IITK-GSDMA GUIDELINES
for SEISMIC DESIGN
of BURIED PIPELINES
Provisions with Commentary and Explanatory Examples

Indian Institute of Technology Kanpur

Gujarat State Disaster Management Authority

November 2007

NATIONAL INFORMATION CENTER OF EARTHQUAKE ENGINEERING

Other IITK-GSDMA Guidelines Available from NICEE:

- IITK-GSDMA Guidelines for Seismic Design of Liquid Storage Tanks
- IITK-GSDMA Guidelines for Structural Use of Reinforced Masonry
- IITK-GSDMA Guidelines for Seismic Design of Earth Dams and Embankments
- IITK-GSDMA Guidelines for Seismic Evaluation and Strengthening of Existing Buildings
- IITK-GSDMA Guidelines on measures to Mitigate Effects of Terrorist Attacks on Buildings

Please see back cover for current list of NICEE publications available for distribution.

IITK-GSDMA GUIDELINES for SEISMIC DESIGN of BURIED PIPELINES

Provisions with Commentary and Explanatory Examples

Prepared by:
Indian Institute of Technology Kanpur
Kanpur

With Funding by:
Gujarat State Disaster Management Authority
Gandhinagar

November 2007

NATIONAL INFORMATION CENTER OF EARTHQUAKE ENGINEERING
Indian Institute of Technology Kanpur, Kanpur (India)

Preparation of this document was supported by the Gujarat State Disaster Management Authority (GSDMA), Gandhinagar, through a project at Indian Institute of Technology Kanpur, using World Bank finances. The views and opinions expressed in this document are those of the authors and not necessarily of the GSDMA, the World Bank, or IIT Kanpur.

Copies of this publication can be requested from:

Coordinator
National Information Center of Earthquake Engineering
Indian Institute of Technology Kanpur
Kanpur 208 016 (India)
Email: nicee@iitk.ac.in
Website: www.nicee.org

ISBN 81-904190-7-2

Participants

Prepared by:

Suresh R. Dash, Indian Institute of Technology Kanpur, India

Sudhir K. Jain, Indian Institute of Technology Kanpur, India

Reviewed by:

John Eidinger, G & E Engineering Systems Inc., Oakland, California, USA

A. P. Sukumar, Structural Engineer, Engineering & Construction Dept., GVRD, Burnaby, BC, Canada

Paul Henderson, Geotechnical Engineer Engineering & Construction Dept., GVRD, Burnaby, BC, Canada

Debasis Roy, Assistant Professor, Indian Institute of Technology Kharagpur, India

O. R. Jaiswal, Assistant. Professor, Visvesvaraya National Institute of Technology, Nagpur India

GSDMA Review Committee:

V. Thiruppugazh, GSDMA, Gandhinagar

Principal Secretary, UDD, Gandhinagar

Sr. Town Planner, Gandhinagar

Secretary, Roads and Buildings, Gandhinagar

A. S. Arya, Ministry of Home Affairs, New Delhi

Alpa Sheth, Vakil Mehta Sheth Consulting Engineers, Mumbai

FOREWORD

The earthquake of 26 January 2001 in Gujarat was unprecedented not only for the state of Gujarat but for the entire country in terms of the damages and the casualties. As the state came out of the shock, literally and otherwise, the public learnt for the first time that the scale of disaster could have been far lower had the constructions in the region complied with the codes of practice for earthquake prone regions. Naturally, as Gujarat began to rebuild the houses, infrastructure and the lives of the affected people, it gave due priority to the issues of code compliance for new constructions.

Seismic activity prone countries across the world rely on "codes of practice" to mandate that all constructions fulfill at least a minimum level of safety requirements against future earthquakes. As the subject of earthquake engineering has evolved over the years, the codes have continued to grow more sophisticated. It was soon realized in Gujarat that for proper understanding and implementation, the codes must be supported with commentaries and explanatory handbooks. This will help the practicing engineers understand the background of the codal provisions and ensure correct interpretation and implementation. Considering that such commentaries and handbooks were missing for the Indian codes, GSDMA decided to take this up as a priority item and awarded a project to the Indian Institute of Technology Kanpur for the same. The project also included work on codes for wind loads (including cyclones), fires, and terrorism considering importance of these hazards. Also, wherever necessary, substantial work was undertaken to develop drafts for revision of codes, and for development of entirely new draft codes. The entire project is described elsewhere in detail.

The Gujarat State Disaster Management Authority Gandhinagar and the Indian Institute of Technology Kanpur are happy to present the *IITK-GSDMA Guidelines for Seismic Design of Buried Pipelines* to the professional engineering and architectural community in the country. It is hoped that the document will be useful in developing a better understanding of the design methodologies for earthquake-resistant structures, and in improving our codes of practice.

GSDMA, Gandhinagar
IIT Kanpur

PREFACE

Lifeline systems in the civil engineering context include those facilities that address societal needs of energy (electricity, gas, liquid fuel, steam, etc.), water (potable, sewage and solid waste, flood, etc.), transportation (highways, bridges, harbors, transit, etc.) and communications (telephone, telegraph, radio, television, telecommunication, mail, press, etc.). The well being of a community requires that these lifeline systems continue to function even after damaging earthquakes. Pipelines carry materials essential to the functioning and support of day-to-day life and maintenance of property and hence are often referred to as "lifelines". These are commonly used in industries, public supplies, and for transportation of oil, gas, water and many other fluids and goods. Among the pipelines, important pipelines are generally buried below ground for aesthetic, safety, economic and environmental reasons. Experiences from past earthquakes show that pipelines are highly vulnerable to earthquake shaking. Pipeline systems are generally spread over a large geographical region and encounter a wide variety of seismic hazards and soil conditions.

In India, there is no specific standard or guideline which adequately deals with the seismic evaluation and design of buried pipeline systems. Most of the agencies are following different codal provisions and guidelines from different countries and some have developed their own standard of analysis and design for seismic effect. Compared to present international practice, seismic design of buried pipelines in India are highly inadequate. Hence, a guideline/standard for earthquake resistant design of buried pipelines is needed to ensure a uniform approach to earthquake resistant practices by all agencies in India.

In the above scenario, it was decided to develop the present document under the project "Review of Building Codes and Preparation of Commentary and Handbooks" assigned by the Gujarat State Disaster Management Authority, Gandhinagar, to the Indian Institute of Technology Kanpur. The provisions included here are based on many international and national codes, guidelines, and research documents. To facilitate understanding of the provisions, clause-by-clause commentary is also provided. Further, four explanatory solved examples are provided based on the provisions of these guidelines.

This document is prepared by a team consisting of Suresh R. Dash (Senior Project Associate) and Professor Sudhir K. Jain of Indian Institute of Technology Kanpur. Dr. John Eidinger (G&E Engineering Systems Inc., USA), Dr. A. P. Sukumar (Engineering & Constructions Dept., Canada), Dr. Paul Henderson (Engineering & Constructions Dept., Canada), Prof. Debasis Roy (Indian Institute of Technology Kharagpur), and Professor O. R. Jaiswal (Visvesvaraya National Institute of Technology Nagpur) reviewed the document and provided their valuable suggestions to improve the same. The document was also placed in the website of National Information Center of Earthquake Engineering (*www.nicee.org*) for comments by the interested professionals. Professor C. V. R. Murty and Professor D C Rai gave many valuable inputs during the preparation of the document.

It is hoped that the designers of pipelines will find the document useful. All suggestions and comments are welcome and should be sent to Professor Sudhir K. Jain, Department of Civil Engineering, Indian Institute of Technology Kanpur, Kanpur 208 016, E-mail: *skjain@iitk.ac.in*.

SUDHIR K. JAIN

Index

PART 1: Guidelines and Commentary

1 Introduction 1

2 General 6

2.1 Scope 6

2.2 Serviceability Requirement 6

2.3 Safety Requirement 6

2.4 Terms and Definitions 7

2.5 Symbols 10

2.6 Acronyms 16

3 General Principles and Design Criteria 17

3.1 General 17

3.1.1 Pipeline Information 17

3.1.2 Site Information 17

3.1.3 Seismic Hazard Information 18

3.2 Classification of Pipelines 18

3.3 Classification of Soil 19

3.4 Classification of Seismic Hazards 21

3.5 Design Seismic Hazard 21

3.5.1 Design Basis Seismic Hazard 21

3.5.2 Design Seismic Hazard 22

3.5.3 Ground Amplification Factor 23

3.5.4 PGA as per Seismic Zones 24

3.5.5 Determining PGV from PGA 24

3.6 General Seismic Design Considerations 25

3.7 Analysis Procedure 26

3.8 Initial Stresses in the Pipeline 27

3.8.1 *Internal Pressure* 27

3.8.2 *Temperature Change* 28

3.9 Allowable Strain for Continuous Pipeline 28

3.10 Allowable Joint Displacement for Segmented Pipeline 30

4 Design Criteria for Permanent Ground Deformation (PGD) 32

4.1 Longitudinal Permanent Ground Deformation 35

4.1.1 *Continuous Pipeline* 35

4.1.2 *Segmented Pipeline* 38

4.2 Transverse Permanent Ground Deformation 39

4.2.1 *Continuous Pipeline* 40
4.2.2 *Segmented Pipeline* 41
4.3 Landslide 42
5 Design Criteria for Buoyancy due to Liquefaction 43
5.1 Buoyant Force on Pipeline 43
5.2 Continuous Pipeline 45
5.3 Segmented Pipeline 45
6 Design Criteria for Fault Crossing 47
6.1 Quantification of Fault Displacement 48
6.2 Continuous Pipeline 51
6.3 Segmented Pipeline 52
6.4 Trench Profile for Pipeline in Fault Zone 52
6.5 Detailed Modeling 53
7 Design Criteria for Seismic Wave Propagation 54
7.1 Design Ground Motion 55
7.2 Continuous Pipeline 57
7.3 Segmented Pipeline 58
Annex-A Finite Element Modeling of Buried Pipeline 60
Annex-B Soil Spring Properties to Model Soil-Pipe Interaction 63

IITK-GSDMA GUIDELINES
for SEISMIC DESIGN
of BURIED PIPELINES
Provisions with Commentary and Explanatory Examples

PART 1: PROVISIONS AND COMMENTARY

PROVISIONS | COMMENTARY

1 – Introduction

1.1 –

Seismic design of buried pipeline has great importance in the field of lifeline engineering. The pipelines are usually buried below ground for economic, aesthetic, safety and environmental reasons. In certain circumstances it may be required to take those pipes above ground but this case is relatively uncommon. Generally the oil and gas pipelines are designed and constructed as continuous pipelines (clause 2.4.3), while water supply pipelines are constructed as segmented pipelines (clause 2.4.17).

1.2 –

Modern pipelines manufactured with ductile steel with full penetration butt welds at joints possess good ductility. It has been observed that the overall performance record of oil and gas pipeline systems in past earthquakes was relatively good. However, catastrophic failures did occur in many cases, particularly in areas of unstable soils. Failures have mostly been caused by large permanent soil displacements.

1.3 –

The pipelines can be designed and constructed to resist most of the earthquake hazards. In India, there are no specific standards or guidelines which adequately deal with the seismic evaluation and design of pipeline systems. This document is aimed at providing seismic design guidelines for continuous and segmented buried pipelines.

C-1.3 –

There are many varieties of pipes used in India. Generally cast iron pipes with bell and spigot joints or flanged joints are used for water pipelines, whereas concrete pipes are preferred for sewer pipelines. For oil and gas pipelines, steel pipes with welded joints are preferred. HDPE pipes are also in use in some special cases where ductility demand on the pipeline is high. These are used generally in oil & gas industry especially within refinery area. Concrete pipes with liners are laid at some selective locations where the soil is relatively wet or the area is susceptible to buoyancy due to water logging reason and to protect the pipeline at road/railway crossings.

1.4 –

In developing this document, assistance has been taken from the following publications:

1) ALA (2001), Guidelines for Design of

PROVISIONS

COMMENTARY

Buried Steel Pipes, A report by public-private partnership between American Society of Civil Engineers (ASCE) & Federal Emergency Management Agency (FEMA), American Lifelines Alliance (ALA), July 2001.

2) ALA (2005), Seismic Guidelines for Water Pipelines, A report by public-private partnership between Federal Emergency Management Agency (FEMA) and National Institute of Building Sciences (NIBS), American Lifelines Alliance (ALA), March 2005.

3) API (1990), Specification for Line Pipe, API Specification 5L, American Petroleum Institute (API), USA.

4) API (1996), Movement of In-Service Pipelines, API Recommended Practice 1117, Second Edition, American Petroleum Institute (API), USA, August 1996.

5) ASCE (1984), Guidelines for the Seismic Design of Oil and Gas Pipeline Systems, Committee on Gas and Liquid Fuel Lifelines, American Society of Civil Engineers (ASCE), US, Nyman, D. J. (Principal Investigator).

6) ASCE (1997), Guide to Post Earthquake Investigation of Lifelines, Technical Council on Lifeline Earthquake Engineering (TCLEE), Monograph No.11, American Society of Civil Engineering (ASCE), Schiff, A. J. (editor), July 1997.

7) ASCE (1999), Guidelines for the Seismic Upgrade of Water Transmission Facilities, J. Eidinger, E. Avila (Editors), ASCE, TCLEE Monograph No. 15, January 1999.

8) Ballantyne. D. (1992). “Thoughts on a Pipeline Design Standard Incorporating Countermeasures for Permanent Ground Deformation” Proc., 4th Japan-US Workshop on Earthquake Design of Lifeline Facilities and Countermeasures for Soil Liquefaction, Hamada, M., and O’Rourke, T. D. (editors.), National Council for Earthquake Engineering Research (NCEER), NCEER-92-0019,

PROVISIONS

COMMENTARY

August 1992.

9) Dikkers, R. J., Chung, R. M., Mohraz, B., Lew, H. S. and Wright, R. N. (1996), "NISTIR 5907: Workshop on Developing and Adopting Seismic Design and Construction Standards for Lifelines." Proc., National Institute of Standards and Technology (NIST), US Department of Commerce Technology Administration, October-1996.

10) Eurocode (2004), Eurocode-8: Design Provisions for Earthquake Resistant Of Structure, Part-4: Silos, Tanks and Pipelines, European Committee for Standard, Stage 34 Draft, December 2004.

11) FEMA–233 (1992), Earthquake Resistant Construction of Gas and Liquid Fuel Pipeline Systems Serving or Regulated by the Federal Government, Federal Emergency Management Agency (FEMA), Earthquake Hazard Reduction Series 67, July 1992.

12) FEMA-450 (2004), NEHRP Recommended Provisions for New Buildings and Other Structures, 2003 Edition, Building Seismic Safety Council (BSSC) for Federal Emergency Management Agency (FEMA), National Earthquake Hazard Reduction Program (NEHRP), National Institute of Building Sciences, Washington DC, US.

13) Hall, W., and Newmark, N. (1977), Seismic Design Criteria for Pipelines and Facilities, American Society of Civil Engineers (ASCE), A report on current state of knowledge of lifeline earthquake engineering, New York, US, pp-18-34.

14) IBC (2003), International Building Code (IBC), International Code Council (ICC), US.

15) IS-1893: part-1 (2002), Criteria for Earthquake Resistant Design of Structures, Part-1: General Provisions and Buildings, Bureau of Indian Standards, New Delhi, India, June 2002.

16) Iwamoto. T., Wakai N., and Yamaji T.

PROVISIONS **COMMENTARY**

(1984), "Observation of Dynamic Behaviour of Buried Ductile-Iron Pipelines during Earthquakes", Eighth World Conference on Earthquake Engineering, San Francisco, California, Volume-VII, pp 231-238.

17) JSCE (2000a), Basic Principles of Seismic Design and Construction for Water Supply Facilities, Earthquake Resistant Codes in Japan, Japan Society of Civil Engineers (JSCE), Japan Water Works Association, January 2000.

18) JSCE (2000b), Recommended Practices for Earthquake Resistant Design of Gas Pipelines, Earthquake Resistant Codes in Japan, Japan Society of Civil Engineering (JSCE), Japan Gas Association, January 2000.

19) Liu, X., and O'Rourke, M. J. (1997), "Behaviour of Continuous Pipelines Subject to Transverse PGD." Earthquake Engineering and Structural Dynamics, Vol-26, pp. 989-1003.

20) McGuire, R. K. (2004), Seismic Hazard and Risk Analysis, Earthquake Engineering Research Institute (EERI), Monograph No-10, California, US.

21) O'Rourke M. J., and Flores-Berrones, R. (1995), "Steel Pipe Wrinkling Due to Longitudinal Permanent Ground Deformation" Journal of Transportation Engineering, ASCE, Vol-121, No-5, September/October 1995.

22) O'Rourke, M. J., and Hmadi, K. EL. (1982), "Analysis of Continuous Buried Pipelines for Seismic Wave Effects" Earthquake Engineering and Structural Dynamics, vol.16, pp 917-929.

23) O'Rourke, M. J., and X. Liu (editors) MCEER (1999), Response of Buried Pipelines Subject to Earthquake Effects, Multidisciplinary center for Earthquake Engineering and Research (MCEER), MCEER Monograph No-3, US.

24) O'Rourke, M. J., Bloom, M. C., and Dobry, R. (1982), "Apparent Propagation Velocity of Body Waves" Earthquake

PROVISIONS

COMMENTARY

Engineering and Structural Dynamics, vol.10, pp 283-294.

25) PRCI, (2004), "Guidelines for the Seismic Design and Assessment of Natural Gas and Liquid Hydrocarbon Pipelines, Pipeline Design, Construction and Operations", Edited by Honegger, D. G., and Nyman D. J., Technical Committee of Pipeline Research Council International (PRCI) Inc, October 2004.

26) Ramberg, W., and Osgood, W R. (1943), Description of Stress-Strain Curves by Three Parameters, Technical Notes: National Advisory Committee for Aeronautics, Washington, July 1943.

27) Reiter, L. (1991), Earthquake Hazard Analysis: Issues and Insights, Columbia press, New York. US.

28) TCLEE (1999), Guidelines for the Seismic Evaluation and Upgrade of Water Transmission Facilities, Technical Council on Lifeline Earthquake Engineering (TCLEE), Monograph No-15, Eidinger, J., Avila, E., (editors), American Society of Civil Engineers (ASCE), January 1999.

29) Wells, D. L., and Coppersmith K. J. (1994), "New Empirical Relationships among Magnitude, Rupture Length, Rupture Width, Rupture Area, and Surface Displacement." Bulletin of the Seismological Society of America, Vol-84, No-4, pp. 974-1002, August 1994.

1.5 –

This document is developed as part of a project entitled "Review of Building Codes and Preparation of Commentary and Handbooks" awarded to Indian Institute of Technology Kanpur by Gujarat State Disaster Management Authority (GSDMA), Gandhinagar through World Bank finances.

PROVISIONS **COMMENTARY**

2 – General

2.1 – Scope

This document deals with the seismic design requirements for new continuous and segmented buried pipelines. It can also be used as a basis for evaluating the level of strengthening or increased redundancy needed by existing facilities to improve their response during seismic events.

C-2.1 –

This document covers design criteria for buried pipelines for various seismic hazards such as: wave propagation, fault crossing, and permanent ground deformation (PGD) due to liquefaction, lateral spreading, etc. Specific seismic hazards like tsunami, tectonic subsidence, uplift, etc., have not been considered in this document.

In this document, detailed design criteria of only iron and steel pipelines are discussed. However, pipelines of other materials may be considered in the way iron and steel pipelines are dealt with, except their stress-strain behaviour and allowable strain/deformation capacity. Specialized literature may be referred for the analysis and design of pipelines with various other implications and category (e.g., offshore buried pipelines, etc.).

The older pipelines, the pipelines which may not confirm to the capacity given in clause 3.9. It can be due to inadequate toughness and presence of corrosion or welding defects. Hence, strain acceptance criteria for older pipes need to be developed on case-by-case basis.

2.2 – Serviceability Requirements

Pipeline system shall be designed and constructed in such a way as to be able to maintain the supplying capability as much as possible, even under considerable local damage due to high intensity earthquakes.

2.3 – Safety Requirements

The location of the pipeline, size of the population that is exposed to the impact of pipeline rupture, and environmental damage due to the pipeline rupture shall be considered in establishing the level of acceptable risk while designing the pipeline system.

C-2.3 –

The safety requirements have been incorporated in the analysis and design by considering the importance factor as given in clause 3.5.1.

PROVISIONS

2.4 – Terms and Definitions

2.4.1 – Apparent Seismic Wave Propagation Velocity ($C_{apparent}$)

This is the propagation velocity of seismic wave with respect to the ground surface.

2.4.2 – Buoyancy Due to Liquefaction

When soil liquefies, it behaves like a thick fluid, and the pipe embedded in it will be subjected to the buoyant force from below. This is commonly referred as buoyancy due to liquefaction. This type of conditions mostly arises at river crossings or areas with high water table and sandy soil.

2.4.3 – Continuous Pipeline

Continuous pipeline has joints possessing higher strength and stiffness relative to the pipe barrel. For example, a steel pipeline with welded (butt, single lap or double lap welded) joints is treated as continuous pipeline. The joints of the continuous pipelines are often referred to as restrained joint.

2.4.4 – Design Basis Earthquake (DBE)

The design basis earthquake (DBE) ground motion for design of a pipeline may be evaluated as that having 10% probability of exceedance in 50 years (that is, return period of 475 years).

2.4.5 – Fault Movement

The fault movement is the abrupt differential movement of soil or rock on either side of fault.

2.4.6 – L-Waves

L-waves (Love waves) are surface seismic waves that cause horizontal movement of

COMMENTARY

C-2.4.5:

The design basis earthquake (DBE) may be defined in a number of ways. It is expected that a major pipeline project will be designed on the basis of site-specific seismic hazard analyses. This is particularly important considering that the seismic zone map in IS 1893:2002 (part-1) is not based on formal seismic hazard analysis. It may also be noted that the definition of DBE given herein is different from that used in IS 1893:2002 (part-1).

PROVISIONS

COMMENTARY

ground from side to side in a horizontal plane but at right angles to the direction of propagation.

2.4.7 – Landslide Hazard

Landslides are the mass movements of the ground which may be triggered by an earthquake or some other causes.

2.4.8 – Lateral Spreading

Lateral spreading is a phenomenon which occurs in a gently sloping ground when the soil deposit liquefies due to seismic shaking. The soil loses its shear strength during liquefaction, which in turn results in lateral movement of liquefied soil and any overlaying soil layer.

2.4.9 – Liquefaction

Liquefaction is a phenomenon that occurs in loose to medium dense sandy saturated soil during seismic shaking. During liquefaction, the soil loses a substantial amount of its shear strength and acts like a viscous fluid.

