# SEISMIC EVALUATION OF BURIED PIPELINE SYSTEMS


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# SEISMIC EVALUATION OF BURIED PIPELINE SYSTEMS

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## BONAFIDE CERTIFICATE

This is to certify that the work contained in this thesis titled "Seismic Evaluation of Buried Pipeline Systems" has been carried out by Jacob Zachariah under my/our supervision and has not been submitted elsewhere for a degree.

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# Abstract

Lifeline systems in 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 wellbeing 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

This project report 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, earthquake resistant design of buried pipelines is needed to ensure a uniform approach to earthquake resistant practices by all agencies in India. The provisions included here are based on many international and national codes, guidelines, and research documents.

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. The pipeline response and design criteria for general seismic hazards are specified in this guideline.

Seismic Analysis Report on Detailed Engineering Services for Bhagyam Field Development Project based on details obtained from seismic study conducted on area around Bhagyam field i.e. (well pads connecting The Mangla Processing Terminal – MPT) near Bharka-Barmer, Rajasthan. The primary objective of the seismic study is to ensure that in field pipeline will have an adequate level of safety during its lifetime against probable earthquake in its vicinity.

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# 1. Introduction

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, while water supply pipelines are constructed as segmented pipelines.

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

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.

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.

# 2. Literature Review

This document is based on a study done by Suresh R. Dash and Sudhir K. Jain in 2007 on Guideline for Seismic Design of buried pipelines reference as IITK-GSDMA (Gujarat State Disaster Management Authority) (2007) Guideline for Seismic Design of Buried Pipeline, Provisions with Commentary and Explanatory Examples. National Information Center for earthquake Engineering, Indian Institute of technology Kanpur, India.

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.

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 as per the updated allowable strain criteria. 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.

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.

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.

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

For the design criteria for permanent ground configuration, 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 is also assumed that the longitudinal ground movement is uniform throughout the Peak ground deformation (PGD) zone

The adherence of soil to pipe wall is neglected in finding the Buoyant force on pipeline while considering design criteria for Buoyancy due to Liquefaction.

This study is also based on the Seismic Studies of Area around Bhagyam Field (Well pads connecting the Mangla Processing Terminal –MPT) near Bharka – Barmer, Rajasthan done by Javed N. Malik and Durgesh Rai, Debasis Roy and Sudhir K. Jain.

These references being used in detailed Engineering Services for Bhagyam Field for Bhagyam Field Development Project for carrying out the Seismic Analysis Report

# 3. Acronyms

Acronyms that are used in this document are:

ALA American Lifelines Alliance

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

PGAr Peak Ground Acceleration at base rock layer

PGD Permanent Ground Deformation

PGV Peak Ground Velocity

PGVr Peak Ground Velocity at base rock layer

## 4. Required Information

### 4.1 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.

### 4.2 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;

### 4.3 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

## 5. Classification of Pipelines

The pipelines have been classified into four groups as per their functional requirement as follow:

**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 (High Pressure: $P \geq 10\ kgf/cm^2$) 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 (Medium Pressure: $3 < P < 10\ kgf/cm^2$) 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 (Low Pressure: $P \leq 3\ kgf/cm^2$).

**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

## 6. Classification of Soil

The soil at the site in the top 30m has been classified as given in Table 1 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 1 for classification of soil.

**Table 1:** Classification of soil at site.

| Soil Class | Soil Type | Velocity of Shear Wave ($V_s$) m/s | Undrained shear strength ($S_u$), $N/m^2$ | Uncorrected Standard penetration resistance (N) |
|---|---|---|---|---|
| A | Hard rock | $V_s > 1500$ | --- | -- |
| B | Rock | $760 < V_s \leq 1500$ | --- | -- |
| C | Very dense soil and soft rock | $360 < Vs \leq 760$ | $S_u \geq 98$ | $N > 50$ |
| D | Dense/ Stiff soil | $180 < Vs \leq 360$ | $49 \leq S_u \leq 98$ | $15 \leq N \leq 50$ |
| E | Loose/ Soft soil | $Vs < 180$ | $S_u < 49$ | $N < 15$ |
| | Soft soil with PI*(Plasticity Index of the soil) >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)<br><br>Peat or highly organic clays (H>3m, where H = thickness of soil)<br><br>Very high plasticity clays (H>7.5m with plasticity index > 75)<br><br>Very thick medium or soft stiff clays (H>35m) | -- | -- | -- |

NOTE: When sufficient detail of the soil is not available to define the site class, Soil 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.

When top 30 soil layer contains distinctly different soil layers, then the soil at the site can be classified as per the normalized values of $V_s$ or $S_u$ or N as defined as follows.

**6.1** 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

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

Vsi = Shear wave velocity in $i^{th}$ layer

**6.2** 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

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

Sui = Undrained shear strength in $i^{th}$ cohesive layer

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

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

Where

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

n = No of layers in top 30m soil

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

Ni = Standard Penetration Resistance in ith layer

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)

## 7. 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

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

## 8. Design Seismic Hazard

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 2 gives the design basis earthquake for different types of pipes (ALA 2005).

**Table 2:** Recommended design levels of seismic hazard.

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

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

The design seismic hazard for various classes of pipelines may be calculated by multiplying importance factor (*Ip*)

**Table 3:** Importance factor for different classes of pipeline (*I*p).

| Class of pipeline | Wave propagation | Faulting | Transverse and Longitudinal | 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 | * | * | * | * |

In this document, the second approach has been adopted.

## 8.1 Ground Amplification Factor

The ground amplification factors (Ig) is used for obtaining ground motion at the surface layer from that at the base rock level.

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 4 and 5; these values are adopted from FEMA:450 (2005).

**Table 4:** Ground amplification factor (*Ig*) 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,*) | | | | |
|---|---|---|---|---|---|
| | $PGVr \leq 0.1$m/s | $PGVr = 0.2$m/s | $PGVr = 0.3$m/s | $PGVr = 0.4$m/s | $PGVr \geq 0.5$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 | * | * | * | * | * |

**Table 5:** Ground amplification factor (*Ig*) for peak ground acceleration for various soil classes.

| Class of Soil | Peak Ground Acceleration Values at site (*PGA*) / Peak Ground Acceleration at | | | | |
|---|---|---|---|---|---|
| | $PGAr \leq 0.1$g | $PGAr = 0.2$g | $PGAr = 0.3$g | $PGAr = 0.4$g | $PGAr \geq 0.5$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.

## 8.2 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 6 for different seismic zones defined in IS 1893-2002 (part-1).

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 6. 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 6:** Peak Ground Acceleration as per Seismic Zones.

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

## 8.3 Determining PGV from PGA

When only the peak ground acceleration is available, Table 7 can be used to estimate peak ground velocity at that site. While using Table 7, user must specify the distance of site from earthquake source and the magnitude of the earthquake. Table 7 is same as that used in some other related literatures (e.g.: ALA 2001, ALA 2005, etc.).

**Table 7:** Relationship between peak ground velocity and peak ground acceleration.

| Moment | Ratio of Peak Ground Velocity (cm/s) to Peak Ground Acceleration (m/s$^2$) |
|---|---|
| | Source-to-Site Distance |

| Magnitude | | 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 | 6.5 | 94 | 102 | 109 |
| | 7.5 | 140 | 127 | 155 |
| | 8.5 | 180 | 188 | 193 |
| Soft | 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.

## 9. General Seismic Design Considerations

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

- In most cases, the seismic hazards ca not be quantified precisely. Hence, based on available data and experience, reasonable assumptions should be made to define proper model for the seismic hazard.
- In the design of pipeline systems, permanent ground deformation is a much more serious concern than seismic shaking.
  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.
- 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).
- In all areas of expected ground rupture, pipelines should be provided with automatic shutdown valves.

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.

- The fittings of the segmented steel pipelines (e.g., water pipelines) should be ductile.
  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.
- 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.
- A ductile coating (e.g., asphalt coating, polyethylene sheet, etc.) is recommended at the joints of the segmented pipeline
  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.
- For segmented pipeline, the displacement absorption capacity of the joint should be more than the expected joint movement due to design seismic action

## 10. Analysis Procedure

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

### 10.1 Ramberg-Osgood's relationship

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

$\varepsilon$ = Engineering strain

$\sigma$ = Stress in the pipe

$E$ = Initial Young's modulus

$\sigma_y$ = Yield strain of the pipe material

$n$, $r$ = Ramberg - Osgood parameters (Table 8)

**Table 8:** 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 |

# 11. Initial Stresses in the Pipeline

## 11.1 Internal Pressure

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

$$S_P = \frac{PDu}{2t}$$

$P$ = Maximum internal operating pressure of the pipe

$D$ = Outside diameter of the pipe

$\mu$ = Poisson's ratio (generally taken as 0.3 for steel)

$t$ = Nominal wall thickness of the pipe

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).

## 11.2 Temperature Change

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

$$S_r = E\alpha\,(T_2 - T_1)$$

Where

$E$ = Modulus of elasticity

$\alpha$ = 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

The same equation has also been used in the API guidelines API Guideline (API-1117, 1996).

## 12. Allowable Strain for Continuous Pipeline

The maximum allowable strains for buried continuous pipelines are specified in Table 9. This table helps 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.

**Table 9:** Allowable strain criteria for buried continuous pipelines

<table>
<tr><th rowspan="2">Strain Component</th><th rowspan="2">Pipe Category</th><th colspan="2">Allowable Strain</th></tr>
<tr><th>Tension</th><th>Compression</th></tr>
<tr><td rowspan="4">Continuous Oil and Gas Pipeline</td><td>Ductile Cast Iron Pipe</td><td>2%</td><td rowspan="4">For PGD: Onset of Wrinkling ($\varepsilon_{cr-c}$)<br><br>For wave propagation: 50% to 100% of the Onset of Wrinkling (0.5 to 1 $\varepsilon_{cr-c}$)</td></tr>
<tr><td>Steel Pipe</td><td>3%</td></tr>
<tr><td>Polyethylene Pipe</td><td>20%</td></tr>
<tr><td>Bends and Tees of pipe</td><td>1%</td></tr>
<tr><td rowspan="2">Continuous Water Pipeline</td><td rowspan="2">Steel and iron pipe</td><td rowspan="2">0.25 $\varepsilon u$ or 5%</td><td>$\varepsilon_{c\text{-}pgd}$</td></tr>
<tr><td>$\varepsilon_{c\text{-}wave}$</td></tr>
</table>

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

Where

$\varepsilon_u$ = Failure strain of the pipe in tension

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

$$\varepsilon_{c-wave} = 0.75\ \left[0.5\frac{t}{D} - 0.0025 + 3000\left(\frac{PD}{2Et}\right)^2\right]$$

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

$D_{min}$ = Minimum inside diameter of pipe = outside diameter of pipe excluding out of roundness thickness (Figure 1)

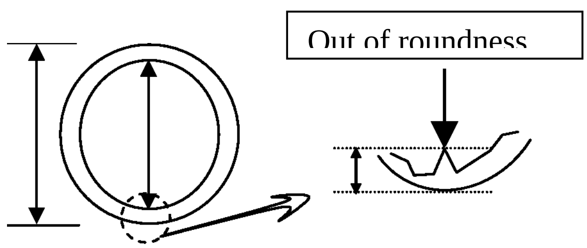


**Figure 1:** Schematic diagram showing minimum inside diameter of pipe.

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 for oil and gas pipelines will become very conservative for water pipelines. Hence, for water pipeline more relaxed strain limits $\varepsilon_{c\text{-}pgd}$- $\varepsilon_{c\text{-}wave}$.

**12.1** The theoretical value of local wrinkling of a pipe begins at a compressive strain of $\varepsilon_c$ 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 1/3rd to 1/4th of the theoretical wrinkling strain. In this document the allowable wrinkling strain is considered as the mean of the experimental wrinkling strains.

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

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

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

Where

$\varepsilon_{allowable}$ = Allowable strain in pipe (as per Table 9)

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

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

$\varepsilon_p$ = Strain in the pipe due to internal pressure

$\varepsilon_t$ = Strain in the pipe due to temperature change

$\varepsilon_{D+L}$ = Strain in the pipe due to service loads

# 13 Allowable Joint Displacement for Segmented Pipeline

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

$\Delta oper$ = Operational joint displacement which is equal to: $\Delta p + \Delta t + \Delta D{+}L$

$\Delta p$ = Joint displacement due to internal pressure

$\Delta t$ = Joint displacement due to temperature change

$\Delta D+L$ = 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.

**13.1** 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).

## 14. 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.

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, whereas the ground deformation associated with soil failure (Figure 2) is gradual. This document addresses the permanent ground deformation associated with the soil failure only.

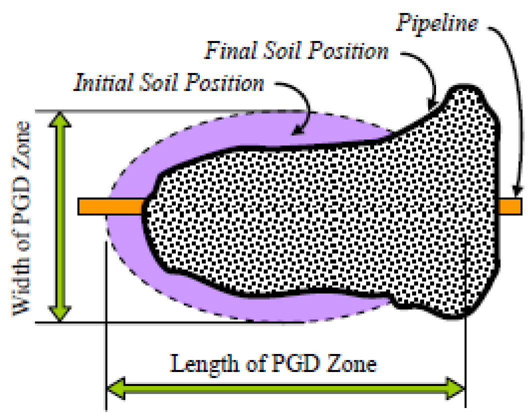

**Figure 2:** Schematic diagram showing permanent ground deformation due to soil failure.

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. However designing the pipeline for critical response due to permanent ground deformation (PGD), two conditions such as parallel crossing (Figure 3) and perpendicular crossing (Figure 4). This document considers these two situations as the pipeline is subjected to longitudinal PGD and transverse PGD respectively.

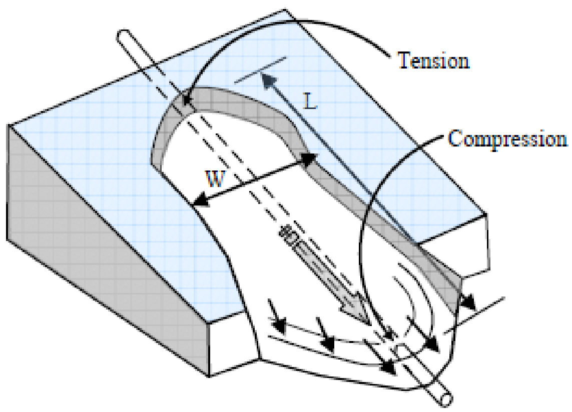


**Figure 3:** Longitudinal permanent ground deformation; pipeline crossing permanent ground deformation zone in the direction of ground movement.

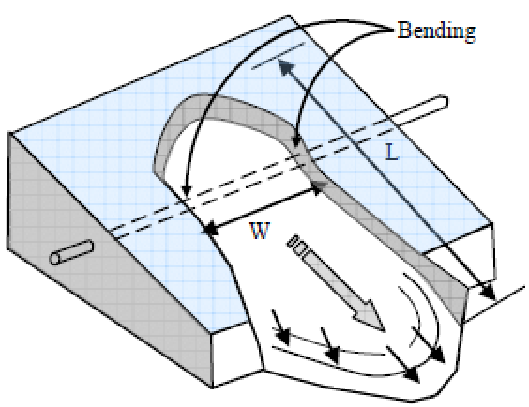


**Figure 4:** Transverse permanent ground deformation; pipeline crossing permanent ground deformation zone transverse to the ground movement.

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.

- If the expected ground displacement exceeds the displacement absorption capacity of the pipe, other alternatives such as soil improvement, etc. shall be employed.
- Pipeline response can be minimized either by minimizing the ground displacement, and/or increasing the load carrying 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.
- 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.
- 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.
- As far as possible, pipeline should be placed below the lowest depth of liquefiable soil.
- 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.
- Trenches, deformable walls or other similar means can be constructed to absorb ground deformations at an upslope location.
- 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.

- 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.
- Where extreme deformation is expected, special pipe joints or fittings are required to be used to allow greater joint deflection, extension or compression.
- The pipelines require moderate to high ductility in areas of permanent ground deformation. Welded polyethylene pipes may be a better option in such areas.
- In segmented pipelines special connections are required to accommodate large ground movement in the areas of permanent ground deformation.

## 14.1 Longitudinal Permanent Ground Deformation

This 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. . For critical response, the block pattern (i.e., the longitudinal ground movement is uniform throughout the PGD zone (Figure 5) of ground deformation is used in this document.

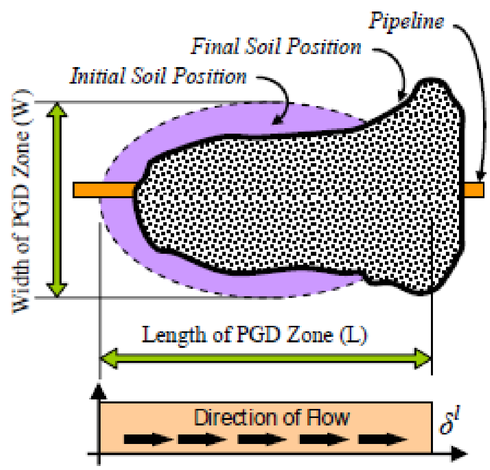


**Figure 5:** Block pattern of longitudinal permanent ground deformation.

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

It is generally difficult to come out with a single number for the amount (S) 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:

$$S_{design} = SI_p$$

Where:
S = Maximum longitudinal ground displacement
*Ip* = Importance factor (Table 3)

### 14.1.1 Continuous Pipeline

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

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

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

**14.1.1.1 Case-1:** Figure 6 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 (S). 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 (section 10.1) may be used to find the maximum strain in pipe from the maximum stress value.

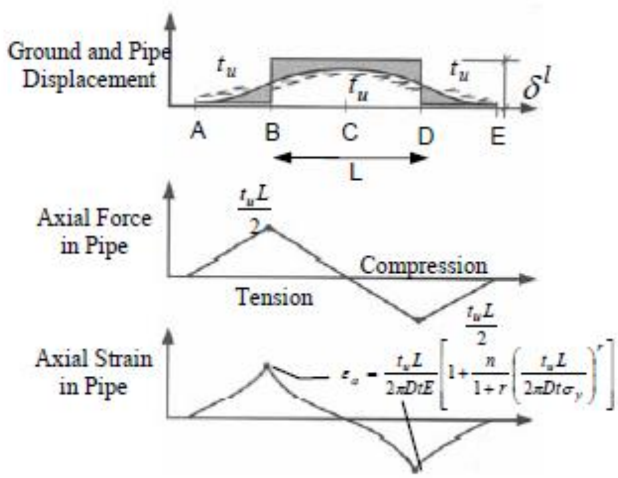


**Figure 6:** Case-1: Inelastic model for longitudinal PGD

When S is large (i.e., **Case-1**), the maximum axial strain in pipe for both tension and compression can be calculated as:

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

Where

*L* = Length of permanent ground deformation zone

$\sigma_y$ = Yield stress of pipe material

*n* , *r* = Ramberg-Osgood parameter (Table 8)

*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

**14.1.1.2- Case-2:** Figure 7 shows the situation for Case-2. Here, the friction force is acting over an as yet unknown length of *Le* 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 (S) over a region of length $L$ - 2$Le$ at the center of the PGD zone.

