American**Lifelines****Alliance**

A public-private partnership to reduce risk to utility and transportation systems from natural hazards and manmade threats

Seismic Guidelines for Water Pipelines

March 2005

U.S. Department of Homeland Security

FEMA

National Institute of BUILDING SCIENCES

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This report was written under contract to the American Lifelines Alliance, a public-private partnership between the Federal Emergency Management Agency (FEMA) and the National Institute of Building Sciences (NIBS). This report was prepared by a team representing practicing engineers in the United States water utility industry and academics.

Acknowledgements

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Tom O'Rourke	Cornell University
Alex Tang	Consultant
John Wesling	Geomatrix Consultants Inc.

Mr. Doug Honegger provided technical oversight of this project. Mr. Joseph Steller (NIBS) provided project management for this project.

G&E would also like to thank the numerous staff of the San Francisco Public Utilities Commission, East Bay Municipal Utilities District, City of San Diego Water Department, the Los Angeles Department of Water and Power, and all the other participating agencies for their generous help.

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Prepared for:
National Institute of Building Sciences

As part of the:
American Lifelines Alliance

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G&E Report 80.01.01, Revision 0
March, 2005

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5.0 Subsurface Investigations

The purpose of a geotechnical investigation is to define the surface and subsurface conditions along the pipe alignment. The level of geotechnical understanding necessary for a proper assessment is dependent upon the pipe Function. Table 5-1 presents recommended geotechnical field and laboratory investigations and testing for pipes serving different Functions. The investigations are presented for consideration and the final determination should be made by experienced geotechnical engineers and geologists based on the pipes specific needs and concerns.

The investigations in Table 5-1 are progressive in that the investigations for higher Function pipes build upon those for lower Functions. For example, an investigation for a Function IV pipe should include all investigations listed for Function II and III pipes. As seen in Table 5-1, there are no recommended geotechnical investigations for seismic design of Function I pipes because there are no seismic design requirements for these pipes; however, it is often prudent to perform a geotechnical investigation for non-seismic concerns. The minimum recommended investigation for Function II pipes includes a literature and map review with follow up site reconnaissance. Investigations for pipes of Function III include some subsurface investigations and mapping and possibly laboratory testing. Function IV pipes may include more detailed and advanced field and laboratory testing and mapping.

Function	Geotechnical Investigation
I	• None

II	Literature and map review with site reconnaissance
	• Review existing maps
	  o Geology
	  o Topographic
	  o Groundwater
	  o Liquefaction hazard
	  o Landslide hazard
	  o Fault
	• Review literature, aerial photographs, and satellite images to identify:
	  o historic landslides
	  o historic ground failures
	  o historic ground water
	  o land use changes
	  o Any past field explorations in vicinity
	• Characterize surface and near surface conditions (bedrock and soil)
	  o General bedrock conditions and strength
	  o Stream and river crossings
	  o General soil classifications and densities.
	  o Identify approximate contacts between differing geologic materials
	• Site reconnaissance
	  o Confirm surface rock and soil conditions
	  o Observation of known historic ground failures
	  o Alignment review for potential undocumented landslides.
	  o Confirm approximate geologic contacts

Table 5-1. Geotechnical Investigations (Part I)

Function	Geotechnical Investigation
III	Field and laboratory investigations and testing in addition to geotechnical investigations recommended for Function II pipes. • Perform drilling and/or CPT a minimum of 500 ft to 1000 ft apart, closer spacing if soil/rock conditions change. Use judgment based on knowledge level of subsurface conditions. • Perform at least 1 boring for every 5-CPT and in each geologic unit. • Focus more detailed investigations in potentially liquefiable areas. Perform SPT and CPT in potentially liquefiable soils, evaluate full depth for potential liquefaction regardless of pipe depth. • Obtain soil samples at 5 to 10 ft intervals; consider alternating SPT and sampling alternating 2.5 ft to 5 ft intervals. • Identify soil type, bedrock depth, groundwater depth. Perform visual soil classification in field. • Possible laboratory index tests include grain size, Atterburg limits, classification, density, and moisture content. • Perform detailed site mapping for fault crossings and landslides. Locate faults and landslides as accurate as possible using geologic mapping methods.
IV	Advanced field investigations and laboratory testing in addition to geotechnical investigations recommended for Function III pipes. • Perform drilling and CPT a minimum of 250 ft to 500 ft apart, closer spacing if soil/rock conditions change. • Perform at least 1 boring for every 3-CPT and in each geologic unit. • Perform detailed investigations in potentially liquefiable areas. Perform SPT and CPT in potentially liquefiable soils, evaluate full depth for potential liquefaction regardless of pipe depth. • Obtain soil samples at 5 alternating SPT and sampling at 2.5 ft intervals. • Identify soil type, bedrock depth, groundwater depth. Consider identifying average shear wave velocity over 30 to 100 m depth. • Perform laboratory soil index tests and soil strength (direct or triaxial shear) tests as considered necessary. • Perform vane shear in weak clay deposits. • Perform geophysical testing methods to identify subsurface layers, depth of bedrock, and material properties. • Identify fracturing and weathering in bedrock. • Perform detailed mapping of liquefiable soil deposits, distinguish high moderate, and low liquefaction potential. • Perform dynamic laboratory testing (torsional shear, triaxial, simple shear) as determined appropriate. • Perform trenching for landslide investigations. Accurately locate slide planes and shear zones, strength of slip plane, etc. • Perform detailed topographic mapping as necessary. • Perform trenching for fault investigations. Accurately locate fault traces, fault zones, historic fault movements, time of last movement, etc.

Table 5-1. Geotechnical Investigations (Part II)

Determining field investigation and laboratory testing requirements are related to the potential seismic hazard and the selection of investigations to perform should be made by geotechnical engineers and geologists experienced in earthquake matters. In some cases the more advanced investigations and testing may be cost effective considering that a more accurate hazard definition provides a better understanding of the design parameters.

For some water pipelines, especially large diameter, there may be a significant difference in design costs for small changes in seismic hazard. In other cases it may be better to design for conservative hazard estimates without spending the time and money on more advanced geotechnical investigations.

Procedures for determining the liquefaction susceptibility of soils have been summarized by Youd, et al. (2001). This publication is a consensus document representing the most appropriate methods for evaluating liquefaction potential from SPT, CPT, and other in situ measurements. The process for correcting for SPT and CPT readings and relating them to liquefaction potential presented in this publication should be followed when assessing liquefaction risk for the design of water pipeline installations.

6.0 General Pipeline Design Approach

Chapter 6 provides a review of normal load conditions that should be considered in conjunction with design for earthquake loading. Chapter 6 is not meant to be comprehensive, but to highlight some of the usual considerations.

6.1 Internal Pressure

The internal pressure used in the design of a water pipe system should be the larger of:

- The maximum operating pressure. This should consider hydrostatic (no flow) pressure, operating pressure, failure of control devices, operator error and anticipated over-pressure transients such as water hammer.

- Any in service pressure leak test.

Design allowable stresses for internal pressure are given in applicable AWWA documents.

6.2 Vertical Earth Load

Under most operating conditions with soil cover of 3 to 4 feet, the vertical earthquake load for buried water pipes can be neglected since it is insignificant compared to the internal pipe pressure. Vertical earth load is an important consideration for the design of pipe casings used for rail and road crossings due to the heavy loads involved.

Welded steel water pipe is considered flexible. For flexible pipes placed in a trench and covered with backfill, the earth dead load applied to the pipe is the weight of a prism of soil with a width equal to the pipe and a height equal to the depth of fill over the pipe.

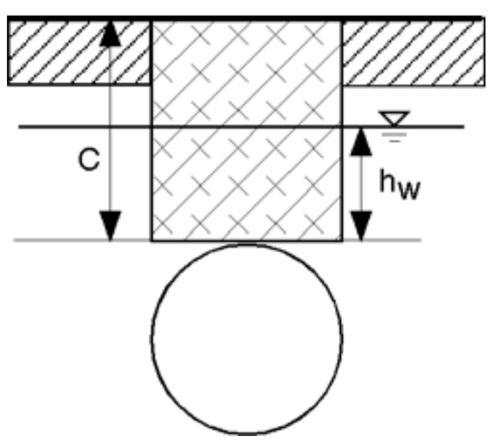

Figure 6-1. Soil Prism Above Flexible Pipe

For the case when the water table is below the top of the pipe, the upper bound estimate of load acting on the pipe from earth dead load is:

$$P_v = \gamma_d C \quad \text{[Eq 6-1]}$$

For conditions with the pipe located below the water table, the effect of soil grain buoyancy can be approximated by:

$$P_v = \gamma_w h_w + R_w \gamma_d C$$

γ_d = dry unit weight of backfill [Eq 6-2]

R_w = water bouyancy factor = $1 - 0.33\left(h_w / C\right)$

Alternately, the vertical dead load can be conservatively estimated from [Eq 6-1] assuming saturated soil conditions.

The effect of soil dead weight and resulting pipe ovality should be considered when establishing ring buckling (wrinkling) capacity of the pipe.

6.3 Surface Live Load

Buried pipes can be exposed to superimposed concentrated or distributed live loads, including truck-wheel loads, railway car, locomotive, aircraft loads, etc.

Depending on the requirements of the design specification, the live-load effect may be based on AASHTO HS-20 truck loads, Cooper E-80 railroad loads or a 180 kip airplane gear assembly load, as listed in Table 6-1. These values include an impact factor of 1.5 to account for bumps and irregularities in the travel surface. H20 loads become negligible for cover over 8 feet. E-80 loads become negligible for cover of 30 feet. Airport loads become negligible for cover of 24 feet.

Height of Cover	Live Load transferred to the pipe, psi		
Feet	Highway H20, Note 1	Railway E80 Note 2	Airport Note 3
1	12.50		
2	5.56	26.39	13.14
3	4.17	23.61	12.28
4	2.78	18.40	11.27
5	1.74	16.67	10.09
6	1.39	15.63	8.79
7	1.22	12.15	7.85
8	0.69	11.11	6.93
10	-	7.64	6.09
12	-	5.56	4.76

Table 6-1. Live Loads

Note 1. Simulates 20 -ton truck traffic, with impact
Note 2. Simulates 80,000 lb/ft railway load, with impact
Note 3. 180,000 pound dual-tandem gear assembly, 26 inch spacing between tires and 66 inch center-to-center spacing between fore and aft tires under a rigid pavement 12 inches thick, with impact

For live loads other than those in Table 6-1, the pressure P_p applied to the buried pipe by a concentrated surface load P_s, without impact, is:

$$P_p = \frac{P_s}{2\pi C^2 \left(1 + \left(\frac{d}{C}\right)^2\right)^{2.5}} \qquad \text{[Eq 6-3]}$$

where P_p is the pressure transmitted to the pipe, P_s is the concentrated load at the surface, C is the height of cover, d is the offset distance from the pipe centerline to the line of application of the surface load.

The pressure P_p should be multiplied by the impact factors in Table 6-2.

Height of Cover	Installation Surface Condition		
Feet	Highway H20, Note 1	Railway E80 Note 2	Runways Note 3
0 to 1	1.50	1.75	1.00
1 to 2	1.35	1.50	1.00
2 to 3	1.15	1.50	1.00
Over 3	1.00	1.35	1.00

Table 6-2. Impact Factors

For checking the pipeline, both empty condition and pressurized condition should be checked. In the pressurized condition, the external down pressure can be reduced by the internal pipe pressure.

When a surcharge load is distributed over the ground surface area near a pipeline, the possibility exists that the external surcharge may cause lateral or vertical displacement of the soil surrounding the pipeline. In this case, additional information, including a suitable geotechnical investigation, may be needed to determine if the pipeline could be subjected to soil displacement. A detailed investigation is in order if the distributed surcharge load over an area larger than 10 square feet exceeds the values below (weight of material placed of height of soil fill added over the pipeline):

- 1,500 psf or 15 feet of fill - nominal pipe diameter 12 inches or less

- 1,000 psf or 10 feet of fill - nominal pipe diameter over 12 inches
- 500 psf or 5 feet of fill - pre-1941 pipes

6.4 Pipe Ovalization

A flexible buried pipe will tend to ovalize under the effect of earth dead and live load (Eq 6-4, modified Iowa deflection). Pipe ovalization due to internal pipe loads (caused by fault offset or PGDs) is not covered in Section 6.4.

$$\frac{\Delta y}{D} = \frac{D_l K P}{\left(\frac{(EI)_{eq}}{R^3}\right) + 0.061E'} \qquad \text{[Eq 6-4]}$$

where D = pipe diameter (inches), Δy is the vertical pipe deflection (inches), D_l is the deflection lag factor (~1.0 to 1.5), K is the bedding constant (~0.1), P is the pressure on the pipe due to soil load Pv and live load Pp, psi, R is pipe radius (inches), $(EI)_{eq}$ is the pipe wall stiffness per inch of length, in-lb; E' is the modulus of soil reaction, psi.

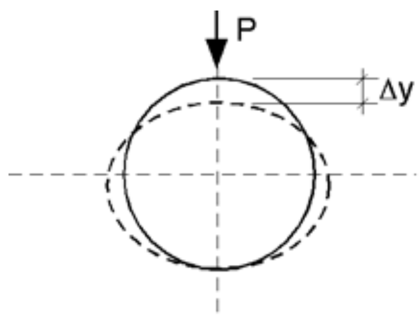

Figure 6-2. Ovality of Pipe Cross Section, Δy per Eq 6-4

The pipe wall stiffness is the sum of the pipe wall, lining and coating:

$$(EI)_{eq} = EI + E_{lining} I_{lining} + E_{coating} I_{coating}$$

and $I = \frac{t^3}{12}$

where t = pipe wall thickness, lining thickness or coating thickness.

The modulus of soil reaction *E'* is a measure of stiffness of the embedment material, which surrounds the pipe. *E' is* actually a hybrid modulus being the product of the modulus of passive resistance of the soil and the radius of the pipe. Values of *E'* vary from close to zero for dumped loose fine grained soil to 3000 psi for highly compacted coarse grained soil. Recent studies show that the confined compression modulus can be used in place of *E'*.

The imposed loads lead to ring deflection and pipe wall bending stress both with capacity limits. Typical allowable deflections, or cross section ovality, to prevent damage to various lining and coating systems are:

- Mortar lined and coated. 0.02D
- Mortar-lined and flexible coated. 0.03D
- Flexible lined and coated. 0.05D

Through-wall bending in the pipe due to ovality can be estimated as follows:

$$\sigma_{bw} = 4E\left(\frac{\Delta y}{D}\right)\left(\frac{t}{D}\right) \quad \text{[Eq 6-5]}$$

where σ_{bw} is the through-wall bending stress, $\Delta y / D$ per equation 6-4, t is wall thickness and D is pipe diameter.