2.4.10 – P-Waves

P-waves (Primary waves) are the fastest body waves that carry energy through the Earth as longitudinal waves by moving particles in the same line as the direction of the wave. These can travel through all layers of the Earth. These waves are often referred to as compressional or longitudinal waves.

2.4.11 – Peak Ground Acceleration (PGA)

Peak ground acceleration is the maximum acceleration of seismic wave at the ground surface expected due to a seismic event. It refers to the horizontal motion unless specified otherwise.

2.4.12 – Peak Ground Velocity (PGV)

Peak ground velocity is the maximum velocity of seismic wave at the ground surface

PROVISIONS

COMMENTARY

expected due to a seismic event. It refers to the horizontal motion unless specified otherwise.

2.4.13 – Permanent Ground Deformation (PGD)

Permanent ground deformation is the large-scale non-recoverable ground deformation due to landslide, faulting or liquefaction induced lateral spreading.

2.4.14 – Phase Velocity

The velocity at which a transient vertical disturbance at a given frequency, originating at the ground surface, propagates across the surface of the medium is referred to as phase velocity.

2.4.15 – R-Waves

R-waves (Rayleigh waves) are surface waves that cause the particles of ground oscillate in an elliptical path in the vertical plane along the direction of the traveling wave.

2.4.16 – S-Waves

S-waves (Shear waves) are the body waves which cause material particles oscillate at right angles to the direction of energy transmission. S-waves cannot travel through fluids, such as air, water, or molten rock.

2.4.17 – Segmented Pipeline

Segmented pipeline has joints possessing lower strength and stiffness relative to the pipe barrel. For example, cast iron pipes with caulked or rubber gasket joints, ductile iron pipes with rubber gasket joints, concrete or asbestos pipes with mechanical joints, etc., are treated as segmented pipelines. The joints of the segmented pipes are generally referred to as unrestrained joints.

2.4.18 – Thrust Block

This is a concrete block made at the pipe joints to resist the unbalanced force developing due to the pipe bends or change

PROVISIONS | COMMENTARY

of pipe size. It also prevents the separation of pipes or pipe movement at the joints.

2.5 – Symbols

The symbols that are used in this document are given below. Some specific symbols those are not listed here are described at the place of their use.

A Cross sectional area of pipe

A_g Design peak ground acceleration

a_{max} Peak ground acceleration

c Coefficient of soil cohesion

C Velocity of Seismic Wave Propagation

$C_{apparent}$ Apparent wave propagation velocity

C_d Coefficient of damping

C_r Rayleigh wave velocity

C_s Shear wave velocity

C_{phase} Phase velocity of seismic wave

D Outside diameter of the pipe

D_c Depth of backfill over pipeline (refer: Figure 5.4 (b))

D_{min} Minimum inside diameter of pipe

Refer to Figure C3.9.1 for D_{min}.

E Modulus of elasticity of pipe material

E_i Modulus of elasticity of pipe material before yielding

E_p Modulus of elasticity of pipe material after yielding

PROVISIONS		COMMENTARY
f	Coating dependent factor relating internal friction angle of soil to friction angle at soil-pipe interface	
F	Focal depth	
F_b	Buoyant force acting on pipeline	
F_{stop}	Maximum design force for mechanical stops	
h_w	Height of water table above pipeline	
H	Depth of the center of pipeline from ground surface	
H_s	Height of the surface soil on base rock layer	
I	Moment of inertia of pipe	
I_g	Ground amplification factor due to site effect different for PGA and PGV	
I_p	Importance factor	
K_0	Coefficient of soil pressure at rest	
L	Length of permanent ground deformation zone	Refer to Figure C 4.1a for L.
$2L_a$	Length of pipeline between two anchor points	
L_b	Length of pipe in buoyancy zone	
L_{cr}	Critical length of permanent ground deformation zone	
L_0	Length of pipe segment	
m	Poisson's ratio of pipe material	
M_w	Moment magnitude of earthquake	

PROVISIONS

COMMENTARY

n, r Ramberg – Osgood parameters

n_c Number of chained restrained joints (Refer to clause 4.1.2.4)

N Uncorrected standard penetration resistance of the soil

N_c, N_q, N_γ Bearing capacity factors for soil

N_{ch} Horizontal bearing capacity factor for clay

N_{cv} Vertical uplift factor for clay

N_{qh} Horizontal bearing capacity factor for sandy soil

N_{qv} Vertical uplift factor for sand

P Maximum internal operating pressure of the pipe

P_y Axial yield strength of pipe

P_t Tensile force in the pipe

P_u Lateral soil-pipe interaction force per unit length of pipe

P_v Vertical earth pressure

P_{vu} Vertical earth pressure for undisturbed state of pipe

Q_d Vertical-bearing soil-pipe interaction force per unit length of pipe

Q_u Vertical-uplift soil-pipe interaction force per unit length of pipe

R Radius of pipe

R_e Epicentral distance

R_c Radius of curvature of pipe due to bending

PROVISIONS		COMMENTARY
S_p	Longitudinal tensile stress in pipe due to internal pressure	
S_r	Longitudinal tensile stress in pipe due to change in temperature	
S_u	Undrained shear strength of soil	
t	Pipe wall thickness	
T_1	Temperature at the time of pipe installation	
T_2	Temperature at the time of pipe operation	
T_g	Fundamental natural period of top soil layer over the bedrock	
t_u	Maximum friction force per unit length of pipe at soil pipe interface	
V_g	Design peak ground velocity	
V_s	Velocity of shear wave	
W	Width of permanent ground deformation zone	Refer to Figure C 4.1a for *W*.
W_w	Weight of water displaced by pipe per unit length	
W_p	Weight of pipe per unit length of pipe	
W_c	Weight of the pipe content per unit length	
Z	Section modulus of pipe cross section	
$\bar{\gamma}$	Effective unit weight of soil	
γ	Total unit weight of soil	
γ_w	Unit weight of water	
ψ	Dip angle of the fault movement	

PROVISIONS | **COMMENTARY**

Δ_p Mobilizing displacement at P_u (Figure B2, Annex-B)

Δ_{qd} Mobilizing displacement of soil at Q_d (Figure B3, Annex-B)

Δ_{qu} Mobilizing displacement of soil at Q_u (refer:Figure B3, Annex-B)

Δ_t Mobilizing displacement of soil at t_u (Figure B1, Annex-B)

$\Delta_{allowable}$ Allowable joint displacement for segmented pipe

Δ_{oper} Joint displacement during operation

$\Delta_{seismic}$ Maximum joint displacement due to seismic action

α Adhesion coefficient of clay

α_t Linear coefficient of thermal expansion of pipe material

α_ε Ground strain coefficient

β Angle of crossing of pipeline and the fault line

δ^l Maximum longitudinal permanent ground deformation

$\delta^l{}_{design}$ Design longitudinal permanent ground deformation

δ^t Maximum transverse permanent ground deformation

$\delta^t{}_{design}$ Design transverse permanent ground deformation

δ' Interface angle of friction between pipe and soil

δ_{fax} Component of fault displacement in axial direction of pipe

PROVISIONS **COMMENTARY**

$\delta_{fax-design}$ Design fault displacement in axial direction of pipe

δ_{fs} Strike slip fault displacement

δ_{fn} Normal fault displacement

δ_{fr} Reverse fault displacement

δ_{fb} Displacement of poorly known fault

δ_{ftr} Component of fault displacement in transverse direction of pipe

$\delta_{ftr-design}$ Design fault displacement in transverse direction of pipe

δ_{fvt} Component of fault displacement in vertical direction of pipe

$\delta_{fvt-design}$ Design fault displacement in vertical direction of pipe

ε Total strain in the pipeline due to both axial and flexural action

ε_{D+L} Strain in pipe due to service loads

ε_{oper} Operational strain in pipe

$\varepsilon_{allowable}$ Allowable strain in pipe

$\varepsilon_{seismic}$ Maximum strain in pipe due to seismic action

ε_{a} Axial strain in pipe

ε_{b} Bending strain in pipe

ε_{cr-c} Critical strain in pipe in compression

ε_{cr-t} Critical strain in pipe in tension

PROVISIONS **COMMENTARY**

ε_{c-wave} Allowable strain for wave effect in pipe

ε_{c-pgd} Allowable strain for permanent ground deformation effect in pipe

ε_p Strain in pipe due to internal pressure

ε_t Strain in pipe due to temperature change

ϕ Internal angle of friction of the soil

λ Apparent wavelength of seismic waves at ground surface

μ Frictional coefficient

σ Stress in pipe

σ_a Axial stress in pipe

σ_y Yield stress of pipe material

σ_{bf} Bending stress in pipe due to buoyancy

2.6 – Acronyms

Acronyms that are used in this document are:

API American Petroleum Institute

ASCE American Society of Civil Engineers

DBE Design Basis Earthquake

IBC International Building Code

JSCE Japan Society of Civil Engineers

PGA Peak Ground Acceleration

PGA_r Peak Ground Acceleration at base rock layer

PGD Permanent Ground Deformation

PGV Peak Ground Velocity

PGV_r Peak Ground Velocity at base rock layer

PROVISIONS

3 – General Principles and Design Criteria

3.1 – General

After the functional (non-seismic) design, the pipeline should be checked for all possible seismic hazards it may encounter. The pipeline safety is to be checked for seismic loads simultaneously with the operating loads (pressure, temperature, initial bending, etc). The pipeline response and design criteria for some general seismic hazards are specified in this guideline. For specific localized hazards, the seismic evaluation of the pipeline should be carried out with reference to specialized reports and literature.

The analysis and design criteria specified in this document require the following engineering information.

3.1.1 – Pipeline Information

a) Pipe geometry (diameter, thickness);

b) Type of pipe joint;

c) Stress-strain relationship of pipe material;

d) Pipeline function and its post seismic performance requirement;

e) External pipe coating specification ;

f) Operating pressure in the pipe;

g) Operational and installation temperature;

h) Pipeline alignment detail (plan, profile, location of fittings, etc.); and

i) Reduced strain limit for existing pipelines.

3.1.2 – Site Information

a) Burial depth of the pipeline;

b) Basic soil properties (unit weight, cohesion, internal friction angle and *in situ* density).

c) Properties of backfill soil in the trench;

d) Depth of water table; and

COMMENTARY

C-3.1-

Some specific seismic hazards for which the design guidelines are not provided in this document are: tsunami and seiches, tectonic uplift and subsidence, ground densification, etc.

PROVISIONS

3.1.3 – Seismic Hazard Information

a) Expected amount of seismic ground motion at the site;

b) Expected amount and pattern of permanent ground deformation and its spatial extent;

c) Length of pipeline exposed to permanent ground deformation;

d) Active fault locations; expected magnitude of fault displacement, and orientation of pipeline with respect to direction of fault movement.

3.2 – Classification of Pipelines

The pipelines have been classified into four groups as per their functional requirement as follows.

Class-I: Very essential water pipelines required to serve for post earthquake response and intended to remain functional and operational during and following a design earthquake.

High pressure oil and gas pipelines which are required to remain functional during and following the design earthquake.

Pipelines which would cause extensive loss of life or a major impact on environment in case of failure or damage.

Class-II: Critical water pipelines serving a large community and having significant economic impact to the community or a substantial hazard to human life and property in the event of failure.

Medium pressure oil and gas pipelines which are vital energy serving facilities, but their service can be interrupted for a sort period until minor repairs are made.

Class-III: Most of the water supply pipelines for ordinary use.

Low pressure oil and gas pipelines.

Class-IV: Water Pipelines that have low or very little importance and effect on the human life and society in the event of failure. Pipelines which do not require quick repair after a seismic event.

COMMENTARY

C-3.2 –

The pipeline classification is meant for outlining the level of seismic performance required for the system. As the classification provided here is descriptive, engineering judgment has to be exercised in classifying a pipeline.

For oil and gas pipelines, internal pressure can be classified as (JSCE, 2000b):

High Pressure: $P \geq 10$ kgf/cm^2

Medium Pressure: $3 < P < 10$ kgf/cm^2

Low Pressure: $P \leq 3$ kgf/cm^2

PROVISIONS

COMMENTARY

3.3 – Classification of Soil

The soil at the site in the top 30m has been classified as given in Table 3.3 according to shear wave velocities. However, in many cases, shear wave velocities of the soil are not available, alternative definitions (e.g., Undrained shear strength, Standard penetration resistance) are also included in Table 3.3 for classification of soil.

C-3.3 –

The classification of soil site here is same as suggested in *NEHRP Recommended Provisions for New Buildings and Other Structures* (FEMA-450, 2004). The same classification is being used in many guidelines and literatures (e.g., ALA 2005, IBC 2003, etc).

Table 3.3: Classification of soil at site.

Soil class	Soil Type	Velocity of shear wave (V_s), m/s	Undrained shear strength (S_u), kN/m^2	Uncorrected Standard penetration resistance (N)
A	Hard rock	$V_s > 1500$	---	--
B	Rock	$760 < V_s \le 1500$	---	--
C	Very dense soil and soft rock	$360 < Vs \le 760$	$S_u \ge 98$	$N > 50$
D	Dense/ Stiff soil	$180 < Vs \le 360$	$49 \le S_u \le 98$	$15 \le N \le 50$
E	Loose/ Soft soil	$Vs < 180$	$S_u < 49$	$N < 15$
	Soft soil with PI* >10 and Natural Moisture Content ≥ 40%	---	$S_u < 24$	--
*F***	Soil vulnerable to potential failure or collapse under seismic loading (i.e. liquefiable soil, quick and highly sensitive soil, collapsible weakly cemented soil) Peat or highly organic clays (H>3m, where H = thickness of soil) Very high plasticity clays (H>7.5m with plasticity index > 75) Very thick medium or soft stiff clays (H>35m)	---	--	--

*PI = Plasticity Index of the soil

** The soil requires site specific investigation

NOTE: When sufficient detail of the soil is not available to define the site class, S*oil Class-D* shall be used. *Soil Class E* or *F* need not be used unless established by geotechnical data or authorized by authority having jurisdiction.

3.3.1 –

When top 30m soil layer contains distinctly different soil layers, then the soil at the site

PROVISIONS

can be classified as per the normalized values of V_s or S_u or N as defined as follows.

3.3.1.1 –

Normalized shear wave velocity for layered top soil may be taken as:

$$\overline{V}_s = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{V_{si}}}$$

Where

$\overline{V}_s$ = Normalized shear wave velocity for top 30m soil

n = No. of layers in top 30m soil

d_i = Thickness of i^{th} layer in top 30m soil

V_{si} = Shear wave velocity in i^{th} layer

3.3.1.2 –

Normalized undrained shear strength for layered top soil may be taken as:

$$\overline{S}_u = \frac{d_c}{\sum_{i=1}^{k} \frac{d_{ci}}{S_{ui}}}$$

Where

$\overline{S}_u$ = Normalized undrained shear strength of top 30m soil

k = No of cohesive soil layers in top 30m soil

d_c = Total thickness of cohesive soil layers in top 30m soil

d_{ci} = Thickness of i^{th} cohesive soil layers in between top 30m soil

S_{ui} = Undrained shear strength in i^{th} cohesive layer

3.3.1.3 –

Normalized standard penetration resistance for layered top soil may be taken as:

COMMENTARY

C-3.3.1.2 –

For cohesive soil, the normalized undrained shear strength is used as a criterion to classify the soil. However, for cohesionless soil, the criterion is normalized standard penetration resistance (N) (clause 3.3.1.3).

PROVISIONS

COMMENTARY

$$\overline{N} = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{N_i}}$$

Where

$\overline{N}$ = Standard Penetration Resistance for top 30m soil

n = No of layers in top 30m soil

d_i = Thickness of i^{th} layer between top 30m soil

N_i = Standard Penetration Resistance in i^{th} layer

3.4 – Classification of Seismic Hazards

The seismic hazards which are directly related to pipeline failure can be classified as:

1) Permanent ground deformation related to soil failures:
    a. Longitudinal permanent ground deformation
    b. Transverse permanent ground deformation
    c. Landslide
2) Buoyancy due to liquefaction
3) Permanent ground deformation related to faulting. (Abrupt PGD)
4) Seismic wave propagation

C-3.4 –

The longitudinal and transverse permanent ground deformation due to soil failure is often referred to as lateral spreading.

3.5 – Design Seismic Hazard

3.5.1 – Design Basis Seismic Hazard

The design basis earthquake ground motion (in terms of acceleration and velocity) and ground deformation (faulting, transverse and longitudinal permanent ground deformation, landslide, etc) corresponding to class III pipeline should be estimated based on site specific hazard analysis for an earthquake of 10% probability of exceedance in 50 years

C-3.5.1 –

Seismic hazard analysis is an important step in evaluating the design seismic hazard for any system or facility. The details about this can be referred from the following publications: Housner and Jennings (1982), Reiter (1991), McGuire (2004), etc.

In areas of high seismicity, 475 year event is considered adequate. However, this may be inadequate in areas subjected to high magnitude

PROVISIONS

(return period of 475 years). This corresponds to PGA/ PGV on soil site class B. If not, then PGA/ PGV value has to be multiplied by ground amplification factor (I_g) as given in Table 3.5.3(a) or 3.5.3(b).

3.5.2 – Design Seismic Hazard

The design seismic hazard for various classes of pipelines may be calculated by multiplying importance factor (I_p) given in Table 3.5.2, to the Design Basis Seismic Hazard (clause-3.5.1).

Table 3.5.2: Importance factor for different classes of pipeline (I_p).

Class of pipeline	Wave propagation	Faulting	Transverse and Longitudinal PGD	Landslide
I	1.5	2.3	1.5	2.6
II	1.25	1.5	1.35	1.6
III	*1.0*	1.0	1.0	1.0
IV	*	*	*	*

* : Seismic conditions need not be considered

COMMENTARY

events on a very long time scale where the 475 year event may be too small to initiate liquefaction or landslides. Hence, special care is warranted when PGA for 475 year event is rather small (say less than 0.10g) but the PGA for 2475 year event is substantial (say more than 0.30g).

C-3.5.2 –

The design level of seismic safety to be provided to a pipeline depends on importance of the pipeline and the consequences of its failure. The importance can be accounted in two ways.

a) Design the pipeline for higher seismic hazard, which is corresponding to higher return period. For instance Table C 3.5.2 gives the design basis earthquake for different types of pipes as per *Seismic Guidelines for Water Pipelines* (ALA 2005).

b) Design the pipeline for the hazards corresponding to design basis earthquake and multiplied by an importance factor (I_p).

In this document, the second approach has been adopted.

Table C 3.5.2: Recommended design levels of seismic hazard.

Pipe class	Probability of exceedance in 50 years	Return period (Years)
I	2%	2475
II	5%	975
III	10%	475
IV	No seismic design consideration required	

PROVISIONS

3.5.3 – Ground Amplification Factor

The ground amplification factors (I_g) given in Table 3.5.3(a) and 3.5.3(b) may be used for obtaining ground motion at the surface layer from that at the base rock level.

COMMENTARY

C-3.5.3 –

The amount of ground motion amplification relative to bedrock depends on the soil conditions at the site. In general, the amplification is more in softer soils (with lower shear wave velocities) than stiffer soils (with higher shear wave velocities). But, increase in ground shaking intensity increases the non-linearity of stress-strain of soil and increases soil damping, which reduces amplification.

Various strong motion recordings obtained on a variety of geological settings during many earthquakes provides the basis of defining ground amplification factor as given in Table 3.5.3(a) and (b); these values are adopted from FEMA:450 (2005).

Table 3.5.3(a): Ground amplification factor (I_g) for peak ground velocity for various soil classes

Class of Soil	Peak Ground Velocity at surface layer (PGV) / Peak Ground Velocity at base rock layer (PGV_r)				
	$PGV_r \le 0.1$m/s	$PGV_r = 0.2$m/s	$PGV_r = 0.3$m/s	$PGV_r = 0.4$m/s	$PGV_r \ge 0.5$m/s
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	*	*	*	*	*

* Site-specific geotechnical investigation and dynamic site response analysis is recommended to develop appropriate values.

Table 3.5.3(b): Ground amplification factor (I_g) for peak ground acceleration for various soil classes.

Class of Soil	Peak Ground Acceleration Values at site (PGA) / Peak Ground Acceleration at base rock layer (PGA_r)				
	$PGA_r \le 0.1$g	$PGA_r = 0.2$g	$PGA_r = 0.3$g	$PGA_r = 0.4$g	$PGA_r \ge 0.5$g
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	*	*	*	*	*

* Site-specific geotechnical investigation and dynamic site response analysis is recommended to develop appropriate values.

PROVISIONS

3.5.4– PGA as per Seismic Zones

When the site specific ground acceleration data are not available, the expected peak ground acceleration (PGA) at the base rock level can be approximated as given in Table 3.5.4 for different seismic zones defined in IS 1893-2002 (part-1).

COMMENTARY

C-3.5.4 –

Ideally, probabilistic seismic hazard analysis should be carried out for important projects. However, for other projects it may not always be possible to do so. In that case the PGA values, which may form appropriate design bases, may be used as per Table 3.5.4. IS: 1893 does not provide probabilistic values, however, in the absence of better data or until IS 1893 provides rational PGA values, this seems to be the best option.

Table 3.5.4: Peak Ground Acceleration as per Seismic Zones.

Seismic Zone	II	III	IV	V
PGA_r	0.1g	0.16g	0.24g	0.36g

3.5.5 – Determining PGV from PGA

When only the peak ground acceleration is available, Table 3.5.5 can be used to estimate peak ground velocity at that site.

C-3.5.5 –

While using Table 3.5.5, user must specify the distance of site from earthquake source and the magnitude of the earthquake. For many sites, the seismic hazard will be the maximum of the earthquakes from varying sources. Hence, Table 3.5.5 has to be looked up for each earthquake sources. Table 3.5.5 is same as that used in some other related literatures (e.g: ALA 2001, ALA 2005, etc.).

Table 3.5.5: Relationship between peak ground velocity and peak ground acceleration.

Moment Magnitude (M_w)		Ratio of Peak Ground Velocity (cm/s) to Peak Ground Acceleration (m/s^2)		
		Source-to-Site Distance		
		0-20 (km)	20-50 (km)	50-100 (km)
Rock	6.5	66	76	86
	7.5	97	109	97
	8.5	127	140	152
Stiff Soil	6.5	94	102	109
	7.5	140	127	155
	8.5	180	188	193
Soft Soil	6.5	140	132	142
	7.5	208	165	201
	8.5	269	244	251

Note: The relationship between peak ground velocity (PGV) and peak ground acceleration (PGA) is less certain in soft soils.

PROVISIONS | COMMENTARY

3.6 – General Seismic Design Considerations

Some general seismic design considerations that should be taken into account while designing the pipeline system are as follows.

3.6.1 –

In most cases, the seismic hazards can not be quantified precisely. Hence, based on available data and experience, reasonable assumptions should be made to define proper model for the seismic hazard.

