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SUBJECT: Forwards addl info re underground fire main analysis.

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WASHINGTON PUBLIC POWER SUPPLY SYSTEM

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September 11, 1989
G02-89-158

Docket No. 50-397

U. S. Nuclear Regulatory Commission
Attn: Document Control Desk
Mail Station P1-137
Washington, D.C. 20555

Gentlemen:

Subject: NUCLEAR PLANT NO. 2
OPERATING LICENSE NPF-21
UNDERGROUND FIRE MAIN ANALYSIS [12" FP(43)-1]

Reference: See Attachment 1

In Reference 1) the Staff indicated that based on their review of the Supply System's submittals, they did not agree with our conclusion that the calculations submitted demonstrate that thrust restraint is not needed for 12" FP(43)-1. The letter requested that the Supply System provide a plan and schedule for completing the installation of an acceptable restraint within 120 days (due September 9, 1989.)

The Supply System advised the Staff informally that we would like the opportunity to provide new information not previously on the Docket that we believe addresses the Staff's concerns as contained in their Safety Evaluation Report (SER) attached to Reference 1). After the Staff has had time to evaluate this new information, the Supply System will be more than willing to meet and/or hold technical discussions as necessary.

According to the SER (bottom of page 2), the reasons for the Staff's findings were as follows:

1) "As stated in Reference 7, the theories of subgrade reaction (on which the licensee's analyses are based) should not be used for the purpose of estimating settlement or displacements, even though they could be reasonably relied on to compute stresses and bending moments in footings or mats; and"

2) "NFPA does not provide guidance regarding the use of the analysis as a means to show that soil alone can be relied upon to prevent movement of tees and bends in underground fire mains."

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With regard to the Staff's first concern, the basis for the Staff's rejection of the Supply System's methodology was described as being Conclusion No. 2 in "Evaluation of Coefficients of Subgrade Reaction" by K. Terzaghi (Reference 2). The conclusion states in part "...the theories of subgrade reaction should not be used for the purpose of estimating settlement or displacements." The Supply System agrees that the use of the coefficient of subgrade reaction is a valid concern because the relationship between displacement and subgrade reaction becomes non-linear at values greater than one-half the ultimate bearing capacity of the soil. The Supply System has revised our calculations to address this concern (CE-02-88-36, Rev. 2), and under separate cover has provided a copy for the Staff's evaluation. This revision addresses the above concern by assuring that the subgrade reaction value is less than one-half the ultimate bearing capacity of the soil.

In addition, the Supply System has discussed the use of this methodology with a nationally recognized expert, Mr. E. C. Goodling, Jr. A particularly appropriate and much more current paper on this subject is "Flexibility Analysis of Buried Pipe" (Reference 3), a copy of which is attached. In this ASME paper, a similar methodology is endorsed. In our discussions with Mr. Goodling, he confirmed the appropriateness of this methodology for this particular application.

The Supply System has concluded that it is valid to use subgrade reaction in the calculation of pipe displacement for this case, and that the expected displacement is very small and therefore the mechanical joint will not be disengaged and a thrust block is not required.

With respect to the Staff's second concern regarding NFPA's lack of guidance regarding the use of analysis, the Supply System has taken the following actions:

1) On November 3, 1988 the Supply System contacted Mr. Robert E. Solomon, (Reference 4), the NFPA's Staff Liaison. Mr. Solomon advised us that other "suitable means" as discussed in NFPA-24-1987 paragraph 8-6.2.8 would include the use of calculations to show that main movement will not cause a problem at the pipe joints and/or changes in direction. He further stated that the calculations should be submitted to the "Authority having jurisdiction" for their approval.

2) On November 28, 1988 the Supply System submitted CE-02-88-36, Rev. 1 to American Nuclear Insurers (the Authority having jurisdiction) for their review and concurrence (Reference 5).

3) On December 16, 1988, Mr. Al Baker of the American Nuclear Insurers (ANI) responded to the Supply System (Reference 6). ANI advised the Supply System that they would not require the installation of a thrust block or other form of pipe restraint for fire main 12"-FP-43-1 based on their review of calculation CE-02-88-36, Rev. 1, the Supply System's commitment to revise the calculations per our discussion with Mr. Baker, the ability to supply the D/G building sprinkler systems through alternate flow paths (never an issue), and the current terms and conditions of ANI's Property Damage Policy.

[illegible]

4) On February 14, 1989, the Supply System submitted Rev. 2 of the subject calculation to the ANI for their review and concurrence (Reference 7). A copy of this calculation has been made available to the Staff for their review.

5) On March 27, 1989 the ANI provided their concurrence of Rev. 2 of the subject calculations and reiterated that a thrust block would not be required (Reference 8).

6) On August 1, 1989 the Supply System requested that the NFPA provide their concurrence in writing that "other suitable means" included the use of calculations (Reference 9).

7) On August 8, 1989 the NFPA responded in the affirmative (Reference 10). In their response, they stated:

> "When the alternative means of pipe restraint is provided, a detailed analysis of the soil conditions and expected pipe forces is necessary to verify the design. The DIPRA manual as well as any civil engineering handbook provides the necessary guidance for proper pipe restraint when using alternative approaches."

The Supply System has reviewed the DIPRA manual (Reference 11), a design guide for thrust restraint of underground piping. This document discusses the same methodology for pipe restraint used by the Supply System in Rev. 2 of the subject calculation (CE-02-88-36), and provides additional confirmation that the Supply System's approach is valid and supports our conclusions.

In addition to the above expressed concerns, Mr. George Knighton informally asked the Supply System to look at other means of monitoring for pipe leakage. The Supply System is in the process of designing a monitoring system that we expect to have installed and operable by July 1, 1990. This system will include an indicating check valve that will enable us to quantify flow and thereby allow us to identify major leakage in the underground system. If such leakage is indicated, the Supply System would then take the necessary steps to determine if 12" FP (43)-1 was contributing significantly to such leakage.

This fire protection system has been in place for a number of years and has proven to be reliable. The system was initially pressure tested and successfully passed. The Supply System has responded to all of the Staff's expressed concerns. We have provided calculations using a methodology supported by a contemporary nationally recognized expert as well as other current industry documents. We have obtained the review and approval of the Authority having jurisdiction. We have obtained written verification from the NFPA that the use of calculations is acceptable. We have agreed to add a monitoring capability. In addition, it is the combined engineering judgement of the Supply System (including our consultant's), that even if the main were to fail catastrophically at the mechanical joint closest to the building it is not a safety issue, but rather a commercial concern.

Based on the above, we conclude that no thrust block is necessary for 12" FP(43)-1.

Very truly yours,

G. C. Sorensen, Manager
Regulatory Programs

HLA/bk

cc: JB Martin - NRC RV
NS Reynolds - BCP&R
RB Samworth - NRC
DL Williams - BPA/399
NRC Site Inspector - 901A

ATTACHMENT 1

1) Letter, GW Knighton (NRC) to GC Sorensen (SS), "WNP-2 Underground Fire Main Analysis TAC No. 64670", dated May 12, 1989

2) Terzaghi, Karl, "Evaluation of Coefficients of Subgrade Reaction", Geotechnique, Vol, 5, No. 4, December 1955

3)* E.C. Goodling Jr., "Flexibility Analysis of Buried Pipe", Contributed by the Pressure Vessels & Piping Division of the American Society of Mechanical Engineers for Presentation at the Joint ASME/CSME Pressure Vessel & Piping Conference, Montreal, Canada, June 25 - 30, 1978

4)* Record of Telephone Conversation between Mr. Dale Eggen (SS Principle Fire Protection Engineer) and Mr. Robert Solomon (NFPA 24 Staff Liaison), November 3, 1988

5) Letter, CM Powers (SS) to Al Baker (ANI), dated November 28, 1988

6)* Letter, 21902088, Al Baker (ANI) to Howard J. Fowler (SS), "Fire Main 12"-FP-43-1 Pipe Displacement Calculations", dated December 16, 1988

7) Letter, CM Powers (SS) to Al Baker (ANI), dated February 14, 1989

8)* Letter, 21900689, Al Baker (ANI) to Howard J. Fowler (SS), "Fire Main 12"-FP-43-1 Revised Pipe Displacement Calculations," dated March 27, 1989

9) Letter, D.H. Walker (SS) to Robert E. Solomon (NFPA) "NFPA 24", dated August 1, 1989

10)* Letter, RE Solomon (NFPA) to DH Walker (SS), same subject, dated August 8, 1989

11) "Thrust Restraint Design for Ductile Iron Pipe", by the Ductile Iron Pipe Research Association (DIPRA), second edition 1989

*Attached

REF 3
(1 of 2)

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Flexibility Analysis of Buried Pipe

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This paper discusses the process of performing a strength analysis for piping which is buried in soil. Beginning with the various geotechnic parameters which define the motion and forces at the soil/pipe interface and control the degree to which the pipe conforms to the soil motion (or non-motion in a thermal expansion analysis), methods are developed to calculate axial forces and bending moments at points of interest in the piping. Equations for Code stresses are presented and their applicability reviewed briefly.

Contributed by the Pressure Vessels & Piping Division of The American Society of Mechanical Engineers for presentation at the Joint ASME/CSME Pressure Vessels & Piping Conference, Montreal, Canada, June 25-30, 1978. Manuscript received at ASME Headquarters April 5, 1978.

Copies will be available until March 1, 1979.

Flexibility Analysis of Buried Pipe

E. C. GOODLING, JR.

NOMENCLATURE

- A = metal cross-section area of pipe
- a_m = maximum ground acceleration
- a, b, c = quadratic coefficients
- C_1, C_2, C_3 = coefficients for Hetenyi's equations
- C_R, C_s, C_p = Rayleigh, shear, and compression wave velocities
- D, D_o = pipe outside diameter
- E, E_p = Young's modulus for pipe
- E_s = Young's modulus for soil
- F_{max} = maximum axial force in pipe
- f = friction force per unit length
- G_s = soil shear modulus
- h = depth below grade
- i = stress intensification factor
- k = soil spring constant per unit length
- k_o = modulus of subgrade reaction
- L' = effective friction length
- ℓ_1, ℓ_2 = length of T and P legs, respectively
- ℓ_m = maximum slippage length
- M = moment
- P = system operating pressure
- p = soil confining pressure
- S = force in pipe at elbow
- S_h = allowable stress in pipe at operating temperature
- t = pipe wall thickness
- V_m = maximum soil velocity
- x = distance along P axis
- Z = section modulus of pipe
- α = coefficient of linear expansion
- γ = weight density of soil
- Δ = movement of element with respect to soil
- δ_T = temperature differential
- ϵ = strain
- ϵ_{ave} = average soil strain
- ϵ_m = maximum soil strain
- ϵ_t = thermal strain in pipe
- θ = angle of bend
- λ = system characteristic for pipe and soil
- μ = friction coefficient at soil/pipe interface
- υ, υ_s = Poisson's ratio for soil
- ρ = soil mass density
- σ_a = axial stress
- σ_b = bending stress
- σ_{bt} = thermal bending stress
- σ_e = net earthquake Code stress
- σ_p = longitudinal pressure stress
- σ_t = net thermal Code stress
- Ω = soil elasticity parameter

INTRODUCTION

The expanded use of continuous welded steel pipe to transport fluids underground and the accompanying awareness of the possibility of high normal stresses resulting from forces acting at the soil/pipe interface has made flexibility analysis of buried piping an important part of the power plant and process plant design function. Piping codes covering such piping are being conservatively applied by procuring and regulatory agencies in such a manner that often precludes the use of concrete or cast iron pipe in relatively short lengths, connected by ball and spigot or similar type joints. Such connections in themselves had introduced enough flexibility into the system that seismic or expansion stresses were seldom considered critical enough to warrant analysis. Welded steel piping conforming to ASME Specification SA-106 Grade B (ASTM A-106 Grade B) has become the usual choice for buried safety class piping in nuclear facilities because of its acceptability under Section III of the ASME Boiler and Pressure Vessel Code.

The calculated seismic or thermal stresses in a continuous welded steel pipe buried in compacted backfill or sand can be quite high, particularly where the pipe contains long straight runs with buried elbows or anchors on either or both ends. Damage to these pipes during an earthquake can be very extensive, most frequently due to axial tension and compression due to a traveling seismic wave with the damage most apparent at the various connections.

In the course of designing buried piping it is necessary to demonstrate analytically the capability of the pipe to withstand seismic soil strain or thermal expansion in the pipe, without exceeding Code allowable stress levels. When the calculated stresses exceed allowable stresses, it becomes necessary then to incorporate flexibility into the pipe design. Differential soil/pipe movements resulting from soil strain or pipe thermal expansion can then be accomodated without inducing high forces and moments at the elbows.

A rigorous analysis of buried piping taking into account complex geometry of the bends and elbows and the passive stiffening effects of the soil is difficult to accomplish. However, a simplified conservative analysis can be accomplished in a rather straightforward fashion, but considerable care is

necessary in establishing the geotechnical parameters and formulating the problem. It is necessary for the pipe stress analyst to determine exactly what is needed in the way of seismic and soils data so that they can be extracted from the site studies. Establishment of a close working relationship with the project engineer responsible for soils and foundations evaluation can be very useful to help in correctly interpreting the soils data in order to arrive at an accurate assessment of pipe stresses.

This paper presents a rational approach to stress analysis of buried piping bends, providing the engineer with an orderly process for assembling the necessary data, classification of a buried pipe by its dimensions and the nature of its end restraints, and derivation of a discrete set of equations for each classification to be used for the calculation of the forces at the soil/pipe interface and resulting forces and moments on the elbows or bends.

ANALYSIS

Assembling the Seismic and Soils Data

The seismic and soils data are used to describe the characteristics of the soil motion during an earthquake (acceleration, velocity, and displacement) and to assess the degree to which a buried pipe conforms to this motion. The maximum acceleration, a_m, to be used in analysis is obtained from the seismic history of the geographical location of the facility under investigation and the soil response curves for the site and elevation. From this, the maximum ground velocity V_m is found as follows[1]:

$$V_m = (48 \text{ in./sec})(a_m)^* \qquad (1)$$

where a_m is given in terms of g.

In order to calculate the displacement of the soil, it is necessary to determine the velocities of the various elastic waves which characterize soil response to an earthquake, including the shear wave, compression wave, and the surface (Rayleigh) wave. Once the shear wave velocity, C_s, and a value for Poisson's ratio, ν, for the soil have been established from geophysical site studies, the compression wave and Rayleigh wave velocities, c_p and c_R, can be determined by using Knopoff's curves (Figure 7). Maximum soil strain for each type of wave is calculated thus[2]:

$$\text{Shear wave-} \quad \epsilon_m = \pm V_m/2c_s \qquad (2)$$

$$\text{Compression wave-} \quad \epsilon_m = \pm V_m/c_p \qquad (3)$$

$$\text{Rayleigh wave-} \quad \epsilon_m = \pm V_m/c_R \qquad (4)$$

A value for maximum soil strain to be used for the piping stress analysis must be chosen with care. Chen[4] suggests combining the shear and compression wave velocities to calculate a design value for strain by:

$$\epsilon_m = \pm 2V_m/(c_s + c_p) \qquad (5)$$

Yeh[5] suggests averaging the strain from all three wave types, which can be expressed by:

$$\epsilon_m = \pm \frac{V_m}{3}\left(\frac{1}{c_p} + \frac{1}{2c_s} + \frac{1}{c_R}\right) \qquad (6)$$

Iqbal[6] proposes that where the depth of the buried pipe below the surface is small (about 15 to 20 feet), the Rayleigh wave predominates, and that maximum soil strain is most accurately expressed by (4).