The depth of burial of the pipe coupled with selection of the pipe wall t should be such that the pressure P on the pipe due to earth and surface load is less than the load needed to crush the pipe side wall. For buried pressurized water pipes with D/t ratios of 100 or less, and a yield stress larger than 30,000 psi, crushing of the sidewall is quite unlikely for normal installations.

If the soil and surface loads are too high, the pipe cross section could buckle (Figure 6-3).

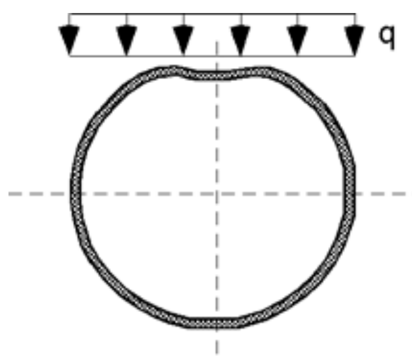

Figure 6-3. Ring Buckling of Pipe Cross Section

Equation 6-6 can be used to determine the external buckling load for typical flexible pipe installations:

$$q_a < \frac{1}{FS}\sqrt{32 R_w B' E' \frac{EI}{D^3}} \quad \text{[Eq 6-6]}$$

where q_a is the allowable buckling pressure (psi); FS is the factor of safety (2.5 for (C/D≥2; 3.0 for C/D<2); C is depth of coil cover above pipe (inch); D is diameter of pipe (inch); R_w is the water buoyancy factor (Eq 6-2), B' is an empirical coefficient of elastic support (dimensionless) from Eq 6-7 (AWWA M11). For steel pipes, buckling typically takes place when the ovality reaches about 20%.

$$B' = \frac{1}{1+4e^{-0.065 C/D}}$$

6.5 Fatigue

Under normal operating loads with pipes buried with at least 3 feet of cover, fatigue is not usually considered a problem. Where pipes cross under highways or railroads, and could be subject to repeated high loading, a deeper minimum burial depth will usually be provided (4 feet under highways, 6 feet under railroads).

6.6 Fluid Transients

Rapid changes in flow rates in water will cause pressure transients, which in turn generate pressure pulses and transient forces in the piping system. Only the simplest cases can be calculated by hand, for example, the rapid closure of a valve in a pipe. A valve closure is considered rapid if its closing time is

$$t_c \le \frac{2L_v}{c_L} \quad \text{[Eq 6-7]}$$

where t_c is the closing time, sec; L_v is the length from the valve to an open water source such as a tank, feet; c_L is the wave velocity, feet/sec.

$$c_L = \frac{12}{\sqrt{\frac{W}{g}\left(\frac{1}{k}+\frac{d}{Et}\right)}} \quad \text{[Eq 6-8]}$$

where k is the bulk modulus of compressibility of water (psi), W is fluid weight (lb/ft^3), g is the acceleration of gravity (32.2 ft/sec^2), d is the inside diameter of the pipe (inch); E is pipe modulus of elasticity for the pipe wall (psi), t is the pipe wall thickness (inch). For steel pipe, this reduces to:

$$c_L = \frac{4660}{\sqrt{1+\frac{1}{100}\left(\frac{d}{t}\right)}} \quad \text{[Eq 6-9]}$$

for k = 300,000 psi and E = 30,000,000 psi. For cast iron pipe, E ranges from 10,000,000 to 15,000,000 psi. For asbestos cement, E is about 3,400,000 psi.

The pressure rise is

$$\Delta P = \frac{c_L W(\Delta v)}{144g} \qquad \text{[Eq 6-10]}$$

where ΔP is the rise in water pressure due to rapid valve closure, psi; W is water weight, lb/ft^3; Δv is the change in liquid velocity from initial flow rate to zero (valve closure case), feet/sec; g is the acceleration of gravity, 32.2 feet/sec^2.

The pressure rise will first occur at the closed valve, propagate and reflect from the pressure source (tank). For more complex situations, a fluid dynamics analysis may be required.

The thrust loads due to fluid transients can cause large displacements in above ground pipes. Water hammer loads have often been seen to break supports off pipes (more common); or could rupture the pipe pressure boundary (less common). Thrust forces can readily open up buried segmented pipe joints if the pipe is not adequately restrained through external skin friction with the soil, by concrete anchor blocks, or by restrained couplings across joints. The unbalanced thrust force at a bend can be estimated from:

$$F = (DLF)\Delta PA \qquad \text{[Eq 6-11]}$$

where A is the pipe cross sectional area, and the DLF=2 reflects an assumed dynamic load factor for fast loading on slow-to-respond above ground pipe or DLF=1 for buried pipe.

7.0 Analytical Models

7.1 Three Models, and When to Use Them

This report provides the user three types of analytical models that can be used in the design of buried pipelines. These models are:

- Chart method (Section 7.2). The simplest approach. Avoids all mathematical models, and allows the designer to pick a style of pipe installation based on parameters such as regional maps for PGA and PGD hazards, and the relative importance of the pipeline in the context of the entire water system.

- Equivalent static method (Section 7.3). Uses simple quantifiable models to predict the amount of force, strain and displacement on a pipe for a particular level of earthquake loading. The pipeline can then be designed to meet these quantified values, or pipe styles can be selected that presumably meet these quantified values without a formal capacity to demand check. Pipe selection is usually made by specification from available manufacturer's catalogs.

- Finite element method (Section 7.4). This method uses finite element models to examine the distribution of loading (whether PGA, PGV or most often PGD) over the length of the pipeline, and then uses beam on inelastic foundation finite element models (or sometimes use 2D or 3D mesh models) to examine the state of stress, strain and displacement within the pipeline and pipeline joints.

7.2 Chart Method

The Chart Method combines the pipe function classification with the level of seismic hazard to indicate a style of pipeline design.

Figure 2-1 lists the basic steps in the Chart Method:

- Step 1. Get the geographic location of the pipeline.
- Steps 2 and 3. Select the Function Class (I, II, III or IV) factor for the pipeline Section 3 describes how to define the Function Class.
- Steps 4 to 7. Get the level of seismic hazard (PGV, PGD). Section 4 describes how to define the seismic hazard.
- Step 8. Pick a category of pipeline construction (A, B, C, D, E). Use Tables 7-1 through 7-10.
- Step 9. Pick the actual style of construction. Use Tables 7-11 to 7-19.
- Steps 10 and 11. Review Sections 8 through 12 for examples of construction styles to guide design.
- Step 12. Prepare the plans, profiles and specifications for the actual design. These Guidelines do not provide Step 12.

7.2.1 Transmission Pipelines

Transmission pipelines may carry raw or treated water. Transmission pipelines are typically assigned Function Class II, III or IV; the exception would be to use Function Class I for those pipes whose failure would not impact any customers for 30 days or more.

Use Tables 7-1 through 7-4 to set the pipeline design category (A, B, C, D or E). For Function Class I, the pipeline design category is always A. If a portion of a pipeline has two or more categories for the various hazards (ground shaking, transverse PGDs, parallel PGDs, fault offset PGDs), then the highest category controls for that portion of the pipeline. Design categories for sub-transmission pipelines may also be set using Tables 7-1 to 7-4.

Inch/sec	Function II	Function III	Function IV
$0 < PGV \leq 10$	A	A	A
$10 < PGV \leq 20$	A	A	B
$20 < PGV \leq 30$	A	B	C
$30 < PGV$	B	C	D

Table 7-1. Transmission Pipelines – Ground Shaking

Inches	Function II	Function III	Function IV
$0 < PGD \leq 2$	A	A	A – welded steel B - segmented
$2 < PGD \leq 6$	A	A	B
$6 < PGD \leq 12$	A	B	C
$12 < PGD$	B	C	D

Table 7-2. Transmission Pipelines – Liquefaction (Settlement of Lateral Spread) and Landslide Perpendicular to Pipeline Alignment (Transverse Loading)

Inches	Function II	Function III	Function IV
$0 < PGD \leq 2$	A	B	B
$2 < PGD \leq 6$	B	B	C
$6 < PGD \leq 12$	C	C	D
$12 < PGD$	D	D	E

Table 7-3. Transmission Pipelines – Liquefaction (Lateral Spread) and Landslide Parallel to Axis of Pipeline (Longitudinal Loading)

Inches	Function II	Function III	Function IV
$0 < PGD \leq 2$	A	B	B
$2 < PGD \leq 6$	B	B	C
$6 < PGD \leq 12$	C	C	D
$12 < PGD \leq 24$	D	D	E
$24 < PGD$	D	E	E

Table 7-4. Transmission Pipelines – Fault Offset

7.2.2 Distribution Pipelines

Distribution pipelines are typically in networks. Failure of a single distribution pipeline will not fail the entire network (once that pipe is valved out), but the customers on that failed distribution pipeline will have no water service until the pipe is repaired. In most cases, the engineer can assume that distribution pipelines are "redundant", except in the following cases:

- The pipeline is the only pipe between lower elevation pump station and upper elevation pump station / reservoir in a pressure zone, and that failure of that pipeline will lead to complete loss of supply to the pump station serving a higher zone, or loss of the water in the reservoir for fire fighting purposes. For example, a 12" diameter pipe from lower elevation pump station that delivers water to a higher elevation tank within a pressure zone, and that also serves water to higher elevation pump stations.

- The pipeline is the only pipe delivering water to particularly important customers, such as critical care hospitals. For example, an 8-inch diameter pipe that has a service connection to a 200 bed hospital.

It has been the experience in past earthquakes that there can be a great quantity of damage to distribution pipelines, especially in areas prone to PGDs. While no single distribution pipeline, will in general, be as important as a transmission pipeline, the large quantity of damage can lead to rapid system-wide depressurization, loss of fire fighting capability, and long outage times due to the great amount of repair work needed. Accordingly, it is recommended that most distribution pipes be classified as Function II and very few as Function I (under ~5% of total pipeline inventory). A few distribution pipes serving essential facilities could be classified as Function III or IV; or they could be designated in suitable emergency response plans as prioritized for rapid repair (generally under one day or two days at most).

Inch/sec	Function I	Function II	Function III, IV
$0 < PGV \leq 10$	A	A	A
$10 < PGV \leq 20$	A	A	A
$20 < PGV \leq 30$	A	A	A (with additional valves)
$30 < PGV$	A	A (with additional valves)	B

Table 7-5. Distribution Pipelines – Ground Shaking

Inches	Function I	Function II	Function III, IV
0 < PGD ≤ 2	A	A	A (with additional valves)
2 < PGD ≤ 6	A	A (with additional valves)	B
6 < PGD ≤ 12	A	B	C
12 < PGD	A	C	C

Table 7-6. Distribution Pipelines – Liquefaction (Settlement and Lateral Spread) and Landslide Perpendicular to Pipeline Alignment (Transverse Loading)

Inches	Function I	Function II	Function III, IV
0 < PGD ≤ 2	A	A	B (with additional valves)
2 < PGD ≤ 6	A	B	C
6 < PGD ≤ 12	A	C	D
12 < PGD	A	D	D

Table 7-7. Distribution Pipelines – Liquefaction (Lateral Spread) and Landslide Parallel to Axis of Pipeline (Longitudinal Loading)

Inches	Function I	Function II	Function III, IV
0 < PGD ≤ 2	A	B	B
2 < PGD ≤ 6	A	B	C
6 < PGD ≤ 12	A	C	D
12 < PGD ≤ 24	A	D	E
24 < PGD	A	E	E

Table 7-8. Distribution Pipelines – Fault Offset

7.2.3 Service Laterals and Hydrant Laterals

Inch/sec	Any Lateral
0 < PGV ≤ 10	A
10 < PGV ≤ 30	A
30 < PGV	B

Table 7-9. Laterals – Ground Shaking

Inches	Any Lateral
0 < PGD ≤ 2	A
2 < PGD ≤ 12	B
12 < PGD	C

Table 7-10. Laterals – Liquefaction, Landslide and Surface Faulting

7.2.4 Design Approach

There are five design categories. Category A denotes standard (non-seismic) design and the others have progressively increasing seismic ruggedness. The following summarizes the general design approach for Categories B, C, D and E:

- B = restrained pipe joints with extra valves
- C = same as B plus use of better pipe materials
- D = same as C plus quantified seismic design; or provide bypass system per Section 9.
- E = same as D plus peer review (it is strongly recommended that FEM method be used for any pipe with Category E)

Tables 7-11 through 7-19 provide guidance for design based for each category A through E. This guidance is based on commonly available pipe and joinery as of 2005. As new pipe products become available, they can be used in the chart method as long as suitable justification (FEM, test, etc.) is provided to show that the pipe meets the intended pipe performance goal.

Pipe Category	Design Features	Notes
A	Standard	
B	Extended Joints	
C	Restrained Joints	
D	Extended and Restrained Joints – or use other material	Standard with bypass[1]
E	Special Joints	Standard with bypass

Table 7-11. Ductile Iron Pipe

Pipe Category	Design Features	Notes
A	Standard	
B	Standard with extra insertion	
C	Restrained Joints	
D	Not recommended	Standard with bypass
E	Not recommended	Standard with bypass

Table 7-12. PVC Pipe

[1] Instead of using special joinery, standard push on joints can be used in conjunction with a bypass system such as described in Section 9.2.

Pipe Category	Design Features	Notes
A	Single Lap Weld	
B	Single Lap Weld	Weld dimension t = pipe t[1]
C	Double Lap Weld	Weld dimension t = pipe t
D	Double Lap Weld / Butt Weld	D/t max 110 in PGD zones[2]
E	Butt Weld	D/t max 95 in PGD zones

Table 7-13. Welded Steel Pipe

Pipe Category	Design Features	Notes
A	Standard	
B	Extended Joints	Avoid in high PGD zones[3]
C	Extended Joints	Avoid in high PGD zones
D	Extended and Restrained Joints – or use other design	Standard with bypass
E	Not recommended	Standard with bypass

Table 7-14. Gasketed Steel Pipe

Pipe Category	Design Features	Notes
A	Gasketed or Single Lap weld	
B	Single Lap Weld	Weld dimension t = cylinder t[4]
C	Double Lap Weld	Weld dimension t = cylinder t
D	Not recommended	Standard with bypass
E	Not recommended	Standard with bypass

Table 7-15. CCP & RCCP Pipe

Pipe Category	Design Features	Notes
A	Standard	
B	Butt Fusion Joints	
C	Butt Fusion Joints	
D	Butt Fusion Joints	
E	Butt Fusion Joints	

Table 7-16. HDPE Pipe

[1] The weld thickness t should equal the pipe wall thickness t

[2] The ratio of pipe diameter D to pipe wall thickness t should be limited to the maximum listed

[3] Each extended joints must be able to accommodate the entire PGD. For PGDs much higher than a few inches, it is not likely that extended joints are practical.