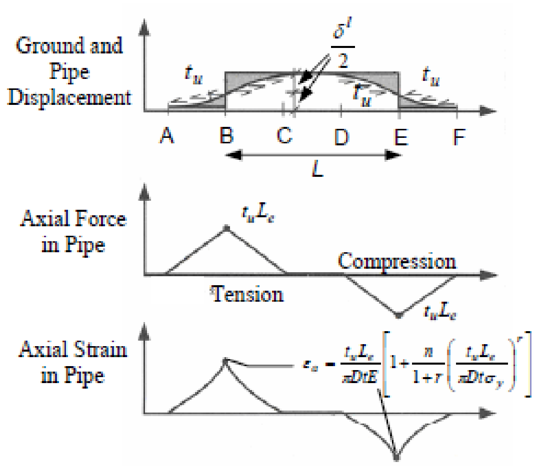


**Figure 7:** Case-2: Inelastic model for longitudinal PGD

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 *Le*, over which the friction force acts is considered. From Figure 7, by symmetry, the pipe displacement at point B is S/2, where

$$\frac{S}{2} = \int_{0}^{L_e} \varepsilon_a(x)dx$$

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

When $L$ is very large (i.e., **Case-2**), $S$ governs the amount of strain in pipe, and the peak pipe strain for both tension and compression can be calculated as:

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

Where

*Le* = Effective length of pipeline over which friction force *tu* acts, and can be evaluated from the following equation

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

**14.1.1.3:** The design pipe strain ($\varepsilon_{seismic}$) for longitudinal permanent ground deformation should be taken as the lower of the strains obtained from above 2 cases (section 14.1.1.2 and 14.1.1.3). The design pipe strain should confirm to the allowable strain criteria as given in section 12.

### 14.1.1.5: Influence of Expansion Joint

The expansion joints are flexible joints that are provided in the continuous pipelines to absorb the ground movement. Depending on the position of expansion joints, they may have no effect or a 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.

### 14.1.1.6: Influence of Field Bend

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

## 14.1.2 Segmented Pipeline

**14.1.2.1:** 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.

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,

$$\Delta_{seismic} = S_{design}$$

Where,

S = Design ground displacement in longitudinal direction (section 14.1).

The design joint displacement should confirm to the allowable joint displacement as specified in section 13.

**14.1.2.2:** 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 above.

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.

**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 cannot 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.

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 section 14.1.2.2, 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

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

$$\Delta_{seismic} = \left( \frac{S_{design}}{\frac{L}{2}} \right) L_a$$

Where

*La* = Length of pipe segment

*L* = Length of permanent ground deformation zone.

**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:

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

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, *Fstop* need not be higher than the yield strength of pipe.

## 14.2 Transverse Permanent Ground Deformation

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

Alike longitudinal ground displacement, the 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 8.

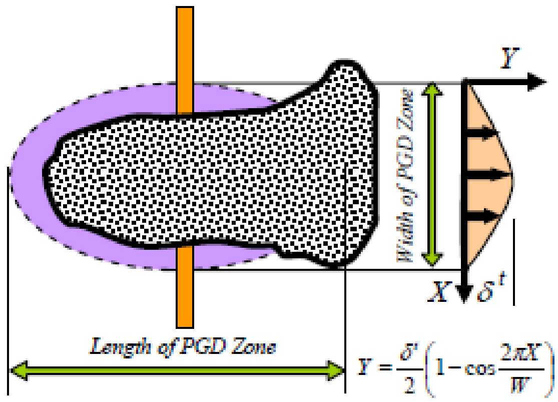


**Figure 8:** Pattern of transverse permanent ground deformation.

Like longitudinal PGD, a range of the amount (S) and spatial extent (L and W) of transverse PGD are quantified and the seismic check is carried out.

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

$$S_{design} = S \times I_p$$

Where

$S$ = Maximum transverse ground displacement

$I_p$ = Importance factor (Table 3)

### 14.2.1 Continuous Pipeline

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.

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

a) $\varepsilon_b = \pm\left(\frac{\pi D S_{design}}{w^2}\right)$

Where

*D* = Outside diameter of pipe

$S_{design}$ = Design transverse ground displacement

*W* = Width of permanent ground deformation zone

*t* = Thickness of pipe

b) $\varepsilon_b = \pm\left(\frac{P_u w^2}{3\pi E t D^2}\right)$

Where

*Pu* = Maximum lateral resistance of soil per unit length of pipe (Section 18)

*E* = Modulus of elasticity of pipe material

**14.2.1.2:** The maximum strain obtained in section 14.2.1.1 should be considered as the design pipe strain ($\varepsilon_{seismic}$) and should confirm to the allowable pipe strain as given in section 13.

**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.

### 14.2.2 Segmented Pipeline

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

**14.2.2.1:** 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:

$$\Delta_{seismic} = \frac{\pi^2 L_0 (S_{design})^2}{w^2} \left(\frac{2D}{S_{design}}\right), \qquad \text{for } 0.268 \leq \frac{D}{S_{design}} \leq 3.73$$

$$\Delta_{seismic} = \frac{\pi^2 L_0 (S_{design})^2}{2w^2} \left(1 + \left(\frac{D}{S_{design}}\right)^2\right), \qquad \text{for other values of } D/S_{design}$$

Where

*L0* = Length of pipe segment

$S_{design}$ = Design transverse ground displacement

**14.2.2.2:** The design joint displacement ($\Delta_{seismic}$) calculated above should confirm to the allowable joint displacement criteria as given in section 13.

## 14.3 Landslide

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

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.

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

If the behaviour of landslide and its displacement pattern is defined, then this can be modeled as permanent ground displacement acting on pipelines.

# 15. 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.

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.

- 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.
- Concrete weights or gravel filled blankets can be utilized to provide additional resistance to buoyancy.
- Buoyancy effect can also be minimized by shallow burial of pipeline above the ground water table.
- Where uplift is the main concern, anchors may be provided with a close spacing (~150 m) to prevent uplift.

## 15.1 Buoyant Force on Pipeline

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 9)

$\gamma_w$ = Dry unit weight of backfill

$h_w$ = Height of water over pipeline (Figure 10)

When the pipeline is jacked into undisturbed and unsaturated soil instead of being placed in the trench and covered with backfill, then the earth 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/cm2 for loose dry sand

= 0.7 kg/cm2 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 9 and 10 for reference.

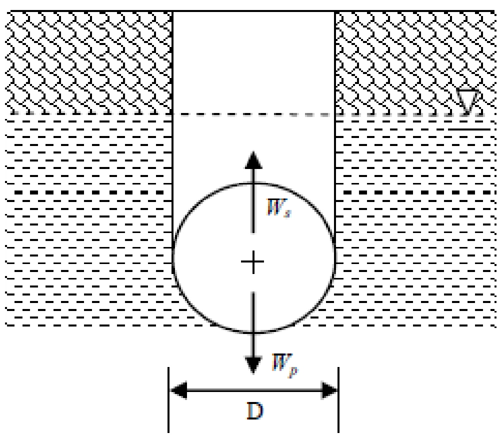


**Figure 9:** Cross section of the pipeline showing the forces acting on it due to buoyancy.

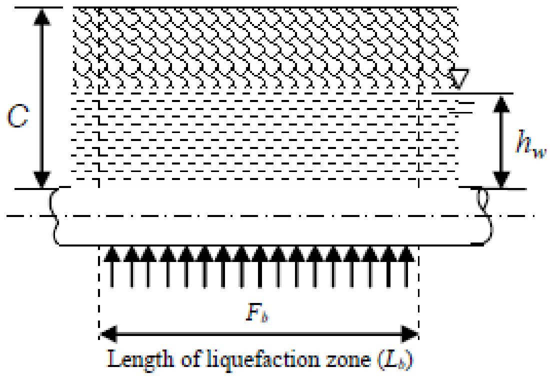


**Figure 10:** Longitudinal section of the pipeline showing the forces acting on it due to buoyancy.

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

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

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

$\gamma_w$ = Unit weight of water

$h_w$ = Height of water above pipeline

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

## 15.2 Continuous Pipeline

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

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

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.

**15.2.2:** The maximum strain corresponding to above bending stress (section 15.2.1) can be obtained by using Ramberg-Osgood's stress-strain relationship (section 10.1).

**5.2.3 –** The maximum strain obtained in section 15.2.2 can be considered as the design strain in pipe ($\varepsilon_{seismic}$) and should confirm to the allowable strain as specified in section 12.

### 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 9 and 10.

**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 section 13.

## 16. 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.

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.

- 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.
- The ductility of pipeline should be increased in the zone of fault-crossing to accommodate the fault movement without rupture.
- Abrupt changes in wall thickness or other stress concentrators should be avoided within the fault zone.
- In all areas of potential ground rupture, pipelines should be laid in relatively straight section avoiding sharp changes in direction and elevation.
- To the extent possible, pipelines should be constructed without field bends, elbows, and flanges that tend to anchor the pipeline to the ground.
- If longer length of pipeline is available to conform to fault movement, level of strain 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.
- 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.
- The burial depth of pipeline may be reduced within fault zones in order to minimize the soil restraint on the pipeline during fault movement.

- 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.

## 16.1 – Quantification of Fault Displacement

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

Where

$\delta_{fs}$= Strike slip fault displacement in meters

$\delta_{fn}$ = Normal slip fault displacement in meters

$\delta_{fr}$= Reverse slip fault displacement in meters

$\delta_{fb}$ = Displacement of a blind fault in meters

$M$ = Moment magnitude of earthquake

**16.1.2:** For a strike slip fault (Figure 11), 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 11)

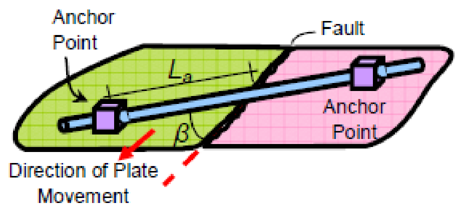


**Figure 11:** Pipeline crossing strike slip fault.

**16.1.3:** For a normal slip fault (Figure 12), 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: $\delta_{fvt} = \delta_{fn} \sin\psi$

Where

β = angle of pipeline crossing a fault line (Figure 12)

β = Dip angle of the fault (Figure 12)

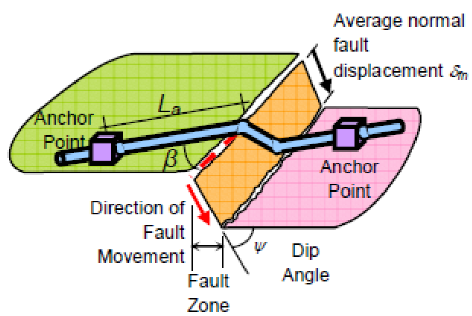


**Figure 12:** Pipeline crossing normal slip fault.

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

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

**16.1.6:** Design fault displacement can be evaluated by multiplying the importance factor ($Ip$) (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: $\delta_{ftr-design} = \delta_{ftr} \times I_p$

Design fault displacement in vertical direction of pipeline: $\delta_{fvt-design} = \delta_{fvt} \times I_p$

## 16.2 Continuous Pipeline

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

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

Where

$L_a$= Unanchored pipe length (refer: section 16.2.2)

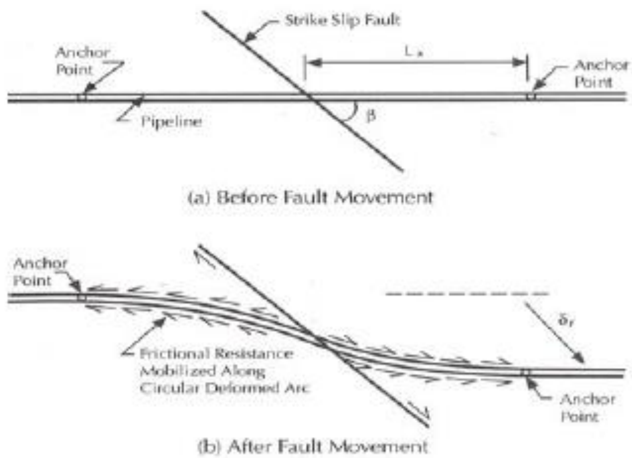


**Figure 13:** Newmark-Hall model for fault crossing

The expression for average strain in pipeline is based on Newmark-Hall's model (Figure 13). A factor of 2 in the formula in clause 6.2.1 is used to counterbalance the unconservatism involved in this model.

**16.2.2:** 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

$\varepsilon_y$ = The yield strain of the material

$E_i$ = Modulus of pipe material before 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 13).

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:

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

Where

$\varepsilon$ = Plastic strain in pipe

$E_p$ = Modulus of pipe material after yielding

The second part of the above equation represents 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.

**16.2.3:** 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 section 12.

## 16.3 Segmented Pipeline

For a segmented pipeline (Figure 14) crossing fault line, it is generally assumed that a) the pipe segments are rigid and b) only the pipe joints accommodate the ground deformation.

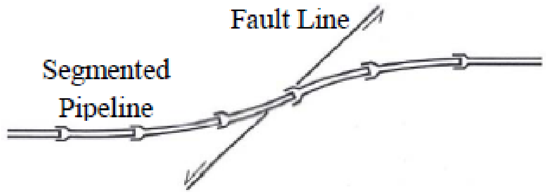


**Figure 14**: Segmented pipeline crossing a strike slip fault.

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)

**16.3.2:** 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 section 13.

## 16.4 Trench Profile for Pipelines in Fault Zone

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.

To achieve minimum soil resistance to reduce the strain in pipe, the pipeline can be buried in a shallow trench as shown in Figure 15 and 16 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 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.

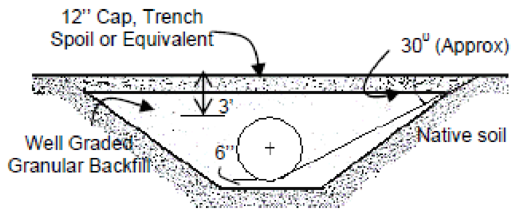


**Figure 15:** Pipeline trench for strike slip fault crossing.

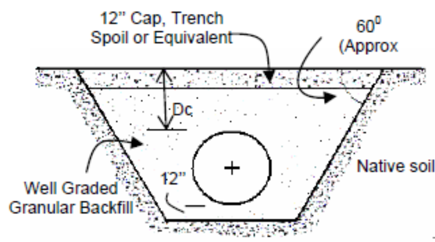


**Figure 16:** Pipeline trench for reverse slip fault crossing

## 17. 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.

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.

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

## 17.1 Design Ground Motion

**17.1.1:** 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 (section 8)

$I_p$ = Importance factor as specified in table 3.

### 17.1.2 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 17).

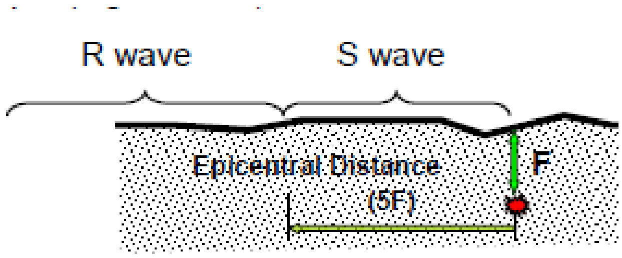


**Figure 17**: Considerations for S-wave and R-wave in pipeline design.

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

### 17.1.2.1 For S-wave

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 angle with respect to vertical. If the angle of incidence is θ and the shear wave velocity of the top layer is *Cs*, 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.

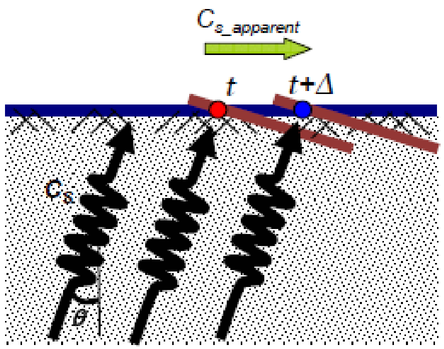


**Figure 18:** Figure illustrating the apparent seismic wave propagation of S-wave

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

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

Where

$C_s$ = Wave propagation velocity of S-wave

θ = Angle of incidence of S-wave

### 17.1.2.2 For R-wave

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

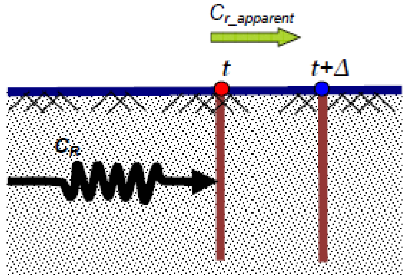


**Figure 19:** Figure illustrating apparent seismic wave propagation of R wave.

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

## 17.2 Continuous Pipeline

**17.2.1:** 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 (section 17.1.1)

$\alpha_{\varepsilon\varepsilon}$ = 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)

The ground strain coefficient $\alpha_\varepsilon$ 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 20). Hence, the apparent propagation velocity in the direction of pipe axis is:

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

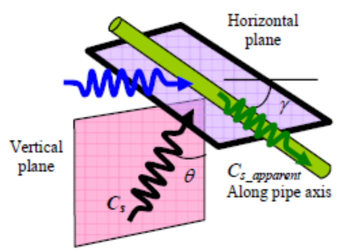


**Figure 20:** Schematic for apparent wave propagation velocity of S-wave along the pipe axis.

The ground strain is maximum for θ and $\gamma$= 45°

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

So $\alpha_\varepsilon$ 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.

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

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

Where,

$t_u$ = 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

The maximum frictional resistance (*tu*) of the soil to pipe movement (Figure 21) 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

The apparent wave length of seismic wave ($\lambda$) is defined as the product of apparent wave propagation velocity (*Vs*) and natural fundamental period of ground surface.

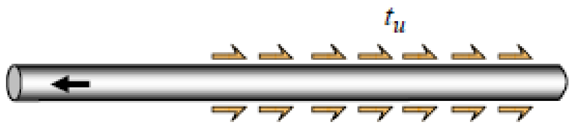


**Figure 21:** Figure illustrating frictional resistance over the pipe surface due to axial ground strain.

**17.2.3:** The axial strain calculated in section 17.2.1 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 (section 17.2.2).

**17.2.5:** The design pipe strain as calculated above (section 17.2.3) should confirm to the allowable strain limit as specified in section 12.

## 17.3 Segmented Pipeline

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. 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.

**17.3.1:** The design joint displacement in pipeline can be calculated as:

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

Where

$\varepsilon_{seismic}$ = Axial strain as calculated in section 17.2.4 for the continuous pipeline.

$L_0$ = Length of pipe segment

**17.3.2:** The design joint displacement ($\Delta_{seismic}$) calculated above should satisfy the allowable joint displacement criteria as given in Section 13.

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

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

Where

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

*C* = Velocity of seismic wave propagation

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

= *Cr_ph*, for R-waves (0.5 km/s may be considered conservatively)

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

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

# 18. Soil Spring Properties to Model Soil-Pipe Interaction

## 18.1 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 22 shows the idealized representation of the axial soil spring.