3.6.2 –

In the design of pipeline systems, permanent ground deformation is a much more serious concern than seismic shaking.

C-3.6.2 –

Many theoretical and experimental investigations show that the inertia forces arising from the interaction between pipe and surrounding soil are far less detrimental to the safety of pipeline than the ground deformation.

3.6.3 –

As a general rule, it is assumed that the sites located at the epicentral region are more affected by body waves (P and S waves), whereas the sites at larger distance are more affected by surface waves (R and L waves).

3.6.4 –

In all areas of expected ground rupture, pipelines should be provided with automatic shutdown valves.

C-3.6.4 –

Automatic shutdown valves should be provided at a reasonable interval to localize the pipeline failure/rupture and consequent hazard. The segregation of the damaged section enables the pipeline to be repaired easily and operation of the undamaged system can be restored soon.

3.6.5 –

The fittings of the segmented steel pipelines (e.g., water pipelines) should be ductile.

C-3.6.5 –

Fittings and valves installed in the pipelines are often constructed of different materials than the pipelines. The fittings for small diameter ductile iron, PVC (Polyvinyl Chloride) and polyethylene pipes are often made of cast iron. As the cast iron fittings are brittle as compared to the ductile iron pipe materials, the fittings fail more easily than ductile iron. Hence, it is recommended to use ductile fittings in high seismic areas.

PROVISIONS

COMMENTARY

3.6.6 –

Corrosion in the pipeline should be controlled by suitable means, as the pipe may get severe damage in case of a seismic event if already corroded.

3.6.7 –

A ductile coating (e.g., asphalt coating, polyethylene sheet, etc.) is recommended at the joints of the segmented pipeline.

C-3.6.7 –

In segmented pipelines, rigid coatings (e.g. mortar lining) to the pipe is likely to fail in the seismic event which may also lead to corrosion problem in future.

3.6.8 –

For segmented pipeline, the displacement absorption capacity of the joint should be more than the expected joint movement due to design seismic action.

3.7 – Analysis Procedure

Acceptable analysis methods described in subsequent clauses may be used to evaluate the response of buried pipeline.

3.7.1 –

In general, it is advisable to take the advantage of post elastic behaviour of pipelines. However, critical components of pipeline, which can cause extensive loss of life or major impact on the environment, should be designed to remain elastic.

3.7.2 –

Where a detailed analysis is required for important pipelines, a nonlinear finite element analysis is preferred.

C-3.7.2 –

Finite element method of modeling and analyzing the pipeline system allows explicit consideration of the non-linear behaviour of pipe-soil interaction and non-linear material properties of both soil and pipe. Annex-A gives some guidelines for modeling of the pipeline. The following publications may also be referred to for detailed description regarding pipeline modeling; MCEER Monograph-3 (O'Rourke et al., 1999), ALA 2001, ALA 2005, PRCI 2004, etc.

3.7.3 –

The stresses (or strains) obtained from the seismic analysis should be combined linearly with the stresses (or strains) in the pipeline during operation (clauses 3.8.1 and 3.8.2).

PROVISIONS

COMMENTARY

3.7.4 –

When the stress-strain relationship for the pipe material is not defined, it may be approximated by Ramberg-Osgood's relationship as :

$$\varepsilon = \frac{\sigma}{E}\left[1 + \frac{n}{1+r}\left(\frac{\sigma}{\sigma_y}\right)^r\right]$$

Where

ε = Engineering strain

σ = Stress in the pipe

E = Initial Young's modulus

σ_y = Yield strain of the pipe material

n, r = Ramberg - Osgood parameters (Table 3.7.4)

C-3.7.4 –

Ramberg-Osgood relationship (Ramberg et al., 1943) is one of the most widely used models for post elastic behaviour of pipes.

Table 3.7.4: Ramberg -Osgood parameters for steel pipes.

Grade of Pipe	Grade – B	X – 42	X – 52	X – 60	X – 70
Yield stress (MPa) of the pipe material	227	310	358	413	517
n	10	15	9	10	5.5
r	100	32	10	12	16.6

Note: For '*n*' and '*r*' values of other pipes, Ramberg et al., (1943) may be referred.

3.8 – Initial Stresses in the Pipeline

3.8.1 – Internal Pressure

The longitudinal stress in pipe due to internal pressure may be calculated as:

$$S_p = \frac{PD\mu}{2t}$$

Where

P = Maximum internal operating pressure of the pipe

D = Outside diameter of the pipe

C-3.8.1 –

The equation presented here is the basic equation used for pipes subjected to internal pressure. The same equation has also been used in API Guideline (API-1117, 1996).

PROVISIONS

COMMENTARY

μ = Poisson's ratio (generally taken as 0.3 for steel)

t = Nominal wall thickness of the pipe

3.8.2 – Temperature Change

The longitudinal stress in pipe due to temperature change may be estimated by the following equation:

$$S_r = E\alpha_t (T_2 - T_1)$$

Where

E = Modulus of elasticity

α_t = Linear coefficient of thermal expansion of steel

T_1 = Temperature in the pipe at the time of installation

T_2 = Temperature in the pipe at the time of operation

C-3.8.2 –

The equation used here is basic thermal equation for any material subjected to temperature variation. The same equation has also been used in the API guidelines API Guideline (API-1117, 1996).

3.9 – Allowable Strain for Continuous Pipeline

3.9.1 –

The maximum allowable strains for buried continuous pipelines are specified in Table 3.9.1.

The allowable strain given in Table 3.9.1 is only applicable to the pipes confirming to API standard (API, 1990). For other types of pipes, the allowable strain limit provided by the manufacturer may be used.

C-3.9.1 –

The allowable strain levels used in Table 3.9.1 are taken from ASCE 1984, ALA 2005 and JSCE 2000.

The specified acceptable strain limit is only a stepping stone to the overall system-wide design criteria, and the designer could check the system as a whole to establish the acceptable stress/strain for a particular pipe. Therefore, the real performance criteria should preferably be set based on economic impacts to the community, not a particular stress/strain.

PROVISIONS

Table 3.9.1: Allowable strain criteria for buried continuous pipelines

Strain Component	Pipe Category	Allowable Strain	
		Tension	Compression
Continuous Oil and Gas Pipeline	Ductile Cast Iron Pipe	2%	For PGD: Onset of Wrinkling ($\varepsilon_{cr\text{-}c}$) (clause-3.9.2)
	Steel Pipe	3%	
	Polyethylene Pipe	20%	
	Bends and Tees of pipe	1%	For wave propagation:50% to 100% of the Onset of Wrinkling (0.5 to 1 $\varepsilon_{cr\text{-}c}$)
Continuous Water Pipeline	Steel and iron pipe	0.25 ε_u or 5%	ε_{c-pgd}
			ε_{c-wave}

Note: Allowable strain criteria for other verities of pipelines may be obtained from the manufacture.

Where

ε_u = Failure strain of the pipe in tension

$$\varepsilon_{c-pgd} = 0.88\frac{t}{R}$$

$$\varepsilon_{c-wave} = 0.75\left[0.5\frac{t}{D'} - 0.0025 + 3000\left(\frac{PD}{2Et}\right)^2\right]$$

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

D_{min} = Minimum inside diameter of pipe = outside diameter of pipe excluding out of roundness thickness (Figure C 3.9.1)

COMMENTARY

Onset of wrinkling might be suitable for the design criteria of high pressure oil and gas pipelines, where the wrinkling of pipe may restrict the passage of its contents or failure of the pipe might result in fire or other serious consequences to nearby facilities and habitat. However, the wrinkling limit as specified in clause- 3.9.2 for oil and gas pipelines will become very conservative for water pipelines. Hence, for water pipeline more relaxed strain limits of ε_{cr-pgd} and $\varepsilon_{cr-wave}$ (Table 3.9.1) are specified in this document.

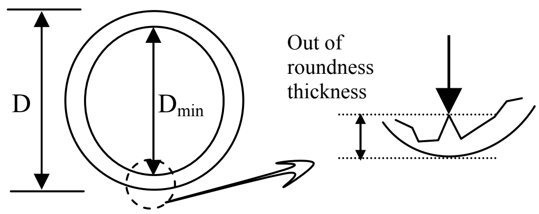


Figure C 3.9.1: Schematic diagram showing minimum inside diameter of pipe

PROVISIONS

3.9.2 -

For oil and gas pipeline, the maximum tensile strain should not exceed 4% in any case. For bends and tees maximum strain is restricted to 2%. The limiting compressive strain is considered as the strain at onset of wrinkling, i.e.:

$$\varepsilon_{cr-c} = 0.175\frac{t}{R}$$

Where

t = Thickness of pipe

R = Radius of pipe

3.9.3 -

The design strain for continuous pipelines should be less than the allowable strain, i.e.:

$$\varepsilon_{seismic} + \varepsilon_{oper} \le \varepsilon_{allowable}$$

Where

$\varepsilon_{allowable}$ = Allowable strain in pipe as per Table 3.9.1

$\varepsilon_{seismic}$ = Design strain in pipe due to seismic hazard

ε_{oper} = Operational strain in the pipeline which is equal to: $\varepsilon_p + \varepsilon_t + \varepsilon_{D+L}$

ε_p = Strain in the pipe due to internal pressure (Clause 3.8)

ε_t = Strain in the pipe due to temperature change (Clause 3.9)

ε_{D+L} = Strain in the pipe due to service loads

3.10 – Allowable Joint Displacement for Segmented Pipeline

3.10.1 -

The design joint displacement for all segmented pipes should be less than the

COMMENTARY

C-3.9.2 –

The theoretical value of local wrinkling of a pipe begins at a compressive strain of ε_c as given in the following equation (ASCE, 1984).

$$\varepsilon_c = 0.6\frac{t}{R}$$

However, the experimental results for thin walled pipes show that the compressive wrinkling begins at a strain of $1/3^{rd}$ to $1/4^{th}$ of the theoretical wrinkling strain. In this document the allowable wrinkling strain is considered as the mean of the experimental wrinkling strains.

C-3.10.1 –

The allowable joint displacement for buried segmented pipeline considered here is same as outlined in *Seismic Guidelines for Water Pipelines*

PROVISIONS

allowable joint displacement.

$$\Delta_{seismic} + \Delta_{oper} \leq \Delta_{allowable}$$

Where

$\Delta_{allowable}$ = Allowable joint displacement

$\Delta_{seismic}$ = Maximum joint displacement due to seismic hazard

Δ_{oper} = Operational joint displacement which is equal to: $\Delta_p + \Delta_t + \Delta_{D+L}$

Δ_p = Joint displacement due to internal pressure

Δ_t = Joint displacement due to temperature change

Δ_{D+L} = Joint displacement due to service loads

The allowable joint displacement of pipes varies widely according to its type and material. It is hence preferable to obtain the allowable joint displacement from the manufacturer.

3.10.2 –

In segmented water pipelines, sometimes an allowance is advised for safety margin, i.e.

$$\Delta_{seismic} + \Delta_{oper} \leq \Delta_{allowable} - Allowance$$

An Allowance of about 0.6 cm covers the additional safety allowance for many pipe joints (ALA, 2005).

COMMENTARY

(ALA, 2005).

PROVISIONS

4 – Design Criteria for Permanent Ground Deformation (PGD)

The permanent ground deformation refers to the unrecoverable soil displacement due to faulting, landslide, settlement or liquefaction-induced lateral spreading. In this clause the attention is restricted to the permanent ground deformation due to liquefaction-induced lateral spreading and landslide.

COMMENTARY

C-4 –

Permanent ground deformations in any seismic event may be due to faulting or due to soil failure. Faulting has been characterized by abrupt permanent ground deformation, where as the ground deformation associated with soil failure (Figure C 4(a)) is gradual. Clause-4 addresses the permanent ground deformation associated with the soil failure only.

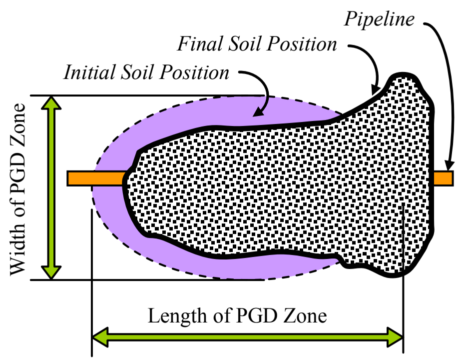


Figure C 4(a): Schematic diagram showing permanent ground deformation due to soil failure.

There are many patterns of permanent ground deformation which depend on local soil condition and geological settings. The pipeline may cross the permanent ground deformation zone in any arbitrary direction (Figure C 4(b)). However, designing the pipeline for critical response due to permanent ground deformation (PGD), two conditions such as parallel crossing (Figure C 4(c)) and perpendicular crossing (Figure C 4(d)). This document considers these two situations as the pipeline is subjected to longitudinal PGD and transverse PGD respectively.

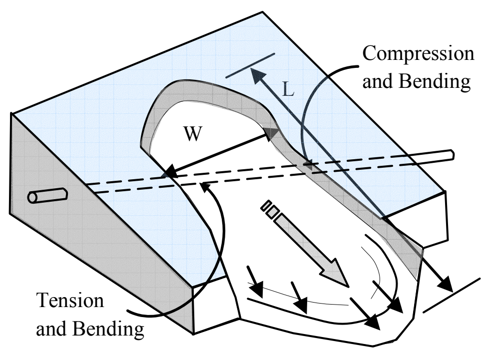


Figure C 4(b): Pipeline crossing permanent ground deformation zone at any arbitrary angle.

PROVISIONS

COMMENTARY

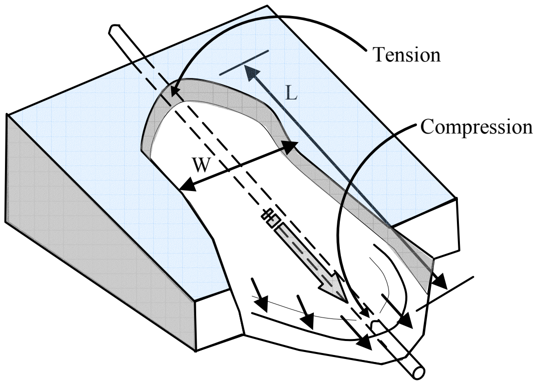


Figure C 4(c): Longitudinal permanent ground deformation; pipeline crossing permanent ground deformation zone in the direction of ground movement.

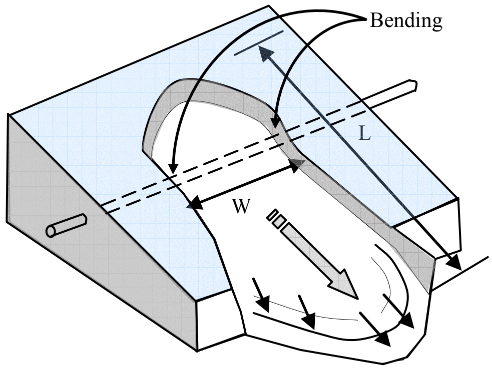


Figure C 4(d): Transverse permanent ground deformation; pipeline crossing permanent ground deformation zone transverse to the ground movement.

In general, the amount of permanent ground deformation is much larger in transverse direction than vertical settlement. The design guidelines for vertical settlement are not provided in this document.

It has been observed from the past earthquakes that the permanent ground deformation is one of the major seismic hazards that may cause substantial damage to pipelines. The following recommendations may be followed to improve the pipeline performance against permanent ground deformation.

i) If the expected ground displacement exceeds the displacement absorption capacity of the pipe, other alternatives such as soil improvement, etc. shall be employed.

ii) Pipeline response can be minimized either by minimizing the ground displacement, and/or increasing the load carrying

PROVISIONS

COMMENTARY

capacity of the pipe system. By minimizing the pipe diameter the soil friction loads can also be minimized. But at the same time, influence of the diameter on hydraulic design should be checked.

iii) The friction between the pipe and soil can be minimized by using appropriate pipe coating or wrapping. Polyethylene wrapping is commonly used for corrosion protection, which is also effective in reducing friction force of pipe-soil interaction.

iv) Strength of soil surrounding the pipeline should be improved to reduce the lateral soil movement and soil flow. For shallow liquefiable deposits, soil densification, and for deeper deposits, stabilized soil buttresses can be constructed at discrete points along the pipeline.

v) As far as possible, pipeline should be placed below the lowest depth of liquefiable soil.

vi) All the pipeline facilities can be located outside the area of ground deformation zone. This may not be an option for an existing utility confined to a right of way.

vii) Trenches, deformable walls or other similar means can be constructed to absorb ground deformations at an upslope location.

viii) The pipeline can be designed and constructed to survive ground movements while remaining in service. They can be designed to move with the ground without breaking or the foundations can be constructed to withstand anticipated soil displacement.

ix) The pipeline may be supported at large distances on well-founded piers to increase the flexibility. Flexible joints should also be considered to allow relative displacement between the supports. Currently, many flexible pipe joints are available commercially, which can accommodate substantial amount of vertical and horizontal displacements.

x) Where extreme deformation is expected, special pipe joints or fittings are required to be used to allow greater joint deflection, extension or compression.

PROVISIONS

COMMENTARY

xi) The pipelines require moderate to high ductility in areas of permanent ground deformation. Welded polyethylene pipes may be a better option in such areas.

xii) In segmented pipelines special connections are required to accommodate large ground movement in the areas of permanent ground deformation.

4.1 – Longitudinal Permanent Ground Deformation

From the geotechnical investigations, the spatial extent, i.e., length (L), width (W) and maximum longitudinal ground displacement (δ^l) of permanent ground deformation (PGD) zone, should be established.

It is generally difficult to come out with a single number for the amount (δ^l) and spatial extent (L and W) of permanent ground displacement. Hence a range of the above quantities are established, and the seismic check is carried out.

The design ground displacement in longitudinal direction may be taken as:

$$\delta^l{}_{design} = \delta^l \times I_p$$

Where:

δ^l = Maximum longitudinal ground displacement

I_p = Importance factor (Table 3.5.2)

C-4.1 –

This clause is applicable when the pipeline is subjected to ground displacement parallel to its pipe axis. The pattern of longitudinal permanent ground deformation may be of various types; e.g., block pattern, ramp pattern, ridge pattern, ramp-block pattern, asymmetric ridge pattern, etc. (O'Rourke, et al., 1995). For critical response, the block pattern (i.e., the longitudinal ground movement is uniform throughout the PGD zone (Figure C 4.1(a))) of ground deformation is used in this document.

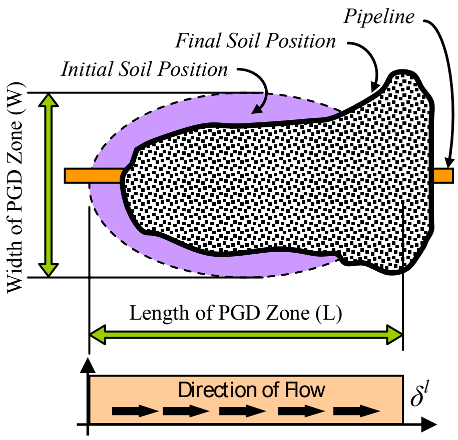


Figure C 4.1(a): Block pattern of longitudinal permanent ground deformation.

4.1.1– Continuous Pipeline

4.1.1.1 –

Generally two types of models are used for buried pipelines subjected to a block pattern of longitudinal permanent ground deformation, such as:

Case-1: The amount of ground movement ($\delta^l{}_{design}$) is large and the pipe strain is controlled by length (L) of the PGD zone.

C 4.1.1.1 –

Two types of inelastic models as given in clause 4.1.1.1 are suggested by O'Rourke et al., (1995) for buried pipelines subjected to a block pattern of longitudinal permanent ground deformation.

PROVISIONS

COMMENTARY

Case-2: The Length (L) of PGD zone is large and the pipe strain is controlled by amount of ground movement $\delta^l{}_{design}$.

4.1.1.2 –

When $\delta^l{}_{design}$ is large (i.e., Case-1), the maximum axial strain in pipe for both tension and compression can be calculated as:

$$\varepsilon_a = \frac{t_u L}{2\pi D t E}\left[1 + \frac{n}{1+r}\left(\frac{t_u L}{2\pi D t \sigma_y}\right)^r\right]$$

Where

- L = Length of permanent ground deformation zone
- σ_y = Yield stress of pipe material
- n,r = Ramberg-Osgood parameter (Table 3.7.4)
- E = Modulus of elasticity of pipe material
- t_u = Peak friction force per unit length of pipe at soil pipe interface
- D = Outside diameter of pipe
- t = Thickness of pipe

C-4.1.1.2 –

Figure C 4.1.1.2 shows the situation for Case-1. The friction force per unit length of pipe in the entire length of permanent ground deformation zone (L) from point B to point D acts to the right due to ground displacement (δ^l). By symmetry and equilibrium , the friction force per unit length acts to the left, over a distance of *L/2* before the head of the PGD zone (from point A to point B) and over a distance of *L/2* beyond the toe of the PGD zone (from point D to point E). In the pipe, the maximum tensile strain occurs at point B and maximum compressive strain occurs at point D.

According to the above conditions, the maximum stress (tensile or compressive) in the pipe is the stress induced due to friction force over a length of *L/2*. Hence, the maximum tensile/compressive stress in the pipe can be calculated as:

$$\sigma = \frac{t_u L}{2\pi D t}$$

Ramberg-Osgood's stress-strain relationship (clause-3.7.4) may be used to find the maximum strain in pipe from the maximum stress value. This yields the maximum strain as specified in clause 4.1.1.2.

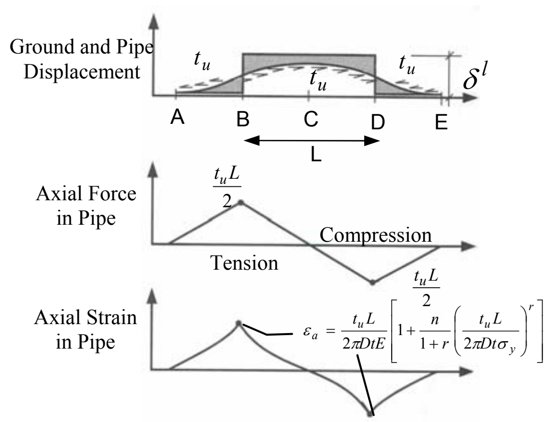


Figure C 4.1.1.2: Case-1: Inelastic model for longitudinal PGD (O'Rourke et. al., 1995)

4.1.1.3 –

When L is very large (i.e., Case-2), δ governs the amount of strain in pipe, and the peak pipe

C-4.1.1.3 –

Figure C 4.1.1.3 shows the situation for Case-2. Here, the friction force is acting over an as yet

PROVISIONS

COMMENTARY

strain for both tension and compression can be calculated as:

$$\varepsilon_a = \frac{t_u L_e}{\pi D t E}\left[1 + \frac{n}{1+r}\left(\frac{t_u L_e}{\pi D t \sigma_y}\right)^r\right]$$

Where

L_e = Effective length of pipeline over which friction force t_u acts, and can be evaluated from the following equation

$$\delta^l{}_{design} = \frac{t_u L_e^{\,2}}{\pi D t E}\left[1 + \left(\frac{2}{2+r}\right)\left(\frac{n}{1+r}\right)\left(\frac{t_u L_e}{\pi D t \sigma_y}\right)^r\right]$$

unknown length of L_e on each side of the PGD zone (from point A to point B and from point E to point F). The pipe displacement matches the ground displacement (δ^l) over a region of length L - $2L_e$ at the center of the PGD zone.

