Pipe Segment Length

The effects of pipe segment length on pipe strain and relative joint displacement for three pipe segment lengths of 10 ft. (3.05 m), 20 ft. (6.10 m), and 40 ft. (12.19 m) are shown in Figs. IV.5 to IV.6 respectively. From these figures, one can easily see that the longer the pipe segment is, the larger the pipe strain, and also the larger the relative joint displacement will be.

The upper bounds of pipe strain and relative joint displacement estimated by the Simplified Approach are also shown in the figures. Note that actual pipe strains and relative joint displacements are always below these two upper bounds.

IV.5.5 Other Effects

The effects of a number of other parameters on the response of buried pipelines, such as pipe size (diameter), non-uniform resistance along the pipeline route, and wave forms have been investigated and reported in detail in Reference 3.

IV.6 Summary and Conclusions

The present general quasi-static analysis model for buried pipelines has substantially extended an earlier rigid pipe segment model[2,13] to include actual physical properties of the pipe, soils and seismic input. This analysis is capable of evaluating the general longitudinal responses of buried pipelines (segmented or continuous) subjected to seismic wave propagation. Based on the parametric study, the following concluding remarks can be made.

1. The most influencial parameters on the response behavior of buried pipelines are the maximum ground velocity and the wave propagation velocity of the seismic waves as suggested by the "Simplified" approach.
2. In general, longer pipe segment lengths and softer soil will produce both larger pipe strains and relative joint displacement.
3. Axial strains in continuous pipelines will be higher than those in segmented pipelines.
4. For a given value of joint stiffness, the difference in seismic response behavior for various commonly used pipeline materials is negligible.

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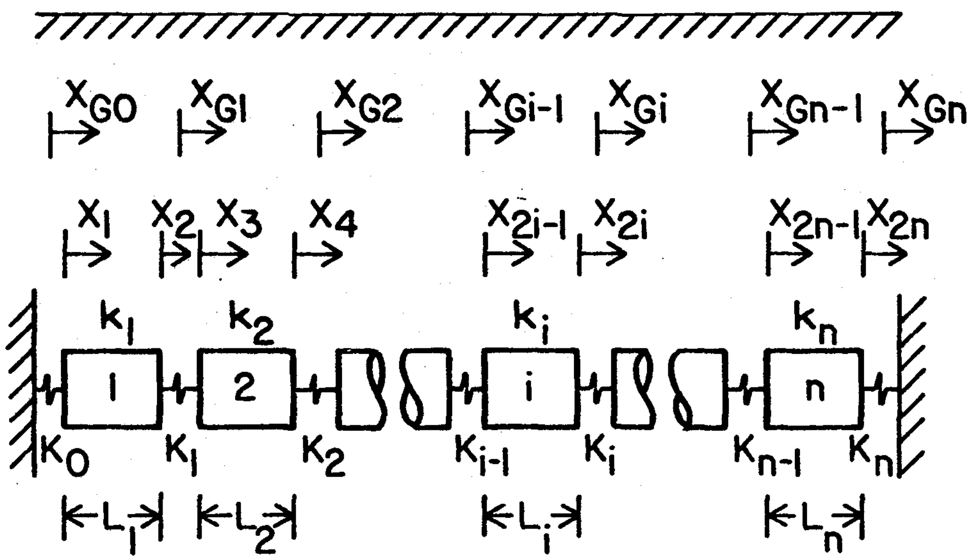


FIG. IV.1 A BURIED LONG PIPING SYSTEM

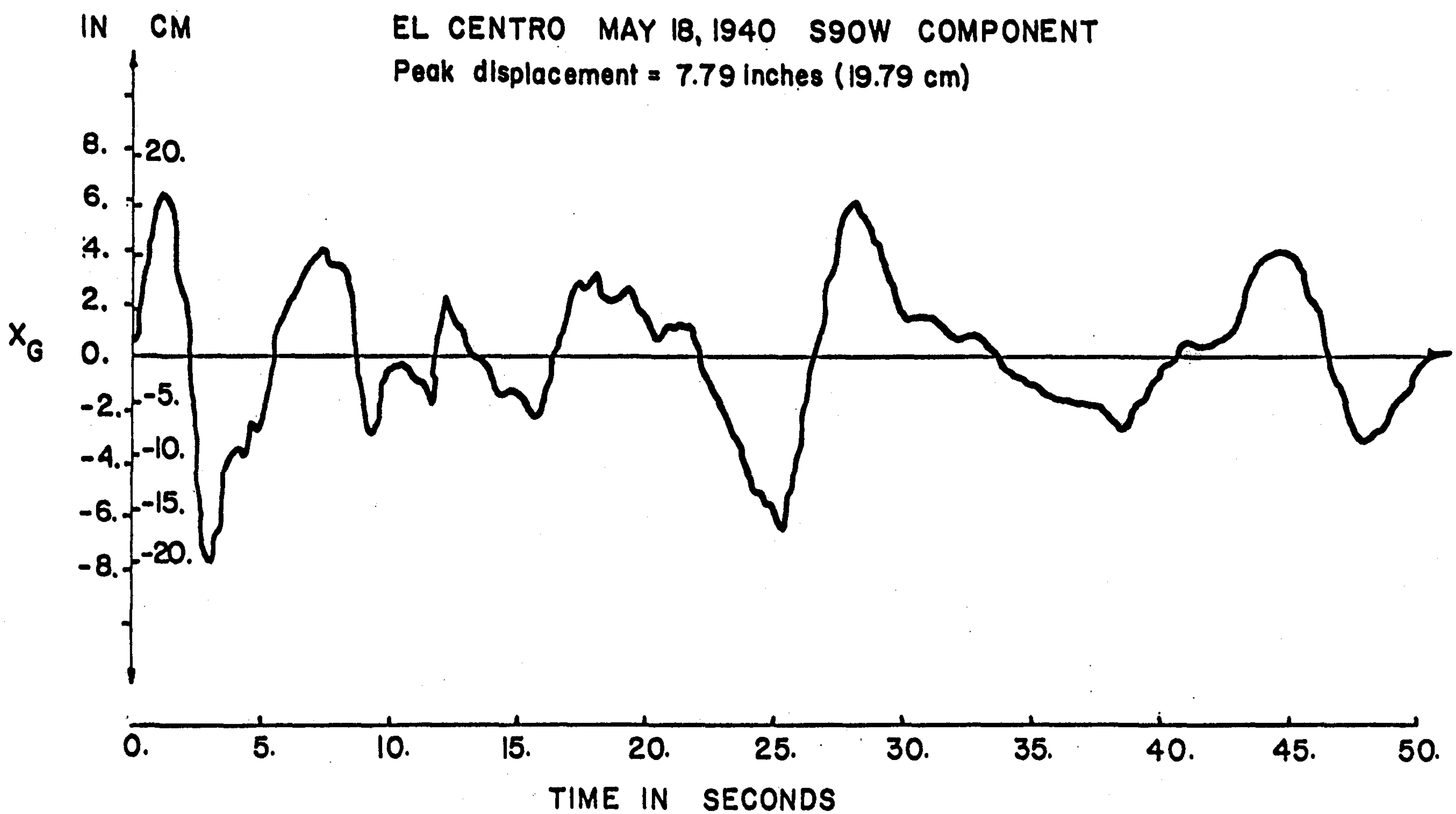


FIG. IV.2 REFERENCE GROUND DISPLACEMENT INPUT

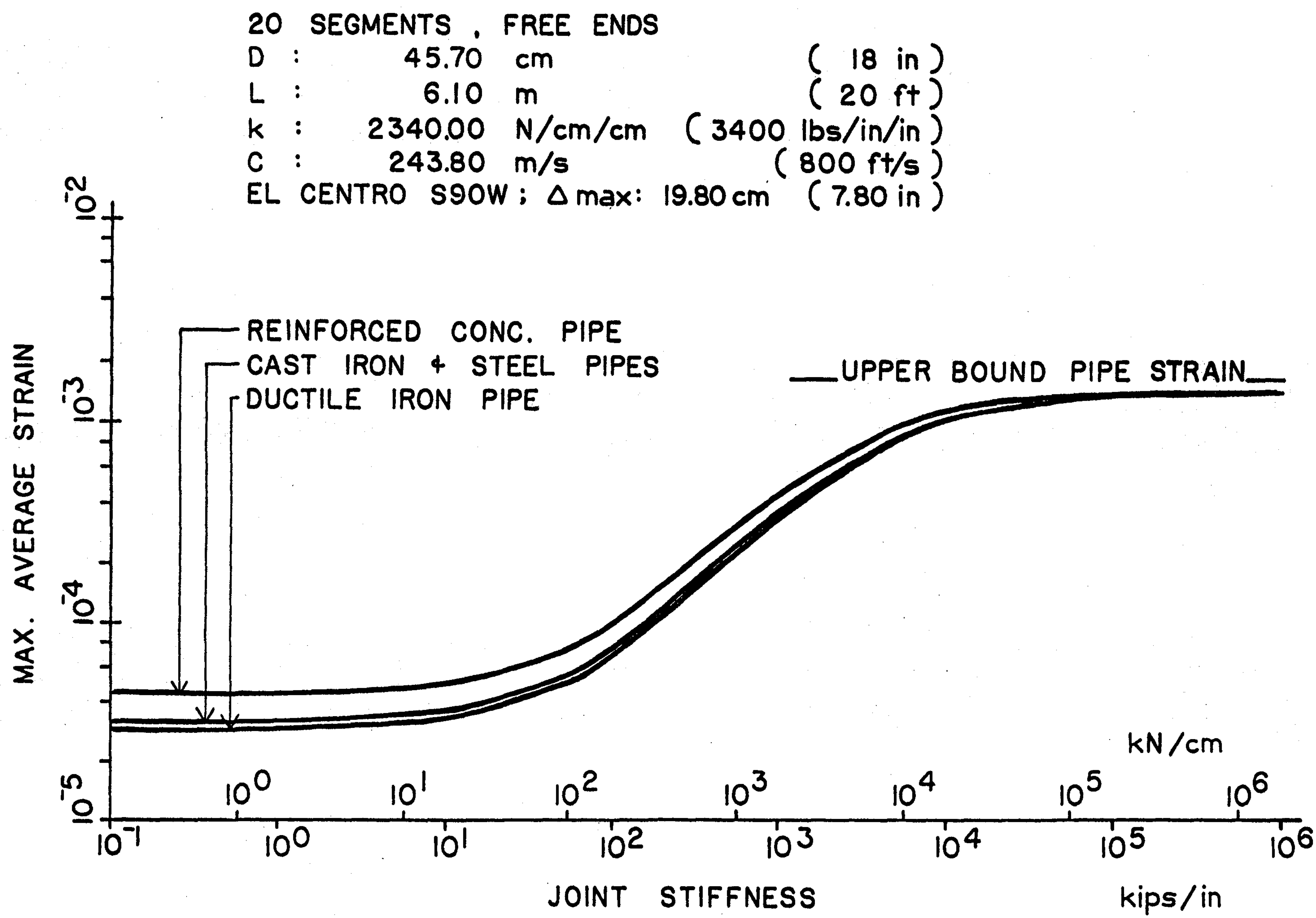


FIG. IV.3 EFFECT OF PIPE MATERIAL AND JOINT STIFFNESS ON PIPE STRAIN

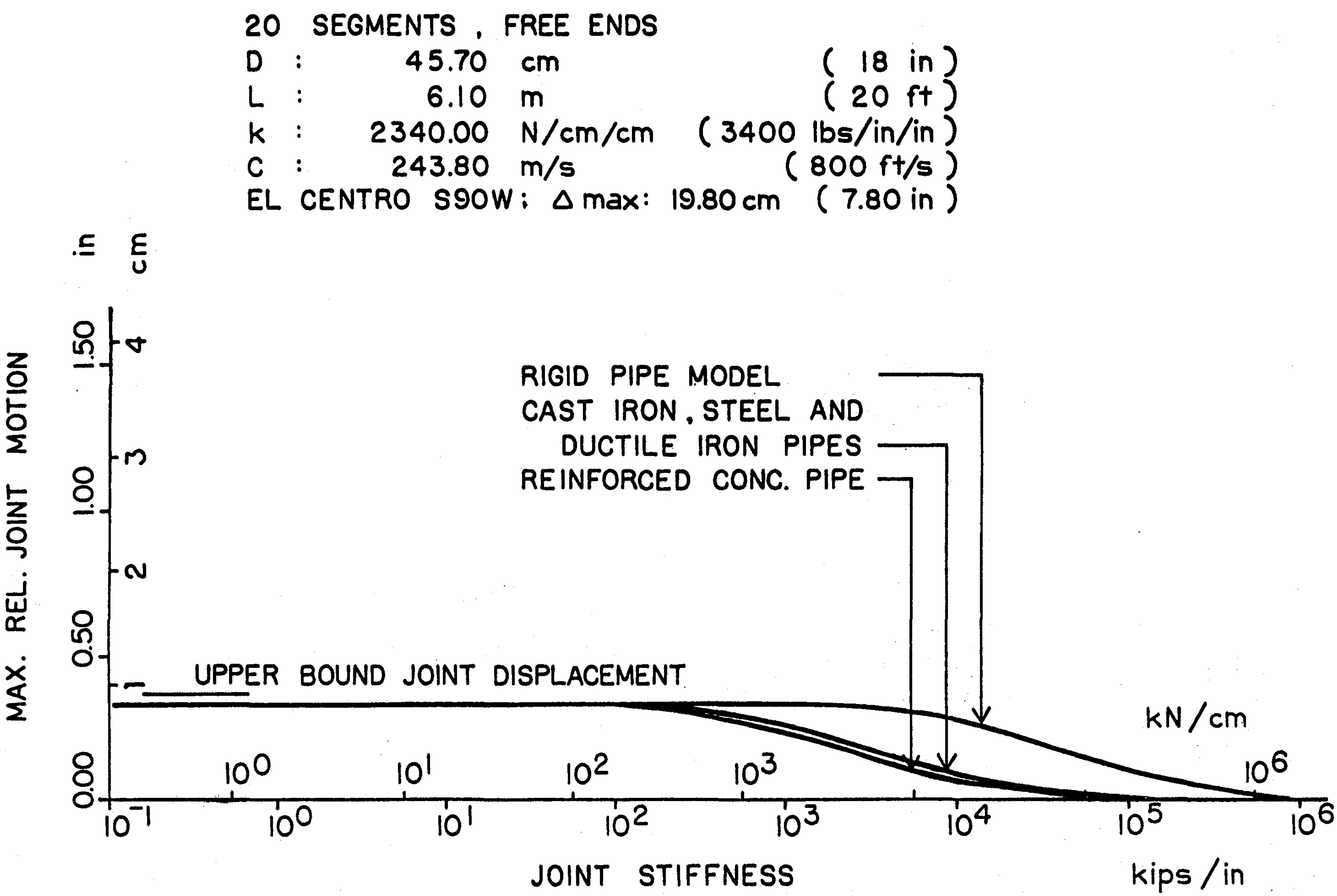
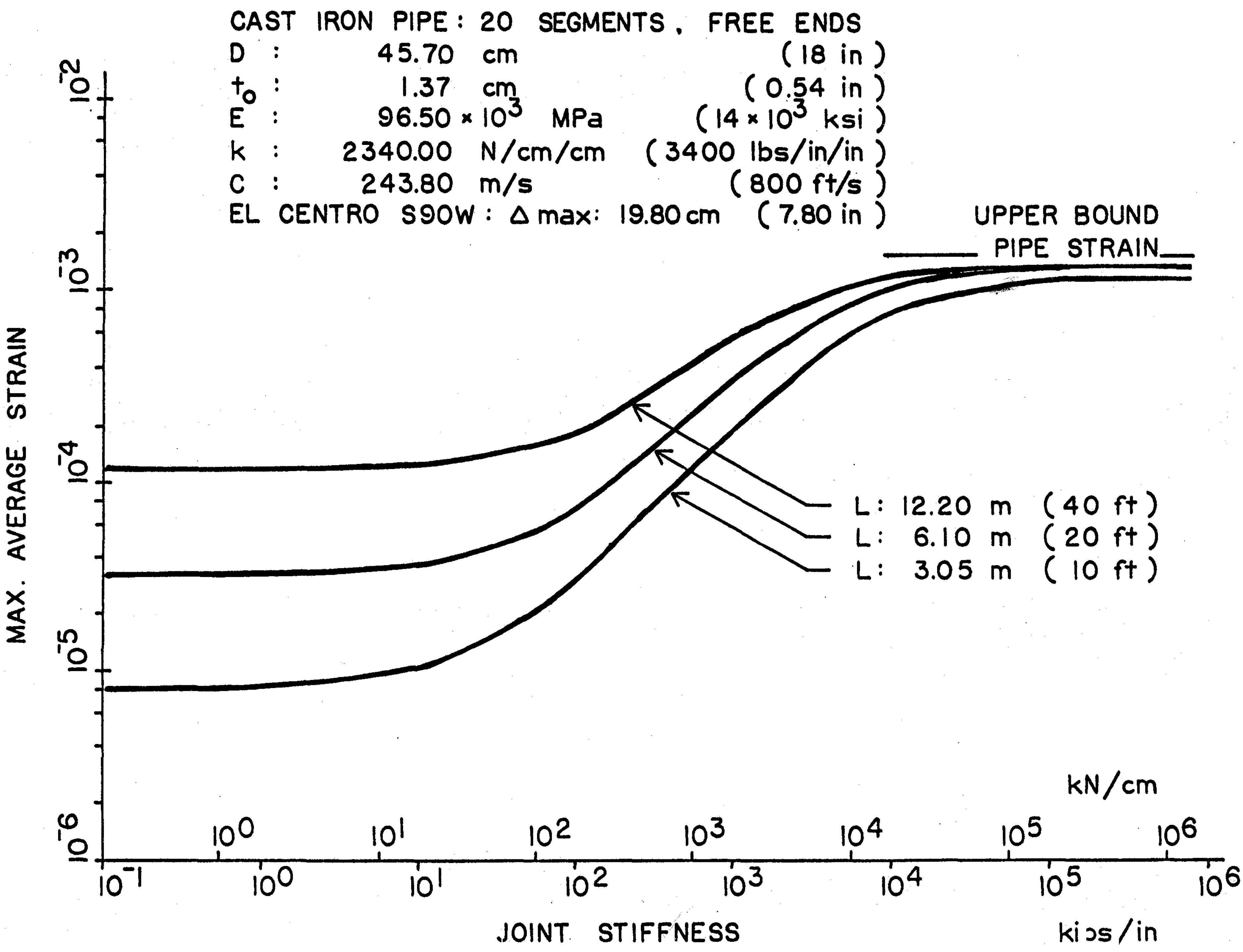


FIG. IV.4 EFFECT OF PIPE MATERIAL AND JOINT STIFFNESS ON RELATIVE JOINT DISPLACEMENT



CAST IRON PIPE: 20 SEGMENTS, FREE ENDS
D : 45.70 cm (18 in)
to : 1.37 cm (0.54 in)
E : 96.50 × 10^3 MPa (14 × 10^3 ksi)
k : 2340.00 N/cm/cm (3400 lbs/in/in)
C : 243.80 m/s (800 ft/s)
EL CENTRO S90W : Δ max: 19.80 cm (7.80 in)
UPPER BOUND
PIPE STRAIN
L: 12.20 m (40 ft)
L: 6.10 m (20 ft)
L: 3.05 m (10 ft)
MAX. AVERAGE STRAIN
JOINT STIFFNESS
kN/cm
kips/in

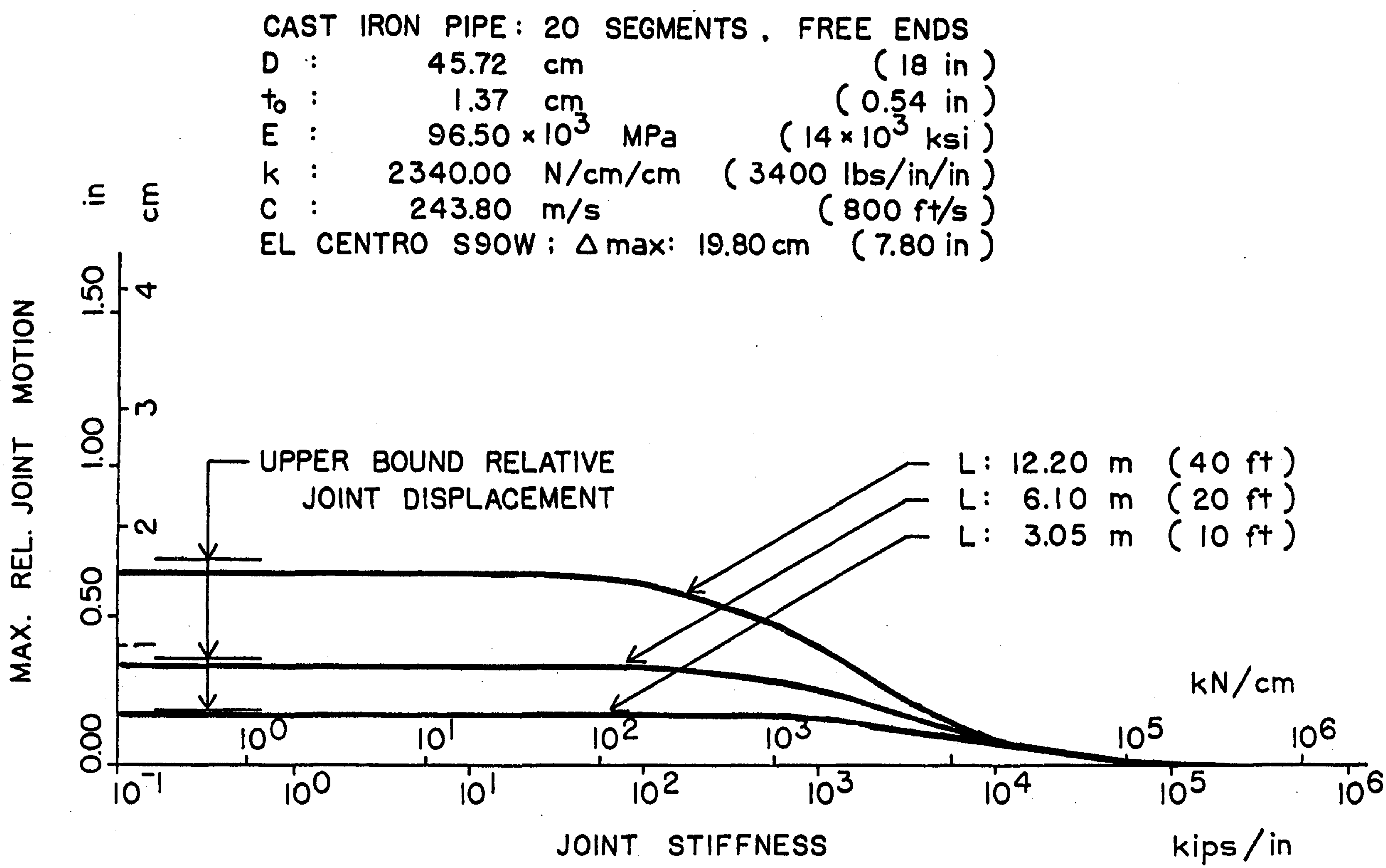


FIG. IV.6 EFFECT OF PIPE SEGMENT LENGTH ON RELATIVE JOINT DISPLACEMENT

CHAPTER V GROUND MOTION CHARACTERISTICS

V.1 Preface

In this chapter certain characteristics of earthquake ground motions which affect the behavior of buried pipelines subject to earthquakes are examined in detail. Specifically, the simplified procedures in Chapter III note that for the critical case of a pipeline lying parallel to a radial line from the epicenter, maximum ground velocity in the radial direction governs the induced axial strain. For a pipeline with the same orientation, the induced curvature in the pipeline is governed by the maximum ground acceleration in the tangential direction. In this context, the question arises as to whether there is any difference between the radial and tangential components of the earthquake ground motion.

