RAM PIPE REQUAL
Pipeline Requalification Guidelines Project

Report 2

Risk Assessment and Management (RAM) Based Guidelines for Requalification of Marine Pipelines

To

Petroleos Mexicanos (PEMEX)

Instituto Mexicano de Petroleo (IMP)

Minerals Management Service (MMS)

By
Professor Robert Bea

Graduate Student Researcher Rune Iversen

Marine Technology & Management Group
University of California at Berkeley

And
Professor Ahsan Kareem

Graduate Student Researcher Xinzhong Chen

Department of Civil Engineering and Biological Sciences
University of Notre Dame

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By
Professor Robert Bea
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Mr. Rune Iversen, Graduate Student Researcher

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Department of Civil Engineering and Biological Sciences
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List of Symbols

Abbreviations

API	- American Petroleum Institute
ASME	- American Society of Mechanical Engineers
ASTM	- American Society of Testing Material
AGA	- American Gas Association
AISC	- American Institute of Steel Construction, Inc.
BSI	- British Standard Institute
DNV	- Det Norske Veritas
IMP	- Instituto Mexicano de Petroleo
ISO	- International Standard Organization
PEMEX	- Petroleos Mexicano
SUPERB	- Submarine Pipeline Probabilistic Based Design Project
ALS	- Accidental Limit States
ASD	- Allowable Stress Design
CTOD	- Critical Tip Opening Displacement
FEA	- Finite Element Analysis
LRFD	- Load Resistance Factor Design
MOP	- Maximum operating pressure
OTC	- Offshore Technology Conference
OP	- Operating Pressure
FS	- Factor of Safety
SMYS	- Specified Minimum Yield Strength of pipe, in psi (N/mm^2)
SMTS	- Specified Minimum Ultimate Tensile Strength of pipe, in psi (N/mm^2)
WSD	- Working stress design
LRFD	- Load Resistance Factor Design
ULS	- Ultimate Limit State
SLS	- Serviceability Limit States
SCF	- Stress Concentration Factor
SNCF	- Strain Concentration Factor
COV	- Coefficient of Variation
X52	- Material grade, yield strength = 52 ksi=358 Mpa
X65	- Material grade, yield strength = 65 ksi=448 Mpa
X52	- Material grade, yield strength = 70 ksi=530 Mpa

Subscripts

0	- mean
d	- design
θ	- circumferential
r	- radial
res	- residual
co	- collapse
u	- ultimate capacity
p	- plastic capacity
g	- global
l	- local
F50	- Median

Superscripts

M	- Moment
P	- Pressure
T	- Tension
C	- Compression

Roman Symbols

General

B_{F50}	- Median bias factor
V	- Coefficient of Variation
γ	- Load Factor
φ	- Resistance Factor
S	- Demand
R	- Capacity
β	- Reliability Index

Design

A	- Cross sectional area of pipe steel, in inches^2 (mm^2)
A_i	- Internal cross sectional area of the pipe, in inches^2 (mm^2)
A_o	- External cross sectional area of the pipe
C_l	-Inelastic local buckling strength in stress units, pond per square inch (N / mm^2)
C_g	- Inelastic global buckling strength in stress units, pond per square inch (N / mm^2)
D	- Outside diameter of pipe (Equation dependent)
D_i	- Inside diameter of pipe, in inches (mm) $= (D - 2t)$
D_{max}	- Maximum diameter at any given cross section, in inches (mm)
D_{min}	- Minimum diameter at any given cross section, in inches (mm)
E	- Elastic modulus, in pounds per square inch (N / mm^2)
$g(\delta)$	- Collapse reduction factor
K	- Effective length factor
L	- Pipe length, in inches (mm)
M	- Applied moment, pond-inch (Nmm)
M_p	- Plastic moment capacity, pond-inch (Nmm)

P	- Applied pressure
P_b	- Minimum burst pressure of pipe, in psi (N / mm^2)
P_c	- Collapse pressure of the pipe, in psi (N / mm^2)
P_e	- Elastic collapse pressure of the pipe, in psi (N / mm^2)
P_i	- Internal pressure in the pipe, in psi (N / mm^2)
P_o	- External hydrostatic pressure, in psi (N / mm^2)
P_p	- Buckle propagation pressure, in psi (N / mm^2)
P_y	- Yield pressure at collapse, in psi (N / mm^2)
r	- Radius of gyration
t_{nom}	- Nominal wall thickness of pipe, in inches (mm)
t_{min}	- Minimum measured wall thickness, in inches (mm)
t	- Pipe wall thickness, in inches (mm)
f_0	- The initial ovalization
n	- The strain hardening parameter
S	- The anisotropy parameter
T_a	- Axial tension in the pipe, in pounds (N)
T_{eff}	- Effective tension in pipe, in pounds (N)
T_y	- Yield tension of the pipe, in pounds (N)
T_u	- Tension Load Capacity
δ	- Ovality
δ_c	- the critical CTOD value
ε_0	- the yield strain
a_{max}	- the equivalent through-thickness crack size.
ε_b	- Bending strain in the pipe
ε_{cr}	- Critical strain
ε_{bm}	- The maximum bending strain
σ_a	- The axial stress
σ_h	- Hoop stress
σ_{he}	- Effective hoop stress
σ_{res}	- Residual stress
σ_θ	- Circumferential stress
σ_{xkL}	- The classic local elastic critical stress
σ_u	- ultimate tensile stress
σ_y	- yield stress
σ_0	- flow stress
λ	- Slenderness parameter

Reassessment

A_d	- effective cross sectional area of damaged (dent) section
A_0	- cross-sectional area of undamaged section
d	- damage depth
ΔY	- Primary out-of-straightness of a dented member
ΔY_0	- 0.001L

I_d	- Effective moment of inertia of undamaged cross-section
K_0	- Effective length factor of undamaged member
K	- Effective buckling length factor
λ_d	- Slenderness parameter of a dented member $=(P_{ud} / P_{ed})^{0.5}$
M_u	- Ultimate moment capacity
M_{cr}	- Critical moment capacity (local buckling)
M_{ud}	- Ultimate negative moment capacity of dent section
M_-	- Negative moment of dent section
M_+	- Positive moment of dent section
M^*	- Neutral moment of dent section
P_{crd}	- Critical axial buckling capacity of a dented member ($\Delta / L > 0.001$)
P_{crd0}	- Critical axial buckling capacity of a dented member ($\Delta / L = 0.001$)
P_E	- Euler load of undamaged member
P_{crl}	- Axial local buckling capacity
P_{cr}	- Axial column buckling capacity
P_u	- Axial compression capacity
P_{ud}	- Axial compression capacity of a short dented member

1.0 Introduction

1.1 Objective

The objective of this joint United States - Mexico cooperative project is to develop and verify Risk Assessment and Management (RAM) based criteria and guidelines for **reassessment and requalification** of marine pipelines and risers. This project was sponsored by the U. S. Minerals Management Service (MMS), Petroleos Mexicanoss (PEMEX), and Instituto Mexicanos de Petroleo (IMP).

1.2 Scope

The RAM PIPE REQUAL project addressed the following key aspects of criteria for requalification of conventional existing marine pipelines and risers:

- Development of Safety and Serviceability Classifications (SSC) for different types of marine pipelines and risers that reflect the different types of products transported, the volumes transported and their importance to maintenance of productivity, and their potential consequences given loss of containment,
- Definition of target reliabilities for different SSC of marine risers and pipelines,
- Guidelines for assessment of pressure containment given corrosion and local damage including guidelines for evaluation of corrosion of non-piggable pipelines,
- Guidelines for assessment of local, propagating, and global buckling of pipelines given corrosion and local damage,
- Guidelines for assessment of hydrodynamic stability in extreme condition hurricanes, and
- Guidelines for assessment of combined stresses during operations that reflect the effects of pressure testing and limitations in operating pressures.

Important additional parts of this project provided by PEMEX and IMP were:

- Review of the criteria and guidelines by an international panel of consulting engineers,
- Conduct of workshops and meetings in Mexico and the United States to review progress and developments from this project and to exchange technologies regarding the design and requalification of marine pipelines,
- Provision of a scholarship to fund the work of graduate student reserarchers that assisted in performing this project, and
- Provision of technical support, background, and field operations data to advance the objectives of the RAM PIPE REQUAL project.

1.3 Background

During the period 1996 - 1998, PEMEX (Petroleos Mexicanos) and IMP (Instituto Mexicanos del Petroleo) sponsored a project performed by the Marine Technology and Development Group of the University of California at Berkeley to help develop first-generation Reliability Assessment and Management (RAM) based guidelines for design of pipelines and risers in the Bay of Campeche. These guidelines were based on both Working Stress Design (WSD) and Load and Resistance Factor Design (LRFD) formats. The following guidelines were developed during this project:

- Serviceability and Safety Classifications (SSC) of pipelines and risers,
- Guidelines for analysis of in-place pipeline loadings (demands) and capacities (resistances), and
- Guidelines for analysis of on-bottom stability (hydrodynamic and geotechnical forces),

This work formed an important starting point for this project.

During the first phase of this project, PEMEX and IMP sponsored two international workshops that addressed the issues and challenges associated with development of criteria and guidelines for design and requalification of marine pipelines.

1.4 Approach

Very significant advances have been achieved in the requalification and reassessment of onshore pipelines. A very general strategy for the requalification of marine pipelines has been proposed by DNV and incorporated into the ISO guidelines for reliability-based limit state design of pipelines (Collberg, Cramer, Bjornoyl, 1996; ISO, 1997). This project is founded on these significant advances.

The fundamental approach used in this project is a Risk Assessment and Management (RAM) approach. This approach is founded on two fundamental strategies:

- Assess the risks (likelihoods, consequences) associated with existing pipelines, and

1) Manage the risks so as to produce acceptable and desirable quality in the pipeline operations.

It is recognized that some risks are knowable (can be forseen) and can be managed to produce acceptable performance. Also, it is recognized that some risks are not knowable (can not be forseen, and that management processes must be put in place to help manage such risks.

Applied to development of criteria for the requalification of pipelines, a RAM approach proceeds through the following steps:

- Based on an assessment of costs and benefits associated with a particular development and generic type of system, and regulatory - legal requirements, national requirements, define the target reliabilities for the system. These target reliabilities should address the four quality attributes of the system including serviceability, safety, durability, and compatibility.
- Characterize the environmental conditions (e.g. hurricane, nominal oceanographic, geologic, sesmic) and the operating conditions (installation, production, maintenance) that can affect the pipeline during its life.
- Based on the unique characteristics of the pipeline system characterize the 'demands' (imposed loads, induced forces, displacements) associated with the environmental and operating conditions. These demands and the associated conditions should address each of the four quality attributes of interest (serviceability, safety, durability, compatibility).
- Evaluate the variabilities, uncertainties, and 'Biases' (differences between nominal and true values) associated with the demands. This evaluation must be consistent with the variabilities and uncertainties that were included in the decision process that determined the desirable and acceptable 'target' reliabilities for the system (Step #1).
- For the pipeline system define how the elements will be designed according to a proposed engineering process (procedures, analyses, strategies used to determine the structure element sizes), how these elements will be configured into a system, how the system will be constructed,

operated, maintained, and decommissioned (including Quality Assurance - QA, and Quality Control - QC processes).