Assuming the seismic waves to be sinusoidal, an argument can be made for refinement of strain calculation by determining an <u>average</u> strain level when the length of a pipe run along the axis of propogation approaches or exceeds half a wave length[6], typically on the order of 200-400 feet (60-120m). The expression for average strain, ϵ_{ave}, can be found by integrating the wave over the appropriate segment of the sine curve and dividing by the pipe length expressed in radians. For example, for a pipe length of a half a wave length (π radians in length)

$$\epsilon_{ave} = \frac{\epsilon_m \int_0^\pi \sin\theta d\theta}{\pi} = 0.637\ \epsilon_m \qquad (7)$$

Theoretically, in a pipe equal in length to a full wave length, average strain would be zero, but of course, there are regions which are very definetely subjected to substantial soil strains, and reduction beyond that of (7) should not be attempted. Nor should reduction of maximum soil strain values be considered unless sufficient site data are available to permit calculation of wave length with confidence.

If only thermal expansion of the pipe is to be considered instead of soil movements due to earthquake, the maximum strain to be used in the piping analysis becomes:

$$\epsilon_t = \alpha\delta_T \qquad (8)$$

Certain soils characteristics must be determined in order to properly assay the degree to which friction and bearing forces are imposed on the pipe. The modulus of subgrade reaction, k_o, is most important in that it defines the stiffness of soil when subjected to unit deflection. It is normally established by field tests or can be estimated by Vesic's equation[7]:

$$k_o = \frac{0.65}{D} \sqrt[12]{\frac{E_s D^4}{E_p I_p}} \left(\frac{E_s}{1-\nu_s^2}\right) \qquad (9)$$

where E_s, the soil modulus of elasticity, is found from $E_s = 2(1 + \nu_s)G_s$ where $G_s = \rho c_s^2$.

Friction effects along the soil/pipe interface may be established by methods proposed in Ref. 2. A simplified equation for friction force per unit length is:

$$f = \pi D \bar{p} \mu \qquad (10)$$

where $\bar{p} = \gamma h$. Whenever possible, the confining pressure, $\bar{p}$, should be obtained from civil engineering sources rather than calculated directly as above.

A value for coefficient of friction must be chosen with care, since friction is a function of many variables such as soil type, moisture content, consistency, degree of compaction, etc. A range of values may be determined, from which the lower bound value should be used as more conservative in the calculation of buried elbow moments, and upper bound values for analyzing straight runs of buried pipe.

* See SI Conversions Section.

In summary, it is best to consult the soils engineer for assistance in assembling the necessary soils and seismic data for use in the piping stress analysis. The state of the art in establishing values and ranges of values for use in the piping analysis is such that it requires considerable informed intuitive input, which is best handled by the soils specialist. However, the analyst must be very specific in requesting the sort of data that is needed and explaining how it will be used. It must be remembered, too, that seismic and soils characteristics vary with depth. Since a buried piping system may have runs, elbows, and anchors at several different depths, it is best to obtain a set of curves in which such items as shear wave velocity, maximum soil strain, soil confining pressure, and soil shear modulus are plotted against elevation. Then, specific values for any given elevation can be picked off the curves as they are needed in the analysis. Figures 8 through 11 constitute a set of typical curves established for a plant site.

Calculation of Intermediate Parameters

Before proceeding with the analysis, it is necessary to calculate some additional soil and pipe related parameters, namely the soil unit spring constant, k, the system characteristic, λ, and the minimum friction slippage length, ℓ_m. The soil unit spring constant (or spring constant per unit length) is found by:

$$k = k_o D \tag{11}$$

This is then used to calculate λ by:

$$\lambda = \sqrt[4]{\frac{k}{4EI}} \tag{12}$$

The characteristic λ is a factor derived by Hetenyi[8] which combines the properties of soil, pipe material, and pipe section properties and influences the shape taken by a beam (the pipe) on a continuous elastic foundation (the soil), when subjected to externally applied forces and moments.

The minimum slippage length, ℓ_m, is defined as the length of pipe necessary for the friction force along the pipe axis to develop fully so that a point is reached where the pipe and soil move together in a region of zero relative motion at the soil/pipe interface. The axial load F_{max} in the pipe in this region can be expressed by $f\ell_m$ and the slippage length is found by:

$$\ell_m = \frac{\epsilon AE}{f} \tag{13}$$

Classification of Pipe Bends

It is in the bends or elbow of buried piping that the highest moments occur as a result of soil/pipe differential motion,e (whether it is from soil strain or from pipe thermal expansion). It is true that the straight sections of buried piping systems experience both bending and axial stresses from soil strains; however, bending stresses conservatively calculated are shown to be quite low for typical seismic conditions and seldom deserve any attention. Also it can be said that, in general, the axial and bending stresses in the straight runs will always be less critical than those experienced by the elbows or bends.

Bends are classified by type according to the nature of their end constraints, that is, how each end of the longitudinal run (along the axis of strain propogation) terminates, either as an elbow, anchor, or free end. Within each type are sub-classifications as to whether the longitudinal run (called the pipe or 'P' leg) and the transverse run (called the transverse or 'T' leg) are long or short with respect to certain criteria.

In this case, buried pipes with bends are classified into three major types, according to the manner in which the end of longitudinal run, ℓ_2 or 'P' leg (along the strain axis) is restrained. The transverse run ℓ_1, or 'T' leg is the run against which the soil bears, producing an in-plane moment on the elbow. The longitudinal 'P' leg is the leg upon which friction at the soil/pipe interface is effective.

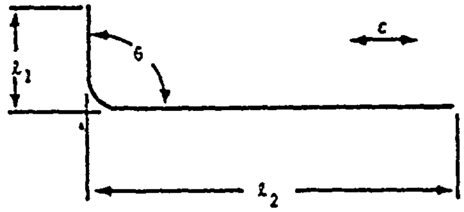

FIGURE 1 - Bend Type 1

This type bend consists of an elbow connecting a P leg and a T leg each with the ends free in the direction of strain.

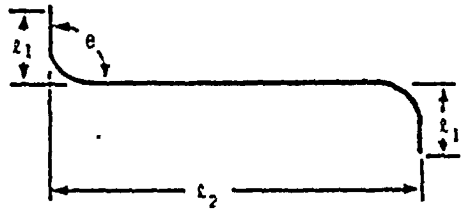

FIGURE 2 - Bend Type 2

This type bend consists of a P leg with elbows and T legs on both ends, with the ends free in the direction of strain.

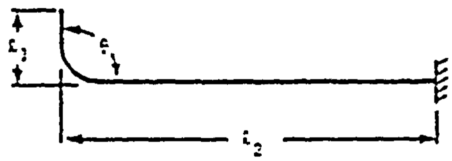

FIGURE 3 - Bend Type 3

This type bend consists of a P leg anchored at one end with a T leg on the other, with the T leg free in the direction of strain.

Each of the three main types of bends are then sub-classified by whether the transverse and longitudinal (T and P) legs are 'long' or 'short'. The sub-classifications may be abbreviated for convenience by using S and L to designate short or long, and P and T to identify the legs. For example, a Type 1-SPLT would refer to a Type 1 bend with a short longitudinal leg and a long transverse one.

The criterion for a short or long T leg is whether ℓ_1 is shorter or longer than $3\pi/4\lambda$ (the length at which the hyperbolic functions in Hetenyi's equations become unity, and the length beyond which there is negligible additional influence on the forces and moments on the elbow). If the T leg happens to be restrained laterally, and if its length ℓ_1 is less than $3\pi/4\lambda$, as perhaps at a building penetration, the additional effect of the lateral restraint would have to be evaluated. This could be done by considering the T end as free, using Hetenyi's equations to determine movement at the free end, applying a force at the free end to push it back to its zero position, and then superimposing the resulting moment at the elbow to the moment resulting from soil or pipe strain effects. If the T leg is longer than $3\pi/4\lambda$, the nature of the lateral restraint on the end is immaterial.

The criterion for a short or long P leg is whether or not ℓ_2 is sufficiently long to experience the maximum force that friction at the soil/pipe interface can develop fully. For maximum friction force to develop in a straight pipe free at both ends, its length ℓ_2 would have to be equal to or greater than $2\ell_m$. If a T leg is attached to one end, thus turning it into a Type 1 bend, the length ℓ_2 necessary for full friction to develop is reduced due to the fact that full friction need be developed only on one end, producing maximum axial force, F_{max}, which is balanced by partial friction force plus a shear force, S_1, due to soil bearing on the T leg. This partial friction force operates over an effective friction length L' which is less than ℓ_m. Shah and Chu[2] propose an equation for effective slippage length for a long pipe with a long transverse leg as follows:

$$L' = \frac{4}{3}\,\Omega\left(\sqrt{1 + \frac{3F_{max}}{2f\Omega}} - 1\right) \qquad (14)$$

where $\Omega = \dfrac{AE\lambda}{k}$

$F_{max} = \varepsilon AE$

However, if the T leg is shorter than $3\pi/4\lambda$, L' approaches ℓ_m as ℓ_1 approaches zero, and L' may be set equal to ℓ_m for purposes of classification. Therefore it appears reasonable to state that a long P leg is one which meets these criteria:

Type 1 $\ell_2 \geq \ell_m + L'$

Type 2 $\ell_2 \geq 2L'$

Type 3 $\ell_2 \geq L'$

In each case, the test value for L' is that calculated per Equation (14).

<u>Calculation of Forces and Moments</u>

Calculation of the forces and moments in buried elbows requires determination of effective slippage length, L', and the relative movement, Δ, between pipe and soil at the elbow. A general equation for Δ in terms of L' and S is written as follows[2]:

$$\Delta = \varepsilon L' - \frac{SL'}{AE} - \frac{fL'^2}{2AE} \qquad (15)$$

The terms in this equation are as follows: $\varepsilon L'$ is the theoretical unrestrained relative movement at the elbow over length L'; SL'/AE is the amount of pipe elongation due to the bearing force of soil against the T leg producing the shear force, S, at the elbow end which is transformed into an axial force in the P leg; and $fL'^2/2AE$ represents the pipe elongation due to friction along the soil/pipe interface which is found from $(f/AE)\int_0^{L'} x\,dx$.

Since Equation (15) contains three unknown quantities, Δ, S, and L', the next step is to write at least two of them in terms of the third, or else calculate them directly where possible. For LPLT subtypes (long pipe, long transverse), both legs can be assumed to be flexible (See Figure 13b). Then L' can be calculated from Equation (14) and S, M, and Δ as follows[2,3]:

$$S = \varepsilon AE - fL' \qquad (16)$$

$$M = \frac{S}{3\lambda} \qquad (17)$$

$$\Delta = \frac{4\lambda^2 M}{k} = \frac{4\lambda S}{3k} \qquad (18)$$

Note that Figure 13b indicates the presence of some shear force, S_2, and soil bearing forces transverse to the axis of the P leg. However, S_2 is substantially less than S (Ref. 3) and need not be calculated.

For SPST and SPLT subtypes, where the relative flexibility of the legs cannot be assumed with confidence, it is convenient to assume conservatively that the T leg is flexible while the P leg is inflexible (See Figure 13a). The force S and movement Δ can be expressed in terms of L' and (15) can be rewritten so that L' can be calculated directly. Starting with a diagram (Figure 10) of a Type 1 - SPST or - SPLT on which axial load is plotted along the P length ℓ_2 -

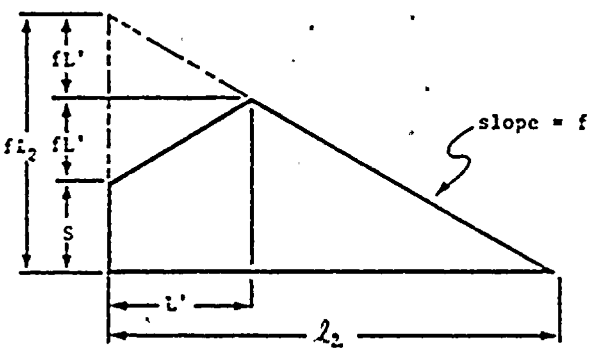

FIGURE 4 - Axial Force Diagram

simple geometry yields an equation for S:

$$S = f\ell_2 - 2fL' \qquad (19)$$

$$= f\,(\ell_2 - 2L')$$

Then, by taking Hetenyi's equations for a concentrated force and a concentrated moment on a finite beam on an elastic foundation, and superimposing them to produce zero rotation at the elbow, equations for S and M are obtained, as follows:

$$S = \frac{k\Delta}{\lambda}\frac{C_3}{C_2} \qquad (20)$$

$$M = \frac{k\Delta}{2\lambda^2}\frac{C_2}{(2C_1C_3 - C_2^2)} \sin\theta \qquad (21)$$

where $C_1 = \dfrac{\text{Sinh}\lambda\ell_1 \ \text{Cosh}\lambda\ell_1 - \sin\lambda\ell_1 \ \cos\lambda\ell_1}{\text{Sinh}^2\lambda\ell_1 - \sin^2\lambda\ell_1}$

$$C_2 = \frac{\text{Sinh}^2\lambda\ell_1 + \sin^2\lambda\ell_1}{\text{Sinh}^2\lambda\ell_1 - \sin^2\lambda\ell_1}$$

$$C_3 = \frac{\text{Sinh}\lambda\ell_1 \ \text{Cosh}\lambda\ell_1 + \sin\lambda\ell_1 \ \cos\lambda\ell_1}{\text{Sinh}^2\lambda\ell_1 - \sin^2\lambda\ell_1}$$

θ = angle between the T and P leg axes

Note that C_1, C_2, and C_3 = 1.0 when $\lambda\ell_1 = 3\pi/4$; therefore these coefficients are generally deleted from equations for bends with long T legs. Equation (20) is rewritten as follows to give Δ in terms of L':

$$\Delta = \frac{S\lambda}{k}\frac{C_2}{C_3}$$
$$= (\ell_2 - 2L')\frac{f\lambda}{k}\frac{C_2}{C_3} \qquad (20a)$$

Equation (15) can now be rewritten with Δ and S represented by Equations (19) and (20a) to give an expression for L' as a quadratic equation, as follows:

$$L'^2\left(\frac{3f}{2AE}\right) + L'\left(c - \frac{f\ell_2}{AE} + \frac{2f\lambda}{k}\right) - \frac{f\lambda\ell_2}{k} = 0 \qquad (22)$$

This can be readily solved for L' by using the quadratic form $L' = (-b + \sqrt{b^2 - 4ac})/2a$. Values for S, Δ, and M can then be obtained from (19), (20a), and (21).