The second question addressed in this chapter deals with the variation of the shape of the seismic waves. In particular, the simplified procedures discussed in Chapter III are based upon the assumption that the shape of the seismic wave remains unchanged as it traverses the pipeline. This assumption is also investigated in this chapter.

V.2 Radial vs. Tangential Components

In a very simple model of earthquake faulting in which only P and S waves propagate away from the fault and the material properties of the earth are uniform between the fault and the pipeline, the radial component would correspond to pressure waves while the tangential component would correspond to shear waves. It is generally accepted that, in the near field, strong shaking corresponds to the arrival of shear waves. Hence, using the simple model of earthquake faulting, one might expect that the tangential component, due to shear waves in this model, would be stronger than the radial component due to pressure waves. Of course, pressure and shear waves are not the only waves generated by actual earthquake faulting. In addition, material properties generally change along the propagation path (for instance, from rock to soil) which leads to reflection and refraction of the propagating seismic waves.

The basic question to be answered is whether there is any difference between the radial and the tangential components of ground motion. That is, should any special consideration be given to the fact that V_{max} in Equation (III.1) corresponds to the maximum ground velocity in the radial direction while A_{max} in Equation (III.2) corresponds to the maximum ground acceleration of the tangential component?

Ground motion time histories recorded at 26 separate sites during the 1971 San Fernando Earthquake were used to answer the above question. Listed in Table V.1 are the 26 sites with their Cal Tech identification number as well as local site conditions. The local site conditions, either rock, stiff soil or deep cohensionless soil, were taken from Seed et al[4] and Idriss and Power[3]. Note, that these site classifications are not available for all sites.

The two original horizontal velocity time histories for each site were transformed into a radial and a tangential time history. The maximum ground velocities in the radial and tangential directions were then determined. The same procedure was used in processing the acceleration time histories. Presented in Table V.2 are the average values of the ratios V_{rad}/V_{tan} and A_{rad}/A_{tan} where V_{rad} and A_{rad} are the maximum ground velocity and acceleration in the radial direction while V_{tan} and A_{tan} are the same quantities for the tangential component. Note, that all the average ratios are close to 1.00 indicating that their is little or no difference between the radial and tangential components. Considering the two components as paired data, there is no statistical difference at the 0.05 significance level between the radial and tangential maximums.

Berrill(1) arrived at a similar conclusion after studying a different set of San Fernando records. Comparing the Fourier amplitude of acceleration, he found that there was no apparent difference between the radial and tangential components. He attributes this to scattering caused by propagation path inhomogeneities and the fact that the San Fernando rupture mechanism had almost equal components of strike-slip and dip-slip dislocations.

V.3 Constant Wave Shape Assumption

The simplified design procedure for seismic wave propagation is based upon the assumption that the soil and pipe move together and that the seismic wave shape remains constant as it traverses the pipeline. This wave shape assumption was investigated using ground motions recorded during the 1971 San Fernando Earthquake. Axial strains were calculated assuming the wave shape remain unchanged and then compared to actual values. The axial strain as opposed to curvature was used for the comparison because axial strain is the larger of the two effects.

A total of 18 pairs of ground displacement time histories were used in the comparison. Each pair of ground displacements are for two nearby stations which lie roughly along the same radial line from the epicenter. Of course, different pairs of stations may lie along different radial lines.

The two horizontal ground displacement components for each station were used to generate a radial ground displacement time history using standard coordinate transformation techniques. Because recording stations triggered independently during the San Fernando Earthquake, the time between the start of the record and the arrival of a particular wave was different for different records. To eliminate the effect of independent triggering the cross correlation function $R_{xy}(\eta)$ for each pair of stations was calculated.

$$R_{xy}(\eta) = \frac{1}{NT} \sum_{t=1}^{NT} X_1(t) \cdot X_2(t+\eta) \qquad (V.1)$$

where $X_1(t)$ is the digitized radial displacement time history for station 1, $X_2(t)$ is the digitized radial displacement time history for station 2, η is the time delay and NT is the number of points in the digitized record. The value of the time delay for which the cross correlation function is a maximum, η_{max}, was then determined.

$$R_{xy}(\eta_{max}) \geq R_{xy}(\eta) \qquad (V.2)$$

η_{max} is then the amount of time by which record 2 must be shifted in order that the wave shape at station 1 and 2 match as closely as possible.

Three strains were calculated for each pair of stations as a function of the propagation speed of the seismic waves. The difference in epicentral distance between the two points was calculated from information available in the Cal Tech records. Since each pair of stations lie roughly on the same epicentral line, the difference in epicentral distance is essentially the same as the separation distance between the two points.

The first strain value, $\varepsilon_1(\xi)$, corresponds to the maximum value of the average axial strain between the two stations assuming the wave shape remains constant.

$$\varepsilon_1(\xi) = \frac{|X_1(t) - X_1(t - \xi)|_{max}}{L_{12}} \qquad (V.3)$$

where X(t) is the radial ground displacement at station 1, L_{12} is the separation distance between station 1 and station 2 and ξ is the time required for the waves to propagate from station 1 to station 2. For any assumed wave propagation speed, C, the time lag is:

$$\xi = \frac{L_{12}}{C} \qquad (V.4)$$

The second strain value, $\varepsilon_2(\xi)$, corresponds to the maximum value of the average strain between the two stations using the actual radial displacement time histories of the two stations.

$$\varepsilon_2(\xi) = \frac{|X_1(t) - \hat{X}_2(t - \xi)|_{max}}{L_{12}} \qquad (V.5)$$

where $\hat{X}_2(t)$ is the radial displacement time history of station 2 modified for the effect of independent triggering of the seismigraphs.

$$\hat{X}_2(t) = X_2(t + \eta_{max}) \qquad (V.6)$$

The third strain is that predicted by the simplified approach,

$$\varepsilon_3(\xi) = V_{max}/C \qquad (V.7)$$

where V_{max} is the maximum ground velocity of the radial component at station 1 that is

$$V_{max} = |dX/_{dt}|_{max} \tag{V.8}$$

Substitution of Eqn. (V.4) into Eqn. (V.7) yields

$$\varepsilon_3(\xi) = \frac{V_{max} \cdot \xi}{L_{12}} \tag{V.9}$$

It should be noted that $\varepsilon_3(\xi)$ is the maximum strain <u>at a point</u> assuming the wave shape remains constant while $\varepsilon_1(\xi)$ is the maximum value of the <u>average strain between two points</u> assuming the wave shape remains constant. If the wave shape remains constant, the maximum strain at a point is an upper bound for the maximum average strain.

Assuming various values for the propagation speed, C, the three strains $\varepsilon_1(\xi)$, $\varepsilon_2(\xi)$ and $\varepsilon_3(\xi)$ were computed for 18 pairs of records. Listed in Table V.3 are the names, soil types[3,4] and approximate distance between the stations used in this comparison. Shown in Fig. (V.1) is typical log-log plot of the three strains for stations R253 and F089 as a function of the time lag ξ. Notice that the third strain, $\varepsilon_3(\xi)$, increases linearly with ξ as expected by Eqn. (V.9) and ε_3, is larger than the first strain, ε_1 except at $\xi = 0.1$ seconds where they are equal, that is

$$\varepsilon_3(\xi) > \varepsilon_1(\xi) \quad \text{for } \xi > 0.1 \text{ sec.} \tag{V.10}$$

and

$$\varepsilon_3(0.1) = \varepsilon_1(0.1) \tag{V.11}$$

Since displacement time histories digitized at 0.1 seconds were used, it is expected that the strains ε_1 and ε_3 are equal for a time lag of

0.1 seconds.

The effect of changes in the wave shape from station R253 to F089 is quantified by comparing strain $\varepsilon_1(\xi)$ and $\varepsilon_2(\xi)$. Recall that $\varepsilon_1(\xi)$ is the maximum value of the average strain between the two stations assuming the radial displacement time history at R253 and F089 are identical while $\varepsilon_2(\xi)$ is the maximum value of the average strain using the <u>actual</u> radial displacement time histories at R253 and F089. For low values of the time lag, ε_2 is approximately 2.5 times larger than ε_1. For a time lag of approximately 0.70 seconds, all three strains are about equal while for a time lag greater than about 1.0 sec. ε_1 and ε_2 tend to level off at a constant value which is less than the value of ε_3. This leveling off of the ε_1 value for large time lags was noted by Christian[2]. It is related to the fact that the maximum value of ε_1 is limited by twice the maximum ground displacement divided by the separation distance.

$$\varepsilon_1(\xi) < \frac{2|X_1(t)|_{max}}{L_{12}} \qquad (V.12)$$

As can be seen from Figure V.1, the change in the wave shape can have a significant effect upon the maximum average strain for low values of the time lag ξ.

In order to be able to include all 18 pairs of records in this comparison, the strains for each pair of stations were normalized by dividing by $\varepsilon_1(0.1)$ for that pair, that is

$$\hat{\varepsilon}_j(\xi) = \varepsilon_j(\xi)/\varepsilon_1(0.1) \text{ for } j = 1,2,3 \qquad (V.13)$$

Now the first and third normalized strain for each pair of stations evaluated for a time lag of 0.1 sec. equals 1.0, that is, from Equation (V.11) and (V.13).

$$\hat{\varepsilon}_1(0.1) = \hat{\varepsilon}_3(0.1) = 1.0 \tag{V.14}$$

The normalized strain for all 18 pairs of records were then averaged. The resulting average normalized strains are plotted in Figure V.2. Note that the average of all 18 pairs of stations has the same general form as Figure V.1.

Figure V.2 could serve as the basis for the design of a pipeline for wave propagation effects. For small time lags, less then approximately 0.60 seconds, the maximum value of the average strain between the two stations given by ε_2, would be used. For large time lags, greater than 0.60 seconds, the maximum strain at a point as given by ε_3 would be appropriate.

V.4 Summary

Certain characteristics of earthquake ground motion which effect the behavior of buried pipelines during earthquakes were studied in this Chapter. Comparison between the radial and tangential components indicated that there was no significant difference between the radial and tangential maximum ground velocity or acceleration in the near field. The effect of changes in the shape of the seismic waves as they traverse a pipeline was found to be significant. However, the effect is important only for high seismic wave velocities. This effect can be included in design by using Figure V.2.

V.5 References

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Table V.1 Sites Used in Study of Radial and Tangential Components

C.I.T. Number	Site	Local Site Conditions
G107	Athenaeum, Cal. Tech.	Deep
H115	15250 Ventura Blvd.	Stiff
C048	8244 Orion Blvd.	Deep
F088	633 E. Broadway, Glendale	
F104	Oso Pumping Plant	Stiff
H121	900 So. Fremont	Deep
J144	Lake Hughes Array #12	Stiff
J143	Lake Hughes, Array #9	Rock
D068	7080 Hollywood Blvd.	
S262	5900 Wilshire Blvd.	Stiff
D059	1901 Ave. of the Stars	
N188	1880 Century Park East	
E075	3470 Wilshire Blvd.	Stiff
R249	1900 Ave. of the Stars	
I134	1800 Century Park East	
S267	5260 Century Blvd.	
P217	3345 Wilshire Blvd.	Stiff
R253	533 So. Fremont Ave.	
F089	808 So. Olive Ave.	
S266	3550 Wilshire Blvd.	Stiff
S265	3411 Wilshire Blvd.	Rock
J142	Lake Hughes, Array #4	Rock
J148	616 S. Normandie Ave.	Stiff
G112	611 W. Sixth St.	
F098	646 S. Olive Ave.	
C054	445 Figueroa St.	Rock

Table V.2 Comparison of Radial and Tangential Components

Sites Considered	$(V_{rad}/V_{tan})_{avg}$	$(A_{rad}/A_{tan})_{avg}$
All	1.04	1.04
Rock Only	1.27	0.93
Stiff Soil Only	0.96	1.12
Deep Soil Only	0.93	0.98

Table V.3 Stations used in Investigation of Constant Wave Shape Assumption

STATION 1			STATION 2			Approximate Separation Distance (km)
CIT #	Name	Soil* Type	CIT #	Name	Soil* Type	
J148	616 S. Normandie	S	S265	3411 Wilshire	R	0.22
J148	616 S. Normandie	S	S266	3550 Wilshire	S	0.15
I134	1800 Century Park East		N188	1880 Central Park East		0.05
G112	611 W. Sixth		F098	646 S. Olive		0.19
K157	420 S. Grand		F098	646 S. Olive		0.37
N188	1880 Century Park East		R249	1900 Ave. Stars		0.19
I134	1800 Century Park East		R249	1900 Ave. Stars		0.23
C054	445 Figueroa	R	G112	611 W. Sixth		0.35
R253	533 S. Fremont		F098	646 S. Olive		0.58
C054	445 Figueroa	R	F098	646 S. Olive		0.58
F098	646 S. Olive		F089	808 S. Olive		0.21
G112	611 W. Sixth		F089	808 S. Olive		0.41
R253	533 S. Fremont		F089	808 S. Olive		0.76
C054	445 Figueroa	R	F089	808 S. Olive		0.75
D068	7080 Hollywood Blvd.		D057	Hollywood Storage	S	1.75
J142	Lake Hughes #4	R	J141	Lake Hughes #1	S	2.50
J141	Lake Hughes #1	S	O207	Fairmont Reservoir	R	2.77
L166	3838 Lankershim	R	D068	7080 Hollywood Blvd.		3.60

*Soil Type R = Rock
S = Stiff Soil

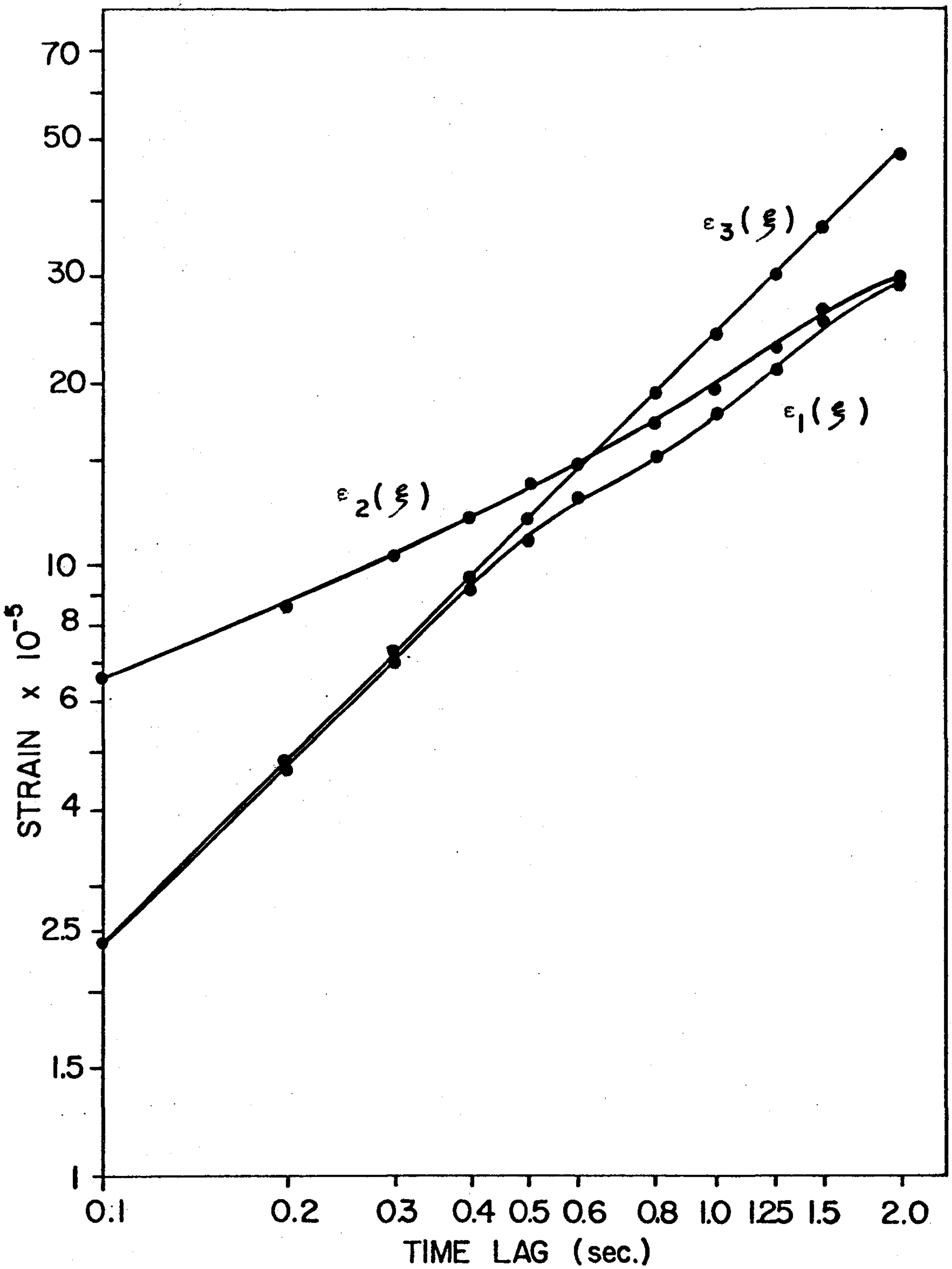

FIG. V.1 STRAINS FOR STATIONS R253 AND F089

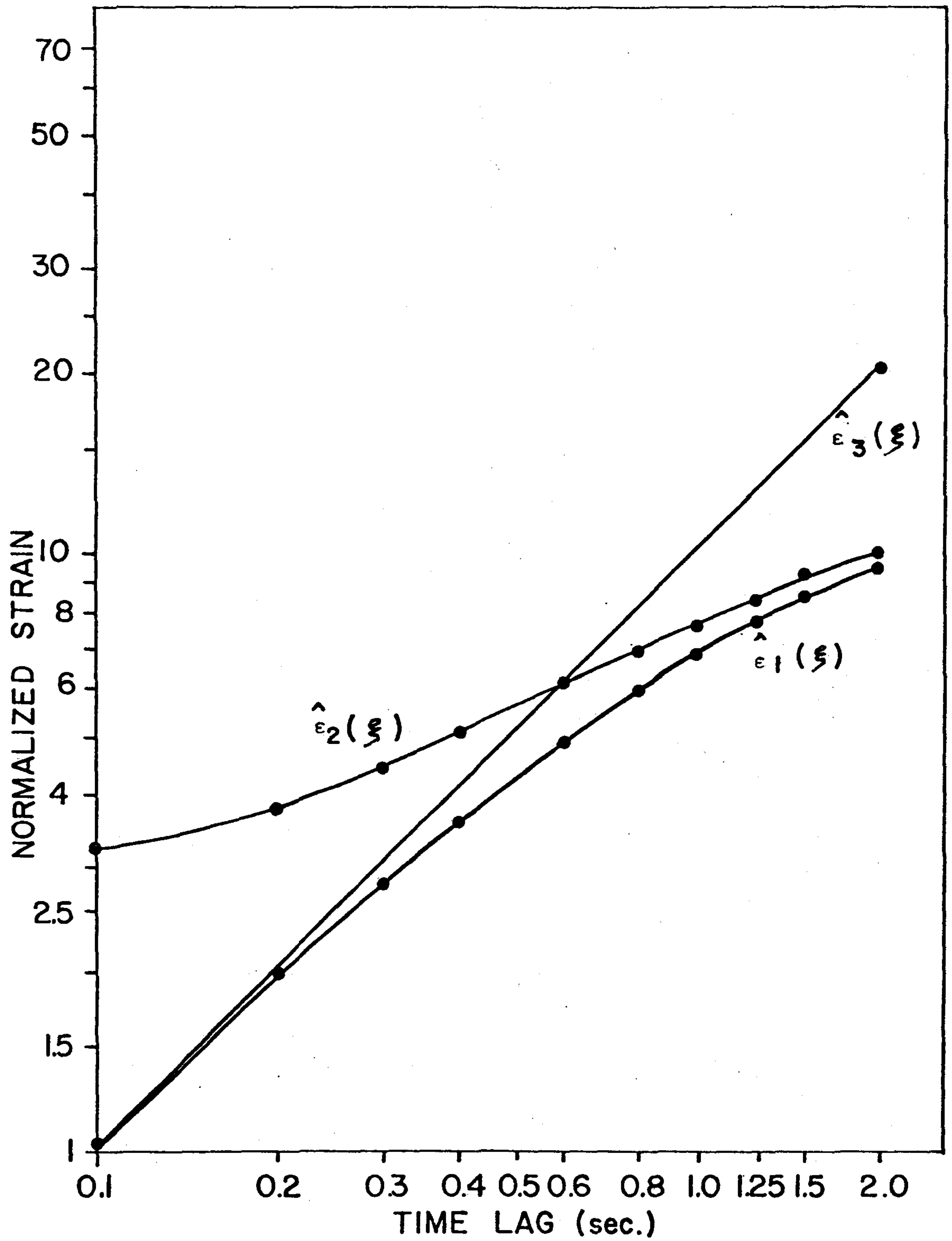

FIG. V.2 AVERAGE NORMALIZED STRAINS FOR ALL 18 PAIRS OF STATIONS

CHAPTER VI SEISMIC RISK ANALYSIS

VI.1 Preface

The chapter presents procedures for determining design values for the peak ground acceleration for a particular location. As shown in Chapter III, the peak acceleration and velocity are needed in the simplified procedure to determine the axial strain and curvature in the buried pipeline due to seismic activity. General seismic risk analysis procedures are discussed in this chapter with data obtained for the Albany, New York area. The results of the seismic risk analysis for Albany are used in Chapter IX which is a case study of the Latham Water District. It should be noted that peak ground acceleration values for particular return periods are available from other sources[1,2] for the United States. It is recommended that a detailed seismic risk analysis for a particular water system be undertaken only if the designer wishes to design for return periods other than those available from other sources[1,2] or if the designer wishes to include a probabilistic error term in the attenuation relationship. Neither the peak acceleration values developed by the Applied Technology Council[2] nor those developed by Algermissen[1] incorporate a probabilistic error term in the attenuation relationship.