[4] The weld thickness t should equal the steel cylinder wall thickness t

Pipe Category	Design Features	Notes
A	Standard	
B	Soldered joints	
C	Soldered joints	Expansion loop / Christie box / Other box

Table 7-17. Copper Pipe

Pipe Category	Design Features	Notes
A	Standard	
B	Dresser-type coupling	
C	Multiple dresser couplings	
D	EBAA flextend type couplings	
E	Do not use - relocate	

Table 7-18. Segmented Pipelines Used as Hydrant Laterals

Pipe Category	Design Features	Notes
A	Bolted, Single Lap Weld, Fusion Weld	
B	Bolted, Single Lap Weld, Fusion Weld	Weld t = pipe t
C	Bolted, Double Lap Weld, Single Lap Weld with fiber wrap, Fusion Weld	Weld t = pipe t
D	Bolted, Double Lap Weld, Single Lap Weld with fiber wrap[1], Butt Weld, Fusion Weld	
E	Bolted, Double Lap Weld, Single Lap Weld with fiber wrap, Butt Weld, Fusion Weld	

Table 7-19. Continuous Pipelines Used as Hydrant Laterals

In addition to the pipe design styles in Tables 7-11 through 7-19, the following additional requirements are made. These recommendations are cumulative (For C, include B and C recommendations).

- B. Add isolation valves on all pipes within 50 feet of every intersection, for example, four valves on a four-way cross.

- C. Maximum pipe length between connections for segmented pipe is 16 feet, or as otherwise justified by ESM or FEM.

[1] Experimental tests have shown that a single lap welded pipe with external epoxy-attached fiber wrap have essentially the same capacity as butt welded pipe.

- D. Maximum pipe length between connections for segmented pipe is 12 feet, or as otherwise justified by ESM or FEM.

7.3 Equivalent Static Method

The Equivalent Static Method (ESM) computes pipe seismic response quantities (forces, displacements, strains) using idealized models describing the interaction of the hazard, soil and pipe. The purpose is to account for the physical aspects governing the pipe behavior in a simplified manner so the designer can apply the method to specific situations with the understanding of the key mechanisms influencing behavior.

The ESM methodology can be refined using techniques discussed in the commentary and/or using the Finite Element Method. Considering the importance of the pipeline, variability and uncertainty in the hazard description as well as the soil conditions, the capability of the pipeline, and the adverse impacts of limited damage, such refinement may not be warranted or cost effective.

In the ESM, the ground motions might be estimated using regional maps. Then, using simplified models, the ground motions are applied to the pipeline to compute forces or in the pipe body and displacements at the pipe joints. The final pipeline design requires the forces and displacements are less than allowable values.

The ESM makes a number of simplifying assumptions, and it should be understood that the ESM sometimes cannot completely account for unusual ground motions or pipeline configurations. The ESM assumes that pipe manufacturers have or will determine certain capacities (like joint movements) and are willing to make such data available to designers. As of 2005, such information is not widely available in vendor catalogs. However, it is the hope that over time, various pipe manufacturers will provide products with the desired earthquake performance using catalog-type product selection.

The ESM can be used for calculation of pipe response resulting from:

- Ground shaking hazard that produces transient ground strains from seismic wave passage (Section 4.2), and

- Ground failure hazards such as landslides, liquefaction, or surface faulting that result in permanent ground deformations (Sections 4.3, 4.4, 4.5).

7.3.1 Analysis for Ground Shaking Hazard

Ground shaking causes transient ground strains from seismic wave passage that is categorized according to peak ground velocity (PGV). These cause transient strains in buried pipe as it deforms with the soil. The buried pipe moves with the soil at locations subject only to ground shaking without ground failures such as liquefaction, landslide or fault offset. Peak strain in the soil may be estimated as follows:

$$\varepsilon_{soil} = \frac{PGV}{c} \qquad \text{[Eq 7-1]}$$

where, PGV = peak ground velocity at pipe location as computed per Section 4.2, and c = seismic wave propagation speed in the soil at the pipe location. The wave propagation speed may be taken as 13,000 feet per second unless otherwise justified.

Continuous Pipe

A continuous pipe has joints possessing significant strength and stiffness relative to the pipe barrel (often referred to as restrained joints). An example is a steel pipe having welded (single lap, double lap or butt welded) joints.

The force for designing the pipe barrel and joints may be taken as the smaller of F_1 or F_2 where F_1 is the force assuming the pipe is fully compliant with the soil (ie., the pipe does not slip through the soil), and F_2 is the ultimate force the soil can transfer to the pipe.

Assume that the ground strain is transferred to the pipe without slip. Then

$$\varepsilon_{pipe} = \varepsilon_{soil} = \frac{PGV}{c}$$

$$F_1 = AE\varepsilon_{pipe} \qquad \text{[Eq 7-2]}$$

$$F_2 = \frac{t_u \lambda}{4} \qquad \text{[Eq 7-3]}$$

where, A = pipe body axial area, E = Youngs modulus of pipe, t_u = ultimate frictional force of soil acting on pipe barrel in axial direction (force per unit pipe length) computed per Section 7.4, and λ = seismic wavelength in soil at pipe location. The wavelength may be taken as 6,500 feet unless otherwise justified. Section C7.3.1 provides an example.

Segmented Pipe

A segmented pipe has joints having low strength and stiffness relative to the pipe barrel (often referred to as unrestrained joints). An example is a ductile iron or PVC pipe having push-on bell-and-spigot gasketed joints. The ground strains are assumed to be transformed into relative axial displacements between pipe segments that must be accommodated in the pipe joints. Should the resulting relative joint displacement be greater than that available in the joint, the pipe segments will separate at the joint in tension, or the segments will bear against each other in compression, possibly leading to telescoping inside of one another, or local buckling (wrinkling) of the pipe barrel. The axial displacement (in both the axial shortening and lengthening directions) that the joint must be able to accommodate may be taken as follows.

$$\Delta_{joint} = 7L_p\varepsilon_{soil} \qquad \text{[Eq 7-4]}$$

where, L_p = length of the pipe segment.

Laboratory tests show that the axial stiffness and strength vary from joint to joint. As a result the weak joints are subject to larger relative joint displacements than their stronger neighbor. El Hamadi and O'Rourke (1990) have shown that for cast iron pipe with lead caulked joints about one in a hundred joints (1% of joints) are subject to three times the average joint displacement while one in a thousand (0.1%) are subject to five times the average. For design purposes, the Guidelines recommend the use of seven times the average joint movement, and this would be expected to result in damage in no more than 1 in 10,000 joints. For example, assuming a PGV of 50 cm/sec and pipe segment length of 16 feet, then:

$$\Delta_{joint} = 7L_p\frac{PGV}{c} = 7*16*12\frac{50}{13{,}000x12x2.54} = 0.17 \ \ inches \qquad \text{[Eq 7-5]}$$

Note that the joint displacement is relatively small.

Continuous Pipe - Design Considerations

If using a single lap welded pipe, the stress in the joint will be amplified over the stress in the main body of the pipe. This is caused by several reasons: the geometry of the joint will introduce net bending, which will increase the maximum longitudinal stress; the stress within the lap weld will include the factors of longitudinal axial, bending and hoop forces; the thickness of the weld; and possibly stress concentrations at within and near the weld due to weld flaws. In the ESM method, we make the overly simplified assumption that most single lap welded joints (outside welds) with minimum leg size equal to the minimum pipe wall thickness can sustain some localized yielding before leading to failure, so we suggest the following acceptance criteria.

$$\sigma_{pipe} \leq \ 0.40F_y \qquad \text{[Eq 7-6]}$$

where F_y = nominal specified yield stress of the pipe. This formula implies a joint efficiency of about 35% as compared to the strength of the pipe. For cases where a single lap welded pipe is used with thinner welds, then use:

$$\sigma_{pipe}\frac{t}{t_{weld}} \leq 0.40F_y \qquad \text{[Eq 7-7]}$$

Single lap welded steel pipes exhibit about the same strength in tension or compression. Once the axial load reach about $\pm 0.60\ F_y$ in the main body of the pipe, strains within the single lap weld will reach about 5% to 6% under compressive loading or 8% to 9% under tension loading. The design longitudinal stress allowable for a single lap welded pipe (external lap weld equal to wall thickness) should not exceed about $0.60F_y$ in the main pipe, under maximum earthquake.

For double lap welded steel pipe with common fit up tolerances, replace 0.40 with 0.90 for tensile loading. Due to eccentricities in a double lap welded pipe at the connection, predicted tensile stresses of 0.90Fy in the main body of the pipe away from the joint will translate to about 3% strain or so within the highest strained part of the lap welded joint. Initial yielding of the double lap welded joint will occur at about 50% to 60% F_y in the main pipe. In the highly nonlinear realm, predicted tensile longitudinal strains of about 5% in the main body of the pipe will translate to about 10% strain in the highest strained part of the welded lap joint.

In compression, a double lap welded steel pipe with common dimension tolerances (D/t = 175) will buckle at a compressive load of about $0.60F_y$. The pipe will continue to shorten in its buckled shape as compressive loading is maintained, albeit with load shedding and with increasing strain in the pipe. By the time the wrinkle has formed to cause about 1 inch bulging in or out, the peak strain in the male or female parts of the spigot joint will reach about 13 to 14% strain (unpressurized) or about 12% to 15% strain (pressurized to 150 psi).

For butt welded pipe, replace 0.90 with 1.00; or use nonlinear strain acceptance criteria.

If the designer opts for some nonlinear performance of the pipe, the stress checks should be replaced with tensile strain limit checks and wrinkling checks. Single lap welded pipe should generally be limited to the above elastic limits. Double lap welded or butt welded pipe can accept some strain or wrinkling, with butt welded pipe performing better than double lap welded pipe. For water pipes, some wrinkling is acceptable if the owner accepts this performance, and if pipe failure does not lead to serious impacts to nearby pipes, structures or habitat.

For pipes connected using bolted flanged joints, then the above equations are used, with the weld being that between the flange and the pipe. It is assumed that the flange and bolts will be sized based on pressure requirements, and that seismic loading from ground shaking will not control.

Segmented Pipe - Design Considerations

The predicted movement of the pipe joint should be less than the movement capacity of the pipe joint. The predicted joint movement is both for tension (pull out) and compression (push in) of the joint. Depending of the style of pipe hardware used, the joint might allow tension and compression movements with similar resistance (common for PVC-style joints) or dissimilar resistance (common for cast iron and ductile iron joints). From observation in past earthquakes, compression failures of ductile iron pipes are rare, so this is not a significant concern. Thus, the larger concern is that the pull out movement should not exceed the capacity of the joint.

The required joint movement (in tension) should be capable of resisting the predicted seismic joint movement as follows:

$$\Delta_{joint} \geq \Delta_{seismic}$$

where Δ_{joint} is the capacity against joint pull out at the time of the earthquake, considering any simultaneous operational forces (temperature, static thrust, hydrodynamic thrust, etc.) and in consideration of construction fit-up tolerances. Without causing too much damage to an overall pipe network, joint pullout failures on the order of one per 6,000 should be tolerable. For pipelines constructed with good quality control, and with the spigot end of essentially every joint is installed to the full depth required, then:

$$\Delta_{design} = \Delta_{seismic} + \Delta_{operational}$$

To provide for some measure of safety, a margin might be included in the design as follows:

$$\Delta_{design} = \Delta_{seismic} + \Delta_{operational} + 0.25 \ \ inch \qquad \text{[Eq 7-8]}$$

Where the 0.25 inch value accommodates the likely range of fit-up tolerances in construction to cover the vast majority of the installations. For cast iron pipe, the previous example suggests that using 7 times the average joint opening movement might be used rather than the average movement plus 0.25 inches. Either design approach seems reasonable.

Concurrent with the joint displacement requirement, segmented pipe joints will be required to take some joint rotation under seismic loads. For these Guidelines in the ESM approach, we do not provide a specific rotation capability numerical check, as it is felt that any rubber gasketed joint capable of providing the axial displacement requirement will also provide adequate joint rotation. However, for installations designed to take PGDs of a few inches or more over one pipe segment length, joint rotations will be important, and the above approach is not suitable.

Continuous Pipeline with One Unrestrained Joint

A common situation arises when an expansion coupling is inserted into long continuous steel pipeline. For example, the expansion coupling may be useful to allow for removal of a valve in a welded steel pipeline. As another example, consider the case of a reinforced concrete cylinder pipe with gasketed and cemented joints, when just one of the joints is cracked.

For this case, the joint opening (or closing) movement can be calculated using the following approach, which is described in ASCE (1984) and quantified by O'Rourke, Wand and Shi (2004).

Assume that the ground strain ε_g acts over a pipeline with axial area A, Young's modulus E, axial skin friction t_u (defined in Section 7.4), wavelength λ (Figure 7-1).

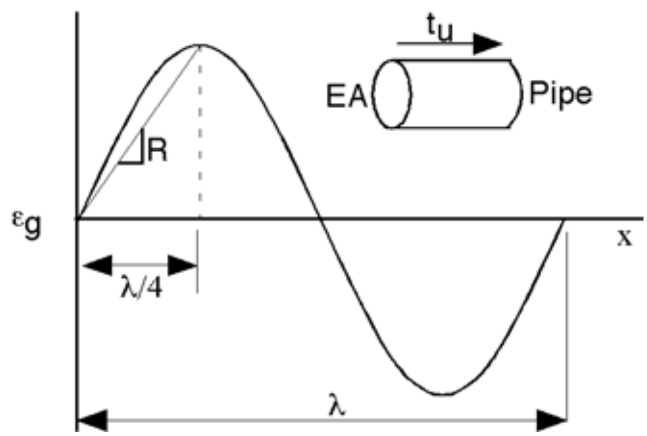

Figure 7-1. Sinusoidal Wave Interaction with Pipe

Assume that at the expansion joint (or single cracked joint) that the strain in the pipe ε_p is zero. The maximum accumulation of strain in the pipe away from that joint is limited to that force transferred from the soil to the pipe, t_u. At some distance L from the joint, the strain in the pipe will increase to the strain in the ground, beyond which the pipe will move with the ground (assuming no further soil-pipe slippage, which is reasonable under most ground shaking hazard situations). This analogy is shown in Figure 7-2.