For LPST subtypes, again the T leg may be conservatively assumed flexible and the P leg inflexible. Then L' may be found from:

$$L' = \ell_m = \frac{cAE}{f} \qquad (23)$$

As the T leg length becomes small, so does the total bearing force, S, of the soil on it, so that Equation (15) may be used to develop an expression for Δ which conservatively ignores the effect of S, as follows:

$$\Delta = cL' - \frac{SL'}{AE} - \frac{fL'^2}{2AE} \qquad (15a)$$

Rewriting (15a) and incorporating (23) gives:

$$\Delta = \frac{cL'}{2} \qquad (24)$$

Then, S and M can be calculated from (20) and (21). The calculation for Δ might be refined somewhat by considering the influence of S from (20) in (15), but the improvement is small. Also, the reasoning expressed by Shah and Chu[3] regarding the assumption of flexibility of both legs when both $\lambda\ell_1$ and $\lambda\ell_2$ exceed $3\pi/4$, M for SPLT may accordingly be calculated from (17).

For Type 2 SP subtypes,

$$L' = \frac{\ell_2}{2} \qquad (25)$$

Values for Δ are again calculated from (15). For ST subtypes, S may be ignored and Δ calculated from (15a). For LT subtypes, S becomes:

$$S = \frac{k\Delta}{\lambda} \qquad (20b)$$

Then substituting (20b) into (15) gives an expression for Δ:

$$\Delta = \frac{cL' - \dfrac{fL'^2}{2AE}}{1 + \dfrac{kL'}{\lambda AE}} \qquad (26)$$

and then S and M can be calculated from (20) and (21).

For Type 3 bends of subtype SP:

$$L' = \ell_2 \qquad (27)$$

For ST subtypes, Δ can be calculated from (15a), while for LT subtypes, Δ is found by (26). Then for Type 3 SPST and SPLT subtypes, S and M can be found from (20) and (21).

<u>Classification of Some Additional Elements</u>

In addition to bends, it is useful to identify some types of straight pipes and tees which need to be analyzed occasionally. Accordingly, a tee is designated Type 4, a straight pipe with both ends free is Type 5, and a straight pipe free on one end and anchored at the other is Type 6 (See Table 2).

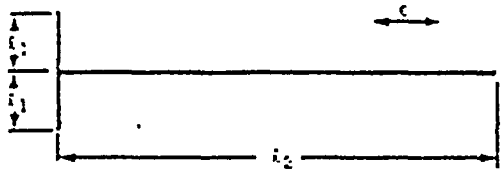

FIGURE 5 - Type 4, Pipe and Tee

For a tee Type 4 - LPLT, expressions for L', S, M, and Δ are given by Shah and Chu[3] as follows:

$$L' = \frac{1}{2}a\left(\sqrt{1 + \frac{4F_{max}}{f\,a}} - 1\right) \qquad (28)$$

where $\Omega = \frac{AE\lambda}{k}$

$$F_{max} = \epsilon AE$$

Furthermore,

$$S = \tfrac{1}{2}(\epsilon AE - fL') \text{ for each leg of the run pipe} \quad (29)$$

$$M = \frac{S}{2\lambda} \quad (30)$$

$$\Delta = \frac{S\lambda}{k} \quad (31)$$

Corresponding expressions for SP and ST subtypes and combinations with Types 2 and 3 can be developed as with the elbows but are not included herein.

For Types 5 and 6, L' is calculated as follows (S and M are zero):

For Type 5 - SP

$$L' = \frac{\ell_2}{2} \quad (32)$$

For Types 5 - LP and 6 - LP

$$L' = \ell_m \quad (33)$$

For Type 6 - SP

$$L' = \ell_2 \quad (34)$$

In all Types 5 and 6 pipes, Δ is calculated by:

$$\Delta = \epsilon L' - \frac{fL'^2}{2AE} \quad (35)$$

Axial Load Diagrams

Tables 1 and 2 present loading diagrams for various types of bends and other elements for which analysis may be required. For all types of buried piping and bends subjected to soil/pipe movements, there is either a point or region of zero relative movement at the soil/pipe interface on the P leg. The diagrams shown in Tables 1 and 2 show where this point or region is located for each type. The maximum axial force in the pipe will occur at this point and can be expressed by:

$$F_{max} = S + fL' \quad (36)$$

For a long pipe with a free end, where friction fully develops to produce a region of zero slippage, this force becomes:

$$F_{max} = \epsilon AE \quad (37)$$

The analyst must keep in mind that soil bearing force, S, and friction force, fL' or $f\ell_m$, are passive in nature and cannot exceed certain practical limits. Neither can occur until an actual relative movement occurs between pipe and soil. The bearing force cannot occur until some movement takes place to deform the soil normal to the T leg. Correspondingly, friction force acts only when there is slippage; it is zero at free ends, and at points or regions of zero slippage in the P leg.

Piping Code Stresses

Analysis of buried piping is generally accomplished to demonstrate structural integrity through compliance with applicable piping codes, either Section III of the ASME Boiler and Pressure Vessel Code (for nuclear facilities), ANSI B31.1 (for non-nuclear power piping) or ANSI B31.3 (for chemical plant and petroleum refinery piping).

Code analysis for earthquakes (called 'occasional' loading) requires that longitudinal stresses such as longitudinal pressure stress, σ_p, bending stress due to earthquake, σ_b, and other stresses (including, conservatively, the axial stress due to earthquake, σ_a), be combined to give a net axial stress which is then compared with established allowables[4]. Thus, where

$$\sigma_p = \frac{PD_o}{4t_n} \quad (38)$$

$$\sigma_b = 0.75i\frac{M}{Z} \quad (39)$$

i = appropriate stress intensifier as calculated for an unburied piping element (if 0.75i < 1.0, use 1.0)

$$\sigma_a = \frac{S}{A} \quad (40)$$

then the net axial stress is given by:

$$\sigma_e = \sigma_p + \sigma_b + \sigma_a \quad (41)$$

In the case of a thermal analysis, only the bending and axial stresses are calculated and combined as follows:

$$\sigma_{bt} = i\frac{M}{Z} \quad (42)$$

$$\sigma_a = \frac{S}{A} \quad (40)$$

$$\sigma_t = \sigma_{bt} + \sigma_a \quad (43)$$

The application of piping codes (which were written with point-supported and restrained piping in mind) to buried piping (which is supported and restrained continuously in an elastic medium) has been questioned. However, until the issue is addressed directly by the Code committees, it is believed conservative and appropriate to continue using the existing Codes as standards of acceptability of buried piping to withstand imposed stresses.

ORGANIZING AN ANALYSIS

Because of the variety of types and subtypes of bends and other elements of buried piping it has been found useful to organize and tabulate the various data items and their sources (See Table 3).

A suggested procedure for accomplishing a Code analysis of buried piping is as follows:

1. Obtain a layout drawing of the piping to be analyzed. The drawing should contain as a minimum, plan and section views with elevations of pipe and soil gradeline, all piping dimensions and material specifications, bend angles and radii, and operating data such as pressure and temperature.

2. Assemble the geotechnic data. This must include soil shear wave velocity, C_s, the soil shear modulus, G_s, confining pressure on the pipe, p, and soil strain, ϵ, preferably in the form of curves showing variation over the range of elevations of every buried element in the piping system. (If a thermal analysis is to be performed, ϵ will be calculated instead by the analyst). Also required will be Poisson's ratio, υ, and a range of values for friction coefficient, μ, at the soil/pipe interface.

3. Calculate the intermediate parameters k, f, λ, and ℓ_m for each pipe size and elevation in the system.

4. Identify and classify each bend or other element which is to be analyzed. An elbow may have to be analyzed twice, with each run being treated successively as a P leg and as a T leg.

5. Calculate L', Δ, S, M, and F_{max} as needed, using the prescribed equations noted in Table 3.

6. Calculate the Code stresses and compare with the allowable stress for the material.

If the stresses exceed the material allowable, it may be necessary to incorporate slip couplings or some other flexibility feature into the piping system design. If this is the case, some of the piping elements will need to be reclassified. For example, if a Type 3 bend is found to sustain stresses which exceed the allowable for the material, necessitating incorporation of a slip coupling between the anchor and the elbow, the result will be a combination of Type 1 and Type 6 elements. Slip couplings have been found to be a satisfactory means of incorporating flexibility into a buried pipe subject to axial soil/pipe differential strains, and in fact, have been incorporated into recent nuclear power plant design.

EXAMPLE

An emergency service water pipe as shown in Figure 6 is to be analyzed for stresses resulting from earthquake. The analysis is carried out in the 6 steps described above, as follows:

1. Description of the pipe and operating conditions.

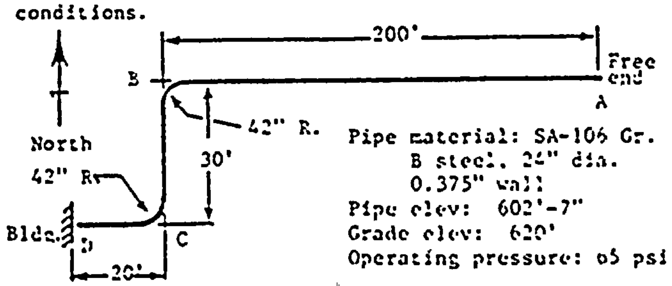

FIGURE 6 - Example Buried Pipe (plan view)

2. Geotechnic data (from soils engineering sources)

C_s = 895 ft/sec (Figure 8)
G_s = 8680 psi (Figure 9)
ϵ_m = 4.40 x 10^{-4} in./in. (Figure 10)
p = 15.55 psi (Figure 11)
υ = 0.40
μ = 0.24

3. Intermediate parameters (equations noted in parentheses)

k = 15,955 psi (9) and (11)
f = 281 lb/in. (10)
λ = 0.0162 in.$^{-1}$ (12)
ℓ_m = 1301 in. (13)

4. Classification of the Bends

With soil strains propogating in the east-west direction, legs AB and CD are treated as longitudinal (P) legs; therefore, elbow B is analyzed as a Type 1 bend, and elbow C as a Type 3 bend. With soil strain in the north-south direction, leg BC is the longitudinal leg and elbows B and C are analyzed as Type 2 bends. Since each leg is treated as a transverse leg at one time or another, and since each exceeds $3\pi/4\lambda$ (or 146"), all will fall into the 'long' transverse (LT) subtype.

a. For the Type 1 bend:

ℓ_2 = 200 ft (2400 in.)
L' = 924 in. (14)
ℓ_m = 1301 in.

Since 2400 in. > 1301 in. + 924 in., this bend may be fully classified as Type 1 - LPLT.

b. For the Type 2 bend:

ℓ_2 = 30 ft (360 in.)
L' = 924 in. (14)

Since 360 in. < 2(924 in.), these bends may be fully classified as Type 2 - SPLT and L' is calculated instead from (22).

c. For the Type 3 bend:

ℓ_2 = 20 ft (240 in.)
L' = 924 in. (14)

Since 240 in. < 924 in., this bend may be fully classified as Type 3 - SPLT and L' is calculated instead from (27).

5. Calculation of L', forces and moments

a. For the elbow at B as a Type 1 - LPLT bend:

L' = 924 in. (14)
S = 106,400 lb (16)
M = 2,190,000 in./lb (17)
F_{max} = 366,000 lb (37)

(Note: Δ does not need to be calculated for this subtype.)

b. For the elbows at B and C as Types 2 - SPLT:

$L' = 180$ in. (25)
$\Delta = 0.061$ in. (26)
$S = 60{,}100$ lb (20)
$M = 1{,}855{,}000$ in./lb (21)
$F_{max} = 110{,}700$ lb (36)

c. For the elbow at $\dot{C}$ as a Type 3 - SPLT bend:

$L' = 240$ in. (27)
$\Delta = 0.075$ in. (26)
$S = 73{,}900$ lb (20)
$M = 2{,}281{,}000$ in./lb (21)
$F_{max} = 141{,}340$ lb (36)

6. Calculation of Code stresses and comparison with allowables

Using equations (38) through (41), the ASME Boiler and Pressure Vessel Code Section III stress can be expressed by:

$$\sigma_e = \frac{PD}{4t} + 0.75i\,\frac{M}{Z} + \frac{S}{A}$$

For this 24 inch pipe with 36 inch radius elbows, $i = 4.2$, $A = 27.83$ in.^{2}, and $Z = 161.9$ in.^{3}. The net stress for each elbow is calculated as follows:

a. Type 1 - LPLT (Elbow B, soil strain east-west)

$$\sigma_e = 1040 + 42609 + 3823 = 47472 \text{ psi}$$

b. Type 2 - SPLT (Elbows B and C, soil strain north-south)

$$\sigma_e = 1040 + 36091 + 2160 = 39291 \text{ psi}$$

c. Type 3 - SPLT (Elbow C, soil strain east-west)

$$\sigma_e = 1040 + 44380 + 2655 = 48075 \text{ psi}$$

Assuming an allowable stress of 27,000 psi (equivalent to 1.8 S_h for SA-106, Grade B steel pipe), it is apparent that neither of elbows B and C satisfy the Code conditions. Therefore, it is necessary to design more flexibility into the pipe, either by using a larger bend radius to reduce the stress intensification, or by incorporating slip couplings. If the latter is chosen, the elbows then must be reclassified and analyzed accordingly.

CONCLUSIONS

This paper has discussed the process of performing a strength analysis for piping which is buried in soil. Beginning with the various geotechnic parameters which define the motion and forces at the soil/pipe interface and control the degree to which the pipe conforms to the soil motion (or non-motion in a thermal expansion analysis), methods are developed to calculate axial forces and bending moments at points of interest in the piping. Equations for Code stresses are presented and their applicability reviewed briefly.

Emphasis has been placed on developing an orderly classification of piping elements by the nature of their various end configurations, and the presentation of a tabulation of applicable equations to provide the analyst with a sequence of operations needed to perform a Code analysis.

This paper has not covered the effects of relative movements at building penetrations, which have been discussed to some degree in Ref. 2 and 5. It is believed, however, that the so-called 'guillotine' assumption (relative movements at the penetrations being sharp, well-defined differential movements at the soil/building interface) is very conservative. First, it is difficult to estimate such relative movements with any degree of confidence, and second, analysis by methods suggested in Ref. 2 nearly always implies an overstressed condition which must be remedied by providing for a 'soft' (flexible) penetration in the piping system design. It is easier to proceed at the outset to design the flexible penetrations and eliminate the need for comprehensive analysis.