The adequacy of any water/sewer system subject to earthquake excitation is a function of both the physical properties of the soil pipeline system itself and the size of the earthquake. While the physical properties of the soil pipeline system (pipe thickness, joint fixity, soil density, soil shear wave velocity) may be viewed as relatively deterministic quantities, the magnitude of the earthquake must be viewed in probabilistic terms. For instance, for a particular site a Richter magnitude 5.5 earthquake may occur on the average once every 25 years (annual risk ≈ 0.04) while a Richter magnitude 6.5 earthquake may occur on the average once every 50 years (annual risk ≈ 0.02).

In order to obtain these probabilities for a particular site, three elements are needed. First, the general seismicity of the area around the site must be determined. Specifically, the rate at which earthquakes of engineering significance occur in the area as well as whether they are due to point, line or area sources must be established. Secondly, a magnitude frequency relationship for the area is required to develop the probability density function of earthquake magnitudes. This allows one to determine the probability that an earthquake is larger than a particular magnitude given the fact that the earthquake has occurred. Finally, an attenuation relationship is needed which specifies the decrease in earthquake parameters such as maximum ground acceleration and maximum ground velocity, with increased distance between the site and the earthquake epicenter.

VI.2 Source Characteristics of Albany, New York Area

Point, line and area sources as well as microzonation are the four basic approaches to modeling earthquake sources. Line sources have been used[4,5,6] to represent an active fault while area sources[4,5,6] are most useful when the epicenters of the historical earthquake are fairly evenly distributed over the area of interest. For both the line and area source models, a uniform rate of occurrence along the line or throughout the area is usually assumed. The fourth approach, microzonation[9], divides a region into a group of area sources, each of which is modeled by a group of point sources within that region.

In order to determine the source characteristics as well as other parameters of the Albany, New York area, a list of historic earthquakes which have occurred in the Northeast[11,19] was compiled. The data for each earthquake consists of location, magnitude and/or intensity and date of occurrence. For many of the older earthquakes, the Modified Mercalli Intensity (MMI) is the only available measure of the earthquake.

The location of the epicenters of the earthquakes used are shown in Fig. VI.1. The magnitude of the earthquake is proportional to the size of the asterisk and the radius of the circle is 100 miles (160 kilometers) with its center at Albany. The radius of the source area was determined using attenuation relationships. The source area radius was established such that an earthquake occurring outside the source area would create a maximum ground acceleration at the Albany site which is small enough to be of no engineering significance.

From the data base, the largest recorded earthquake within 200 miles (322 Km) of Albany has a magnitude of 5.5 on the Richter scale. Using five attenuation relationships developed by other researchers and using 0.02g maximum ground acceleration as the cut-off point for earthquakes of engineering significance, Solla[13] has shown that the radius of the area source may be conservatively taken as 160 kilometers around Albany. Since the epicenters of the historic earthquakes within 160 kilometers of Albany had a relatively uniform distribution, and since there are no active faults in this region, a uniform source area of 160 kilometers was used in this study.

VI.3 Earthquake Occurrence Rate

The occurrence rate is a measure of the seismic activity of a region. More specifically, it is the average number of earthquakes per unit time per unit source area with a magnitude of engineering significance. Using the data base of historic earthquakes, Solla[13] has determined the variation of the occurrence rate, ν, with the radius of the source area and the variation of the occurrence rate with the time interval considered (i.e., considering earthquakes within the last 50 years, last 100 years, etc.). For a given time interval, the occurrence rate is relatively constant within 160 kilometers of Albany but increases with increasing radius beyond 160 kilometers.

For a given source radius, the occurrence rate decreases slightly as a time interval is increased. This is most likely due to incomplete reporting of past earthquakes. The occurrence rate, considering the last 100 years and an area source of 160 kilometer radius, was used in this study and has a value of 0.204×10^{-4} earthquakes per year per square kilometer. The lower bound for earthquakes of engineering significance is taken as 2.0 on the Richter scale.

Having determined the average occurrence rate, the Poisson model was used to establish the probability of having a specific number of earthquakes in a given number of years[12]. For an area source, the probability that J, the number of earthquakes in the area source in t years, is equal to j is given by

$$P(J=j) = \frac{e^{-\hat{\nu}t}(\hat{\nu}t)^j}{j!} \qquad j = 0,1.2,... \qquad \text{(VI.1)}$$

where $\hat{\nu}$ is the average occurrence rate for the region multiplied by the source area.

In general, earthquakes of engineering interest are those for which structural damage is possible. The occurrence of these earthquakes can be modelled using the Poisson process with an average occurrence rate of νp_y where p_y is the probability that the ground motion will exceed y at the site. The relationship then becomes

$$P(J=j) = \frac{e^{-p_y \nu t} (p_y \nu t)^j}{j!} \qquad j = 0,1,2,... \tag{VI.2}$$

where J = the number of earthquakes which cause a ground motion greater than y in a time interval of t years.

VI.4 Magnitude Frequency Relationship

Richter's relation is most commonly used to determine the cumulative distribution function of earthquake magnitudes. Richter's relationship is

$$\log_{10} N_m = c - bm \tag{VI.3}$$

where N_m is the number of earthquakes whose magnitude is greater than or equal to m while b and c are empirical constants which vary from region to region.

In general, upper and lower bounds are imposed on Richter's relationship[3,4,5,6,10,12,14,15,16,17]. The upper bound represents a magnitude which is improbable for a particular region while the lower bound represents the magnitude which is not of engineering significance. Using upper and lower bounds the modified version of Richter's relationship becomes

$$\log_{10} N_m = c + b\ (m - m_o) \qquad m_o \leq m \leq m_1 \tag{VI.4}$$

where m_o is the lower bound and m_1 is the upper bound on the magnitude. A lower and upper bound of 2.0 and 6.3 were used in this study.

The linear form of Richter's relationship is only an approximate fit to actual data. It has been suggested by Merz and Cornell[10] that a quadratic relationship be used. Due to the relative lack of data on earthquakes in the eastern U.S., the linear form of Richter's relation was used.

The cumulative distribution function of earthquake magnitudes may be derived from Richter's relationship and is presented below for the linear case where both an upper and lower bound on the magnitude are included.

$$F_M(m) = K_{m1}\,[1-e^{-\beta(m-m_o)}] \qquad m_o \leq m \leq m_1 \tag{VI.5}$$

where $K_{m1} = (1-\exp(-\beta(m_1-m_o)))^{-1}$ and $\beta = b \cdot \ell n10$ (VI.6)

Once the cumulative distribution function is obtained, the probability that the earthquake is of magnitude greater than m given the fact that an earthquake has occurred, is equal to $1 - F_M(m)$.

Presented in Fig. VI.2 is a plot of Richter magnitude, M, vs the cumulative number of earthquakes with a magnitude greater than M for the Albany source area. The slope of this curve, b, is obtained using the method of least squares. Then the parameter β needed for the cumulative distribution may be calculated using Eqn. (VI.6).

For the Albany area, magnitudes between 2.0 and 4.5 were used to determine the slope of the frequency magnitude relationship. Earthquakes with magnitudes of less than 2.0 are normally not felt and therefore, many of these were not reported. Earthquakes of magnitude greater than 4.5 are relatively rare for the source area and are also excluded from the calculation of β.

Solla[13] presents a plot of β vs time interval for a constant source radius of 160 km. The time interval is the number of previous years considered, i.e., a time interval of 100 years corresponds to using only the past 100 years of earthquake data. The values of β tend to decrease as the time interval increases. The value for β for a 100 year interval with an area source radius of 160 kilometers is 1.5 and this is the value used herein. It should be noted that the value for β suggested by Algermissen and Perkins[1] ranges between 1.35 and 1.54 for the Northeastern U.S.

VI.5 Attenuation Relationship

For most engineering applications, the maximum ground acceleration, velocity, and/or displacement are important design parameters. An attenuation relationship relates the earthquake magnitude and the distance from the site to these design parameters. The most common relationship has the form:

$$A_{max} = b_1 \exp(b_2 M) \, (S)^{-b_3} \tag{VI.7}$$

where A_{max} = the maximum ground acceleration at the site; M = the magnitude of the earthquakes; S = distance from the epicenter of the earthquake to the site of interest and b_1, b_2, b_3 are empirical constants.

When actual data is examined, the spread of data points around the attenuation is quite large. Some authors[18] have suggested attenuation relationships which account for site conditions as well. Other authors [3,7,10,17] have suggested the use of an error term. This changes the attenuation relationship to:

$$A_{max} = b_1 \exp(b_2 M) \, S^{-b_3} \, e \tag{VI.8}$$

where the natural log of e is normally distributed with a mean of zero and a standard deviation of s. This error term accounts for the spread in the data due to varying site conditions and other variations in the data. Typical values for s range from 0.5 to 1.0[3,6,17].

VI.6 Evaluation of Seismic Risk - Deterministic Attenuation

Using the deterministic attenuation relationship, Eqn. (VI.8) the probability that an earthquake will produce a maximum ground acceleration, A_{max}, greater than $\bar{A}$ at the site given that an earthquake has occurred at a distance S from the site is[17]:

$$P[A_{max} \geq \bar{A}|S] = 1 - K_{m1}[1 - cG_1(\bar{A}/b_1)^{-\beta/b_2}] \qquad \text{(VI.9)}$$

where $c = \exp(\beta m_o)$

$$G_1 = S^{-\beta b_3/b_2}$$

Assuming the occurrence rate is uniform throughout the source area, Equation (VI.9) may be integrated over the source area yielding:

$$p_y = P[A_{max} \geq \bar{A}]$$

$$p_y = (1 - K_{m1}) + \frac{2}{S_y^2 - D_f^2} K_{m1}\, cb_1^{\beta/b_2} \cdot \bar{A}^{-\beta/b_2} G_2 \qquad \text{(VI.10)}$$

where

$$G_2 = (D_f^{-\beta b_3/b_2+2} - S_y^{-\beta b_3/b_2+2})/(\beta b_3/b_2-2)$$

in which $\bar{A}$ is a given acceleration, p_y is probability that $\bar{A}$ will be exceeded, D_f is focal depth of the earthquakes, S_y is radius of source area, m_1 and m_o are the upper and lower bounds on earthquake magnitude.

Assuming a Poisson process, the probability that the ground motion will exceed a level $\bar{A}$ in t years is:

$$P[A_{max} \geq \bar{A}] = 1 - \exp(-p_y \cdot \nu \cdot t) \qquad \text{(VI.11)}$$

The annual risk is the probability that the ground motion will exceed a given acceleration A_{max} in one year or:

$$\text{Annual risk} = 1 - \exp(-p_y \cdot \nu) \qquad \text{(VI.12)}$$

VI.7 Evaluation of Seismic Risk - Probabilistic Attenuation

Use of the probabilistic attenuation relationship, Eqn. (VI.8) causes an increase in the associated probabilities and annual risk. A detailed description of the applicable equations for a probabilistic attenuation relationship is found in References (13) and (17).

VI.8 Application of Procedure To Albany, New York

In this section, the procedures described previously will be used to determine the seismic risk of Albany, New York and the effect of various input parameters on the seismic risk. From the data base of historical earthquakes, a range of possible values for both β and ν were established and it has been shown[13] that changes of these parameters within that range has a negligible effect upon the overall seismic risk. Unfortunately, seismic risk is greatly dependent upon the attenuation coefficients (b_1, b_2, b_3) and upon whether the probabilistic error term is included in the attenuation relationship.

Little data is available on earthquake attenuation in the Eastern United States. Hence, the coefficients for the attenuation relationship, Equation (VI.8), were determined by a search of current literature. Most of these relationships were derived for the West Coast[4,7,8,12] while only a few are applicable to the East Coast[7,15]. Since the East Coast is thought to have a distance attenuation coefficient b_3 of approximately half that of the West Coast[1], only the relationship for the Eastern U.S. was examined in detail in this study. Since Donovan's relationship[7] yields more conservative results, his values for b_1, b_2, b_3 will be used in this report.

$$A_{max} = 1100\ e^{0.5M}\ (S+25)^{-1.32} \qquad \text{(VI.13)}$$

where A_{max} is maximum ground acceleration in cm/sec^2, M is the Richter's magnitude and S is the distance in kilometers from the epicenter of the earthquake to the site.

The effect of including the probabilistic log-normal error term in the attenuation relationship is shown in Fig. VI.3. This graph presents the annual risk for various peak acceleration values for Albany using the deterministic attenuation relationship (no error term) and using the probabilistic attenuation relationship with the standard deviation of the log-normal error term, s, taking values of 0.5 and 1.0. Note that inclusion of the error term increases the associated annual risks. Henceforth, in this study, the standard deviation of the log-normal error term is taken as 0.75 which is the midpoint of the range of suggested values of s[2,16].

VI.9 Recommended Values

The recommended peak acceleration for various return periods are presented in graphical form in Fig. VI.4 and in tabular form in Table VI.1. The annual risk presented in Fig. IV.4 may be used to calculate the probability that a particular maximum ground acceleration will be exceeded in a given number of years. If the annual risk of a particular maximum acceleration, $\bar{A}$, is q_a, the probability that the acceleration will not be exceeded in T years is given by

$$P(A_{max} \leq A) = (1 - q_a)^T \qquad \text{(VI.14)}$$

This information is presented in Figure VI.5 and in tabular form in Table VI.2. For example, a maximum ground acceleration of 228 cm/sec^2 has a 1 in 10 chance of being exceeded in 50 years, while a maximum ground acceleration of 205 cm/sec^2 has a 1 in 5 change of being exceeded in 100 years.

Procedures for calculating the peak ground velocity and for including the effect of local soil conditions on the peak ground acceleration values are presented in Chapter VII of this report.

VI.10 Summary and Conclusions

The seismic risk of Latham, New York is presented in terms of annual risk, average return periods and probabilities of exceedance. The average occurrence rate, ν, for the area as well as β, the parameter specified from the magnitude frequency relation, were determined using a list of historic earthquakes for the area. A conservative attenuation relationship for the Eastern United States with a probabilistic error term was also used. It is felt that the seismic risk values recommended in this chapter are appropriate for engineering purposes.

VI.111 References

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TABLE VI.1 RECOMMENDED PEAK GROUND ACCELERATION VS RETURN PERIOD FOR ALBANY

RETURN PERIOD (Years)	MAXIMUM GROUND ACCL. (cm/sec^2)
10	40
25	65
50	90
100	125
200	160

TABLE VI.2 RECOMMENDED MAXIMUM GROUND ACCELERATION FOR SPECIFIC EXCEEDANCE PROBABILITIES AND ECONOMIC LIFETIMES

ECONOMIC LIFETIME T (years)	MAXIMUM GROUND ACCELERATION (cm/sec^2) HAVING PROBABILITY p OF BEING EXCEEDED IN T YEARS		
	$p = 0.05$	$p = 0.10$	$p = 0.20$
25	225	180	152
50	270	228	178
100	330	265	205

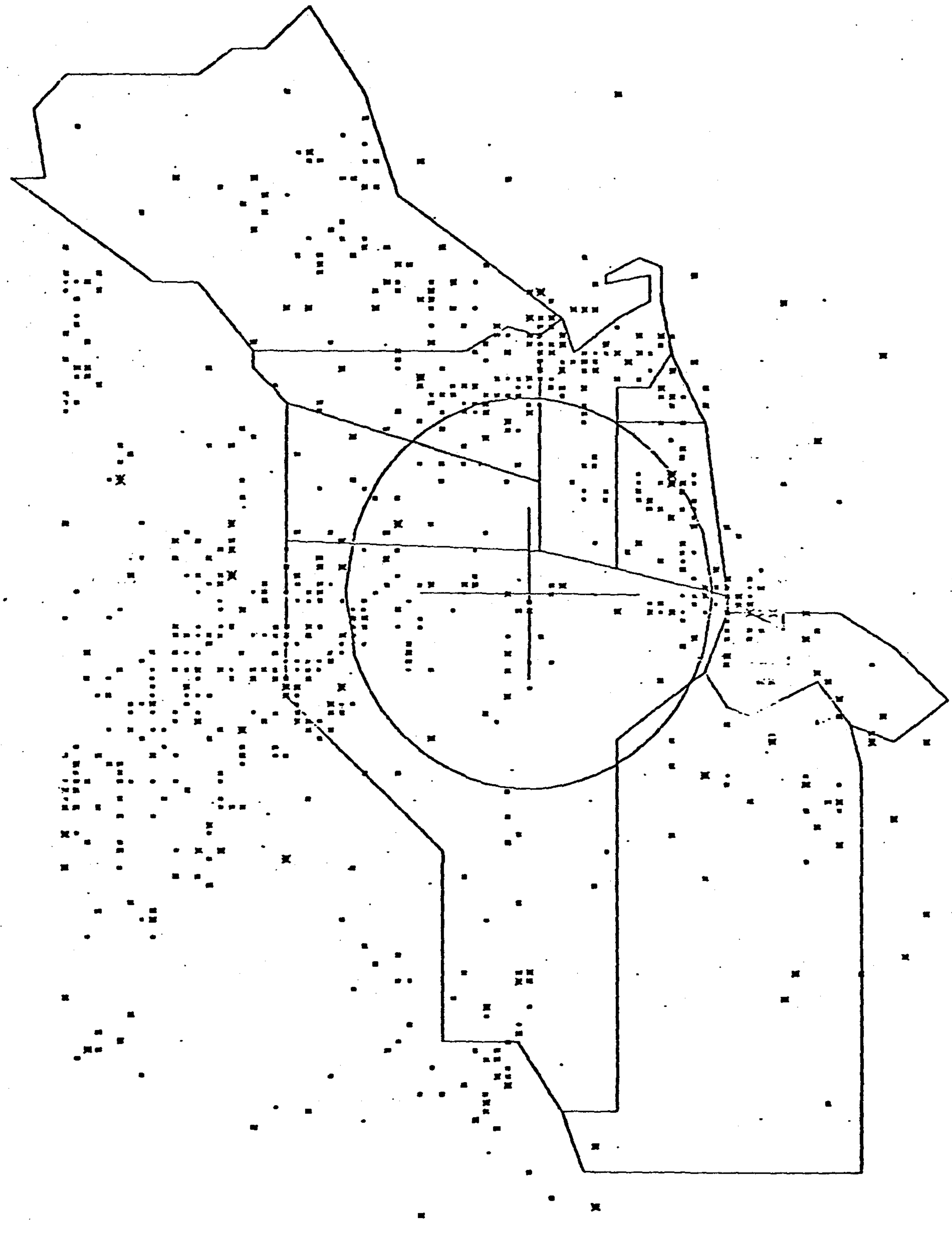

FIG. VI.1 PLOT OF HISTORIC EARTHQUAKES - NORTHEASTERN U.S.