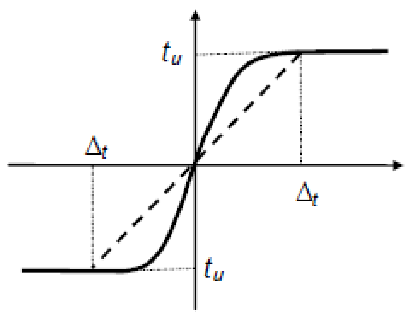


**Figure 22:** Idealized representation of axial soil spring

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

$$t_u = \pi D c \alpha + \pi D H \breve{y} \left(\frac{1 + K_0}{2}\right) tan S$$

where

$D$ = Outside diameter of pipe

$c$ = Coefficient of cohesion of backfill soil

$H$ = Depth of soil above the center of the pipeline

Ў = Effective unit weight of soil

α = Adhesion factor, $\alpha = 0.608 - 0.123c - \frac{0.274}{c^2+1} + \frac{0.695}{c^3+1}$

S = Interface angle of friction between pipe and soil, $S = f\phi$

$\Phi$ = Internal friction angle of the soil

$f$ = Friction factor for various types of pipes (Table 10).

$K_0$ = Coefficient of soil pressure at rest. This may be taken from Table B 1b or may be determined by Jaky's formula as: $K_0 = 1 - sin\phi$

The maximum mobilizing displacement of soil ($\Delta t$) in axial direction of pipe can be taken as:

$\Delta_t$ = 3mm for dense sand

= 5mm for loose sand

= 8mm for stiff clay

= 10mm for soft clay

**Table 10:** Friction factor 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 11:** Values of lateral pressure coefficient at rest ($K_0$) for different soil conditions.

| **Type of soil** | ***K*** |
|---|---|
| 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.

## 18.2 Lateral Soil Spring

The properties of lateral soil spring are estimated considering the native soil at the site. Figure 23 shows the idealized representation of the lateral soil spring.

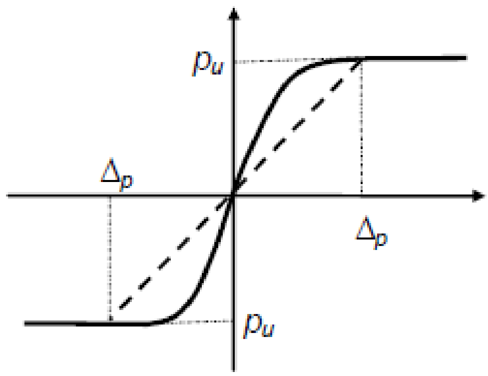


**Figure 23:** Idealized representation of lateral soil spring

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

$$P_u = N_{ch}cD + N_{qh}\breve{y}HD$$

where

$N_{ch}$ = Horizontal bearing capacity factor for clay, 0 for *c* = 0 (Table 12),

$N_{qh}$ = Horizontal bearing capacity factor for sandy soil, 0 for = 0 (Table 12),

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

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

Where, $x = \frac{H}{D}$

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

**Table 12:** 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 | $1.059\times10^{-3}$ | $-1.754\times10^{-5}$ |
| $N_{qh}$ | 25 | 3.332 | 0.839 | -0.09 | $5.606\times10^{-3}$ | $-1.319\times10^{-4}$ |
| $N_{qh}$ | 30 | 4.565 | 1.234 | -0.089 | $4.275\times10^{-3}$ | $-9.159\times10^{-5}$ |
| $N_{qh}$ | 35 | 6.816 | 2.019 | -0.146 | $7.651\times10^{-3}$ | $-1.683\times10^{-4}$ |
| $N_{qh}$ | 40 | 10.959 | 1.783 | 0.045 | $-5.425\times10^{-3}$ | $-1.153\times10^{-4}$ |
| $N_{qh}$ | 45 | 17.658 | 3.309 | 0.048 | $-6.443\times10^{-3}$ | $-1.299\times10^{-4}$ |

## 18.3 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 24 shows the idealized representation of the vertical soil spring.

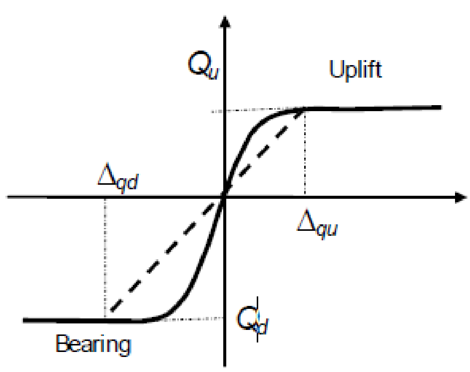


**Figure 24:** Idealized representation of soil springs in vertical direction.

### 18.3.1 Vertical Uplift

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

$$Q_u = N_{cv} cD + N_{qv} \breve{y} HD$$

Where

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

$N_{qv}$ = Vertical uplift factor for sand (0 for Φ = 0°),

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

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

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

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

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

### 18.3.2 Vertical Bearing

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

$$Q_d = N_c cD + N_q \breve{y} HD + N_r y \frac{D^2}{2}$$

Where

*Nc*, *Nq*, $N\gamma$ and are bearing capacity factors from Figure 25 or as

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

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

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

$\gamma$ = Total unit weight of soil

The mobilizing soil displacement, $\Delta_{qd}$, at $Q_d$ can be taken as:
0.1$D$ for granular soils, and
0.2$D$ for cohesive soils.

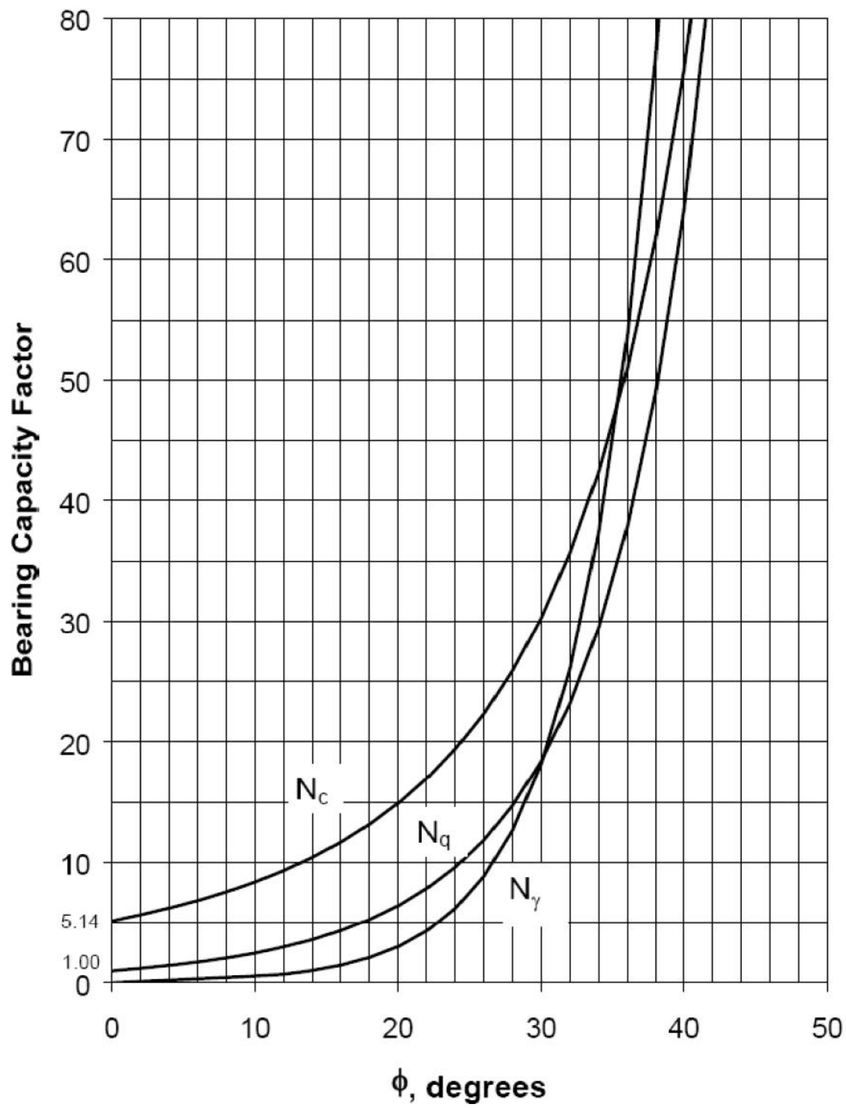


**Figure 25:** Bearing capacity factors of soils of different soil friction values (ALA 2001).



PL Engineering

# 19. Attenuation Relationships

## 19.1 Attenuation Relationships Derived By Boore, Joyner And Fumal (1997)

The following equation was derived by Boore et al for spectral ordinates:

$$Ln(y) = b_1 + b_2(M-6) + b_3(M-6)^2 + b_5 Ln(r) + b_V Ln\left(\frac{V_S}{V_A}\right)$$

$$r = \sqrt{r_{jb}^2 + h^2}$$

$$b_1 = \begin{Bmatrix} b_{1SS} & \text{for strike-slip earthquakes} \\ b_{1RV} & \text{for reverse-slip earthquakes} \\ b_{1ALL} & \text{if mechanism is not specified} \end{Bmatrix}$$

The variable y is spectral acceleration in g; $M$ is moment magnitude, $V_s$ is the average shear wave velocity (in m/sec), $V_A$ is reference shear wave velocity (in m/sec), and $r_{jb}$ is the closest horizontal distance (in km) from the site to the surface projection of the source. Values of the coefficients $b_{1SS}$, $b_{1RV}$, $b_{1ALL}$, $b_2$, $b_3$, $b_5$, $b_V$, $V_A$, and $h$ (in km) are listed in **Table 13**. Also listed in **Table 13** are the values of the standard error terms.

**Table 13:** Coefficients Derived by Boore, Fumal and Joyner (1997)

| Period | $b_{1ss}$ | $b_{1RV}$ | $b_{1ALL}$ | $b_2$ | $b_3$ | $b_5$ | $b_V$ | $V_A$ | $h$ | $SE$ |
|---|---|---|---|---|---|---|---|---|---|---|
| 0 | -0.313 | -0.117 | -0.242 | 0.527 | 0.000 | -0.778 | -0.371 | 1396 | 5.57 | 0.520 |
| 0.10 | 1.006 | 1.087 | 1.059 | 0.753 | -0.226 | -0.934 | -0.212 | 1112 | 6.27 | 0.479 |
| 0.11 | 1.072 | 1.164 | 1.130 | 0.732 | -0.230 | -0.937 | -0.211 | 1291 | 6.65 | 0.481 |
| 0.12 | 1.109 | 1.215 | 1.174 | 0.721 | -0.233 | -0.939 | -0.215 | 1452 | 6.91 | 0.485 |
| 0.13 | 1.128 | 1.246 | 1.200 | 0.711 | -0.233 | -0.939 | -0.221 | 1596 | 7.08 | 0.486 |
| 0.14 | 1.135 | 1.261 | 1.208 | 0.707 | -0.230 | -0.938 | -0.228 | 1718 | 7.18 | 0.489 |
| 0.15 | 1.128 | 1.264 | 1.204 | 0.702 | -0.228 | -0.937 | -0.238 | 1820 | 7.23 | 0.492 |
| 0.16 | 1.112 | 1.257 | 1.192 | 0.702 | -0.226 | -0.935 | -0.248 | 1910 | 7.24 | 0.495 |
| 0.17 | 1.090 | 1.242 | 1.173 | 0.702 | -0.221 | -0.933 | -0.258 | 1977 | 7.21 | 0.497 |

| | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| 0.18 | 1.063 | 1.222 | 1.151 | 0.705 | -0.216 | -0.930 | -0.270 | 2037 | 7.16 | 0.499 |
| 0.19 | 1.032 | 1.198 | 1.122 | 0.709 | -0.212 | -0.927 | -0.281 | 2080 | 7.10 | 0.501 |
| 0.20 | 0.999 | 1.170 | 1.089 | 0.711 | -0.207 | -0.924 | -0.292 | 2118 | 7.02 | 0.502 |
| 0.22 | 0.925 | 1.104 | 1.019 | 0.721 | -0.198 | -0.918 | -0.315 | 2158 | 6.83 | 0.508 |
| 0.24 | 0.847 | 1.033 | 0.941 | 0.732 | -0.189 | -0.912 | -0.338 | 2178 | 6.62 | 0.511 |
| 0.26 | 0.764 | 0.958 | 0.861 | 0.744 | -0.180 | -0.906 | -0.360 | 2173 | 6.39 | 0.514 |
| 0.28 | 0.681 | 0.881 | 0.780 | 0.758 | -0.168 | -0.899 | -0.381 | 2158 | 6.17 | 0.518 |
| 0.30 | 0.598 | 0.803 | 0.700 | 0.769 | -0.161 | -0.893 | -0.401 | 2133 | 5.94 | 0.522 |
| 0.32 | 0.518 | 0.725 | 0.619 | 0.783 | -0.152 | -0.888 | -0.420 | 2104 | 5.72 | 0.525 |
| 0.34 | 0.439 | 0.648 | 0.540 | 0.794 | -0.143 | -0.882 | -0.438 | 2070 | 5.50 | 0.530 |
| 0.36 | 0.361 | 0.570 | 0.462 | 0.806 | -0.136 | -0.877 | -0.456 | 2032 | 5.30 | 0.532 |
| 0.38 | 0.286 | 0.495 | 0.385 | 0.820 | -0.127 | -0.872 | -0.472 | 1995 | 5.10 | 0.536 |
| 0.40 | 0.212 | 0.423 | 0.311 | 0.831 | -0.120 | -0.867 | -0.487 | 1954 | 4.91 | 0.538 |
| 0.42 | 0.140 | 0.352 | 0.239 | 0.840 | -0.113 | -0.862 | -0.502 | 1919 | 4.74 | 0.542 |
| 0.44 | 0.073 | 0.282 | 0.169 | 0.852 | -0.108 | -0.858 | -0.516 | 1884 | 4.57 | 0.545 |
| 0.46 | 0.005 | 0.217 | 0.102 | 0.863 | -0.101 | -0.854 | -0.529 | 1849 | 4.41 | 0.549 |
| 0.48 | -0.058 | 0.151 | 0.036 | 0.873 | -0.097 | -0.850 | -0.541 | 1816 | 4.26 | 0.551 |
| 0.50 | -0.122 | 0.087 | -0.025 | 0.884 | -0.090 | -0.846 | -0.553 | 1782 | 4.13 | 0.556 |
| 0.55 | -0.268 | -0.063 | -0.176 | 0.907 | -0.078 | -0.837 | -0.579 | 1710 | 3.82 | 0.562 |
| 0.60 | -0.401 | -0.203 | -0.314 | 0.928 | -0.069 | -0.830 | -0.602 | 1644 | 3.57 | 0.569 |
| 0.65 | -0.523 | -0.331 | -0.440 | 0.946 | -0.060 | -0.823 | -0.622 | 1592 | 3.36 | 0.575 |
| 0.70 | -0.634 | -0.452 | -0.555 | 0.962 | -0.053 | -0.818 | -0.639 | 1545 | 3.20 | 0.582 |
| 0.75 | -0.737 | -0.562 | -0.661 | 0.979 | -0.046 | -0,813 | -0.653 | 1507 | 3.07 | 0.587 |
| 0.80 | -0.829 | -0.666 | -0.760 | 0.992 | -0.041 | -0.809 | -0.666 | 1476 | 2.98 | 0.593 |
| 0.85 | -0.915 | -0.761 | -0.851 | 1.006 | -0.037 | -0.805 | -0.676 | 1452 | 2.92 | 0.598 |
| 0.90 | -0.993 | -0.848 | -0.933 | 1.018 | -0.035 | -0.802 | -0.685 | 1432 | 2.89 | 0.604 |
| 0.95 | -1.066 | -0.932 | -1.010 | 1.027 | -0.032 | -0.800 | -0.692 | 1416 | 2.88 | 0.609 |
| 1.00 | -1.133 | -1.009 | -1.080 | 1.036 | -0.032 | -0.798 | -0.698 | 1406 | 2.90 | 0.613 |
| 1.10 | -1.249 | -1.145 | -1.208 | 1.052 | -0.030 | -0.795 | -0.706 | 1396 | 2.99 | 0.622 |

| | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| 1.20 | -1.345 | -1.265 | -1.315 | 1.064 | -0.032 | -0.794 | -0.710 | 1400 | 3.14 | 0.629 |
| 1.30 | -1.428 | -1.370 | -1.407 | 1.073 | -0.035 | -0.793 | -0.711 | 1416 | 3.36 | 0.637 |
| 1.40 | -1.495 | -1.460 | -1.483 | 1.080 | -0.039 | -0.794 | -0.709 | 1442 | 3.62 | 0.643 |
| 1.50 | -1.552 | -1.538 | -1.550 | 1.085 | -0.044 | -0.796 | -0.704 | 1479 | 3.92 | 0.649 |
| 1.60 | -1.598 | -1.608 | -1.605 | 1.087 | -0.051 | -0.798 | -0.697 | 1524 | 4.26 | 0.654 |
| 1.70 | -1.634 | -1.668 | -1.652 | 1.089 | -0.058 | -0.801 | -0.689 | 1581 | 4.62 | 0.660 |
| 1.80 | -1.663 | -1.718 | -1.689 | 1.087 | -0.067 | -0.804 | -0.679 | 1644 | 5.01 | 0.664 |
| 1.90 | -1.685 | -1.763 | -1.720 | 1.087 | -0.074 | -0.808 | -0.667 | 1714 | 5.42 | 0.669 |
| 2.00 | -1.699 | -1.801 | -1.743 | 1.085 | -0.085 | -0.812 | -0.655 | 1795 | 5.85 | 0.672 |

## 19.2 Attenuation Relationships Derived By Campbell (1997)

The following equations were derived by Campbell for the median value of peak horizontal acceleration ( $A_H$ in g):

$$\begin{aligned} Ln(A_H) = &-3.512 + 0.904M \\ &-1.328Ln\left\{\sqrt{R_{SEIS}^2 + \left[0.149\exp(0.647M)\right]^2}\right\} \\ &+\left[1.125 - 0.112Ln(R_{SEIS}) - 0.0957M\right]F \\ &+\left[0.440 - 0.171Ln(R_{SEIS})\right]S_{SR} \\ &+\left[0.405 - 0.222Ln(R_{SEIS})\right]S_{HR} \end{aligned}$$

In these equations, $M$ is moment magnitude, and the source-to-site distance, $R_{SEIS}$ , is the shortest distance between the recording site and the assumed zone of seismogenic rupture on the fault. Campbell indicates, based on the work of Marone and Scholz (1988), that the upper 2 to 4 km of the fault zone is typically non-seismogenic. The style of faulting variable, $F$ , is equal to zero for strike slip faulting and is equal to unity for all other style of faulting. The parameters $S_{SR}$ and $S_{HR}$ define the local site conditions as follows:

$S_{SR} = 1$ & $S_{HR} = 0$ for soft rock sites; and

$S_{SR} = 0$ & $S_{HR} = 1$ for hard rock sites

## 19.3 Attenuation Relationships Derived By Sadigh, Chang, Egan, Makdisi, And Youngs (1997)

The following equation was derived by Sadigh et al for spectral ordinates:

$$Ln(y) = C_1 + C_2 M + C_3 (8.5 - M)^{2.5} + C_4 Ln\left[r_{rup} + exp(C_5 + C_6 M)\right] + C_7 Ln\left(r_{rup} + 2\right)$$

$y$ is the median spectral acceleration in g , or peak ground acceleration (PGA), in g's, $M$ is moment magnitude, $r_{rup}$ is the closest distance to the rupture plane in km, and $C_1 ... C_7$ are coefficients. The values of the standard error terms are listed in **Table 14**. The values of the coefficients $C_1 ... C_7$ are provided in **Table 15**.