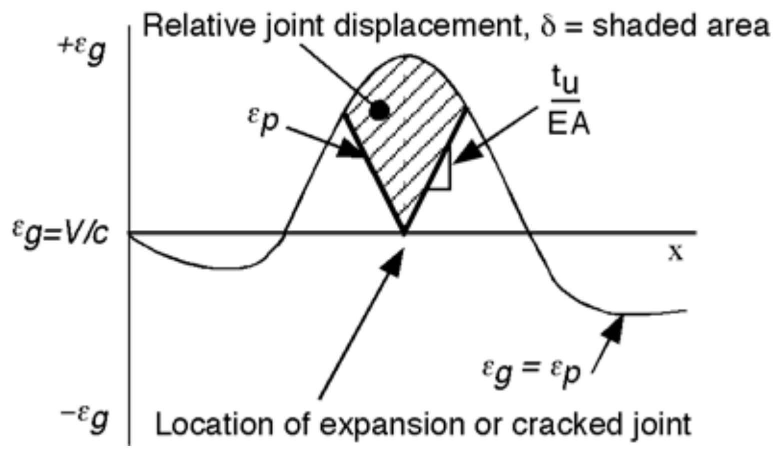

Figure 7-2. Relative Joint Displacement at Expansion Joint in Continuous Pipeline

If one can estimate all the parameters in Figure 7-2, then the relative axial joint displacement is just the area between the pipe strain and ground strain curves. If one assumes that the wave length is very long such that the strain in the pipe equals (or nearly equals) the maximum ground strain, then the area under the curve in Figure 7-2 is:

$$\delta = \left(\frac{V}{c}\right)^2 \left(\frac{EA}{t_u}\right) \qquad \text{[Eq 7-9]}$$

This will overestimate the true relative joint displacement, in all practical cases. O'Rourke, Wang and Shi (2004) ran a series of finite element analyses for pipes with varying E and A, soil conditions t_u and seismic wave characteristics (λ, c, T= λ/c , where T = dominant wave period). The results are summarized in Figure 7-3.

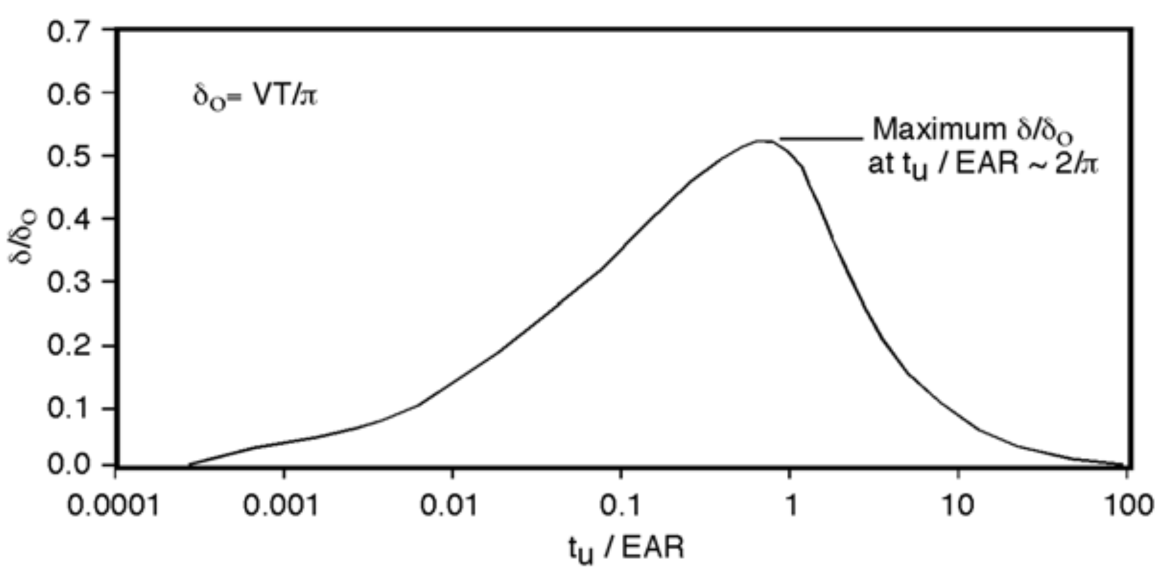

Figure 7-3. Relationship Between δ/δ_0 and t_u/EAR

7.3.2 Landslide and Liquefaction Permanent Ground Deformations

Earthquakes can trigger landslides and soil liquefaction that are the mass movement of soil over an extended area. These can be very damaging to buried pipes as they are dragged along within the soil mass and experience applied relative deformations. The pipe response depends on its orientation relative to the direction of the soil mass permanent ground displacement (PGD), as highlighted in Figure 7-4.

- A pipe run oriented parallel to the soil movement is defined as experiencing *longitudinal* PGD.
- A pipe run oriented perpendicular to the soil movement is defined as experiencing *transverse* PGD.
- We do not provide ESM formula for intermediate cases, but a vector addition of the two cases could be applied should a pipe be exposed to some movement in both the longitudinal and transverse directions. The FEM method can treat any orientation.

In general, longitudinal PGD are more damaging than transverse PGD. Empirical observations suggests that damage rates for non-seismically designed pipes for longitudinal PGDs have been 5 to 10 times higher than corresponding damage rates due to transverse PGDs. This is in part due to the fact that a pipe is inherently more flexible or compliant when subject to bending (transverse PGD) then when subject to axial tension or compression (longitudinal PGD).

In the ESM method, highly simplified and semi-empirical methods are provided to treat permanent ground deformations. These approaches can be used for pipes that traverse through liquefaction zones. For especially important pipes that are subject to large PGDs (over a foot or so) (like landslides or surface fault offset), the more detailed FEM is recommended.

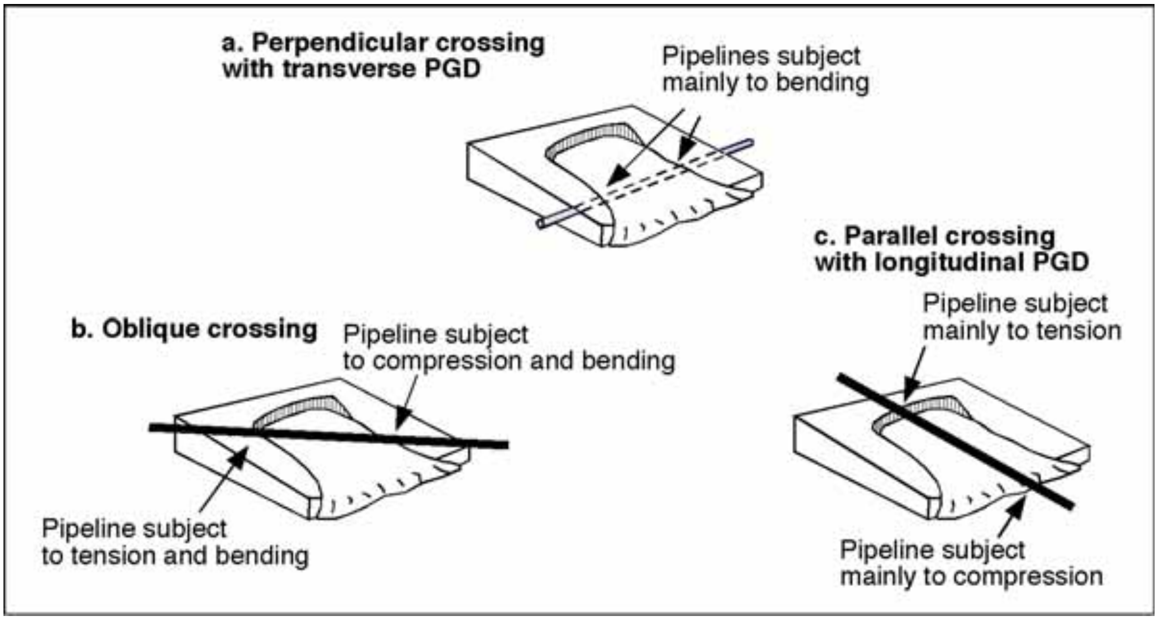

Figure 7-4. Principal effects of PGDs on pipelines according to their orientation

The direction of PGD for landslides is assumed to be down-slope. The direction of PGD due to liquefaction lateral spreading may be assumed as follows:

- Locations ≤ 1,000 feet from of a water boundary (such as a stream, lake, or ocean front that constitutes a "free-face") will have PGD directed toward the free-face.
- Locations > 1,000 feet from a water boundary having an average slope > 1% will have PGD directed down-slope.
- Locations not meeting above may have PGD oriented in any direction.

Buried Pipe Response to Longitudinal PGD

The maximum pipe forces and displacements generally occur at the margins of the soil mass undergoing movement causing either pipe tension (pull-out at the head of the moving soil mass, point A in Figure 7-5), or pipe compression (push-in at the toe of the moving soil mass, point B in Figure 7-5). Design for longitudinal PGD will generally be controlling.

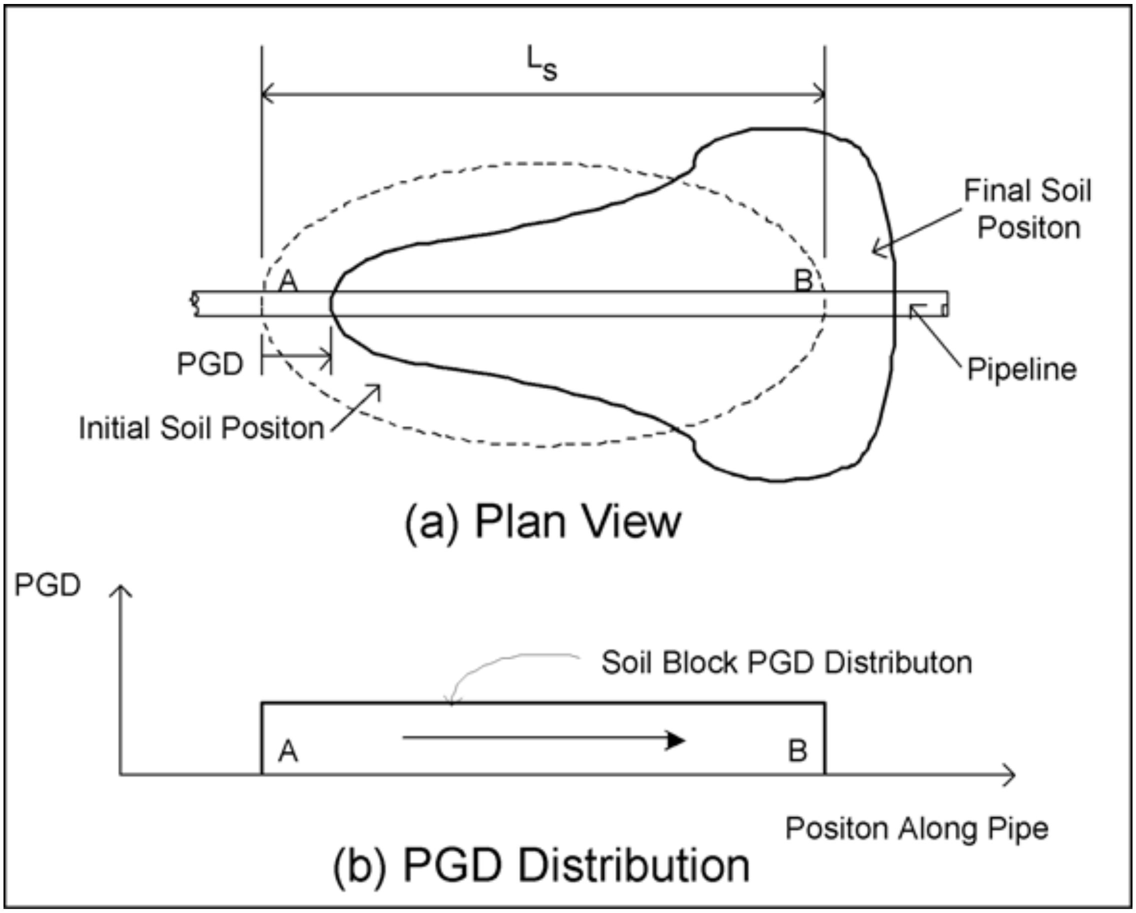

Figure 7-5. Pipe Response to Longitudinal PGD

Continuous pipe. The force for designing the pipe barrel and joints may be taken as the smaller of F_1 or F_2 representing upper bound estimates of the axial force in the pipe. F_1 is the force assuming the pipe is elastic and fully compliant with the soil, and F_2 is the ultimate force the soil can transfer to the pipe.

$$F_1 = \sqrt{AEt_u\delta} \qquad \text{[Eq 7-10]}$$

where, δ = PGD displacement estimated from Sections 4.3 and 4.4. In lieu of specific knowledge about the particular site, e.g., via geotechnical studies, the commentary contains suggested values for δ .

$$F_2 = \frac{t_u L_s}{2} \qquad \text{[Eq 7-11]}$$

where, L_s = length of pipe in soil mass undergoing movement estimated from Section 4.3 and 4.4. In lieu of specific knowledge about the particular site, e.g., via geotechnical studies, the commentary provides suggested values for L_s. F_2 assumes that half the total applied soil load is resisted in tension and half in compression; for pipes with bends, up to the entire applied soil load may be imposed on the pipe.

In situations where elastic design using the computed force above is not practical (requiring pipe and joints to have excessive strength), plastic design is recommended. For plastic design, the pipe should consist of ductile material capable of large plastic strains without fracture, and the joints should be capable of developing the strength of the adjoining pipe segments (e.g., steel pipe having welded joints).

For Function Class III or IV continuous pipe with bends in or near the PGD zone, the FEM method is suggested; equations 7-10 and 7-11 do not account for bends.

<u>Segmented pipe</u>. The ground displacement is assumed to be accommodated by pipe joint expansion and contraction. The axial displacement that the joint must be able to accommodate may be taken as follows.

For push-on type pipe joints (not having mechanical stops preventing pipe segments from pulling apart), the design displacement may be taken as:

$$\Delta_{joint} = \delta$$

For pipe joints having mechanical stops preventing pipe segments from pulling apart, the PGD may be assumed to be distributed over several joints. We call such joints "chained joints". The design displacement may be taken as:

$$\Delta_{joint} = \frac{\delta}{n} \qquad \text{[Eq 7-12]}$$

Where, n = the number of chained restrained joints near the head or near the toe of the moving soil mass that will expand to absorb the total PGD. Figure 7-6 illustrates the definition of n. In Figure 7-6, we illustrate that the sharply imposed PGD is equally taken up by n=3 joints at both the head and toe of the soil mass. This implies that all the joints and all the soil is equally stiff and strong. In reality, it is quite possible that the soil mass on one side of the head or two will be stiffer than on the other side (hence the proclivity for the ground crack at the interface. This might force all n joints to absorb the PGD to be on just one side of the ground crack, not equally distributed as illustrated.