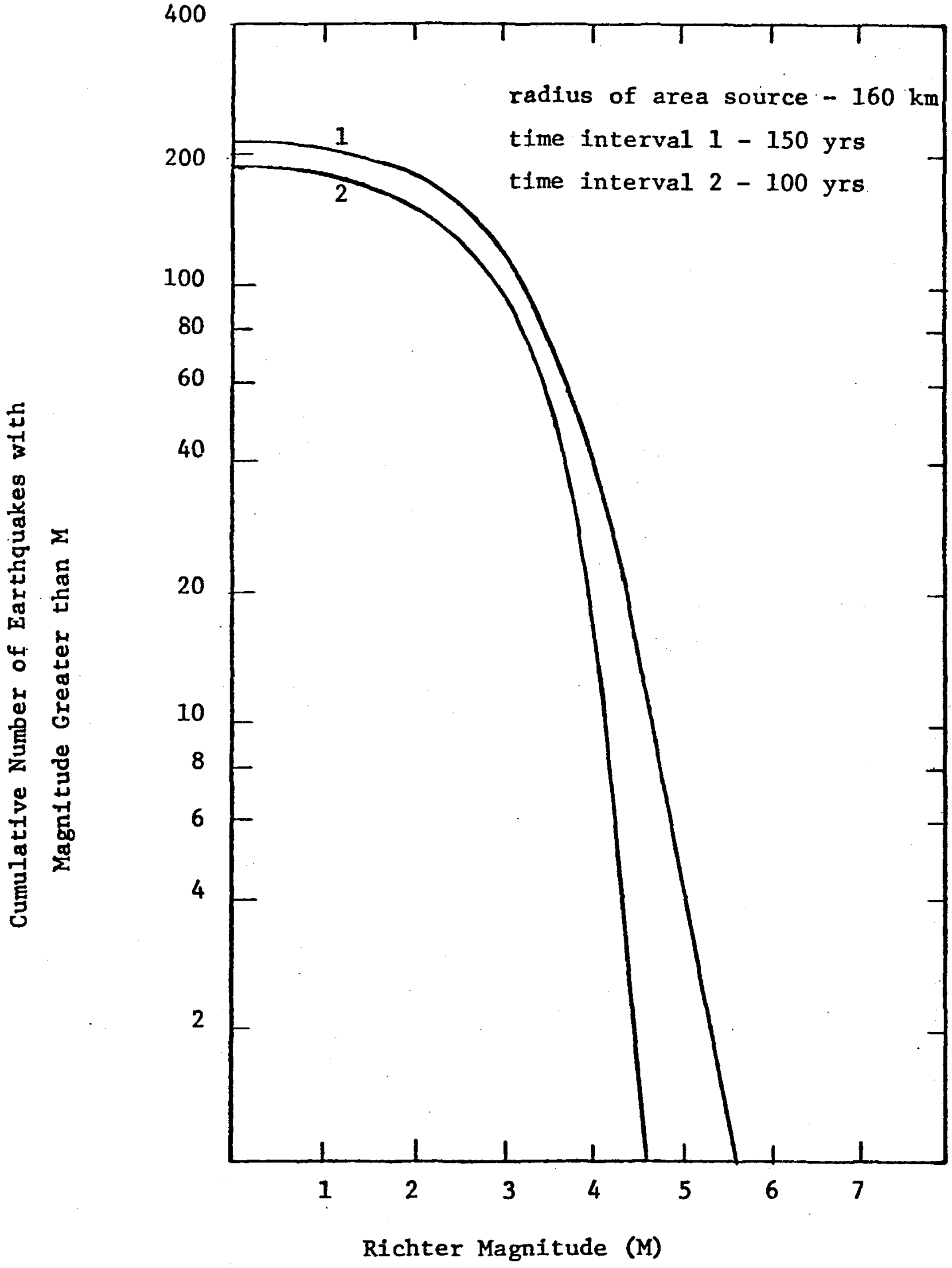

FIG. VI.2 MAGNITUDE FREQUENCY RELATIONSHIP

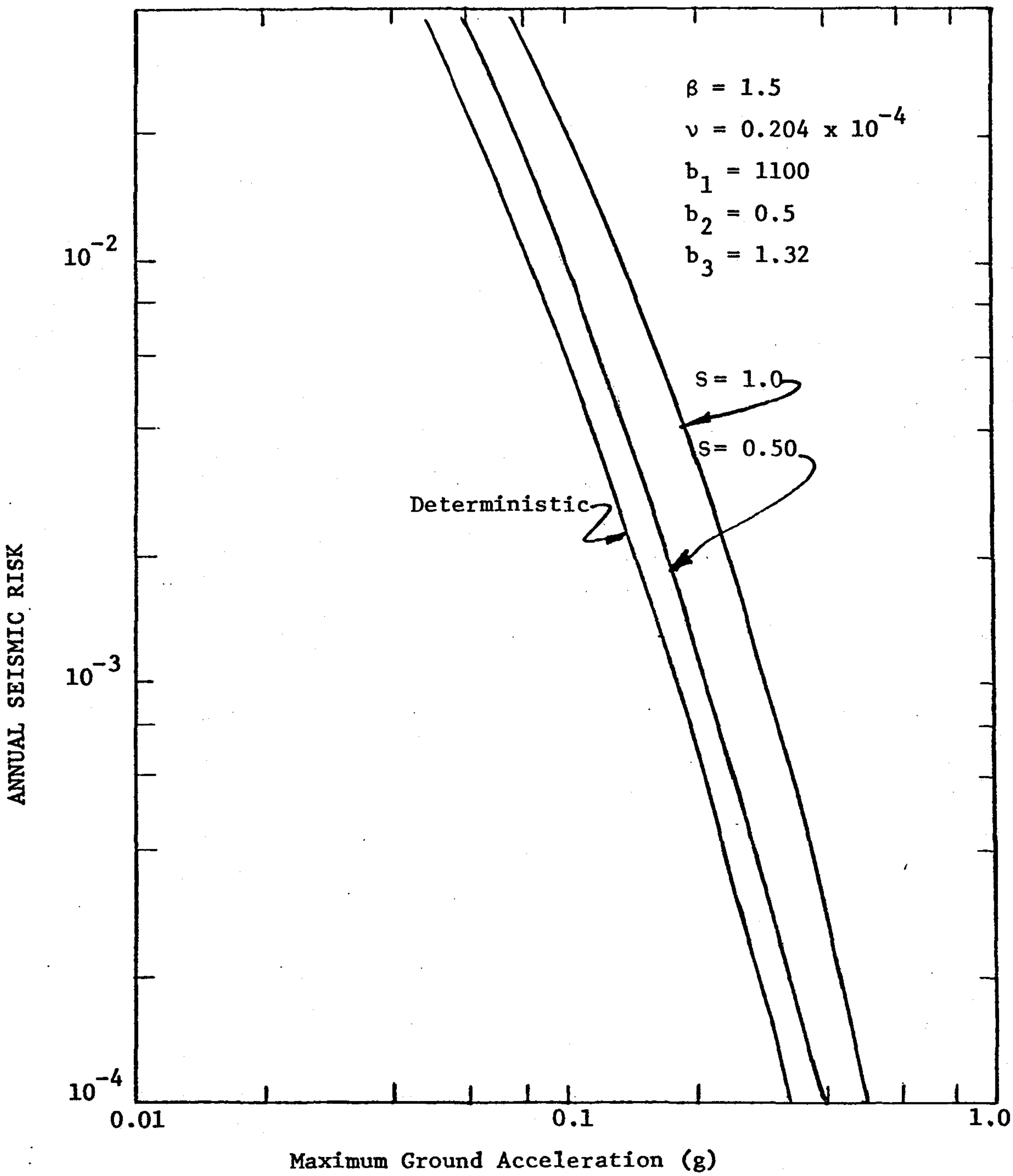

FIG. VI.3 EFFECT OF PROBABILISTIC ERROR TERM UPON SEISMIC RISK

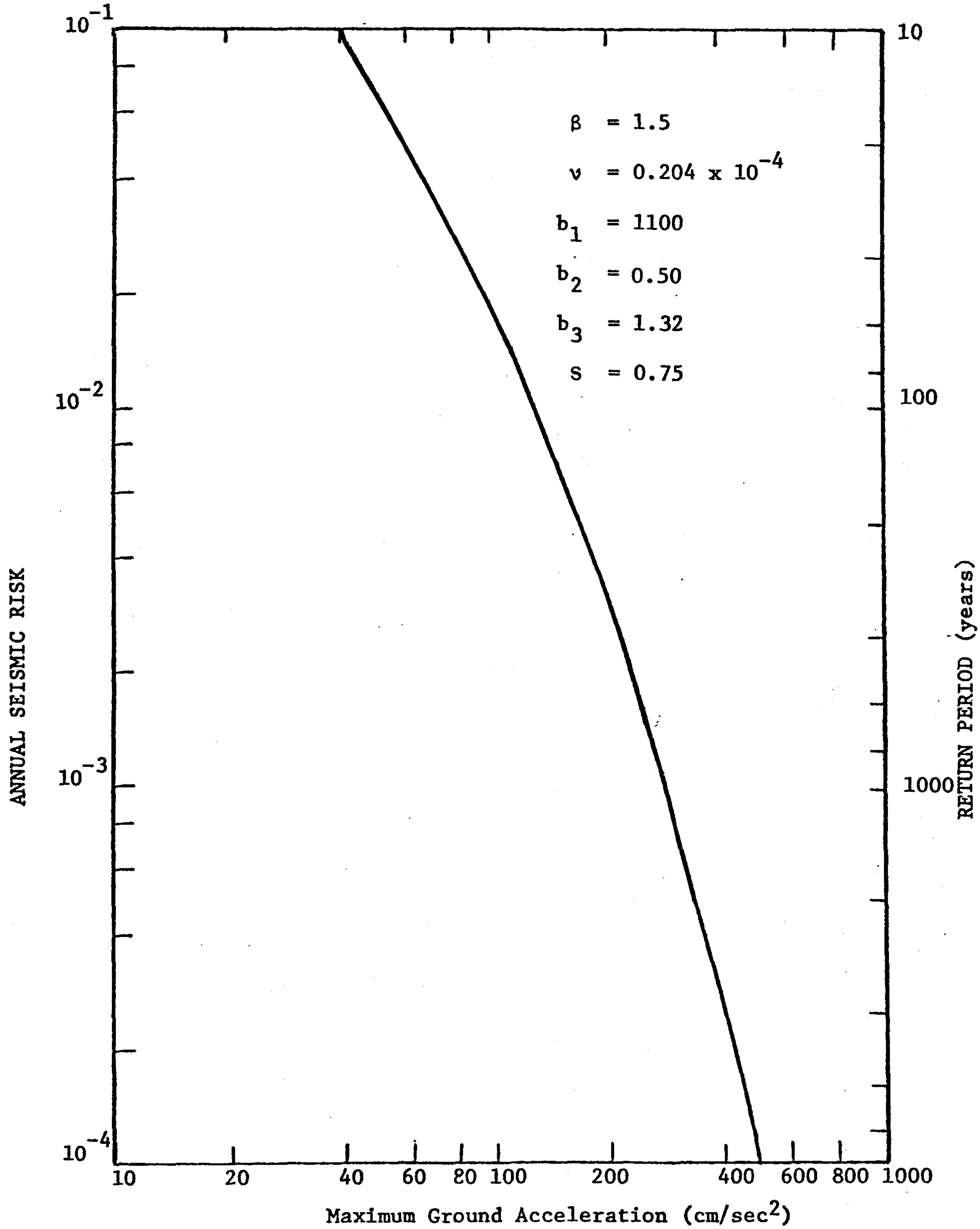

FIG. VI.4 RECOMMENDED ANNUAL SEISMIC RISK AND RETURN PERIODS FOR LATHAM, N.Y.

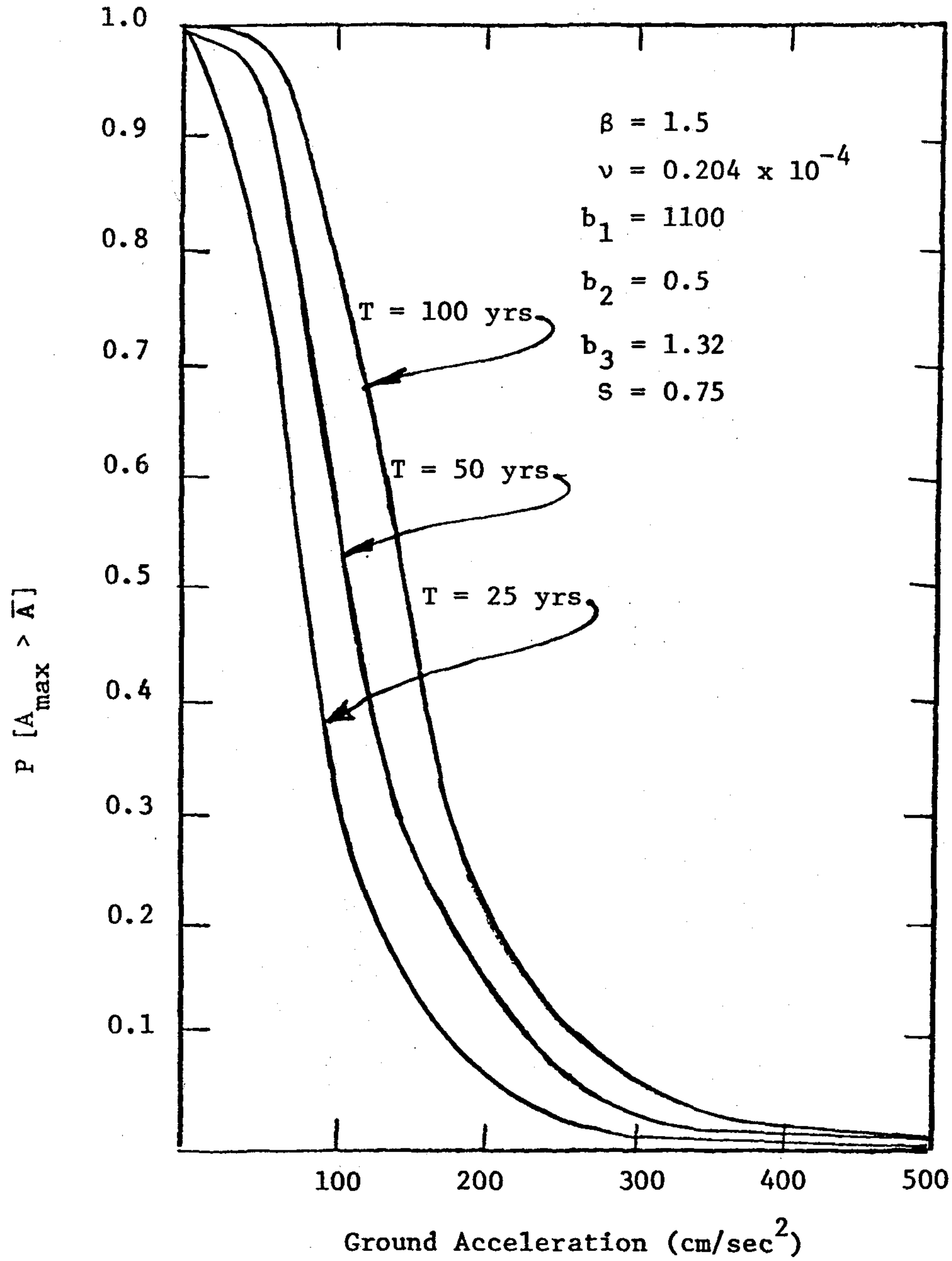

FIG. VI .5 PROBABILITIES OF EXCEEDING PARTICULAR GROUND ACCELERATION IN T YEARS

CHAPTER VII PERTINENT SOIL PARAMETERS

VII.1 Preface

This chapter briefly discusses the soil parameters that are pertinent and used in this report. A detailed investigation of these parameters, however, is outside the scope of this report.

The pertinent soil parameters affecting the seismic response of buried pipelines are the soil resistance to axial motion of the pipeline and the propagation velocities of the seismic waves with respect to the pipeline. The wave velocities are used directly in the "Simplified Analysis" approach to determine the upper bound pipe strains and curvatures. In the "Quasi-Static Analysis" approach, the wave propagation velocity is used to determine the delay time for the seismic wave between two points and the axial soil resistant spring constants are needed to study the soil-structure interaction effects.

Parameters such as wave propagation velocities for the "simplified approach" can be approximated using published data. Values for the longitudinal soil-structure resistant constants are not readily available in the literature and must be obtained from experimental studies.

VII.2 Soil Resistance to Axial Deformation of Pipe

VII.2.1 General

If a buried pipeline deforms in the longitudinal direction, the resistance to such motion develops from the surrounding soil medium. This soil resistance will reach a plateau when either the pipe slips at the soil pipe interface or the soil material yields near the pipe surface.

In a simple linearly elastic perfectly plastic model (Fig. VII.1), the soil resistance constants of interest are a soil axial spring constant, K_a, and a maximum soil resistant stress, called the soil slippage stress, Γ_a.

Initially, one might think that there is a large amount of published information on the resistance of the soil to buried pipe motion in the axial direction. Surprisingly, very little information was found on the subject after an extensive literature search. The subject has not drawn attention until recently, when the earthquake damage and response of buried pipelines in the axial direction have been found to be predominant[12,17]. This section presents a summary of the limited information on the axial soil resistance available in the literature.

To supplement this limited information, this section also presents an analytical model to roughly determine the behavior of the resistance of an idealized soil medium to the axial motion of buried pipelines.

VII.2.2 K_a-Factors

The soil axial resistant spring constant, K_a is a proportionality constant used to determine the soil force resistant to the motion of a buried pipe element with a diameter, D, and length, dx, in the longitudinal direction. In this report, this factor is termed the "axial soil spring constant".

One hypothesis[11,21] is that for small strain conditions, K_a is independent of the diameter of the pipe. Mathematically, the soil resistant force dF_o, would then be expressed:

$$dF_o = K_a \cdot u_o \cdot dx \tag{VII.1}$$

where u_o is the relative pipe displacement with respect to the ground and K_a is the axial soil spring constant with units of force/unit length/unit displacement.

In writing the equation of motion, several investigators[4,12,16,17] have used this axial soil resistant spring constant, K_a, in their formulations. However, most of these investigators did not supply or discuss a physical value for K_a. Only Tamura and Okamoto[17,18], in studying the seismic response of a tunnel, have used a K_a value ranging from 8.5-11.5 kips/in/in (6-8 kN/cm/cm) without further explanation. O'Rourke and Wang[11] used $K_a = 2G$ in an analysis where G was the shear modulus of soil.

Recently, Novak et al.[4,9] have analytically shown the axial soil resistant spring constant K_a as:

$$K_a = 2\pi G \cdot \alpha \tag{VII.2}$$

where G is the shear modulus of the soil medium and α is a constant depending on a number of parameters such as; load area to pipe length ratio; buried depth to diameter ratio; and Poisson's ratio. However, the manner in which the information is presented makes it somewhat difficult for an engineer to select a practical value for use in daily analysis and design problems.

To supplement the discussion of the K_a-factor with respect to other basic soil parameters, a simple analytical model (Fig. VII.2) is proposed.

It is assumed that the pipeline is buried in an infinite elastic medium. Upon a displacement of the pipeline, the displacement of soil in the axial direction, u, is assumed to be:

$$u = u_o \, c^{(1 - \frac{r}{R})} \tag{VII.3}$$

where u_o is the displacement of the pipe; r is a variable in the normal direction; R is the radius of the pipe; and c is an arbitrary constant whose magnitude has no effect on the problem.

After differentiating Eqn. (VII.3), one obtains the shear strain in the soil medium as

$$\gamma = -\frac{du}{dr} = \frac{u_o}{R} c^{(1 - \frac{r}{R})} \tag{VII.4}$$

and the shear stress:

$$\tau = G\gamma = \frac{G\,u_o}{R} c^{(1 - \frac{r}{R})} \tag{VII.5}$$

where G is shear modulus of soil. When r = R, the shear stress at the interface between the pipe and the soil is:

$$\tau_o = G\,u_o/R \tag{VII.6}$$

(note that the arbitrary constant, c, drops out of the equation).

The equilibrium over a small length on the surface of the pipe is:

$$dF_o = 2\pi\,R \cdot \tau_o \cdot dx \tag{VII.7}$$

Substituting τ_o, (Eqn. VII.6) into Eqn. VII.7), one will obtain the K_a-factor

$$K_a = dF_o/dx/u_o = 2\pi G \tag{VII.8}$$

which is a special case of Novak's development if $\alpha = 1$ and supports the hypothesis that K_a is independent of diameter for small strain.

Other investigators[3,19] have used the modulus of subgrade reaction, k_a, as a basis for determining the soil resistant spring constant as:

$$K_a = \pi\,D \cdot k_a \tag{VII.9}$$

where D is the diameter of the pipe.

Note that k_a has units of force/unit surface area/unit displacement and is normally established by tests. For a given k_a, from Eqn. (VII.9), the axial soil resistant spring constant is directly proportional to the diameter of the pipe. In an earlier paper[19], Wang has used a k_a ranging from 8-36 lbs/in^2/in (2-10 N/cm^2/cm) in a parametric study of vibration frequencies of buried pipelines.

The question whether K_a factor is dependent upon the pipe diameter must be verified by experimental evidence. At the present time, some experimental data has been gathered[1,20] but the results are not conclusive.

VII.2.3 Slippage Stress, Γ_a

Several investigators[3,5,7] have defined the maximum soil stress resistant to slippage of the pipe in the soil medium as the friction force over a small surface area of the pipe, dF. Mathematically, it is expressed as:

$$dF = \pi D \cdot p_h \tan \phi \cdot dx \qquad \text{(VII.10)}$$

in which D is the diameter of the pipe; p_h is the average normal soil pressure exerted against the wall of the pipe and ϕ is the angle of friction between the pipe and the surrounding soil or the internal angle of friction of the soil, whichever is smaller. Newmark[7] and others[5] further defined the average soil pressure as:

$$p_h = \frac{1 + k_o}{2} \gamma_w H \qquad \text{(VII.11)}$$

where H is depth of cover to the center of the pipe; γ_w is weight per unit volume of soil (net or submerged) and k_o is the coefficient of lateral earth pressure on the side of the pipe.

In this context, the maximum soil stress due to friction resisting the axial motion of buried pipelines would be

$$\Gamma_a = p_h \tan \phi \qquad \text{(VII.12)}$$

VII.3 Wave Propagation Velocities

As a conservative approximation, the wave propagation velocity resulting in pipeline curvature may be represented by C_s, the transverse wave velocity of the soil with respect to the pipeline, and the velocity resulting in axial strain may be represented by C_p, the longitudinal wave velocity. The actual speed of the seismic waves with respect to the pipeline is a function of the epicenter distance, focal depth as well as the material properties along the transmission path of the waves. Preliminary studies have indicated that the use of the wave speeds in the top soil layer is conservative. Further studies are required to determine the relationship for the apparent wave velocity relative to the pipeline for various angles of incidence.

Basic theory[8] indicates that transverse shear wave velocity, C_s, can be related to soil shear modulus of elasticity, G, and soil mass density, ρ, as follows:

$$C_s = \sqrt{\frac{G}{\rho}} \qquad \text{(VII.13)}$$

For a realistic range of soil parameters[8], C_p, the longitudinal pressure wave velocity, can be approximated as:

$$C_p = \sqrt{3}\, C_s \qquad \text{(VII.14)}$$

In lieu of a detailed soil analysis, an approximate value for G can be obtained using published relationships[6,8,10,13,14] relating Standard Penetration Resistance, N, to shear modulus G.

Ohaski and Iwasaki[10] have attempted to relate G to N using a direct relationship:

$$G = cN^b \qquad \text{(VII.15)}$$

where b and c are functions of the soil type.

Seed and Idriss[14] have presented an expression for cohesionless soils relating G, for various levels of shear strain, γ, to the soil relative density and overburden pressure:

$$G = 1000\ \kappa_2\ (\sigma_m')^{1/2}\ \text{psf} \tag{VII.16}$$

where σ_m' is the effective confining pressure and κ_2 is an empirical factor. Figure (VII.3) is a representation of their curve to evaluate κ_2 for sands as a function of relative density, D_r, and expected shear strain, γ. Relative density can be related to N, the Standard Penetration Resistance, and to the effective overburden pressure through the use of the "Gibbs-Holtz" type relationship reproduced from reference 12 as Figure (VII.4).