**Table 14:** Coefficients for Standard Error Terms Using Equations Derived by Sadigh et al (1997)

| Period – sec | Standard Error Term | Minimum Value for $M \geq 7.21$ |
|---|---|---|
| Zpa | $1.39 - 0.14M$ | 0.38 |
| 0.07 | $1.40 - 0.14M$ | 0.39 |
| 0.10 | $1.41 - 0.14M$ | 0.40 |
| 0.20 | $1.43 - 0.14M$ | 0.42 |
| 0.30 | $1.45 - 0.14M$ | 0.44 |
| 0.40 | $1.48 - 0.14M$ | 0.47 |
| 0.50 | $1.50 - 0.14M$ | 0.49 |
| 0.75 | $1.52 - 0.14M$ | 0.51 |
| ≥1.00 | $1.53 - 0.14M$ | 0.52 |

**Table 15**: Coefficients for the Median Spectral Ordinates Using Equations Derived by Sadigh et al (1997)

| Period | $C_1$ | $C_2$ | $C_3$ | $C_4$ | $C_5$ | $C_6$ | $C_7$ |
|---|---|---|---|---|---|---|---|
| | | | | $M \leq 6.5$ | | | |
| zpa | -0.624 | 1 | 0.000 | -2.100 | 1.29649 | 0.250 | 0.000 |
| 0.03 | -0.624 | 1 | 0.000 | -2.100 | 1.29649 | 0.250 | 0.000 |
| 0.07 | 0.110 | 1 | 0.006 | -2.128 | 1.29649 | 0.250 | -0.082 |
| 0.1 | 0.275 | 1 | 0.006 | -2.148 | 1.29649 | 0.250 | -0.041 |
| 0.2 | 0.153 | 1 | -0.004 | -2.080 | 1.29649 | 0.250 | 0.000 |
| 0.3 | -0.057 | 1 | -0.017 | -2.028 | 1.29649 | 0.250 | 0.000 |
| 0.4 | -0.298 | 1 | -0.028 | -1.990 | 1.29649 | 0.250 | 0.000 |
| 0.5 | -0.588 | 1 | -0.040 | -1.945 | 1.29649 | 0.250 | 0.000 |
| 0.75 | -1.208 | 1 | -0.050 | -1.865 | 1.29649 | 0.250 | 0.000 |
| 1 | -1.705 | 1 | -0.055 | -1.800 | 1.29649 | 0.250 | 0.000 |
| 1.5 | -2.407 | 1 | -0.065 | -1.725 | 1.29649 | 0.250 | 0.000 |
| 2 | -2.945 | 1 | -0.070 | -1.670 | 1.29649 | 0.250 | 0.000 |
| 3 | -3.700 | 1 | -0.080 | -1.610 | 1.29649 | 0.250 | 0.000 |
| 4 | -4.230 | 1 | -0.100 | -1.570 | 1.29649 | 0.250 | 0.000 |
| | | | | $M > 6.5$ | | | |
| zpa | -1.237 | 1.1 | 0.000 | -2.100 | -0.48451 | 0.524 | 0.000 |
| 0.03 | -1.237 | 1.1 | 0.000 | -2.100 | -0.48451 | 0.524 | 0.000 |
| 0.07 | -0.540 | 1.1 | 0.006 | -2.128 | -0.48451 | 0.524 | -0.082 |
| 0.1 | -0.375 | 1.1 | 0.006 | -2.148 | -0.48451 | 0.524 | -0.041 |
| 0.2 | -0.497 | 1.1 | -0.004 | -2.080 | -0.48451 | 0.524 | 0.000 |
| 0.3 | -0.707 | 1.1 | -0.017 | -2.028 | -0.48451 | 0.524 | 0.000 |
| 0.4 | -0.948 | 1.1 | -0.028 | -1.990 | -0.48451 | 0.524 | 0.000 |
| 0.5 | -1.238 | 1.1 | -0.040 | -1.945 | -0.48451 | 0.524 | 0.000 |

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| 0.75 | -1.858 | 1.1 | -0.050 | -1.865 | -0.48451 | 0.524 | 0.000 |
| 1 | -2.355 | 1.1 | -0.055 | -1.800 | -0.48451 | 0.524 | 0.000 |
| 1.5 | -3.057 | 1.1 | -0.065 | -1.725 | -0.48451 | 0.524 | 0.000 |
| 2 | -3.595 | 1.1 | -0.070 | -1.670 | -0.48451 | 0.524 | 0.000 |
| 3 | -4.350 | 1.1 | -0.080 | -1.610 | -0.48451 | 0.524 | 0.000 |
| 4 | -4.880 | 1.1 | -0.100 | -1.570 | -0.48451 | 0.524 | 0.000 |

Note that the above coefficients are applicable to ground motions generated by a strike slip event. Sadigh et al suggest that the calculated spectral ordinates be multiplied by a factor of 1.2 for reverse / thrust events.

# 20. Seismic Studies of area around Bhagyam Field (Well Pads connecting the Mangla Processing Terminal – MPT) near Bharka-Barmer, Rajasthan

## 20.1 Executive Summary

A seismic hazard study has been carried out for Bhagyam oil field around Bharka village to identify active faults and to ascertain design PGA (peak ground acceleration) value. As no major fault topography was evident from field survey/satellite imageries, it has been concluded no faults cross the N-S aligned pipeline connecting the well pads. On the contrary, probable traces of active faults have been identified through satellite data interpretation and field survey along Bharka – Baisali and Barmer – Chauhtan transects. The Baisali fault is a north dipping reverse fault having lateral extend of about 15 km and located at about 16 km from the proposed Bhagyam oil field. The Barmer – Chauhtan fault is also a north dipping reverse fault with ENE-WSW strike extends for about 45 km and located at approximately 30 km southwest of Bhagyam oil field. Both faults are capable of producing earthquake of magnitude 6-6.5 and hence, should be considered for estimating the peak ground acceleration.

Along with the faults identified during present study the active faults that exist (Konoi Fault, Nagar-Parker Luni-Sukri Fault, Allah Bund Fault) in the vicinity of about 250 km of radius from the well pad site were also considered for estimating the PGA value.

Based on the faults identified and using various attenuation relationships, the design PGA values were estimated for the design of pipeline. The Baisala fault was found to govern the PGA value, which is recommended as 0.25g for this project. The site showing no susceptibility to liquefaction, negligible ground deformation and with no fault crossing the pipeline, the safety of the pipeline shall be checked against seismic wave propagation only

.

## 20.2. Introduction to Seismic Studies of area around Bhagyam Field

Scope of the work undertaken is as follows

- Determine the location and nature of faults in and around the area based on published information (e.g., Seismotectonic Atlas of India), satellite imageries and a field visit
- Carry out a deterministic seismic hazard analysis and provide recommendations on ground motion to be considered for seismic studies
- Liquefaction potential evaluation of the soil based on SPT or CPT data and make recommendations on seismic analysis parameters

With the readily available LANDSAT and Shuttle Radar Topographic Mission (SRTM) data for proposed area, an attempt was made to identify the locations of fault topography around the Bharka-Barmer area through a detailed fieldwork carried during 17-20 May, 2010. The present study suggests that no prominent fault topography is preserved in the immediate vicinity of the pipeline connecting the well pads located west of Bharka village (Figure 26.1). However a few signatures such as occurrence of shear zone and nearly vertically stacked succession in rocks of Barmer Foundation comprised of sandstone along with igneous rocks like granite and rhyolite; linearly aligned ridges along Barmer-Chauhtan transect as well as along Baisala-Harsana transect are suggestive of probable traces of active faults striking in ENE-WSW direction (Figure 26.2; 26.3a, 26.3b and 26.4).

## 20.3. Tectonic setting and seismicity around Barmer-Baisala-Bharka

The area of study around Barmer is partly occupied by the hills, whereas, most of the tract is covered by alluvium. The structural framework and deformational regime of the area around Rajasthan-Gujarat suggests that the area has been tectonically controlled by the interplay of two major Precambrian tectonic trends i.e. , NNW-SSE Dharwarian trends and ENE-WSW Delhi-Aravalli trends (Figures 26.1 and 26.2). These two trends are represented in form of several faults and lineaments, and the area indicates a sequential reactivation of these faults, some of which display even recent activity (GSI, 2000). These lineaments were classified as L1 to L4 starting from north part in Rajasthan and similarly faults were classified as F1 to F8 (Figure 26.1). Two major lineaments L1 - Jaisalmer-Barwani Lineament striking NNW-SSE and L2 – Rajkot Lathi Lineament striking NNE-SSW are marked close to Barmer (Figure 26.1). It has been suggested that the Western Marginal Fault of Cambay Basin merges with Jaisalmer-Barwani Lineament near Barmer. This lineament delimits the western boundary of the Mesozoic-Cenozoic basin in the Barmer area as well as demarcates the boundary between the Barmer Graben in the east and Birmania-Barmer-Nagar Parker Horst in the west. Thus it has been considered as the surface trace of a deep seated fault controlling basin configuration. The Konoi Fault (F1) which marks the boundary between the up-thrown eastern block of Jaisalmer-Mari Arch and the down thrown western block of Shahgarh sub-basin, lies close to this lineament.

## 20.4. Seismicity of the area

The study area around Barmer does not show any prominent occurrence of earthquakes during recent historic past. The Jaisalmer earthquake of 08 November, 1991 with Mb 5.5 is the only so far reported earthquake from this area (GSI 2000), It has been suggested that this event was probably triggered along Konoi Fault (refer F1 fault in Figure 26.1) with maximum shaking intensity of VIII. Apart from this earthquake events of this area are shallow focus and dominate by magnitude from ranging from 4.0 to 5.0. Spatial distributions of the epicenters of these events delineate two belts: one is in the vicinity of the tectonically active linear domain in the Barmer-Ramgarh tract; the other in the southern part in the proximity of the Luni-Sukri Lineament. The fundamental tectonic linears delineating the Dharwar trend and the Delhi-Aravalli trend seem to have reactivated during Recent times (GSI, 2000).

## 20.5. Methodology

For the present study, available satellite data covering the area around Barmer-Bharka-Baisala has been used. Digital Elevation Model (DEM) prepared from Shuttle Radar Topographic Mission Data (SRTM) with resolution of about 90 m and LANDSAT data with resolution of 28.5 m was also used. Several sites showing probable location of fault scarps were identified with help of above mentioned satellite data. To get the locations of the identified sites, coordinates were extracted from Google Earth. Based on satellite data interpretation and coordinates, site verification of these locations around Barmer-Bharka-Baisala carried out depending on their accessibility and approachability.

Detailed field survey was conducted from 17 to 20 May, 2010 covering the area around Barmer-Bharka-Baisala in Rajasthan (Figure 26.2-26.4)

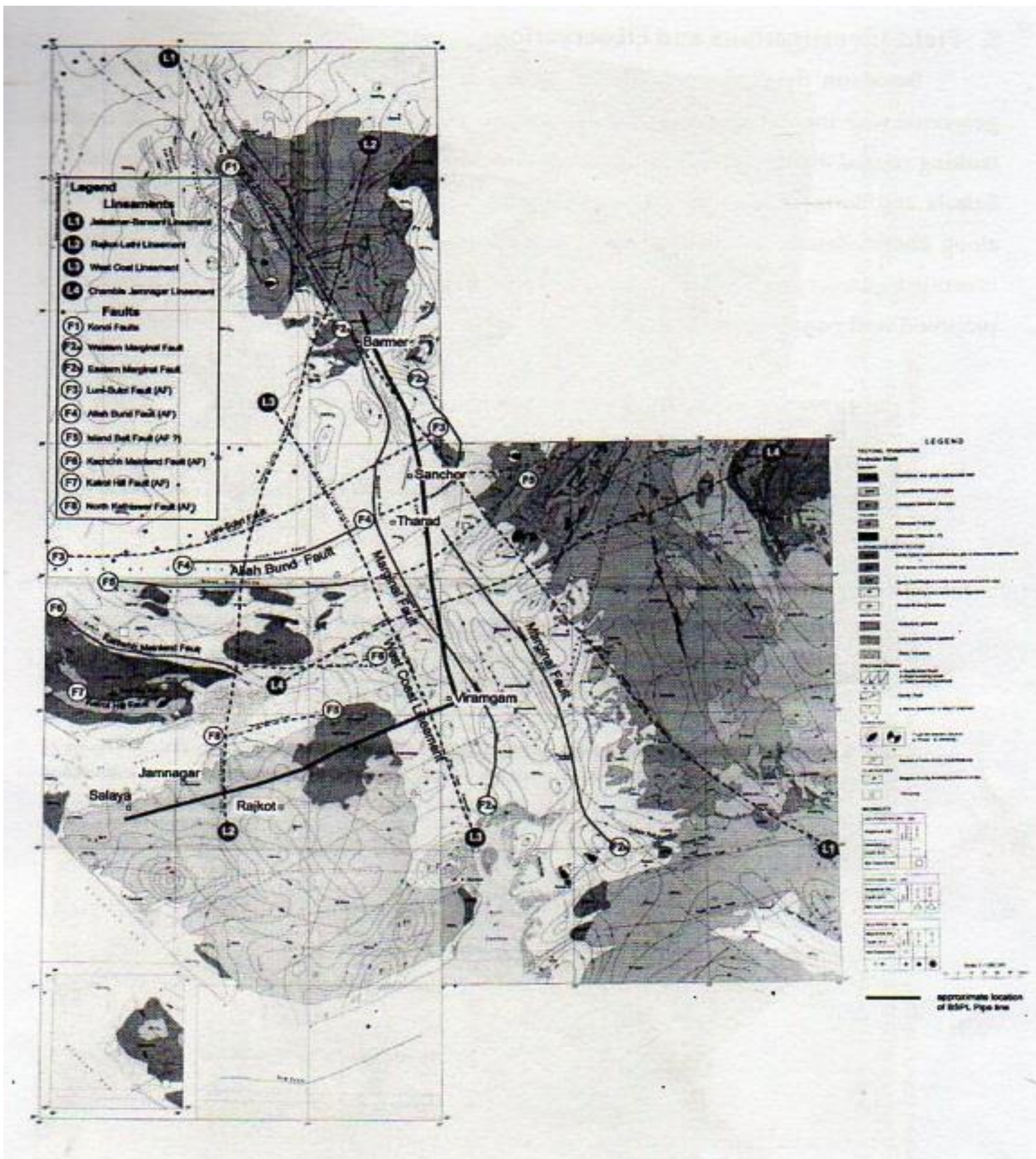




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**Figure 26.1:** Seismo-tectonic and geological map of northwest Rajasthan and Gujarat. Bold line shows approximate location of proposed crude oil pipeline from Barmer-Viramgam-Salaya (BSPL). The area falls in seismic zone III and IV with an adjoining region of Kachchh in zone V. Broken lines show traces of major NNW-SSE, NNE-SSW and ENE-WSW trending lineaments. Fault are marked by light tone lines, broken light tone lines are the inferred faults. Except the Cambay Graben marginal faults trending in NNW-SSE all other faults strikes in E-W and ENE-WSW direction. The lineaments are given number from L1 to L4 and the faults from F1 to F8.

## 20.6. Field Investigations and Observations

Based on detailed interpretation of the satellite data and 3D perspective views generated with the SRTM and LANDSAT images, several locations showing probable active faulting related topography were identified. Ground trothing was carried out along Bharka-Baisala and Barmer-Chauhtan transects (Figures 26.2 and 26.4). The survey was first conducted along Bharka-Baisala and then along Barmer-Chauhtan transects with an aim to locate the identified geomorphic features indicative of probable fault topography in the vicinity of proposed well pads (Figure 26.2-26.4)

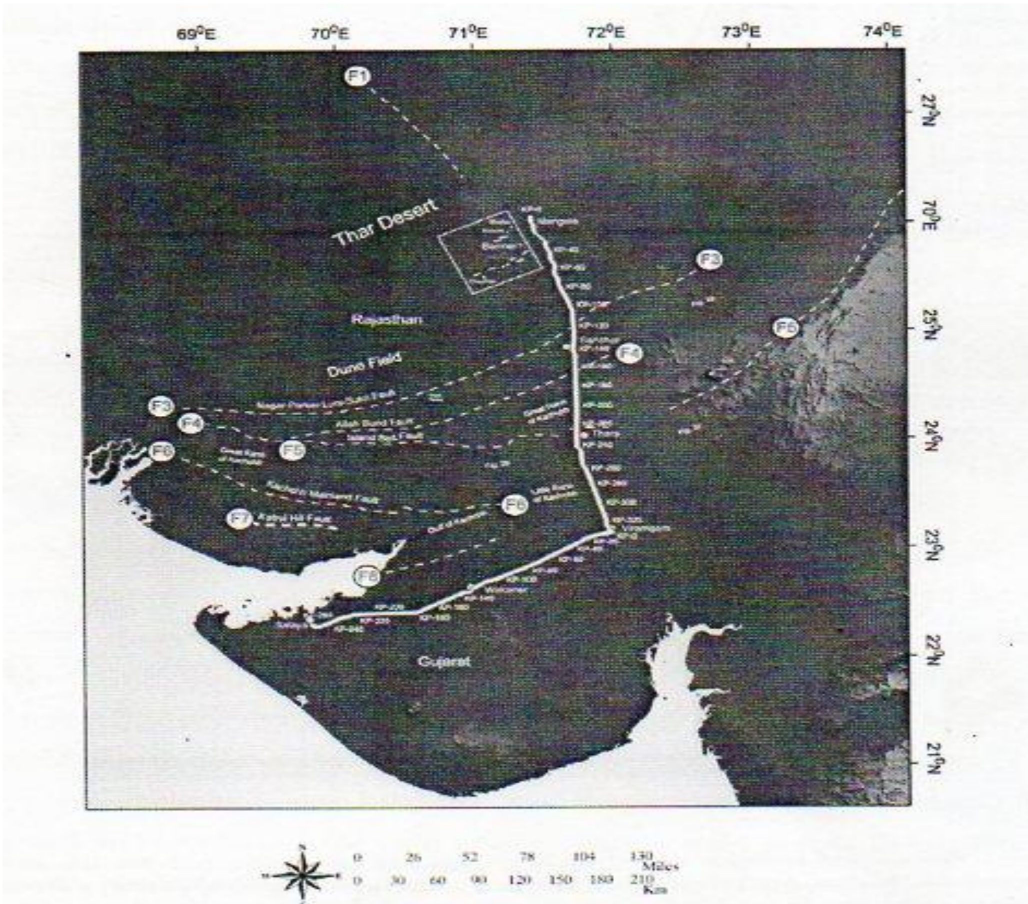


**Figure 26.2:** Digital Elevation Model (DEM) of the area around Rajasthan and Gujarat. Location and distribution major active faults are shown with respect to the proposed Barmer-Viramgam-Salaya Crude Oil Pipeline (BSPL). The pipeline is marked by thick white line from Barmer to Viramgam in NNW-SSE direction and from Viramgam to Salaya in ENE-WSW direction. Box marks the area of study around Bhagyam oil field around Barmer-Bharka towns near Barmer Basin. The ground truthing around Barmer-Bharka suggests discontinues trace of two active faults striking in ENE-WSW direction.