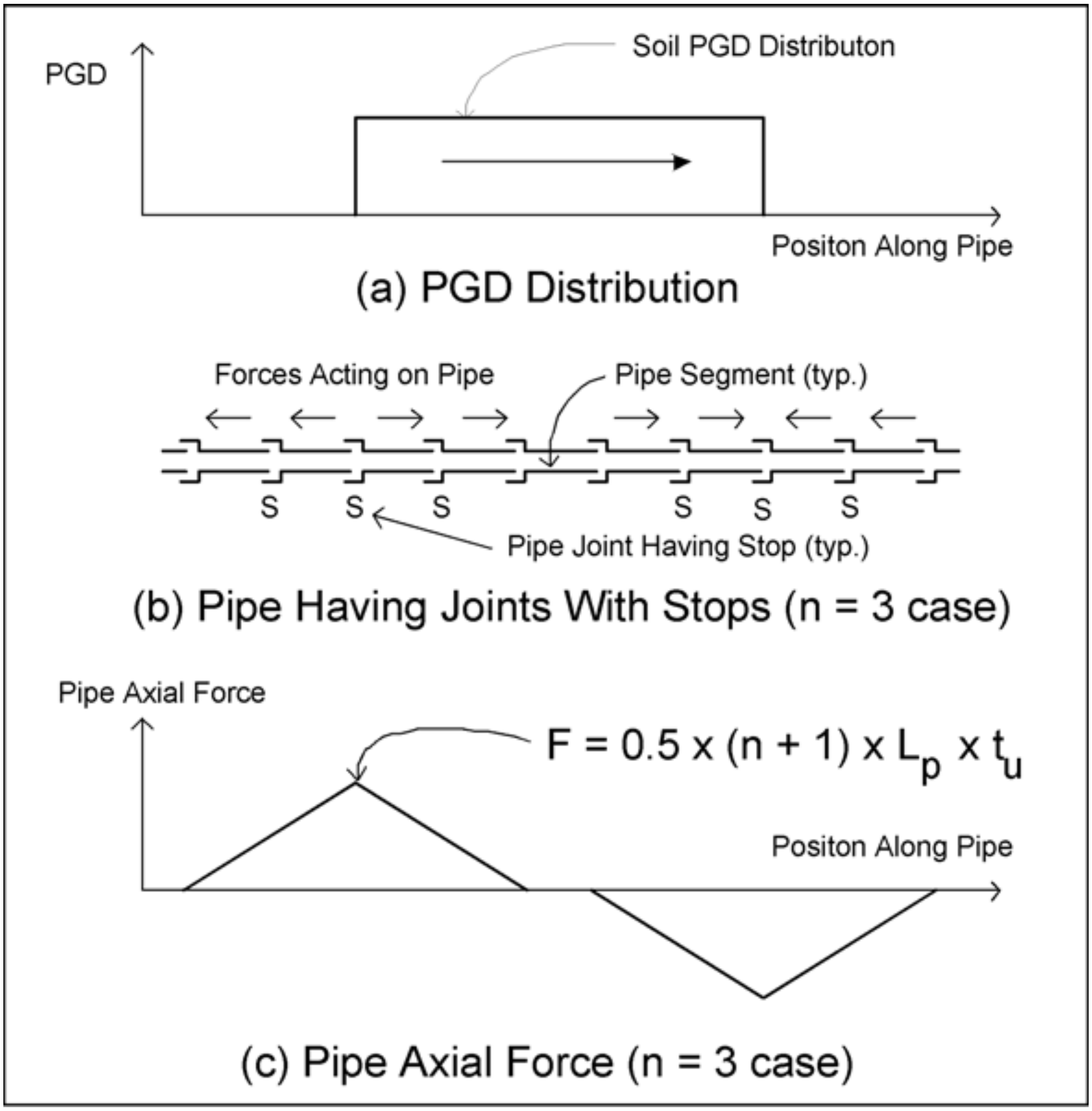

Figure 7-6. Chained Segmented Pipe Subject to Longitudinal PGD

The mechanical stops (restrainer rings, etc.) for each joint in this case must be designed to accommodate $F_{stop} \geq 1.0 * t_u * L_p * (n+1)$ with a suitable factor of safety (implying a factor of safety = 2 used in this computation). F_{stop} need not be higher than the yield strength of the pipe barrel.

<u>Segmented pipe, alternate method</u>. For terrain units identified as having "high" or "very high" liquefaction susceptibility, an alternative method to estimate axial joint displacement is as follows.

- At locations within 1,000 feet of a water boundary or on land with average slope more than 1%, the resulting ground strain ε_g, in the down-slope (toward the water) horizontal direction may be assumed to be 1.5%. The ground strain is assumed to be uniform throughout.

- At locations more than 1,000 feet from of a water boundary or on land with average slope less than 1%, the resulting ground strain ε_g, in any horizontal direction may be assumed to be 0.75%. The ground strain is assumed to be uniform throughout..

For segmented pipes installed in such liquefaction areas, a chained joint can be designed to accommodate the ground strain as follows, (a chained joint is a segmented joint with the additional requirement of having mechanical stops to prevent the pipes from pulling apart should the amount of PGD require movement at more than one joint). The chained joints are installed throughout the zone subject to PGD, plus at least the first three joints (or for a pipe length needed to provide full anchorage) outside the PGD zone.

$$\Delta_{joint} = \varepsilon_g L_p \qquad \text{[Eq 7-13]}$$

The strength of the chained joint stop should be high enough to accommodate the accumulated pulling load, $F_{stop} \geq 1.0 * t_u * L_p * (n+1)$. However, in this alternate design approach, n is not formally computed; one would have to rely upon the manufacturer's catalog item to provide a suitably strong stop, or the designer can work with the manufacturer to establish a suitably strong stop.

Buried Pipe Response to Transverse PGD

The pipe is characterized as taking the displaced shape of a beam under lateral loading with the peak displacement occurring at the middle of the span (i.e. at the center of the soil mass), Figure 7-7. This assumes a distributed PGD across the slide having a maximum displacement near the center and small displacements near the margins of soil mass. (A more severe situation is where the PGD occurs abruptly near the margins of the soil mass analogous to a pipe fault crossing discussed below. However, effectively designing for this situation requires rather specific knowledge about the locations of the soil mass margins so that treating it this way ought to be considered only if there is site-specific geologic hazard information.)

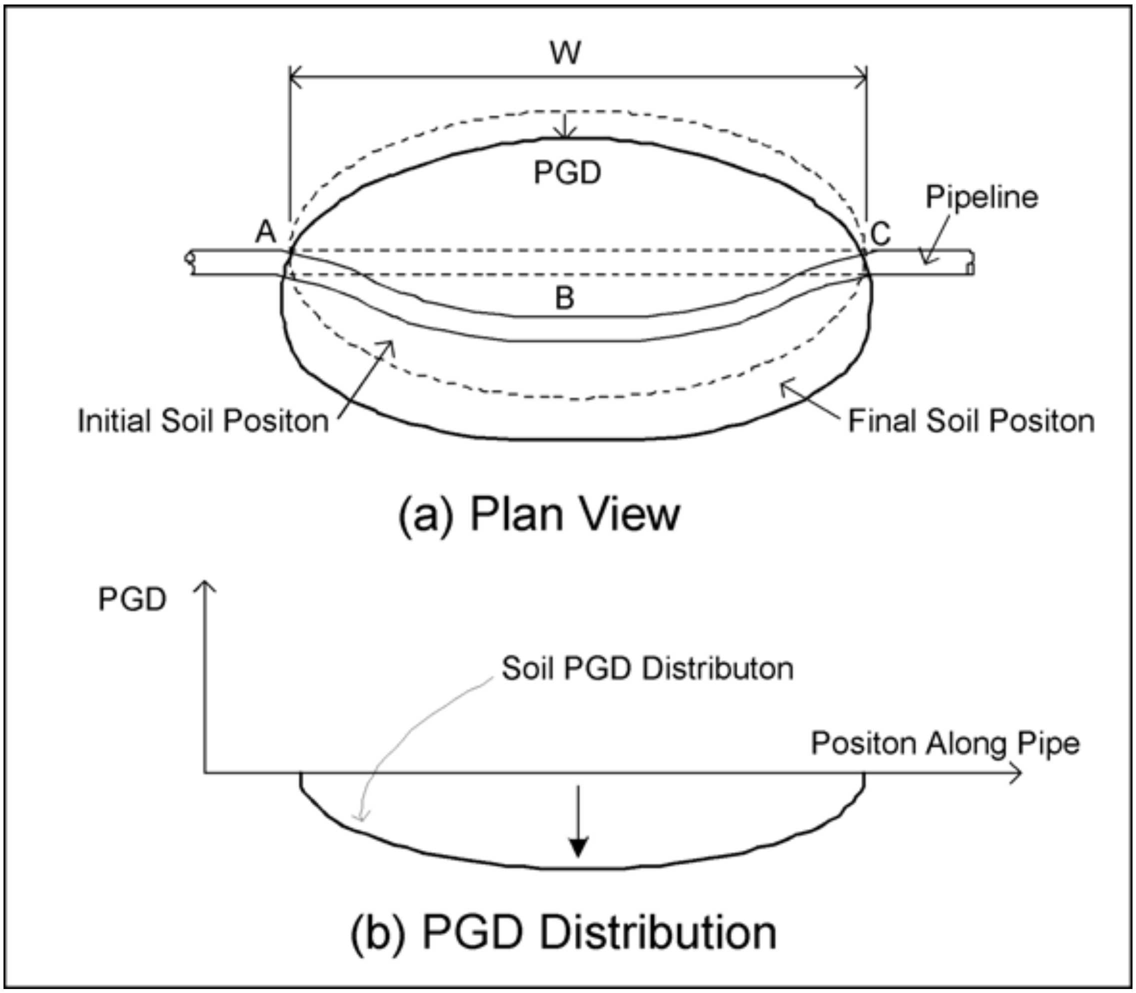

Figure 7-7. Pipe Response to Transverse PGD

Continuous pipe. The peak bending strain in the pipe can occur either at the center or near the margins of the transverse-moving soil mass, points A, B and C in Figure 7-7. These may be conservatively estimated as the smaller of the following.

$$\varepsilon_b = \pm \frac{\pi D \delta}{W^2} \qquad \text{[Eq 7-14]}$$

where, W = the width of the soil mass as estimated from Section 4.3 and 4.4, D is the outside diameter of the pipe and δ is the peak displacement of the PGD. In lieu of specific knowledge about the particular site, the commentary contains suggested values for δ and W.

$$\varepsilon_b = \pm \frac{p_u W^2}{3\pi E t D^2} \qquad \text{[Eq 7-15]}$$

where, t = pipe wall thickness, and p_u = ultimate lateral bearing force of soil acting on pipe barrel in transverse direction (force per unit pipe length) computed per Section 7.4.

The peak moment for checking the pipe barrel and joints is given as follows.

$$M = \varepsilon_b E S \qquad \text{[Eq 7-16]}$$

where, S = pipe section modulus.

In situations where design using the computed bending strain above is not practical (requiring pipe and joints to have excessive strength, or compressive bending strain exceeds wrinkling capacity), refined analysis (FEM) is recommended (the above approach likely over predicts pipe bending strain by a substantial amount). In such cases, site specific estimates for both W and δ are recommended. In lieu of such refinement, the pipe could be designed to accept some plastic deformations, such as: the pipe should consist of ductile material capable of large plastic strains (at least 4% to 5% in tension and about -1% in compression) without fracture, and the joints should be capable of developing the strength of the adjoining pipe segments (e.g., steel pipe having butt welded joints, or if the pipe diameter is large enough, double lap welded joints).

<u>Segmented pipe</u>. The transverse PGD causes a combination of axial extensions and angular rotations in the pipe joints. Assuming that the transverse PGD is in the form of a sine wave, then the axial displacement that the joint must be able to accommodate may be taken as follows.

For $0.3 < D/\delta < 4$:

$$\Delta_{joint} = \frac{\delta^2}{W^2}\left[\frac{2D}{\delta}\right]\pi^2 L_p \qquad \text{[Eq 7-16]}$$

Otherwise:

$$\Delta_{joint} = \frac{\delta^2}{W^2}\left[1+\left(\frac{D}{\delta}\right)^2\right]\frac{\pi^2 L_p}{2} \qquad \text{[Eq 7-17]}$$

where, D = pipe diameter, L_p is the pipe segment length, and Δ_{joint} is the maximum joint opening displacement. In lieu of specific knowledge about the particular site, the commentary contains suggested values for δ and *W*.

7.3.3 Analysis for Fault Crossing Ground Displacement Hazard

Earthquake fault movements are assumed to occur in relatively narrow fault zones characterized by permanent horizontal and vertical offset as one soil mass moves relative to the other. This can be very damaging to buried pipes spanning the fault that experience relative applied deformations. The pipe response depends on its orientation relative to the fault and the amount and spatial variation of fault PGD.

Continuous Pipe

A continuous pipe will experience plastic deformations in most actual fault crossing situations. Therefore, at a minimum, the pipe must be ductile and the joints capable of developing the strength of the pipe.

The average pipe strain may be easily (but not rigorously) estimated as follows if the fault offset results in net tension in the pipe:

$$\varepsilon_{pipe} = 2\left[\frac{\delta}{2L_a}Cos\beta + \frac{1}{2}\left(\frac{\delta}{2L_a}Sin\beta\right)^2\right] \qquad \text{[Eq 7-18]}$$

where, β = the acute angle (≤ 90 degrees, =90 degrees when the pipe alignment is perpendicular to the fault offset) between the pipe run and line of ground rupture, and L_a = the effective unanchored pipe length, that is, the distance between the fault trace and an anchor point, see Figure 7-8.

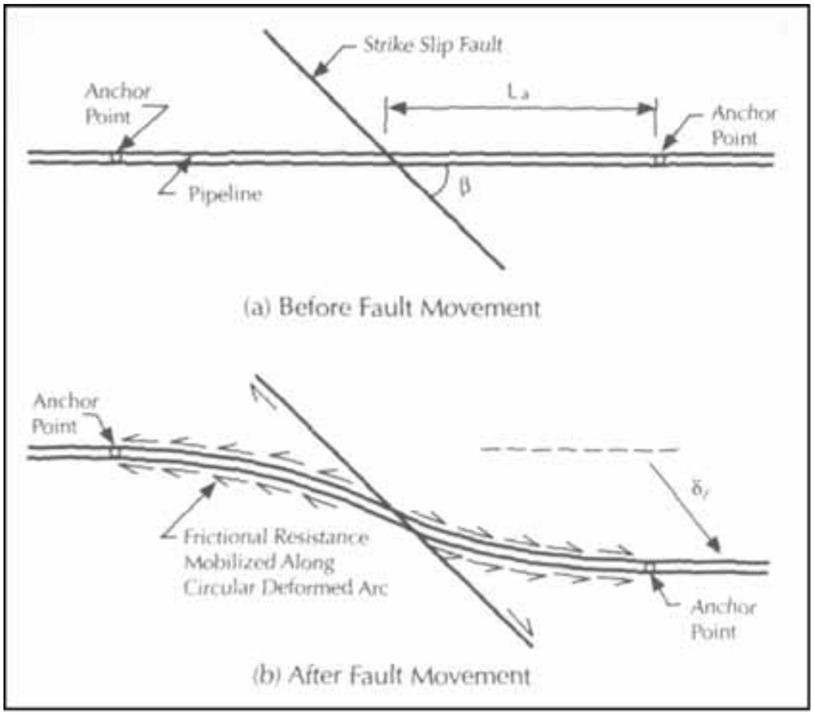

Figure 7-8. Plan View of Pipeline Equation [7-18]

When no bends, tie-ins or other constraints are located near the fault, then the axial resistance is provided by the soil-pipe friction and the effective unanchored pipe length may be taken as follows.

$$L_a = \frac{P_y}{t_u} + \frac{P - P_y}{t_u} \qquad \text{[Eq 7-19]}$$

where, P_y = yield strength of pipe in tension and P = actual tensile force in the pipe at the fault crossing (requires iteration). The above is the Newmark-Hall method, but scaled higher by a factor of 2 to reflect unconservatisms in the Newmark-Hall analogy (see commentary). Even with this factor of 2, this model may overestimate pipe capacity to withstand fault offset, and in general should only be used as a first order approximation; FEM methods are recommended to be used for important (Function Class III or IV) pipelines.