Standard Penetration Resistance or "blow count" data as well as boring logs are often available for an area from previous building, bridge, highway and utility construction. Hence, reasonable approximations can often be made for the shear modulus, G, and related wave velocities, C_s and C_p.

VII.4 Ground Acceleration and Velocity

To estimate peak ground velocity and acceleration it is first necessary to establish the probable peak ground acceleration in rock based on a seismic risk analysis. Knowing the peak acceleration in rock, the peak acceleration in soil can be determined using standard techniques[14,15].

Seed, et al.[15], have presented curves relating "maximum acceleration" to "maximum acceleration in rock" for four basic soil types. Figure (VII.5) is a representation of this relationship and assumes a layered system with little or no reflection or refraction at sloping interfaces. The effects of sloping rock-soil interfaces are presented in a paper by Dezfulian and Seed[2]. Data from finite element investigations of various sloping interface conditions are presented that allow for an approximation of acceleration amplification effects. It is noted that the amplification is dependent on the direction of wave propagation relative to the sloping boundary.

Once maximum ground accelerations have been established, maximum ground velocities can be estimated through the use of published relationships as presented by Newmark[8] and Seed[15] which are summarized in reference 15 and presented in this report as Table VII.1. The range of values (24-55 in/sec/g) is such that a reasonable estimate can be made for given local soil conditions.

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TABLE VII.1 - RELATIONSHIP BETWEEN MAXIMUM VELOCITY & MAXIMUM ACCELERATION FOR VARIOUS SOILS

GEOLOGIC CONDITIONS	V_{MAX}/A_{MAX}	REF.
ROCK	$26 \frac{IN/SEC}{g}$	SEED (14)
STIFF SOIL	$45 \frac{IN/SEC}{g}$	SEED (14)
DEEP COHESIONLESS SOIL	$55 \frac{IN/SEC}{g}$	SEED (14)
ROCK	$24 \frac{IN/SEC}{g}$	NEWMARK (8)
ALLUVIUM	$48 \frac{IN/SEC}{g}$	NEWMARK (8)

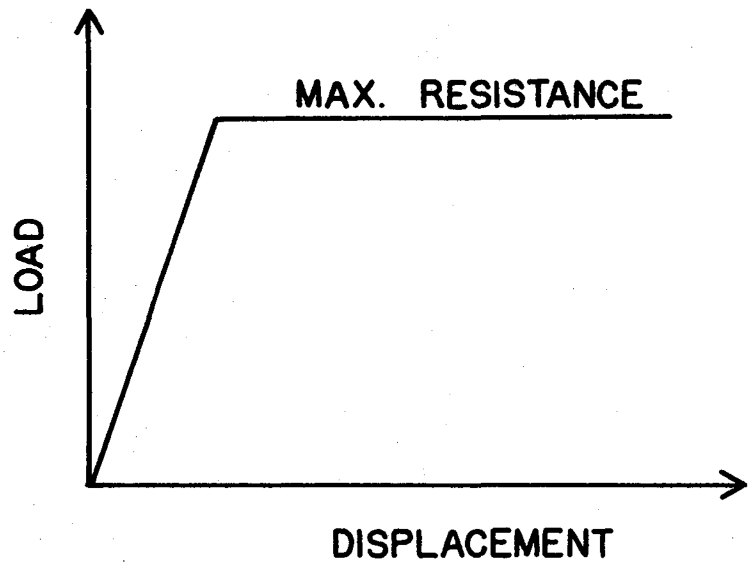

FIG. VII.1 IDEALIZED SOIL RESISTANCE TO AXIAL PIPE MOTION

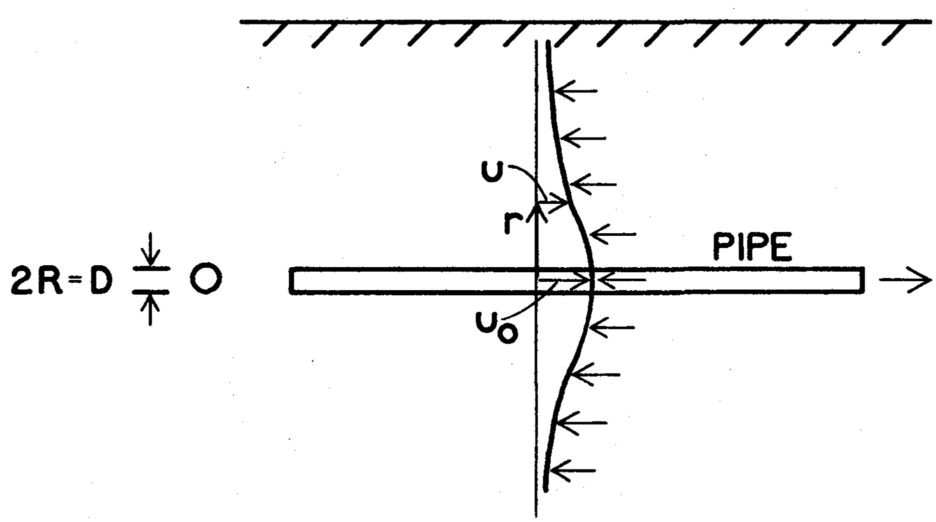

FIG. VII.2 SOIL RESISTANCE TO PIPE MOTION

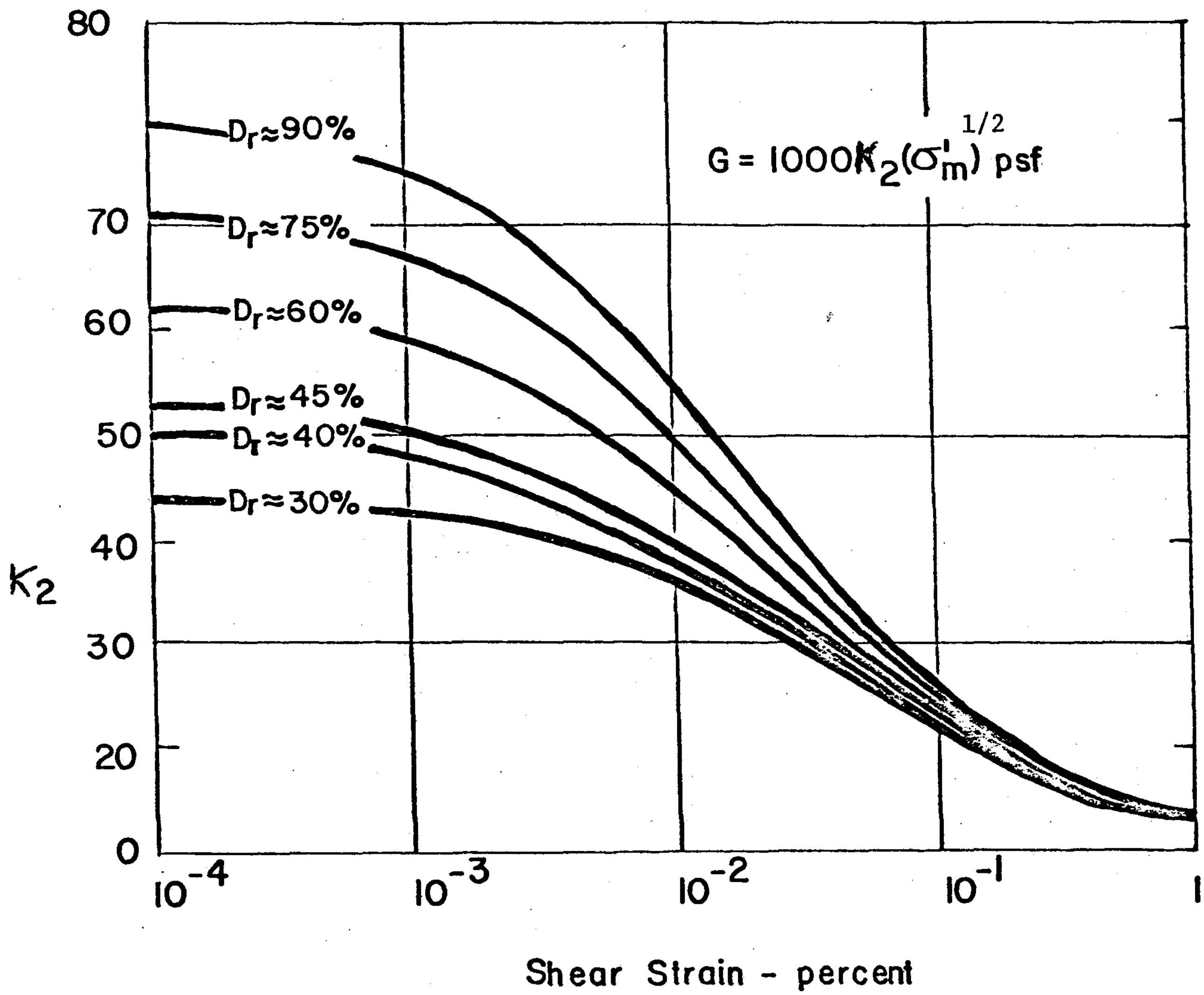

FIG. VII.3 SHEAR MODULI OF SANDS AT DIFFERENT RELATIVE DENSITIES (After Seed & Idriss (14))

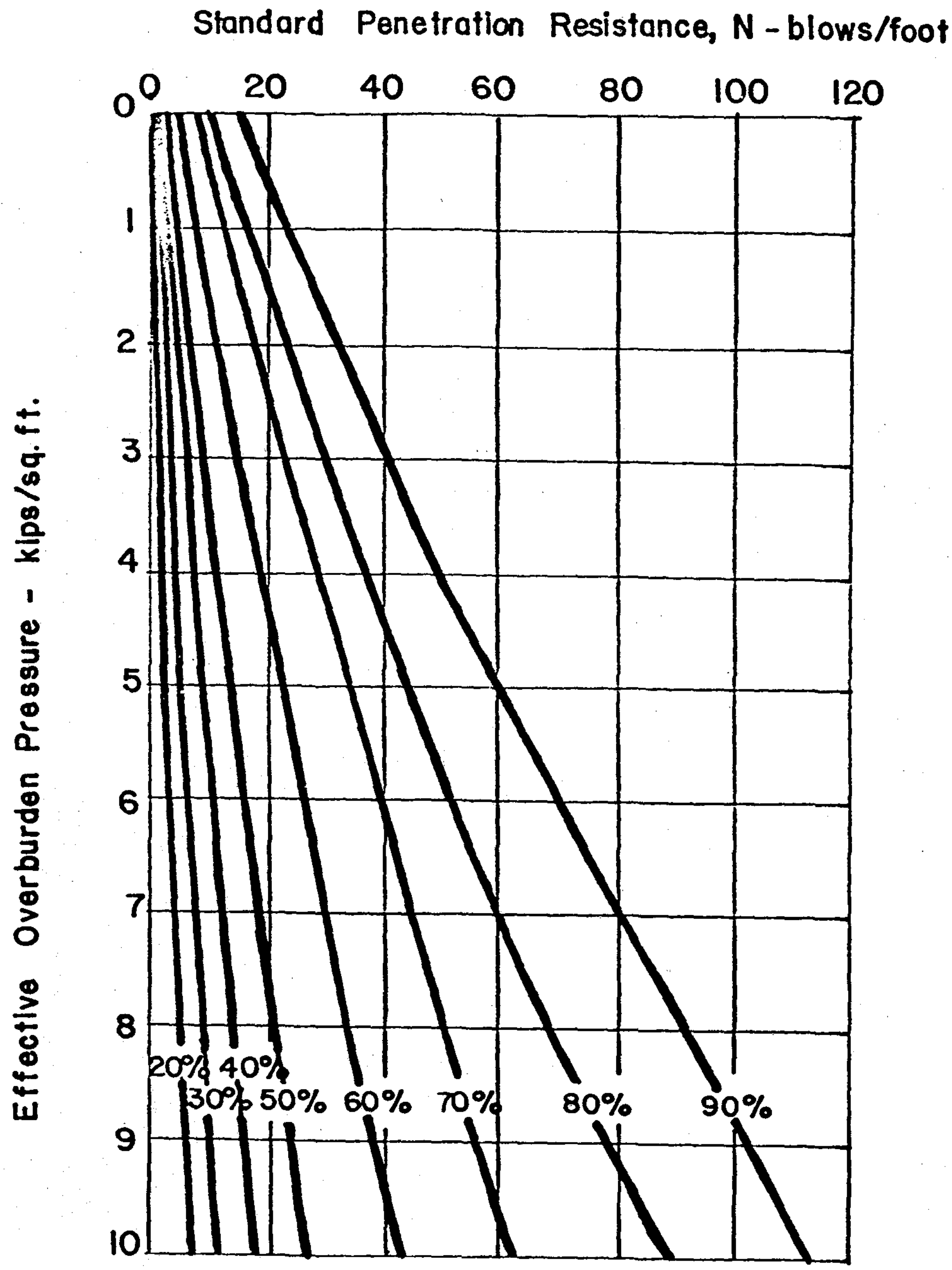

FIG. VII.4 RELATIONSHIP BETWEEN STANDARD PENETRATION RESISTANCE, RELATIVE DENSITY AND EFFECTIVE OVERBURDEN PRESSURE.

(After Seed and Idriss (13))

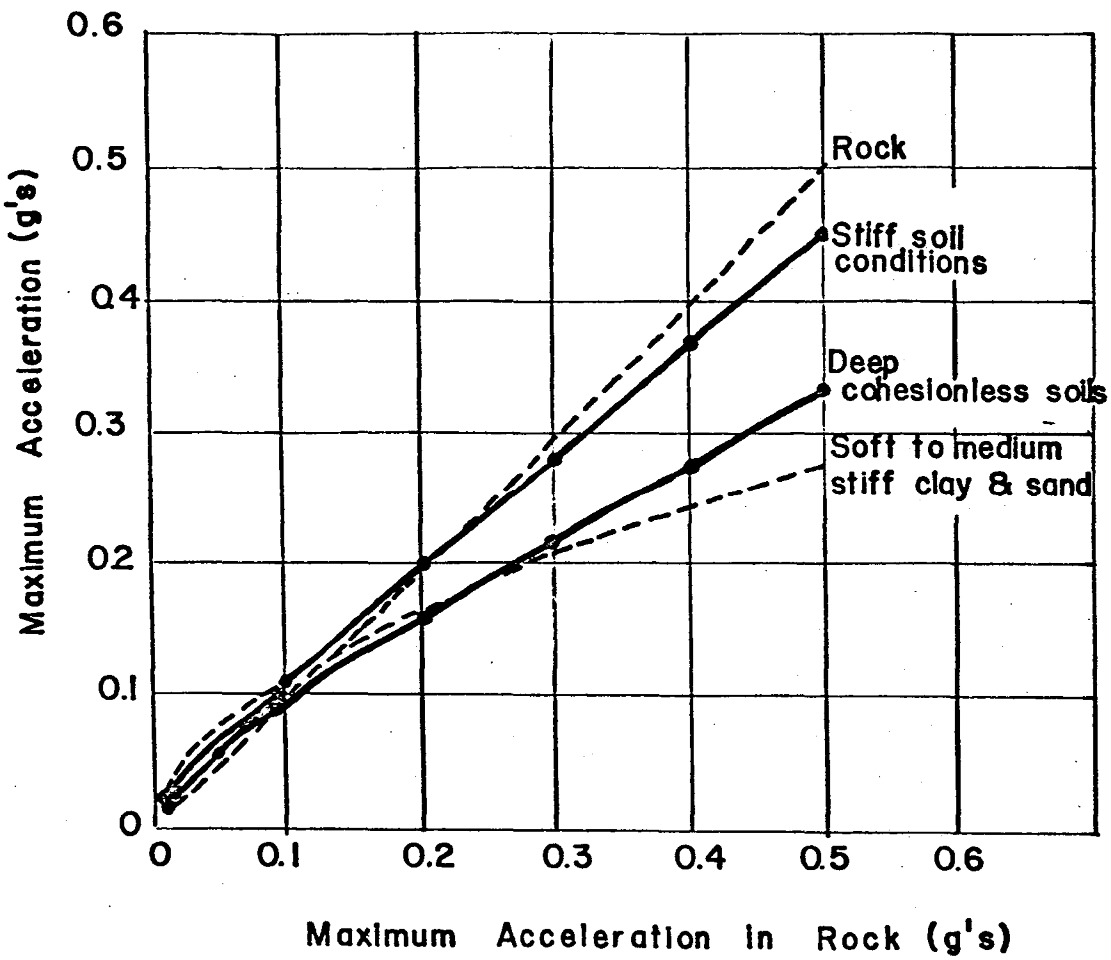

FIG. VII.5 EFFECT OF SOIL ON MAXIMUM ACCELERATION

(After Seed, Murarka, Lysmer & Idriss (15))

CHAPTER VIII SEISMIC DESIGN CRITERIA

VIII.1 Preface

To aid in the design of buried pipelines against earthquakes, this Chapter evaluates the reserve strength/strain of buried pipes beyond normal stress/strain conditions. This reserve strength is the capacity available in buried pipes to resist seismic loads. In buried pipelines under a combination of conventional and seismic loadings, bi-axial stresses are developed. Conventional loads produce mainly hoop stresses whereas seismic wave propagation produces predominantly longitudinal stress. To evaluate the failure of buried pipelines consisting of materials with different tensile and compressive strengths such as cast iron and concrete, under a bi-axial stress state, a modified Von Mises failure criterion is proposed.

For practical applications, this Chapter evaluates parametrically the reserve strengths/strains of typical rigid pipes (cast iron or concrete) and typical flexible pipes (ductile iron or steel) with respect to several important parameters such as aging (corrosion effect), laying and loading conditions, buried depth and dynamic effect (earthquake induced water pressure). Details of these parametric studies are available in earlier reports[3,11]. This Chapter presents the basic formulation and sample problems.

VIII.2 Conventional Non-Seismic Design Methods

VIII.2.1 General

The conventional methods used for determining the loads on buried pipes can be attributed to the early works of Marston[5], Schlick[8] and Spangler[9]. The AWWA standards[1,2] are based on these and other developments. Two essentially independent methods are used for the conventional analysis/design of buried pipes. The separation is based on the relative stiffness of the pipe and the surrounding soil. In most cases, the thickness determines the characteristics of the pipe, that is, either rigid or flexible.

For rigid pipe design, the deflection of the pipe is assumed to be so small that the lateral soil resistance does not play a significant role in the analysis. Thus, the ring stresses in the pipe come from the combination of internal water pressure and external earth and/or truck loads. For flexible pipes, the lateral resistance of the soil is a major design factor because of the pipe's relatively large lateral deflection characteristics. Due to the fact that the vertical deflection of the pipe will reduce the vertical trench load, while the horizontal deflection will increase the soil resistance, the AWWA design method[2] for flexible pipes is based on a stress produced either by the internal water pressure or by the external trench loads, but not the combination of both as in the rigid pipe design.

Most buried water/sewer pipes consist of non-linear materials with different tension and compression strengths, such as cast iron, ductile iron and concrete. The capacities of these materials are represented by a uniaxial (tensile or compressive) strength and a modulus of rupture or bending strength. In general, a buried pipe needs to be checked for both strengths. For rigid pipe design, the failure from combined stresses (ring tension and ring bending) is determined by an interaction equation.

Finally, the conventional design of buried pipes takes the aging effect into account. From current AWWA Codes[1,2], the corrosion allowance added to the design wall thickness ranges from 0.05 inches to 0.10 inches depending on pipe material and size. In this study, it is assumed that the amount of corrosion increases linearly with time over a 30 year life span, the reduction of wall thickness by corrosion would be 0.02 to 0.03 inches for every 10 years.

Note that the conventional design does not take joint effects into account. This is due to the fact that the design is based on a (plane strain) ring segment of the pipeline of unit length.

VIII.2.2 Conventional Non-Seismic Stress Analysis for Cast Iron Pipes

The design for cast iron pipe is typical of "rigid" pipe design. The design is controlled by one of two loading conditions. Loading Condition #1 includes the earth pressure (without truck load) plus working and surge water pressures. Loading Condition #2 considers earth pressure, traffic and impact loads plus operating water pressure (without surge).

According to published semi empirical research results from Iowa State University[5,8,9], the ring bending stress, $\sigma_{b,r}$, due to an equivalent vertical load, W, is:

$$\sigma_{b,r} = 0.0795W\ (d+t_o)/t_o^2 \quad \text{(psi)} \qquad \text{(VIII.1)}$$

where d = nominal diameter of pipe (in.)
t_o = thickness of pipe (in.)
W = equivalent vertical trench load (lbs/lin.ft)

The load W is a function of laying condition and buried depth.

The ring tension produced by the internal pressure, $\sigma_{t,r}$ is:

$$\sigma_{t,r} = \frac{pd}{2t_o} \qquad \text{(VIII.2)}$$

where p is internal water pressure with or without surge.

The combined stress in the buried pipe, $\sigma_{c,r}$, is the sum of ring bending and ring tension:

$$\sigma_{c,r} = \frac{pd}{2t_o} + \frac{0.0795W\ (d+t_o)}{t_o^{\ 2}} \qquad \text{(VIII.3)}$$

The tensile strength, σ_{ty} of the material may be used to check the ring tension (Eqn. VIII.2) and the modulus of rupture, σ_{by} may be used to check the combined stress (Eqn. VIII.3). However, for the combined effect of ring tension and bending for the non-homogeneous cast iron material, the AWWA[1] presents a quadratic parabolic interaction equation as its failure criterion. Thus, for a given ring tensile stress, $\sigma_{t,r}$, the reduced modulus of rupture, $\bar{\sigma}_{by}$, is specified as:

$$\bar{\sigma}_{by} = \sigma_{by}\sqrt{1 - \sigma_{t,r}/\sigma_{ty}} \qquad \text{(VIII.4)}$$

and should be used to check the combined stress. In other words, the non-seismic safety factor for buried rigid pipes is:

$$\text{(S.F.)} = \bar{\sigma}_{by}/\sigma_{c,r} \qquad \text{(VIII.5)}$$

Recently, Parmelee[6] indicated that these conventional calculated stresses might be different from the measured stresses by a multiple of

4 or 5 times either way. However, this report considers only the conventional stresses suggested by the AWWA codes[1,2].