- Evaluate the variabilities, uncertainties, and 'Biases' (differences between nominal and true values) associated with the capacities of the pipeline elements and the pipeline system for the anticipated environmental and operating conditions, construction, operations, and maintenance activities, and specified QA - QC programs). This evaluation must be consistent with the variabilities and uncertainties that were included in the decision process that determined the desirable and acceptable 'target' reliabilities for the system (Step #1).
- Based on the results from Steps #1, #4, and #6, and for a specified 'design format' (e.g. Working Stress Design - WSD, Load and Resistance Factor Design- LRFD, Limit States Design - LSD), determine the design format factors (e.g. factors-of-safety for WSD, load and resistance factors for LRFD, and design conditions return periods for LSD).

It is important to note that several of these steps are highly interactive. For some systems, the loadings induced in the system are strongly dependent on the details of the design of the system. Thus, there is a potential coupling or interaction between Steps #3, #4, and #5. The assessment of variabilities and uncertainties in Steps #3 and #5 must be closely coordinated with the variabilities and uncertainties that are included in Step #1. The QA - QC processes that are to be used throughout the life-cycle of the system influence the characterizations of variabilities, uncertainties, and Biases in the 'capacities' of the system elements and the system itself. This is particularly true for the proposed IMR (Inspection, Maintenance, Repair) programs that are to be implemented during the system's life cycle. Design criteria, QA - QC, and IMR programs are highly interactive and are very inter-related.

The RAM PIPE REQUAL guidelines are based on the following current criteria and guidelines:

1) American Petroleum Institute (API RP 1111, 1996, 1998),

2) Det Norske Veritas (DNV, 1981, 1996, 1998),

3) American Gas Association (AGA, 1990, 1993),

4) American Society of Mechanical Engineers (ASME B31),

5) British Standards Institute (BSI 8010, PD 6493), and

6) International Standards Institute (ISO, 1998).

1.5 Guideline Development Premises

The design criteria and guideline formulations developed during this project are conditional on the following key premises:

- The design and reassessment – requalification analytical models used in this project were based in so far as possible on analytical procedures that are founded on fundamental physics, materials, and mechanics theories.
- The design and reassessment – requalification analytical models used in this –project were founded on in so far as possible on analytical procedures that result in unBiased (the analytical result equals the median – expected true value) assessments of the pipeline demands and capacities.

- Physical test data and verified – calibrated analytical model data were used in so far as possible to characterize the uncertainties and variabilities associated with the pipeline demands and capacities.
- The uncertainties and variabilities associated with the pipeline demands and capacities will be concordant with the uncertainties and variabilities associated with the background used to define the pipeline reliability goals.

1.6 Pipeline Operating Premises

- The pipelines will be operated at a minimum pressure equal to the normal hydrostatic pressure exerted on the pipeline.
- The pipelines will be maintained to minimize corrosion damage through coatings, cathodic protection, use of inhibitors, and dehydration so as to produce moderate corrosion during the life of the pipeline. If more than moderate corrosion is developed, then the reassessment capacity factors are modified to reflect the greater uncertainties and variabilities associated with severe corrosion.
- The pipelines will be operated at a maximum pressure not to exceed the maximum design pressure. If pipelines are reassessed and requalified to a lower pressure than the maximum design pressure, they will be operated at the specified lower maximum operating pressure. Maximum incidental pressures will not exceed 10 % of the specified maximum operating pressures.

1.7 Schedule

This project will take two years to complete. The project was initiated in August 1998. The first phase of this project was completed on 1 July, 1999. RAMP PIPE REQUAL Report1 and Report 2 document results from the first year study. The second phase of this project will be initiated in August 1999 and completed during July 2000.

The schedule for each of the project tasks is summarized in Table 1.1.

Table 1.1 - Project Task Schedule

Task	Part 1, Year 1	Part 2, Year 1	Part 3, Year 2	Part 4, Year 2
1 Classifications	-------------X			
2 Buckling	---------------X			
3 Pressure	---------------X			
4 Op. Pressures	---------------X			
5 Pipe Charar.		---------------X		
6 Stability		---------------X		
7 Buckling Gl.		---------------X		
8 Press. Gl.		---------------X		
9 Stab. Gl.			---------------X	
10 Requal. Gl.			---------------X	---------------X
11 Workshps.	X X X	X	X	X
12 GSR	---------------X	---------------X	---------------X	---------------X
13 Review	X-------------X	---------------X	---------------X	---------------X

1.8 Project Reports

A report will document the developments from each of the four parts or phases of this project. The reports that will be issued at the end of each of the project phases are as follows:

- **Report 1 –** Requalification Process and Objectives, Risk Assessment & Management Background, Pipeline and Riser Classifications and Targets, Templates for Requalification Guidelines, Pipeline Operating Pressures and Capacities (corrosion, denting, gouging – cracking).
- **Report 2 –** Pipeline characteristics, Hydrodynamic Stability, Geotechnical Stability, Guidelines for Assessing Capacities of Defective and Damaged Pipelines.
- **Report 3 –** Guidelines for Assessing Pipeline Stability (Hydrodynamic, Geotechnical), Preliminary Requalification Guidelines.
- **Report 4 –** Guidelines for Requalifying and Reassessing Marine Pipelines.

2.0 RAM PIPE REQUAL

2.1 Attributes

Practicality is one of the most important attributes of an engineering approach. Industry experience indicates that a practical RAM PIPE REQUAL approach should embody the following attributes:

- **Simplicity** – ease of use and implementation,
- **Versatility** – the ability to handle a wide variety of real problems,
- **Compatibility** – readily integrated into common engineering and operations procedures,
- **Workability** – the information and data required for input is available or economically attainable, and the output is understandable and can be easily communicated,
- **Feasibility** – available engineering, inspection, instrumentation, and maintenance tools and techniques are sufficient for application of the approach, and
- **Consistency** – the approach can produce similar results for similar problems when used by different engineers.

2.2 Strategies

The RAM PIPE REQUAL approach is founded on the following key strategies:

- **Keep pipeline systems in service** by using preventative and remedial IMR (Inspection, Maintenance, Repair) techniques. RAM PIPE attempts to establish and maintain the integrity of a pipeline system at the least possible cost.
- RAM PIPE REQUAL procedures are intended to **lower risks to the minimum that is practically attainable**. Comprehensive solutions may not be possible. Funding and technology limitations may prevent implementation of ideally comprehensive solutions. Practicality implicates an **incremental investment in identifying and remedying pipeline system defects in the order of the hazards they represent**. This is a prioritized approach.
- RAM PIPE REQUAL should be one of **progressive and continued reduction of risks to tolerable levels.** The **investment of resources must be justified by the scope of the benefits achieved.** This is a repetitive, continuing process of improving understanding and practices. This is a process based on economics and benefits.

2.3 Approach

The fundamental steps of the RAM PIPE REQUAL approach are identified in Figure 2.1. The steps can be summarized as follows:

- **Identification** – this selection is based on an assessment of the likelihood of finding significant degradation in the quality (serviceability, safety, durability, compatibility) characteristics of a given pipeline system, and on an evaluation of the consequences that could be associated with the degradation in quality. The selection can be triggered by either a regulatory requirement or by an owner's initiative, following an unusual event, an accident, proposed upgrading of the operations, or a desire to significantly extend the life of the pipeline system beyond that originally intended. ISO (1997) has identified the following triggers for requalification of pipelines: extension of design life, observed damage, changes in operational and environmental

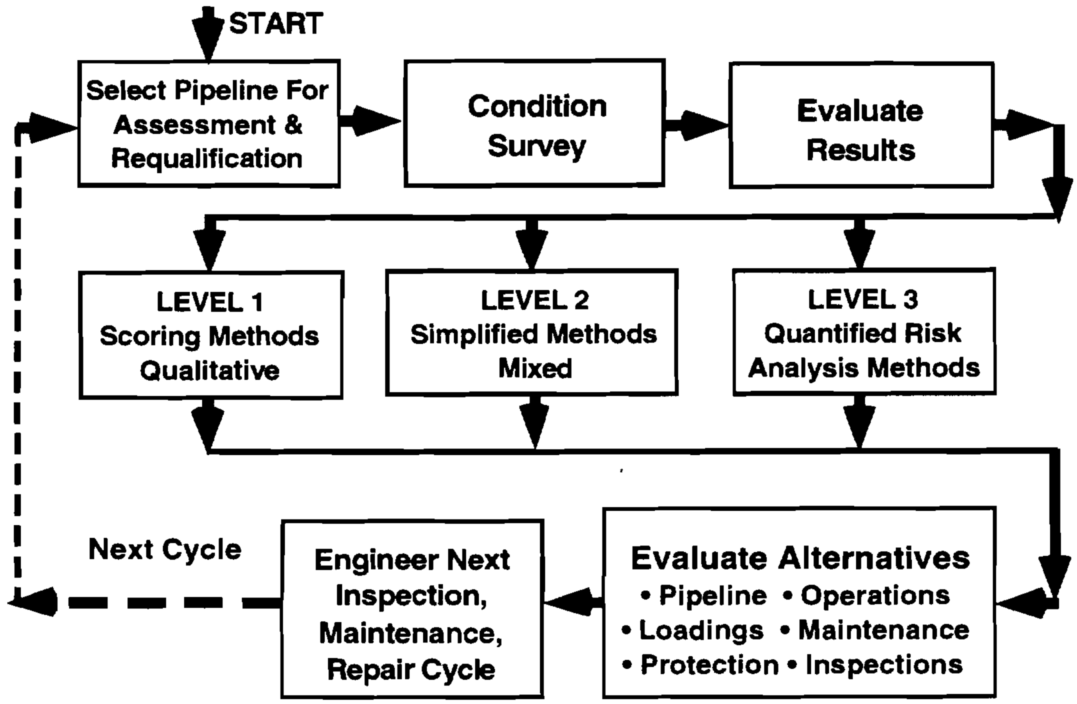

Figure 2.1 – RAM PIPE Approach

conditions, discovery of errors made during design or installation, concerns for the safety of the pipeline for any reason including increased consequences of a possible failure.

- **Condition survey** – this survey includes the formation of or continuance of a databank that contains all pertinent information the design, construction, operation, and maintenance of a pipeline system. Of particular importance are identification and recording of exceptional events or developments during the pipeline system history. Causes of damage or defects can provide important clues in determining what, where, how ,and when to inspect and/or instrument the pipeline system. This step is of critical importance because the RAM PIPE process can only be as effective as the information that is provided for the subsequent evaluations (garbage in, garbage out). Inspections can include external observations (eye, ROV) and measurements (ultrasonic, eddy current, caliper), and internal measurements utilizing in-line instrumentation (smart pigs: magnetic flux, ultrasonic, eddy current, caliper, inertia – geo).
- **Results assessment** – this effort is one of assessing or screening the pipeline system based on the presence or absence of any significant signs of degradation its quality characteristics. The defects can be those of design, construction, operations, or maintenance. If there appear to be no potentially significant defects, the procedure becomes concerned with engineering the next IMR cycle. If there appear to be potentially significant defects, the next step is to determine if mitigation of these defects is warranted. Three levels of assessment of increasing detail and difficulty can be applied: Level 1 – Qualitative (Scoring, Muhlbauer 1992; Kirkwood, Karam 1994), Level 2 – Simplified Qualitative – Quantitative (Bea, 1998), and Level 3 – Quantitative (Quantitative Risk Assessment, QRA, Nessim, Stephens 1995; Bai, Song 1998; Collberg, et al 1996). ISO guidelines (1997) have noted these levels as those of simple calculations, state of practice methods, and state of art methods, respectively.