In general, plastic design should be used for fault offset loading. Pipe material must be ductile and capable of plastic strains exceeded the computed average pipe strain without fracture. The joints should be capable of developing the strength of the adjoining pipe segments (e.g., steel pipe having double lap welded or butt welded joints may be acceptable, but single lap welded joints or riveted joints are generally not satisfactory to allow for ductile behavior of the pipe).

Segmented Pipe

The fault offset is assumed to be accommodated equally by the pipe joints located immediately on each side of the line of ground rupture (faulting crosses the pipe barrel and not at the joint). In this case, each joint is subjected to an axial displacement and angular rotation that may be calculated as follows.

$$\Delta_{joint} = \frac{\delta}{2}\cos\beta \qquad \text{[Eq 7-20]}$$

$$\gamma_{joint} = Arc\sin\left(\frac{\delta}{L_p}\sin\beta\right) \qquad \text{[Eq 7-21]}$$

The shear force and moment in the pipe barrel that crosses the line of ground rupture may be calculated as follows (assuming fault crosses at midpoint of pipe segment).

$$V = \frac{p_u L_p}{4} \qquad \text{[Eq 7-22]}$$

$$M = \frac{p_u L_p^2}{32} \qquad \text{[Eq 7-23]}$$

where, p_u = ultimate lateral bearing force of soil acting on pipe barrel.

7.4 Finite Element Method

The Finite Element Method (FEM) can be used for the analysis and design of any pipeline. With the advent of low cost high-powered personal computers, the use of the FEM method can be adopted for many pipeline applications, covering ground shaking, liquefaction, landslide and surface faulting hazards. In practice, it will normally be used for the most important pipelines (Function Class III and IV) subject to PGD.

Figure 7-9 shows the typical form of the FEM. The pipe can be modeled with beam-type (or pressurized pipe) line elements. Near the fault offset, the length of the beam/pipe elements should not be longer than the pipe diameter. The model should accommodate both material and geometry (large deformation) nonlinearities. (In a more generalized case, axisymmetric and shell-type elements could be used to examine special fittings and other factors. However, this latter case is not covered here.) The soil is modeled by lateral and axial springs having the ability to mimic the nonlinear soil force-deformation behaviors. The loading, usually PGD, is modeled by displacements applied to the ends of the soil springs to simulate the soil-pipe interaction.

The normally reported stresses and strains from a beam-type FEM are the longitudinal-direction actions. The user should be cautioned that these stresses and strains are not the same as the stresses and strains in the pipe wall once the wall begins to substantially distort (such as in wrinkling). Distinction as to the allowable strains in a pipe must be made, when such strains are calculated using beam-element or pipe-element type models, or when such strains are calculated with consideration of localized bending, etc. at wrinkled locations. In these Guidelines, unless otherwise noted, all compressive strains are reported as allowable compressive strains in the main body of the pipe near but not in the wrinkle, recognizing that the strain in the wrinkling joint will be higher.

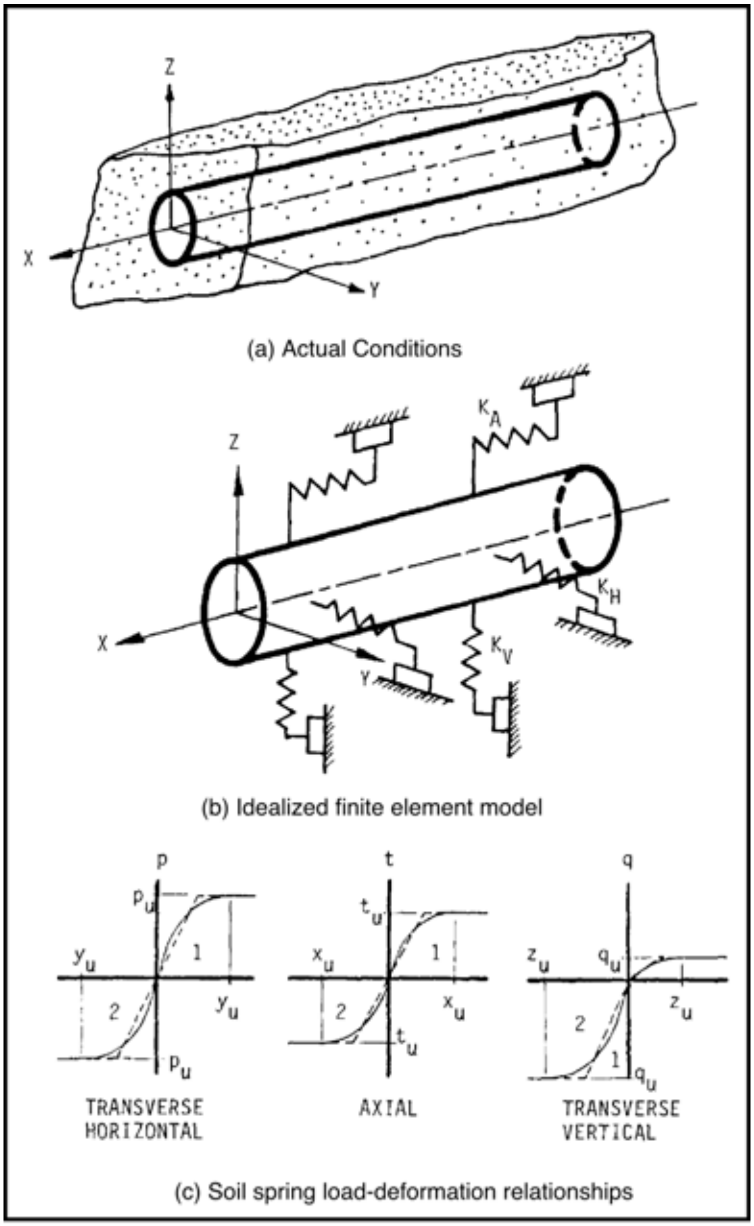

Figure 7-9. Finite Element Model (Beam Type) of Buried Pipeline and Soil Loads and Restraint

7.4.1 Pipe Modeling Guidelines

For models of the type in Figure 7-9(a), (b), a low pressure water pipe can be reasonably modeled using beam elements (effects of internal pressure ignored) or pipe elements (effects of internal pressure included). The typically reported stresses and strains from such elements are the longitudinal-direction actions. These stresses and strains are not the same as the stresses and strains in the pipe wall once the wall begins to substantially distort (such as when the pipe wall undergoes local buckling, often referred to as "wrinkling"). Hence, the results from such elements must be evaluated with consistent compressive wrinkling and tensile strain limits criteria presented in sections 7.4.3 and 7.4.4.

Depending on the pipeline geometry and loading, the model may need to include three dimensions. Pipe elements should be discretized at relatively short lengths near the transition point where the PGDs occurs (e.g., proximate to the fault). For example, a pipe could be discretized at one-fifth diameter intervals, for ten pipe diameters either side of the imposed PGD. This will generally provide adequate capability to capture the localized peak bending gradient in the pipeline. At locations distant from the imposed PGD transition point, the pipe element lengths could be up to 5 times the pipe diameter, without loss of accuracy.

The material properties of the pipeline should be set to capture the nonlinear capability of the pipe material. Tests of actual steel from water pipelines suggests that the flat yield plateau exhibited by virgin A36 steel might not be present, in part because of the rolling involved in the original pipe manufacture; and for high strain rate applications like many types of seismic loading, the flattened plateau might not be present.

7.4.2 Soil Modeling Guidelines

For cases where there are imposed PGDs on the pipeline (such as at fault crossings, landslide transition points, etc.), it would be expected that the pipe will slip through the soil. Thus, the soil load-deflection curves (Figure 7-9(c)) will need to be nonlinear. The following outlines the usual formulation for soil springs; it is suitable for the engineer to modify these formulations to reflect actual field conditions, whether from test, experience, judgment or analysis.

The following are the soil springs including example values assuming a 42-inch inside diameter butt welded steel pipeline, with wall t = 0.5 inches, in a firm clay type backfill, for application in a fault crossing situation. The inner lining and outer coating in the fault crossing area are assumed to be fusion bonded epoxy. In the formulations below, it is assumed that undrained conditions prevail for clays while drained conditions prevail in sands.

Axial Spring (t-x curve)

$$t_u = \begin{cases} \pi D \alpha S_u \text{ for clay} \\ \dfrac{\pi D}{2} \bar{\gamma} H (1 + K_o) \tan k\phi \text{ for sand} \end{cases} \qquad \text{[Eq 7-24]}$$

$$x_u = \begin{cases} 0.1 \text{ to } 0.2 \text{ inches for dense to loose sand} \\ 0.2 \text{ to } 0.4 \text{ inches for stiff to soft clay} \end{cases}$$

These soil springs are inferred from pile shaft load transfer theory, where, t_u = maximum soil resistance to the pipe axial direction having units of force per unit length of pipe, x_u = axial displacement at which maximum soil resistance is developed, D = pipe outer diameter, α = adhesion factor from Figure 7-10, S_u = soil undrained shear strength, $\bar{\gamma}$ = soil effective unit weight, H = soil depth to centerline of pipe, K_o = coefficient of lateral soil pressure at rest, ϕ = angle of soil shear resistance, k is a factor to represent the friction between the outer surface of the pipe and the surrounding soil (if that is the failure plane), such that ($\tan k\phi$) is in the range of about 0.6 to 0.7 for concrete coated steel pipe in compacted sand; or 0.4 to 0.5 for hard epoxy coated steel pipe in compacted sand.

For design purposes, variation in t_u should be considered, at least -33% / +50%, to consider the range of soil properties on the impact of pipe strain and other forces. The coefficient of soil pressure may be substantially higher in zones of large relative displacement between the pipeline and the soil. Lower bound values tend to result in lower stresses and strains in the pipe and increase the length of pipeline needed to transfer pipeline forces to the soil. Upper bound values tend to increase the stresses and strain in the pipe and reduce the length of pipeline needed to transfer the pipeline forces to the soil.

Example. The following illustrate the soil spring formulation for a 42 inch inside diameter steel pipeline (wall thickness of 0.5 inches) in a firm clay type backfill, for application in a fault crossing situation. Assume S_u = 2,000 psf, and $\alpha = 0.5$, the empirical adhesion factor coefficient.

$t_u = \pi * D_{outside} * \alpha * S_u = \pi * 43 * 0.5 * (2{,}000\, psf\ /144) = 938$ pounds per inch of pipe length
$x_u = 0.30$ inches

Transverse (Horizontal) Spring (p-y curve)

$$p_u = \begin{cases} S_u N_{ch} D \text{ for clay} \\ \bar{\gamma} H N_{qh} D \text{ for sand} \end{cases} \qquad \text{[Eq 7-25]}$$

$$y_u = \begin{cases} 0.07 \text{ to } 0.10(\mathrm{H} + \mathrm{D}/2) \text{ for loose sand} \\ 0.03 \text{ to } 0.05(\mathrm{H} + \mathrm{D}/2) \text{ for medium sand} \\ 0.02 \text{ to } 0.03(\mathrm{H} + \mathrm{D}/2) \text{ for dense sand} \\ 0.03 \text{ to } 0.05(\mathrm{H} + \mathrm{D}/2) \text{ for stiff to soft clay} \end{cases}$$

These soil springs are inferred from footing and vertical anchor plate pull-out capacity theory and laboratory tests on model pipelines simulating horizontal pipe movements, where, p_u = maximum soil resistance to the pipe transverse (horizontal) direction having units of force per unit length of pipe, y_u = transverse displacement at which maximum soil resistance is developed, N_{qh} and N_{ch} are coefficients from Figures 7-11 and 7-12.

Example. The bearing factor, N_{ch} is taken as 5.5. The depth from the soil surface to the springline of the pipe is 5.75 feet (4 feet of cover in this case).

$$p_u = S_u * N_{ch} * D_{outside} = (2{,}000/144) * 5.5 * 43) = 3{,}284 \text{ pounds per inch of pipe length}$$

$$y_u = 0.03 * \left(H + D_{outside}/2 \right) = 0.03 * \left(5.75 + 43/12*2\right) * 12 = 2.72 \text{ inches}$$

Transverse (Vertical Downwards) Spring (q-z curve)

$$q_u = \begin{cases} S_u N_c D \text{ for clay} \\ \bar{\gamma} H N_q D + 1/2\, \gamma \mathrm{D}^2 \mathrm{N}\gamma \text{ for sand} \end{cases} \qquad \text{[Eq 7-26]}$$

$$z_u = 0.10D \text{ to } 0.15D \text{ for both sand and clay}$$

where, γ = total unit weight of sand. These soil springs are inferred from bearing capacity theory for footings, where, q_u = maximum soil resistance to the pipe transverse (vertical downwards) direction having units of force per unit length of pipe, z_u = transverse displacement at which maximum soil resistance is developed, and N_c, N_q, N_y = coefficients from Figure 7-13.

Example. The downward bearing factor, N_c is taken as 20.

$$q_u = S_u * N_c * D_{outside} = (2{,}000/144) * 20 * 43) = 11{,}944 \text{ pounds per inch of pipe length}$$

$$z_u = 6.0 \text{ inches}$$

Transverse (Vertical Upwards) Spring (q-z curve)

$$q_u = \begin{cases} S_u N_{cv} D \text{ for clay} \\ \bar{\gamma} H N_{qv} D \text{ for sand} \end{cases} \qquad \text{[Eq 7-27]}$$

$$z_u = \begin{cases} 0.01H \text{ to } 0.015\text{H for dense to loose sand} \\ 0.1H \text{ to } 0.2H \text{ for stiff to soft clay} \end{cases}$$

These soil springs are from pull-out capacity theory and laboratory tests on anchor plates and model buried pipes, where, q_u = maximum soil resistance to the pipe transverse (vertical upwards) direction having units of force per unit length of pipe, z_u = transverse displacement at which maximum soil resistance is developed, N_{cv} = coefficient from Figure 7-15, and N_{qv} = coefficient from Figure 7-14. Example. The bearing factor, N_{cv} is taken as 2.75.