VIII.2.3 Conventional Non-Seismic Stress Analysis for Ductile Iron Pipes

The design for ductile iron pipe is typical of "flexible" pipe design. The design of flexible pipe is controlled by three criteria, namely, (i) the internal water pressure (operating and surge pressure), (ii) trench loads from earth, truck and impact and (iii) pipe deflection.

The ring tension due to internal pressure is given by Eqn. (VIII.2). The ring bending produced by the equivalent vertical trench load as suggested by Iowa State University research[2] is:

$$\sigma_{b,r} = 3\,P_v\,\frac{D}{t_o}\left(\frac{D}{t_o} - 1\right)\left[k_b - \frac{k_x}{\dfrac{8E}{E'\left(\dfrac{D}{t_o} - 1\right)^3} + 0.732}\right] \text{ (psi)} \qquad \text{(VIII.6)}$$

where D = Outside diameter of pipe (in.)
E = Young's modulus of pipe (psi)
E' = Modulus of soil reaction (psi)
k_b = Bending moment coefficient
k_x = Deflection coefficient
P_v = Equivalent vertical trench load (psi)

Note that P_v is a function of buried depth and E', k_b and k_x depend on the laying condition.

In flexible pipe design, the tensile strength of the material is used to check the ring tension and the bending strength is used to check the ring bending. As discussed earlier, no interaction of stress is necessary. Thus, the non-seismic safety factor for buried flexible pipes will be the smaller of:

$$(S.F.) = \sigma_{ty}/\sigma_{t,r} \text{ or } \sigma_{by}/\sigma_{b,r} \tag{VIII.7}$$

in which the ring tension or ring bending controls the design.

VIII.3 Additional Stresses in Buried Pipes

VIII.3.1 General

The conventional plane strain non-seismic stress analyses only give the ring tension and ring bending stresses. However, for seismic resistance, the axial (longitudinal) strength of buried pipes is most important. Following are additional stresses which have not been considered in the conventional design analyses.

VIII.3.2 Longitudinal Stress Due to Partial Live Load

Based on the theory of beams on elastic foundation[4], the longitudinal bending stress, $\sigma_{b,L}$, due to partially distributed truck and impact loads along the pipeline is found to be:

$$\sigma_{b,L} = \frac{wD}{2I}\sqrt{\frac{EI}{k_b}} \tag{VIII.8}$$

where I = Moment of inertia, pipe
w = imposed live load
k_b = Spring constant for lateral soil resistance

VIII.3.3 Axial Stress Due to Internal Pressure

When a buried pipe comes to a closed end or directional change, local axial stress due to internal pressure, $\sigma_{a,L}$, develops[10] as:

$$\sigma_{a,L} = \frac{pd}{4t_o} \tag{VIII.9}$$

This stress diminishes in the longitudinal direction of the pipe because the axial deformation of the pipe will be resisted by the soil friction around the pipe. However, for seismic resistance evaluation, this local axial stress may be required in the analysis.

VIII.3.4 Dynamic Effect Due to Seismic Excitation

The conventional stress analysis is for static loads only. However, under seismic excitations, there may be a dynamic effect that will increase the internal water and surge pressures. To study this effect, a dynamic load factor, β, ranging from 1 to 2, has been assigned to the internal water pressure and surge pressure. The investigation of the true dynamic factor for various earthquakes is outside the scope of this report.

VIII.4 Seismic Reserve Strength/Strain of Buried Pipelines

VIII.4.1 Stresses and Strengths

The biaxial stresses on a buried pipe element subjected to both seismic and conventional non-seismic loads are shown in Fig. VIII.1 in which σ_1, σ_2 are the stresses in the longitudinal and hoop direction respectively.

Since during seismic wave propagation, the axial stresses have been shown to be predominant, the seismic bending stress is neglected. Thus, this section develops the seismic reserve strength/strain of buried pipes in the longitudinal direction only.

The calculated stress, σ_1, for combined seismic and non-seismic effects in the longitudinal direction is:

$$\sigma_1 = \sigma_{as} + \beta\, \sigma_{a,L} \pm \sigma_{b,L} \qquad \text{(VIII.10)}$$

where σ_{as} is the seismic axial stress produced by an earthquake in the longitudinal direction. This stress, σ_{as}, can be calculated from the earthquake induced strain using the procedures discussed in Chapter III or IV.

For evaluation of the safety of buried pipes against earthquakes, and to establish the seismic design criteria for buried pipes, σ_{as} is the required seismic reserve strength and ε_{as} is the required reserve strain.

Since Eqn.(VIII.10) represents a combined axial and bending stress condition, the tensile strength according to the quadratic parabolic interaction(1) in the longitudinal direction, σ_{1y}, will be:

$$\sigma_{1y} = \sigma_{ty} \left[1 - \left(\frac{\sigma_{b,L}}{\sigma_{by}}\right)^2\right] \qquad \text{(VIII.11)}$$

Depending on the pipe construction, the total (seismic plus non-seismic) hoop stress, σ_2, is calculated as follows:

$$\sigma_2 = \beta\, \sigma_{t,r} \pm \sigma_{b,r} \qquad \text{(VIII.12)}$$

for the rigid pipes and

$$\sigma_2 = \beta\, \sigma_{t,r} \qquad \text{(VIII.13a)}$$

or

$$\sigma_2 = \sigma_{b,r} \qquad \text{(VIII.13b)}$$

for the flexible pipes.

Note that for flexible pipe design, either ring tension or ring bending may control, the available strength, σ_{2y}, will either be σ_{ty} or σ_{by} depending on which type of stress controls. However, for rigid pipe design, the stresses (tension and bending) are combined and the available strength in the hoop direction will be obtained by modifying Eqn. (VIII.4).

$$\sigma_{2y} = \sigma_{by}\sqrt{1 - \beta\sigma_{t,r}/\sigma_{ty}} \qquad \text{(VIII.14)}$$

VIII.4.2 <u>Modified Von Mises Criteria</u>

For homogeneous materials, the Von Mises yield criteria(7) has been developed to define the failure of an element under a biaxial stress state as:

$$\frac{(\sigma_1 - \sigma_2)^2}{2} + \frac{\sigma_1^2}{2} + \frac{\sigma_2^2}{2} = \sigma_y^2 \qquad \text{(VIII.15)}$$

which may be rewritten as:

$$\left(\frac{\sigma_1}{\sigma_y} - \frac{\sigma_2}{\sigma_y}\right)^2 + \left(\frac{\sigma_1}{\sigma_y}\right)^2 + \left(\frac{\sigma_2}{\sigma_y}\right)^2 = 2 \qquad \text{(VIII.16)}$$

However, for buried cast-iron or ductile iron pipe, the material is not homogeneous and Eqn. (VIII.15) and Eqn. (VIII.16) do not apply. For design purposes, this paper proposes a "modified" Von Mises failure criterion to include the non-homogeneous characteristics of material as follows:

$$\left(\frac{\sigma_1}{\sigma_{1y}} - \frac{\sigma_2}{\sigma_{2y}}\right)^2 + \left(\frac{\sigma_1}{\sigma_{1y}}\right)^2 + \left(\frac{\sigma_2}{\sigma_{2y}}\right)^2 = 2 \qquad \text{(VIII.17)}$$

where σ_{1y} and σ_{2y} are the yield strengths in the 1 and 2 direction respectively.

Substituting the calculated stresses and available strengths developed in Eqns. (VIII.10) thru (VIII.14) into Eqn. (VIII.17), the seismic reserve strength σ_{as} of a buried pipe beyond its normal stress condition can be readily determined.

VIII.5 Reserve Strength/Strain of Cast Iron Pipes

This section examines the effects of a number of important parameters on the seismic reserve strengths such as, thickness, aging (corrosion), loading condition, laying condition, and dynamic effects on water pressure.

As an example for the parametric study, a cast iron pipe with the following data is used:

Nominal diameter,	$d =$	18 in. (46 cm)
Tensile strength,	$\sigma_{ty} =$	18 ksi (124 MPa)
Modulus of Rupture,	$\sigma_{by} =$	40 ksi (276 MPa)
Operating Water pressure	$p_o =$	200 psi (1380 kPa)
Surge pressure,	$p_s =$	100 psi (690 kPa)
Buried depth,	$H =$	5 ft. (1.5 m)
Initial safety factor,	(S.F.) =	2.5

For cast-iron pipe construction, AWWA(1) suggests three possible laying conditions as noted below. Laying Condition B has been selected as the standard case.

Laying Condition	Description
A	Pipe laid on flat bottom trench, backfill not tamped.
B	Pipe laid on flat bottom trench, backfill tamped.
F	Pipe bedded in gravel or sand, backfill tamped.

The design of cast-iron pipe is controlled by one of the two possible loading conditions listed:

Loading Condition	Description
#1	Operating water and surge pressure + earth load (No live loads)
#2	Operating water pressure + earth and live loads (No surge pressure)

Stresses under both loading conditions are calculated but only the critical results are presented.

Following the AWWA specification[1], the initial design is found to be controlled by Loading Condition #1 and the thickness is 0.47 in. (1.19 cm). With a 0.08 in. (0.20 cm) corrosion allowance and a 0.08 in. (0.20 cm) manufacturing allowance added, the initial design thickness becomes 0.63 in. (1.60 cm) and AWWA Class #23 pipe is designated.

Using the proposed "modified" Von Mises failure criterion, the effects of corrosion and thickness on the reserve axial strength under conventional Loading Condition #2 are noted in Fig. VIII.2 with an "age" parameter shown by T = 0, 20, 40, 60 and 80 years of life. Although the initial non-seismic design is controlled by Loading Condition #1, the effect of axial stress produced by the live loads (truck + impact) which have not been considered in the conventional design greatly reduces the seismic factor for the seismic reserve strength.

The effect of laying condition on the reserve axial strength is shown in Fig. VIII.3. From this figure, one can conclude that the reserve strength is a function of surrounding soil stiffness.

In conclusion, the seismic reserve axial strength of buried cast-iron pipe is influenced by all parameters investigated. The effects from corrosion and loading conditions are a little higher than those from buried depth, laying condition and dynamic load factor.

VIII.6 References

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2. American Water Works Association, "American National Standard for the Thickness Design of Ductile Iron Pipes", AWWA C150-76, August 1976.

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10. Untrauer, R.E., Lee, T.T., Sanders, W.W. and Jawad, M.H., "Design Requirements for Cast-Iron Soil Pipe", Bull. 199, Engineering Research Institute, Iowa State University, May 1970.

11. Wang, L.R.L. and Fung, R.C.Y., "Seismic Design Criteria for Buried Pipelines", Proc. of ASCE Pipeline Div. Specialty Conf., New Orleans, LA., Jan. 1979, pp. 130-145.

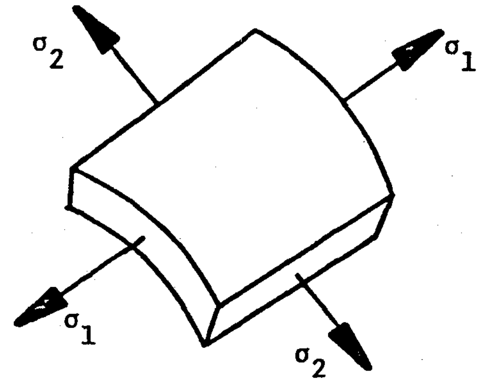

FIG. VIII.1 STRESSES IN A BURIED PIPE ELEMENT

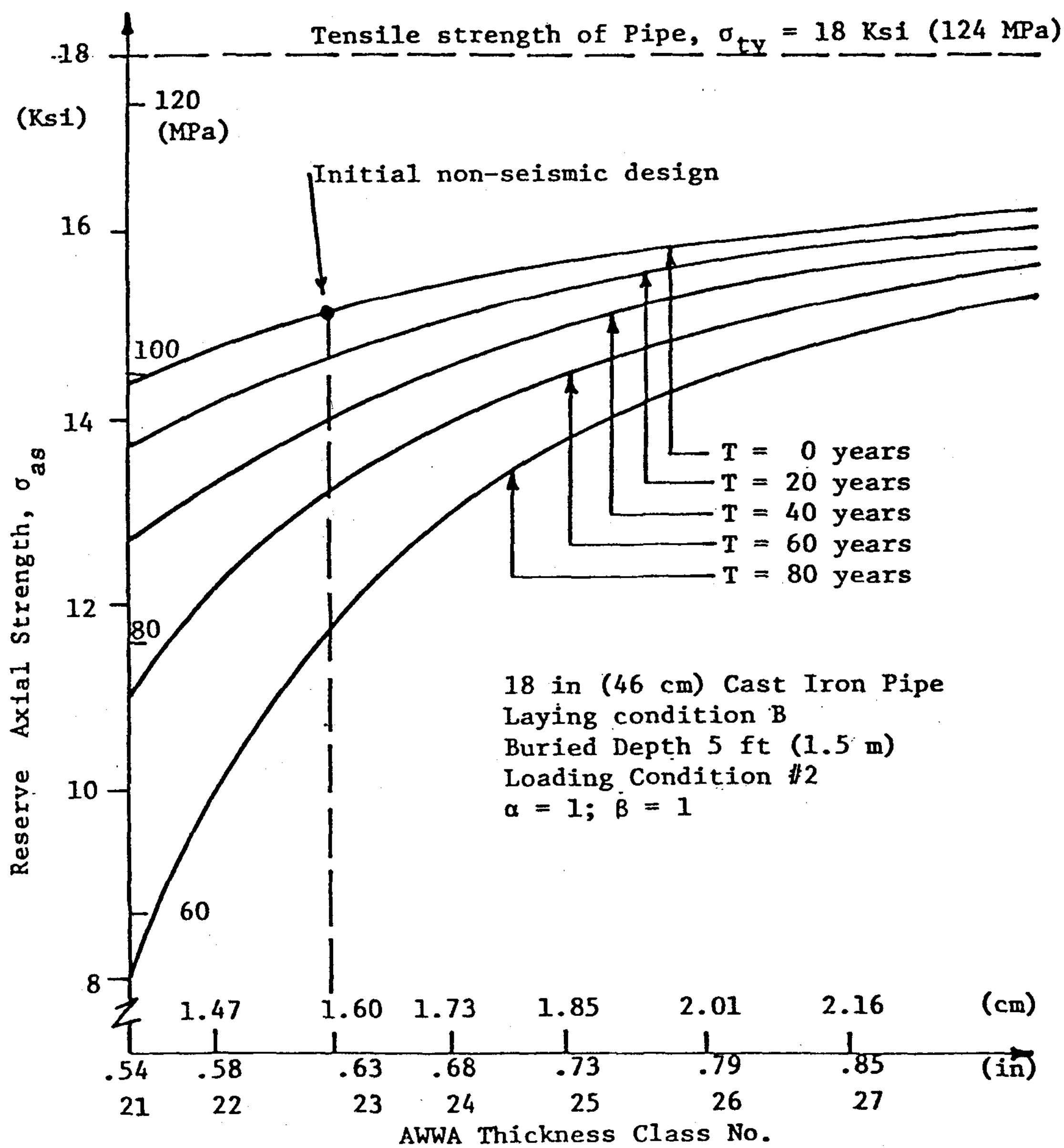

FIG. VIII.2 RESERVE AXIAL STRENGTH VS. THICKNESS

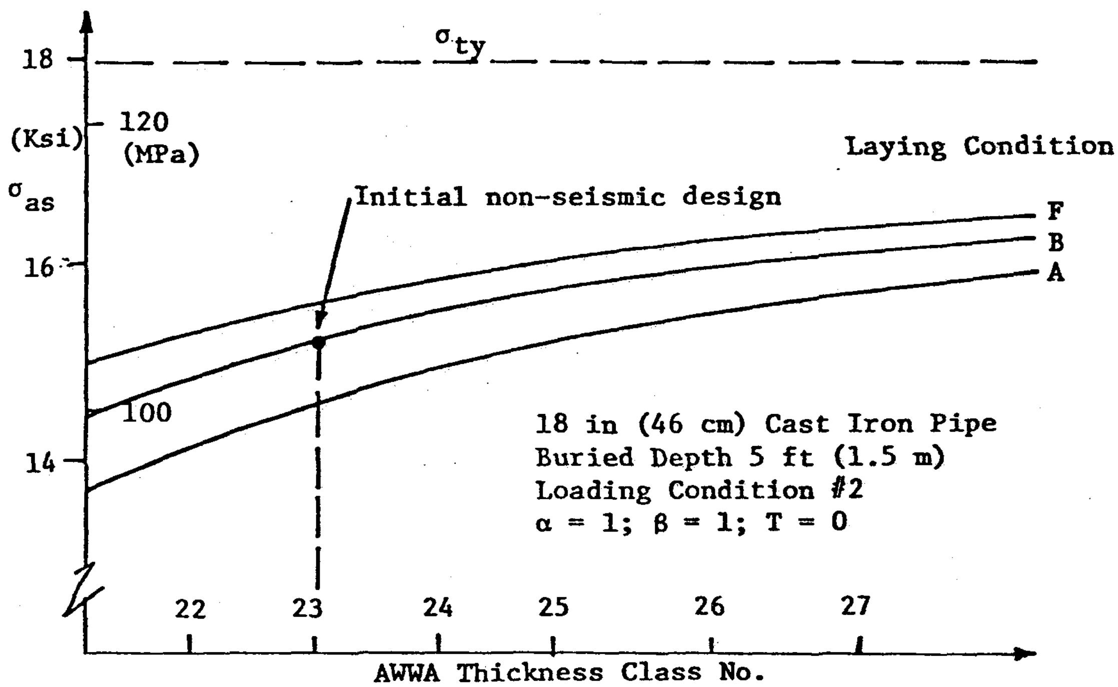

FIG. VIII.3 EFFECT OF LAYING CONDITION ON σ_{as}

CHAPTER IX CASE STUDY (LATHAM WATER DISTRICT)

IX.1 Preface

This chapter uses the "simplified approach" of Chapter III to analyze an existing water distribution system having a variety of pipe/joint configurations and a variation in subsurface properties. Application of the simplified approach required utilization of research and analysis methodologies discussed in Chapters I through VIII of this report. The Latham Water District was chosen for this case study[2] due to its proximity to the research institution, its well documented facilities and its readiness to cooperate in all phases of the project.

This study indicates that a substantial portion of the water district could experience earthquake related damage based on a 450 year return period earthquake (20% probability of exceedance in a 100 year economic life time). The potential failure area is over a deep, loosely consolidated, sand, silt and clay area that has filled in a pre-glacial river valley to a depth of 300-350 feet (90-105 m) in some areas. In addition, distribution piping in this area is of a relatively non-flexible leadite or lead joint construction resulting in potential leakage under tensile forces.

The use of flexible joint systems for new portions of the system as well as replacements for damaged older portions tends to continually upgrade the system and decrease vulnerability.

IX.2 Existing Water Distribution System[5]

The Latham Water District was formed in 1929 and presently includes the major portion of the Town of Colonie, Albany County, New York, with a total area of approximately 50 square miles (130 square kilometers).

The system has two major intakes, several well locations and several water storage tanks. Figure IX.1 shows the relative location of the treatment plants as well as major distribution piping sized 10 inches (25 cm) diameter and above. Water storage tanks are located in seven general areas in the district with storage capacities ranging from 100,000 to 3,000,000 gallons (378,500 to 11,356,000 liters). Distribution of the storage tanks is noted in Figure IX.1 and identified in Table IX.1.

Distribution piping ranges in size from 6 inch (15 cm) diameter to 30 inch (76 cm) diameter and includes a variety of pipe and joint materials. Initial distribution system construction consisted of cast iron pipes with mechanical joints of 6 through 16 inch (15-41 cm) diameters; cast iron pipe with lead joints used for reservoir supply lines and major distribution lines of 12 through 24 inch (30-61 cm) diameters; and cast iron pipes with "leadite" joints of 6 through 12 inch (15-30 cm) diameters. "Leadite" refers to a sulfur cement-non lead aggregate compound used as a lead substitute for water pipe joints from approximately 1929 through 1950 in the Latham Water District. The leadite joints used in the Latham Water District were relatively rigid and brittle materials as compared to the lead system.

Beginning in 1950, new portions of the distribution system were typically installed with cast iron pipes and traditional lead joints with sizes ranging from 6 through 8 inch (15-20 cm) diameters. Circa 1967, gasketed connections were initiated and since 1973 new and replacement pipe installations have generally been of ductile iron pipe with rubber gasket connections in sizes ranging from 6 to 24 inch (15-61 cm) diameters. A section of prestressed concrete pipe with mortar/rubber gasket joints associated with the 1969 water treatment plant construction is located in the northeastern portion of the district and is of 24 and 30 inch (61-76 cm) diameters. An additional 24 inch (61 cm) diameter trunk section is located in the western portion of the district.

The historical development of the system has resulted in "cast iron pipe-leadite" joint systems in the older more densely populated regions of the town with the newer residential and commercial developments serviced by "cast iron-lead" and/or "ductile iron-rubber gasket" pipe-joint systems.

Of the non mechanical pipe-joint systems, the "ductile iron-rubber gasket" combination provides the greatest flexibility followed by the "cast iron-lead" system. The "prestressed concrete-mortar/rubber gasket" system is expected to act as a continuous pipe in compression due to mortared joints and as a flexible joint in tension after the mortar cracks and behavior is controlled by the rubber gasket.

Due to climatic conditions in the Northern portion of the United States, the majority of the water distribution piping in the Latham Water District is installed with a natural soil cover of 5 feet (1.5 m) from ground surface to the top of the pipe. Typical installation details and junction/crossing details are noted in reference 5.