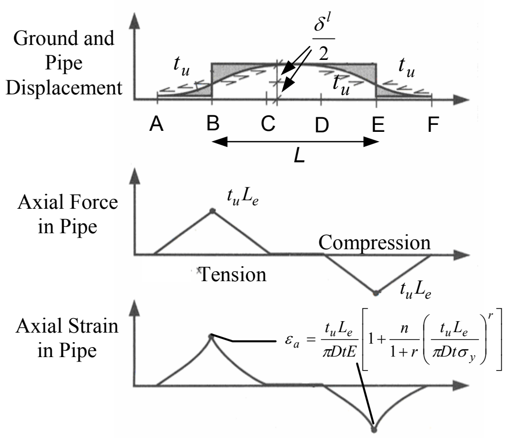


Figure C 4.1.1.3: Case-2: Inelastic model for longitudinal PGD (O'Rourke et. al., 1995)

The maximum strain in pipe (tensile or compressive) for this case can also be calculated as worked out in Case 1. However, the only difference is that, instead of *L/2*, the effective length of L_e, over which the friction force acts is considered. From Figure C 4.1.1.3, by symmetry, the pipe displacement at point B is $\delta^l /2$, where

$$\frac{\delta^l}{2} = \int_0^{L_e} \varepsilon_a(x)dx \; .$$

By using the peak strain of pipe in above equation, the effective length of pipeline over which the friction force acts (L_e) can be estimated.

4.1.1.4 –

The design pipe strain ($\varepsilon_{seismic}$) for longitudinal permanent ground deformation should be taken as the lower of the strains obtained from clause 4.1.1.2 and 4.1.1.3. The design pipe strain should confirm to the allowable strain criteria as given in clause 3.9.

4.1.1.5 – Influence of Expansion Joint

Depending on the position of expansion joints, they may have no effect, beneficial effect or a

C-4.1.1.5 –

The expansion joints are flexible joints that are provided in the continuous pipelines to absorb the ground movement. MCEER Monograph-3

PROVISIONS

detrimental effect on pipeline. Often, the expansion joints are provided to mitigate the effect of longitudinal PGD in continuous pipeline. It is advisable to provide at least two expansion joints, one close to the head of the PGD zone and the other close to the toe.

COMMENTARY

(O'Rourke et al., 1999) may be referred for the influence of expansion joint on the pipeline performance.

4.1.1.6 – Influence of Field Bend

Special attention should be given while calculating the response of pipeline subjected to longitudinal PGD with field bends.

C-4.1.1.6 –

If an elbow or bend is located close to but beyond the margins of permanent ground displacement zone, large pipe stresses may develop due to bending. Often local wrinkling is also expected at bends.

4.1.2 – Segmented Pipeline

4.1.2.1 –

The design joint displacement in pipe can be calculated as the maximum opening at the joint of the pipe ($\Delta_{seismic}$) due to longitudinal permanent ground deformation. Hence,

$$\Delta_{seismic} = \delta^{l}{}_{design} .$$

Where,

$\delta^{l}{}_{design}$ = Design ground displacement in longitudinal direction (clause 4.1).

The design joint displacement should confirm to the allowable joint displacement as specified in clause 3.10.

C-4.1.2.1 –

If the ground movement within the PGD zone is relatively uniform, it is expected that the damage will mainly concentrate at the joints of the segmented pipeline. Ground displacement is generally assumed to be accommodated only by joint contraction or expansion.

4.1.2.2 –

For small amount of ground displacement, push-on type joints (joints without mechanical stops) may be used in the PGD zone. One such joint may be provided at the head and one at the toe of the PGD zone. Each joint should be designed for a design joint displacement of $\Delta_{seismic}$ as specified in clause 4.1.2.1.

C 4.1.2.2 –

For push-on type joints subjected to longitudinal PGD, joints in the immediate vicinity of the head and the toe must accommodate the expected ground movement. It is assumed that the block pattern longitudinal ground movement is accommodated by expansion of a single joint at head of the PGD zone or by contraction of a single joint at the toe of the PGD zone.

In reality, it is quite possible that the soil mass on one side of head or toe will be stiffer than on the other side. Hence, the joint at each end should be designed to accommodate the full ground movement.

PROVISIONS

4.1.2.3 –

In the areas of large ground displacement, a chained joint can be designed to accommodate it. Normally, chained joints are required when one single joint can not accommodate the expected ground displacement.

The designed chained joints should be provided at both head and toe of the PGD zone, of which at least three joints are to be installed outside the PGD zone at the PGD zone boundary.

The design joint displacement of each pipe segment may be calculated as:

$$\Delta_{seismic} = \left(\frac{\delta^{l}{}_{design}}{L/2} \right) \times L_a$$

Where

L_a = Length of pipe segment

L = Length of permanent ground deformation zone.

The design joint displacement ($\Delta_{seismic}$) should confirm to the allowable joint displacement criteria as given in clause 3.10.

4.1.2.4 –

The mechanical stops, which are used in chained joints, must be designed to accommodate maximum friction force (F_{stop}) given by:

$$F_{stop} = 2\left(\frac{n_c + 1}{2} \right) \times L_a \times t_u$$

Where

n_c = No. of chained joints at head or toe of the moving soil mass, that will expand to absorb total amount of PGD.

But in any case, F_{stop} need not be higher than the yield strength of pipe.

4.2 – Transverse Permanent Ground Deformation

Like longitudinal PGD, a range of the amount (δ^t) and spatial extent (L and W) of transverse PGD are quantified and the seismic check is

COMMENTARY

C 4.1.2.3 –

A chained joint is a segmented joint with the additional requirement of having mechanical stops to prevent the pipes from pulling apart. The pullout capacity of the whole series of joints is going to resist the expected ground movement in the axial direction of the pipeline.

As discusses in C 4.1.2.2, it is quite possible to have head or toe of the PGD zone to be stiffer than the other one. In this situation, pipe joints at one side of the PGD zone is expected to resist the total amount of ground displacement.

C-4.2 –

When subjected to transverse ground deformation, a continuous pipeline will stretch and bend as it attempts to accommodate it.

Alike longitudinal ground displacement, the

PROVISIONS	COMMENTARY

carried out.

The design ground displacement in transverse direction can be calculated as:

$$\delta^t{}_{design} = \delta^t \times I_p$$

Where

δ^t = Maximum transverse ground displacement

I_p = Importance factor (Table 3.5.2)

pattern of the transverse ground displacement can also be of different types. A cosine function is assumed here to define the transverse permanent ground deformation profile as shown in Figure C 4.2.

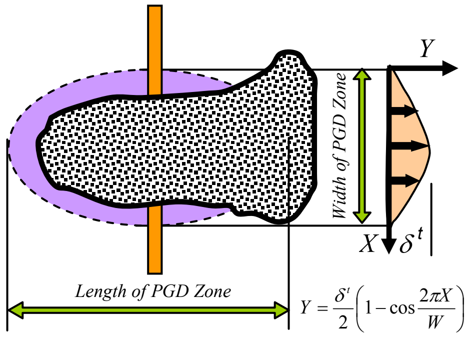


Figure C 4.2: Pattern of transverse permanent ground deformation.

4.2.1 – Continuous Pipeline

4.2.1.1 –

The maximum bending strain in pipe may be conservatively calculated as the least of the following two:

a) $\varepsilon_b = \pm \frac{\pi D \delta^t{}_{design}}{W^2}$

Where

D = Outside diameter of pipe

$\delta^t{}_{design}$ = Design transverse ground displacement

W = Width of permanent ground deformation zone

t = Thickness of pipe

b) $\varepsilon_b = \pm \frac{P_u W^2}{3\pi E t D^2}$

Where

P_u = Maximum lateral resistance of soil per unit length of pipe (Annex-B)

E = Modulus of elasticity of pipe material

C-4.2.1.1 –

The analytical expressions used here are based on O'Rourke's (O'Rourke et al., 1999) simplified model pipeline response to spatially distributed permanent ground deformation. Two conditions have been considered, such as: a) large width of permanent ground deformation zone and pipeline is assumed to be flexible, and b) narrow width of permanent ground deformation zone and pipeline is assumed to be stiff.

PROVISIONS

4.2.1.2 –

The maximum strain obtained in clause 4.2.1.1 should be considered as the design pipe strain ($\varepsilon_{seismic}$) and should confirm to the allowable pipe strain as given in clause 3.9.

4.2.1.3 –

Simplified analytical expressions given above may be used for determining strain in the pipeline required for preliminary design. However, finite element analysis considering nonlinearity in the pipe and the soil is advised while designing important pipelines.

4.2.2 – Segmented Pipeline

4.2.2.1 –

The design joint displacement of pipe ($\Delta_{seismic}$) for transverse PGD can be calculated as the sum of axial extension and extension due to rotational effect. Thus, the resulting joint displacement can be written as:

$$\Delta_{seismic} = \frac{\pi^2 L_0 {\delta^t_{design}}^2}{W^2}\left[\frac{2D}{\delta^t_{design}}\right]$$

For $0.268 \le D/\delta^t_{design} \le 3.73$

$$\Delta_{seismic} = \frac{\pi^2 L_0 {\delta^t_{design}}^2}{2W^2}\left[1+\left(\frac{D}{\delta^t_{design}}\right)^2\right]$$

For other values of D/δ^t_{design}

Where

L_0 = Length of pipe segment

δ^t_{design} = Design transverse ground displacement

4.2.2.2 –

The design joint displacement ($\Delta_{seismic}$) calculated above should confirm to the allowable joint displacement criteria as given in clause 3.10.

COMMENTARY

C-4.2.2.1 –

Transverse PGD causes both axial extensions and angular rotation in the pipe joint. In the analytical formulation given in clause 4.2.2.1, the transverse PGD pattern is assumed to be a cosine function (ALA, 2005).

PROVISIONS

COMMENTARY

4.3 –Landslide

Landslides are the large movements of the ground, generally due to slope failure, which may be triggered by earthquake shaking.

4.3.1 –

In landslides, the soil mass movements are catastrophic. The damage to the pipeline system in this area is of high magnitude. These slide zones should be avoided through careful route selection.

4.3.2 –

The effect of landslide on pipelines can sometimes be avoided by deep burial of pipes below the expected lower boundary of the sliding soil.

4.3.3 –

If the behaviour of landslide and its displacement pattern is defined, then this can be modeled as permanent ground displacement acting on pipelines. Annex-A may be referred for pipeline modeling.

PROVISIONS	COMMENTARY

5 – Design Criteria for Buoyancy due to Liquefaction

When liquefaction of soil occurs around the pipeline, buoyant forces are exerted on pipeline and must be resisted by suitable anchoring device.

C-5 –

Buoyancy effects are probably of greatest concern in areas such as flood plains and estuaries where massive liquefaction could take place in a major earthquake.

The following recommendations may be followed to minimize the buoyancy effects on pipeline.

i) Pipelines may be encased with concrete pipes to reduce the buoyancy effects, but the increased diameter will also increase lateral drag force on pipeline during lateral spreading due to liquefaction.

ii) Concrete weights or gravel filled blankets can be utilized to provide additional resistance to buoyancy.

iii) Buoyancy effect can also be minimized by shallow burial of pipeline above the ground water table.

iv) Where uplift is the main concern, anchors may be provided with a close spacing (~150 m) to prevent uplift.

5.1 – Buoyant Force on Pipeline

The net upward force per unit length of pipeline due to buoyancy may be calculated as:

$$F_b = W_s - [W_p + W_c + (P_v - \gamma_w h_w)D]$$

Where

W_s = Total weight of soil displaced by pipe per unit length

W_p = Weight of pipe per unit length

W_c = Weight of pipe content per unit length

P_v = Vertical earth pressure

D = Outside diameter of pipe

γ_w = Unit weight of water

h_w = Height of water above pipeline

C-5.1 –

When the pipeline is located below water table and placed in a trench, the vertical earth pressure on the pipeline can be calculated as:

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

Where

R_w = A factor for water buoyancy

$= 1 - 0.33\left(\frac{h_w}{C}\right)$

C = Height of soil fill over pipeline (Figure C 5.1(b))

γ_d = Dry unit weight of backfill

h_w = Height of water over pipeline (Figure C 5.1(b))

When the pipeline is jacked into undisturbed and unsaturated soil instead of being placed in the trench and covered with backfill, then the earth

PROVISIONS

The adherence of soil to pipe wall is neglected in the above calculations for simplicity.

COMMENTARY

load on pipe can be calculated as:

$$P_v = \gamma_w h_w + R_w \gamma_d C - 2c\frac{C}{D}$$

Where

c = Coefficient of soil cohesion

= 0 kg/cm^2 for loose dry sand

= 0.7 kg/cm^2 for hard clay

The earth load on the pipeline mentioned here is taken from the *Guidelines for the Design of Buried Steel Pipe* (ALA, 2001).

The buoyancy force acting on the pipeline is shown in Figure C 5.1(a) and (b) for reference.

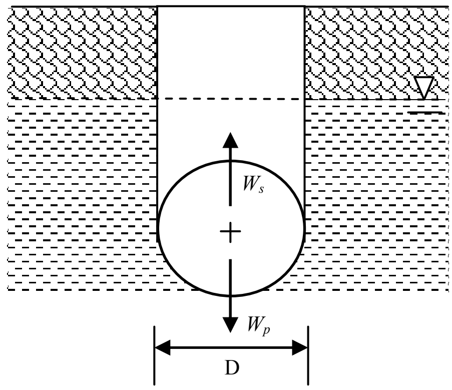


Figure C 5.1 (a): Cross section of the pipeline showing the forces acting on it due to buoyancy.

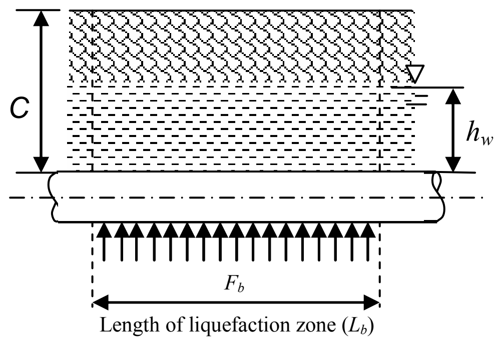


Figure C 5.1(b): Longitudinal section of the pipeline showing the forces acting on it due to buoyancy.

PROVISIONS

COMMENTARY

5.2 – Continuous Pipeline

5.2.1 –

Bending stress induced for a relatively short section of continuous pipeline subjected to buoyancy can be calculated as:

$$\sigma_{bf} = \frac{F_b L_b^{\ 2}}{10Z}$$

Where

L_b = Length of pipe in buoyancy zone

Z = Section modulus of pipe cross section

F_b = Buoyant force acting on pipeline

For longer sections of pipeline subjected to buoyancy force, the pipe can exhibit both cable and beam action to resist the upward force.

5.2.2 –

The maximum strain corresponding to above bending stress (clause 5.2.1) can be obtained by using Ramberg-Osgood's stress-strain relationship (clause 3.7.4).

5.2.3–

The maximum strain obtained in clause 5.2.2 can be considered as the design strain in pipe ($\varepsilon_{seismic}$) and should confirm to the allowable strain as specified in clause 3.9.

5.3 – Segmented Pipeline

5.3.1–

The response of segmented pipeline subjected to buoyancy force can be analyzed according to the location of the joint by using the equilibrium of forces and moment as shown in Figure C 5.1.(a) and (b).

PROVISIONS

COMMENTARY

5.3.2–

In the analysis, the joint of the segmented pipe may be considered as a hinge joint and the extension and rotation of the joint is obtained. The extension of the joint can be considered as the design joint displacement of the pipeline and should confirm to the allowable joint displacement as specified in clause 3.10.

PROVISIONS

6 – Design Criteria for Fault Crossing

Fault movement is the phenomenon related to the offset or tearing of the ground surface by differential movement across the fault line. The following criteria may be followed to design the pipeline crossing a fault of expected ground movement.

COMMENTARY

C-6 –

A fault is a crack or zone of crack between two blocks of rock. Faults allow the blocks to move relative to each other. This movement may be due to sudden displacement or may be due to gradual accumulation. The sudden fault movement is mostly associated with the seismic event. Whereas, the gradual displacement is mainly associated with the plate movement.

Faults may be classified according to the direction of motion as normal slip, strike slip, or reverse slip faults. The normal, strike, and reverse slip faults are formed due to tensile, shear, and compressive stresses respectively. Often the normal or reverse fault occurs in combination with the strike slip fault. This kind of faulting is referred to as oblique fault. This is formed due to the combination of stresses acting both vertically and horizontally.

The magnitude of fault displacement depends primarily on the type of fault, size of earthquake, focal depth and the geology.

For buried structures, for example pipelines, get severe damage due to fault displacement. The following recommendations may be followed to reduce the risk of pipeline crossing a fault.

i) The pipeline crossing fault line should be oriented in such a way to avoid compression in the pipeline. The optimum angle of fault-crossings will depend on the dip of the fault plane and the expected type of movement.

ii) The ductility of pipeline should be increased in the zone of fault-crossing to accommodate the fault movement without rupture.

iii) Abrupt changes in wall thickness or other stress concentrators should be avoided within the fault zone.

iv) In all areas of potential ground rupture, pipelines should be laid in relatively straight section avoiding sharp changes in direction and elevation.

v) To the extent possible, pipelines should be constructed without field bends, elbows, and flanges that tend to anchor the pipeline to the ground.

vi) If longer length of pipeline is available to conform to fault movement, level of strain

PROVISIONS | **COMMENTARY**

gets reduced. Hence, the points of anchorage should be provided away from the fault zone to the extent possible in order to lower the level of strain in the pipeline.

vii) A hard and smooth coating on pipeline such as an epoxy coating may be used in the vicinity of fault crossing to reduce the friction between the pipe and soil.

viii) The burial depth of pipeline may be reduced within fault zones in order to minimize the soil restraint on the pipeline during fault movement.

ix) If the expected fault displacement is very large, it is advisable to take the pipeline above ground and design with sliding supports to sustain the expected level of ground displacement.

In Indian subcontinent, the surface faulting is a relatively infrequent phenomenon. Most of the fault lines are deep below the ground level. Hence, more importance is given to the permanent ground deformation due to soil failures than surface faulting effects on pipeline.

6.1 – Quantification of Fault Displacement

6.1.1 –

Evaluating the expected fault displacement requires specialized and rigorous analysis. In the absence of such an analysis, available site specific empirical relationship may be used. One of the widely used empirical relationships is the one that was given by Wells and Coppersmith (1994). According to that the fault displacement can be evaluated as follows.

For strike slip fault:

$$\log \delta_{fs} = -6.32 + 0.90M$$

For Normal fault

$$\log \delta_{fn} = -4.45 + 0.63M$$

For reverse fault

$$\log \delta_{fr} = -0.74 + 0.08M$$

For a poorly known fault or blind fault

$$\log \delta_{fb} = -4.80 + 0.69M$$

C-6.1.1 –

Based on worldwide database of 421 historical earthquakes, Wells and Coppersmith selected 244 earthquakes and developed empirical relationship between fault displacement and moment magnitude. As per their observation, fault displacement varies from .05 to 8.0m for strike slip faults, .08 to 2.1m for normal faults, and .06 to 1.5m for reverse faults.

PROVISIONS

Where

δ_{fs} = Strike slip fault displacement in meters

δ_{fn} = Normal slip fault displacement in meters

δ_{fr} = Reverse slip fault displacement in meters

δ_{fb} = Displacement of a blind fault in meters

M = Moment magnitude of earthquake

6.1.2 –

For a strike slip fault (Figure 6.1.2), the fault movement along and transverse to the pipeline may be calculated as follows.

Component of fault displacement in the axial direction of pipeline is:

$$\delta_{fax} = \delta_{fs} \cos\beta$$

Component of fault displacement in transverse direction of pipeline

$$\delta_{ftr} = \delta_{fs} \sin\beta$$

Where

β = angle of pipeline crossing a fault line (Figure 6.1.2)

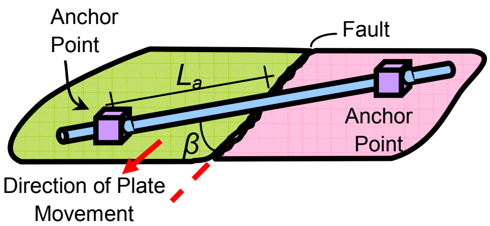


Figure 6.1.2: Pipeline crossing strike slip fault.

6.1.3 –

For a normal slip fault (Figure 6.1.3), the fault movement along, transverse and vertical to the pipeline may be obtained as follows.

Component of fault displacement in the axial direction of pipeline:

$$\delta_{fax} = \delta_{fn} \cos\psi \sin\beta$$

Component of fault displacement in transverse direction of pipeline:

$$\delta_{ftr} = \delta_{fn} \cos\psi \cos\beta$$

Component of fault displacement in vertical direction of pipeline:

COMMENTARY

C-6.1.2 –

Refer clause 6.2.2 for the considerations taken for anchor points.

PROVISIONS

$$\delta_{fvt} = \delta_{fn} \sin\psi$$

Where

β = angle of pipeline crossing a fault line (Figure 6.1.3)

β = Dip angle of the fault (Figure 6.1.3)

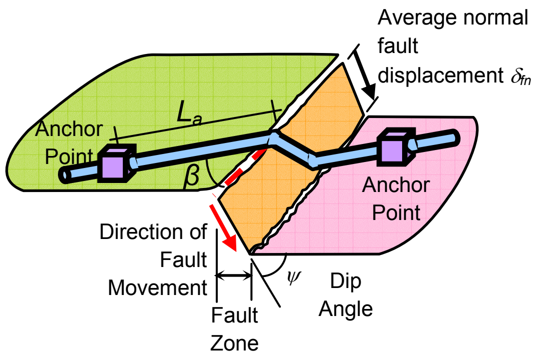


Figure 6.1.3: Pipeline crossing normal slip fault.

6.1.4 –

In reverse faults, the displacement components are evaluated in the similar way as in normal-slip fault, but, with a negative slip.

6.1.5 –

For oblique faults, the strike slip displacement and normal slip (or reverse slip) displacement may be added algebraically in axial, transverse and vertical direction of the pipeline axis.