A few conservatisms should be noted here. High calculated bending stresses in elbows generally result from the application of a stress intensification factor, i, to account for the secondary stresses which occur as a result of the ovalization tendency in the elbow cross-section. However, a natural passive resistance of the soil to ovalization exists, but is difficult to evaluate, so it has been conservatively ignored. Also, treatment of the P leg in the SP element subtypes as inflexible, for convenience of analysis, results in higher bending moments than are believed to exist. Further refinements in methods of calculating forces and moments in buried elbows using finite element methods of analysis can be expected to result in lower calculated values for a given set of conditions.

On the unconservative side, it has been suggested that the inertial reluctance of the pipe to conform to the cyclically straining soil could conceivably result in higher forces and moments than those computed from treating soil strain as a static phenomenon. The dynamic aspects of an earthquake with respect to buried piping may merit further investigation.

In summary, it is believed that the approach to flexibility analysis of buried piping presented by this paper is, on balance, conservative and reasonable in its application of seismic and soils parameters and its use of generally accepted piping code analysis procedures.

SI CONVERSIONS

1 in = 0.0254 M
1 ft = 0.3048 M
1 psi = 6.895 kPa
1 lb = 4.448 N

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FIGURE 7 CURVES PLOTTED BY KNOPOFF AS PRESENTED IN REFERENCE 2

RELATIONSHIPS BETWEEN VELOCITIES OF COMPRESSION WAVE C_P, SHEAR WAVE C_S, AND RAYLEIGH WAVE C_R FOR VALUES OF POISSON'S RATIO ν

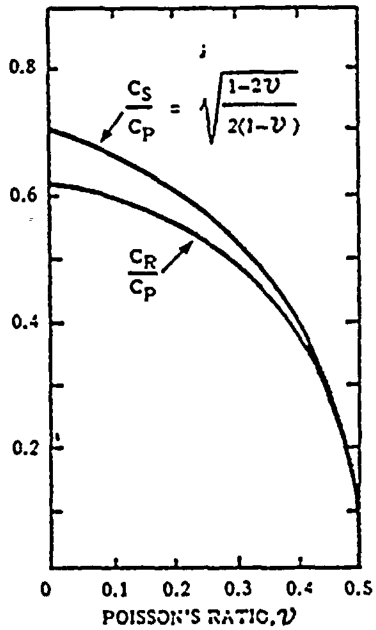


FIGURE 8 SHEAR WAVE VELOCITY C_s VS. ELEVATION

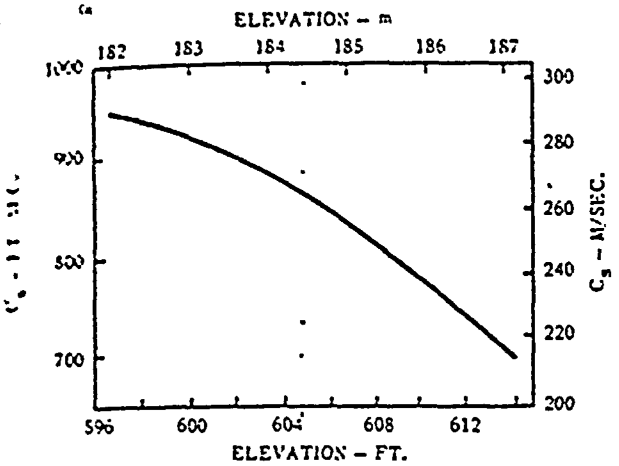


FIGURE 10 MAXIMUM SOIL STRAIN ϵ_m VS. ELEVATION

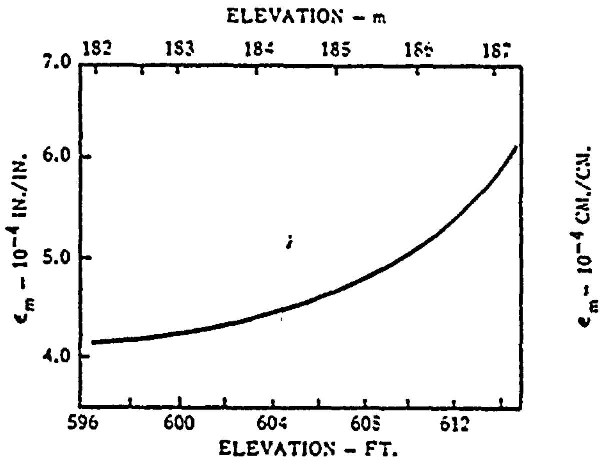


FIGURE 9 SOIL SHEAR MODULUS G_s VS. ELEVATION

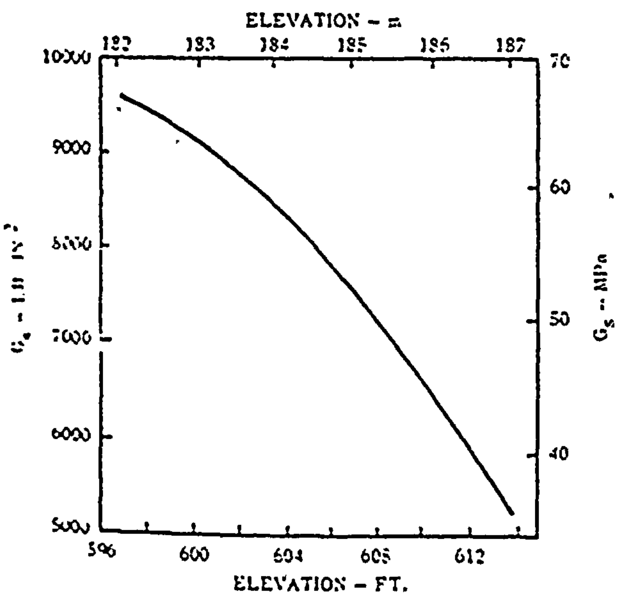


FIGURE 11 SOIL/PIPE CONFINING PRESSURE p VS. ELEVATION

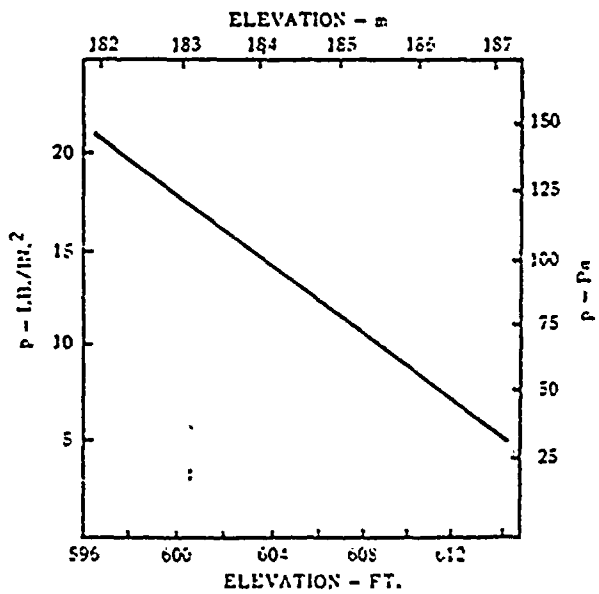
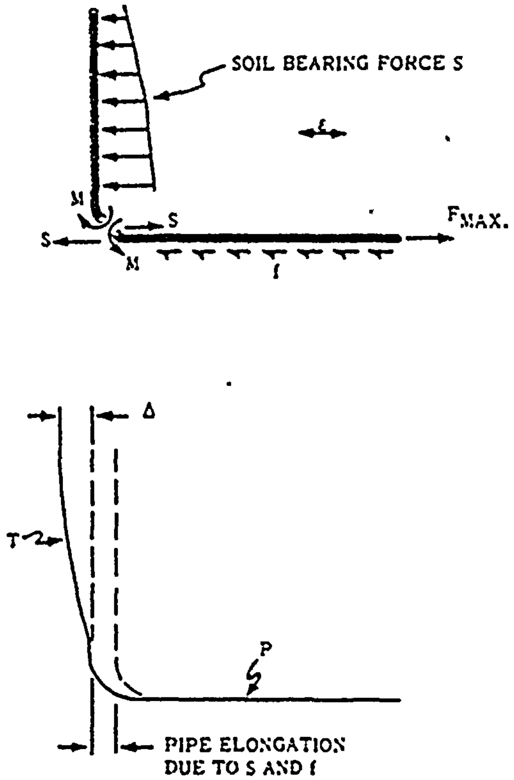
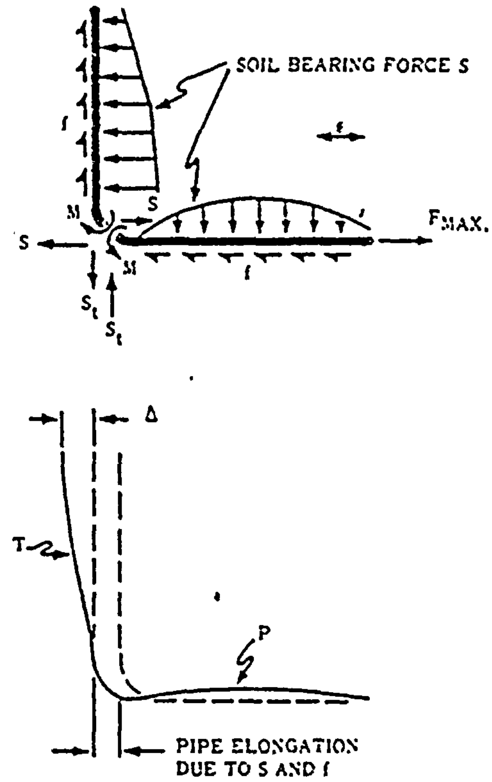


FIGURE 12 LOADING AND DEFORMATION DIAGRAMS FOR BURIED ELBOWS



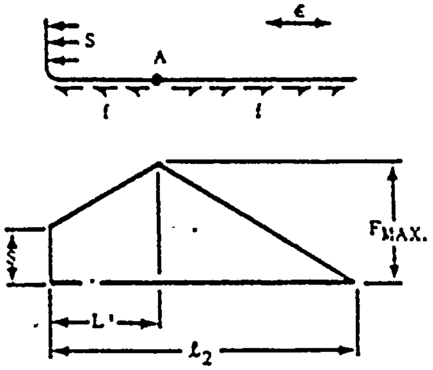
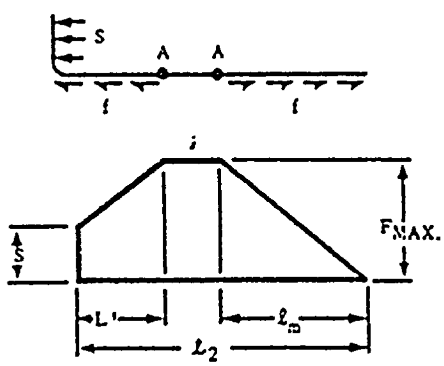
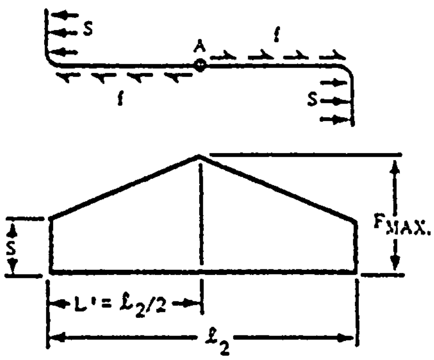
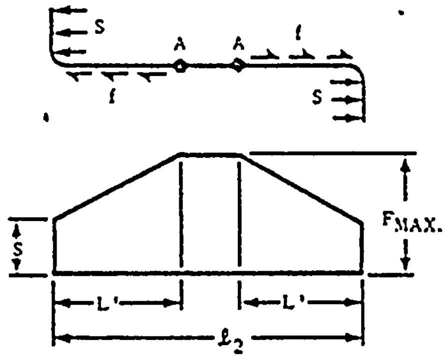
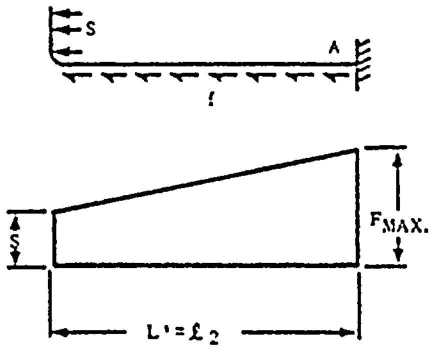
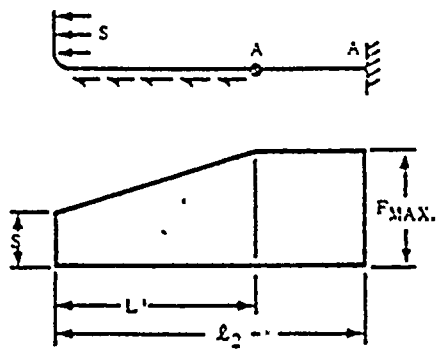
A. LONGITUDINAL (P) LEG
ASSUMED INFLEXIBLE



B. LONGITUDINAL (P) LEG
ASSUMED FLEXIBLE

TABLE 1 AXIAL LOADING DIAGRAMS FOR ELBOWS

(POINTS AND REGIONS LABELED A EXPERIENCE ZERO SLIPPAGE BETWEEN SOIL AND PIPE)

TYPE	SHORT PIPE	LONG PIPE
1		
2		
3		

REF 3
(2 of 2)

BURIED PIPING — AN ANALYSIS PROCEDURE UPDATE

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ABSTRACT

This paper is intended as a guide to be used for flexibility analysis of buried steel piping which is subjected to the effects of an earthquake or to thermal expansion. It is essentially a consolidation of previous papers on this subject presented by the author, to present clarifications, expand the coverage to include additional pipe configurations and to incorporate some new equations and updated treatment of certain variables involved in the analysis of buried piping.

Methods used to estimate analysis values for soil strain, modulus of subgrade reaction and friction at the pipe/soil interface are revised and simplified to a degree consistent with the certainty of site data. Classification of buried pipe elements is broadened to include more design cases, and several equations are refined to improve their usefulness to the analyst. An updated discussion of applicable piping code considerations is included.