IX.3 Geological and Soil Conditions

The Latham Water District is underlain by shale bedrock of the Normanskill, Snake Hill and Indian Ladder formations. Overlying the shale bedrock are deposits of till and glacial outwash consisting of sands, silts and clays. Due to the pressure of a pre-glacial river channel orientated roughly north-south in the central portion of the water district, depths to bedrock range from 300 to 350 feet (90-105 m) in this region as compared to 30 to 50 feet (9-15 m) in the western portion of the district and to occasional shale outcroppings in the eastern portion of the district. Bedrock contour lines shown in Fig. IX.2 note the presence of this preglacial steep sided valley. Figure IX.3 notes a typical eastwest cross section through the water district. Figure IX.4 presents soil isopachs, contours of equal soil depth above bedrock. "Blow count" results from numerous soil investigations in the area conducted for airport, roadway and sewer construction indicate Standard Penetration Resistance, N, values of 4-12 from the surface layers to depths of 250 feet (76 m) with sample descriptions typically including sand, silt and traces of clay. The water table generally lies within 10 feet (3 m) of the surface. Based on information from the soils borings it is concluded that the major portion of the soil is loosely consolidated, saturated, and of fine grained sands and silts.

IX.4 Earthquake Risk

The seismic risk for the Latham Water District area[1] has been presented in terms of annual risk, average return periods and probabilities of exceedance in Chapter VI - Seismic Risk Analysis. The annual risk results are based on a study of historic earthquakes in the Northeast United States. Specifically, all historic earthquakes with epicenters within a circle of 160 kilometer radius and centered at Latham, were used to establish an average earthquake occurrence rate of 0.204×10^{-4} earthquakes per year per square kilometer for Richter magnitude 2 or greater. The historical data was also used to develop a magnitude-frequency relationship for the Latham area. Finally, an attenuation relationship was incorporated into the analysis to account for the decrease in acceleration magnitude with distance between site and epicenter. The attenuation function selected used a set of conservative parameters with a probabilistic error term.

Using a Poisson process to model the random occurrence of a natural event, the annual risk (the probability that a given ground acceleration will be exceeded in any particular year), was developed and plotted as a graph. The reciprocal of the annual risk, the return period, was presented in a table giving maximum ground acceleration for return periods of 10 to 200 years. An additional table was presented listing recommended maximum ground accelerations for specific exceedence probabilities and economic lifetimes (see Table VI.2). This latter table provides the engineer with a design earthquake acceleration once the economic lifetime and the acceptable criteria for exceedence of the design parameter is established. The determination of the economic lifetime is seldom a problem but the philosophy of allowing for the possibility of exceeding a design parameter, especially when the loss of life may be involved, is a more difficult criteria to establish.

Since many civil engineers think more easily in terms of a design year event Table IX.2[(2)] was developed showing design year earthquakes as well as maximum ground accelerations for specific exceedance probabilities and economic lifetimes.

Discussions with Mr. Warren Lavery, Superintendent of the Latham Water District, have established a 100 year economic lifetime for the distribution system and an acceptable probability of exceedance of 0.20, corresponding to a 450 year design earthquake. Determination of the seismic vulnerability of the distribution system is based on this design earthquake event.

IX.5 Seismic Vulnerability of the Distribution System

IX.5.1 Strain and Displacement Criteria

Using information regarding the engineering aspects of the distribution system, the local soil and geologic conditions, and the results of the seismic risk study, an analysis of the distribution system for earthquake effects was accomplished using the "simplified approach" of Chapter III for the previously established guideline of a 100 year economic lifetime and an acceptable probability of exceedence of 20% (i.e., the 450 year event).

As noted in Chapter III, assuming that the soil and pipe move together and the shape of the seismic wave remains relatively constant, the maximum pipe axial strain, ε_a, and the maximum soil axial strain will be the same.

Thus,

$$\varepsilon_a = \frac{V_{max}}{C_p} \tag{IX.1}$$

where V_{max} represents the maximum ground velocity during an earthquake and C_p is the longitudinal wave velocity relative to the pipeline.

In addition, assuming the pipe curvature is the same as the soil curvature, the pipe flexural strain, ε_f, can be obtained by multiplying the curvature, χ, by the pipe radius, R. Thus,

$$\varepsilon_f = R\chi = \frac{R\ A_{max}}{C_s^2} \tag{IX.2}$$

where A_{max} represents the maximum ground acceleration for the site and C_s is the transverse wave propagation velocity in the controlling medium.

The combined pipe strain, ε_t, is conservatively:

$$\varepsilon_t = \varepsilon_a + \varepsilon_f \tag{IX.3}$$

This combined strain is deemed to be conservative since the maximum values of acceleration and velocity will not occur simultaneously.

The product of the combined strain and modulus of elasticity of the pipe material gives the longitudinal stress due to earthquake effects.

For a continuous piping system such as the "cast iron-mechanically jointed" and the "prestressed concrete mortar/rubber gasket" system acting in compression, the longitudinal stress represents an upper bound on the required capacity of the pipe and joints since no joint movement is available to relieve the strains. For a jointed system acting in compression, with pipe segments in contact, the above magnitude is an upper bound on compressive longitudinal stress capacity required to prevent failure due to crushing of bells or local buckling of thin walled pipes.

For a flexible jointed system subject to tensile strains, pullout of the pipes at a joint will tend to relieve the axial strain and joint rotation will relieve the flexural strain. If a joint system can withstand the induced axial strain and allow the necessary axial movement without losing its ability to maintain a watertight seal it will survive the earthquake ground displacements without leakage.

As noted in Chapter III, in a jointed system, an upper bound for the required axial joint movement, U, can be obtained by multiplying the peak axial soil strain, ε_a, by the length of the pipe segment, L, thus,

$$U = \varepsilon_a L \qquad \text{(IX.4)}$$

An idealized joint, with no axial force resistance, would then have an upper bound requirement on joint movement equal to U.

In addition, an upper bound on the required rotation capacity, θ, of a stress free joint is the product of the maximum curvature and length of the pipe segment:

$$\theta = L\chi \text{ radians} \tag{IX.5}$$

For the case of actual joints, it is expected that a combination of force transfer and joint movement will occur. For the "leadite" and lead joints it is expected that an initial tensile capacity will be available until the joint compound is pulled free of the bell and spigot connection. After this joint has been "cracked" no additional tensile force will be available during further earthquake strain cycling. The rubber gasketed joint will also have a limited load capability represented by deformation of the gasket material until it is pulled free or "rolled" out of position. In this case study the tensile capacity as well as the rotational moment restraint of the non-rigid joints was ignored as a conservative approximation. Continuous mechanically jointed and prestressed concrete pipe and joint systems were thus analyzed for tensile and compressive strains/stresses due to the design earthquake. Discontinuous, non-rigid "leadite", lead, and rubber gasketed systems were analyzed for compressive strains/stresses and tensile joint movement and rotation.

IX.5.2 Wave Velocities

In order to evaluate the strains, displacements and rotations in the Latham Water District, it was necessary to establish the propagation velocities of the seismic waves relative to the pipe, C_p and C_s, for various locations within the distribution system.

Referring to Fig. IX.3, calculation of the wave velocity in the Water District was simplified by dividing the case study area into relatively "deep" and "shallow" zones of soil overburden. It was assumed that the wave velocity over the preglacial valley areas would be controlled by the deep layers of the sand, silt and clay mixture. In the "shallow" area, the wave velocity was assumed to be generally controlled by the underlying shale bedrock. Several hills noted in the surface topography of the "shallow" area are localized and represent former sand dunes. These high points may be noted as the 100 foot isopaths of limited area of Fig. IX.4. For purposes of this study, the wave velocity at these localized high points was assumed to be controlled by the "shallow" zone bedrock. The 100 foot isopach, bordering the preglacial valley, was selected to generally deliniate the two major zones.

Coupling the technical information on wave velocities with the soil conditions in the Latham Water District, an approximation was made for the wave velocity in areas controlled by the deep soil layers. Blow count information at various locations within the Latham Water District indicated Standard Penetration Resistance, N, values of 4-12 to depths of 250 feet (76 m) in the sand, silt and silty clay glacial lake deposits. The water table was generally found to be within 10 feet (3 m) of the surface. Assuming an average Standard Penetration of 8 and a soil depth of 10 feet (3 m) to represent the zone of pipe burial and a soil weight of 120 lbs/ft^3, (18.85 kN/m^3) Figures VII.4 and VII.5 were used to calculate a transverse (shear) wave velocity, C_s, of 360 ft/s (100 m/s).

Using an alternate approach and assuming the transverse wave velocity to be controlled by the material of the middepth of the average soil layer thickness in the pre-glacial valley resulted in a transverse (shear) wave velocity of 508 ft/s (155 m/s).

Based on shear wave velocities observed in other locations, this latter value appeared more reasonable and still represented a relatively loose material such as the sediments noted in the boring logs.

For simplicity; a shear wave velocity of 500 ft/s (150 m/s) was used in further portions of this Chapter to represent the approximate magnitude of the shear wave velocity in the deep cohesionless layers within the pre-glacial valley. For the shallow soil depths adjacent to the valley, the shear wave velocity was assumed to be controlled by the underlying shale bedrock. A shear wave velocity of 2500 ft/sec. (760 m/sec.) was assumed.

For the two zones, shallow and deep, the pertinent wave velocities were then:

	C_s	C_p
Shallow	2500 ft/s (760 m/s)	4330 ft/s (1320 m/s)
Deep	500 ft/s (150 m/s)	870 ft/s (260 m/s)

IX.5.3 <u>Ground Acceleration and Velocity</u>

Based on the seismic risk analysis coupled with a 100 year economic lifetime and 20% probability of exceedence, the probable maximum ground acceleration in rock at the Latham Water District site is 0.21g. To develop the acceleration and velocities that will control strains in the piping system it was necessary to determine the ground accelerations in the region of pipe burial.

The ground motion in the shallow zone of the Latham Water District was assumed to be controlled by the rock and hence maximum ground acceleration was kept at 0.21g, that is not modified for local soil conditions. For the "deep" zone, a value of 0.17g was used as the maximum ground acceleration.

The maximum ground velocities were calculated as 10.5 in/s (26.7 cm/s) for the shallow deposits and 8.5 in/s (21.6 cm/s) for the deep deposits. These values were generated assuming 50 in/sec/g (127 cm/s/g) as the ratio of V_{max}/A_{max} as shown in Table VII.1. Note that this yields conservative values for the deep zone.

IX.6 Summary

Synthesizing the above information resulted in Table IX.3 indicating the maximum strains, stresses and displacements for the idealized "shallow" and "deep" soil zones. Calculations were based on a pipe segment length of 20 feet (6.1 m) and a diameter for flexural strain calculations of 30 inches (76 cm). Even with the use of the largest pipe diameter, the effect of curvature related flexural strain was an order of magnitude less than axial induced strain.

"Shallow" areas, controlled by the shale shear wave velocities, would develop upper bound stress and/or displacement requirements of a tolerable magnitude for all pipe joint combinations of initial sound condition. Pipes and joints severely weakened by corrosion and non-earthquake loadings could be potentially damaged by these additional strength requirements.

"Deep" areas controlled by the 100-350 ft (30-107 m) layer of loosely consolidated sands and silts would develop appreciable upper bound stresses in continuous pipe action (rigid joints and joints in contact in compression) and would require upper bound values of joint movement in the range of 1/4 inch (0.6 cm) for jointed conditions.

Wang and Fung[4] indicate a considerable reserve stress capacity in the axial direction for normally designed pipes. Since the flexural strain is so low, its effect on a locked joint would appear tolerable. The angular relative rotation between pipe segments of 0.03° is minor and well within the leakage range presented by Untrauer, et al.[3] for cast iron pipe with lead caulked joints.

The required axial joint motion of approximately 1/4 in. (0.6 cm) appears to be more critical. It is expected that such a movement would open both the "leadite" and lead caulked joints and result in numerous leaks in the distribution system.

Based on the above analysis, joint leakage would be expected to occur in several areas where lead and leadite joint systems coincide with the deep cohesionless soil deposits.

Major distribution lines meeting these failure criteria are cross hatched on Fig. IX.5. From this figure it is noted that potential pipe damage would divide the district into two relatively undamaged zones separated by a damaged central region. In the north, portions of the reservoir supply line of "cast iron-lead" configuration could experience leakage. In the south portion of the central region a relatively densely populated, older residential section serviced by 6 inch and 8 inch (15-20 cm) diameter "cast iron-leadite" distribution lines would be expected to suffer a loss of water due to the design earthquake event. This area is noted by grid hatching on Fig. IX.5. This area represents the greatest potential for damage and loss of life and property from diminished fire fighting capabilities due to a loss of water supply.

Thus, simplified analysis techniques indicate the "shallow" zone would experience little if any damage from the design earthquake. The same technique indicates the potential for damage within the "deep" area with the possibility of distribution line leakage dividing the district and resulting in a loss of water supply for a relatively densely populated residential area.

IX.7 References

1. O'Rourke, M.J. and Solla, E., "Seismic Risk Analysis of Latham - Albany, New York", Technical Report (SVBDUPS Project) No. 3, Dept. of Civil Engineering, Rensselaer Polytechnic Institute, Troy, N.Y., May 1977.

2. Pikul, R.R., Wang, L.R.L. and O'Rourke, M.J., "Seismic Vulnerability of the Latham Water Distribution System - A Case Study", Technical Report (SVBDUPS Project) No. 7, Dept. of Civil Engineering, Rensselaer Polytechnic Institute, Troy, N.Y., Sept. 1978.

3. Untrauer, R.E., Lee, T., Sanders, Jr., W.W. and Sawad, M.H., "Design Requirements for Cast Iron Soil Pipe", Bulletin 199, Engineering Research Institute, Iowa State University, Ames, Iowa (p. 78, Table 7.1).

4. Wang, L.R.L. and Fung, R.C.Y., "Seismic Design Criteria for Buried Pipelines", Tech. Report (SVBDUPS Project) No. 8, Dept. of Civil Engineering, Rensselaer Polytechnic Institute, Troy, N.Y., Sept. 1978.

5. Wang, L.R.L. and Lavery, W.T., "Engineering Profile of Latham Water District, Albany, New York", Technical Memorandum (SVBDUPS Project) No. 3, Dept. of Civil Engineering, Rensselaer Polytechnic Institute, Troy, N.Y., April 1978.

TABLE IX.1 WATER TANK STORAGE CAPACITIES

Tank Site			Capacity (gallons)	Capacity (liters)
T-1	Latham	#1	100,000	378,500
		#2	3,000,000	11,356,000
T-2	Vly	#1	200,000	757,000
		#2	1,000,000	3,785,000
T-3	Boght		200,000	757,000
T-4	Miller Road		1,000,000	3,785,000
T-5	Newtonville		1,000,000	3,785,000
T-6	Osborne Road		500,000	1,893,000
T-7	Loudon		400,000	1,514,000

TABLE IX.2 DESIGN YEAR EVENT AND RECOMMENDED MAXIMUM GROUND ACCELERATION (A_{max}) FOR SPECIFIC EXCEEDENCE PROBABILITIES AND ECONOMIC LIFETIMES

Economic Lifetime T (years)	P = probability of design year event being exceeded at least once in T yrs.		
	P = 0.05	P = 0.10	P = 0.20
25	488 yr (0.23g)	238 yr (0.18g)	112.5 yr (0.155g)
50	975 yr (0.275g)	475 yr (0.23g)	225 yr (0.18g)
100	1950 yr (0.34g)	950 yr (0.27g)	450 yr (0.21g)

Table IX.3 - Maximum Strains, Stresses and Displacements

Relative Depth to Bedrock	a_{max} g's	v_{max} in/s (cm/s)	C_s ft/s (m/s)	C_p ft/s (m/s)	Strains ε_a (10^{-3})	Strains ε_f (10^{-3})	Strains ε_t (10^{-3})	Stresses* σ_{ci} ksi (MP_a)	Stresses* σ_{pc} ksi (MP_a)	Displacements Δ in (cm)	Displacements θ degrees
Shallow<100 ft (shale controls)	0.21	10.5 (26.7)	2500 (762)	4330 (1320)	0.24	-	0.24	3.36 (23.2)	0.72 (5.0)	0.06 (0.15)	-
Deep > 100 ft	0.17	8.5 (21.6)	500 (152)	866 (264)	0.82	0.03	0.85	11.9 (82.0)	2.55 (17.6)	0.20 (0.51)	0.03

*σ_{ci} = cast iron stress, $E = 14 \times 10^3$ ksi (96.5 GPa)

σ_{pc} = prestressed concrete stress, $E = 3 \times 10^3$ ksi (20.7 GPa)