The basis for selection of one these levels is one that is intended to allow assessment of the pipeline with the simplest method. The level of assessment is intended to identify pipelines that are clearly fit for purpose as quickly and easily as is possible, and reserve more complex and intense analyses for those pipelines that warrant such evaluations. The engineer is able to choose the method that will facilitate and expedite the requalification process. There are more stringent Fitness for Purpose (FFP) criteria associated with the simpler methods because of the greater uncertainties associated with these methods, and because of the need to minimize the likelihood of 'false positives' (pipelines identified to FFP that are not FFP).

- **Mitigation measures evaluation** – mitigation of defects refers to prioritizing the defects to remedied (first things first), and identifying practical alternative remedial actions. The need for the remedial actions depends on the hazard potential of a given pipeline system, i.e., the likelihood that the pipeline system would not perform adequately during the next RAM PIPE REQUAL cycle. If mitigation appears to be warranted, the next step is to evaluate the alternatives for mitigation.
- **Evaluating alternatives** – mitigation alternatives include those concerning the pipeline itself (patches, replacement of sections), its loadings (cover protection, tie-downs), supports, its operations (pressure de-rating, pressure controls, dehydration) maintenance (cathodic protection, corrosion inhibitors), protective measures (structures, procedures, personnel), and its information (instrumentation, data gathering). Economics based methods (Kulkarni, Conroy 1994; Nessim, Stephens 1995), historic precedents (data on the rates of compromises in pipeline quality), and current standards of practice (pipeline design codes and guidelines, and reassessment outcomes that represent decisions on acceptable pipeline quality) should be used as complimentary methods to evaluate the alternatives and the pipeline FFP. An important alternative is that of improving information and data on the pipeline system (information on the internal characteristics of the pipeline with instrumentation – 'smart pigs' and with sampling, information on the external characteristics of the pipeline using remote sensing methods and on-site inspections).
- **Implementing alternatives** – once the desirable mitigation alternative has been defined, the next step is to engineer that alternative and implement it. The results of this implementation should be incorporated into the pipeline system condition survey – inspection databank. The experiences associated with implementation of a given IMR program provide important feed-back to the RAM PIPE REQUAL process.
- **Engineering the next RAM PIPE REQUAL cycle** – the final step concluding a RAM PIPE REQUAL cycle is that of engineering and implementing the next IMR cycle. The length of the cycle will depend on the anticipated performance of the pipeline system, and the need for and benefits of improving knowledge, information and data on the pipeline condition and performance characteristics.

The ISO guidelines for requalification of pipelines (1997) cite the following essential aspects of an adequate requalifcation procedure – process:

- Account for all the governing factors for the pipeline, with emphasis on the factors initiating the requalification process
- Account for the differences between design of anew pipeline and the reassessment of an existing pipeline
- Apply a decision-theoretic framework and sound engineering judgement
- Utilize an approach in which the requalification process is refined in graduate steps
- Define a simple approach allowing most requalification problems to be solved using conventional methods.

The proposed RAM PIPE REQUAL process, guidelines, and criteria developed during this project are intended to fully satisfy these requirements. A Limit State format will be developed based on Risk Assessment and Management (RAM) background outlined in the next section of this report.

3.0 Pipeline Requalification Formulations & Criteria

The following tables summarize the pipeline requalification guidelines for determination of pipeline strength – capacity characteristics developed during the first phase of this project for in-place operating and accidental conditions. While the tables are not complete at this time, these tables will provide the format that will be used to compile requalification formulations and criteria developed as a result of this project. At this stage, one SSC has been identified for requalification strength criteria. This SSC represents the highest reliability requirements for pipelines and risers for the SSC evaluated during the first phase of this project. The SSC annual Safety Indices are summarized in Table 3.3.

Table 3.1 – Pipeline Capacities

Loading States (1)	Capacity Analysis Eqn. (2)	Data Bases (3)	Capacity Analysis Eqn. Median Bias (4)	Capacity Analysis Eqn. Coef. Var. (5)
Single				
Longitudinal • Tension - **Td**	1	1.1	1.0	0.25
• Compression -**Cd** local - **Cld**	2	1.2	1.0	0.25
• Compression global - **Cgd**	3	1.3	1.0	0.25
Transverse • Bending - **Mud**	4	1.4	1.0	0.25
Pressure • Burst - **Pbd**	5	1.5	1.2	0.25
• Collapse – **Pcd***	6	1.6	1.0	0.25
• Propagating–**Pp***	7	1.7	1.0	0.12
Combined				
T - Mu	8	2.1	1.0	0.25
T – Pc*	9	2.2	1.0	0.25
Mu – Pc*	10	2.3	1.0	0.25
T–Mu-Pc*	11	2.4	1.0	0.25
C–Mu–Pb	12	2.5	1.0	0.25
C–Mu-Pc*	13	2.6	1.0	0.25

*** Accidental Limit State (evaluated with 10-year return period conditions)**

Table 3.2 – Pipeline Loadings & Pressures Biases and Uncertainties

Loading States (1)	In-Place Loading Median Bias B_{F50} (2)	In-Place Loading Annual Coefficient of Variation V_F (3)
Single		
Longitudinal • Tension - **Td**	1.0	0.10
• Compression- **Cd** local - **Cld**	1.0	0.10
• Compression global - **Cgd**	1.0	0.10
Transverse • Bending - **Mud**	1.0	0.10
Pressure • Burst - **Pbd**	1.0	0.10
• Collapse – **Pcd***	0.98	0.02
• Propagating-**Pp***	0.98	0.02
Combined		
T - Mu	1.0	0.10
T – Pc*	0.98	0.02
Mu – Pc*	0.98	0.02
T – **Mu – Pc***	0.98	0.02
C– Mu -Pb	1.0	0.10
C– Mu –Pc*	0.98	0.02

*** Accidental Limit State (evaluated with 10-year return period conditions)**

Table 3.3 – Pipeline Design and Reassessment Ultimate Limit State Annual Safety Indices

Loading States (1)	Annual Safety Index In-Place ULS Pipelines (2)	Annual Safety Index In-Place ULS Risers (3)
Single		
Longitudinal • Tension - **Td**	3.4	3.8
• Compression -**Cd** local - **Cld**	3.4	3.8
• Compression global - **Cgd**	3.4	3.8
Transverse • Bending - **Mud**	3.4	3.8
Pressure • Burst - **Pbd**	3.4	3.8
• Collapse – **Pcd***	1.7	1.7
• Propagating-**Pp***	1.7	1.7
Combined		
T - Mu	3.6	3.8
T – Pc*	2.0	2.0
Mu – Pc*	2.0	2.0
T – Mu – Pc*	2.0	2.0
C – Mu - Pb	3.6	3.6
C – Mu – Pc*	**2.0**	**2.0**

***Accidental Limit State (evaluated with 10-year return period conditions)**

Table 3.4 –In-Place Reassessment Working Stress Factors

WSD Single In-Place Loadings	Demand/ Capacity	Demand & Capacity	In-Place Pipelines	In-Place Risers
	Median Bias	Uncertainty V	ULS - f	ULS - f
Tension	1.00	0.27	0.40	0.36
Compression (local)	1.00	0.27	0.40	0.36
Compression (global)	1.00	0.27	0.40	0.36
Bending	1.00	0.27	0.40	0.36
Burst Pressure (no corrosion)	0.91	0.27	0.44	0.39
Burst Pressure (20 yr corrosion)	0.83	0.27	0.48	0.43
Collapse Pressure (high ovality)*	0.98	0.31	0.60	0.60
Collapse Pressure (low ovality)*	0.98	0.27	0.64	0.64
Propagating Buckling*	0.98	0.12	0.83	0.83
WSD Combined In-Place Loadings				
Tension-Bending-Collapse Pressure*	0.98	0.27	0.64	0.64
Compression-Bending-Collapse Pressure*	0.98	0.27	0.64	0.64
Compression-Bending-Burst Pressure	1.00	0.27	0.40	0.36
*accidental condition with 10-yr demands				

Table 3.5 – In-Place Reassessment Loading Factors

LRFD Single In-Place Loadings	Demand	Demand	In-Place Pipelines	In-Place Risers
	Median Bias	Uncertainty V	LRFD - γ	LRFD - γ
Tension	1.00	0.10	1.29	1.33
Compression (local)	1.00	0.10	1.29	1.33
Compression (global)	1.00	0.10	1.29	1.33
Bending	1.00	0.10	1.29	1.33
Burst Pressure (no corrosion)	1.00	0.10	1.29	1.33
Burst Pressure (20 yr corrosion)	1.00	0.10	1.29	1.33
Collapse Pressure (high ovality)*	0.98	0.02	1.01	1.01
Collapse Pressure (low ovality)*	0.98	0.02	1.01	1.01
Propagating Buckling*	0.98	0.02	1.01	1.01
LRFD Combined In-Place Loadings				
Tension-Bending-Collapse Pressure*	0.98	0.02	1.01	1.01
Compression-Bending-Collapse Pressure*	0.98	0.02	1.01	1.01
Compression-Bending-Burst Pressure	1.00	0.10	1.29	1.33
*accidental condition with 10-yr demands				

Table 3.6 – In-Place Reassessment Resistance Factors

LRFD Single In-Place Loadings	Capacity	Capacity	Pipelines	Risers
	Median Bias	Uncertainty V	LRFD - ϕ	LRFD - ϕ
Tension	1.00	0.25	0.53	0.49
Compression (local)	1.00	0.25	0.53	0.49
Compression (global)	1.00	0.25	0.53	0.49
Bending	1.00	0.25	0.53	0.49
Burst Pressure (no corrosion)	1.10	0.25	0.58	0.54
Burst Pressure (20 yr corrosion)	1.20	0.25	0.63	0.59
Collapse Pressure (high ovality)*	1.00	0.25	0.73	0.73
Collapse Pressure (low ovality)*	1.00	0.25	0.73	0.73
Propagating Buckling*	1.00	0.12	0.86	0.86
LRFD Combined In-Place Loadings				
Tension-Bending-Collapse Pressure*	1.00	0.25	0.73	0.73
Compression-Bending-Collapse Pressure*	1.00	0.25	0.73	0.73
Compression-Bending-Burst Pressure	1.00	0.25	0.53	0.49
*accidental condition with 10 yr demands				

Table 3.7 –Analysis Equations References (See Section 9)