$q_u = S_u * N_{cv} * D_{outside} = (2,000/144) * 2.75 * 43) = 1,642$ pounds per inch of pipe length, upwards direction

$z_u = 0.1 * (H) = 0.1 * (5.75) * 12 = 6.90$ inches

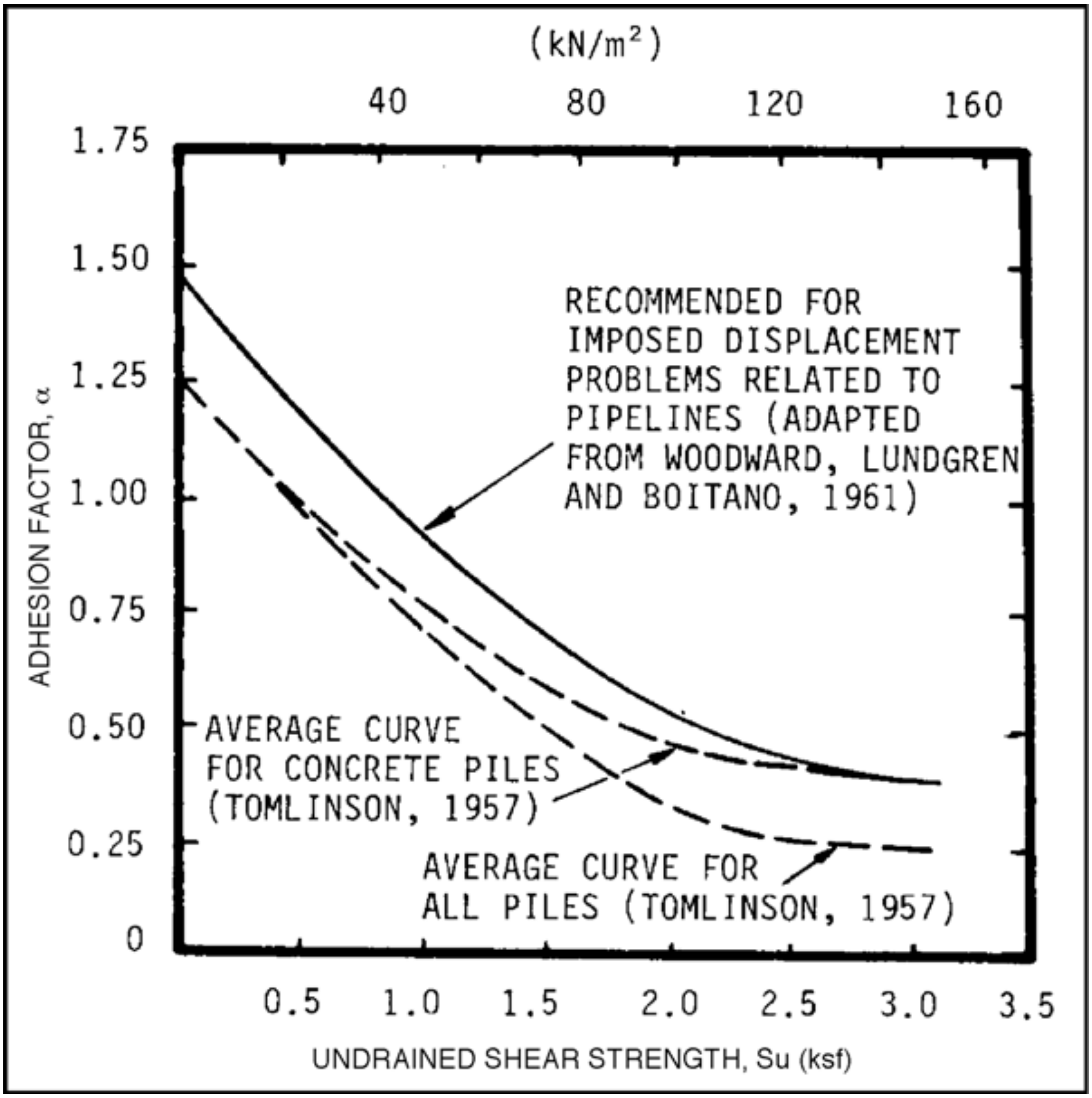

Figure 7-10. Adhesion Factors Versus Undrained Shear Strength (ASCE 1984)

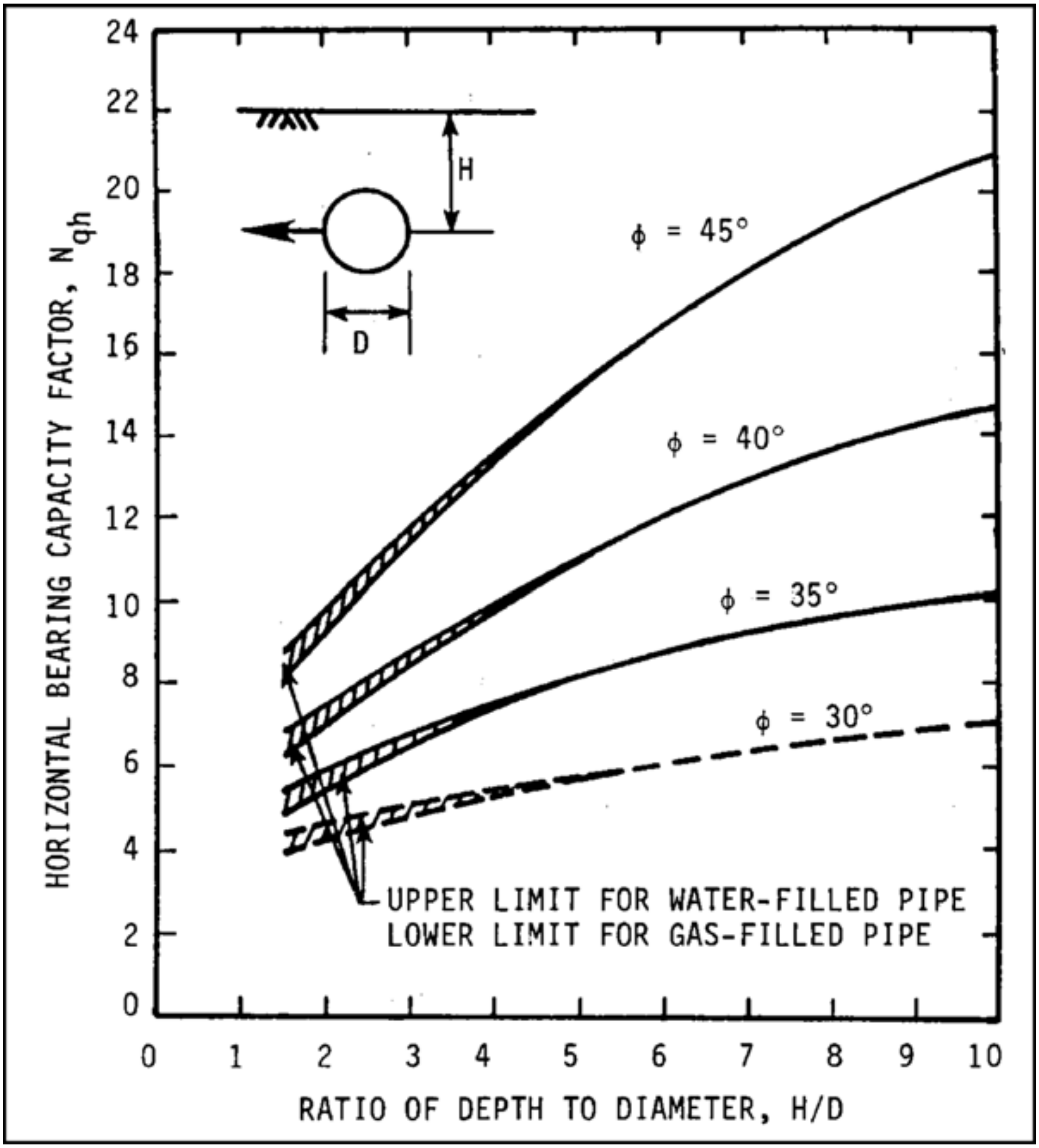

Figure 7-11. Horizontal Bearing Capacity Factor for Sand as a Function of Depth to Diameter Ratio of Buried Pipelines (ASCE 1984)

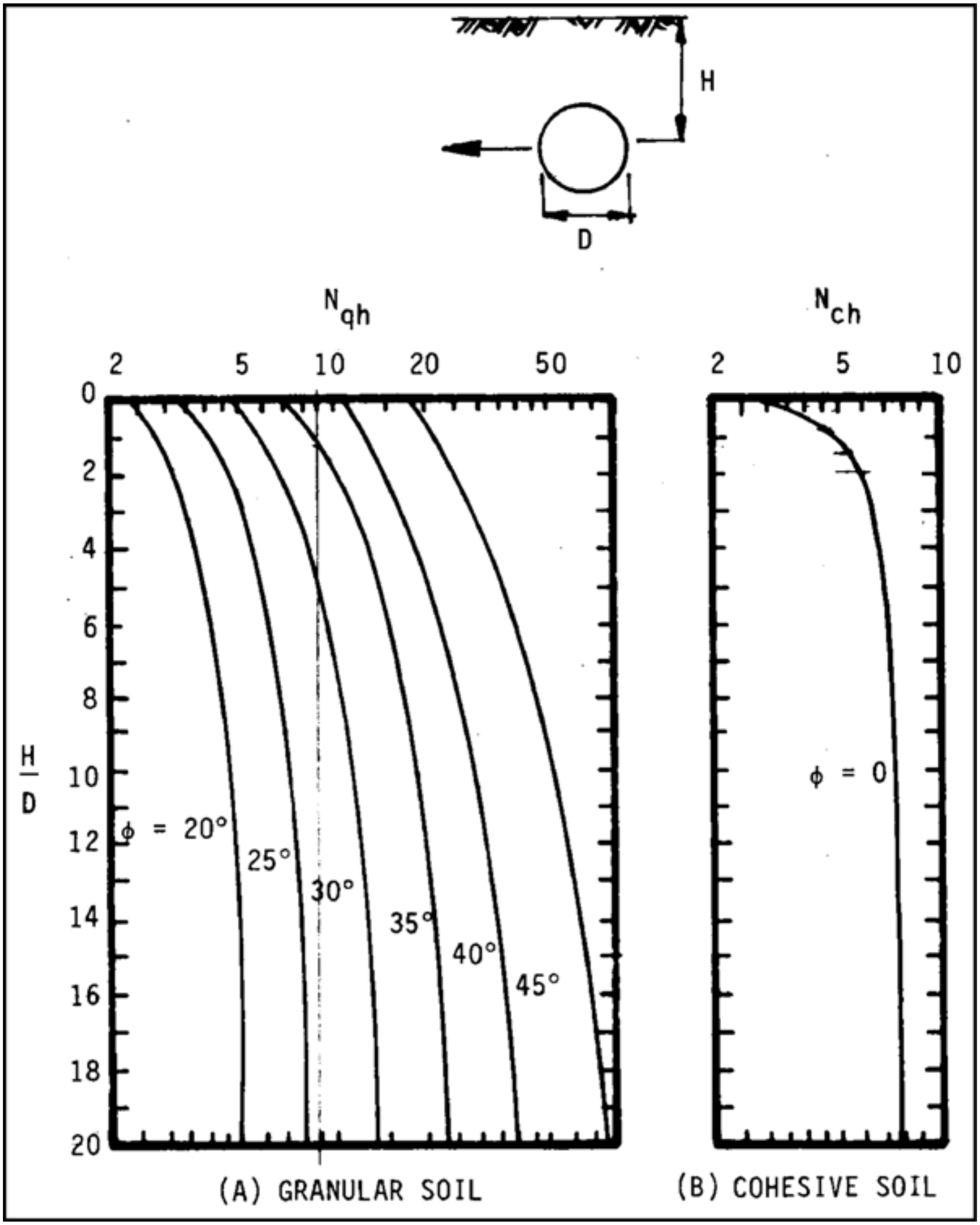

Figure 7-12. Horizontal Bearing Capacity Factor for Sand as a Function of Depth to Diameter Ratio of Buried Pipelines (ASCE 1984)

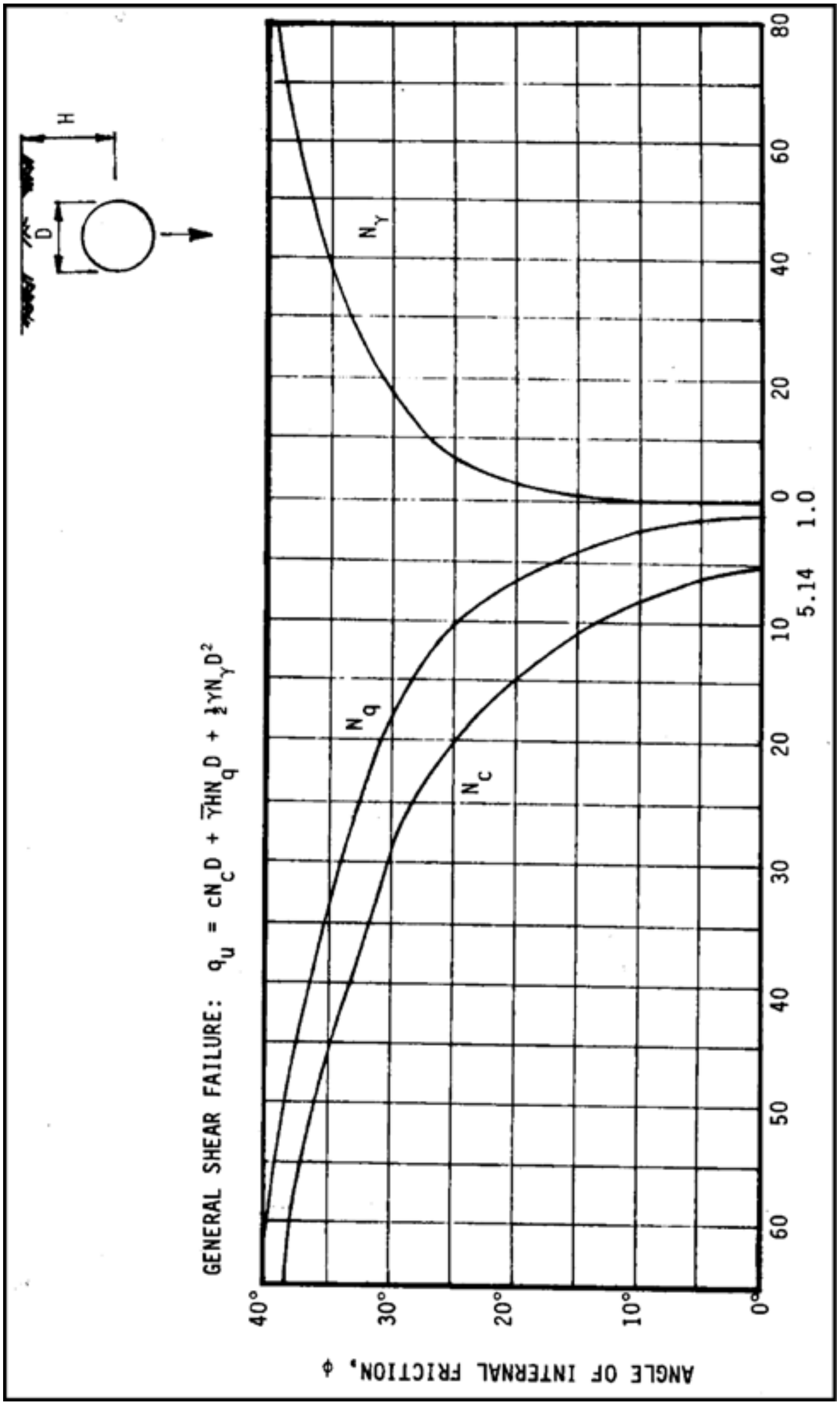

Figure 7-13. Vertical Bearing Capacity Factors vs. Soil Angle of Internal Friction (ASCE 1984)