6.1.6 –

Design fault displacement can be evaluated by multiplying the importance factor (I_p) (Table 3.5.2) with the expected fault displacement. Hence,

Design fault displacement in the axial direction of pipeline is:

$$\delta_{fax-design} = \delta_{fax} \times I_p$$

Design fault displacement in transverse direction of pipeline is:

$$\delta_{ftr-design} = \delta_{ftr} \times I_p$$

COMMENTARY

C-6.1.5 –

In general, the fault displacements are three-dimensional and it depends on the magnitude of strike-slip and normal or reverse-slip.

PROVISIONS

Design fault displacement in vertical direction of pipeline:

$$\delta_{fvt-design} = \delta_{fvt} \times I_p$$

6.2 – Continuous Pipeline

6.2.1 –

The average pipe strain due to fault crossing can be calculated as:

$$\varepsilon = 2\left[\frac{\delta_{fax-design}}{2L_a} + \frac{1}{2}\left(\frac{\delta_{ftr-design}}{2L_a}\right)^2\right]$$

Where

L_a = Unanchored pipe length (refer: clause-6.2.2)

6.2.2–

The unanchored length of pipeline in the zone of fault crossing can be taken as the least of the following:

a) When there are no bends, tie-ins or any type of constraints to the pipeline near the fault zone, the effective unanchored length of the pipeline may be taken as:

$$L_a = \frac{E_i \varepsilon_y \pi D t}{t_u}$$

Where

t_u = The ultimate friction force acting in axial direction of the pipe

ε_y = The yield strain of the material

E_i = Modulus of pipe material before

COMMENTARY

C-6.2.1 –

The expression for average strain in pipeline is based on Newmark-Hall's model (Figure C 6.2.1). A factor of 2 in the formula in clause 6.2.1 is used to counterbalance the unconservatism involved in this model. However, this model should only be used for initial approximation and the detailed design should be based on suitable nonlinear analysis.

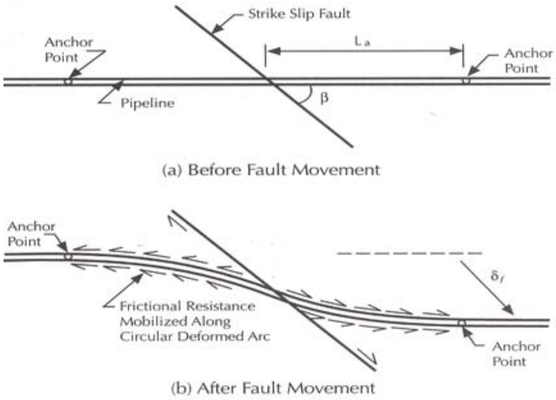


Figure C 6.2.1: Newmark-Hall model for fault crossing (O'Rourke et al., 1999).

C-6.2.2 –

The unanchored length of the pipeline is controlled by both the pipeline system structures and also by connections such as services, hydrants and tees/bends/crosses.

The effective unanchored length of pipeline may be calculated considering that the axial restraint to the pipe is provided by soil-pipe friction. The effective unanchored length of pipeline can be calculated as:

$$L_a = \frac{(E_i \varepsilon_y \pi D t)}{t_u} + \frac{(E_p(\varepsilon - \varepsilon_y)\pi D t)}{t_u}$$

Where

ε = Plastic strain in pipe

E_p = Modulus of pipe material after yielding

The second part of the above equation represents

PROVISIONS

yielding

D = Diameter of the pipe

t = Thickness of the pipe

b) Any anchorage provided by the pipe configuration (e.g., bends, elbows, change in soil cover, etc) shall be considered as the actual point of anchorage. And the length of pipeline from the point of anchorage to the fault line will be taken as the effective unanchored length (Figure C 6.2.1).

6.2.3 –

The average strain calculated in clause 6.2.1 can be considered as the design strain in pipe ($\varepsilon_{seismic}$) and should satisfy the allowable strain criteria as specified in clause 3.9.

6.3 – Segmented Pipeline

6.3.1 –

In segmented pipelines, the fault offset is assumed to be accommodated equally by pipe joints located on each side of the fault line. The design displacement of the joints can be calculated as:

$$\Delta_{seismic} = \delta_{fax} \times I_p$$

Where

I_p = Importance factor (Table 3.5.2)

6.3.2 –

The design joint displacement ($\Delta_{seismic}$) for each joint of the pipe in fault zone should confirm to the allowable joint displacement as specified in clause 3.10.

6.4 – Trench Profile for Pipelines in Fault Zone

To achieve minimum soil resistance to reduce the strain in pipe, the pipeline can be buried in a shallow trench as shown in Figure 6.4 (a) and Figure 6.4(b) with loose to medium granular soil without cobbles or boulders. Close control must be exercised over the backfill material of the pipe trench over a

COMMENTARY

the actual tensile force in the pipeline, which needs an iterative method of evaluation. However, the contribution of this part to the total length of anchorage is not significant. Hence, the effective unanchored length is often calculated by considering only the first term of the equation as(refer: PRCI 2004):

$$L_a = \frac{E_i \varepsilon_y \pi D t}{t_u}$$

C-6.3.1 –

For a segmented pipeline (Figure C 6.3.1) crossing fault line, it is generally assumed that a) the pipe segments are rigid and b) only the pipe joints accommodate the ground deformation.

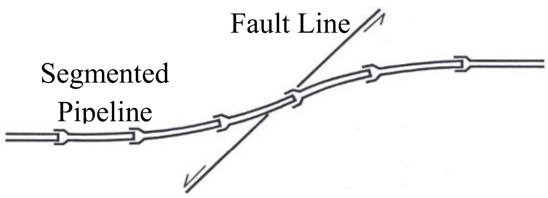


Figure C 6.3.1: Segmented pipeline crossing a strike slip fault.

C-6.4-

Fault displacement absorption capacity of the pipeline can be maximized by minimizing the longitudinal, lateral, and uplift resistance between the surrounding soil and the pipe by suitable means.

PROVISIONS

considerable distance (~300m) on each side of the fault.

Good geotextile membrane may be used in between native and backfill soil (i.e., over the trench wall). This will prevent mixing of the fines from native soil with the well graded backfill material for a long period of time.

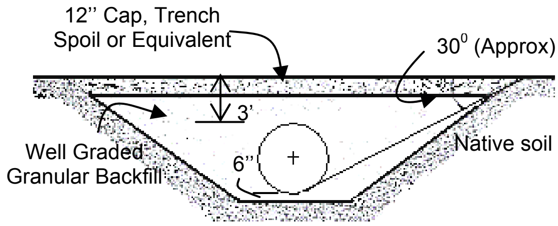


Figure 6.4 (a): Pipeline trench for strike slip fault crossing (NIST, 1996).

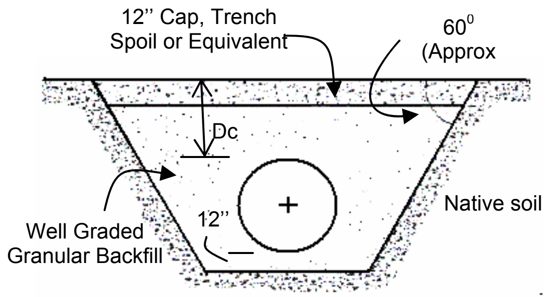


Figure 6.4 (b): Pipeline trench for reverse slip fault crossing (NIST, 1996).

6.5 – Detailed Modeling

While analyzing important pipelines for fault crossing, finite element methods are recommended for use (see Annex-A).

COMMENTARY

C 6.5 –

The above analytical procedure of designing pipeline for fault movement is based on simplified Newmark–Hall's model (Figure C 6.2.1). This procedure may be followed to estimate the design values of first order approximation. A nonlinear finite element based analysis is hence recommended for detailed analysis and design.

PROVISIONS

7 – Design Criteria for Seismic Wave Propagation

The response of pipeline due to wave propagation is generally described in terms of longitudinal axial strains in pipes. Flexural strains in pipes due to ground curvature are neglected since these are relatively small.

COMMENTARY

C-7 –

Every earthquake is associated with ground motion which generally includes body waves and surface waves. Body waves attenuate with distance more rapidly as compared to the surface waves. The above ground structures are more susceptible to the seismic wave hazards than the underground structures. However, pipelines buried at a very shallow depth may get damaged due to the ground shaking. The burial depth is hence an important design parameter for buried pipelines, as siting at greater depth can reduce the design levels of ground shaking.

While designing for seismic wave propagation, the pipeline is assumed to fail primarily due to wave passage and is not combined with any other seismic effect.

The following recommendations may be followed to mitigate the pipeline against seismic wave propagation.

i) Seismic wave propagation generally does not have serious effect on welded buried pipelines in good condition. Some situations where the wave propagation imply serious damage to the pipeline system include: a) transition between very stiff and very soft soils, b) penetration of pipe into valve boxes, c) pipes located at or near pump stations, d) T-connections, e) pipe fittings and valves, etc. Therefore, special care should be taken while designing the pipeline system in above situations.

ii) The pipelines weakened by corrosion, and the old cast iron pipes with bell and spigot joints are vulnerable to seismic wave propagation. Therefore special attention should be given to them.

iii) As far as possible, the selection of the seismic waves and the corresponding wave propagation speeds should be based on geophysical considerations.

iv) The effect of wave propagation on pipelines can be minimized by minimizing the interaction force at soil-pipe interface with suitable pipe coating or wrapping or using suitable backfill soil.

PROVISIONS | **COMMENTARY**

7.1 – Design Ground Motion

7.1.1 –

The design seismic motion at a site is often characterized as the velocity of seismic wave propagation. The design wave propagation velocity can be calculated as:

$$V_g = PGV \times I_p$$

Where

PGV = Peak ground velocity expected at the site (clause 3.5)

I_p = Importance factor as specified in table 3.5.2.

7.1.2 – Apparent Wave Propagation Velocity

The apparent wave propagation velocity is an important parameter which is used to calculate the strain in pipe induced by seismic waves.

To evaluate the axial strain in pipe, as a general rule, the velocity of shear wave (S-wave) is used for the sites within the epicentral distance of 5 times focal depth. In the other hand, the velocity of Rayleigh wave (R-wave) is considered for the sites having epicentral distance more than 5 times focal depth (Figure 7.1.2).

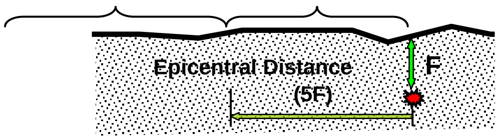


Figure 7.1.2: Considerations for S-wave and R-wave in pipeline design.

C-7.1.2 –

The apparent propagation velocity of both body and surface waves are of interest, since the pipelines are typically buried at shallow depth (1 – 3 m) below ground surface.

For body waves, only S-waves are considered since they carry more energy and generate larger ground motion than P waves.

For surface waves, only R-waves are considered since they induce axial strain in the pipeline significantly higher than that of the bending strain induced by L-waves.

7.1.2.1 – For S-wave

The apparent wave propagation velocity ($C_{s\_apparent}$) for S-waves can be calculated as:

$$C_{s\_apparent} = \frac{C_s}{\sin\theta}$$

Where

C-7.1.2.1

When the site is subjected to body waves only, the apparent wave propagation velocity of shear wave with respect to ground surface is many times higher than the shear wave velocity of the near surface material. The seismic energy originating at depth passes through increasing layers of softer materials and refraction causes a concave travel path. Hence the net result being body waves which arrive at the ground surface with small incident

PROVISIONS

COMMENTARY

C_s = Wave propagation velocity of S-wave

θ = Angle of incidence of S-wave

angle with respect to vertical. If the angle of incidence is θ and the shear wave velocity of the top layer is C_s, then the apparent wave propagation velocity is:

$$C_{s\_apparent} = \frac{C_s}{\sin\theta}$$

When the angle of incidence approaches zero, the apparent propagation velocity becomes infinity.

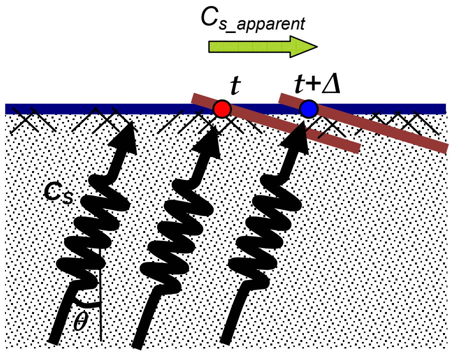


Figure C 7.1.2.1: Figure illustrating the apparent seismic wave propagation of S-wave

7.1.2.2 – For R-wave

The apparent wave propagation velocity for R waves can be taken as:

$$C_{r\_apparent} = C_{r\_ph}$$

Where

C_{r_ph} = Phase velocity of R-wave

C-7.1.2.2

As the R-waves always travel parallel to the ground surface, the apparent propagation velocity is same as its phase velocity.

The calculation of phase velocity of R-wave is complicated. For simplified calculation, specific literatures (e.g., O'Rourke et al., 1999) may be referred.

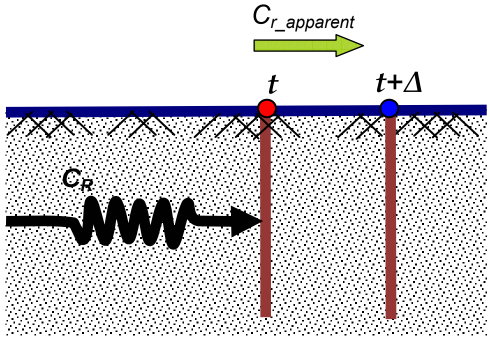


Figure C 7.1.2.2: Figure illustrating apparent seismic wave propagation of R wave.

PROVISIONS

COMMENTARY

7.2 – Continuous Pipeline

7.2.1 –

The maximum longitudinal axial strain, that can be induced in the pipeline due to wave propagation, can be approximated as:

$$\varepsilon_a = \frac{V_g}{C_{apparent}} = \frac{V_g}{\alpha_\varepsilon C}$$

Where

V_g = Design peak ground velocity (clause 7.1.1)

α_ε = Ground strain coefficient

= 2.0 (for S-waves)

= 1.0 (for R-waves)

C = Velocity of seismic wave propagation

= C_s, for S waves, (2.0 km/s may be considered conservatively)

= C_{r_ph}, for R-waves (0.5 km/s may be considered conservatively)

C-7.2.1 –

The ground strain coefficient α_ε depends on the angle of incidence and type of seismic waves. For S-waves, when there is an angle in the horizontal plane between pipe axis and the direction of apparent wave propagation, there exists a component of ground motion parallel to the pipe axis (Figure C 7.2.1). Hence, the apparent propagation velocity in the direction of pipe axis is:

$$C_{s\_apparent} = \frac{C_s}{\sin\theta\cos\gamma}$$

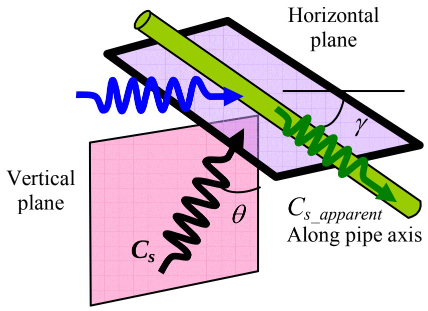


Figure C 7.2.1: Schematic for apparent wave propagation velocity of S-wave along the pipe axis.

The ground strain is maximum for θ and γ = 45° (Yeh, 1974),

i.e., $C_{s\_apparent} = \dfrac{C_s}{\left(\frac{1}{\sqrt{2}}\right)\left(\frac{1}{\sqrt{2}}\right)} = 2C_s$.

So α_ε is taken as 2 for S-wave.

For R-wave, the phase velocity is considered parallel to the pipe axis, and hence the apparent propagation velocity along pipeline axis becomes equal to its phase velocity.

7.2.2 –

The maximum strain induced in the pipeline by friction at the soil-pipe interface is calculated as:

$$\varepsilon_a \le \frac{t_u \lambda}{4AE}$$

Where

C-7.2.2 –

The maximum frictional resistance (t_u) of the soil to pipe movement (Figure C 7.2.2) is based on the limiting conditions, such as: a) the slippage is occurring over the entire length of the pipe, and b) the friction force acting on the pipe surface is uniform (O'Rourke et al., 1999).

The apparent wave length of seismic wave (λ) is defined as the product of apparent wave propagation velocity (V_s) and natural fundamental

PROVISIONS

t_u = Peak frictional force per unit length at soil-pipe interface (Annex-B)

λ = Apparent wavelength of seismic waves at ground surface (often taken as 1.0 km in the absence of detailed information)

A = Cross sectional area of pipe

E = Modulus of elasticity of pipe material

7.2.3 –

The axial strain calculated in clause 7.2.1 can be considered as the design strain ($\varepsilon_{seismic}$). However, the design strain need not exceed the maximum strain that can be induced in pipeline by soil friction (clause 7.2.2).

7.2.5 –

The design pipe strain as calculated above (clause 7.2.3) should confirm to the allowable strain limit as specified in clause 3.9.

7.3 – Segmented Pipeline

7.3.1 –

The design joint displacement in pipeline can be calculated as:

$$\Delta_{seismic} = \varepsilon_{seismic} L_0$$

Where

$\varepsilon_{seismic}$ = Axial strain as calculated in clause 7.2.4 for the continuous pipeline.

L_0 = Length of pipe segment

7.3.2 –

The design joint displacement ($\Delta_{seismic}$) calculated above should satisfy the allowable joint displacement criteria as given in clause 3.10.

7.3.3 –

The design joint rotation in the pipeline due to wave propagation can be calculated as:

COMMENTARY

period of ground surface.

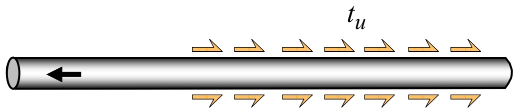


Figure C 7.2.2: Figure illustrating frictional resistance over the pipe surface due to axial ground strain.

C-7.3.1 –

For a long straight run segmented pipe, the ground strain is accommodated by combination of pipe strain and relative axial displacement (expansion/contraction) at pipe joints. As per Iwamoto et al. (1984), since the overall axial stiffness for pipe segments are typically much larger than that for the joints, the ground strain results primarily in relative displacement of the joints.

C-7.3.2 –

For conservative estimation, the design joint rotation is multiplied by a safety factor of 1.5.

PROVISIONS

COMMENTARY

$$\theta_{seismic} = 1.5 \frac{A_g}{C^2} L_0$$

Where

A_g = Maximum ground acceleration in direction normal to the direction of propagation of ground wave generated by design earthquake

C = Velocity of seismic wave propagation

= C_s, for S waves, (2.0 km/s may be considered conservatively)

= C_{r_ph}, for R-waves (0.5 km/s may be considered conservatively)

7.3.4 –

The design joint rotation should be less than the allowable joint rotation specified by the manufacturer.

Annex-A: Finite Element Modeling of Buried Pipeline

A.1 – General

The soil-pipe interaction in buried pipelines is a complex problem. During earthquake, the nonlinear behaviour of the soil further increases the complexity. The complexity is mainly attributed to the soil rather than the pipe. While analyzing important pipelines, a detailed finite element model that best represents the nonlinearity of the system is recommended. In practice, several models are used to represent the soil-pipe interaction. They include:

(a) *Continuum model*, where a rigorous mathematical formulation is devised for a flexible pipe of finite length embedded in a semi-infinite soil medium;

(b) *Soil mesh finite element model*, where the complicated nonlinearity of the system is modeled; and

(c) *Beam on Nonlinear Winkler Foundation (BNWF) model*, where the soil is represented by independent springs lumped at discrete locations of the pipe.

Among the above models, BNWF model is extensively used in practice due to its simplicity, mathematical convenience and ability to incorporate nonlinearity. The following section details the BNWF model for continuous and segmented pipelines.

In a BNWF model, The actual three dimensional soil-pipe interactions (Figure A 1.1) can be ideally modeled as a pipe resting on nonlinear soil springs as shown in Figure A 1.2. The pipe can either be modeled as a three dimensional shell element or as a two dimensional beam element depending on the pipeline geometry and loading condition. The soil surrounding the pipe is modeled as nonlinear springs. Basically four types of springs are used to model the surrounding soil as:

a) *Axial soil spring*: It is to represent soil resistance over the pipe surface along its length.

b) *Lateral soil spring*: It is to represent the lateral resistance of soil to the pipe movement.

c) *Vertical bearing spring*: It is to represent the vertical resistance of soil at the bottom of the pipe,

d) *Vertical uplift spring*: It is to represent the vertical resistance of the soil at the top of the pipe.

The properties of above soil springs are detailed in Annex-B.

While analyzing the pipeline for permanent ground deformation (PGD), it is usually assumed that the development of ground deformation is gradual. Hence, pseudo-static analysis is preferred for pipelines subjected to PGD. The ground deformation is assigned at the fixed ends of the soil springs. The damping and inertia effect are ignored in this analysis.

However, while analyzing the pipeline for seismic wave propagation effect, a dynamic analysis is required. In the model, the ground excitation time-history is assigned at the fixed ends of the soil springs. This analysis includes the inertia and damping effect of the system. The effective damping of the soil during earthquake is mainly a function of the shear strain induced in it due to seismic shaking. According to the expected level of ground strain, effective damping of the soil may be specified.

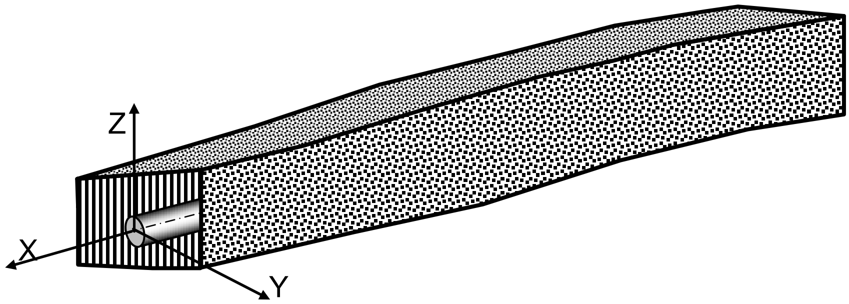


Figure A 1.1: Actual three dimensional soil-pipe interactions.

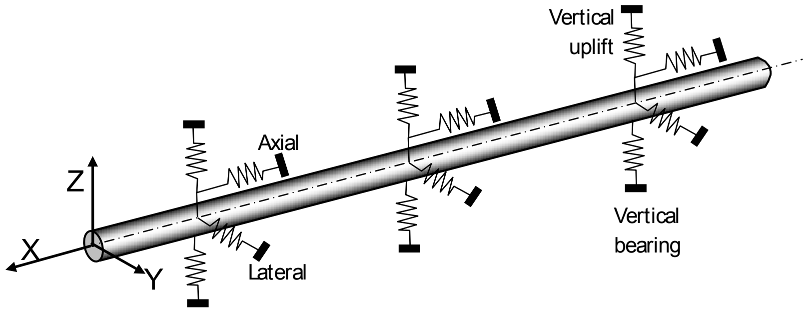


Figure A 1.2: BNWF model representing pipe-soil interaction.