Loading States (1)	Analysis Eqn. (2)	Capacity Analysis Equations References (3)
		Single - Reassessment
Longitudinal • Tension - Td	**1**	**Andersen, T.L., (1990), API RP 1111 (1997), DNV96 (1996), ISO (1996), Crentsil, et al (1990)**
•Compression - Cd local - Cld	**2**	**API RP 2A (1993), Tvergaard, V., (1976), Hobbs, R. E., (1984)**
• Compression 1) global - Cgd	**3**	**API RP 2A (1993), Tvergaard, V., (1976), Hobbs, R. E., (1984)**
Transverse • Bending - Mud	**4**	**BSI 8010 (1993), DNV 96 (1996), API RP 1111 (1997), Stephens, D.R., (1991), Bai, Y. et al (1993), Bai, Y. et al (1997a), Sherman, D.R., (1983), Sherman, D.R., (1985), Kyriakides, S. et al (1991), Gresnigt, A.M., et al (1998)**
Pressure • Burst - Pbd	**5**	**Bea, R. G. (1997), Jiao, et al (1996), Sewart, G., (1994), ANSI/ASME B31G (1991), API RP 1111 (1997), DNV 96 (1996), BSI 8010 (1993)**
• Collapse - Pcd	**6**	**Timoshenko,S.P., (1961), Bai, Y., et al (1997a), Bai, Y., et al (1997b), Bai, Y., et al (1998), Mork, K., (1997), DNV 96 (1996), BSI 8010 (1993), API RP 1111 (1997), ISO (1996), Fowler, J.R., (1990)**
•Propagating–Pp *	**7**	**Estefen, et al (1996), Melosh, R. , et al (1976), Palmer, A.C., et al (1975), Kyridkides, et al (1981), Kyriakides, S. et al (1992), Chater, E., (1984), Kyriakides, S. (1991)**
		Combined
T - Mp	**8**	**Bai, Y., et al (1993), Bai, Y., et al (1994), Bai, Y., (1997), Mork, K et al (1997), DNV 96 (1996), Walker, A. C., (1995), Yeh, M.K., et al (1986), Yeh, M.K., et al (1988), Murphey, C.E., et al (1984)**
T - Pc	**9**	Kyogoku, T., et al (1981), Tamano, et al (1982)
B - Pc	**10**	Ju, G. T., et al (1991), Kyriakides, S., et al (1987), Bai, Y., et al (1993), Bai, Y., et al (1994), Bai, Y., et al (1993), Corona, E., et al (1988), DNV96 (1996), BSI 8010 (1993), API RP 1111 (1997), Estefen, S. F. et al (1995)
T – Mu - Pc	**11**	Li, R., et al (1995), DNV 96 (1996), Bai et al (1993), Bai, Y. et al (1994), Bai, Y. et al (1997), Kyriakides, et al (1989)
C – Mu - Pb	**12**	DNV 96 (1996), Bruschi, R., et al (1995), Mohareb, M. E. et al (1994)
C – Mu - Pc	**13**	Kim, H. O., (1992), Bruschi, R., et al (1995), Popv E. P., et al (1974),

Table 3.8 – Capacity Database References (See Section 9)

Loading States (1)	Database	Capacity Analysis Equations References (3)
Single - Reassessment		
Longitudinal • Tension - **Td**	**1.1**	Taby, J., et al (1981)
• Compression - **Cd** 1. local - **Cld**	**1.2**	Loh, J.T., (1993), Ricles, J. M., et al (1992), Taby, J., et al (1981)
• Compression global - **Cgd**	**1.3**	Loh, J.T., (1993), Ricles, J. M., et al (1992), Smith, C.S., et al (1979)
Transverse • Bending - **Mp d**	**1.4**	Loh, J.T., (1993), Ricles, J. M., et al (1992), Taby, J., et al (1981)
Pressure • Burst - **Pbd**	**1.5**	DNV (93-3637)
• Collapse - **Pcd**	**1.6**	
• Propagating–**Pp***	**1.7**	Kyriakides, S., (1984), Estefen S. F., et al (1995), Mesloh, et al (1976)
Combined		
T - Mp	**2.1**	Dyau, J.Y., (1991), Wilhoit, Jr. J.C., et al (1973)
T - Pc	**2.2**	Edwards, S.H., et al (1939), Kyogoku, T., et al (1981), Tamano, T., et al (1982), Kyriakides, S., et al (1987), Fowler, J. R., (1990)
B - Pc	**2.3**	Kyriakides, S., et al (1987), Fowler, J. R., (1990), Winter, P. E., (1985), Johns, T. G., (1983)
T – Mp - Pc	**2.4**	Walker, G.E., et al (1971), Langner, C.G., (1974)
C – Mp - Pb	**2.5**	Walker, G.E., et al (1971), Langner, C.G., (1974)

Table 3.9 – Formulations for Single Loading States

Loading States (1)	Formulation (2)	Formulation Factors (3)
Longitudinal • Tension - **Td**	$Td = 1.1SMYS(A - \Delta)$	
• Compression- **Cd** local - **Cld**	$Cl = 1.1 \bullet SMYS\left(2.0 - 0.28\left(D/t_{\min}\right)^{1/4}\right) \bullet A \bullet Kd$	Kd = 1 + 3 fd (D / t)
• Compression global - **Cgd**	$Cg = 1.1SMYS\left(1.2 - 0.25\lambda^2\right) \bullet A$ $\lambda = \frac{KL}{\pi r} \frac{SMYS}{E}^{0.5}$	$\frac{P_{crd}}{P_{crdo}} + \frac{P_{crd}\Delta Y}{\left(1 - \frac{P_{crd}}{P_{Ed}}\right)M_{ud}} \leq 1.0$ $\lambda_d = \left(P_{ud} / P_{ed}\right)^{0.5}$ $P_{ud} = P_u \frac{A_d}{A_o} = P_u \exp\left(-0.08\frac{\Delta}{t}\right)$
Transverse • Bending - **Mud**	$\frac{M_d}{M_u} = \exp\left(-0.06\frac{\Delta}{t}\right)$	
Pressure **• Burst – Pbd** **Corroded** **Dented** **Gouged** **Dented & Gouged**	$P_{bC} = \frac{2.2 \bullet t \bullet SMTS}{(D-t) \bullet SCFc}$ $Pb_D = \frac{2t\sigma_u}{(D-t) \bullet SCF_D}$ $Pb_G = \frac{2t\sigma_u}{(D-t) \bullet SCF_G}$ $Pb_{DG} = \frac{2t\sigma_u}{(D-t) \bullet SCF_{DG}}$	$SCFc = 1 + 2 (d/R)^{0.5}$ $SCF_D = 1 + 0.2 (H/t)^3$ $SCF_G = 1 + 2 (h / r)^{0.5}$ $SCF_{DG} = [1-d/t-(16H/D)(1-d/t)]^{-1}$
• Collapse – Pcd **High Ovality Pipe* (f_{50} = 1 %)** **Low Ovality Pipe* (f_{50} = 0.1 %)**	$P_c = 0.5\left\{P'_{ud} + P_{ed}K_d - \left[\left(P'_{ud} + P_{ed}K_d\right)^2 - 4P'_{ud}P_{ed}K_d\right]^{0.5}\right\}$ $P_c = 0.5\left\{P_{ud} + P_{ed}K_d - \left[\left(P_{ud} + P_{ed}K_d\right)^2 - 4P_{ud}P_{ed}K_d\right]^{0.5}\right\}$	$P'_u = 5.1\frac{\sigma_u t_d}{D_0}$ $P_E = \frac{2E}{1-\nu^2}\left(\frac{t_d}{D_0}\right)^3$ $K = 1 + 3f\left(\frac{D_0}{t_{nom}}\right)$ $P_{ud} = \frac{2SMTSt_{\min}}{D_0}$
• Propagating-**Pp***	$Pp = 34 \bullet SMYS\left(\frac{t_{nom}}{D_0}\right)^{2.5}$	

*** Accidental Limit State (evaluated with 10-year return period conditions)**

Table 3.10 – Formulations for Combined Loading States

Loading States (1)	Formulation (2)	Formulation Factors (3)
T - Mu	$\left[\left(\frac{M}{Mu}\right)^2+\left(\frac{T}{Tu}\right)^2\right]^{0.5}=1.0$	
T - Pc	$\frac{P}{Pc}+\frac{T}{Tu}=1.0$	
Mu - Pc	$\frac{P}{Pc}+\frac{M}{Mu}=1.0$ *(load controlled)* $\left(\frac{P}{Pc}\right)^2+\left(\frac{M}{Mu}\right)^2=1.0$ *(displacement cont.)*	
T – Mu - Pc	$\left[\left(\frac{M}{M_u}\right)^2+\left(\frac{P}{P_c}\right)^2+\left(\frac{T}{Tu}\right)^2\right]^{0.5}\leq 1$	
C– Mu -Pb	$M_c=M_p f_M$ $M_p=SMYS\bullet D^2t\left(1-0.001\frac{D}{t}\right)$ $\left[\left(\frac{P}{P_{co}}\right)^2+\left(\frac{M}{M_u}\right)^2+\left(\frac{C}{C_g}\right)^2-2\mu\left(\frac{P}{P_{co}}\bullet\frac{M}{M_u}+\frac{M}{M_u}\bullet\frac{C}{C_g}+\frac{P}{P_{co}}\bullet\frac{C}{C_g}\right)\right]^{0.5}\leq 1$	$f_M=k_1\cos\left[\frac{\frac{\pi}{2}\left(k_2-\frac{1}{2}\frac{\sigma_{he}}{SMTS}\right)}{k_1}\right]$ $k_1=\sqrt{1-\frac{3}{4}\left(\frac{\sigma_{he}}{SMTS}\right)^2}$ $k_2=\frac{C}{\pi\bullet SMTS\bullet Dt}$
C– Mu -Pc	$\left(\frac{M}{M_{co}}\right)^2+\left(\frac{P}{P_{co}}\right)^2\leq 1$ $\left[\left(\frac{P}{P_{co}}\right)^2+\left(\frac{M}{M_u}\right)^2+\left(\frac{C}{C_g}\right)^2-2\mu\left(\frac{P}{P_{co}}\bullet\frac{M}{M_u}+\frac{M}{M_u}\bullet\frac{C}{C_g}+\frac{P}{P_{co}}\bullet\frac{C}{C_g}\right)\right]^{0.5}\leq 1$	$M_{co}=M_p\cos\left(\frac{\pi}{2}\frac{T}{T_y}\right)$ $M_p=SMYS\bullet D_0^2t_{nom}\left(1-0.001\frac{D_0}{t_{nom}}\right)$ P_{co} : Timoshenko Ultimate or Elastic equation

4.0 Hydrodynamic Loadings

4.1 Demands: Wave and Current Forces

The demands on the pipeline for hydrodynamic loadings are derived from hurricane wave and current kinematics. The wave and current velocities and accelerations normal to the axis of the pipeline are of primary importance to these loadings.

In development of these guidelines, it has been assumed that the hurricane conditions characterizations are based on calibrated 'second generation' hurricane hindcasting methods. As appropriate for specific locations, unique bathymetric, geographic, and geotechnical conditions must be recognized in order to account for shallow water effects on the deep water wave and current characteristics.

The stability guidelines for in-place conditions are based on the 100-year expected maximum and significant wave heights and the currents that occur at the time of these sea states. The wave kinematics developed for the 100-year design conditions should be modified for directional spreading and the principal angle of approach of the waves relative to the axis of the pipeline. Generally, the principal direction of the waves for extreme condition hurricanes will be approximately normal to the bathymetry. Generally, the principal direction of the near sea floor currents will be parallel to the bathymetry. Thus, the vectorial addition of wave and current velocities should take account of the lack of alignment of the principal wave propagation direction and the near sea floor current direction relative to the axis of the pipeline.

It is assumed that the AGA guidelines and Level 2 approach will be used to evaluate the stability of the pipelines (Pipeline Research Committee, 1993). For proper use or application of these guidelines, it is critical that unbiased estimates (neither conservative or unconservative, but expected or best estimate values) should be used for all parameters that are used in the analyses of hydrodynamic forces developed on a pipeline or riser.

This guideline applies particularly to the definition of drag (Cd), inertia (Cm), and lift (Cl) coefficients - as functions of turbulence (Reynold's Number), flow persistence (Keulegan-Carpenter Number), the strength of the currents relative to the wave velocities, the proximity of the pipeline to the sea floor (normally on the bottom or partially buried), and the roughness of the pipeline. Unbiased estimates are suggested in the criteria developed by Weiss (1997) and documented in the American Gas Association (AGA) guidelines (Pipeline Research Committee, 1993).