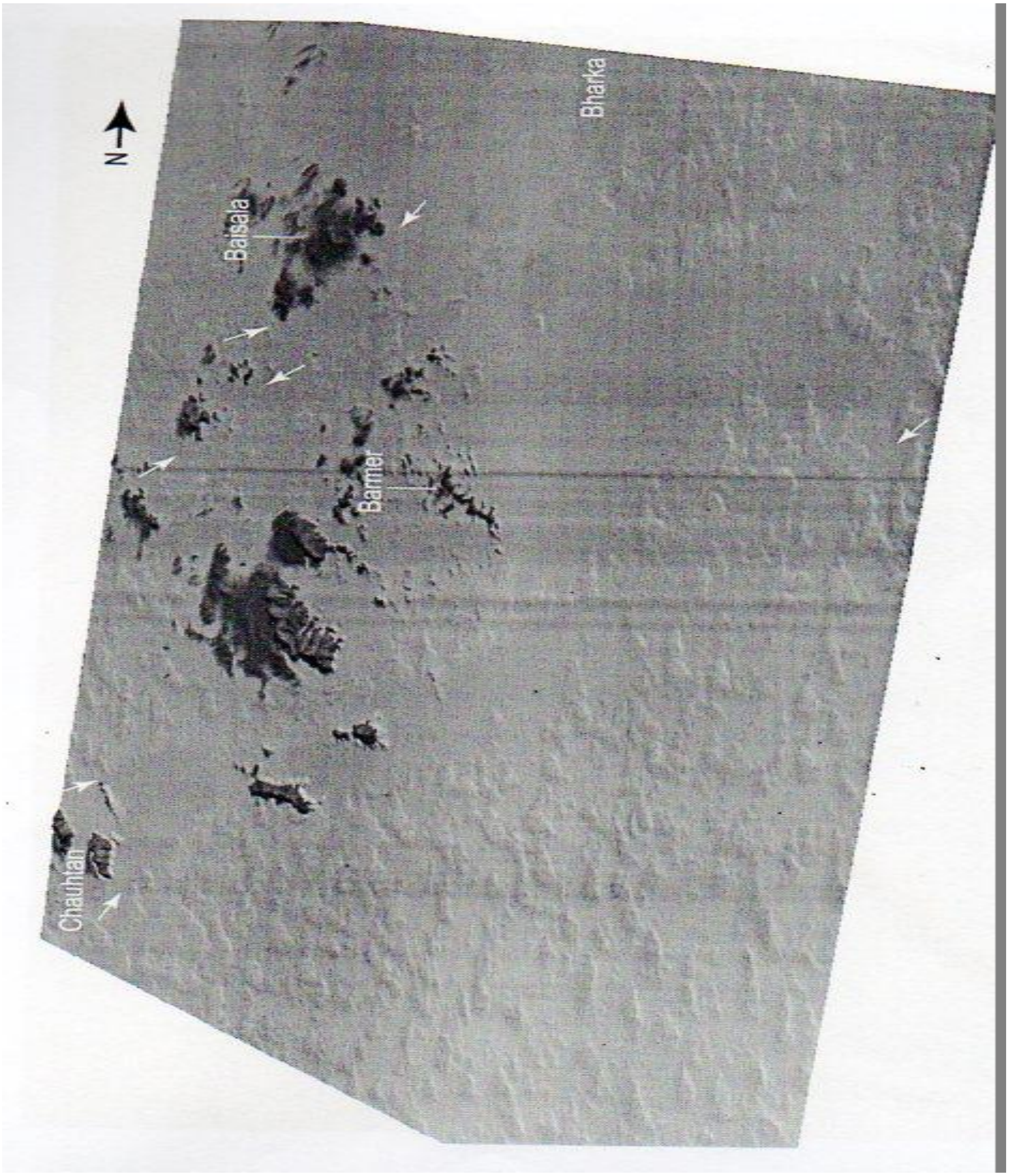


**Figure 26.3a:** DEM generated from SRTM data of the area around Barmer-Bharka. Linear features marked by arrows are the probable indicative of active fault traces. Active fault traces

are discontinuous on the surface, at places marked by highly sheared zone with vertically stacked beds near Baisala village, whereas along Barmer-Chauhtan transect linearly aligned ridges covered by sand-dunes were observed striking in ENE-WSW direction.

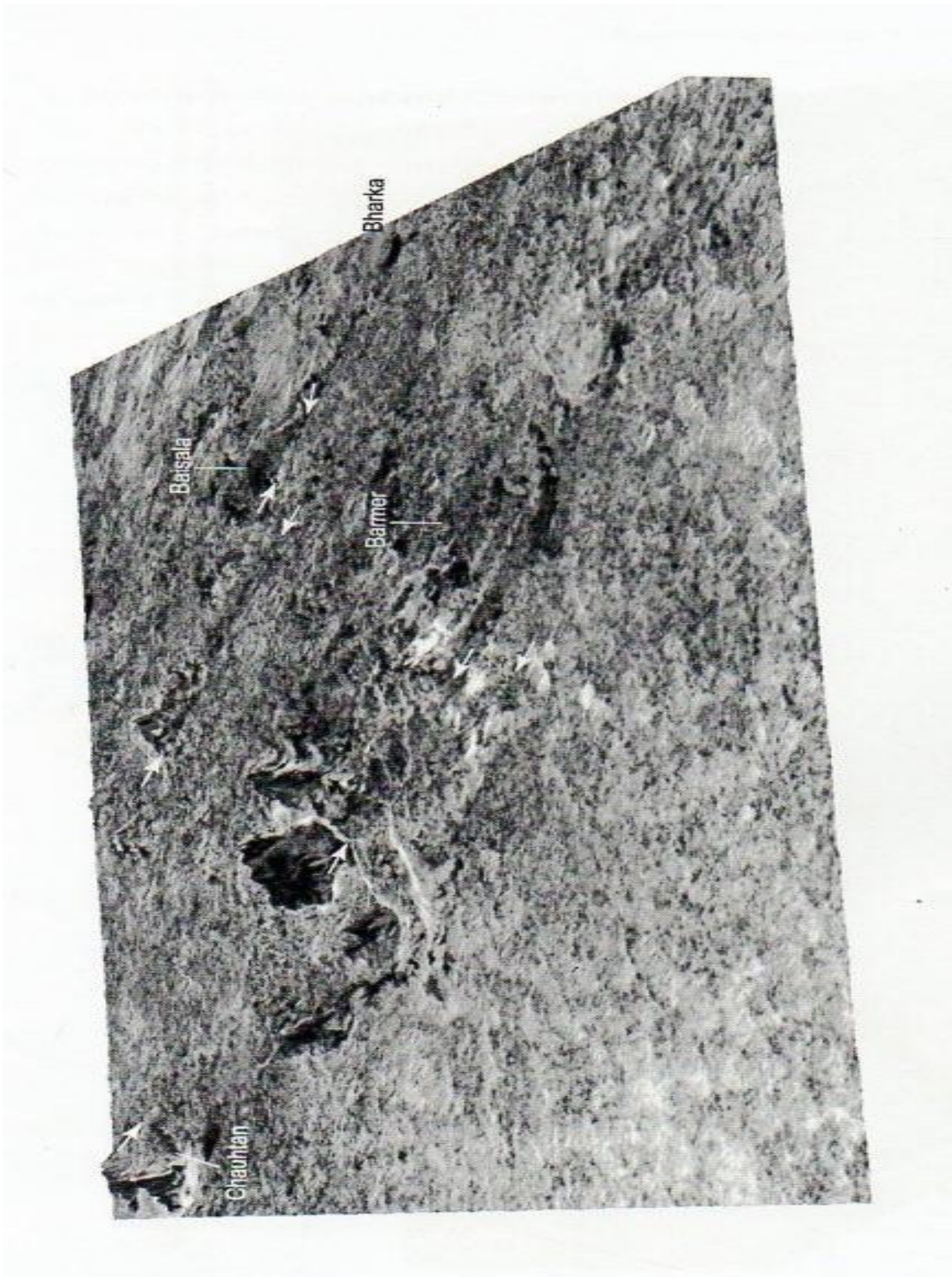


**Figure 26.3b:** 3D perspective view of the terrain around Barmer-Baisala-Chauhtan region in the vicinity of Bhagyam oil field. Probable traces of active fault are marked by arrows.


**Figure 26.4:** Google Earth image of the area showing discontinuous traces of active fault between Barmer and Chauhtan towns, and between Bharka and Baisala villages. WP1-15 locations of oil well pads around Bharka village. Locations identified for ground truthing are

marked as: BB1a-BB1d around Bharka; Ba1-Ba3a around Baisala village; and Bam1-Bam6 and Ch1-Ch4 around Barmer and Chauhtan respectively.

## 20.6.1 Bharka-Baisala transect:

Several locations for ground truthing showing probable indication of active fault related topography were identified on satellite data (Figures 26.3a and b; 26. 4). Ground truthing around Bharka (BB1a-BB1e) suggested that the elevated landscape with northwest side up is an erosional landform marked by gully erosion (Figure 26.5). Due to lack of any prominent active fault related features like linearly aligned ridges/fault scarp/uplifted terraces it is difficult to suggest the recent tectonic movement around this area.


**Figure 26.5:** Google Earth image showing location Well Pads 1-15 of Bhagyam oil field and location identified to undertake ground truthing around Bharka village. The elevated landscape marked by gully erosion due to headward erosion within drainage basin is encircled by black line.

Fieldwork carried out around Baisala village (Ba1, Ba2, Ba2a, Ba3, Ba3a) revealed prominent active fault related features. Along with occurrence of linear ridges a prominent shear zone marked by crushed material and nearly vertically stacked sandstone beds of Barmer Formation were observed at location Ba2a (25°52’47.4”N, 71°14’43.32”E), along with this at a few locations crushed igneous rocks (granite and rhyolite) were also observed (Figures 26.3, 26.4, 26.6 and 26.7). The exposed sections along the ENE-WSW striking ridges show higher inclination of beds with dip of 65°-75° towards north (Figures 26.6-26.8). Crushed and vertically stacked basic rocks along with sedimentary succession were also observed on the way to Bola village (Ba2: 25°51’42.94”N, 71°14’43.32”E). The fault traces are discontinuous with variable strike ranging from ENE-WSW to E-W (Figures 26.3a, 26. b and 26.6). The faulting along these traces has resulted in north side up causing vertical stacking of sandstone succession and shearing of rocks. The presence of shear zone and vertically stacked rocks suggest deformation along a north dipping reverse fault.

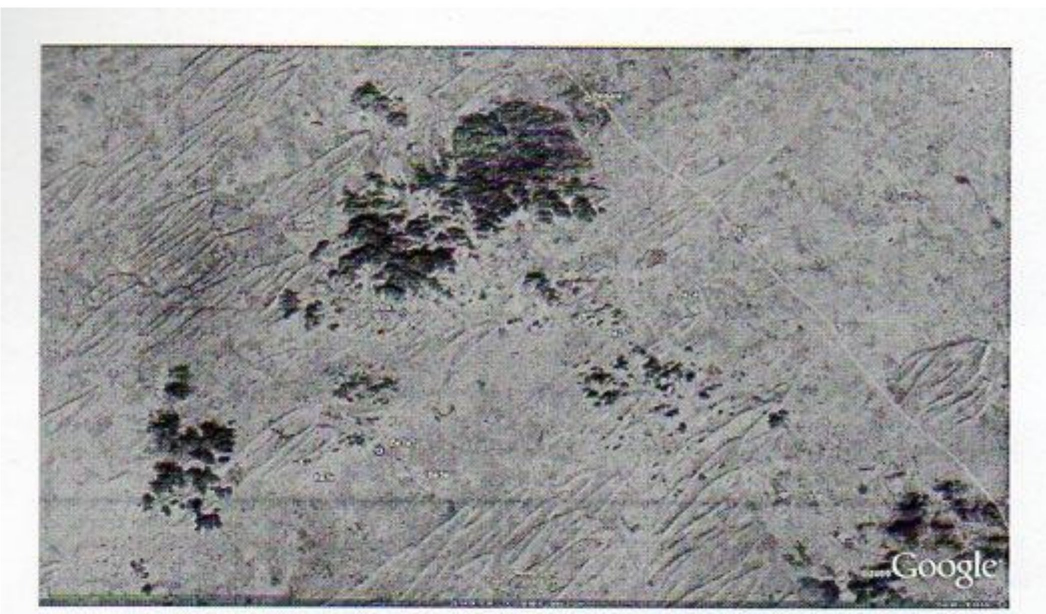


**Figure 26.6:** Google earth image showing locations identified for ground truthing around Baisala village. Inferred active fault trace is marked by light tone line (Baisala fault). The fault traces are discontinuous with variable strike ranging from ENE-WSW to E-W. The faulting along these traces has resulted in north side up causing vertical stacking of sandstone succession and shearing of rocks.

**Figure 26.7:** Exposed section along ENE-WSW striking linear ridges showing vertically stacked sandstone succession. Dip of the beds ranges from 65-70 towards north. Shearing and tilting of rocks is related to the deformation along north dipping Baisala fault (reverse fault). Photo looking towards ENE.

Along with this at places linearly aligned ridges were also observed at Ba3a1: 25°50’20.6”N, 71°11’34.00”E (Figures 26.3a, 26. b, 26.6 and 26.9). The surface topography is marked by stabilized to partially stabilized dune ranging in height of about 15-20 m. This fault trace has been named as Baisala fault, extends for about 15 km and located at a distance of about 16 km southwest of Bharka (well pads) (Figure 26.4).

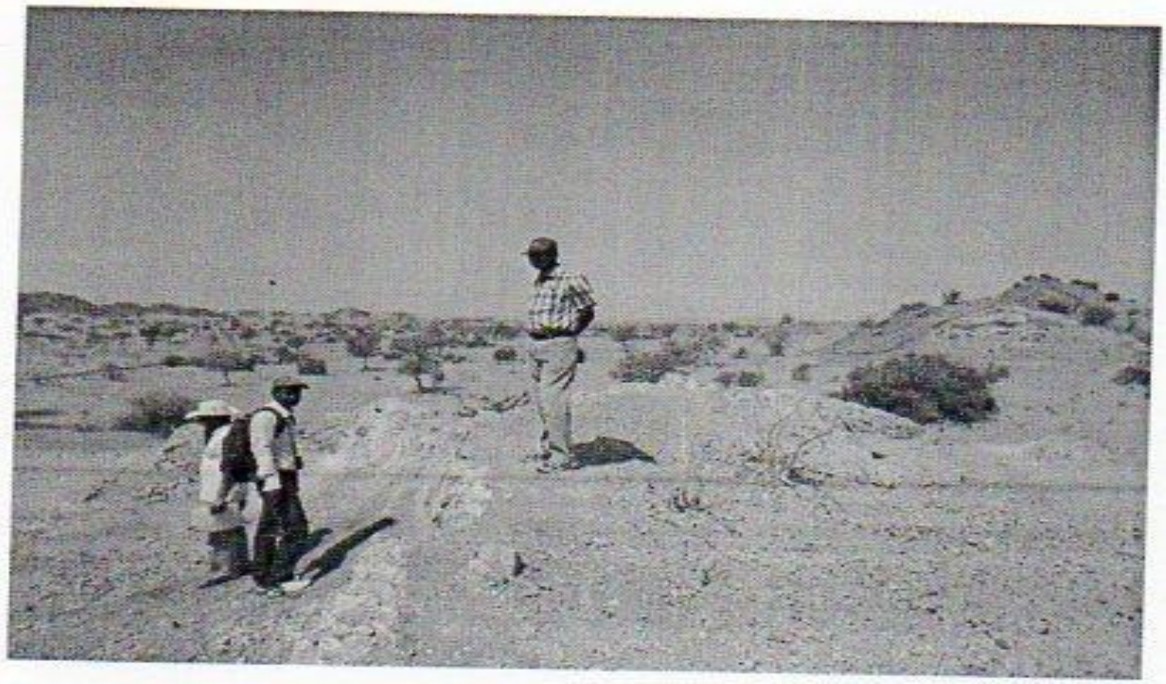

**Figure** 26.**8**: E-W striking linear ridges of basic igneous rock near location Ba2. Photo looking towards west.

**Figure 26.9:** Southeast facing scarp suggestive of probable fault trace near Baisala village (Ba3a1: 25°50’20.6”N, 71°11’34.00”E). The surface topography is marked by stabilized to partially stabilized dune ranging in height of about 15-20 m. Photo looking northwest.

## 20.6.2 Barmer-Chauhtan transect:

Several locations – Bam 1-Bam 6 and Ch1-Ch4 were identified for ground truthing based on the linear features observed on satellite data (Figure 26.2-26.4 and 26.10). Linearly aligned hills composed of igneous rocks (jointed granitoid, basalt and rhyolite) as well as sedimentary succession was observed along the Barmer-Chauhtan transect. Prominent shear zone and vertically stacked beds were observed at location Bam2a: 25°42’57.2”N, 71°22’06.4”E (Figures 26.10 and 26.11).

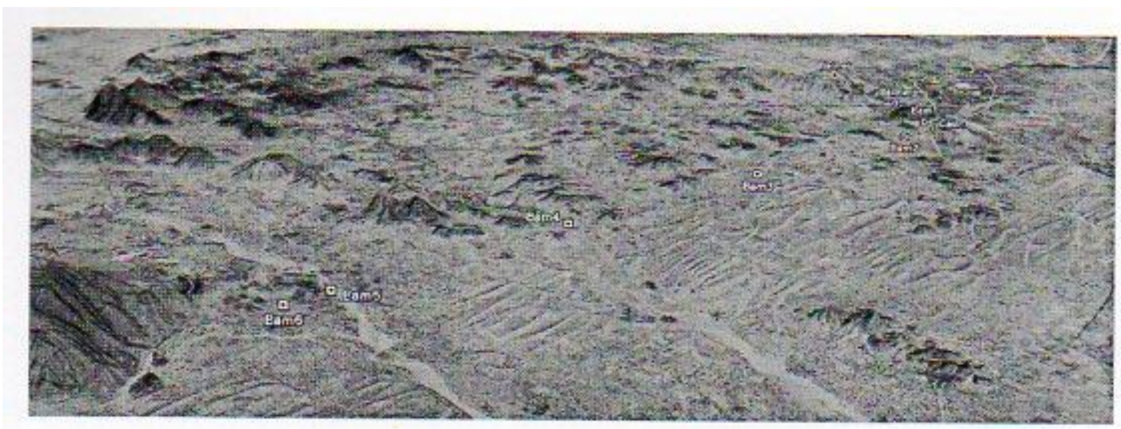


**Figure 26.10:** Google earth image showing locations (Bam 1 to Bam 6) identified for ground truthing along Barmer-Chauhtan transect. Red arrow marks probable trace of active fault.