A.2 – Modeling of Continuous Pipeline

In continuous pipelines, the lateral and rotational stiffness of the joint is equal to or more than that of the pipe material. Hence, the continuous pipeline can be treated as a single continuous beam. Schematics of the BNWF model of a continuous pipeline for ground deformation and vibration in both longitudinal and transverse direction are shown in Figure A 2.1, A 2.2, A 2.3 and A 2.4 respectively.

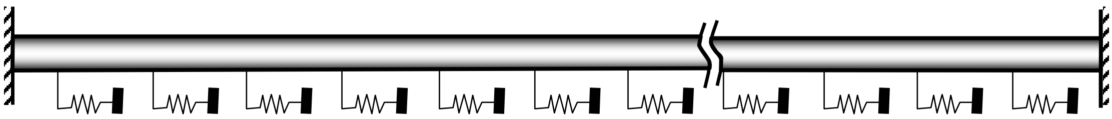


Figure A 2.1: Pipe-soil interaction model for longitudinal ground deformation.

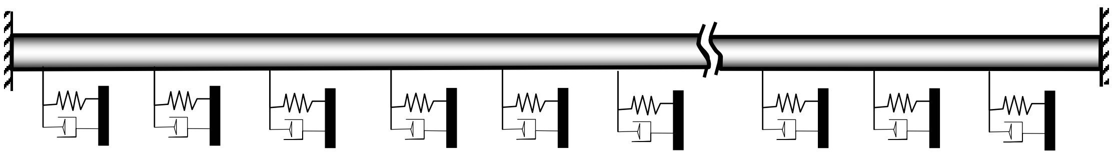


Figure A 2.2: Pipe-soil interaction model for axial vibration.

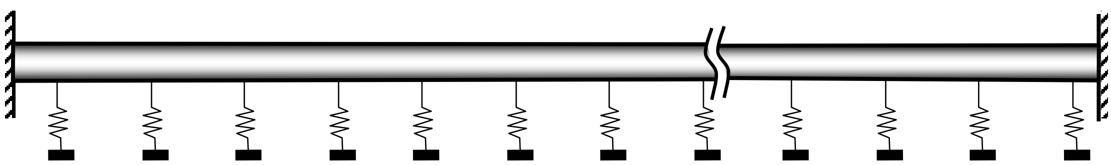


Figure A 2.3: Pipe-soil interaction model for transverse ground deformation.

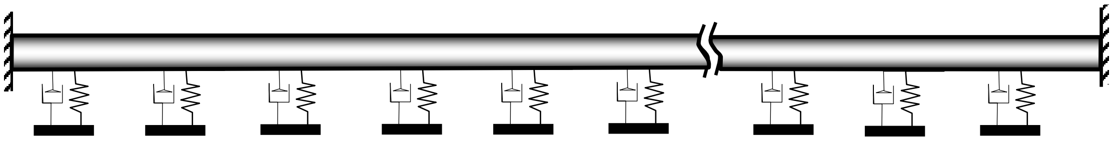


Figure A 2.4: Pipe-soil interaction model for transverse vibration.

A.3 – Modeling of Segmented Pipeline

The segmented pipelines are modeled as small segments of stiff pipes with flexible joints. The joints are modeled as single equivalent axial spring while analyzing for longitudinal PGD or axial vibration. However, while analyzing for transverse PGD or lateral vibration, the joints are modeled as lateral and rotational springs. Schematics of the BNWF model of a segmented pipeline for permanent ground deformation and vibration in both longitudinal and transverse direction are shown in Figure A 3.1, A 3.2, A 3.3 and A 3.4 respectively.

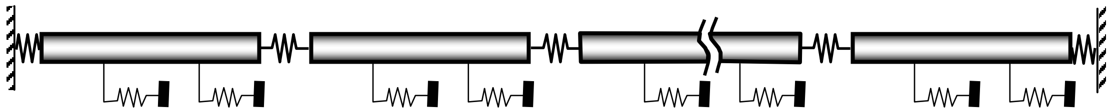


Figure A 3.1: Pipe-soil interaction model for longitudinal ground deformation.

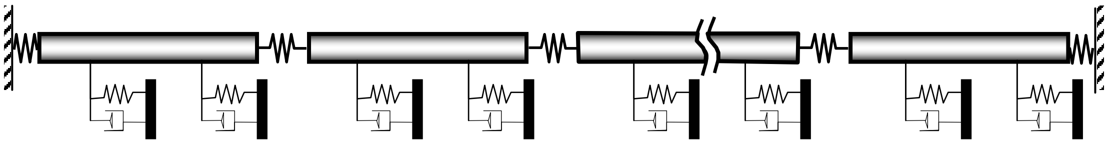


Figure A 3.2: Pipe-soil interaction model for axial vibration.

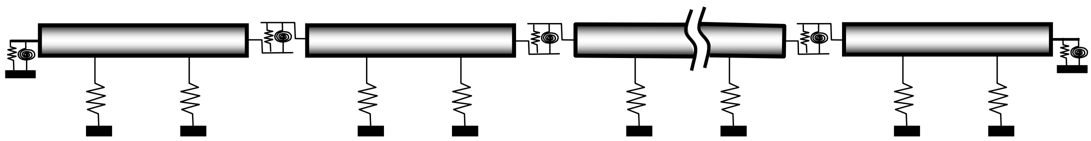


Figure A 3.3: Pipe-soil interaction model for transverse ground deformation.

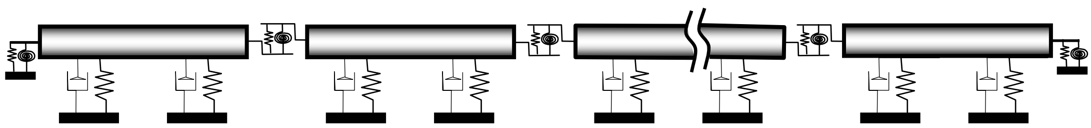


Figure A 3.4: Pipe-soil interaction model pipelines for transverse vibration.

Annex-B: Soil Spring Properties to Model Soil-Pipe Interaction.

B.1 - Axial Soil Spring

The properties of axial soil spring are estimated considering the soil properties of the backfill material used in the pipeline trench. However, this is appropriate only when the response of pipeline movement relative to the surrounding backfill soil is not significantly influenced by the soil outside the trench. Figure B1 shows the idealized representation of the axial soil spring.

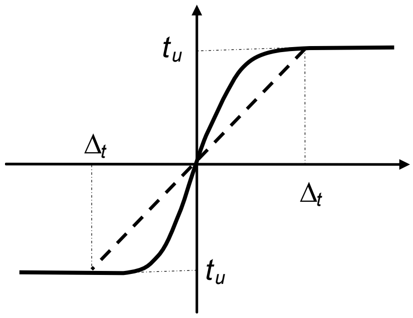


Figure B 1: Idealized representation of axial soil spring

The maximum axial soil resistance (t_u) per unit length of the pipe can be calculated as:

$$t_u = \pi D c \alpha + \pi D H \bar{\gamma} \frac{1+K_0}{2} \tan\delta'$$

where

D = Outside diameter of pipe

c = Coefficient of cohesion of backfill soil

H = Depth of soil above the center of the pipeline

$\bar{\gamma}$ = Effective unit weight of soil

α = Adhesion factor = $0.608 - 0.123c - \frac{0.274}{c^2+1} + \frac{0.695}{c^3+1}$, in which c is in kPa/100

δ' = Interface angle of friction between pipe and soil = $f \times \phi$

ϕ = Internal friction angle of the soil

f = Friction factor for various types of pipes (Table B 1a).

K_o = Coefficient of soil pressure at rest. This may be taken from Table B 1b or may be determined by Jaky's formula as: $K_o = 1 - sin\phi$

The maximum mobilizing displacement of soil (Δ_t) in axial direction of pipe can be taken as:

Δ_t = 3mm for dense sand

= 5mm for loose sand

= 8mm for stiff clay

= 10mm for soft clay

Table B 1a: Friction factor f for various external coatings (ALA, 2001).

Pipe Coating	f
Concrete	1.0
Coal Tar	0.9
Rough Steel	0.8
Smooth Steel	0.7
Fusion Bonded Epoxy	0.6
Polyethylene	0.6

Table B 1b: Values of lateral pressure coefficient at rest (K_o) for different soil conditions.

Type of soil	K_0
Loose soil	0.5 – 0.6
Dense soil	0.3 – 0.5
Clay (drained)	0.5 – 0.6
Clay (undrained)	0.8 – 1.1
Over consolidated soil	1.0 – 1.3

For deep buried pipelines with soil properties varying between the ground surface and the pipeline depth, the equations presented above do not hold good. Special literatures may be followed in such situations.

B.2 - Lateral Soil Spring

The properties of lateral soil spring are estimated considering the native soil at the site. Figure B2 shows the idealized representation of the lateral soil spring.

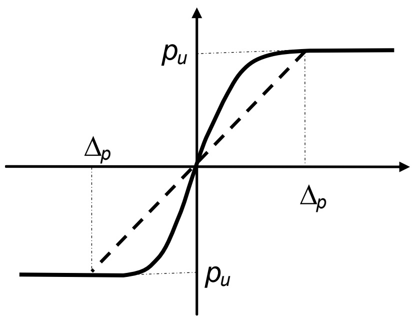


Figure B 2: Idealized representation of lateral soil spring

The maximum lateral resistance of soil per unit length of pipe can be calculated as:

$P_u = N_{ch} cD + N_{qh} \overline{\gamma} HD$,

where

N_{ch} = Horizontal bearing capacity factor for clay, 0 for c = 0 (Table B2),

N_{qh} = Horizontal bearing capacity factor for sandy soil, 0 for ϕ = 0 (Table B2),

$N_{ch} = a + bx + \frac{c}{(x+1)^2} + \frac{d}{(x+1)^3} \le 9$, and

$N_{qh} = a + bx + cx^2 + dx^3 + ex^4$

Where, x = H/D

The displacement Δ_p at P_u is taken as: $\Delta_p = 0.04\left(H + \frac{D}{2}\right) \le 0.01D$ to $0.02D$

Table B2: Lateral bearing capacity factor of soil (ALA, 2001)

Factor	f	***A***	***b***	***C***	***D***	***E***
N_{ch}	0	6.752	0.065	-11.063	7.119	
N_{qh}	20	2.399	0.439	-0.03	1.059×10^{-3}	-1.754×10^{-5}
N_{qh}	25	3.332	0.839	-0.090	5.606×10^{-3}	-1.319×10^{-4}
N_{qh}	30	4.565	1.234	-0.089	4.275×10^{-3}	-9.159×10^{-5}
N_{qh}	35	6.816	2.019	-0.146	7.651×10^{-3}	-1.683×10^{-4}
N_{qh}	40	10.959	1.783	0.045	-5.425×10^{-3}	-1.153×10^{-4}
N_{qh}	45	17.658	3.309	0.048	-6.443×10^{-3}	-1.299×10^{-4}

B.3 - Vertical Soil Spring

The soil spring properties are different for uplift and bearing cases. For bearing soil spring, the properties of native soil at the site may be used. However, for uplift soil spring, the properties of backfill soil are to be considered. Figure B3a shows the idealized representation of the vertical soil spring.

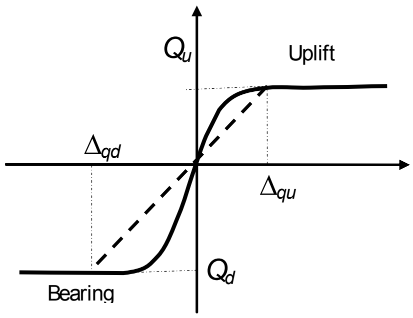


Figure B 3a: Idealized representation of soil springs in vertical direction.

B.3.1 - Vertical Uplift

The maximum soil resistance per unit length of the pipeline in vertical uplift can be calculated as:

$$Q_u = N_{cv} cD + N_{qv} \bar{\gamma} HD$$

where

N_{cv} = Vertical uplift factor for clay (0 for $c = 0$),

N_{qv} = Vertical uplift factor for sand (0 for $\phi = 0^0$),

$N_{cv} = 2\left(\frac{H}{D}\right) \le 10$ for $\left(\frac{H}{D}\right) \le 10$, and

$N_{qv} = \left(\frac{\phi H}{44D}\right) \le N_q$ (see clause B.3.2 for the definition of N_q)

The mobilizing displacement of soil, Δ_{qu}, at Q_u can be taken as:

(a) $0.01H$ to $0.02H$ for dense to loose sands $< 0.1D$, and

(b) $0.1H$ to $0.2H$ for stiff to soft clay $< 0.2D$.

B.3.2 - Vertical Bearing

The maximum soil resistance per unit length of pipeline in vertical bearing can be calculated as

$$Q_d = N_c cD + N_q \bar{\gamma} HD + N_\gamma \gamma \frac{D^2}{2},$$

Where

N_c, N_q and N_γ are bearing capacity factors from Figure B 3b or as

$$N_c = \left[cot(\phi + 0.001)\right]\left\{exp\left[\pi tan(\phi + 0.001)\right]tan^2\left(45 + \frac{\phi + 0.001}{2}\right) - 1\right\},$$

$N_q = exp(\pi tan\phi)tan^2\left(45 + \frac{\phi}{2}\right)$,

$N_\gamma = exp(0.18\phi - 2.5)$, and

γ = Total unit weight of soil

The mobilizing soil displacement, Δ_{qd}, at Q_d can be taken as:

$0.1D$ for granular soils, and

$0.2D$ for cohesive soils.

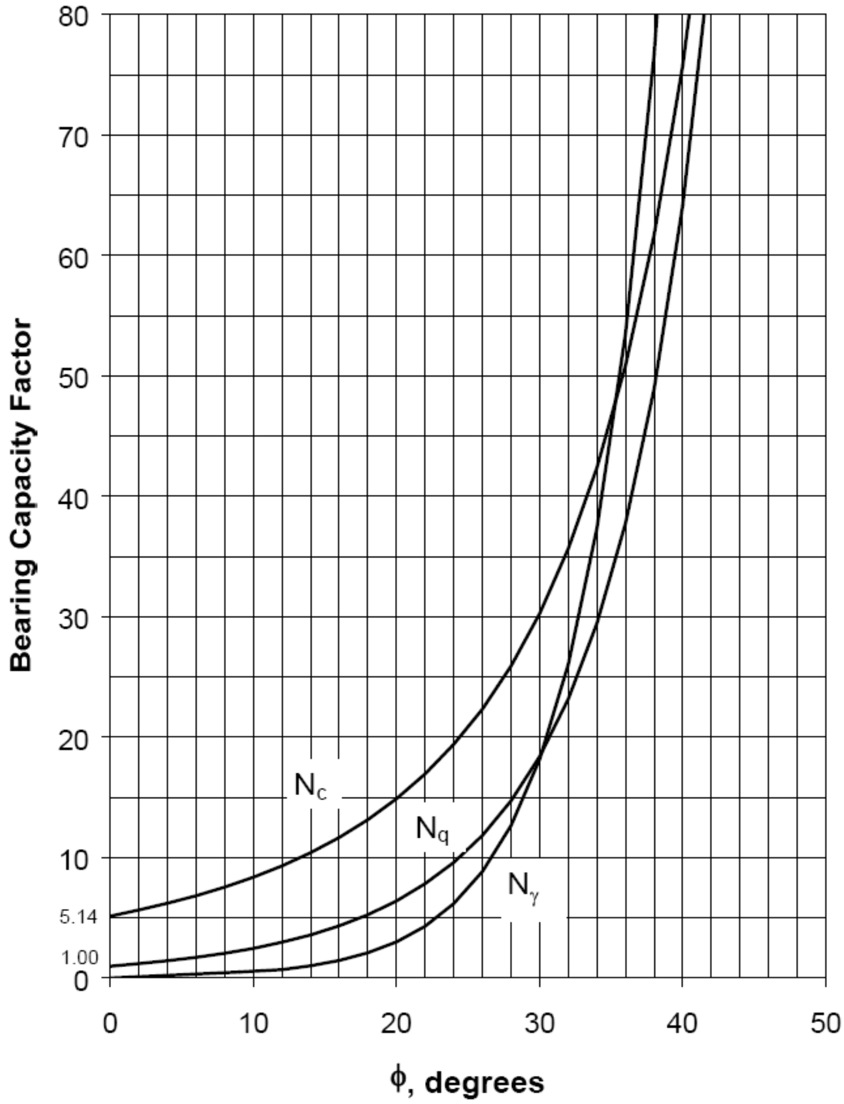


Figure B 3b: Bearing capacity factors of soils of different soil friction values (ALA 2001).

IITK-GSDMA GUIDELINES
for SEISMIC DESIGN
of BURIED PIPELINES

Provisions with Commentary and Explanatory Examples

PART 2: EXPLANATORY EXAMPLES

Index

PART 2: Explanatory Examples

Example - 1	Calculation of operational strain in the pipeline	71
Example – 2	Calculation of soil spring properties to represent pipe-soil interaction	73
Example – 3	Seismic safety evaluation of a continuous oil pipeline	78
Example – 4	Seismic safety evaluation of a segmented water pipeline	84

Example 1 – Calculation of operational strain in the pipeline.

Problem statement:

A buried pipeline is designed to carry natural gas at a pressure (P) of 7.5 MPa. The installation temperature and operating temperature of the pipeline are 30°C and 60°C respectively. The pipe is of API X-52 grade with outer diameter (D) of 0.6m and wall thickness (t) of 0.0064m. Poisson's ratio and coefficient of thermal expansion of the pipe material can be considered as 0.3 and 12×10^{-6} respectively.

Estimate

(a) the strain in a pipe due to internal pressure and temperature if it is a continuous pipeline, and

(b) the joint displacement due to internal pressure and temperature if it is a segmented pipeline ?

Solution:

For API X-52 grade pipe:

Yield stress of pipe material

= σ_y = 358 MPa (Table 3.7.4)

Ramebrg-Osgood parameter (*n*) = 9

Ramberg-Osgood parameter (*r*) = 10

a) Continuous pipeline

Pipe strain due to internal pressure:

The longitudinal stress induced in the pipe due to internal pressure will be (Clause 3.8.1):

$$S_p = \frac{PD\mu}{2t} = \frac{7500000\times0.6\times0.3}{2\times0.0064}$$

= 105.5×10^6 N/m^2 = 105.5 MPa

Using Ramberg-Osgood's stress-strain relationship (clause 3.7.4), the longitudinal strain in the pipe will be:

$$\varepsilon_p = \frac{S_p}{E}\left[1+\frac{n}{1+r}\left(\frac{S_p}{\sigma_y}\right)^r\right]$$

$$= \frac{105.5\times10^6}{2\times10^{11}}\left[1+\frac{9}{10+1}\left(\frac{105.5\times10^6}{358\times10^6}\right)^{10}\right]$$

= 0.00053 = 0.053 % (tensile)

Pipe strain due to temperature change:

The longitudinal stress induced in the pipe due to change in temperature will be (Clause 3.8.1) :

$$S_T = E\alpha(T_2 - T_1)$$

= $2\times10^{11}\times12\times10^{-6}(60-30)$

= 72×10^6 N/m^2 = 72 MPa

Using Ramberg-Osgood's stress-strain relationship (clause 3.7.4), the longitudinal strain in the pipe will be:

$$\varepsilon_t = \frac{S_t}{E}\left[1+\frac{n}{1+r}\left(\frac{S_t}{\sigma_y}\right)^r\right]$$

$$= \frac{72\times10^6}{2\times10^{11}}\left[1+\frac{9}{10+1}\left(\frac{72\times10^6}{358\times10^6}\right)^{10}\right]$$

= 0.00036 = 0.036 % (tensile)

Therefore, the total strain in the continuous pipeline due to internal pressure and temperature is:

= 0.053 + 0.036

= 0.09 %.

Ignoring the stain in pipe due to installation imperfection or initial bending, the above calculated strain can be considered as the operational strain in pipe (i.e., $\varepsilon_{oper} = 0.09\,\%$).

a) Segmented pipeline

For segmented pipeline, the joint displacement can be calculated as:

Δ = strain in pipe × Length of pipe segment

Hence, using the strain calculated above for continuous pipe, the joint displacement of the segmented pipe due to internal pressure and temperature is:

$L_0 \times 0.0009$

Where

L_0 = Length of the pipe segment

Ignoring the joint displacement of pipe due to installation imperfection or initial bending, the above calculated joint displacement can be considered as the operational joint displacement in pipe (i.e., $\Delta_{oper} = L_0 \times 0.0009$).

Example 2 – Calculation of soil spring properties to represent pipe-soil interaction.

Problem statement:

The pipeline as described in Example-1 is buried with a soil cover of 1.2 m up to the center of the pipeline. Calculate the soil spring properties (ultimate soil resistance, and mobilizing soil displacement) to represent the soil-pipe interaction in (a) axial, (b) lateral and (c) vertical direction for the following two site conditions.

Site 1: Medium sandy soil with coefficient of cohesion = 30 kPa, angle of friction (ϕ) = 30° and effective unit weight of 18 kN/m^3.

Site 2: Sandy soil with an angle of friction (ϕ) = 32° and effective unit weight of 18 kN/m^3.

Solution:

Site 1:

(a) Axial soil spring:

The maximum axial soil resistance (t_u) per unit length of the pipe can be calculated as (clause B.1, Annex B):

$$t_u = \pi D c \alpha + \pi D H \bar{\gamma}\left(\frac{1+K_0}{2}\right)\tan\delta'$$

Where:

D = Diameter of pipe = 0.6 m

c = Coefficient of cohesion = 30 kPa

α = Adhesion Factor

$$= 0.608 - 0.123c - \frac{0.274}{c^2+1} + \frac{0.695}{c^3+1} = 0.9964$$

H = Soil cover above the center of the pipeline = 1.2 m

$\bar{\gamma}$ = Effective unit weight of soil = 18000 N/m^3

Interface angle of friction between soil and pipe = $\delta' = f\phi$

Where,

f = friction factor = 0.7 for smooth steel pipe (Table B.1a, Annex B)

ϕ = Internal friction angle of soil = 30°

Hence, $\delta' = f\phi = 0.7 \times 30° = 21°$

K_0 = Coefficient of soil pressure at rest

= 1 - sin30° = 0.5

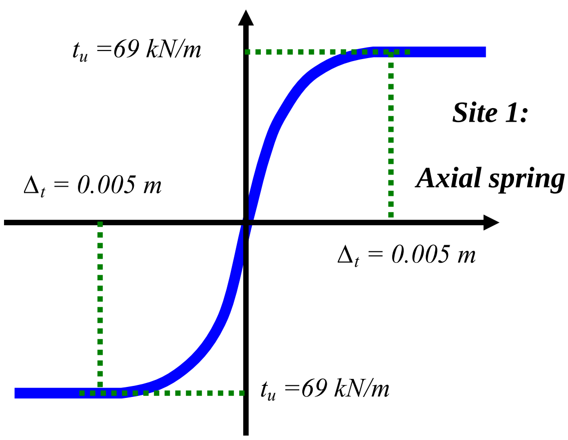
Hence,

$$t_u = (\pi \times 0.6 \times 30000 \times 0.9964) + \left(\pi \times 0.6 \times 1.2 \times 18000 \times \frac{1+0.5}{2}\tan 21°\right)$$

= 68975 N/m = 69 kN/m

The mobilizing soil displacement in axial direction can be taken as:

Δ_t = 5 mm = 0.005 m (clause B1, Annex B)



(b) Lateral soil spring:

The maximum transverse soil resistance per unit length of pipe is (clause B.2, Annex B):

$$P_u = N_{ch} cD + N_{qh} \bar{\gamma} HD$$

Where

N_{ch} = Horizontal bearing capacity factor for clay

$$N_{ch} = a + bx + \frac{c}{(x+1)^2} + \frac{d}{(x+1)^3} \leq 9$$

Where, $x = \frac{H}{D} = \frac{1.2}{0.6} = 2$,

a = 6.752,

b = 0.065,

c = -11.063, and

d = 7.119.