INTRODUCTION

The flexibility analysis of buried steel piping fabricated from continuous welded runs and branches has progressed over the past several years from a highly idealized approach with many conservative assumptions to a moderately rigorous procedure with more rational use of seismic, soil and pipe physical characteristics. The use of the conservative theoretical approach does not really create many problems where new design is

NOMENCLATURE

- A = metal cross-section area of pipe
- a_{ij} = independent variable group
- a = radius of pipe section
- a, b, c = quadratic coefficients
- C_1, C_2, C_3, C_4 = coefficients for Hetenyi's equations
- C_R, C_s = Rayleigh and shear wave velocities
- d, D = pipe outside diameter
- E, E_p = Young's modulus for pipe
- E_s = Young's modulus for soil
- F_{max} = maximum axial force in pipe
- f = friction force per unit length
- G_s = soil shear modulus
- I, I_p = moment of inertia of pipe section
- i = stress intensification factor
- K, k_1 = dimensionless coefficients
- k = soil spring constant per unit length
- k_o = modulus of subgrade reaction
- L' = effective friction length
- ℓ_1, ℓ_2 = length of T and P legs, respectively
- ℓ_m = maximum slippage length
- M = moment
- P = system operating pressure
- r = elbow or bend radius
- S = force in pipe at elbow
- S_h = allowable stress in pipe at operating temperature
- t = pipe wall thickness
- V_m = maximum soil velocity
- Z = section modulus of pipe
- α = coefficient of linear expansion
- β = dimensionless coefficient
- Δ = movement of element with respect to soil
- δ = relative pipe/soil movement at a free end
- δ_T = temperature differential
- ϵ = strain
- ϵ_{ave} = average soil strain
- ϵ_m = maximum soil strain
- ϵ_t = thermal strain in pipe
- θ = angle of bend
- λ = system characteristic for pipe and soil
- μ = friction coefficient at soil/pipe interface
- ν_s = Poisson's ratio for soil
- ρ = soil mass density
- σ_a = axial stress
- σ_b = bending stress
- σ_{bt} = thermal bending stress
- σ_e = net earthquake Code stress
- σ_p = longitudinal pressure stress
- σ_t = net thermal Code stress
- Ω = soil elasticity parameter
- ϕ = seismic wave phase angle

involved. Buried piping shown by analysis to be overstressed by seismic or thermal effects can be redesigned to incorporate flexible couplings (Dresser style or similar) to minimize the buildup of stresses due to differential movement between the soil and the pipe. Such couplings have been incorporated effectively in buried water and petroleum transport piping for over a hundred years. The real problems with buried piping occur when it becomes necessary to demonstrate analytically that piping which has already been fabricated and buried will not be subjected to seismic or expansion stresses which exceed allowable levels.

Actual failure of the pressure boundary in buried pipelines subject to earthquake appears to occur primarily when there are severe dislocations or ruptures of the ground across which the pipe is laid (1). Even where there is severe ground displacement but no rupture, buried steel piping yields but rarely fails completely. However, piping can yield and buckle under high compressive loads, whether due to seismic soil strain or excessive thermal expansion of the pipe, resulting in an unacceptable reduction of fluid flow cross-section area (loss of functional capability) and consequent head loss.

It is therefore necessary for the practicing engineer to have at hand a rational procedure for calculating stresses in buried piping, particularly for bends and elbows. This paper presents such a procedure, including refinements, changes and corrections to previous published procedures (2 thru 6). It must be noted that the refinements are those which have evolved through analysis of buried safety class piping for a number of nuclear power plants and reflect the experiences and suggestions of a number of engineers throughout the United States and Canada.

ANALYSIS

Assembling the Seismic and Soils Data

The seismic and soils data are necessary to describe the characteristics of the soil motion during an earthquake and to assess the degree to which the pipe conforms to this motion. The maximum acceleration to be used in seismic analysis is the dynamic response of the soil at the elevation of interest as determined from the seismic history of the geographical location of the facility under investigation, as determined from the soil response curves for the site. From this, the maximum ground velocity V_m is calculated as follows (7):

$$V_m = (48\ \text{in/sec})g \qquad (1)$$

where g is the dimensionless ratio of the maximum soil particle acceleration to the gravitational constant.

A value for maximum soil strain to be used for buried piping analysis must be calculated with care. Chen (8), Yeh (9), and Iqbal (10) have recommended different ways of assuming the participation of the various seismic waves (shear, compression and Rayleigh) in the net seismic wave which results in seismic soil strain. However, although the velocities of propagation of the various waves can be determined from in situ and laboratory tests, it may be overly conservative to consider only the wave velocities of the soils immediately surrounding the buried piping (11). O'Rourke et al. (12) state that for a uniform soil layer overlaying bedrock, the propagation velocity of seismic waves along the pipeline is found to be strongly influenced by the wave velocity in rock, a low value for this being 2,000 ft/sec (600 m/sec). Measurements of seismic wave propagation velocities occurring during some recent Japanese earthquakes and during the San Fernando earthquake of 1973 suggest actual wave velocities in excess of 7,000 ft/sec. There has therefore evolved a consensus among consultants that the use of any wave velocity of less than 2,000 ft/sec to calculate soil strain is overly conservative where the elevation above bedrock is less than one wavelength, typically 200 to 400 feet (60-120 m) (11).

Maximum seismic soil strain ϵ_m is calculated by:

$$\epsilon_m = \pm \frac{V_m}{C} \qquad (2)$$

where C is the velocity of propagation of the seismic wave along the pipeline axis. Where the elevation above bedrock is known to be one wavelength or less, a value for C of 2,000 ft/sec (600 m/sec) is recommended for use. If the elevation above bedrock is unknown or significantly exceeds one wavelength, the velocity of the Rayleigh (surface) wave C_R as determined from site tests should be used for C, especially if the pipe is buried within 10 or 15 feet (3 or 4 m) of the ground surface.

Seismic waves are assumed for convenience to be sinusoidal, and an argument can be made for further refinement of soil strain calculations for pipe runs which are longer than one half wavelength. Since (2) represents a maximum value which, at any instant, occurs only at a single point along the pipe, a more realistic value for strain along the pipe run can be found by calculating the average strain ϵ_{ave} by:

$$\epsilon_{ave} = \frac{\epsilon_R \int_0^{\pi} \sin\theta\, d\theta}{\pi} = 0.637\ \epsilon_m \qquad (3)$$

where ϵ_R is soil strain based on C_R.

Theoretically, in a pipe equal in length to a full wavelength, average strain would be zero, but of course there are regions which are definitely subjected to substantial soil strains, and reductions in calculated strain beyond that afforded by (3) are not recommended nor should (3) be used where the pipe run length is less than one half wavelength, or where site data is insufficient to permit calculation of wavelength with confidence.

If only thermal expansion of the pipe is to be considered instead of soil movements due to earthquake, the strain value ϵ_t to be used in the following analysis of buried piping is calculated by:

$$\epsilon_t = \alpha \delta T \qquad (4)$$

Unlike seismic analysis, though, where seismic strain is assumed to act along the longest run at any given moment, thermal strain acts along every run. Therefore, a thermal analysis of a buried elbow requires analysis of each run separately and combination of the resulting stresses in the connecting elbow.

Certain soils characteristics must be determined in order to establish the degree to which friction and bearing forces are imposed by the soil on the pipe. The modulus of subgrade k_o defines the stiffness of soil when subjected to bearing loads in the course of restraining a pipe. There are a number of methods used for determining k_o, the most accurate of course, being actual field testing of the compacted soil surrounding the pipe. In the absence of accurate field data, some empirical methods have been developed for estimating k_o. Vesic (13) proposes the following (in English units):

$$k_o = \frac{0.65}{D} \sqrt[12]{\frac{E_s D^4}{E_p I_p}} \left(\frac{E_s}{1-\upsilon_s^2}\right) \quad (5)$$

where k_o may be thought of as the spring rate of soil under the bearing pressure imposed by a pipe in dimensions of pounds/sq inch of bearing area/inch of deflection. In order to find k_o by Vesic's equation (5), E_s may be calculated from:

$$E_s = 2(1 + \upsilon_s)G_s \text{ where } G_s = \rho C_s^2 \quad (6)$$

The analyst must be careful to maintain dimensional consistency since the shear wave velocity is usually given in ft/sec or m/sec., while the usual dimensions given for E_s and G_s are psi or ksi. Similarly, when k_o is developed from site testing, the dimensions are often given as pounds per foot cubed, which must then be converted to pounds/in.^{2}/in. for subsequent calculations. From k_o, the spring constant k is calculated by:

$$k = k_o D \quad (7)$$

This value for k, which is used to determine the pipe/soil system characteristic and the soil deformation Δ in subsequent calculations, is usually referred to as the soil unit spring constant, with dimensions of pounds/inch squared (psi).

Returning momentarily to Vesic's equation (5), it will be noted after a few trials by the analyst that the 12th root of the value under the radical, for a typical value for G_s of 8,680 psi, ranges from 0.76 for 2" Schedule 40 pipe to 0.87 for 30" standard wall (0.375) pipe, being affected only slightly by variations in E_s or υ_s. Since the values used for υ and E_s are often approximate at best, it appears that a simpler means of calculating k_o and k might be suitable. Accordingly, Parmelee and Ludtke (14) have proposed a more direct method, by which for υ_s of 0.5 and ratio of depth below grade to pipe radius of 30 or greater, $k = 1.12\ E_s$. Dr. M. Ayub Iqbal (10) suggests that setting k equal to 0.6 E_s would be appropriate for the usual range of υ_s (0.3 to 0.5).

Soil Friction Effects

In analyzing straight runs of buried pipe, it is acceptable to use upper bound values for friction force per unit length at the pipe/soil interface (5). However, in analyzing buried elbows for seismic stresses, it is necessary that friction force be established conservatively on the low side. Soil forces normal to the pipe may be determined by the modified Marston equation as follows:

$$W_c = C_d w B_d D \quad (8)$$

in which W_c = deadload on pipe per unit length

w = soil density, weight per unit volume

B_d = trench width at top

D = pipe diameter

C_d = dimensionless coefficient based on backfill

(See Figure 10)

Friction force f at the pipe/soil interface can then be found by:

$$f = \mu W_c \quad (9)$$

The value for μ must be selected carefully since friction is influenced by such variables as soil type, moisture content, consistency, compaction, etc. (3). Where friction coefficient values exceed 0.5, the internal friction within the soil itself dominates. It is recommended that a soils engineer be consulted for assistance in establishing an appropriate range of values for μ from site-specific data.

Calculation of Intermediate Parameters

After the strain ε, soil unit spring constant k, and the friction force per unit length f have been determined, it is necessary to calculate two additional parameters, namely the pipe/soil system characteristic λ and the minimum slippage length L_m.

The system characteristic λ is a factor derived by Hetenyi (15) which combines the stiffness of the soil in k and of the pipe material in E with pipe section property I to account for these influences on the shape taken by the pipe on the continuous elastic foundation when subjected to externally applied forces and moments. The system characteristic is found by:

$$\lambda = \sqrt[4]{\frac{k}{4EI}} \quad (10)$$

The minimum slippage length L_m is defined as the length of pipe necessary for the friction force along the pipe axis to develop fully so that a point is reached where the pipe and soil move together in a region of zero relative motion at the pipe/soil interface. The minimum slippage length L_m and corresponding maximum axial force F_{max} in the pipe are found by:

$$L_m = \frac{\varepsilon AE}{f} \quad (11)$$

$$F_{max} = f L_m = \varepsilon AE \quad (12)$$

Classification of Pipe Elements

It is in the bends or elbows of buried piping that the highest moments occur as a result of soil/pipe differential motion Δ (whether it is from soil strain or from pipe thermal expansion). It is true that the straight sections of buried piping systems experience both bending and axial stresses from soil strains; however, bending stresses conservatively calculated are shown to be quite low (Ref. 4, 9) for typical seismic conditions and seldom deserve any attention. Also, it can be said that, in general, the axial and bending stresses in the straight runs will always be less critical than those experienced by the elbows or bends.

Pipe elements are classified by type according to the nature of their end constraints, that is, how each end of the longitudinal run (along the axis of strain propagation) terminates, either as an elbow, anchor, or free end. Within each type are sub-classifications as to whether the longitudinal run (called the pipe or 'P' leg) and the transverse run (called the transverse or 'T' leg) are long or short with respect to certain criteria (4).

In this case, buried pipes with bends are classified into three major types, according to the manner in which the end of longitudinal run, L_2, or 'P' leg (along the strain axis) is restrained. The transverse

run ℓ_1, or 'T' leg is the run against which the soil bears, producing an in-plane moment on the elbow. The longitudinal 'P' leg is the leg upon which friction at the soil/pipe interface is effective.

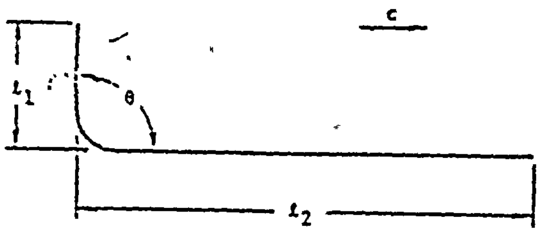


Fig. 1 Bend Type 1

This type bend consists of an elbow connecting a P leg and a T leg each with the ends free in the direction of strain.

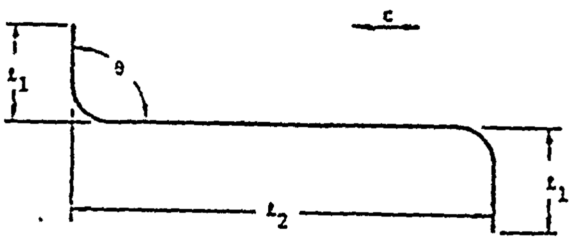


Fig. 2 Bend Type 2

This type bend consists of a P leg with elbows and T legs on both ends, with the ends free in the direction of strain.

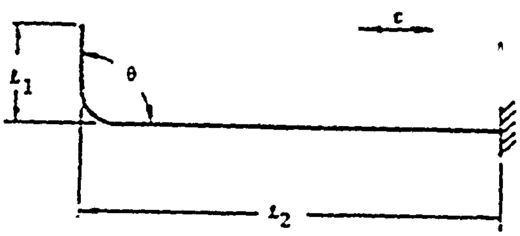


Fig. 3 Bend Type 3

This type bend consists of a P leg anchored at one end with a T leg on the other, with the T leg free in the direction of strain.

Each of the three types of bends are then sub-classified by whether the transverse and longitudinal (T and P) legs are 'long' or 'short'. The sub-classifications may be abbreviated for convenience by using S and L to designate short or long, and P and T to identify the legs. For example, a Type 1 - SPLT would refer to a Type 1 bend with a short longitudinal leg and a long transverse one.

Additional element types are T types as shown in Figures 4 and 5, with one pipe end axially free or anchored respectively.

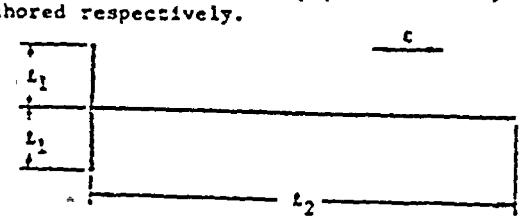


Fig. 4 Element Type 4

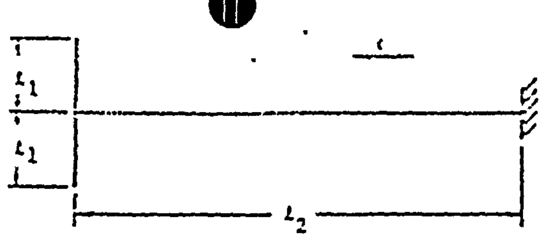


Fig. 5 Element Type 5

The strict criterion for a long or short transverse (T) leg is whether ℓ_1 is longer or shorter than $3\pi/4\lambda$, the length at which the hyperbolic functions in Hetenyi's equations approach unity (and the length beyond which there is negligible additional influence on the forces and moments at the elbow). Actually, a boundary condition for ℓ_1 of as little as $3\pi/8\lambda$ may be used if a 10% error in calculated results can be tolerated. What this all implies is that the largest portion of the bearing load on the transverse leg occurs in the first few feet of the pipe at the elbow or bend.