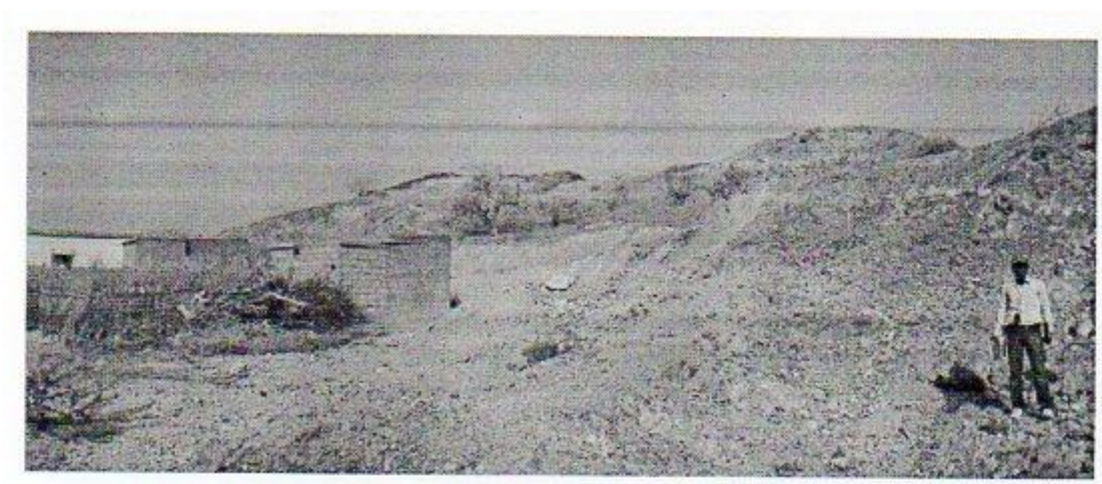

**Figure 26.11:** Exposed section along ENE-WSW striking linear ridges showing shear zone at location Ba2a (25°42’57.2”N, 71°22’06.4”E).

At places further west along Barmer Chauhtan highway linearly aligned ridges striking ENE-WSW with a surface topography marked by stabilized to partially stabilized dune ranging in height of about 15-20 m were also observed at Ch3 (25°31’47.91”N, 71°3’26.14”E) and Ch4 (25°35’39.60”N, 71°14’1.30”E) (Figures 26.3a, b, 26.4, 26.12, and 26.13). The fault traces observed between Barmer and Chauhtan are discontinuous with ENE-WSW strike (Figure 26.4). This fault has been named as Barmer-Chauhtan fault, extends in a stretch of about 45 km and located at a distance of about 30 km southwest of Bharka (well pads) (Figure 26.4). The southeast facing fault topography along the Barmer-Chauhtan fault suggests deformation along a north dipping reverse fault.

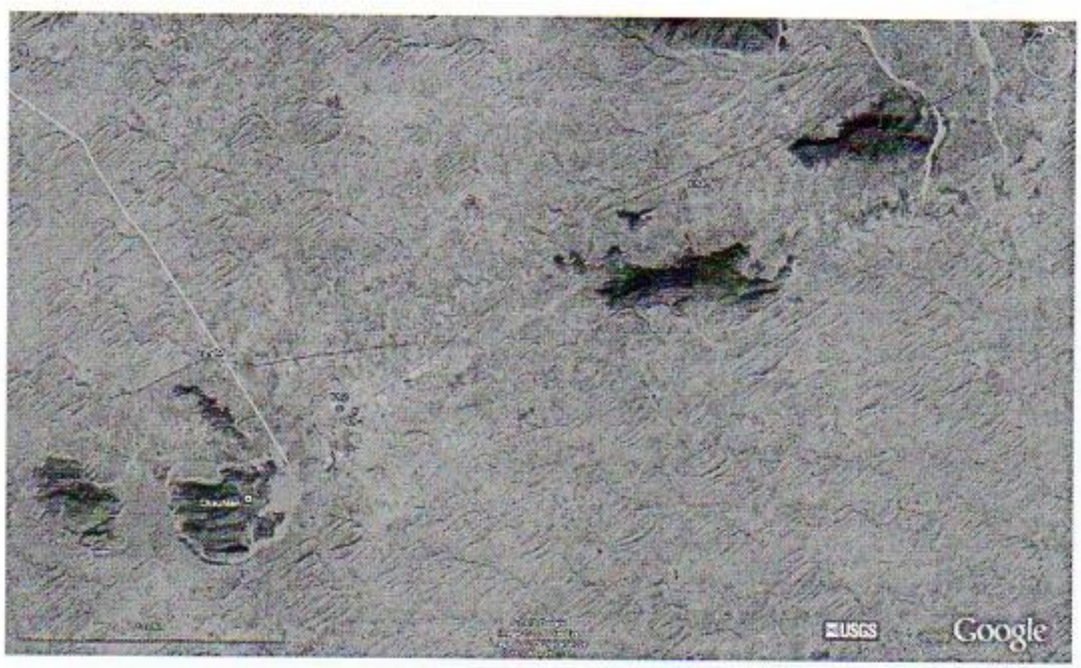


**Figure 26.12:** Google earth image showing locations identified for ground truthing around Chauhtan village. Inferred active fault trace is marked by light tone line (Barmer-Chauhtan fault). The fault traces are discontinuous with ENE-WSW strike.

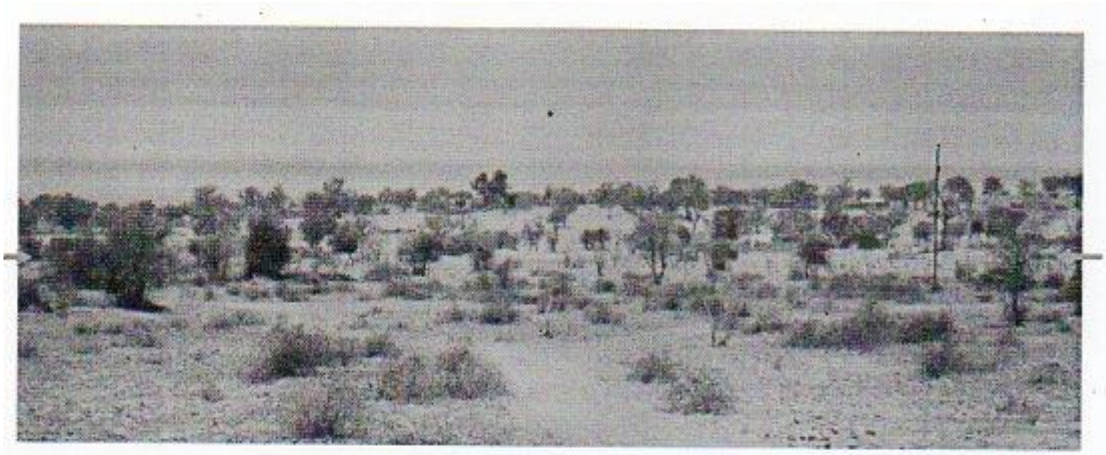

**Figure 26.13:** Southeast facing scarp suggestive of probable fault trace at location Ch4 (25°35'39.60"N, 71°14'1.30"E) on the way to Barmer from Chauhtan village. The topography is marked by stabilized to partially stabilized dune ranging in height of about 15 m. Photo looking northwest.

## 20.7. Seismic sources and hazard estimation

Based on the present study, probable traces of active faults identified in this study i.e. Baisala-Chauhtan faults must be considered for seismic hazard analysis, even though these do not cross the pipeline of well pads around Bharka. The Baisala fault is a north dipping reverse fault having lateral extend of about 15 km. This fault is located at about 16 km from the proposed Bhagyam oil field and is capable of producing earthquake of magnitude 6-6.5. The Barmer-Chauhtan fault is also a north dipping reverse fault with ENE-WSW strike, located at about 30 km southwest of Bhagyam oil field. It extends for about 45 km and is capable of producing earthquake of magnitude 6-6.5. Hence, these faults should be considered for estimating the ground motion.

Along with the faults identified during present study the active faults that exist in the vicinity of about 250 km of radius from the well pad site were also considered for estimating the ground motion (Table 16).

**Table 16:** Location of active fault traces with respect to Bhagyam oil field around Bharka

| S. No. | Fault No. | Name of the fault | Type of Fault | Dist. In km from Bharka (BOF) | Probable Magnitude |
|---|---|---|---|---|---|
| 1 | -- | Baisala Fault | North dipping thrust fault | ~16 | 6-6.5 |
| 2 | -- | Barmer-Chauhtan Fault | North dipping thrust fault | ~ 30 | 6-6.5 |
| 3 | F1 | Konoi Fault | Strike-slip fault | ~ 30 | 6.5 |
| 4 | F3 | Nagar-Parker Luni-Sukri Fault | North dipping thrust fault | ~ 105 | 7.5 |
| 5 | F4 | Allah Bund Fault (ABF) | North dipping thrust fault | ~ 155 | 8 |
| 6 | F5 | Island Belt Fault (IBF) | South dipping thrust fault | ~ 210 | 7.5 |

The Konoi Fault striking in NNE-SSE direction extends for about 100 km. This fault has been marked by GSI as neotectonic fault. Based on the focal mechanism of 1991 event (Mb 5.5), this fault could be strike-slip fault. It does not cross the pipeline and is located about 30 km from the pipeline northwest of Barmer. With 100 km of its length this fault could generate a moderate earthquake of M 6.5. Hence, this fault should be considered for estimating the ground motion.

The Nagar-Parker Luni-Sukri Fault (NPLS) – F3 with strike E-W and ENE-WSW is a north dipping thrust fault located about 100 km from the Bhagyam oil field (Figures 26.1 and 26.2; Table 16). With about 400 km of its length this fault could generate earthquake of M 7.5.Hence,this fault should be considered for estimating the ground motion.

The Allah Bund Fault (ABF) – F4 striking E-W and ENE-WSW is a north dipping thrust fault located about 155 km from the proposed site (Figure 26.2; Table 16). This fault extends for about 170 km along its strike. Considering the occurrence of 1819 Allah Bund earthquake and paleoseismic investigations it is suggested that the fault could generate earthquake of M 8.0. Hence, this fault should be considered for estimating the ground motion.

The Island Belt Fault (IBF) – F5 striking E-W and ENE-WSW is a south dipping thrust fault located about 210 km from the proposed site (Figure 26.2). Considering the length this fault could generate an earthquake of about M 7.5. Hence, this fault should be considered for estimating the ground motion.

## 20.8. PGA Estimates

In the estimate of seismic risk, the determination of ground motion parameters like spectral characteristics, peak ground displacement and peak ground acceleration is very important for a quantitative assessment of the problem. Among these parameters an important parameter, peak ground acceleration (PGA) can be estimated using relationship between the magnitude of an earthquake and the distance away from the fault rupture which is called an attenuation relationship. These relationships are developed by statistical analyses performed on a large number of records which were obtained in compatible geomorphic regions. Most of these relationships are updated as new strong ground motion data becomes available and many now include additional parameters such as fault type and site soil conditions. In this study, the attenuation relations proposed by Boore et al. (1997), Campbell (1997) and Sadigh et al (1997) were used to estimate PGA value for each fault. The major faults with significant influence at the site are: Konoi fault, Nagar-parker Luni-Sukri fault and Barmer-Chauhtan fault as listed in Table 17.

The controlling fault for PGA estimates in Baisala thrust fault at ~ 16 km from the site, with a probable M6.5 event on this fault. At assumed depth of 20 km for this event for deep and stiff soils with 600 m/s shear wave velocity, various attenuation relations gives mean

PGA varying from 0.15-o.18 g, whereas mean plus values range between 0.23-0.30 g (Table 17). Considering the importance of the project, it is prudent to base the design on mean plus sigma values of PGA and a value of 0.25g is suggested for this project.

**Table 17:** Mean and Mean + Sigma Values of Peak Horizontal Ground Acceleration by Attenuation Relations

| **S. No.** | **Fault No.** | **Name of the fault** | **Type of Fault** | **Dist. In km from Bharka (BOF)** | **$M_g$** | **PGA Values (g)** | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | **Mean** | | | **Mean + Sigma** | | |
| | | | | | | **Boore et al. 1997** | **Campbell 1997** | **Sadigh et al. 1997** | **Boore et al. 1997** | **Campbell 1997** | **Sadigh et al. 1997** |
| 1 | -- | Baisala Fault | North dipping thrust fault | ~16 | 6.5 | 0.175 | 0.150 | 0.162 | 0.295 | 0.233 | 0.242 |
| 2 | -- | Barmer-Chauhtan Fault | North dipping thrust fault | ~ 30 | 6.5 | 0.111 | 0.096 | 0.112 | 0.186 | 0.158 | 0.167 |
| 3 | F1 | Konoi Fault | Strike-slip fault | ~ 30 | 6.5 | 0.091 | 0.087 | 0.087 | 0.153 | 0.145 | 0.141 |
| 4 | F3 | Nagar-Parker | North dipping thrust fault | ~ 105 | 7.5 | 0.072 | 0.046 | 0.061 | 0.121 | 0.085 | 0.091 |

| | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Luni-Sukri Fault | | | | | | | | | |
| 5 | F4 | Allah Bund Fault (ABF) | North dipping thrust fault | ~ 155 | 8 | 0.069 | 0.040 | 0.054 | 0.116 | 0.075 | 0.081 |
| 6 | F5 | Island Belt Fault (IBF) | South dipping thrust fault | ~ 210 | 7.5 | 0.042 | 0.018 | 0.023 | 0.070 | 0.037 | 0.035 |

## 20.9. Liquefaction Potential

Relevant stratigraphic details at well pad locations extracted from the first source listed in the preceding section are presented in together with the estimated clean-sand-equivalent, stress-normalized and energy-corrected SPT blow counts, $(N1)_{60CS}$, and the factor of safety against liquefaction estimated thence following Youd et al. (2001). The energy typically delivered by donut hammers used in Indian SPT setups was assumed to be 45% in the assessment. Further, the groundwater table was assumed to be at the ground surface for making the assessment in recognition for the potential for flash flooding. The estimated factors of safety at well pad locations were found to exceed 1.5 indicating a potential for volumetric strains of 0.25% or less. These values are expected to translate into permanent ground deformations of 50 mm or less within the top 2m from ground surface. Within deeper layers earthquake-related permanent ground deformation is expected to be negligible.

## 20.10. Design Parameters

The seismic design of buried pipeline can be carried out with the design parameters as given in Table 18.

**Table 18:** Seismic analysis and design parameter for buried pipeline in area around Bhagyam Field near Bharka-Barmer, Rajasthan

| Parameter | Value | Remark |
|---|---|---|
| PGA | 0.25g | Pipeline shall be designed for wave propagation only. The recommended PGA value of 0.25g includes the necessary importance factor. |
| Ground Amplification Factor, $I_g$ | 1.15 | Soil Class C |
| PGV (m/s)/PGA(m/s$^2$) | 94 | Considering stiff soil condition and characteristics of the fault controlling the seismic hazard at the site. (Baisala Fault, M=6.5,source to site distance =16 km) |
| Ground Strain Coefficient, $\alpha_e$ | 2.0 (for S Waves) | Shear wave velocity effect will be dominating as the governing site is within the epicentral distance of 5 times the focal depth |
| Apparent wave length of seismic wave, $\lambda$ | 1.0 km | Due to lack of reliable information about factors determining the apparent wavelength of seismic waves, a conservative value is recommended. |

## 20.11 Conclusion

Based on the field study conducted along Bharka-Baisala and Barmer-Chauhtan transects as well as around Bharka in vicinity of Bhagyam well pads, our conclusions are as follows:

1. No prominent fault topography was observed in the vicinity of well pads around Bharka village. The elevated landscape with northwest side up is an erosional landform marked by gully erosion. Hence, due to lack of prominent active fault topography it is concluded that no active fault trace is present around Bharka and no fault cross the N-S aligned pipeline connecting the well pads.

2. Satellite data interpretation and field survey along Bharka-Baisala and Barmer-Chauhtan transects suggests probable traces of active fault marked by shear zones, linear ridges comprised of sedimentary and igneous rocks. At places topography is marked by stabilized to partially stabilized dune.

3. The Baisala fault with a lateral extend of about 15 km with variable strike (ENE-WSW and E-W) is a north dipping reverse fault located at about 16 km from the proposed Bhagyam oil field. The Barmer-Chauhtan fault with ENE-WSW strike is also a north dipping reverse fault located at a distance of about 30 km southwest of Bhagyam oil field (well pads) and extends for about 45 km. Considering the length of these faults it is suggested that these faults could generate earthquake of ~ M 6-6.5. Hence should be considered for ground motion estimation.

4. Other active faults exist in the vicinity of about 250 km of radius from the proposed site namely Konoi Fault (KF); Nagar-Parker Luni-Sukri Fault (NPLS); Allah Bund Fault (ABF) and Island Belt Fault (IBF) are also considered for estimating ground motion.

5. The controlling fault for PGA estimates was Baisala thrust fault and various attenuation relations gives mean plus sigma PGA varying from 0.23-0.3g. Considering the importance of the project, a PGA of 0.25g is recommended for ground motion estimation and the same should be considered to check the safety of the pipeline against seismic wave propagation.

6. At well pad locations, factor of safety against liquefaction was found to exceed 1.5 indicating a potential volumetric strains of 0.25% or less which can be translate into permanent ground deformations of 50 mm or less within the top 2 m from ground surface. Considering no liquefaction and negligible PGD, pipeline safety check against buoyancy against liquefaction and PGD case are not envisaged.

# 21. SEISMIC ANALYSIS REPORT ON DETAILED ENGINEERING SERVICES FOR BHAGYAM FIELD DEVELOPMENT PROJECT

## 21.1 OBJECTIVE

The primary objective of the seismic study is to ensure that in field pipeline will have an adequate level of safety during its lifetime against probable earthquake in its vicinity.

## 21.2 SEISMIC STUDY ANALYSES

As per the seismic study carried out , since there is no fault crossing and no liquefaction , it is recommended that impact on pipeline due to active fault crossing, buoyancy due to liquefaction and permanent ground deformation caused by liquefaction are not to be considered and are hence not analyzed. However, the impact of seismic wave propagation is required to be analyzed.