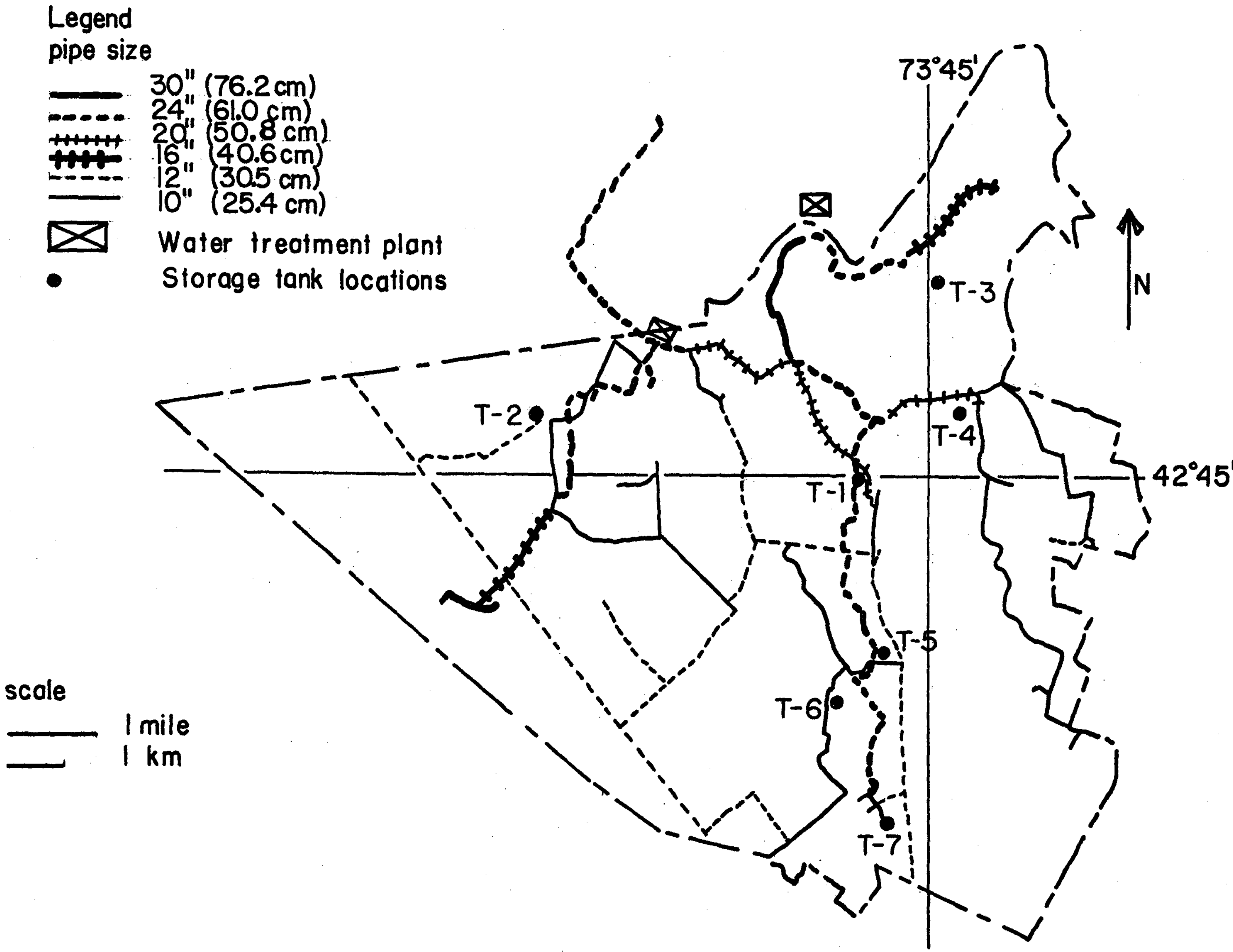

FIG. IX.1 LATHAM WATER DISTRICT DISTRIBUTION MAP

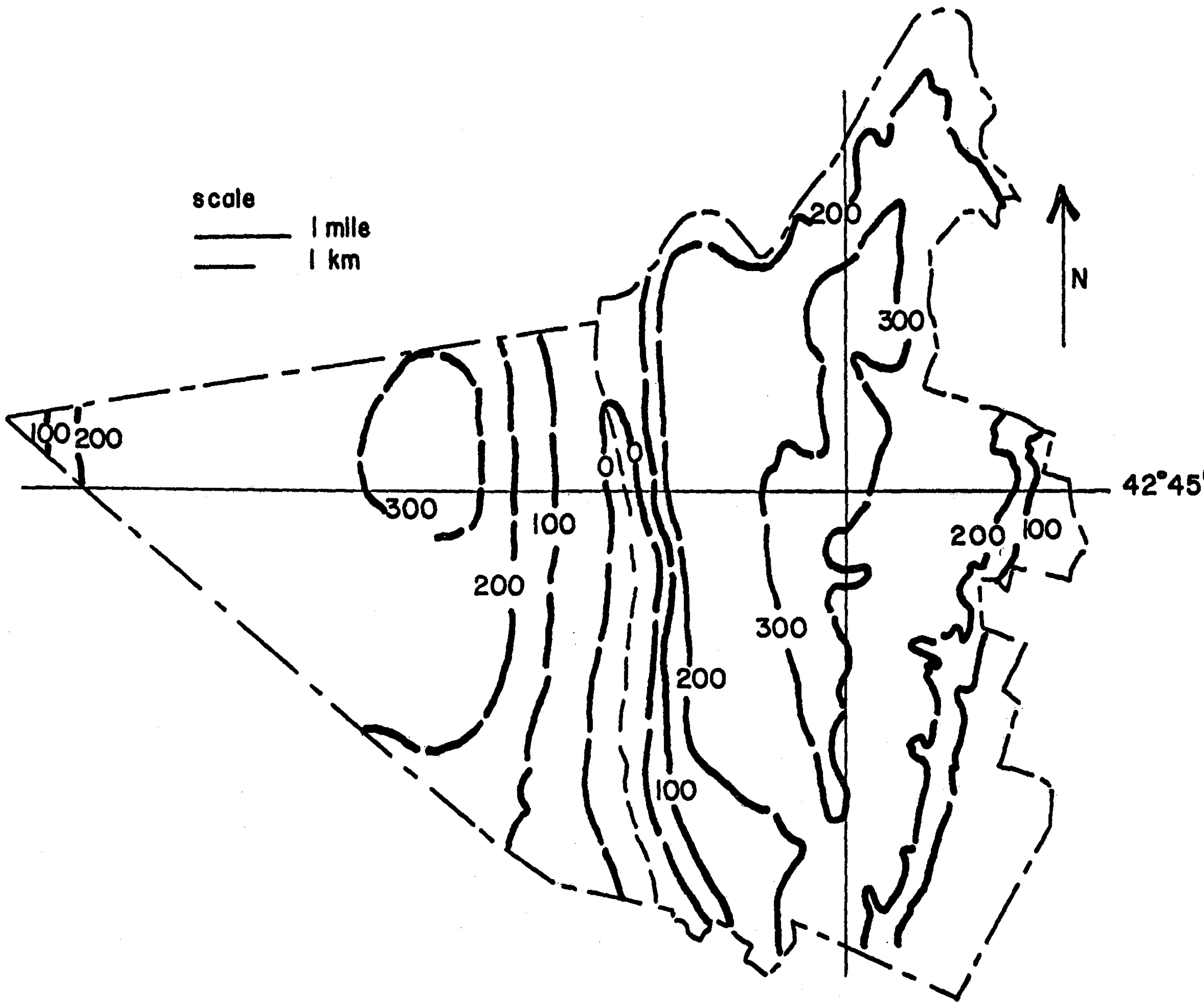

FIG. IX.2 BEDROCK CONTOURS - USGS FEET

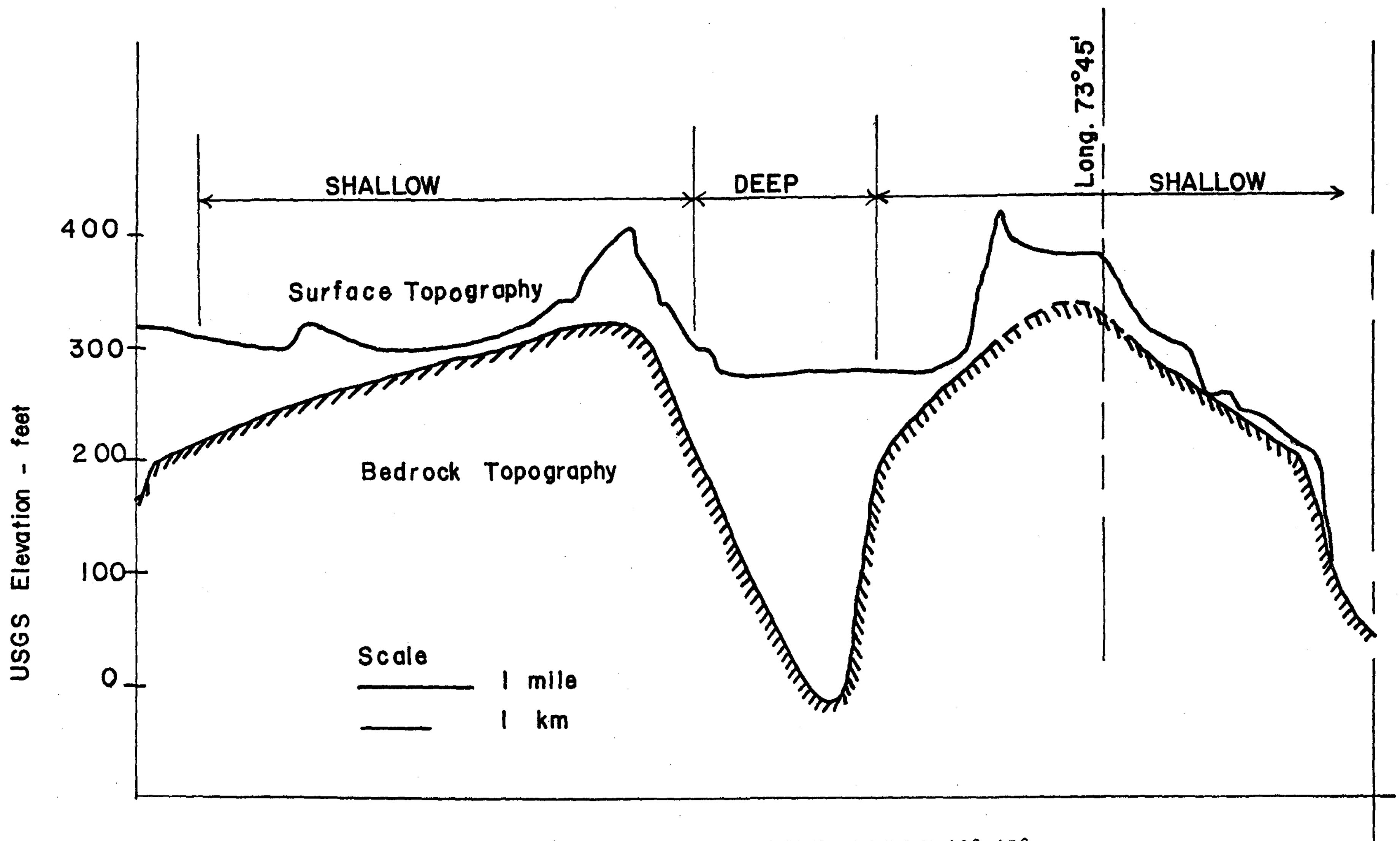

FIG. IX.3 SOIL-BEDROCK PROFILE LATITUDE 42°-45°

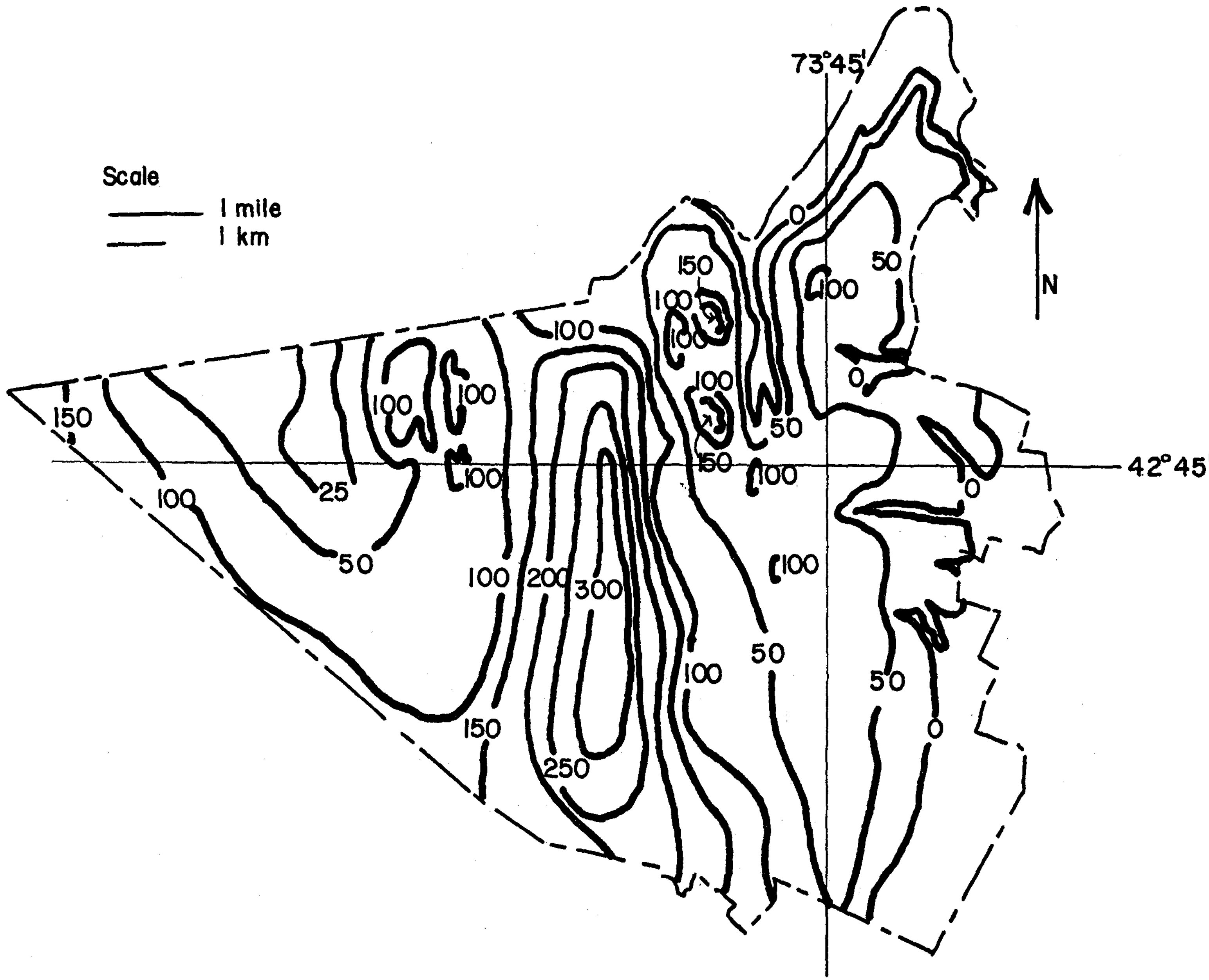

FIG. IX.4 LINES OF CONSTANT SOIL DEPTH - SOIL ISOPACHS (FEET)

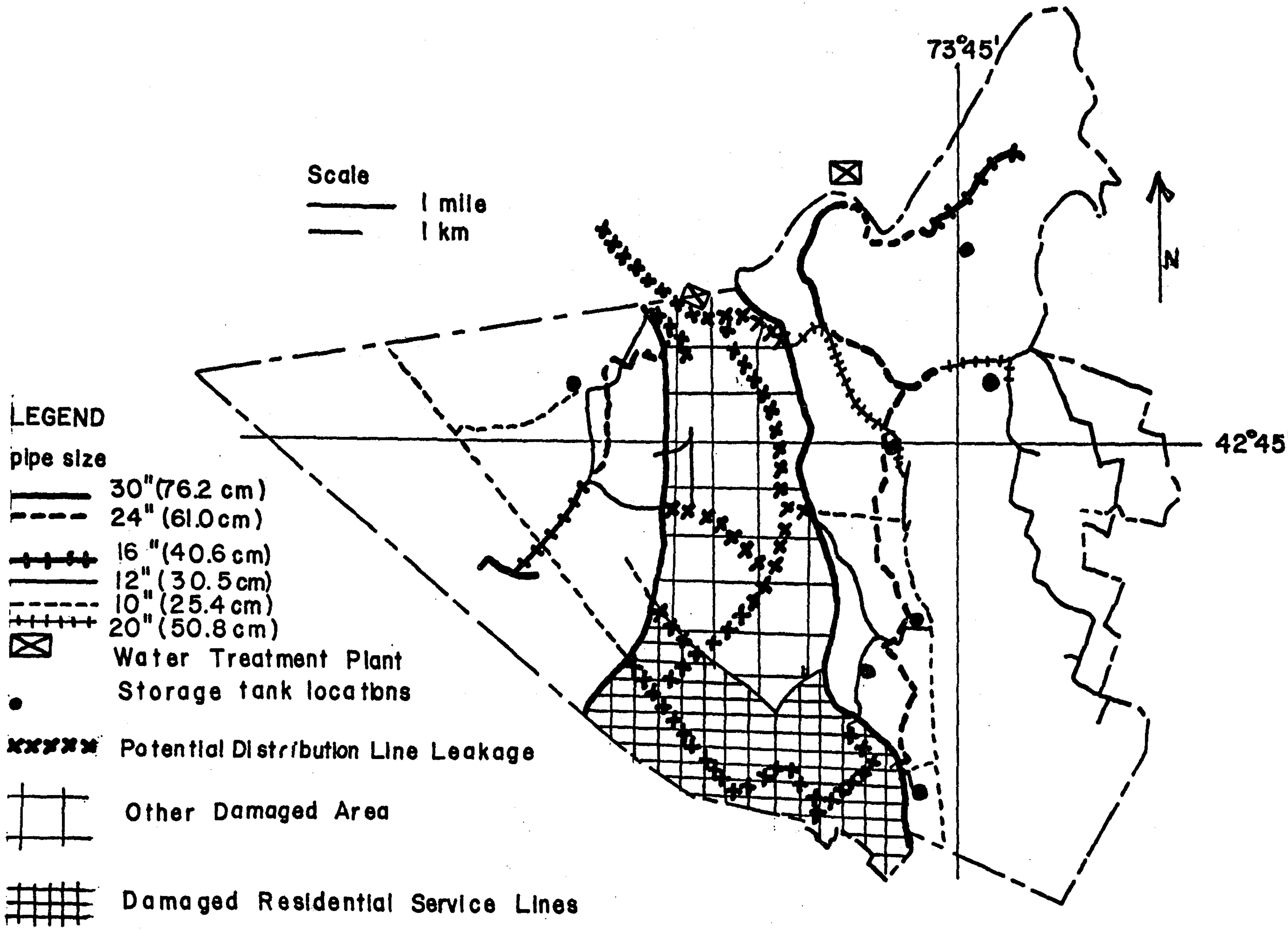

FIG. IX.5 POTENTIAL PIPELINE DAMAGE (100 FT. ISOPACH)

CHAPTER X CONCLUSIONS AND RECOMMENDATIONS

In this chapter the major conclusions of the report are summarized. A design procedure for simple pipelines subject to seismic wave propagation effects is outlined. In addition, recommendations for further research are proposed.

X.1 Seismic Behavior

Damage surveys supplemented by analytical investigations indicate the following general conclusions can be made about the behavior of buried pipelines subjected to seismic wave propagation effects.

- Pipelines with flexible joints experience less damage during earthquakes than pipelines with more rigid joints.
- Pipelines in regions of transition from one soil type to another experience more damage. Otherwise pipelines in soft soil experience more damage than pipelines in firm soil.
- The relative motion between the pipeline and the surrounding soil during earthquake excitation is small. In other words, the inertia forces generated by motion of the buried pipeline have little effect upon the response of the pipeline itself.
- Axial strains induced in a pipeline by seismic wave propagation are found to about an order of magnitude larger than the induced bending strains.

X.2 Design Considerations

There are three major causes of pipeline damage during earthquakes; soil liquefaction and landsliding, fault crossing, and wave propagation effects. Listed below are general items which should be considered in the design of buried pipeline for earthquakes.

- Pipeline construction on steep hillsides should be avoided when feasible.
- Redundancy in the distribution system is desirable.
- Installation of blow-off valves near the fault line, where higher seismic activity is anticipated, should be considered.
- Ductile pipe material such as steel, ductile iron, copper or plastic, etc. should be considered to allow for larger pipeline deformations.
- Flexible joints using rubber gaskets and ball-socket-type joints should be considered in areas of potentially strong seismic activity. Extra long restrained sleeves could be provided for sliding pipe connections.

X.3 Design Procedure for Wave Propagation Effects

Outlined below is a procedure which can be used to design "simple" pipelines for these effects. A description of what is meant by "simple pipelines is given in Chapter I.

- The designer in consultation with other interested parties must select an acceptable level of risk for the design life of the pipeline system. That is, a design event with a specific mean recurrance interval must be selected.
- The peak ground velocity and acceleration for the design event must be established. These values can be determined by referring to seismic risk studies published in the technical literature or by performing a detailed seismic risk analysis for the particular site. A detailed seismic risk analysis requires data on the

seismic activity of the region as well as an attenuation relationship for the region.

- The simplified procedures in Chapter III can be used to estimate the maximum pipe strain and relative joint displacement. In this case, geotechnical information about the site is required to determine the wave propagation velocities with respect to the pipeline.
- For a refined study, the "Quasi-static" analysis approach is recommended. Then additional geotechnical information is required to determine the stiffness of the soil springs.
- By comparing these pipe strains and relative joint displacements with the reserve pipe strains and ultimate joint expansions and contractions one can determine the possibility of pipe failure or cracking, or joint separation/crushing.

X.4 Recommendations for Further Studies

The investigation so far has concentrated on the "simple" piping systems. Significant progress toward the understanding of the behavior of the "simple" system has been made. However, for solving general piping systems involving complex soil and seismic environments, many more tasks need to be investigated before a comprehensive analysis/design procedure can be developed. The recommendations for further study are described below:

- Study of Ground Motion Characteristics With Varying Soil Conditions.
- Study of Ground Motion Characteristics With Varying Geological Environments.
- Detailed Analysis of Pipe/Joint Interaction at Junctions.

- Vulnerability/Serviceability of General Buried Piping Systems.
- Component Analysis/Design Procedure.
- System Analysis/Design Procedure.

APPENDIX - NOTATIONS

a = Cross section area of pipe

A = Ground acceleration

A_{max} = Maximum ground acceleration

A_{rad} = Radial ground acceleration

A_{tan} = Tengential ground acceleration

b = a constant or a parameter

c = a constant or a parameter

C = Propagation velocity of waves with respect to the pipeline

C_i = Wave propagation velocity for ith pipe segment

C_p = Longitudinal wave propagation velocity of controlling medium with respect to the pipeline

C_s = Transverse wave propagation velocity of controlling medium with respect to the pipeline

d = Nominal diameter of pipe

D = Outer diameter of pipe

D_f = Focal depth

D_r = Relative soil density

e = Log normal error term for attenuation relationship

E = Young's modulus of material

E' = Modulus of soil reaction in ring stress equation

f = soil resistance force to axial motion of pipe

$F(m)$ = cumulative distribution function

$g_a(x)$ = displacement of ground equivalent soil springs

G = shear modulus of soil

G_1, G_2 = constants

$h(t)$ = ground displacement time history function

H = Buried depth

i = an index

I = Moment of inertia of pipe

j = an index

J = Number of earthquakes

k_a = axial soil subgrade reaction

k_b = bending moment coefficient

k_o = coefficient of lateral soil pressure

k_x = deflection coefficient

$\overline{K}$ = Joint spring constant

K_a = axial soil resistant spring constant

K_b = lateral soil resistant spring constant

L = length of a pipe segment

L_{12} = separation distance between Station 1 and Station 2

m = magnitude of an earthquake

m_o, m_1 = lower and upper bound for earthquake magnitudes

M = Richter magnitude

n = number of pipe segment in a piping system

N = standard penetration resistance constant

N_m = the number of earthquakes whose magnitude is greater than or equal to m

NT = number of points in a digital record of a seismic event

p = internal pressure in pipe

p_h = lateral soil pressure on pipe

p_o = operating water pressure

p_s = surge pressure

p_y = probability that the ground motion will exceed y at the site

P = probability

P_v = equivalent vertical trench load on pipe

q_a = probability that the acceleration will be exceeded in T years

r = variable in radial direction or normal to pipe axis

R = outer radius of pipe

R_{xy} = cross correlation function

s = standard deviation

S = distance from the epicenter of the earthquake to the site of interest

t = time variable

t_o = thickness of pipe

T = time duration

u = soil displacement along the axis of the pipe

u_o = soil displacement at the interface with pipe

u_1, u_2 = rigid motion of pipe segment 1 and 2

U = relative joint displacement in axial direction

U_{max} = maximum relative joint displacement in axial direction of pipe

V = particle ground velocity

V_{max} = maximum particle ground velocity

V_{rad} = particle ground velocity in radial direction

V_{tan} = particle ground velocity in tangential direction

w = Imposed live load

W = equivalent vertical load

x = spatial variable along the axis of pipe

X_i = nodal displacement at end of ith pipe segment

X_{Gi} = ground displacement at ith node of a piping system

$X_1(t)$; $X_2(t)$ = digitized radial displacement time history for station 1 and station 2

Y = relative displacement between pipe and ground

Y_{max} = maximum relative displacement between pipe and ground

α = a constant or a parameter

β = a constant or a parameter

γ = shear strain of soil

γ_w = unit weight of soil

Γ_a = slippage stress around or near pipe surface

Δ_{max} = maximum ground displacement

ε = strain

ε_a = maximum axial strain in pipe

ε_{ai} = axial strain of ith segment

ε_{as} = seismic reserve axial strain of pipe

ε_f = flexural strain in pipe

ε_t = total combined axial and flexural strain in pipe

η = delay time

η_i = delay time of seismic wave from the beginning of a pipe system to ith pipe segment

θ = joint rotation of pipe

θ_{max} = maximum relative joint rotation of pipe

κ_1, κ_2 = constants

ν = occurrence rate

$\hat{\nu}$ = average occurrence rate times source area

ξ = time required for wave to propagate from Station 1 to Station 2

ρ = mass density of soil

σ = stress

$\sigma_{a,L}$ = local axial stress in pipe due to internal pressure

$\sigma_{a,s}$ = seismic reserve axial strength of pipe

$\sigma_{b,L}$ = longitudinal bending stress of pipe

$\sigma_{b,r}$ = ring bending stress in pipe due to equivalent vertical load

σ_{by} = modulus of rupture of pipe material

$\bar{\sigma}_{by}$ = reduced modulus of rupture of pipe material

$\sigma_{c,r}$ = combined ring stress in pipe

σ_m' = effective confining pressure

$\sigma_{t,r}$ = ring tensile stress in pipe due to internal pressure

σ_{ty} = tensile strength of pipe material

σ_y = yield strength of material

σ_1 = longitudinal stress in pipe

σ_2 = hoop stress in pipe

σ_{1y} = longitudinal strength of pipe material

σ_{2y} = hoop strength of pipe material

τ = shear stress

τ_o = shear stress at interface between pipe and soil

ϕ = friction angle of soil

χ = curvature of ground

χ_{max} = maximum curvature of pipe

$\{X\}$ = nodal displacement vector of pipeline

$\{X_G\}$ = ground displacement vector

$[K_{soil}]$ = soil resistance matrix

$[K_{system}]$ = structural system matrix of buried pipeline

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Jan. 1977

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An Overview of Buried Lifeline Earthquake Engineering
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Vibration Frequencies of Buried Pipelines
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Seismic Risk Analysis of Latham Water District,
Albany, New York
June 1977

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Earthquake Response of Buried Pipelines
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September 1978

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Seismic Vulnerability, Behavior and Design of Buried Pipelines
March 1979