4.2 Capacities - Uplift and Sliding

The lift stability is determined by the weight of the pipeline, its contents, and weight coating (W) relative to the hydrodynamic uplift forces of buoyancy and flow lift (Fl) (Figure 4.1)

The sliding or lateral stability is determined by the characteristics of the soils (sliding or frictional resistance and passive resistance due to burial of the pipeline, Ru) relative to the lateral forces developed by the waves and currents (Fd). For cohesionless soils, there is a coupling between the net weight of the pipeline (W - Fl) and Ru:

$$Ru = (W - Fl) \tan \phi'$$

where represents the effective value of internal sliding friction between the soil and the pipeline. The effective value reflects the effects of wave cycling, pipeline motions, and currents.

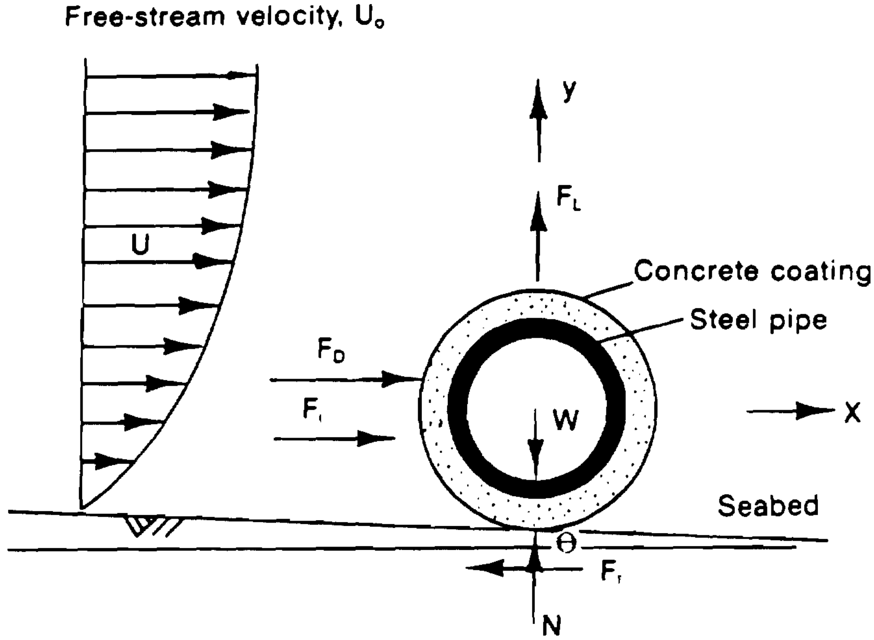

Figure 4.1 - Formulation of Pipeline Stability

The lateral sliding or shear resistance of a pipeline in cohesive soil is independent of the vertical stress or net pipeline weight:

$$Ru \approx Ap\ Su$$

where Ap is the area of the pipeline in contact with the soil and Su' is the effective undrained shear strength of the soil. The effective undrained shear strength of the soil reflects the influences of wave cycling, pipeline motion, and currents.

The AGA pipeline burial analysis process should be calibrated to produce results that agree with results from experiments that have been performed to determine the rates of burial of pipelines in very soft sea floor soils (Ghazzaly, 1975; Morris, Webb, Dunlap, 1988).

The AGA stability analysis process provides an advanced state-of-the-art approach to estimating both the hydrodynamic loadings and the pipeline stability characteristics. This stability analysis process will be taken as the reference for development of the risk based on-bottom stability criteria.

4.3 Probability of Stability Failure

The probability of failure of the pipeline in uplift can be expressed as:

$$Pfl = P\ (Fl \geq W)$$

The probability of failure of the pipeline in sliding can be expressed as:

$$Pfd = P\ (Fd \geq Ru)$$

Given the high degree of correlation between these two modes of failure, the probability of stability failure is equal to the maximum of either Pfl or Pfd. Lack of recognition of the high correlation would indicate that the probability of stability failure (Ps) would be (assuming uncorrelated, independent variables):

$$Ps = Pfd + Pfl$$

In a Lognormal demand and capacity format, and based on characterization of the demands and capacities with loadings and loading resistances, the probabilities of stability 'failure' can be expressed in terms of the annual Safety Index as:

$$\beta l = (\ln\ (W_{50} / Fl_{50})) \setminus \sigma_{\ln FlW}$$

and

$$\beta s = (\ln\ (Ru_{50} / Fd_{50})) \setminus \sigma_{\ln FdRu}$$

The subscripts ($_{50}$) indicate median expected annual maximum values. The ratios of median capacities to demands represents a central tendency measure of the factor of safety (FS). Based on the use of expected maximum 100 year hurricane conditions (no biases), the 'required' factors of safety in uplift and sliding can be expressed as:

$$FS_{99} = B \exp\ (\beta\ \sigma - 2.33\ \sigma_{\ln F})$$

where indicates the FS for the 99th annual maximum percentile condition (average return period of 100 years). B is the bias in demands and capacities:

$$B = B_F / B_{Ru}$$

σ is the total uncertainty in the demands and capacities (standard deviation of the logarithms of the demands and capacities):

$$\sigma^2 = \sigma^2_{\ln Ru} + \sigma^2_{\ln F}$$

$\sigma_{\ln F}$ is the uncertainty in the expected annual maximum hydrodynamic forces and $\sigma_{\ln Ru}$ is the uncertainty in the expected maximum resistance (uplift or sliding).

Based on the previous work on the uncertainties associated with hydrodynamic loadings developed on platforms, pipelines, and risers in the Gulf of Mexico and the Bay of Campeche, an evaluation of the uncertainties associated with pipeline loadings (Grace, Nicinski, 1976; Grace, Zee, 1981; Chao, 1989), the uncertainty in the hydrodynamic loadings is estimated to be $\sigma_{\ln F} = 0.8$. The uncertainty in the pipeline stability is estimated to be $\sigma_{\ln Ru} = 0.25$. The total uncertainty is thus estimate to be $\sigma = 0.84$.

There is an important source of bias that has not been discussed. This bias is due to the lack of correlation of the demand or imposed hydrodynamic forces over the entire portion of the pipeline or riser and the 'transient' nature of these loadings. This lack of correlation and transient nature of the loadings leads to different forces at different times and positions over the pipeline or riser (Figure 4.2). As hydrodynamic forces on one portion of the pipeline become sufficient to cause uplift or sliding, they are not sufficient over adjacent portions to cause uplift or sliding. The adjacent portions of pipeline act to restrain the pipeline and limit the displacements or deformations induced in the pipeline or riser.

This spatial - temporal correlation bias will be dependent on the orientation of the pipeline relative to the imposed currents and wave kinematics (Figure 4.2) and the relative strengths of the currents and wave kinematics. Pipelines that are routed parallel to the bathymetry (Case 1) and whose loadings are primarily caused by the wave kinematics, could be expected to have a relatively high correlation in the forces along the length of the pipeline, depending on the crest lengths of the hurricane waves. A similar observation could be made for pipelines that are routed perpendicular to the bathymetry and whose forces would be primarily those from the bottom currents (Case 2). For these two routing cases, the force correlation bias is evaluated conservatively to be unity ($B_{FC} = 1.0$). For the third routing case (Case 3, at an angle to the bathymetry), the force correlation bias is evaluated to be $B_{FC} = 0.5$). Research on the spatial characteristics of hydrodynamic loadings developed on pipelines has been conducted during this project by Prof. Kareem and Graduate Student Researcher Xinzhong Chen. Results from the first phase (January – July 1999) of this research are summarized in Appendix A. The final results from this research will be included in the December 1999 report.

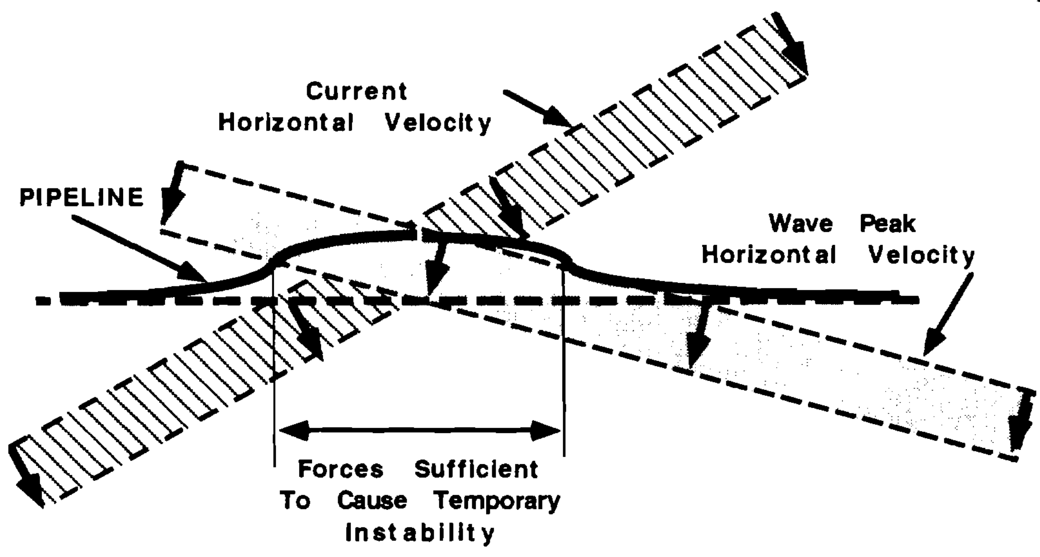

Figure 4.2 - Lack of Correlation in Time and Space of Destabilizing Forces on Pipeline or Riser

Based on the proposed design and requalification analytical procedures, it is stipulated that best estimate values will be used to determine all of the design values that determine the pipeline resistance to the destabilizing forces (weight, soil characteristics, embedment); thus, $B_{ru} = 1.0$.

The resultant bias will then be the product of the two force biases: the bias due to the pipeline direction and the bias due to the lack of correlation of forces along the length of the pipeline:

$$B = B_{f\theta} \cdot B_{FC}$$

4.4 Stability Factors of Safety

Based on the foregoing developments, the Factor of Safety for operating conditions stability assessments based on 100-year design conditions are summarized in Figure 4.3 for Biases of B = 0.1 to 0.5 (due to pipeline route and force correlation) and a range of annual Safety Indices from 2 to 4 (range for design of new pipelines and requalification of existing pipelines).