## 21.3 PIPELINE PARAMETERS

The pipeline details as per FEED Document provided by M/s CEIL are as given below:

### 6.1 FOR 16”, 12”, 10”, 8” NB PRODUCTION FLUID PIPELINES:

**<u>Pipe Properties</u>**

| **Line Pipe Size** | **16”** | **12”** | **10”** | **8”** |
|---|---|---|---|---|
| Pipe Material Grade | API 5L<br>Gr. X-65 | API 5L<br>Gr. X-65 | API 5L<br>Gr. X-65 | API 5L<br>Gr. X-65 |
| Thickness of Coating (mm) | 1.5 | 1.5 | 1.5 | 1.5 |
| Pipe Outside Diameter (mm) | 406.4 | 323.9 | 273.1 | 219.1 |
| Wall Thickness (mm) | 9.7 | 7.1 | 7.1 | 6.4 |

**Other Properties:**

Modulus of Elasticity : $2x10^5$

Poisson's Ratio : 0.3

Coefficient of Thermal Expansion : $11.7 \times 10^{-6}$ /°C

Density of Pipe : 78500 $N/m^3$

Corrosion Allowance : 3mm

**Functional Parameters:**

Internal Design Pressure : 4.6 MPa

Design Temperature : 90°C

Installation Temperature : 25°C

Service : Production Fluid

Burial Depth : 1.2 m

(Corroded pipe wall thickness has been considered in analysis)

## 6.2 FOR 18", 10", 8", 6" NB WATER INJECTION PIPELINE:

## <u>Pipe Properties</u>

| **Line Pipe Size** | **18"** | **10"** | **8"** | **6"** |
|---|---|---|---|---|
| Pipe Material Grade | API 5L | API 5L | API 5L | API 5L |

| | Gr. X-65 | Gr. X-65 | Gr. X-65 | Gr. X-65 |
|---|---|---|---|---|
| Thickness of Coating (mm) | 1.5 | 1.5 | 1.5 | 1.5 |
| Pipe Outside Diameter (mm) | 457.2 | 273.1 | 219.1 | 168.3 |
| Wall Thickness (mm) | 14.3 | 11.1 | 9.5 | 6.4 / 7.1 |

## Other Properties:

Modulus of Elasticity : $2x10^5$

Poisson's Ratio : 0.3

Coefficient of Thermal Expansion : $11.7 \times 10^{-6}$ /°C

Density of Pipe : 78500 $N/m^3$

Corrosion Allowance : 3mm

## Functional Parameters:

Internal Design Pressure : 9.3 MPa

Design Temperature : 90°C

Installation Temperature : 25°C

Service : Water Injection Fluid

Burial Depth : 1.2 m

(Corroded pipe wall thickness has been considered in analysis)

## 21.4 SOIL PARAMETERS

### 21.4.1 FOR WAVE PROPOGATION

The type of soil and soil properties considered for analysis are as given below:

| | **Case 1** | **Case 2** |
|---|---|---|
| Soil Type | Sandy Silt or Silty Sand or Sand or Sand and Gravel | Remolded Clayey Silt or Silty Clay or Clayey Sand |
| Effective Unit weight of soil | 18 KN/$m^2$ | 17 KN/$m^2$ |
| Cohesion | 0 KPa | 20 Kpa |
| Angle of Internal Friction | 35° | 10° |

## 21.5 SEISMIC PARAMETERS

### 21.5.1 SEISMIC WAVE PROPOGATION

Effect of ground motion on pipeline due to fault movement is considered for faults in the vicinity of pipeline. The parameters considered are as follows (Refer Seismic Studies of area around Bhagyam Field (Well Pads connecting the Mangla Processing Terminal – MPT) near Bharka-Barmer, Rajasthan)

- Ground Strain Coefficient equal to 2 has been considered for S-Waves.
- Apparent wavelength of 1 km has been taken for seismic wave propagation.
- Ground Amplification Factor considered for calculation is 1.15.
- Ratio of PGV/PGA is considered as 94.
- Peak Ground acceleration considered for calculation is 0.25g.
- Velocities of seismic wave propagations for both the sectors are as follows:

| **Sl. No.** | **Waves** | **Velocity** |
|---|---|---|
| 1 | S-Wave (Shear Wave) | 600m/s |

# 22. ANNEXURE

## SAMPLE ANALYTICAL CALCULATIONS

# ANNEXURE-1: CALCULATION FOR 12" NB PRODUCTION FLUID PIPELINE

## 22.1 OPERATING STRAIN CALCULATIONS

**Input**

| | **Symbol** | **Value** | **Unit** |
|---|---|---|---|
| Internal pressure in pipe | P | 4.60E+06 | $N/m^2$ |
| Outer diameter of pipe | D | 0.3238 | m |
| Pipe wall thickness | T | 0.0041 | m |
| Modulus of elasticity of pipe material | E | 2E+11 | $N/m^2$ |
| Poisson's ratio | $\mu$ | 0.3 | |
| Specified Minimum Yield Strength of pipe material | $\sigma_y$ | 4.50E+08 | $N/m^2$ |
| Rameberg-Osgood parameter | N | 38.32 | |
| Rameberg-Osgood parameter | R | 31.5 | |
| Installation Temperature | $T_1$ | 25 | °C |
| Operating Temperature | $T_2$ | 90 | °C |
| Coefficient of thermal expansion | A | 1.17E-05 | m/m/°C |

**Output**

| | Symbol | Value | Unit |
|---|---|---|---|
| Longitudinal stress in pipe due to internal pressure | $S_p$ | 5.45E+07 | $N/m^2$ |
| Longitudinal strain in pipe due to internal pressure | $\varepsilon_p$ | 0.00027 | |

| | | | |
|---|---|---|---|
| Longitudinal stress in pipe due to temperature change | $S_T$ | -1.52E+08 | $N/m^2$ |
| Longitudinal strain in pipe due to temperature change | $\varepsilon_T$ | 0.00076 | |
| Total longitudinal strain in pipe due to operating loads | $\varepsilon_{oper}$ | -0.00049 | |

$$S_P = \frac{PDu}{2t} = \frac{(4.6E + 06) \times 0.3238 \times 0.3}{2 \times 0.0041} = 5.45E + 07\ N/m^2$$

$$\varepsilon_p = \frac{S_p}{E}\left(1 + \frac{n}{1+r}\left(\frac{S_p}{\sigma_y}\right)^r\right) = \frac{5.45E + 07}{2E + 11}\left(1 + \frac{38.32}{1 + 31.5}\left(\frac{5.45E + 07}{4.5E + 08}\right)^{31.5}\right) = 0.00027$$

$$S_r = E\alpha\ (T_2 - T_1) = (2E + 11) \times (1.17E - 05) \times\ (90 - 25) = -1.52E + 08\ Nlm^2$$

$$\varepsilon_p = \frac{S_r}{E}\left(1 + \frac{n}{1+r}\left(\frac{S_r}{\sigma_y}\right)^r\right) = \frac{-1.52E + 08}{2E + 11}\left(1 + \frac{38.32}{1 + 31.5}\left(\frac{-1.52E + 08}{4.5E + 08}\right)^{31.5}\right) = 0.00027$$

$$\varepsilon_{oper} = \varepsilon_p - \varepsilon_T = 0.00027 - 0.00076 = -0.00049$$

## 22.2 SOIL SPRING PROPERTIES TO REPRESENT PIPE-SOIL INTERACTION

### 22.2.1 AXIAL SOIL SPRING

**Input**

| | Symbol | Value | Unit |
|---|---|---|---|
| Outer diameter of pipe | D | 0.3253 | m |
| Soil cover above center of pipeline | H | 1.36265 | m |
| Coefficient of cohesion of backfill soil | C | 0 | $N/m^2$ |
| Effective unit weight of soil | Ў | 18000 | $N/m^3$ |
| Friction Factor | F | 0.6 | |
| Internal friction angle of soil | Φ | 35 | Degree |

**Output**

| | Symbol | Value | Unit |
|---|---|---|---|
| Interface angle of friction b/w soil & pipe | S | 21 | |
| Coefficient of soil pressure | $K_0$ | 0.4264236 | |
| Adhesion factor | A | 1.029 | |
| Axial soil resistance | $t_u$ | 6863 | N/m |
| Mobilizing displacement | $\Delta_t$ | 0.005 | m |
| Axial Spring stiffness | $2t_u/\Delta_t$ | 2745027.4 | N/m/m |

$$S = f\phi = 0.6 \times 35 = 21$$

$$K_0 = 1 - sin\phi = 1 - \sin 35 = 0.4264236$$

$$\alpha = 0.608 - 0.123c - \frac{0.274}{c^2+1} + \frac{0.695}{c^3+1} = 0.608 - 0.123 \times 0 - \frac{0.274}{0^2+1} + \frac{0.695}{0^3+1} = 1.029$$

$$t_u = \ \pi D c \alpha + \pi D H \breve{y} \left(\frac{1+K_0}{2}\right) tanS$$

$$= \ 3.14 \times 0.3253 \times 0 \times 1.029 + 3.14 \times 0.3253 \times 1.36265 \times 18000$$

$$\times \left(\frac{1+0.4264236}{2}\right) tan\ 21 = 6863\ N/m$$

## 22.2.2 LATERAL SOIL SPRING

**Inputs**

| | Symbol | Value | Unit |
|---|---|---|---|
| Outer diameter of pipe | D | 0.3253 | M |
| Soil cover above center of pipeline | H | 1.36265 | M |
| Coefficient of cohesion of backfill | C | 0 | $N/m^2$ |
| Effective unit weight of soil | Ÿ | 18000 | $N/m^3$ |
| Parameters for Horizontal bearing | | | |
| Factor for $N_{ch}$ | A | 0 | |
| Factor for $N_{ch}$ | b | 0 | |
| Factor for $N_{ch}$ | C | 0 | |
| Factor for $N_{ch}$ | d | 0 | |
| Parameters for Horizontal bearing | | | |
| Factor for $N_{qh}$ according to Φ | A | 6.816 | |
| Factor for $N_{qh}$ according to Φ | b | 2.019 | |
| Factor for $N_{qh}$ according to Φ | C | -0.146 | |
| Factor for $N_{qh}$ according to Φ | d | 7.651E-03 | |
| Factor for $N_{qh}$ according to Φ | e | -1.683E-04 | |

**Outputs**

| | Symbol | Value | Unit |
|---|---|---|---|
| Factor for $N_{ch}$ & $N_{qh}$ | X | 4.1889 | |
| Horizontal bearing capacity factor $N_{ch}$ (should be ≤ 9 ) | $N_{ch}$ | 0 | |
| Horizontal bearing capacity factor $N_{qh}$ | $N_{qh}$ | 13.222 | |
| Maximum displacement of soil per unit length of pipeline | $P_u$ | 105497 | N/m |
| Mobilizing displacement of soil in lateral direction (should be ≤ 0.01 D to 0.02D) | $\Delta_p$ | 0.1525 | M |
| Lateral soil Spring stiffness | | 1383298.3 | N/m/m |

$$x = \frac{H}{D} = \frac{1.36265}{0.3253} = 4.1889$$

$$N_{ch} = a + bx + \frac{c}{(x+1)^2} + \frac{d}{(x+1)^3} == 0 + 0 \times 4.1889 + \frac{0}{(4.1889+1)^2} + \frac{0}{(4.1889+1)^3} = 0$$

$$N_{qh} = a + bx + cx^2 + dx^3 + ex^4 = 6.816 + 2.019 \times 4.1889 + (-0.146) \times 4.1889^2 + (7.651E-03) \times 4.1889^3 + (-1.683E-04) \times 4.1889^4 = 13.222$$

$$P_u = N_{ch}cD + N_{qh}\breve{y}HD = 0 \times 0 \times 0.3253 + 13.222 \times 18000 \times 1.36265 \times 0.3253 = 105497\ N/m$$

$$\Delta_p = 0.1\left(H + \frac{D}{2}\right) = 0.1\left(1.36265 + \frac{0.3253}{2}\right) = 0.1525\ m$$

## 22.2.3 VERTICAL UPLIFT

**Input**

| | Symbol | Value | Unit |
|---|---|---|---|
| Outer diameter of pipe | D | 0.3253 | m |
| Soil cover above center of pipeline | H | 1.36265 | m |
| Coefficient of cohesion of backfill soil | C | 0 | $N/m^2$ |
| Effective unit weight of soil | Ÿ | 18000 | $N/m^3$ |
| Internal friction angle of soil | Φ | 35 | Degree |

**Outputs**

| | Symbol | Value | Unit |
|---|---|---|---|
| Vertical uplift factor | $N_{cv}$ | 8.3778 | |
| Vertical uplift factor (should be ≤ $N_q$) | $N_{qv}$ | 3.332 | |
| Maximum soil resistance per unit length of pipeline in vertical uplift | $Q_u$ | 26586 | N/m |
| Mobilizing displacement of soil in vertical uplift | $\Delta_{qu}$ | 0.0272 | m |
| Vertical uplift soil spring | | 1951067 | N/m/m |

| | | | |
|---|---|---|---|
| stiffness | | | |

$$N_{cv} = 2\left(\frac{H}{D}\right) = 2\left(\frac{1.36265}{0.3253}\right) = 8.3778$$

$$N_{qv} = \left(\frac{\emptyset H}{44D}\right) = \left(\frac{35 \times 1.36265}{44 \times 0.3253}\right) = 3.332$$

$$Q_u = N_{cv}cD + N_{qv}\check{y}HD = 8.3778 \times 0 \times 0.3253 + 3.332 \times 18000 \times 1.36265 \times 0.3253$$
$$= 26586\ N/m$$

## 22.2.4 VERTICAL BEARING

**INPUT**

| | Symbol | Value | Unit |
|---|---|---|---|
| Outer diameter of pipe | D | 0.3253 | m |
| Soil cover above center of pipeline | H | 1.36265 | m |
| Coefficient of cohesion of backfill soil | c | 0 | $N/m^2$ |
| Effective unit weight of soil | Ў | 18000 | $N/m^3$ |
| Total unit weight of soil | Y | 18000 | |
| Internal friction angle of soil | Φ | 35 | Degree |

**Output**

| | Symbol | Value | Unit |
|---|---|---|---|
| Bearing capacity factor | $N_c$ | 46.13 | |
| Bearing capacity factor | $N_q$ | 33.3 | |
| Bearing capacity factor | $N_Y$ | 44.7 | |
| Maximum soil resistance per unit length of pipeline vertical bearing | $Q_d$ | 308237 | N/m |
| Mobilizing displacement of soil in vertical bearing | $\Delta_{qd}$ | 0.033 | m |
| Vertical bearing soil spring stiffness | | 18950964 | N/m/m |

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

$$= [\cot(35 + 0.001)]\left\{exp[\pi\ \tan(35 + 0.001)]\left(\tan\left(45 + \frac{35 + 0.001}{2}\right)\right)^2 - 1\right\}$$

$$= 46.13$$

$$N_r = \exp(\pi \tan \phi)\left(\tan\left(45 + \frac{\pi}{2}\right)\right)^2 = \exp(\pi \tan 35)\left(\tan\left(45 + \frac{\pi}{2}\right)\right)^2 = 33.3$$

$$N_r = exp(0.18\phi - 2.5) = exp(0.18 \times 35 - 2.5) = 44.7$$

$$Q_d = N_c cD + N_q \breve{y} HD + N_r y \frac{D^2}{2}$$

$$= 46.13 \times 0 \times 0.3253 + 33.3 \times 18000 \times 1.36265 \times 0.3253 + 44.7 \times 18000 \times \frac{0.3253^2}{2} = 308237\ N/m$$

## 22.3.0 SEISMIC SAFETY CALCULATION

### 22.3.1 SEISMIC WAVE PROPOGATION (CASE-III)

**Input**

| | Symbol | Value | Unit |
|---|---|---|---|
| Outside diameter of pipe | D | 0.3238 | m |
| Internal diameter of pipe | d | 0.3156 | m |
| Thickness of pipe | t | 0.0041 | m |
| Cross sectional area of pipe | A | 0.0041179 | $m^2$ |
| Operational Strain in Pipeline ($\varepsilon_{operational}$) | $\varepsilon_{oper}$ | -0.0005 | |
| Expected peak ground acceleration at base of rock layer | $PGA_r$ | 0.25 | g |
| Ground amplification factor for various soil | $I_g$ | 1.15 | g |
| Important factor for fault movement | $I_p$ | 1.5 | |
| Ratio of PGV (m/s) to PGA ($m/s^2$) | | 94 | |
| Maximum axial soil force per unit length of pipe | $t_u$ | 6863 | N/m |
| Modulus of pipe material before yielding | $E_i$ | 2.00E+11 | $N/m^2$ |
| Yield strain of pipe material | $\varepsilon_y$ | 0.002 | |
| Apparent wavelength of seismic wave | $\lambda$ | 1000 | m |
| Ground strain coefficient | $\alpha_y$ | 2 | |
| Velocity of seismic wave | C | 600 | m/s |

| | | | |
|---|---|---|---|
| propagation | | | |

**Output**

| | Symbol | Value | Unit |
|---|---|---|---|
| Peak ground acceleration at ground | PGA | 0.2875 | g |
| Peak ground velocity | PGV | 27.02 | cm/s |
| Design peak ground velocity | $V_g$ | 0.4054 | m/s |
| Maximum axial strain in pipe due to wave propagation | $\varepsilon_a$ | 0.0003 | |
| Maximum axial strain that can be transmitted by soil friction | $\varepsilon_{soil\ friction}$ | 0.0021 | |
| Is $\varepsilon_a \leq \varepsilon_{soil\ friction}$? ($\varepsilon_a$ should be less than $\varepsilon_{soil\ friction}$) | | YES | |
| Total tensile strain in pipe (Operating + Seismic Wave) | $\varepsilon_{t(Total)}$ | -0.0002 | |
| Allowable strain in pipe | $\varepsilon_{t(allowable)}$ | 0.03 | |
| Is tensile strain in pipe within limit? | | YES | |
| Total compressive strain in pipe (Seismic Wave – Operating) | $\varepsilon_{c(Total)}$ | 0.0008 | |
| Allowable strain in pipe | $\varepsilon_{c(allowable)}$ | 0.004 | |
| Is compressive strain in pipe within limit? | | YES | |

$$PGV = \frac{PGV}{PGA} \times PGA = 94 \times 0.2875 = 27.02\ cm/s$$

$$\mathrm{V}_g = PGV \times I_p = 27.02 \times 1.5 = 0.4054\, m/s$$

$$\varepsilon_a = \frac{V_g}{\alpha_\varepsilon C} == \frac{0.4054}{2 \times 600} = 0.0003$$

$$\varepsilon_{soil\ friction} = \frac{t_u \lambda}{4AE} = \frac{6863 \times 1000}{4 \times 0.0041179 \times (2E + 11)} = 0.0021$$

$$\varepsilon_{t(Total)} = \varepsilon_{oper} + \varepsilon_a = -0.00049 + 0.003 = -0.002$$

$$\varepsilon_{c(Total)} = \varepsilon_{oper} - \varepsilon_a = -0.00049 - 0.003 = 0.0008$$

$$\varepsilon_{c(allowable)} = 0.175 \times \frac{t}{\frac{D}{2}} = 0.175 \times \frac{0.0041}{\frac{0.3238}{2}} = 0.0044$$