The criterion for a short or long pipe (P) leg (the leg running in the direction of wave propagation or maximum movement at the pipe/soil interface) is whether or not ℓ_2 is sufficiently long to experience the maximum force that develop at the friction interface. For maximum friction force ($F_{max} = cAE$) to develop in a straight pipe free axially at each end, its length ℓ_2 would have to be $2\ell_m$ as calculated by (11). If one end terminates in a transverse (T) leg, thus turning it into a Type 1 element, the total length ℓ_2 necessary for full friction to develop is reduced to $L' + \ell_m$; the friction force over length ℓ_m at the free end is balanced by the soil bearing force S, on the transverse leg plus the soil friction force acting over a reduced (or 'effective') slippage length L' in the P leg at the elbow. Shah and Chu (2) propose equations for calculating effective slippage length for long pipes with long transverse legs, as follows:

LPLT

For bends:
$$L' = \frac{4}{3}\,\Omega\left(\sqrt{1 + \frac{3F_{max}}{2f\Omega}} - 1\right) \qquad (13a)$$

For tees:
$$L' = \frac{1}{2}\,\Omega\left(\sqrt{1 + \frac{4F_{max}}{f\Omega}} - 1\right) \qquad (13b)$$

where $\Omega = \frac{AE\lambda}{k}$ and F_{max} is found from (12)

Another equation for this L' can be derived by solving Equation (23) for the boundary condition where $\ell_2 = L' + \ell_m$, this being a singular condition where ℓ_2 is the length at which the transition from short pipe to long pipe occurs. Using the Shah and Chu notation gives the following:

$$L' = \Omega\left(\sqrt{1 + \frac{2F_{max}}{f\Omega}} - 1\right) \qquad (14)$$

This equation applies to both bends and tees. Although (14) was derived for the case where $\ell_2 = L' + \ell_m$, it applies also for any case where $\ell_2 > L' + \ell_m$, since the length of the region of zero slippage at the friction interface is immaterial. Using L' as calculated by (14), it can now be established that a P leg can be classified as long if it meets these criteria:

Type 1 $\ell_2 \geq \ell_m + L'$

Type 2 $\ell_2 \geq 2L'$

Type 3 $\ell_2 \geq L'$

Type 4 $\ell_2 \geq \ell_m + L'$

Type 5 $\ell_2 \geq L'$

Calculation of Forces and Moments for Elbows and Tees

Calculation of forces and moments in buried elbow and tee elements requires that the effective slippage length L' and relative movement Δ between pipe and soil (magnitude of soil distortion) at the element be found. A general equation for Δ in terms of L' and the bearing force S at the elbow is written as follows:

$$\Delta = \epsilon L' - \frac{SL'}{AE} - \frac{fL'^2}{2AE} \qquad (15)$$

The terms in this equation represent various contributions to the net movement of the restrained elbow, as follows:

1. $\epsilon L'$ is the theoretical unrestrained relative movement of soil at the elbow over length L'.
2. SL'/AE is the amount of pipe elongation due to the bearing force S on the transverse leg or legs, which is transferred through shear at the elbow or tee into an axial force in pipe run.
3. $fL'^2/2AE$ is the amount of pipe elongation obtained by integrating the friction force at the pipe/soil interface over length L'.

Since (15) contains three dependent variables Δ, S, and L', it is necessary to express at least two of them in terms of the third or to calculate them directly where possible. For LPLT elbow subtypes (long pipe, long transverse) and assuming both legs to be flexible with the elbow considered inflexible (See Figure 2), L' can be calculated from (14) and S, M, and Δ can be calculated from the following (2), (3):

$$S = \epsilon AE - fL' \qquad (16)$$

$$M = S\sin\theta/3\lambda \qquad (17)$$

$$\Delta = 4\lambda^2 M/k = 4\lambda S/3k \qquad (18)$$

For Types 1 and 4 elements with SPST and SPLT subtypes, where the relative flexibility of the legs cannot be assumed with confidence, it is convenient to assume conservatively that the T leg is flexible while the P leg or legs are inflexible (See Figures 7 & 8). The force S and movement Δ can be expressed in terms of L' and (15) can be rewritten so that L' can be calculated directly. Starting with a diagram (Figure 6) of a Type 1 - SPST or - SPLT on which axial load is plotted along the P length ℓ_2:

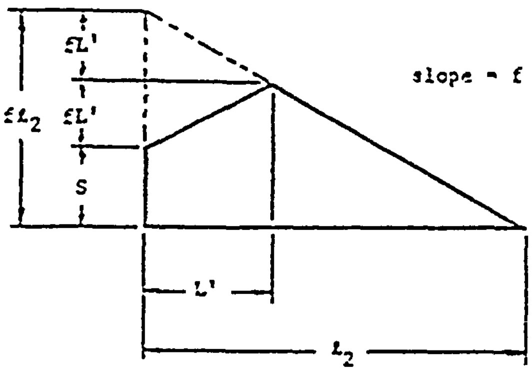


Fig. 6 Axial Force Diagram

Simple geometry yields an equation for S:

$$S = f\ell_2 - 2fL' = f(\ell_2 - 2L') \qquad (19)$$

Then, by taking Hetenyi's equations for a concentrated force and a concentrated moment on a finite beam on an elastic foundation, and super-imposing them to produce zero rotation at the elbow, equations for S and M are obtained, as follows:

$$S = \frac{k\Delta}{\lambda}\,\frac{C_2\sin\theta}{C_4} = 2M\lambda\,\frac{C_3}{C_2\sin\theta} \qquad (20)$$

$$M = \frac{k\Delta}{2\lambda^2}\,\frac{C_2\sin\theta}{C_4} \qquad (21)$$

where
$$C_1 = \frac{\sinh\lambda\ell_1\cosh\lambda\ell_1 - \sin\lambda\ell_1\cos\lambda\ell_1}{\sinh^2\lambda\ell_1 - \sin^2\lambda\ell_1}$$

$$C_2 = \frac{\sinh^2\lambda\ell_1 + \sin^2\lambda\ell_1}{\sinh^2\lambda\ell_1 - \sin^2\lambda\ell_1}$$

$$C_3 = \frac{\sinh\lambda\ell_1\cosh\lambda\ell_1 - \sin\lambda\ell_1\cos\lambda\ell_1}{\sinh^2\lambda\ell_1 - \sin^2\lambda\ell_1}$$

$$C_4 = 2C_1C_3 - C_2^2$$

θ = angle between the P and T leg axes.

Note that C_1, C_2, C_3 and C_4 approach 1.0 when $\lambda\ell_1 = 3\pi/4$; therefore, these coefficients are generally deleted from equations for bends with long T legs. Equation (20) is rewritten to incorporate (19) as follows to give Δ in terms of L':

$$\Delta = \frac{S\lambda}{k}\,\frac{C_4}{C_3\sin\theta} = (\ell_2 - 2L')\frac{f\lambda C_4}{kC_3} \qquad (22)$$

Equation (15) can now be rewritten with S and Δ represented by Equations (19) and (22) to give an expression for L' as follows (assuming the constants C_1 through C_4 to be unity):

$$\frac{3f}{2AE}L'^2 + \left(c - \frac{f\ell_2}{AE} + \frac{2f\lambda}{k}\right)L' - \frac{f\lambda\ell_2}{k} = 0 \qquad (23)$$

which can readily be solved for L' by using the following quadratic solution form:

$$L' = \frac{-b + \sqrt{b^2 - 4ac}}{2a}$$

Values for S, Δ, and M for SP subtypes can then be found from (20), (22) and (21).

For LPST subtypes, again the T leg may be conservatively assumed flexible and the P leg inflexible. Then L' may be found from:

$$L' = \ell_m = \frac{cAE}{f} \qquad (24)$$

As the T leg length becomes small, so does the total bearing force, S, of the soil on it, so that Equation (15) may be used to develop an expression for Δ which conservatively ignores the effect of S, as follows:

$$\Delta = cL' - \cancel{\frac{SL'}{AE}}^{\,0} - \frac{fL'^2}{2AE} \qquad (15a)$$

Rewriting (15a) and incorporating (24) gives:

$$\Delta = \frac{cL'}{2} \qquad (25)$$

Then, S and M can be calculated from (20) and (21). The calculation for Δ might be refined somewhat by considering the influence of S from (20) in (15), but the improvement is small. Also, the reasoning expressed by Shah and Chu (2) regarding the assumption of flexibility of both legs when both $\lambda\ell_1$ and $\lambda\ell_2$ exceed $3\pi/4$, M for SPLT subtypes may accordingly be calculated from (17).

For Type 2 SP subtypes:

$$L' = \frac{\ell_2}{2} \qquad (26)$$

Values for Δ are again calculated from (15). For ST subtypes, S may be ignored and Δ calculated from (15a). For LT subtypes, S becomes:

$$S = \frac{k\Delta \sin\theta}{\lambda} \qquad (20a)$$

Then, substituting (20a) into (15) gives an expression for Δ:

$$\Delta = \frac{cL' - \dfrac{fL'^2}{2AE}}{1 + \dfrac{kL'\sin\theta}{\lambda AE}} \qquad (27)$$

and then S and M can be calculated from (20) and (21).

For Type 3 and Type 5 elements of subtype SP:

$$L' = \ell_2 \qquad (28)$$

For ST subtypes, Δ can be calculated from (15a), while for LT subtypes, Δ is found by (27). Then, for Types 3 and 5 SPST and SPLT subtypes, S and M can be found from (20) and (21).

Elbow Flexibility

Approximate methods of analyzing buried elbows described above conservatively treat the legs entering and leaving the elbow as relatively flexible but treat the elbow itself as inflexible, that is, there is no relative rotation due to bending moment between the entrance and exit tangent points. However, some relative rotation does take place, consideration of which results in lower calculated bending moments for a given set of conditions (Refer to Figure 9). Consideration of the effects of elbow flexibility may be accomplished by deriving a system of equations reflecting the interdependence of forces, moments, soil deformation, and rotations of the pipe in the immediate vicinity of the elbow.

By applying Timoshenko's treatment of bending stress in thin wall curved tubes (16), such a system of equations (15), (29) and (31) can be written, as follows (5, 6):

$$\Delta = cL' - \frac{SL'}{AE} - \frac{fL'^2}{2AE} \qquad (15)$$

$$M = \frac{\lambda\Delta\sin\theta}{r\theta/KEI} \qquad (29)$$

where r = elbow or bend radius

θ = angle of bend

$$K = 1 - \frac{9}{10 + 12\left(\frac{tr}{a^2}\right)^2} \qquad (30)$$

t = pipe wall thickness

a = pipe outside radius

$$S = \frac{k_s\Delta\sin\theta}{2\lambda} + \lambda M \qquad (31)$$

In order to solve this system of equations, it is convenient to first rename the groups of independent variables as follows:

$$a_{11} = cL' - \frac{fL'^2}{2AE}$$

$$a_{12} = -\frac{L'}{AE}$$

$$a_{21} = \frac{\lambda}{r\theta/KEI} \quad \text{where K is found from (30)}$$

$$a_{31} = \frac{k}{2\lambda}$$

$$a_{32} = \lambda$$

Then, Equations (15), (29) and (31) can be expressed by:

$$\Delta = a_{11} + a_{12} S$$

$$M = a_{21} \Delta \sin\theta$$

$$S = a_{31} \Delta \sin\theta + a_{32} M$$

Substituting (29) into (31) and then into (15) enables the variables Δ and S to be calculated directly as follows:

$$\Delta = \frac{a_{11}}{1 - a_{12}a_{31}\sin\theta - a_{12}a_{21}a_{32}\sin\theta} \qquad (32)$$

$$S = \Delta\sin\theta(a_{31} + a_{21}a_{32}) \qquad (33)$$

Equation (29) can then be used to calculate the flexible elbow bending moment M resulting from strain M.

The maximum bending stress (corresponding to the intensified elbow stress of the ASME/ANSI Codes) can then be found from the VonKarman equation (Ref. 16, p. 408) as follows:

$$\sigma_b = k_1 \frac{Md}{2I} = k_1 \frac{M}{Z} \qquad (34)$$

where $k_1 = \dfrac{2}{3K\sqrt{3B}}$

where K is found from (30) and $B = \dfrac{6}{5 + 6\left(\frac{tr}{az}\right)^2}$

Consideration of elbow flexibility gives much lower values for calculated bending moments and stresses. However, caution must be used in using this analysis procedure due to the likely presence of some restraining effect by the backfill to the slight flattening or ovalization which is characteristic of an elbow or bend under high bending moment. It is suggested that the analyst intuitively assign a multiplier of 2 to elbow bending stresses calculated with consideration of flexibility to account for passive restraint of compacted backfill to the flattening at the bend midpoint.

The elbow flexibility aspect of buried piping subjected to thermal expansion has been treated by Tung and Yeh in Reference 17. Their findings appear to support the validity of considering elbow flexibility (in which they refer appropriately to the elbows as 'torsion springs') suggesting though that the torsion spring model underestimates S and overestimates effective friction length, which are better obtained from the rigid elbow model. On the other hand, the higher value of S obtained from the rigid elbow model would result in a lower calculated value for Δ, upon which the value of the bending moment depends.

Classification of Some Additional Elements

In addition to bends and tees, it is useful to identify some types of straight pipes and pipes with offsets which need to be analyzed and which may be treated as follows:

1. Type 6 - Straight pipe axially free at both ends. For short pipes, where $\ell_2 < 2\ell_m$:

$$L' = \ell_2/2 \qquad (35)$$

For long pipes, where $\ell_2 \geq 2\ell_m$:

$$L' = \ell_m \qquad (36)$$

2. Type 7 - Straight pipe axially free at one end and anchored at the other. For short pipes, where $\ell_2 \leq \ell_m$:

$$L' = \ell_2 \qquad (37)$$

For long pipes, where $\ell_2 \geq \ell_m$:

$$L' = \ell_m \qquad (36)$$

For both Type 6 and Type 7 pipes, the relative movement between pipe and soil at the free end is calculated by:

$$\Delta = \epsilon L' - \frac{fL'^2}{2AE} \qquad (38)$$

3. Type 8 - Pipes with offsets. Pipes with offsets may be analyzed conservatively as pipes with elbows, Types 1, 2 or 3. A more rigorous treatment of offsets considering balanced friction effects on each side of the offset may be appropriate, but is not covered here.