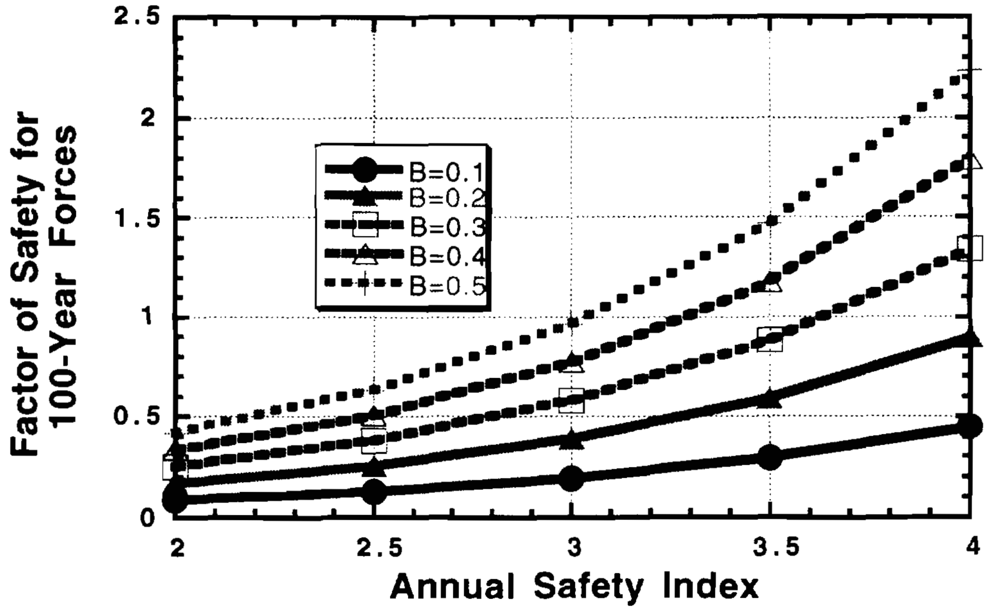

Figure 4.3 - Factors of Safety for Pipeline Operating Stability Based on Best Estimate 100-year Design Conditions and AGA Stability Analysis Procedures For Bias (B) Due to Pipeline Route and Force Correlation

For annual Safety Indices of 3.0 to 4.0 (Pf = 1 E-3 to 1 E-4), and B = 0.5, the Factor of Safety is ≥ 1.0. This is implied by the current API guidelines that specify that stability is to be evaluated typically for the 100-year conditions with a 1.0 factor of safety.

Figure 4.4 summarizes the Factors of Safety for pipeline installation stability based on the use of 10-year return period conditions and bias due to the pipeline route and force correlation.

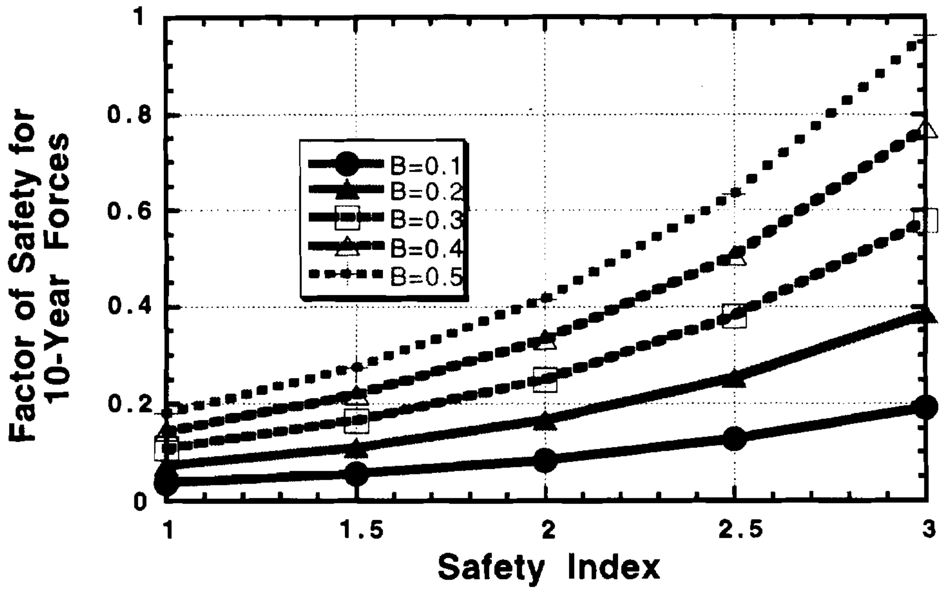

Figure 4.4 - Factors of Safety for Pipeline Installation Stability Based on Best Estimate 10-year Design Conditions and AGA Stability Analysis Procedures For Bias (B) Due to Pipeline Route and Force Correlation

Factors of Safety for operating stability evaluations for design of new pipelines for the Safety and Serviceability Classifications (SSC) associated with the target reliabilities identified in this study and for 100-year design conditions are summarized in Table 4.1. The Factors of Safety are very sensitive to the pipeline route and ratio of current to wave velocities due to the design analysis based on superposition of the wave and current kinematics. Note for the case in which the current and wave velocity ratio is unity (equal to each other), that the Factor of Safety is independent of the pipeline route and varies from FS = 0.88 to FS = 0.52 for SSC 1 and SSC 3, respectively.

For reassessment of existing pipelines, the analyses will be performed on the basis of the pipeline route and the wave and current conditions along this route, taking account of directionality of these conditions. Thus, the bias introduced for the pipeline route can be omitted. This leaves only the bias introduced to recognize the lack of perfect spatial and temporal correlation of the forces along the pipeline segment length (length of pipeline with similar hydrodynamic conditions). As noted earlier, it is expected that this correlation bias will be a function of the pipeline route relative to the hurricane wave and current kinematics and the relative strengths of these kinematics. The current kinematics could be expected to be highly correlated over the length of the pipeline. The wave kinematics could be expected to be poorly correlated over the length of the pipeline. For pipelines that are routed so that they are perpendicular to the wave and / or current kinematics, it could be expected that the spatial - temporal correlation of the forces would be high and the force bias close to unity. However, for pipelines whose routes are such that they are at an angle relative to the wave kinematics, then a large spatial - temporal correlation effect on the forces could be expected.

Table 4.1 - Factors of Safety for Pipeline Operating Stability Evaluations for Pipeline Design for 100-year Hurricane Conditions

Factors of Safety for SSC 1 ($\beta = 3.72$)			
pipeline route	$\psi = 1$	$\psi = 2$	$\psi = 3$
Case 1	0.88	0.39	0.22
Case 2	0.88	0.88	0.88
Case 3	0.88	1.6	2.0
Factors of Safety for SSC 2 ($\beta = 3.29$)			
pipeline route	$\psi = 1$	$\psi = 2$	$\psi = 3$
Case 1	0.61	0.27	0.15
Case 2	0.61	0.61	0.61
Case 3	0.61	1.1	1.4
Factors of Safety for SSC 3 ($\beta = 3.10$)			
pipeline route	$\psi = 1$	$\psi = 2$	$\psi = 3$
Case 1	0.52	0.23	0.13
Case 2	0.52	0.52	0.52
Case 3	0.52	0.92	1.2

Given that the wave and current kinematics perpendicular to the axis of the pipeline are used in the requalification analyses, the spatial - temporal correlation bias in the forces was analyzed as follows. The ratio of the current velocity (high correlation) to the wave velocity (low correlation) is again designated as ψ. The correlation of the current velocity over the pipeline segment was taken as unity (Kc = 1.0). The correlation of the wave velocity over the pipeline segment was taken as ranging from Kw = 0 (no correlation over length of pipeline segment) to Kw = 1.0 (perfect correlation over length of pipeline segment). The force bias due to the lack of correlation of the wave kinematics can then be expressed as:

$$B_{FC} = (\psi + Kw)^2 / (\psi + 1)^2$$

The results are summarized in Figure 4.5. For pipelines that are generally parallel to the wave crests and perpendicular to the wave kinematics and consequently have relatively high spatial correlation (Kw $\approx$ 1), the force bias is close to unity. For pipelines that are not perpendicular to the wave kinematics and consequently have relatively low spatial correlation (Kw $\approx$ 0.25), the force bias is $B_{FC} \approx 0.5$. As the ratio of the velocity of the current to the wave velocity increases, the force bias increases, indicating a lesser effect of the low correlated wave forces and a greater effect of the highly correlated current forces.

Since it is not possible to make any general statement regarding the expected pipeline route relative to the hurricane wave and current kinematics, Factors of Safety for requalification of existing pipelines were developed for two cases of wave kinematics correlations: low ($B_{FC} \approx 0.4$) and high ($B_{FC} \approx 0.8$). The resulting Factors of Safety for operating stability of existing pipelines are summarized in Table 4.2.

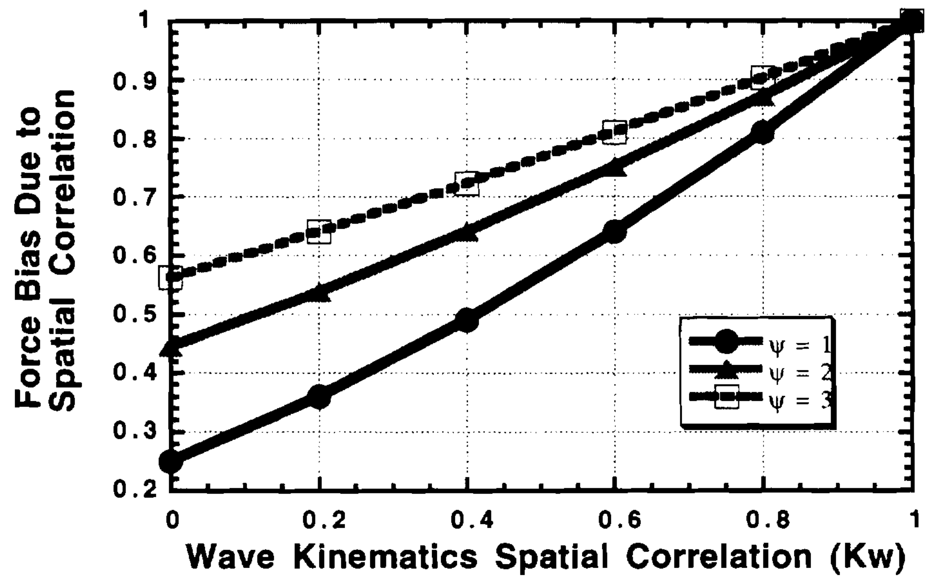

Figure 4.5 - Force Bias Due to Lack of Perfect Correlation of Wave Kinematics over Pipeline Segment

Table 4.2 - Factors of Safety for Requalification of Existing Pipelines for Operating Stability for 100-year Hurricane Directional Conditions

SSC	Annual Safety Index (β)	FS (low force correlation)	FS (high force correlation)
1	3.54	1.2	2.4
2	3.10	0.83	1.7
3	2.87	0.69	1.4

Given that design of pipelines would be based on analyses that took into account the directionality of the design wave kinematics and currents relative to the pipeline segment, the bias due to the kinematics directionality would be unity. This would leave only the spatial - temporal force correlation bias. Table 4.3 summarizes the results for design of new pipelines based on directionality of the wave and current kinematics for 100-year hurricane conditions relative to the pipeline segment for routes that would have low and high correlation of forces.

Table 4.3 - Factors of Safety for Design of New Pipelines for Operating Stability for 100-year Hurricane Directional Conditions

SSC	Annual Safety Index (β)	FS (low force correlation)	FS (high force correlation)
1	3.72	1.4	2.8
2	3.29	1.0	2.0
3	3.10	0.84	1.7

Factors of Safety to evaluate the stability of pipelines during the installation period have been developed on the same basis as for the operating conditions except that the reduced exposure period and consequences of failure have been recognized. The results are summarized in Table 4.4 for assessment of the pipeline stability for 10-year return period conditions and the assumption of colinearity of the wave and current kinematics.