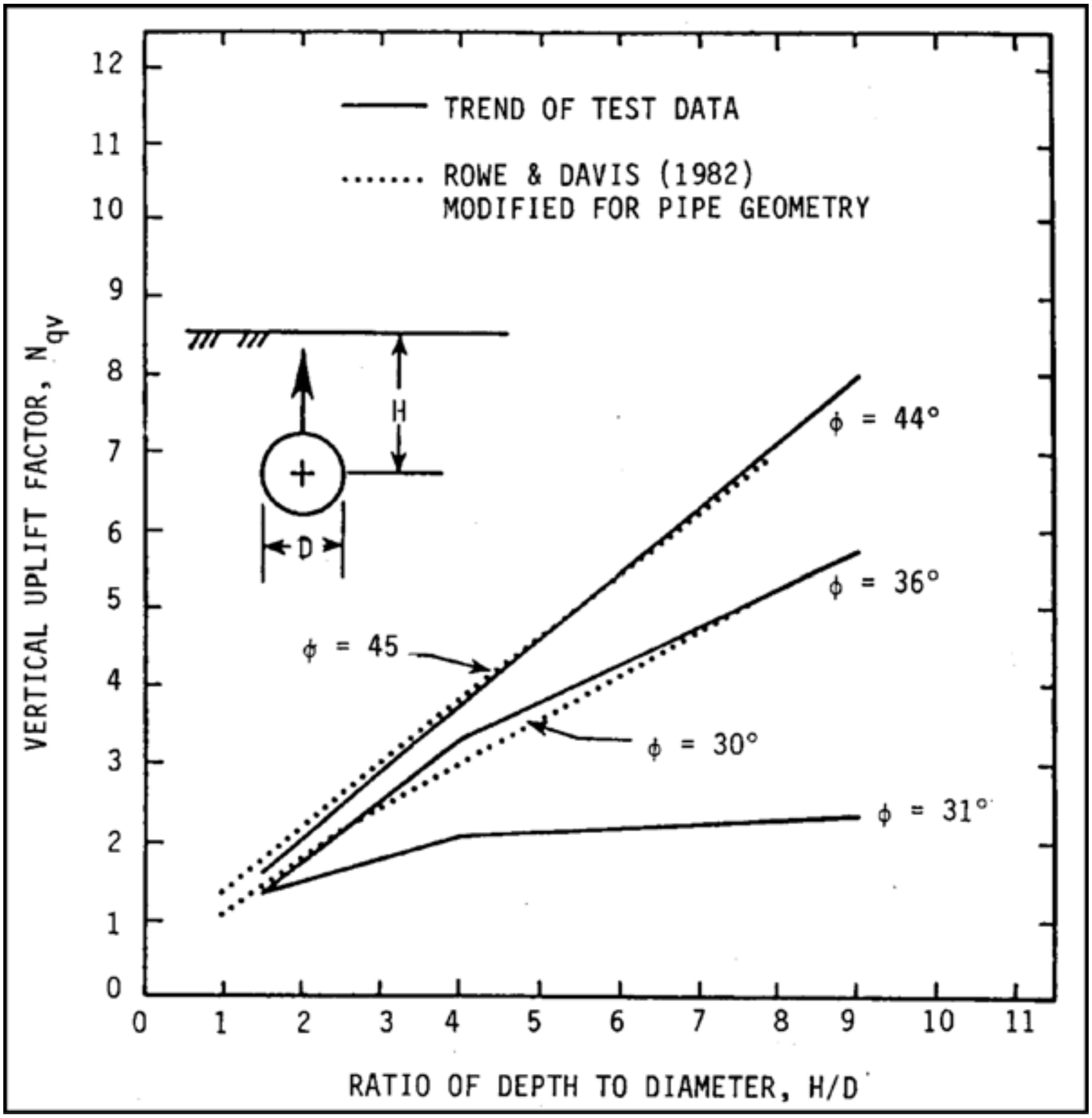

Figure 7-14. Vertical Uplift Capacity Factor for Sand as a Function of Depth to Diameter Ratio for Buried Pipelines (ASCE 1984)

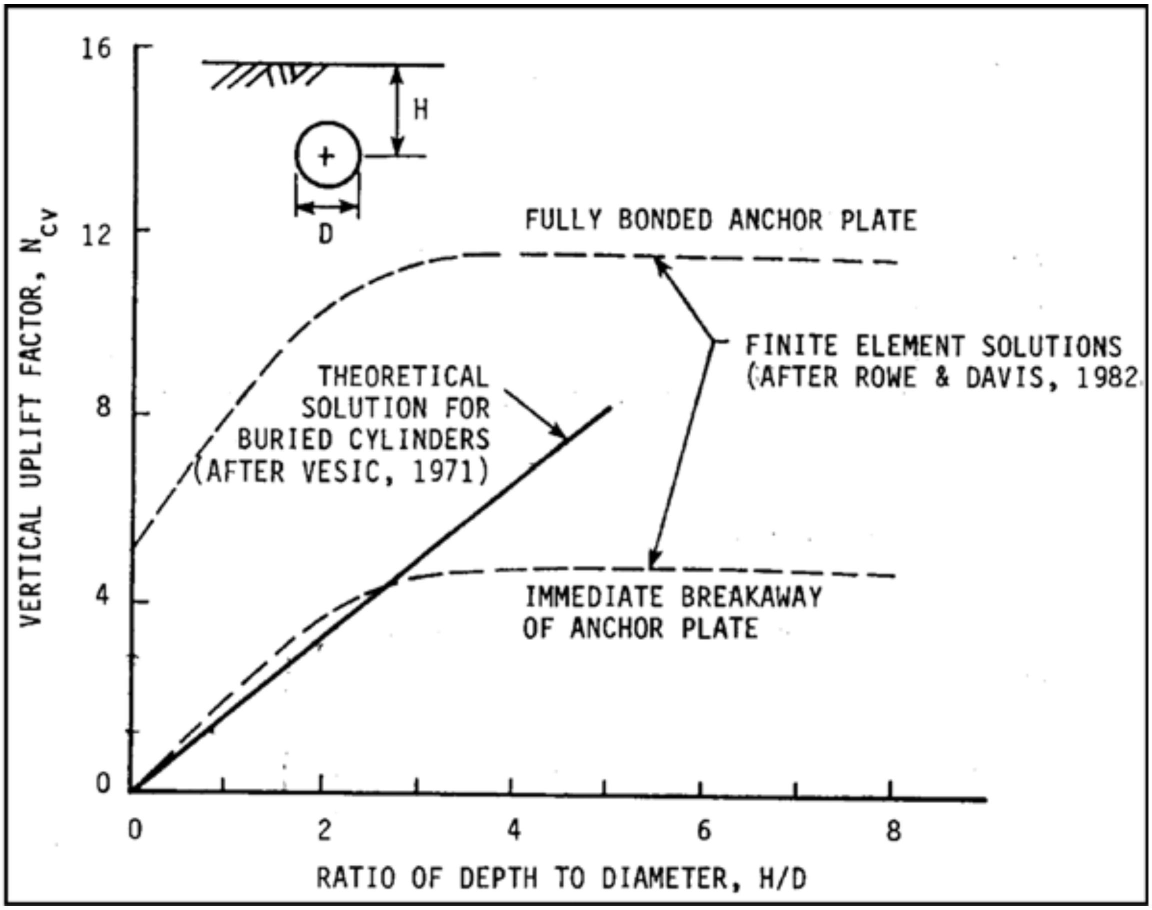

Figure 7-15. Vertical Uplift Capacity Factor for Clay as a Function of Depth to Diameter Ratio for Buried Pipelines (ASCE 1984)

7.4.3 Wrinkling Limit

The theoretical onset of compressive buckling in a thin-walled cylinder (not including lap joints) is between one-third to one-fourth of the theoretical value of:

$$\varepsilon_{theory} = 0.6\frac{t}{R} \qquad \text{[Eq 7-28]}$$

where t = pipe wall thickness, and R = pipe radius. This is derived from the classical buckling stress of a perfect cylinder (Timoshenko and Gere) of:

$$\sigma_{classical} = \frac{1}{\sqrt{3\left(1-\mu^2\right)}}\frac{tE}{R} \qquad \text{[Eq 7-29]}$$

where μ is Poisson's ratio and E is Young's modulus.

A conservative estimate of the onset of local buckling in a butt welded pipe is:

$$\varepsilon_{onset} = 0.175\frac{t}{R} \text{ to } 0.2\frac{t}{R} \qquad \text{[Eq 7-30]}$$

Once local buckling (wrinkling) starts, there is usually a 50% to 500% increase in capability before the pipe wrinkles sufficiently to initiate a through wall crack. Recent tests of a 30" diameter, t=0.327" (D/t=92) pipe with Fy=70 ksi, DelCol (1998) showed that for internal pressures in the range of 0 psi to 312 psi for that pipe, the initial buckle formed at an average compressive strain of about -0.5%, which corresponds to 0.229 t/R. For an unpressurized pipe, average compressive strains over one pipe diameter length, at the wrinkle, reached 3.5%, without breach of the pressure boundary.

Once a wrinkle forms, additional shortening of the pipeline will tend to accumulate at the wrinkle.

Onset of wrinkling might be a suitable design allowable for a high pressure gas pipe, or oil pipe, where wrinkling of the pipe may restrict the passage of pigs; or failure of the pipe might result in fire or other serious consequences to nearby facilities and habitat. However, with recognition that for water pipes that the wrinkling limit in Equation [7-30] is conservative, and with recognition that it is rare that release of water poses serious consequences to the nearby environment, then some post-wrinkling performance may be acceptable; so the more relaxed compression limits in Equations [7-31 and 7-32] are considered suitable for design. Under wave propagation, peak longitudinal compressive strains in the pipe should be lower than the onset of significant wrinkling, equation [7-31]. Equation [7-32] implies that post-earthquake inspection and possible subsequent repair may be needed. Under fault offset or other limited area PGD loading, peak compressive longitudinal strains should be kept below equation [7-32] if D/t is ≤ 100.

$$\varepsilon_c^{wave\ \ passage} = 0.75\left[0.50\frac{t}{D'} - 0.0025 + 3000\left(\frac{pD}{2Et}\right)^2\right] \qquad \text{[Eq 7-31]}$$

$$D' = \frac{D}{1 - \frac{3}{D}\left(D - D_{\min}\right)}$$

$$\varepsilon_c^{PGD} = 0.88\frac{t}{R} \qquad \text{[Eq 7-32]}$$

where D is pipe outside diameter.

Example. Assume a 96-inch inside diameter butt welded steel pipe with t = 0.75 inches. The nominal onset of compressive wrinkling (0.175t/R) is -0.27%. Assuming that Dmin is 95 inches (2.5 inch out of roundness), and an internal pressure of 150 psi, equation [7-31] gives the allowable strain at -0.10%. For fault offset, equation [7-32] gives the allowable strain at -1.35%. Equation [7-32] allows for post-wrinkling behavior, and assumes that this is acceptable to the owner.

For compressive strains higher than -5% (when measured ignoring wrinkle geometry), tears in the pipe should be expected. For most water pipelines at moderate temperatures (over 40°F), the tear length has not been observed to propagate, with a resulting leak. Tear openings have been observed as about 0.25 inches wide x 12 inches long (36-inch diameter pipe with double lap weld impacted by fault creep), resulting in leak rates on the order of a 1,000 gpm to 2,000 gpm.

The above equations do not apply for single or double lap welded pipes, where the onset of wrinkling occurs at lower forces owing to the major geometric discontinuity at the joint. For double lap welded pipes, the longitudinal compressive stress in the main pipe should be kept to 0.60 Fy to prevent wrinkling; or the peak bending strain within the wrinkled joint kept below 5% when considering joint geometry.

For single lap welded pipes, the longitudinal compressive stress in the main pipe should be kept to 0.40 Fy to prevent wrinkling; or the peak bending strain within the wrinkled joint kept below 5% when considering joint geometry.

In all cases where yielding of the steel is allowed, the weld consumables, welding procedures and inspection criteria should be suitable to ensure development of gross section yielding of the pipe section both for field girth joints and shop fabricated longitudinal spiral or straight seam joints.

7.4.4 Tensile Strain Limit

The longitudinal strain in a butt welded steel pipe should be limited to a level to achieve the target performance level of the pipeline. For offset displacements which are defined as having about a 16% chance of exceedance given the design basis earthquake (or 2 * AD if using Table 4-6) , maximum tensile longitudinal strains should be kept to about 0.25 times ultimate uniform strain (strain before necking) of the steel, or no more than 5%. This design limit provides for some capacity to withstand larger fault offset, or to accommodate minor flaws in the pipe and girth joint.

Should double lap welded steel pipe be used, then the maximum longitudinal strain in the pipe must be kept low enough such that there is a reasonable chance of survival of the joint. Test data on double lap welded joints suggests that perhaps one quarter of the joints will break when the strain in the pipe away from the joint reaches about 8%. This suggests that the maximum allowable strain in the main body of the pipe should be kept to 2%, or perhaps no more than 4% to have a reasonable chance of maintaining the pressure boundary. At 2% strain, the reliability of a double lap welded pipe will be similar to a similar quality butt welded pipe at 5% strain.

The girth joints in single lap welded steel pipe will generally not be strong enough to allow longitudinal tensile yielding in the main pipe (see Section 7.3.1).