Axial Load Diagrams

Tables 1 and 2 present loading diagrams for various types of bends and other elements for which analysis may be required. For all types of buried piping and bends subjected to soil/pipe movements, there is either a point or region of zero relative movement at the soil/pipe interface on the P leg. The diagrams shown in Tables 1 and 2 show where this point or region is located for each type. The maximum axial force in the pipe will occur at this point and can be expressed by:

$$F_{max} = S + fL' \qquad (39)$$

For a long pipe with a free end, where friction fully develops to produce a region of zero slippage, this force becomes:

$$F_{max} = \epsilon AE \qquad (40)$$

The analyst must keep in mind that soil bearing force, S, and friction force, fL' or $f\ell_m$, are passive in nature and cannot exceed certain practical limits. Neither can occur until an actual relative movement occurs between pipe and soil. The bearing force cannot occur until some movement takes place to deform the soil normal to the T leg. Correspondingly, friction force acts only when there is slippage; it is zero at free ends, and at points or regions of zero slippage in the P leg.

DESIGN CONSIDERATIONS

Whenever analysis shows a buried pipe to be stressed beyond allowable limits, it is necessary to provide additional flexibility in the system or to modify the restraining effects of the backfill. Several considerations are discussed in References 9 and 14, and one of these, the use of flexible couplings, has been used in several recently designed

2. Soils and seismic data:

Modulus of subgrade reaction $k = 13{,}500$ psi (93,150kPa)
Friction at soil/pipe interface $f = 200$ lb/in. (350 N/cm)
Seismic soil strain – Eq. (1) and (2):

For a 0.24g earthquake

$V_m = (48\text{in/sec.})(.24) = 11.52$ in/sec (29.26cm/sec)

Assuming wave velocity C of 2,000 ft/sec(600m/sec)

$\epsilon_m = \pm V_m/c = \pm(11.52\text{in/sec}/2{,}000\text{ ft/sec})(12\text{in/ft})$

$= \pm 4.8 \times 10^{-4}$ in./in. (cm/cm)

System characteristic λ – Eq. (10):

$\lambda = 0.0158 \text{ in.}^{-1}$ $(6.22 \times 10^{-3}\text{cm}^{-1})$

Minimum slippage length L_m – Eq. (11):

$L_m = \epsilon AE/f$
$= (4.8 \times 10^{-4})(27.83)(27.9 \times 10^{-6})/200$
$= 1{,}864$ inches (47.33m)

3. Classification of the pipe element:

The element is shown to be a Type 1 bend, that is, a pipe free at one end with an elbow at the other.

Transverse 'T' leg:

$L_1 = 30 \text{ ft} \times 12 \text{ in./ft}$
$= 360$ inches (914cm)
$3\pi/4\lambda = 149$ inches (378cm)

Since $L_1 > 3\pi/4\lambda$, the transverse leg is long ('LT' subtype).

Longitudinal 'P' leg: Assume initially that it is 'long' and calculate L' from (14).

$\Omega = AE\lambda/k = 909$ inches (2,308cm)
$F_{max} = fL_m = 372{,}800\#$ (1,658kN)

Then $L' = 909(1.2586) = 1{,}144$ inches (2,906cm)
Therefore, since $L_2 = 2{,}400$ inches (6,100cm) and $L_m + L' = 1{,}864 + 1{,}144 = 3{,}008$ inches (7,640cm), $L_2 < L_m + L'$ and the 'P' leg is short (SP).

Thus, the elbow is now finally classified as a Type 1-SPLT bend.

4. Calculation of L', S and M:

a. Elbow assumed inflexible: From Table 3, it is seen that for Type 1-SPLT, values for L', S and M are found by (23), (19) and (17) respectively. Accordingly:

L' = 852 inches (2,165cm)
S = 139,200lbs. (619kN)
M = 2,936,700in.lbs. (331.8kNM)

b. Elbow assumed flexible: Values for S and M are calculated from (33) and (29) as follows:

$\Delta = 0.211$ inches (0.536cm)
S = 95,668lbs. (425.5kN)
M = 349,670in-lbs. (39.5kNM)

5. Calculation of code stresses in the elbow:

a. Elbow assumed inflexible:

(1.) Seismic stress assumed primary –

$\sigma_p = 1{,}040$psi (7,170kPa)
$\sigma_b = 0.75i\ M/Z = 57{,}137$psi (393,946kPa)
$\sigma_a = S/A = 5{,}002$psi (34,488kPa)
$\sigma_e = \sigma_p + \sigma_b + \sigma_a = 63{,}179$psi (435,604kPa)

(2.) Seismic stress assumed secondary –

$\sigma_b = iM/Z = 76{,}184$psi (525,270kPa)
$\sigma_a = 5{,}002$psi (34,488kPa)

Total stress range –

$\sigma = 2(\sigma_b + \sigma_a) = 162{,}372$psi (1,119,952kPa)

In either case, the stresses are far above allowable levels, and the pipe would have to be modified to provide axial flexibility in the P leg near the elbow.

b. Elbow assumed flexible:

(1.) Seismic stress assumed primary –

$\sigma_p = 1{,}040$psi (7,170kPa)
$\sigma_b = F_s(k_1 M/Z)$

where the safety factor F_s (to account for passive soil resistance to ovalization) is taken as 2 and $k_1 = 3.229$ from (30) and (34),

then –

$\sigma_b = 13{,}946$psi (96,154kPa)
$\sigma_a = 5{,}002$psi (34,488kPa)
$\sigma_e = 19{,}988$psi (137,812kPa)

(2.) Seismic stress assumed secondary –

$\sigma_b = 13{,}946$psi (96,154kPa)
$\sigma_a = 5{,}002$psi (34,488kPa)

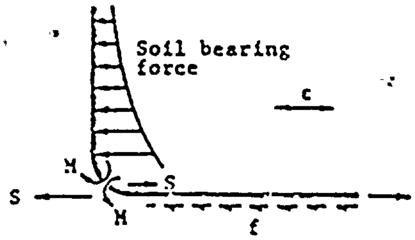
Total stress range –

$\sigma = 2(\sigma_b + \sigma_a) = 37{,}896$psi (261,284kPa)

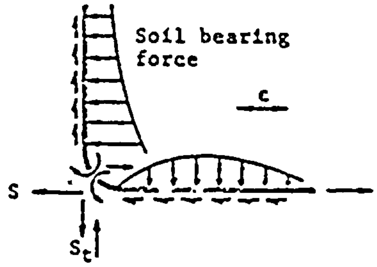
It is the responsibility of the analyst to make the necessary assumptions regarding elbow flexibility and primary vs. secondary treatment of stresses, and then to apply the appropriate allowable stresses to determine if code conditions are met.

ACKNOWLEDGMENTS

The kind assistance of several individuals is gratefully acknowledged for their having pointed out various typographical and transcription errors in previous works, and assisted in developing and clarifying certain concepts involving the interaction of buried piping and the surrounding soils. Particularly helpful were Dr. M. A. Iqbal, Dr. M. C. Lee, Dr. R. A. Meyer, R. J. Hunt, T. Chadda and Dr. A. Sallahuddin. Also, the persistance and patience of Sandra Sustello, who typed this manuscript is gratefully acknowledged.

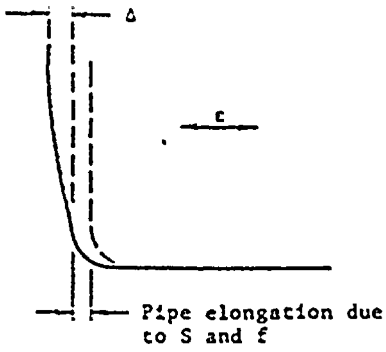


a. Transverse leg flexible, pipe leg inflexible.

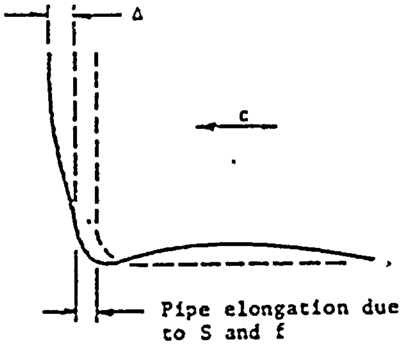


b. Both legs flexible.

FIG. 7 LOAD DIAGRAMS



a. Transverse leg flexible, pipe leg inflexible.



b. Both legs inflexible.

FIG. 8 DEFORMATION DIAGRAMS

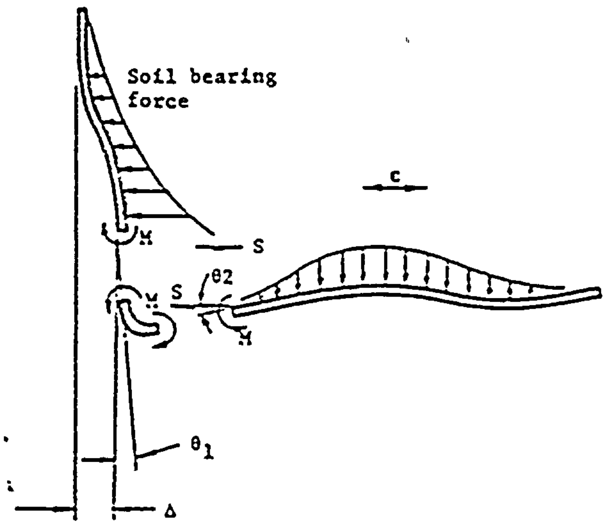


FIG. 9 FLEXIBLE ELBOW LOAD AND DEFORMATION

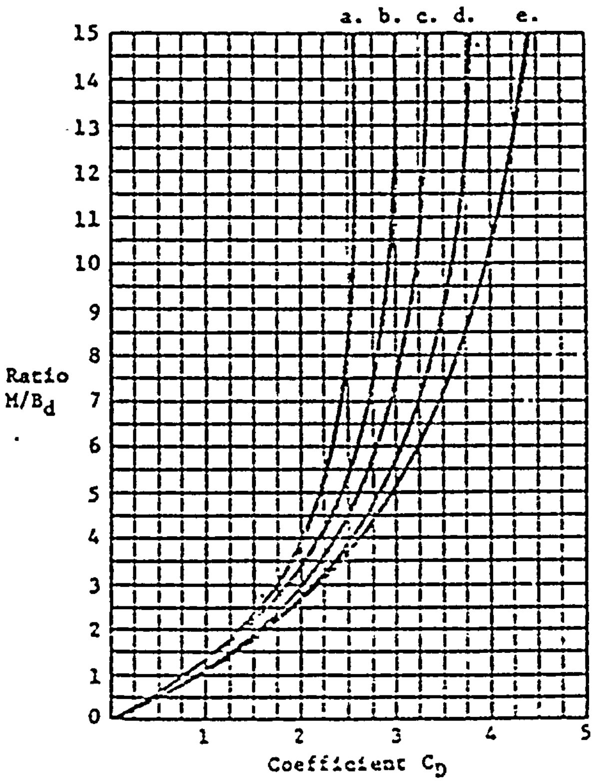


FIG. 10 CURVES OF C_D FOR VARIOUS BACKFILL MATERIALS (18)

a. Granular materials without cohesion
b. Sand and gravel
c. Saturated top soil
d. Ordinary clay
e. Saturated clay

TABLE 1 AXIAL LOADING DIAGRAMS FOR PIPING ELEMENTS
(POINTS AND REGIONS LABELED A EXPERIENCE ZERO SLIPPAGE BETWEEN SOIL AND PIPE)

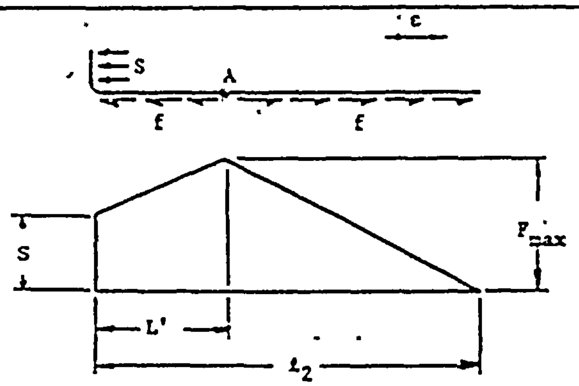
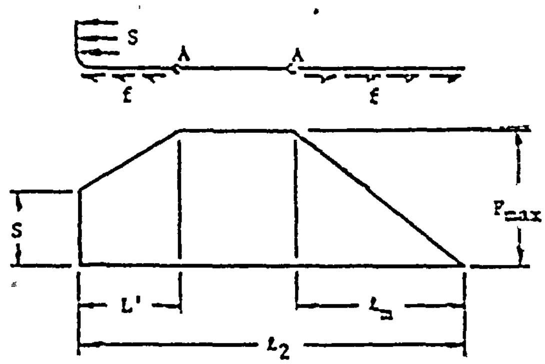
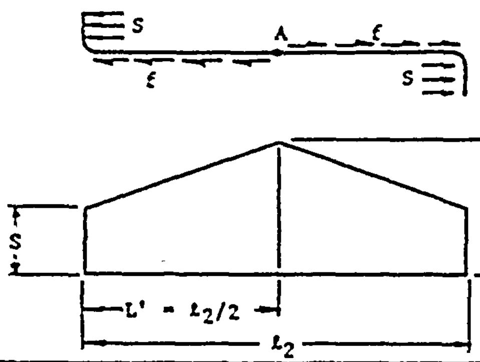
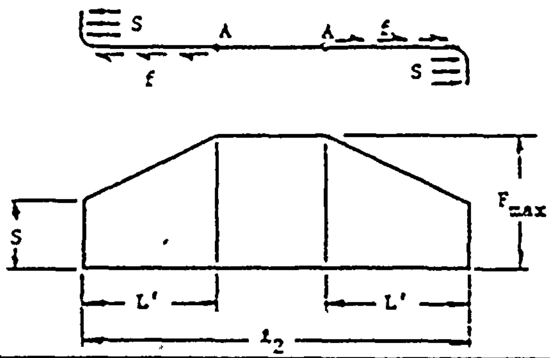
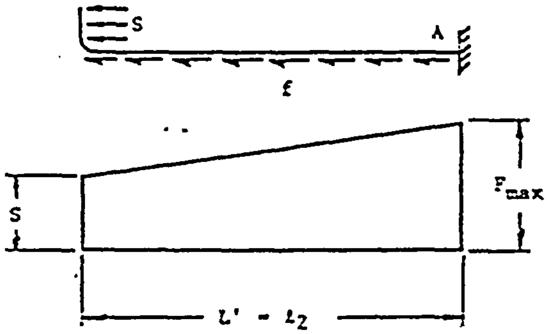
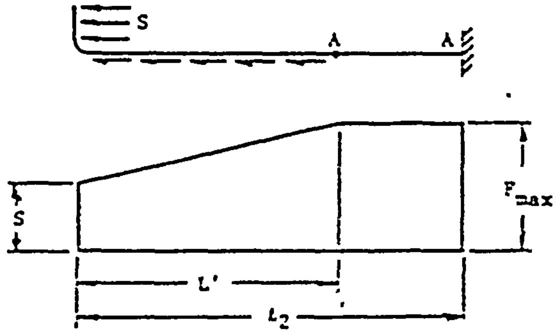
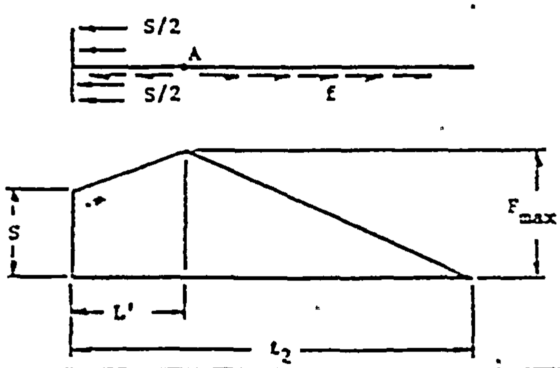
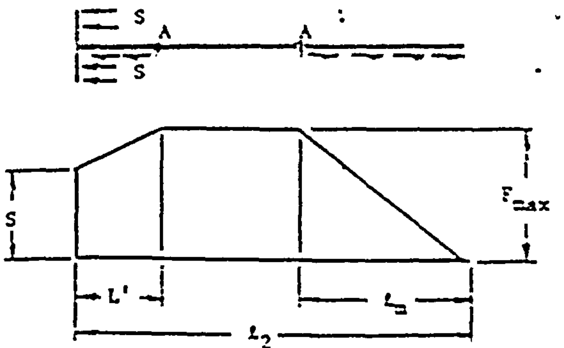
TYPE	SHORT PIPE	LONG PIPE
1		
2		
3		
4		