Table 4.4 - Factors of Safety for Pipeline Installation Stability Evaluations for Pipeline Design for 10-year Hurricane Conditions

Factors of Safety for SSC 1 (β = 2.32)			
pipeline route	$\psi = 1$	$\psi = 2$	$\psi = 3$
Case 1	0.63	0.28	0.16
Case 2	0.63	0.63	0.63
Case 3	0.63	1.1	1.4
Factors of Safety for SSC 2 (β = 1.65)			
pipeline route	$\psi = 1$	$\psi = 2$	$\psi = 3$
Case 1	0.36	0.16	0.09
Case 2	0.36	0.36	0.36
Case 3	0.36	0.63	0.80
Factors of Safety for SSC 3 (β = 1.28)			
pipeline route	$\psi = 1$	$\psi = 2$	$\psi = 3$
Case 1	0.26	0.12	0.07
Case 2	0.26	0.26	0.26
Case 3	0.26	0.46	0.59

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5.0 On-Bottom Stability Soil Forces Criteria

5.1 Background

Sea wave - sea floor interactions have been found to be an important mechanism for attenuation of waves in areas where the sea floor is overlain with a layer of deformable clays (Bea, 1971; 1974; 1981; 1996; Suhayda, 1976; 1996; Forristall, et al, 1980; 1981; 1985; Gu, Thompson, 1995; Clukey, et al, 1990). Much of the sea floor in the Bay of Campeche is overlain by such a layer that varies in thickness from 0 m to 20 m. This is one of the primary reasons that the majority of pipelines in this area have become buried (Valdez, et al, 1997). In addition, much of the Eastern section of the Louisiana coast near the present Mississippi River delta is overlain by a very significant thickness of deformable clays.

As waves propagate over the sea floor, deformations are induced in the sea floor soils (Figure 5.1). In the case of soft sea floor soils, these deformations can be very large for severe sea state conditions. These deformations are of two types. The first are the deformations that develop during the passage of individual waves (Figure 5.1). The second are the accumulated deformations that develop during the passage of many waves (Figure 5.2). These accumulated deformations will be greater for greater sea floor slopes. These deformations can have both horizontal and vertical components. In areas of soft sea floor soils, following severe hurricanes, pipelines have been found displaced down-slope several hundred meters (Bea, et al, 1975; Bea, Audibert 1980; Bea, et al, 1980; Bea, 1996).

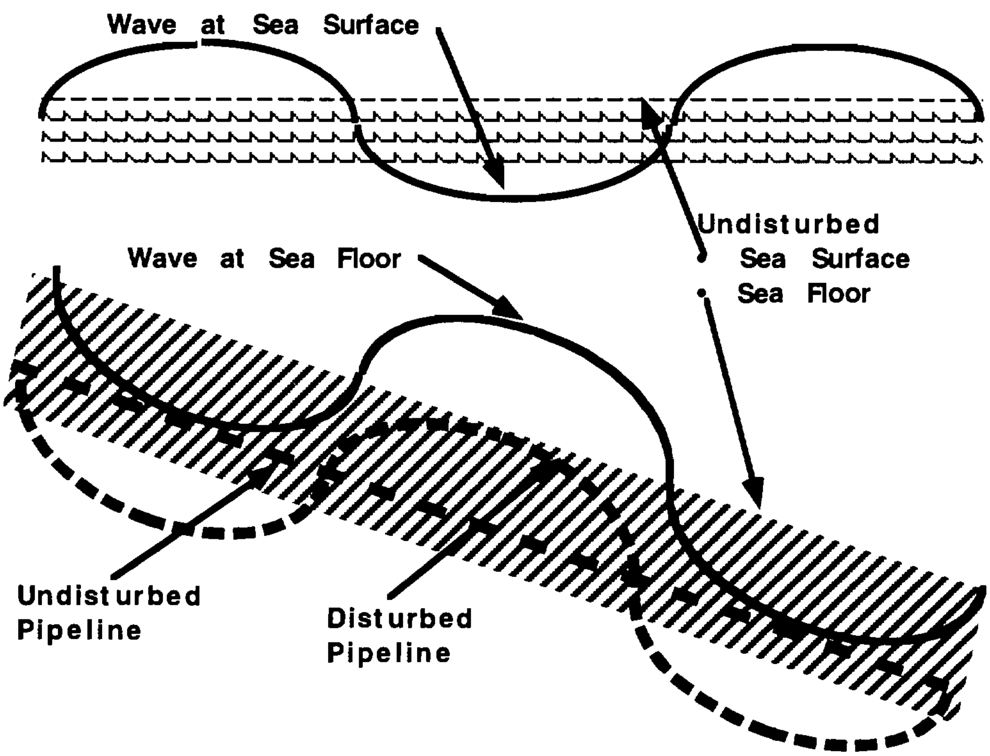

Figure 5.1 - Surface and Sea Floor Waves with Pipeline Interactions

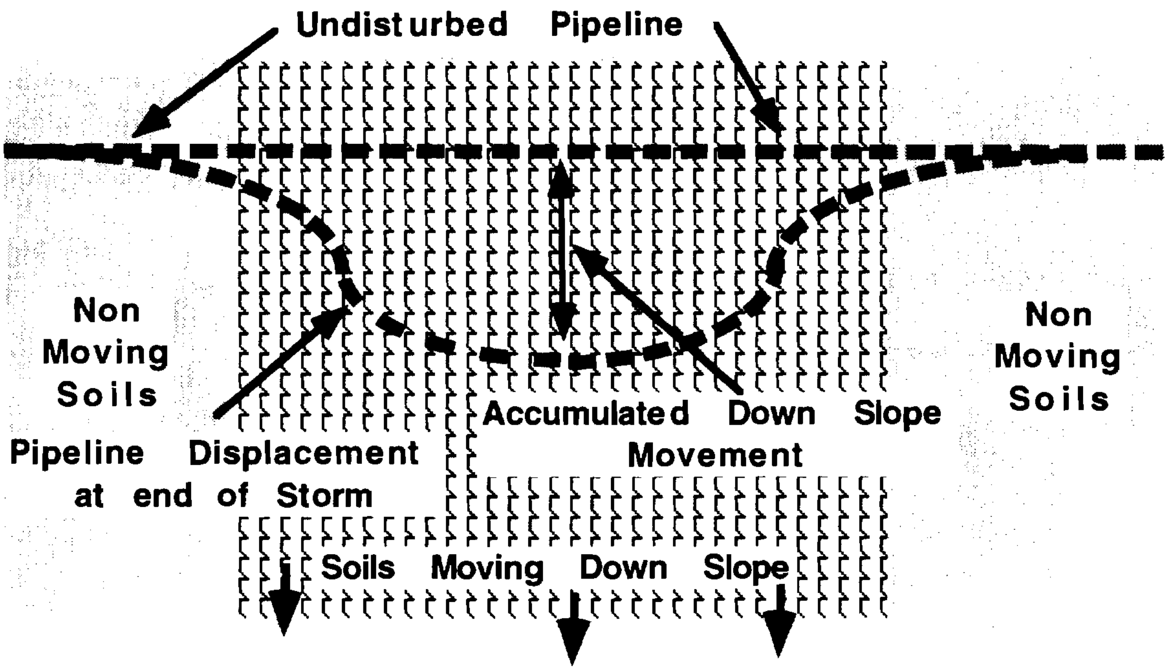

Figure 5.2 - Down Slope Movement of Pipeline after Hurricane

Analyses can be performed to determine the deformations induced in a pipeline by individual waves and by a series of waves (Arnold, 1971; Audibert, Lai, Bea, 1978; 1980). This section addresses the issues of movements of the soft sea floor soils in the Bay of Campeche during intense hurricanes and their potential effects on the design of pipelines and risers.

As a part of this study, detailed analyses were performed to determine the 'wave frequency' displacement characteristics of the sea floor soils characteristic of those in the Mississippi Delta area and in the Bay of Campeche. These analyses utilized the same oceanographic and geotechnical characterizations used in an earlier study of the effects of soft sea floor soils on the maximum wave heights in the Bay of Campeche (Zhaohui Jin, Bea, 1997; Bea, 1997; Suhayada, 1997a; 1997b).

5.2 Wave Frequency Displacements

In this simplified analysis, it is conservatively assumed that the pipeline has the same movement characteristics of the soils that it is embedded in (Figure 5.1). Further, it is assumed that the pipeline motions are the same as that at the sea floor. This is equivalent to assuming that the pipeline is buried relatively shallow in the soft sea floor soils.

With selection of appropriate parameters, the maximum vertical movement (Δv) of the sea floor can be determined using results from analyses of an elastic half space subjected to excitation from a sinusoidal surface wave (Dawson, Suhayda, Coleman, 1981):

$$\Delta v = (\Delta_H / \alpha^2 - k^2) ((\alpha^2 - k^2) e^{kz} - 2 k^2 e^{\alpha z})$$

where

$$\Delta_H = (H/2) (\cosh kh - (g/k c^2) \sinh kh)$$

$$\alpha = k\,(1 - \rho'\,c^2 / G)^{0.5}$$

H is the wave height, k is the wave number ($2\pi / L$), h is the water depth, and c is the wave celerity ($c = L/T$) of the Airy surface wave, G is the soil shear modulus, and ρ' is the soil density.

For realistic results from the foregoing, the soil properties need to reflect the effects of the hurricane prior to the arrival of the maximum wave heights that induce the maximum displacements in the sea floor (Esrig, Ladd, Bea, 1975). This is also very important when assessments are made of the soil loadings that will occur on the pipelines in areas where the soils are moving.

Figure 5.3 summarizes the results from the SWBI (Sea Wave Bottom Interactions) analyses of the soil movements along the Dos Bocas pipeline in the Bay of Campeche for 100-year, 1,000-year, and 10,000 year return period hurricane conditions. The movements shown are the maximum vertical amplitude of motion that occur during the hurricanes for the best estimate soil characteristics. The horizontal scale references the locations along the length of the Dos Bocas pipeline. The UTMX = 597 is at the northeastern end of the pipeline in a water depth of 47 m. The UTMX = 547 is in a water depth of 38 m. The UTMX = 490 is in a water depth of 92 m. The dramatic increase in the vertical movements at UTMX = 560 is due to a dramatic change in the soil characteristics at this point - a change from stiff to very soft soils at this point along the pipeline. The pipeline could be expected to experience very large relative motions in this portion of the pipeline. For the 100-year hurricane conditions, the maximum vertical displacements (double amplitude) range from about 0.05 m to about 0.16 m. For the 10,000-year hurricane conditions, the maximum vertical displacements range from about 0.1 m to about 0.8 m. For all conditions, the maximum vertical displacements are less than 1 m.

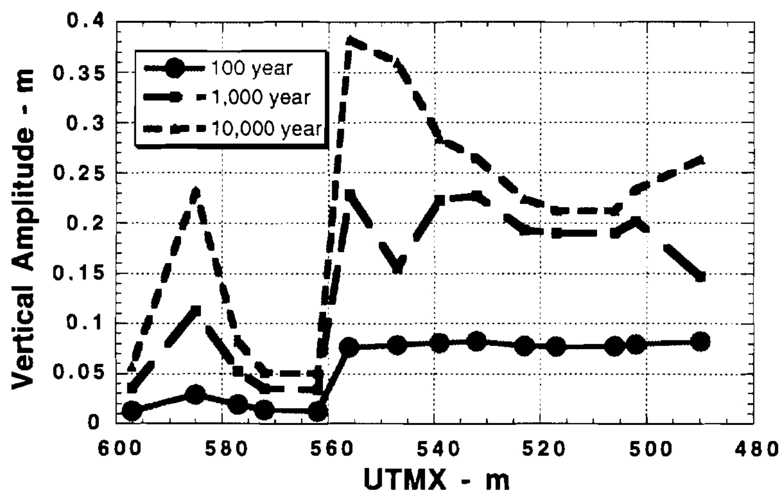

Figure 5.3 - Amplitude of Maximum Vertical Soil Movements Along the Axis of the Dos Bocas Pipeline for 100-year, 1,000-year, and 10,000-year Hurricane Conditions

Figures 5.4 through 5.6 summarize the amplitude of maximum vertical soil movements along three transects in the Bay of Campeche. Transect A is normal to the bathymetry in the central portion of the Bay of Campeche and represents a broad shelf condition. Transect B is in the southwestern part of the Bay of Campeche and represents a narrow shelf example. Transect C is a line that is midway between Transects A and C. The variations in the vertical movements for the different transects reflect the effects of the wave shoaling and the soil characteristics. In all cases, the maximum vertical displacements are less than 1 m.