(See Table B2, Annex B)

Hence,

$$N_{ch} = 6.752 + (0.065 \times 2) + \left(\frac{-11.063}{(2+1)^2}\right) + \left(\frac{7.119}{(2+1)^3}\right)$$

= 5.916

N_{qh} = Horizontal bearing capacity factor for sandy soil

$$N_{qh} = a + bx + cx^2 + dx^3 + ex^4$$

Where

$x = \frac{H}{D} = \frac{1.2}{0.6} = 2$,

a = 4.565,

b = 1.234,

c = -0.089,

d = 4.275 × 10^{-3}

e = -9.159 × 10^{-5}

(See table B2, Annex B for the values of a, b, c, d and e for ϕ = 30°)

Hence,

$$N_{qh} = 4.565 + (1.234 \times 2) + (-0.089 \times 2^2) + (4.275 \times 10^{-3} \times 2^3) + (-9.159 \times 10^{-5} \times 2^4)$$

= 6.709

Hence,

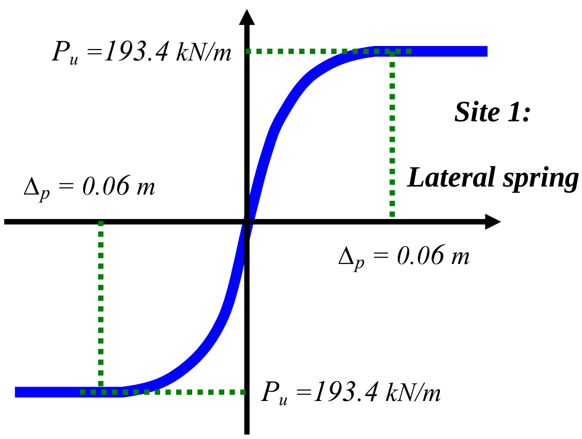
$$P_u = (5.916 \times 30000 \times 0.6) + (6.709 \times 18000 \times 1.2 \times 0.6)$$

= 193436 N/m = 193.4 kN/m

The mobilizing displacement of soil in lateral direction can be calculated as (clause B2, Annex B):

$$= 0.04\left(H + \frac{D}{2}\right) = 0.04 \times \left(1.2 + \frac{0.6}{2}\right)$$

= 0.06 m



(c) Vertical soil spring

Uplift:

The maximum uplift soil resistance per unit length of pipe is (clause B3.1, Annex B):

$$Q_u = N_{cv} cD + N_{qv} \bar{\gamma} HD$$

Where

N_{cv} = Vertical uplift factor for clay

$$N_{cv} = 2\left(\frac{H}{D}\right) \leq 10$$

Hence, $N_{cv} = 2 \times \frac{1.2}{0.6} = 4$

N_{qv} = Vertical uplift factor for sand

$$N_{qv} = \left(\frac{\phi H}{44D}\right) \leq N_q$$

Hence, $N_{qv} = \left(\dfrac{30\times1.2}{44\times0.6}\right) = 1.364$

Hence,

$Q_u = (4\times30000\times0.6) +$
$(1.364\times18000\times1.2\times0.6)$

= 89677 N/m = 89.7 kN/m

The mobilizing displacement of soil in vertical uplift can be calculated as:

$\Delta_u = 0.15\times H = 0.15\times1.2 = 0.18$ m

Bearing:

The maximum bearing soil resistance per unit length of pipe is (clause B3.2, Annex B):

$$Q_d = N_c cD + N_q\bar{\gamma}HD + N_\gamma\gamma\frac{D^2}{2}$$

Where $N_c = 30$, $N_q = 18$, $N_\gamma = 18$

(See Figure B3b, Annex-B)

Hence,

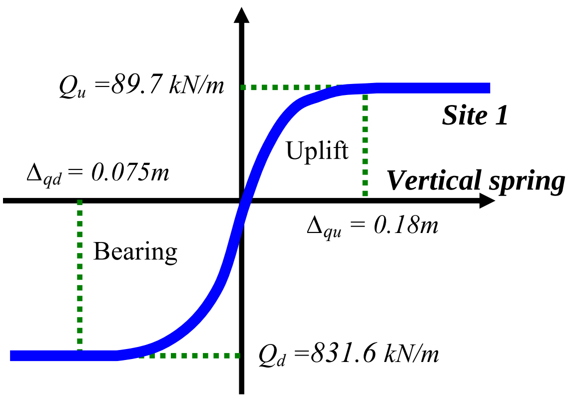
$Q_d = (30\times30000\times0.6) + (18\times18000\times1.2\times0.6)$
$+\left(18\times18000\times\dfrac{0.6^2}{2}\right)$

= 831600 N/m = 831.6 kN/m

The mobilizing soil displacement in vertical bearing can be calculated as:

$\Delta_d = 0.125\times D = 0.125\times0.6$

= 0.075 m



Site 2:

(a) Axial soil spring:

The maximum axial soil resistance (t_u) per unit length of the pipe can be calculated as (clause B.1, Annex B):

$$t_u = \pi Dc\alpha + \pi DH\bar{\gamma}\left(\frac{1+K_0}{2}\right)\tan\delta'$$

For sandy soil (i.e. c = 0), t_u can be rewritten as:

$$t_u = \pi DH\bar{\gamma}\left(\frac{1+K_0}{2}\right)\tan\delta'$$

Where:

D = Diameter of pipe = 0.6 m

H = Soil cover above the center of the pipeline = 1.2 m

$\bar{\gamma}$ = Effective unit weight of soil = 18000 N/m^3

Interface angle of friction between soil and pipe = $\delta' = f\phi$

Where,

f = friction factor = 0.7 for smooth steel pipe (Table B.1a, Annex B)

ϕ = Internal friction angle of soil = 32°

Hence, $\delta' = f\phi = 0.7\times32° = 22.4°$

K_0 = Coefficient of soil pressure at rest

= 1 - sin32° = 0.47

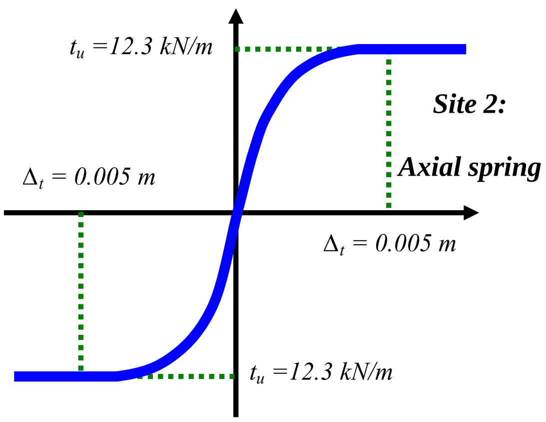
Hence,

$$t_u = \left(\pi\times0.6\times1.2\times18000\times\frac{1+0.47}{2}\tan22.4°\right)$$

= 12334 N/m = 12.3 kN/m

The mobilizing soil displacement in axial direction can be taken as:

Δ_t = 5 mm = 0.005 m (clause B1, Annex B)



(b) Lateral soil spring:

The maximum transverse soil resistance per unit length of pipe is (clause B.2, Annex B):

$$P_u = N_{ch}cD + N_{qh}\bar{\gamma}HD$$

For sandy soil (i.e. c = 0), P_u can be rewritten as:

$$P_u = N_{qh}\bar{\gamma}HD$$

Where

N_{qh} = Horizontal bearing capacity factor for sandy soil

$$N_{qh} = a + bx + cx^2 + dx^3 + ex^4$$

Where

$$x = \frac{H}{D} = \frac{1.2}{0.6} = 2\,,$$

a = 5.465,

b = 1.548,

c = -0.1118,

d = 5.625 × 10^{-3}

e = -1.2227 × 10^{-5}

(See table B2, Annex B for the values of a, b, c, d and e for ϕ = 32°)

Hence,

$$N_{qh} = 5.465 + (1.548 \times 2) + (-0.1118 \times 2^2) + (5.625 \times 10^{-3} \times 2^3) + (-1.2227 \times 10^{-4} \times 2^4)$$

= 8.156

Hence,

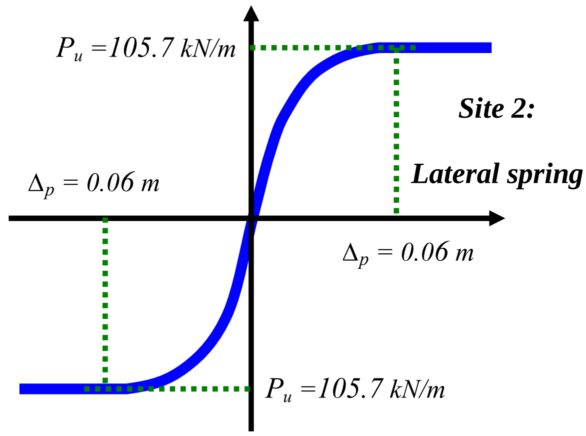
$$P_u = (8.156 \times 18000 \times 1.2 \times 0.6)$$

= 105702 N/m = 105.7 kN/m

The mobilizing displacement of soil in lateral direction can be calculated as (clause B2, Annex B):

$$= 0.04\left(H + \frac{D}{2}\right) = 0.04 \times \left(1.2 + \frac{0.6}{2}\right)$$

= 0.06 m



(c) Vertical soil spring

Uplift:

The maximum uplift soil resistance per unit length of pipe is (clause B3.1, Annex B):

$$Q_u = N_{cv}cD + N_{qv}\bar{\gamma}HD$$

For sandy soil (i.e. c = 0), Q_u can be rewritten as:

$$Q_u = N_{qv}\bar{\gamma}HD$$

Where

N_{qv} = Vertical uplift factor for sand

$$N_{qv} = \left(\frac{\phi H}{44D}\right) \le N_q$$

$$= \left(\frac{32 \times 1.2}{44 \times 0.6}\right) = 1.455$$

Hence,

$$Q_u = (1.455 \times 18000 \times 1.2 \times 0.6)$$

= 18857 N/m = 18.9 kN/m

The mobilizing displacement of soil in vertical uplift can be calculated as:

$$\Delta_{qu} = 0.15 \times H = 0.15 \times 1.2 = 0.18 \text{ m}$$

Bearing:

The maximum bearing soil resistance per unit length of pipe is (clause B3.2, Annex B):

$$Q_d = N_c cD + N_q \bar{\gamma} HD + N_\gamma \gamma \frac{D^2}{2}$$

For sandy soil (i.e. c = 0), Q_u can be rewritten as:

$$Q_d = N_q \bar{\gamma} HD + N_\gamma \gamma \frac{D^2}{2}$$

Where $N_q = 23$, $N_\gamma = 25$

(See Figure B3b, Annex-B, for ϕ=32°)

Hence,

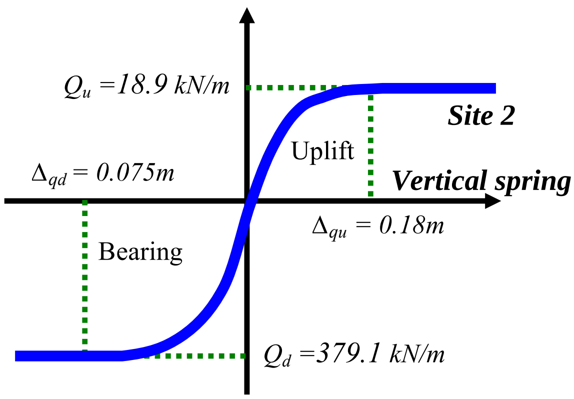
$$Q_d = (23 \times 18000 \times 1.2 \times 0.6) + \left(25 \times 18000 \times \frac{0.6^2}{2}\right)$$

= 379080 N/m = 379.1 kN/m

The mobilizing soil displacement in vertical bearing can be calculated as:

$$\Delta_{qd} = 0.125 \times D = 0.125 \times 0.6$$

= 0.075 m



Summary of results

Soil site	Direction of pipe movement		Maximum soil resistance (kN/m)	Mobilizing soil displacement (m)
Site 1	Axial		$t_u = 69$	$\Delta_t = 0.005$
	Lateral		$P_u = 193.4$	$\Delta_p = 0.06$
	Vertical	Uplift	$Q_u = 89.7$	$\Delta_{qu} = 0.18$
	Vertical	Bearing	$Q_d = 831.6$	$\Delta_{qd} = 0.075$
Site 2	Axial		$t_u = 12.3$	$\Delta_t = 0.005$
	Lateral		$P_u = 105.7$	$\Delta_p = 0.06$
	Vertical	Vertical	$Q_u = 18.9$	$\Delta_{qu} = 0.18$
	Vertical	Vertical	$Q_d = 379.1$	$\Delta_{qd} = 0.075$

Example 3 – Seismic safety evaluation of a continuous oil pipeline.

Problem statement:

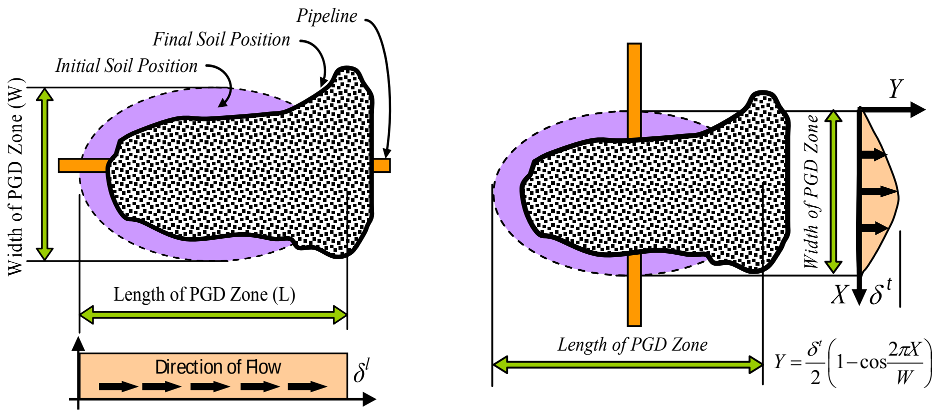
A continuous buried pipeline is designed to carry natural gas at a pressure (P) of 7.5 MPa. The installation temperature and operating temperature of the pipeline are 30°C and 60°C respectively. The pipe is of API X-52 grade with 0.6 m diameter (D) and 0.0064 m wall thickness (t). The pipeline is buried at 1.2 m of soil cover. Check the pipeline safety for the following conditions.

Case I: Permanent ground displacement (PGD)

The length and width of PGD zone is 100 m and 40 m respectively. The pipeline in PGD zone encounters the soil same as that of Site 2, as described in Example 2. The ground displacement (δ^l and δ^t) due to liquefaction can be taken as 2m. Check the pipeline safety if it is oriented

i) parallel to the direction of ground movement (Figure E3.1 (a)), and

ii) transverse to the direction of ground movement (Figure E3.1 (b).



(a) **(b)**

Figure E3.1: a) Pipeline crossing parallel to the ground movement. **b)** Pipeline crossing transverse to the ground movement.

Case II: Buoyancy due to liquefaction

The pipe is expected to experience buoyancy force during liquefaction over a length of about 40 m. The unit weight of saturated soil at the site is 18 kN/m^3. Assume the water table to be 1 m below the surface layer.

Case III: Fault crossing

The pipeline crosses a normal slip fault with fault displacement of 2.5 m and a dip angle of 35° (Figure E3.2). The pipeline crosses the fault line at an angle of 40°. The source to site distance can be considered as 20km. The pipeline in fault zone encounters the soil same as that of Site 1, as described in Example 2.

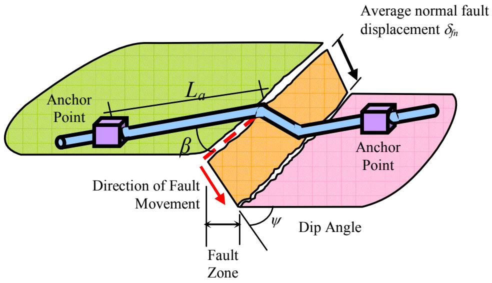


Figure E3.2: Pipeline crossing a normal slip fault

Case IV: Seismic wave propagation

The expected peak ground acceleration (PGA) in the site is 0.45g at the base rock layer. The soil at the site can be taken as the same soil as that of Site 1 as given in Example 1.

Solution:

General:

For API X-52 Grade pipe:

Yield stress of pipe material $= \sigma_y = 358$ MPa (Table 3.7.4)

Ramberg-Osgood parameters (Table 3.7.4) $n = 9$, and $r = 10$.

Class of soil is: E (Table 3.3)

Class of pipeline is: I (clause 3.2)

The operational strain in pipeline = 0.09% (tensile) (See Example 1)

Case I: Permanent Ground Displacement (PGD):

i) Parallel crossing (i.e., Longitudinal PGD)

The expected amount of permanent ground movement parallel to pipe axis = $\delta^l = 2$ m

Length of permanent ground displacement zone is = 100 m

The design ground movement =

$\delta^l{}_{design} = \delta^l \times I_p$ (clause 4.1)

Where

I_p = Importance factor (Table 3.5.2)

For pipeline of class-1, $I_p = 1.5$

$$\delta^l{}_{design} = 2 \times 1.5 = 3\,\text{m}$$

As described in clause 4.1.1.1, two types of longitudinal PGD model are used.

According to Case-1:

The amount of ground movement ($\delta^l{}_{design}$) is considered to be large and the pipe strain is controlled by length (L) of permanent ground deformation zone. The peak pipe strain (tensile/compressive) for this case is calculated as: (clause 4.1.1.2)

$$\varepsilon_a = \frac{t_u L}{2\pi D t E}\left[1 + \frac{n}{1+r}\left(\frac{t_u L}{2\pi D t \sigma_y}\right)^r\right]$$

Where

t_u = Maximum axial soil force per unit length of pipe for 'Site 2' soil condition (see Example 2) = 12334 N/m

Hence,

$$\varepsilon_a = \frac{12334 \times 100}{2 \times \pi \times 0.6 \times 0.0064 \times \left(2 \times 10^{11}\right)}$$
$$\left[1 + \frac{9}{1+10}\left(\frac{12334 \times 100}{2 \times \pi \times 0.6 \times 0.0064 \times 358 \times 10^6}\right)^{10}\right]$$

= 0.00025 = 0.025%

According to Case-2:

The length (L) of permanent ground deformation zone is large, and the pipe strain is controlled by the amount of ground movement ($\delta^l{}_{design}$) (clause 4.1.1.3). The peak pipe strain (tensile/compressive) for this case is calculated as:

$$\varepsilon_a = \frac{t_u L_e}{2\pi DtE}\left[1 + \frac{n}{1+r}\left(\frac{t_u L_e}{2\pi Dt\sigma_y}\right)^r\right]$$

Where

L_e = Effective length of pipeline over which the friction force (t_u) acts, which can be calculated by the following equation.

$$\delta^l{}_{design} = \frac{t_u {L_e}^2}{2\pi DtE}\left[1 + \left(\frac{2}{2+r}\right)\left(\frac{n}{1+r}\right)\left(\frac{t_u L_e}{2\pi Dt\sigma_y}\right)^r\right]$$

Hence, the effective length of pipeline is calculated as: L_e = 445 m

Hence,

$$\varepsilon_a = \frac{12334 \times 445}{2 \times \pi \times 0.6 \times 0.0064 \times 2 \times 10^{11}}$$
$$\left[1 + \frac{9}{1+10}\left(\frac{12334 \times 445}{2 \times \pi \times 0.6 \times 0.0064 \times 358 \times 10^6}\right)^{10}\right]$$

= 0.001147

The design strain in pipe is taken as the least value between the two cases (clause 4.1.1.4) = $\varepsilon_{seismic}$ = 0.00025 (tensile/compressive)

The operational strain in the pipeline (Example 1) = ε_{oper} = 0.0009

Hence,

total tensile strain in the pipeline

= 0.00025 + 0.0009 = 0.00115, and

total compressive strain = 0.00025-0.0009 = - 0.00065 (i.e. total strain is tensile)

The limiting strain in tension for permanent ground deformation is (clause 3.9) = 3 % = 0.03

Hence the total strain in pipe due to longitudinal strain is less than the allowable strain.

ii) Transverse crossing (i.e., Transverse PGD)

The expected amount of transverse permanent ground deformation (δ^t) = 2 m

The design transverse ground displacement (clause 4.2) $= \delta^t{}_{design} = \delta^t \times I_p = 2 \times 1.5 = 3$ m

The maximum bending strain in the pipe is calculated as the least value of the following two (clause 4.2.1.1).

a) $\varepsilon_b = \pm \dfrac{\pi D \delta^t{}_{design}}{W^2}$

$= \pm \dfrac{\pi \times 0.6 \times 1.5}{40^2} = \pm 0.001767$, and

b) $\varepsilon_b = \pm \dfrac{P_u W^2}{3\pi EtD^2}$

Where,

P_u = maximum resistance of soil in transverse direction for ‘Site 2’ soil condition (see Example 2) = 105702 N/m

Hence,

$$\varepsilon_b = \pm \frac{10572 \times 40^2}{3 \times \pi \times 2 \times 10^{11} \times 0.0064 \times 0.6^2}$$

$= 0.00389$

Hence, the maximum strain induced in the pipeline due to transverse PGD is taken as = $\varepsilon_{seismic} = \pm 0.001767$ (tensile/compressive)

The operational strain in the pipeline (Example 1) = ε_{oper} = 0.0009

Hence,

total longitudinal strain in the pipe in tension = 0.001767+0.0009 = 0.00267

total longitudinal strain in the pipe in compression = 0.001767-0.0009 = 0.000867

The allowable strain in tension for permanent ground deformation is (clause 3.9) = 3 % = 0.03

The allowable strain in compression for steel pipe is (clause 3.9):

$$= \varepsilon_{cr-c} = 0.175\frac{t}{R} = 0.175\frac{0.0064}{0.3}$$

= 0.00373

The total strain in pipe due to transverse PGD is less than the allowable strain for both tension and compression.