TABLE 2 AXIAL LOADING DIAGRAMS FOR PIPING ELEMENTS
(POINTS AND REGIONS LABELED A EXPERIENCE ZERO SLIPPAGE BETWEEN SOIL AND PIPE)

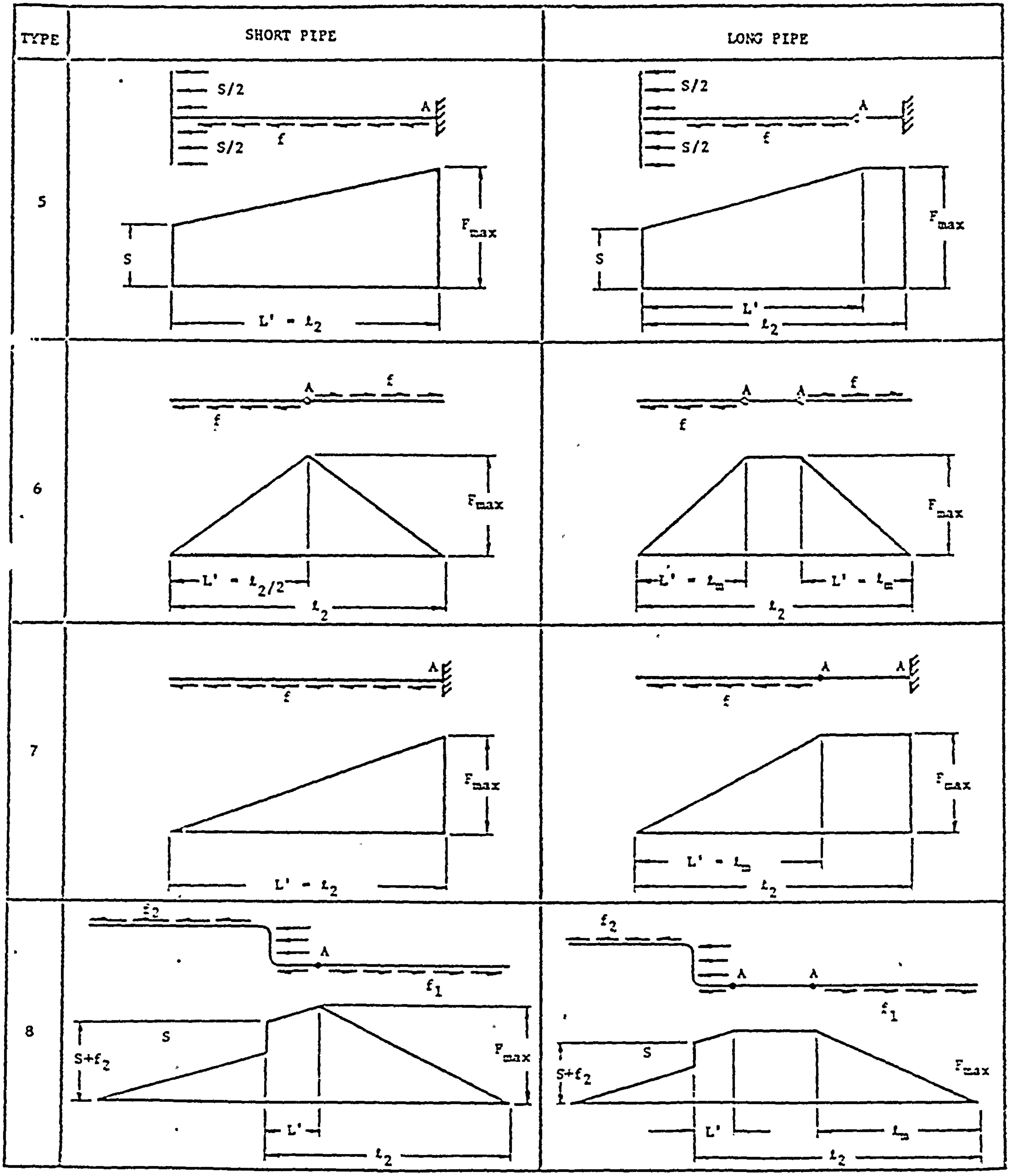


TABLE 3

MATRIX OF APPLICABLE EQUATIONS BY ELEMENT TYPE AND SUBTYPE

VARIABLE	ELEMENT TYPE AND SUBTYPE										
	1-SPST 1-SPLT 4-SPST 4-SPLT	1-LPST 2-LPST 3-LPST 4-LPST 5-LPST	1-LPLT 2-LPLT 3-LPLT	2-SPLT	2-SPST	3-SPST 5-SPST	3-SPLT 5-SPLT	4-LPLT 5-LPLT	6-SP	6-LP 7-LP	7-SP
c_m	(2)										
c_t	(4)										
k	(7) or $0.6E_s$										
f	(9)										
λ	(10)										
L_m	(11)										
L'	(23)	(24)	(14)	(26)	(26)	(28)	(28)	(14)	(35)	(36)	(37)
S	(19)	(20)	(16)	(20a)	(20)	(20)	(20)	(16)			
Δ	(22)	(25)	(18)	(27)	(27)	(15a)	(27)	(22)	(38)	(38)	(38)
M	LT:(17) ST:(21)	(21)	(17)	(17)	(21)	(21)	(21)	(21)			
F_{max}	(39)	(40)	(40)	(39)	(39)	(39)	(39)	(40)	$F_{max} = fL'$		

NOTE: Table 3 is based on the assumption of inflexible elbows. For the flexible elbow assumption, Δ, S and M for Types 1, 2 and 3 may be found from Equations (32), (33) and (29) respectively.

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REF 4

WASHINGTON PUBLIC POWER
SUPPLY SYSTEM

RECORD OF TELEPHONE CONVERSATION

58400-CDE-0067-88

DATE	TIME	TO BE CONFIRMED
11-03-88	0650	☐ YES ☒ NO (XX)

FROM: Dale Eggen, Prin Fire Pctn Engr. (NAME) — Washington Public Power Supply System (COMPANY OR DEPARTMENT)

TO: Robert Soloman, Staff Liaison, (NAME) — NFPA 24 (COMPANY OR DEPARTMENT)

SUBJECT(S) DISCUSSED

NFPA 24: The use of calculation to determine main movement rather than the use of thrust blocks to restrain the movement.

REMARKS:

NFPA-24-1987 states in paragraph 8-6.2.8 "Thrust blocks or other suitable means of thrust restraint shall be provided at each change in the direction of a pipeline and at all tees, plugs, caps, and bends. The thrust blockes shall be of concrete of a mix not leaner than one part cement, two and one-half parts sand, and five parts stone. Backing shall be placed between undisturbed earth and the fitting to be anchored and shall be of such bearing area as to assure adequate resistance to the thrust to be encountered. In general, backing shall be so placed that the joints will be accessible for inspection and repair. Thrust blocks are not suitable for vertical pipe."

Mr. Soloman stated that other "suitable means" includes the use of calculations to show that main movement will not cause a problem at the pipe joints and/or changes in direction. He stated that this method is not generally used as most plants do not have the engineering staff to do the needed study and calculation. Thus, they use the "cook book" method of thrust block as noted in the code. Additionally, Mr. Soloman stated that the calculations should be submitted to the "Authority having jurisdiction" for their approval.

NFPA 1973 has the same words about using "other suitable means".

cc: JC Bell 982B
AW Clarkson 982B
HL Aeschliman 956B
JV Hanson 988U
LD Noble 981A
DJ Wilsey 981B
CDE/lb/File

WP-106 R2

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AMERICAN NUCLEAR INSURERS

BURT C. PROOM, CPCU
President and Chief Executive Officer

ENGINEERING DEPARTMENT
Ronald Sanocore, Vice President

December 16, 1988
(21902088)

Mr. Howard J. Fowler, MD #927S
WNP-2 Fire Marshal
Washington Public Power Supply System
P. O. Box 968
Richland, WA 99352

Dear Skip:

WASHINGTON PUBLIC POWER SUPPLY SYSTEM
NUCLEAR PROJECT NO. 2
RICHLAND, WASHINGTON
ANI PROPERTY FILE NO. N-219

FIRE MAIN 12"-FP-43-1
PIPE DISPLACEMENT CALCULATIONS

This will confirm American Nuclear Insurers (ANI) review of your calculations CE-02-88-36, Revision 1, and my telephone conversations with you and Mr. Larry Nobel. Our telephone conversation yesterday morning centered on needed revisions to the calculations. It is understood that Mr. Nobel will send me a new set of calculations as soon as the revisions have been made.

The revised calculations will show a maximum pipe displacement of approximately 0.07 inches when subjected to a working pressure of 150 psi and a surge pressure of 100 psi. This supports your conclusion that pipe separation will not occur and installation of a thrust block for this existing 12 inch main is not necessary.

Under normal conditions, NFPA 24 requires the installation of some type of approved pipe restraint, such as a thrust block, to prevent pipe movement and separation. Your calculations show that the action of soil friction and soil spring reaction against the pipe will provide an equal measure of restraint. It is also noted that the NFPA Fire Protection Handbook, 14th. Edition, states in part, "Joints are expected to be kept in place by the soil in which the pipe is buried".

Based on the calculations reviewed, the receipt of revised calculations as per our discussion, the ability to supply the Diesel Generator Building sprinkler systems through alternate flow paths, and the current terms and

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TKS

conditions of ANI's Property Damage Policy, ANI will not require the installation of a thrust block or other form of pipe restraint for fire main 12"-FP-43-1.

It should be understood this acceptance is case specific and will not be automatically extended to any future concerns dealing with site underground fire mains. Future concerns, if any, will be resolved after individual evaluation.

If you have any questions, please contact me.

Yours truly,

Al Baker

Al Baker
Senior Regional Field Engineer

cc: C. M. Powers - WNP-2, Richland, WA
C. D. Eggen - WNP-2, Richland, WA
A. C. Marzette - WPPSS, Richland, WA
H. D. Pickerl - M&MNC, Chicago, IL

REF 8

AMERICAN NUCLEAR INSURERS

BURT C. PROOM, CPCU
President and Chief Executive Officer

ENGINEERING DEPARTMENT
Ronald Sonocore, Vice President

March 27, 1989
(21900689)

Mr. Howard J. Fowler, MD #927S
WNP-2 Fire Marshal
Washington Public Power Supply System
P. O. Box 968
Richland, WA 99352

Dear Skip:

WASHINGTON PUBLIC POWER SUPPLY SYSTEM
NUCLEAR PROJECT NO. 2
RICHLAND, WASHINGTON
ANI PROPERTY FILE NO. N-219

FIRE MAIN 12-FP-43-1
REVISED PIPE DISPLACEMENT CALCULATIONS

I have reviewed the revised pipe displacement calculations, dated December 19, 1988 (calculation CE-02-88-36, revision 2).

The calculations are acceptable to American Nuclear Insurers (ANI) for property insurance purposes only.

As indicated in my letter 21902088, dated December 16, 1988, ANI will not require the installation of a thrust block or other form of pipe restraint for existing fire main 12-FP-43-1 at this time. This is based on the calculations reviewed, the ability to supply the Diesel Generator Building sprinkler systems through alternate flow paths, and the current terms and conditions of ANI's property damage policy.

Also, as stated previously, this acceptance is case specific and will not be automatically extended to any future concerns dealing with site underground fire mains.

Yours truly,

Al Baker

Al Baker
Senior Regional Field Engineer

cc: C. M. Powers - WNP-2, Richland, WA
C. D. Eggen - WNP-2, Richland, WA✓
D. H. Walker - WPPSS, Richland, WA
A. C. Marzette - WPPSS, Richland, WA
H. D. Pickerl - M&MNC, Chicago, IL

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REF 10

W. J. Hoskins - H. L. Aeschliman
Bill - Bob Thuyco-has.

Committee on Private Water Supply Piping Systems

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Reply To:

RECEIVED
AUG 1 5 1989
IS & FP

August 8, 1989

D. H. Walker, Manager
Industrial Safety & Fire Protection
Washington Public Power Supply System
P.O. Box 968
3000 George Washington Way
Richland, WA 99352

Dear Mr. Walker:

This replies to your letter of August 1, 1989 requesting information on NFPA 24, Standard for the Installation of Private Fire Service Mains and Their Appurtenances. The NFPA cannot approve a particular design or arrangement but I can offer you my personal opinion of the Standard as it relates to your situation.

NFPA 24, Paragraph 8-6.2.8 requires the use of thrust blocks or other suitable means to restrain the expected movement of the pipe due to hydrostatic and hydrodynamic forces. According to the Ductile Iron Pipe Research Association (DIPRA) Thrust Restraint Manual, the use of a special push on, or mechanical restrained joint is considered to be an acceptable alternative to the use of concrete thrust blocks.

When the alternative means of pipe restraint is provided, a detailed analysis of the soil conditions and expected pipe forces is necessary to verify the design. The DIPRA manual as well as any civil engineering handbook provides the necessary guidance for proper pipe restraint when using alternative approaches.

This response does not represent a Formal Interpretation as noted below.

Sincerely,

8
8
89

Robert E. Solomon, P.E.
Senior Fire Protection Engineer

RES/pmm

— NOTICE ON INTERPRETATIONS —

A statement, written or oral, that is not processed in accordance with Section 16 of the Regulations ... considered the official position of NFPA ...