## 22.4.0 SUMMARY OF 12 INCH PRODUCTION FLUID PIPELINE

| | Strain In Pipe In Tension | Strain In Pipe In Compression | Allowable Strain In Pipe In Tension | Allowable Strain In Pipe In Compression | Safe/Unsafe |
|---|---|---|---|---|---|
| Seismic Wave | -0.0002 | 0.0008 | 0.0300 | 0.0044 | Safe |

## CALCULATION FOR 18" NB WATER INJECTION PIPELINE

### 23.1.0 OPERATING STRAIN CALCULATIONS

**Input**

| | **Symbol** | **Value** | **Unit** |
|---|---|---|---|
| Internal pressure in pipe | P | 9.30E+06 | $N/m^2$ |
| Outer diameter of pipe | D | 0.4572 | m |
| Pipe wall thickness | T | 0.0113 | m |
| Modulus of elasticity of pipe material | E | 2E+11 | $N/m^2$ |
| Poisson's ratio | $\mu$ | 0.3 | |
| Specified Minimum Yield Strength of pipe material | $\sigma_y$ | 4.50E+08 | $N/m^2$ |
| Rameberg-Osgood parameter | n | 38.32 | |
| Rameberg-Osgood parameter | r | 31.5 | |
| Installation Temperature | $T_1$ | 25 | °C |
| Operating Temperature | $T_2$ | 90 | °C |
| Coefficient of thermal expansion | α | 1.17E-05 | m/m/°C |

**Output**

| | Symbol | Value | Unit |
|---|---|---|---|
| Longitudinal stress in pipe due to internal pressure | $S_p$ | 5.64E+07 | $N/m^2$ |
| Longitudinal strain in pipe due to internal pressure | $\varepsilon_p$ | 0.00028 | |

| | | | |
|---|---|---|---|
| Longitudinal stress in pipe due to temperature change | $S_T$ | -1.52E+08 | $N/m^2$ |
| Longitudinal strain in pipe due to temperature change | $\varepsilon_T$ | 0.00076 | |
| Total longitudinal strain in pipe due to operating loads | $\varepsilon_{oper}$ | -0.00048 | |

$$S_P = \frac{PDu}{2t} = \frac{(5.64E+07) \times 0.4572 \times 0.3}{2 \times 0.0113} = 5.46E+07\ N/m^2$$

$$\varepsilon_p = \frac{S_p}{E}\left(1 + \frac{n}{1+r}\left(\frac{S_p}{\sigma_y}\right)^r\right) = \frac{5.64E+07}{2E+11}\left(1 + \frac{38.32}{1+31.5}\left(\frac{5.64E+07}{4.5E+08}\right)^{31.5}\right) = 0.00028$$

$$S_r = E\alpha\ (T_2 - T_1) = (2E+11) \times (1.17E-05) \times\ (90-25) = -1.52E+08\ Nlm^2$$

$$\varepsilon_p = \frac{S_r}{E}\left(1 + \frac{n}{1+r}\left(\frac{S_r}{\sigma_y}\right)^r\right) = \frac{-1.52E+08}{2E+11}\left(1 + \frac{38.32}{1+31.5}\left(\frac{-1.52E+08}{4.5E+08}\right)^{31.5}\right) = 0.00076$$

$$\varepsilon_{oper} = \varepsilon_p - \varepsilon_T = 0.00028 - 0.00076 = -0.00048$$

## 23.2.0 SOIL SPRING PROPERTIES TO REPRESENT PIPE-SOIL INTERACTION

### 23.2.1 AXIAL SOIL SPRING

**Input**

| | Symbol | Value | Unit |
|---|---|---|---|
| Outer diameter of pipe | D | 0.4587 | m |
| Soil cover above center of pipeline | H | 1.42935 | m |
| Coefficient of cohesion of backfill soil | c | 0 | $N/m^2$ |
| Effective unit weight of soil | Ў | 18000 | $N/m^3$ |
| Friction Factor | F | 0.6 | |
| Internal friction angle of soil | Φ | 35 | Degree |

**Output**

| | Symbol | Value | Unit |
|---|---|---|---|
| Interface angle of friction b/w soil & pipe | S | 21 | |
| Coefficient of soil pressure | $K_0$ | 0.42642 | |
| Adhesion factor | α | 1.029 | |
| Axial soil resistance | $t_u$ | 10150 | N/m |
| Mobilizing displacement | $\Delta_t$ | 0.005 | m |
| Axial Spring stiffness | $2t_u/\Delta_t$ | 4060183.2 | N/m/m |

$S = f\phi = 0.6 \times 35 = 21$

$K_0 = 1 - sin\phi = 1 - \sin 35 = 0.42642$



PL Engineering

$$\alpha = 0.608 - 0.123c - \frac{0.274}{c^2+1} + \frac{0.695}{c^3+1} = 0.608 - 0.123 \times 0 - \frac{0.274}{0^2+1} + \frac{0.695}{0^3+1} = 1.029$$

$$t_u = \pi D c \alpha + \pi D H \breve{y} \left(\frac{1+K_0}{2}\right) tanS$$

$$= 3.14 \times 0.4587 \times 0 \times 1.029 + 3.14 \times 0.4587 \times 1.42935 \times 18000$$

$$\times \left(\frac{1+0.42642}{2}\right) \tan 21 = 10150\ N/m$$

### 23.2.2 LATERAL SOIL SPRING

**Inputs**

| | Symbol | Value | Unit |
|---|---|---|---|
| Outer diameter of pipe | D | 0.4587 | m |
| Soil cover above center of pipeline | H | 1.42935 | m |
| Coefficient of cohesion of backfill | C | 0 | $N/m^2$ |
| Effective unit weight of soil | Ÿ | 18000 | $N/m^3$ |
| Parameters for Horizontal bearing | | | |
| Factor for $N_{ch}$ | a | 0 | |
| Factor for $N_{ch}$ | b | 0 | |
| Factor for $N_{ch}$ | c | 0 | |
| Factor for $N_{ch}$ | d | 0 | |
| Parameters for Horizontal bearing | | | |
| Factor for $N_{qh}$ according to Φ | a | 6.816 | |
| Factor for $N_{qh}$ according to Φ | b | 2.019 | |
| Factor for $N_{qh}$ according to Φ | c | -0.146 | |
| Factor for $N_{qh}$ according to Φ | d | 7.651E-03 | |
| Factor for $N_{qh}$ according to Φ | e | -1.683E-05 | |

**Outputs**

| | Symbol | Value | Unit |
|---|---|---|---|
| Factor for $N_{ch}$ & $N_{qh}$ | x | 3.1161 | |
| Horizontal bearing capacity factor $N_{ch}$ (should be ≤ 9 ) | $N_{ch}$ | 0 | |
| Horizontal bearing capacity factor $N_{qh}$ | $N_{qh}$ | 11.905 | |
| Maximum displacement of soil per unit length of pipeline | $P_u$ | 140502 | N/m |
| Mobilizing displacement of soil in lateral direction (should be ≤ 0.01 D to 0.02D) | $\Delta_p$ | 0.1658 | m |
| Lateral soil Spring stiffness | | 1694120.2 | N/m/m |

$$x = \frac{H}{D} = \frac{1.42935}{0.4587} = 3.1161$$

$$N_{ch} = a + bx + \frac{c}{(x+1)^2} + \frac{d}{(x+1)^3} == 0 + 0 \times 3.1161 + \frac{0}{(3.1161+1)^2} + \frac{0}{(3.1161+1)^3} = 0$$

$$N_{qh} = a + bx + cx^2 + dx^3 + ex^4 = 6.816 + 2.019 \times 3.1161 + (-0.146) \times 3.1161^2 + (7.651E-03) \times 3.1161^3 + (-1.683E-04) \times 3.1161^4 = 11.905$$

$$P_u = N_{ch}cD + N_{qh}\breve{y}HD = 0 \times 0 \times 0.4587 + 11.905 \times 18000 \times 1.42935 \times 0.4587 = 140502\ N/m$$

$$\Delta_p = 0.1\left(H + \frac{D}{2}\right) = 0.1\left(1.42935 + \frac{0.4587}{2}\right) = 0.1658\ m$$

### 23.2.3 VERTICAL UPLIFT

**Input**

| | Symbol | Value | Unit |
|---|---|---|---|
| Outer diameter of pipe | D | 0.4587 | M |
| Soil cover above center of pipeline | H | 1.42935 | M |
| Coefficient of cohesion of backfill soil | c | 0 | $N/m^2$ |
| Effective unit weight of soil | Ў | 18000 | $N/m^3$ |
| Internal friction angle of soil | Φ | 35 | Degree |

**Outputs**

| | Symbol | Value | Unit |
|---|---|---|---|
| Vertical uplift factor | $N_{cv}$ | 6.2321 | |
| Vertical uplift factor (should be ≤ $N_q$) | $N_{qv}$ | 2.4787 | |
| Maximum soil resistance per unit length of pipeline in vertical uplift | $Q_u$ | 29253 | N/m |
| Mobilizing displacement of soil in vertical uplift | $\Delta_{qu}$ | 0.0285 | M |
| Vertical uplift soil spring | | 2046569 | N/m/m |

| | | | |
|---|---|---|---|
| stiffness | | | |

$$N_{cv} = 2\left(\frac{H}{D}\right) = 2\left(\frac{1.42935}{0.4587}\right) = 6.231$$

$$N_{qv} = \left(\frac{\emptyset H}{44D}\right) = \left(\frac{35 \times 1.42935}{44 \times 0.4587}\right) = 2.4787$$

$$Q_u = N_{cv}cD + N_{qv}\check{y}HD = 6.2321 \times 0 \times 0.4587 + 2.4787 \times 18000 \times 1.42935 \times 0.4587$$

$$= 29253\ N/m$$

## 23.2.4 VERTICAL BEARING

**INPUT**

| | Symbol | Value | Unit |
|---|---|---|---|
| Outer diameter of pipe | D | 0.4587 | m |
| Soil cover above center of pipeline | H | 1.42935 | m |
| Coefficient of cohesion of backfill soil | C | 0 | $N/m^2$ |
| Effective unit weight of soil | Ў | 18000 | $N/m^3$ |
| Total unit weight of soil | Y | 18000 | |
| Internal friction angle of soil | Φ | 35 | Degree |

**Output**

| | Symbol | Value | Unit |
|---|---|---|---|
| Bearing capacity factor | $N_c$ | 46.13 | |
| Bearing capacity factor | $N_q$ | 33.3 | |
| Bearing capacity factor | $N_Y$ | 44.7 | |
| Maximum soil resistance per unit length of pipeline vertical bearing | $Q_d$ | 477595 | N/m |
| Mobilizing displacement of soil in vertical bearing | $\Delta_{qd}$ | 0.046 | m |
| Vertical bearing soil spring stiffness | | 20823835 | N/m/m |

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

$$= [\cot(35 + 0.001)]\left\{exp[\pi\ \tan(35 + 0.001)]\left(\tan\left(45 + \frac{35 + 0.001}{2}\right)\right)^2 - 1\right\}$$

$$= 46.13$$

$$N_r = \exp(\pi \tan \phi)\left(\tan\left(45 + \frac{\pi}{2}\right)\right)^2 = \exp(\pi \tan 35)\left(\tan\left(45 + \frac{\pi}{2}\right)\right)^2 = 33.3$$

$$N_r = exp(0.18\phi - 2.5) = exp(0.18 \times 35 - 2.5) = 44.7$$

$$Q_d = N_c cD + N_q \breve{y} HD + N_r y \frac{D^2}{2}$$

$$= 46.13 \times 0 \times 0.4587 + 33.3 \times 18000 \times 1.42935 \times 0.4587 + 44.7 \times 18000 \times \frac{0.4587^2}{2} = 477595\ N/m$$

## 23.3.0 SEISMIC SAFETY CALCULATION

## 23.3.1 SEISMIC WAVE PROPOGATION (CASE-III)

**Input**

| | Symbol | Value | Unit |
|---|---|---|---|
| Outside diameter of pipe | D | 0.4572 | m |
| Internal diameter of pipe | d | 0.4346 | m |
| Thickness of pipe | t | 0.0113 | m |
| Cross sectional area of pipe | A | 0.015829449 | $m^2$ |
| Operational Strain in Pipeline ($\varepsilon_{operational}$) | $\varepsilon_{oper}$ | -0.0005 | |
| Expected peak ground acceleration at base of rock layer | $PGA_r$ | 0.25 | g |
| Ground amplification factor for various soil | $I_g$ | 1.15 | g |
| Important factor for fault movement | $I_p$ | 1.5 | |
| Ratio of PGV (m/s) to PGA (m/s$^2$) | | 94 | |
| Maximum axial soil force per unit length of pipe | $t_u$ | 10150 | N/m |
| Modulus of pipe material before yielding | $E_i$ | 2.00E+11 | N/$m^2$ |
| Yield strain of pipe material | $\varepsilon_y$ | 0.002 | |
| Apparent wavelength of seismic wave | λ | 1000 | m |
| Ground strain coefficient | $\alpha_y$ | 2 | |
| Velocity of seismic wave | C | 600 | m/s |

| | | | |
|---|---|---|---|
| propagation | | | |

**Output**

| | Symbol | Value | Unit |
|---|---|---|---|
| Peak ground acceleration at ground | PGA | 0.2875 | g |
| Peak ground velocity | PGV | 27.02 | cm/s |
| Design peak ground velocity | $V_g$ | 0.4053 | m/s |
| Maximum axial strain in pipe due to wave propagation | $\varepsilon_a$ | 0.0003 | |
| Maximum axial strain that can be transmitted by soil friction | $\varepsilon_{soil\ friction}$ | 0.0008 | |
| Is $\varepsilon_a \leq \varepsilon_{soil\ friction}$? ($\varepsilon_a$ should be less than $\varepsilon_{soil\ friction}$) | | YES | |
| Total tensile strain in pipe (Operating + Seismic Wave) | $\varepsilon_{t(Total)}$ | -0.0001 | |
| Allowable strain in pipe | $\varepsilon_{t(allowable)}$ | 0.03 | |
| Is tensile strain in pipe within limit? | | YES | |
| Total compressive strain in pipe (Seismic Wave – Operating) | $\varepsilon_{c(Total)}$ | 0.0008 | |
| Allowable strain in pipe | $\varepsilon_{c(allowable)}$ | 0.0087 | |
| Is compressive strain in pipe within limit? | | YES | |

$$PGV = \frac{PGV}{PGA} \times PGA = 94 \times 0.2875 = 27.025\ cm/s$$

$$\mathrm{V}_g = PGV \times I_p = 27.02 \times 1.5 = 0.4053\ m/s$$

$$\varepsilon_a = \frac{V_g}{\alpha_\varepsilon C} == \frac{0.4054}{2 \times 600} = 0.0003$$

$$\varepsilon_{soil\ friction} = \frac{t_u \lambda}{4AE} = \frac{6863 \times 1000}{4 \times 00.015829449 \times (2E + 11)} = 0.0003$$

$$\varepsilon_{t(Total)} = \varepsilon_{oper} + \varepsilon_a = -0.00048 + 0.0003 = -0.0001$$

$$\varepsilon_{c(Total)} = \varepsilon_{oper} - \varepsilon_a = -0.00048 - 0.0003 = 0.0008$$

$$\varepsilon_{c(allowable)} = 0.175 \times \frac{t}{\frac{D}{2}} = 0.175 \times \frac{0.0113}{\frac{0.4572}{2}} = 0.0087$$

## 23.4.0 SUMMARY OF 18 INCH WATER INJECTION PIPELINE

| | Strain In Pipe In Tension | Strain In Pipe In Compression | Allowable Strain In Pipe In Tension | Allowable Strain In Pipe In Compression | Safe/Unsafe |
|---|---|---|---|---|---|
| Seismic Wave | -0.0001 | 0.0008 | 0.0300 | 0.0087 | Safe |

# 24.0 SEISMIC ANALYSIS CONCLUSION

Result of Calculations regarding Seismic Wave Propagation on Production Fluid and Water Injection pipeline.

**Production Fluid**

| Sl. No | Size (Inch) | Soil Type | Strain in Pipe | | | | Result |
|---|---|---|---|---|---|---|---|
| | | | Max. Tensile Strain | Max. Compressive Strain | Allowable Tension Strain | Allowable Compressive Strain | |
| 1 | 16 | Case I | -0.0002134 | 0.000889047 | 0.03 | 0.005770177 | SAFE |
| 2 | 16 | Case II | -0.0002134 | 0.000889047 | 0.03 | 0.005770177 | SAFE |
| 3 | 12 | Case I | -0.0001502 | 0.000825847 | 0.03 | 0.004431748 | SAFE |
| 4 | 12 | Case II | -0.0001502 | 0.000825847 | 0.03 | 0.004431748 | SAFE |
| 5 | 10 | Case I | -0.000193 | 0.000868593 | 0.03 | 0.00525641 | SAFE |
| 6 | 10 | Case II | -0.000193 | 0.000868593 | 0.03 | 0.00525641 | SAFE |
| 7 | 8 | Case I | -0.0002004 | 0.00087599 | 0.03 | 0.00543131 | SAFE |
| 8 | 8 | Case II | -0.0002004 | 0.00087599 | 0.03 | 0.00543131 | SAFE |

**Water Injection Fluid**

| Sl. No | Size (Inch) | Soil Type | Strain in Pipe | | | | Result |
|---|---|---|---|---|---|---|---|
| | | | Max. Tensile Strain | Max. Compressive Strain | Allowable Tension Strain | Allowable Compressive Strain | |

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| 1 | 18 | Case I | -0.000140 | 0.0008161 | 0.03 | 0.008650481 | SAFE |
| 2 | 18 | Case II | -0.000140 | 0.0008161 | 0.03 | 0.008650481 | SAFE |
| 3 | 10 | Case I | -0.000187 | 0.0008632 | 0.03 | 0.010384615 | SAFE |
| 4 | 10 | Case II | -0.000187 | 0.0008632 | 0.03 | 0.010384615 | SAFE |
| 5 | 8 | Case I | -0.000187 | 0.0008632 | 0.03 | 0.010383387 | SAFE |
| 6 | 8 | Case II | -0.000187 | 0.0008632 | 0.03 | 0.010383387 | SAFE |
| 7 | 6 (thk. 7.1) | Case I | -0.000137 | 0.0008126 | 0.03 | 0.008547237 | SAFE |
| 8 | 6 (thk. 7.1) | Case II | -0.000137 | 0.0008126 | 0.03 | 0.008547237 | SAFE |
| 9 | 6 (thk. 6.4) | Case I | -7.74E-05 | 0.0007530 | 0.03 | 0.007070707 | SAFE |
| 10 | 6 (thk. 6.4) | Case II | -7.74E-05 | 0.0007530 | 0.03 | 0.007070707 | SAFE |

## 25.0 CONCLUSION OF THE ANALYSIS

The pipeline has been analyzed for seismic wave propagation hazard and found to be safe

# 26. References


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