Case II: Buoyancy due to liquefaction:

The net upward force per unit length of pipeline can be calculated as:

The extent of liquefaction = L_b = 40m

$$F_b = \frac{\pi D^2}{4}\left(\gamma_{sat} - \gamma_{content}\right) - \pi D t \gamma_{pipe}$$

$$F_b = \frac{\pi \times 0.6^2}{4}\left(18000 - 0\right) - \pi \times 0.6 \times 0.0064 \times 78560 = 4142 N/m$$

It is assumed that the weight of gas flowing through pipe has negligible weight. The unit weight of steel pipe (γ_{pipe}) is taken as 78560 N/m^3.

The bending stress in the pipeline due to uplift force (F_b) can be calculated as (clause 5.2.1):

$$\sigma_{bf} = \pm\frac{F_b L_b^{\,2}}{10Z}$$

Where

L_b = Length of pipe in buoyancy zone

Z = Section modulus of pipe cross section

$$= \frac{\pi}{32}\frac{(0.6^4 - 0.5872^4)}{0.6} = 0.001752 \text{ m}^4$$

$$\sigma_{bf} = \pm\frac{4142 \times 40^2}{10 \times 0.001752} = 378 \times 10^6 \text{ N/m}^2$$

Maximum strain in pipe corresponding to the above bending stress can be evaluated as (clause 5.2.2):

$$\varepsilon = \frac{\sigma_{bf}}{E}\left[1 + \frac{n}{1+r}\left(\frac{\sigma_{bf}}{\sigma_y}\right)^r\right]$$

$$= \frac{378 \times 10^6}{2 \times 10^{11}}\left[1 + \frac{9}{1+10}\left(\frac{378 \times 10^6}{358 \times 10^6}\right)^{10}\right]$$

= 0.00455 = 0.45% (tensile/compressive)

The operational strain in the pipeline (Example 1) = ε_{oper} = 0.0009

Hence,

total longitudinal strain in the pipe in tension = 0.00455+0.0009 = 0.00545

total longitudinal strain in the pipe in compression = 0.00455-0.0009 = 0.00365

The allowable strain in pipe in tension is (clause 3.9) = 3 % = 0.03

The allowable strain in pipe in compression is (clause 3.9):

$$= \varepsilon_{cr-c} = 0.175\frac{t}{R} = 0.175\frac{0.0064}{0.3}$$

= 0.00373 = 0.37%

The maximum strain in the pipeline due to buoyancy effect is less than the allowable strain for steel pipes in tension and compression.

Case III: Fault crossing

The expected normal-slip fault displacement = δ_{fn} = 2.5 m

Dip angle of the fault movement = ψ = 35°

The angle between pipeline and fault line $= \beta = 40°$

Component of fault displacement in the axial direction of the pipeline (clause 6.1.3):

$\delta_{fax} = \delta_{fn} \cos\psi \sin\beta$

= 2.5 × cos 35° × sin 40° = 1.316 m

Component of fault displacement in transverse direction of pipeline:

$\delta_{ftr} = \delta_{fn} \cos\psi \cos\beta$

= 2.5 × cos 35° × cos 40° = 1.569 m

Importance factor for fault movement for pipe class-I = $I_p = 2.3$ (Table 3.5.1)

Applying importance factor,

the design fault displacement in axial direction becomes (clause 6.1.6):

$= \delta_{fax\text{-}design} = \delta_{fax} \times I_p = 1.316 \times 2.3 = 3.03$ m

and the design fault displacement in transverse direction becomes (clause 6.1.6):

$= \delta_{ftr\text{-}design} = \delta_{ftr} \times I_p = 1.569 \times 2.3 = 3.6$ m

The average pipe strain due to fault movement in axial direction can be calculated as (clause 6.2.1):

$$\varepsilon = 2\left[\frac{\delta_{fax-design}}{2L_a} + \frac{1}{2}\left(\frac{\delta_{ftr-design}}{2L_a}\right)^2\right]$$

Where

L_a = effective unanchored length of the pipeline in the fault zone (clause 6.2.2).

$$L_a = \frac{E_i \varepsilon_y \pi D t}{t_u} = \frac{2\times10^{11}\times0.002\times\pi\times0.6\times0.0064}{68975}$$

= 70 m

Or

L_a = the actual length of anchorage = 100m

Hence, the anchored length to be considered is the lower of the above two values (clause 6.2.2). So L_a = 70 m

Hence the axial strain in the pipe =

$$\varepsilon = 2\times\left[\frac{3.03}{2\times70} + \frac{1}{2}\left(\frac{3.6}{2\times70}\right)^2\right]$$

= 0.044 = 4.4% (tensile)

The operational strain in the pipeline (Example 1) = ε_{oper} = 0.0009

Hence total strain in pipe in tension = 0.044 + 0.0009 = 0.0449 = 4.5%

The allowable strain in pipe in tension is (clause 3.9) = 3 % = 0.03

The total tensile strain in pipe due to fault crossing is more than the allowable strain. The strain in pipe has to be minimized by any suitable means.

One of the possible solutions in this case is to redesign the pipeline with increased thickness. One may also use high ductile pipes that allows higher strain limit.

Case IV: Seismic wave propagation

The expected peak ground acceleration of the site at base rock layer = PGA_r = 0.45g

For soil class E, peak ground acceleration (*PGA)* at ground = 0.45g × I_g

= 0.45g × 0.9 =0.405 g (Table 3.5.3 (b))

Converting PGA to PGV using the correlation given in table 3.5.5

$$\frac{PGV}{PGA} = 140$$

(Considering the soil as soft and the magnitude of design basis earthquake (M) is equals to 6.5, and distance of site from earthquake source is about 20km)

Hence, PGV = 0.405 × 140 =56.7 cm/s

Design peak ground velocity (clause 7.1.1) $= V_g = PGV \times I_p$

Where

$I_p = 1.5$ (Table 3.5.2)

Hence $V_g = 56.7 \times 1.5 = 85.05\,\text{cm/sec}$

= 0.85 m/s

Maximum axial strain in the pipe due to wave velocity can be calculated as (clause 7.2.1):

$$= \varepsilon_a = \frac{V_g}{\alpha_\varepsilon C} = \frac{0.85}{2 \times 2000} = 0.00021$$

Where

C = Velocity of Seismic wave propagation.

= 2 km/s (assuming shear wave velocity effect is dominating) (clause 7.1.2 and 7.2.1)

α_ε = Ground strain coefficient = 2.0 (clause 7.2.1)

Maximum axial strain that can be transmitted by soil friction can be calculated as (clause 7.2.2):

$$\varepsilon_a = \frac{t_u \lambda}{4AE} = \frac{68975 \times 1000}{4 \times 0.0119 \times 2 \times 10^{11}}$$

= 0.0072

Where

$$t_u = \pi D \alpha c + \pi D H \bar{\gamma} \left(\frac{1 + K_0}{2} \right) \tan \delta$$

t_u = 68975 N/m (see Example-2)

Cross sectional area (A) of the pipeline is:

$$= A = \frac{\pi}{4}\left(0.6^2 - 0.5872^2\right) = 0.0119\,\text{m}^2$$

λ = Apparent wavelength of seismic waves at ground surface (clause 7.2.2) = 1000 m

The calculated axial strain due to wave passage (i.e., 0.00021) need not be larger than the strain transmitted by soil friction (i.e., 0.0072).

The operational strain in the pipeline (Example 1) = ε_{oper} = 0.0009

Hence total strain in pipe in tension = 0.00021 + 0.0009 = 0.0011 = 0.1%

The allowable strain in pipe in tension is (clause 3.9) = 3 % = 0.03

The maximum strain in pipe due to wave propagation pipe is less than the allowable strain.

Summary of results

Case	Maximum strain in pipe in tension	Maximum strain in pipe in compression	Allowable strain in pipe in tension	Allowable strain in pipe in compression	Safe/Unsafe
I	0.00267	0.000867	0.03	0.00373	Safe
II	0.00545	0.00365	0.03	0.00373	Safe
III	0.0449	-	0.03	0.00373	Unsafe
IV	0.0011	-	0.03	0.00373	Safe

Example 4 – Seismic safety evaluation of a segmented water pipeline.

Problem statement:

A segmented buried pipeline is designed along a gentle sloped ground. It is intended to carry water at an internal pressure of 3 MPa. The pipeline is a secondary water supply pipeline constructed with bell and spigot joint, and the length of each pipe segment is 6 m .The pipe is of API Grade-B material with 0.6 m diameter (D) and 0.0064 m of wall thickness (t). The burial depth of pipeline is 1.2 m up to the center of the pipeline. Design the pipe joint for the following conditions:

Case I: Permanent ground displacement (PGD)

The length and width of PGD zone is 100 m and 40 m respectively. The pipeline in PGD zone encounters the soil same as that of Site 2, as described in Example 2. The ground displacement (δ^l and δ^t) due to liquefaction can be taken as 2m. Design the pipe joint for the conditions where the pipeline crosses

i) parallel to the direction of ground movement, and

ii) transverse to the direction of ground movement.

Case II: Fault crossing

The pipeline crosses a normal slip fault with fault displacement of 2.5 m and a dip angle of 35° (Figure E3.2). The pipeline crosses the fault line at an angle of 40°. The source to site distance can be considered as 20km. The pipeline in fault zone encounters the soil same as that of Site 1, as described in Example 2.

Case III: Seismic wave propagation

The expected peak ground acceleration (PGA) in the site is 0.45g at the base rock layer. The soil at the site can be taken as the same soil as that of Site 1 as given in Example 1.

Solution:

General:

For API Grade B pipe (Table 3.7.4):

Yield stress of pipe material $= \sigma_y = 227$ MPa

Ramberg-Osgood parameters: $n = 10$, and $r = 100$.

Class of soil is: E (Table 3.3)

Class of pipeline is: II (clause 3.2)

The operational pipe joint displacement pipeline = Δ_{oper} = 6 × 0.0009 = 0.0054m (tensile) (See Example 1)

Where,

Length of each segment of pipeline = 6 m

Case I: Permanent Ground Displacement (PGD):

i) Parallel crossing (i.e., Longitudinal PGD)

The expected amount of permanent ground movement parallel to pipe axis = $\delta^l = 2$ m

Length of permanent ground displacement zone is = 100 m

The design ground movement =

$\delta^l{}_{design} = \delta^l \times I_p$ (clause 4.1)

Where

I_p = Importance factor (Table 3.5.2)

For pipeline of class-II, I_p = 1.35

Hence the design longitudinal ground displacement is:

$= \delta^l{}_{design} = 2 \times 1.35 = 2.7$ m

The joint displacement due to expected ground movement =

$\Delta_{seismic} = \delta^l{}_{design}$ (clause 4.1.2.1)

The operational joint displacement in the pipe segment = $\Delta_{oper} = 0.0054$ m

Hence the total joint displacement due to both seismic action and operational pressure and temperature is:

= 2.7 + 0.0054 = 2.705 m

Considering the additional joint displacement of 0.6cm, the total joint displacement becomes (clause 3.10.2):

= 2.705 + 0.006 = 2.711 m

As the ground displacement is large, chained joint is advisable to be provided in the PGD zone.

No of joints to be provided near the head and toe of the PGD zone are:

$$= \frac{L}{2L_a} = \frac{100}{2 \times 6} = 8.3 \cong 8$$

Hence, each joint in the head and toe region should be designed for a displacement absorption capacity of $\frac{2.711}{8} = 0.34$ m = 34 cm (clause 4.1.2.3).

Along with the designed joints in the PGD zone, three extra restrained joints with 34cm displacement absorption capacity should be provided outside the PGD zone boundaries (clause 4.1.2.3).

ii) Transverse crossing (i.e., Transverse PGD)

The expected amount of transverse permanent ground deformation (δ^t) = 2 m

The design transverse ground displacement (clause 4.2)

$= \delta^t{}_{design} = \delta^t \times I_p = 2 \times 1.35 = 2.7$ m

Where

I_p = Importance factor = 1.35 (Table 3.5.2)

Hence the design transverse ground displacement is:

$= \delta^t{}_{design} = 2.7m$

According to clause 4.2.2.1, the maximum joint opening of the pipe due to transverse PGD is:

$$\Delta_{seismic} = \frac{\pi^2 L_0 \delta^t{}_{design}{}^2}{2W^2}\left[1+\left(\frac{D}{\delta^t{}_{design}}\right)^2\right]$$

For $\frac{D}{\delta^t{}_{design}} = \frac{0.6}{2.7} = 0.22$

Hence,

$$\Delta_{seismic} = \frac{\pi^2 \times 6 \times 2.7^2}{2 \times 40^2}\left[1+\left(\frac{0.6}{2.7}\right)^2\right]$$

= 0.141m = 14.1 cm

The operational joint displacement in the pipe segment = $\Delta_{oper} = 0.0054$ m

Hence the total joint displacement due to both seismic action and operational pressure and temperature is:

= 0.141 + 0.0054 = 0.1464 m

Considering the additional joint displacement of 0.6cm, the total joint displacement becomes (clause 3.10.2):

= 0.1464 + 0.006 = 0.1524 m = 15 cm

Hence, the displacement absorption capacity of the joint of the pipeline in transverse PGD zone should be more than 21 cm.

Case II: Fault crossing

The expected normal-slip fault displacement $= \delta_{fn} = 2.5$ m

Dip angle of the fault movement $= \psi = 35°$

The angle between pipeline and fault line $= \beta = 40°$

Component of fault displacement in the axial direction of the pipeline (clause 6.1.3):

$$\delta_{fax} = \delta_{fn} \cos\psi \sin\beta$$

= 2.5 × cos 35° × sin 40° = 1.316 m

Component of fault displacement in transverse direction of pipeline:

$$\delta_{ftr} = \delta_{fn} \cos\psi \cos\beta$$

= 2.5 × cos 35° × cos 40° = 1.569 m

Importance factor for class-II pipeline for fault displacement $= I_p = 1.5$ (Table 3.5.2)

Applying importance factor,

the design fault displacement in axial direction becomes (clause 6.1.6):

$\delta_{fax\text{-}design} = \delta_{fax} \times I_p = 1.316 \times 1.5 = 1.97$ m

and the design fault displacement in transverse direction becomes (clause 6.1.6):

$\delta_{ftr\text{-}design} = \delta_{ftr} \times I_p = 1.569 \times 1.5 = 2.35$ m

Joint displacement expected due to the fault movement can be taken as:

$\Delta_{seismic} = \delta_{fax-design} = 1.97$ m

The operational joint displacement in the pipe segment = $\Delta_{oper} = 0.0054$ m

Hence the total joint displacement is:

= 1.97 + 0.0054 = 1.98 m

Considering the allowance of 0.6 cm, the total displacement demand on the joint becomes (clause 3.10.2):

= 1.98 + 0.006 = 1.986 m = 198.6 cm

If the displacement absorption capacity of each joint is 40 cm, then a series of 5 joints should be constructed in the fault zone to absorb the design fault displacement. Also the trench profile should strictly be followed with suitable backfill material.

Case III: Seismic wave propagation

The expected peak ground acceleration of the site at base rock layer = PGA_r = 0.45g

For soil class E, peak ground acceleration (*PGA*) at ground = 0.45g × I_g

= 0.45g × 0.9 =0.405 g (Table 3.5.3 (b))

Converting PGA to PGV using the correlation given in table 3.5.5

$$\frac{PGV}{PGA} = 140$$

(Considering the soil as soft and the magnitude of design basis earthquake (M) is equals to 6.5, and distance of site from earthquake source is about 20km)

Hence, PGV = 0.405 × 140 =56.7 cm/s

Design $PGV = V_g = I_p \times PGV$

Where

I_p = Importance factor = 1.25 (Table 3.5.2)

Hence $V_g = 56.7 \times 1.25 = 70.87$ cm/sec

= 0.71 m/s

Maximum axial strain in the pipe due to wave velocity can be calculated as (clause 7.2.1):

$$\varepsilon_a = \frac{V_g}{\alpha_\varepsilon C} = \frac{0.71}{2 \times 2000} = 0.00018$$

Where

C = Velocity of Seismic wave propagation.

= 2 km/s (assuming shear wave velocity effect is dominating) (clause 7.1.2 and 7.2.1)

α_ε = Ground strain coefficient = 2.0 (clause 7.2.1)

Maximum axial strain that can be transmitted by soil friction can be calculated as (clause 7.2.2):

$$\varepsilon_a = \frac{t_u \lambda}{4AE} = \frac{68975 \times 1000}{4 \times 0.0119 \times 2 \times 10^{11}}$$

= 0.0072

Where

$$t_u = \pi D \alpha c + \pi D \bar{H} \gamma \left(\frac{1+K_0}{2} \right) \tan\delta$$

t_u = 68975 N/m (see Example-2)

Cross sectional area (A) of the pipeline is:

$$= A = \frac{\pi}{4}\left(0.6^2 - 0.5872^2\right) = 0.0119 \text{ m}^2$$

λ = Apparent wavelength of seismic waves at ground surface (clause 7.2.2) = 1000 m

The axial strain due to wave propagation (0.00018) need not be greater than the strain transmitted by soil friction i.e. 0.0072.

So the joint displacement due to wave propagation will be = $L_0 \times 0.00018$ m

= 0.001m

The operational joint displacement in the pipe segment = $\Delta_{oper} = 0.0054$ m

Hence the total joint displacement becomes:

= 0.001 + 0.0054 = 0.0064 m

Considering the allowance of 0.6 cm, the total displacement demand on the joint becomes (clause 3.10.2):

= 0.0064 + 0.006 = 0.0124 m = 1.3 cm

The axial extension of the joint due to wave propagation is very small. The joint of the pipeline should have displacement absorption capacity equals to or more than 1.3 cm.

NATIONAL INFORMATION CENTER OF EARTHQUAKE ENGINEERING

MISSION

The National Information Center of Earthquake Engineering (NICEE) at Indian Institute of Technology Kanpur maintains and disseminates information resources on Earthquake Engineering. It undertakes community outreach activities aimed at mitigation of earthquake disasters. NICEE's target audience includes professionals, academics and all others with an interest in and concern for seismic safety.

SPONSORS

NICEE receives no budget from any sources and operates entirely on the interest income of its endowment, sponsorships, publication sales, and the donations. One-time grants from the following organizations made it possible to launch the operations of NICEE:

Housing and Urban Development Corporation, New Delhi
Department of Telecom, New Delhi
Railway Board, New Delhi
Ministry of Agriculture, Government of India, New Delhi
Department of Atomic Energy, Mumbai

List of donors to NICEE may be viewed at *www.nicee.org/Giving.php*

All donations to NICEE from within India and USA are 100% tax deductible.

For more information, please contact:

Prof. Sudhir K. Jain
Coordinator, NICEE
Indian Institute of Technology Kanpur
Kanpur 208016 (INDIA)
Phone: *91-512-259 7866*; Fax: *91-512-259 7794*
Email: *nicee@iitk.ac.in*; Web: *www.nicee.org*

To receive free monthly electronic newsletter, please register at:
www.nicee.org

NICEE Publications and Products

Publications

Earthquake Engineering Practice (A Quarterly Periodical)

IITK-BMTPC Earthquake Tips by C. V. R. Murty

Design of Foundations in Seismic Areas: Principles and Applications by Subhamoy Bhattacharya (Editor)

Seismic Conceptual Design of Buildings - Basic principles for engineers, architects, building owners, and authorities by H. Bachmann

AT RISK: The Seismic Performance of Reinforced Concrete Frame Buildings with Masonry Infill Walls by C. V. R. Murty et al.

Earthquake Dynamics of Structures, A Primer by A. K. Chopra

Earthquake Design Criteria by G. W. Housner and P. C. Jennings

Guidelines for Earthquake Resistant Non-Engineered Construction (English) by IAEE

Guidelines for Earthquake Resistant Non-Engineered Construction (Hindi) by IAEE

Keeping Schools Safe in Earthquakes by OECD

Fundamentals of Seismic Protection for Bridges by M. Yashinsky and M. J. Karshenas

Seismic Hazard and Risk Analysis by R. K. McGuire

Reconnaissance Report of Sikkim Earthquake of 14 February 2006 by H. B. Kaushik et al.

Earthquake Rebuilding in Gujarat, India by C.V.R. Murty et al.

The Great Sumatra Earthquake and Andaman Ocean Tsunami of December 26 2004 by S. K. Jain et al.

Annotated Images from the Bhuj, India Earthquake of January 26, 2001 (CD) by EERI

Bhuj, India Republic Day January 26, 2001 Earthquake Reconnaissance Report (CD) by S. K. Jain et al. (editors)

Audio-Video Lectures on CD

Concept of Earthquake Resistant Design by S. K. Jain

Seismic Retrofit Techniques for Masonry Buildings - An Overview by S. N. Brzev

Buildings on Rollers - Use of Passive Control Devices for Seismic Protection of Structures by S. N. Brzev

Seismic Design & Retrofit of Nonstructural Building Components by S. N. Brzev

Building Performance in Boumerdes (Algeria) Earthquake of 21 May 2003 by S. N. Brzev

The History of Earthquake Engineering from an International Perspective by R. Reitherman

Structure & Architecture, Architecture & Earthquakes by A. W. Charleson

Seismic Hazard and Its Quantification by Late B. A. Bolt

Earthquake Resistant Design of Steel Buildings in the US by J. E. Rodgers

Powerpoint Slides on CD

E-course: Indian Seismic Code IS:1893-2002(part-I) by S. K. Jain

E-course: Seismic Design of Liquid Storage Tanks by S. K. Jain and O. R. Jaiswal

CD and Hard Copy

Architectural Teaching Resource Material on Earthquake Design Concepts for Teachers of Architecture Colleges by C. V. R. Murty and A. W. Charleson

Manual for Experimental Studies in Earthquake Engineering Education by C. S. Manohar and S. Venkatesha

IITK-GSDMA Guidelines

Guidelines for Seismic Design of Liquid Storage Tanks

Guidelines for Structural Use of Reinforced Masonry

Guidelines for Seismic Evaluation and Strengthening of Existing Buildings

Guidelines for Seismic Design of Dams and Embankments

Guidelines for Seismic Design of Buried Pipelines

Guidelines on Measures to Mitigate Effects of Terrorist Attacks on Buildings

ISBN 81-904190-7-2

A NICEE Publication

IITK-GSDMA GUIDELINES for

SEISMIC DESIGN of BURIED PIPELINES

Provisions with Commentary and Explanatory Examples

November 2007