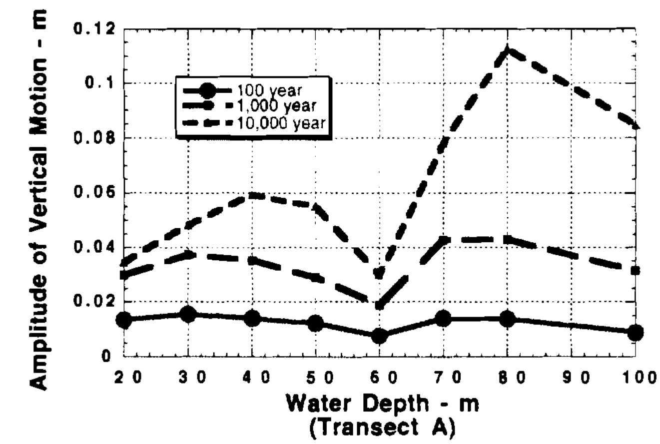

Figure 5.4 - Amplitude of Maximum Vertical Soil Movements Along Transect A for 100-year, 1,000-year, and 10,000-year Hurricane Conditions

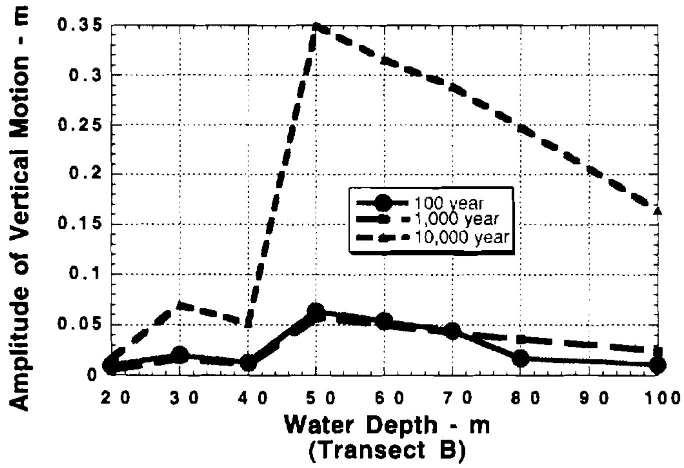

Figure 5.5 - Amplitude of Maximum Vertical Soil Movements Along Transect B for 100-year, 1,000-year, and 10,000-year Hurricane Conditions

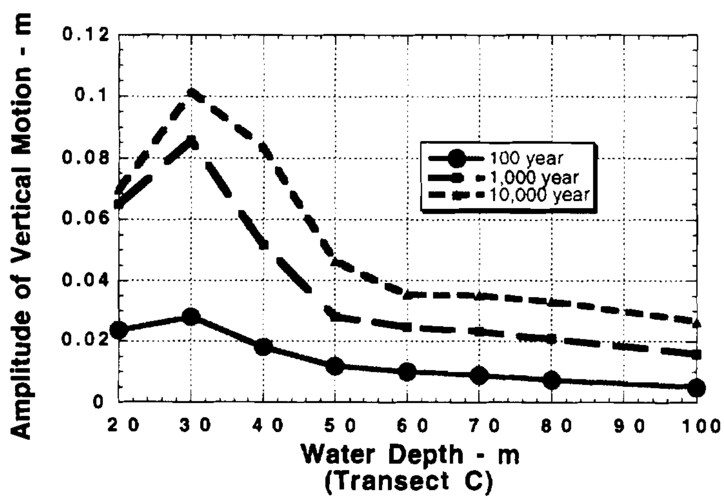

Figure 5.6 - Amplitude of Maximum Vertical Soil Movements Along Transect C for 100-year, 1,000-year, and 10,000-year Hurricane Conditions

5.3 Sea Floor Movement Induced Displacements

Horizontal soil movement displacements are a function of the bottom slope, the sea floor soil characteristics, the intensity of the hurricane reflected in the history of wave heights and periods, and the duration of the hurricane. On a perfectly horizontal sea floor, given 'unskewed' waves (equal amplitudes above and below still water), the soil movements would be closed elliptical orbits. However, on a slope, the soil movements would describe open elliptical orbits with a 'steadily accumulating' displacement down slope (Doyle, 1973). This is analogous to slow drift forces associated with water waves or 'wave induced currents.'

Values that have been observed in the soft sea floor areas of the Mississippi River following hurricanes whose intensities approximate those of a 100-year hurricane (e.g. hurricane Camille) for bottom slopes comparable with those in the Bay of Campeche range from 500 m to 2,000 m and occur over widths of 500 m to 10,000 m (Arnold, 1971; Bea, et al, 1975; 1980; Bea, 1996).

Given such a range, it is not possible nor warranted to develop general guidelines for the Bay of Campeche or other locations in the Northern Gulf of Mexico. The soil movement displacement characteristics for the design of a particular pipeline should depend on the route of a particular pipeline and the soil and bathymetric characteristics along this route. Side-scan sonar surveys of pipeline routes together with results from studies of the geotechnical characteristics and hazards along the pipeline route should provide the bases for route specific analyses of potential soil movement induced displacements (Audibert, Lai, Bea, 1979, 1980; Bea, 1980, 1982, 1983).

5.4 Pipeline Displacement Capacities

The displacement and stress capacities of pipelines have been studied extensively. Both simple and complex analytical models have been developed for this purpose (Arnold, 1971; Bea, 1981; Mousselli, 1981). These methods have proven to be reliable for a wide variety of applications from determining pipeline laying stresses to determining pipeline stresses in sag and hog bends.

5.4.1 Wave Frequency Displacement Capacities

For the wave frequency displacements, the assumption made in this preliminary analysis is that the pipeline follows the displacements induced in the soils. The maximum vertical displacement of the pipeline would be equal to the double amplitude of the wave induced at the sea floor (Figure 5.3). It is assumed that the pipeline has been laid with sufficient slack so that there is no significant tension in the pipeline. Given these assessments, the maximum stress (Sm) induced in the pipeline is:

$$Sm = 8\ E\ r\ Hsf / L^2$$

where E is the modulus of elasticity, r is the pipeline radius, Hsf is the vertical displacement (double amplitude) of the sea floor, and L is the surface wave length (assumed to be associated with a linear sinusoidal wave):

$$L = (g\ T^2 / 2\pi)\tanh(2\pi d / L)$$

In deep water ($d/L \geq 0.5$):

$$L = g\,T^2 / 2\,\pi$$

In shallow water ($d/L \leq 1/25$):

$$L = T\,(g\,d)^{0.5}$$

Figure 5.7 summarizes the foregoing development in terms of the maximum induced stress (Sm) as a function of the dimensionless ratio:

$$Kv = r\,Hsf / L^2$$

For example, assume a 12-inch pipeline is displaced by a sea floor vertical motion of 36 inches that has a wave length of 500 ft (6,000 in). The dimensionless ratio would be 6 E-6 and the resulting maximum stress would be 1,400 pounds per square inch (psi).

An alternative formulation could be developed by determining the displacement capacity of the pipeline based on the strain induced in the pipeline as a function of the radius of curvature induced by the soil - wave motions. The strain (ε) induced in the pipeline of radius ($r = D / 2$) when it is bent to a minimum radius of curvature (Rmin) is:

$$\varepsilon = r / Rmin$$

Assuming that the wave induced motion in the soil is sinusoidal with displacement Hsf:

$$Rmin = L^2 / 2\,Hsf\,\pi^2$$

Basing the analysis on the shallow water wave lengths (conservative):

$$\varepsilon = D\,Hsf\,\pi^2 / g\,d\,T^2$$

Given Hsf = 1 m and T = 13 s in a water depth of d = 60 m and D = 0.3 m, ε = 3 E-5. The yield strain would be about 2 E-3. Thus, the stress would be very low (less than 1,000 psi).

It is apparent that the vertical sea floor motions would have to be very large, much larger than shown in Figures 5.3 through 5.6 for the stresses induced by the vertical sea floor movements to be important.

This observation correlates well with the performance of the pipelines in the Bay of Campeche during hurricane Roxanne. While pipelines were displaced down-slope from their initial conditions and there were numerous failures of the pipelines at their connections to other lines, there was no evidence of simple overstress of the pipelines leading to loss of containment.

There is one other phenomenon that could be important that could be associated with the vertical sea floor movements: cracking and loss of weight coating. The movements could lead to loss of weight coating, leading to 'floating' of the pipelines and exposing them to hydrodynamic loadings which if great enough could lead to loss of containment.

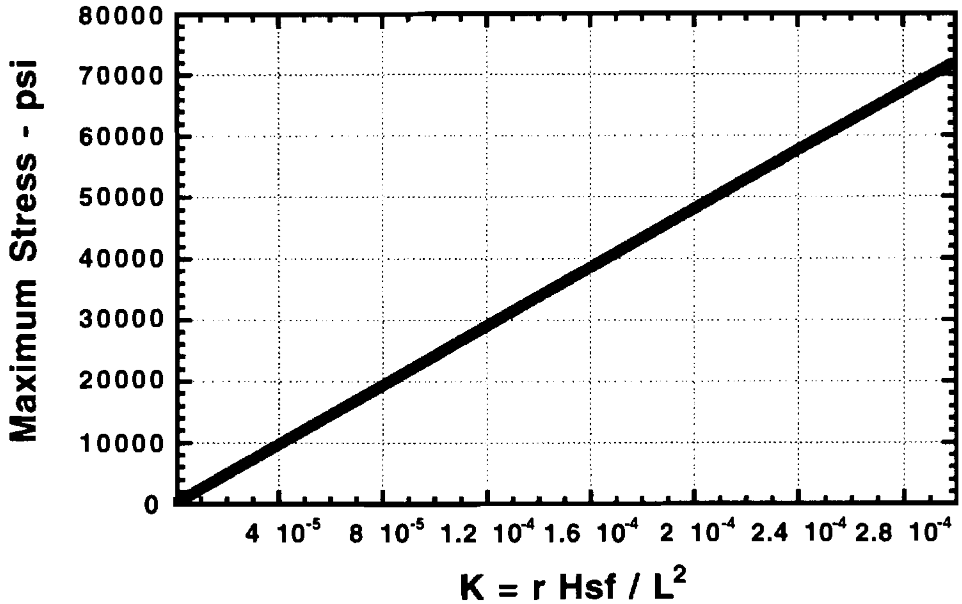

Figure 5.7 - Maximum Stress Due to Pipeline Vertical Displacement by Sea Floor Soils

5.4.2 Sea Floor Displacement Capacities

The preliminary formulation for the maximum stress induced in the pipeline by down slope moving soils follows along the same lines. In this case, the pipeline is treated as a catenary whose axial tension (T) is (Mousselli, 1981):

$$T = (w\,L / 2)\,(h^2 + 1)^{0.5}$$

where w is the imposed soil lateral loading on the pipeline, L is the total pipeline span across the zone of moving soils, and h:

$$h = r / \varepsilon\,L$$

r is the pipeline radius and ε is the strain induced in the pipeline.

The total span width that the pipeline can withstand can be determined from:

$$L = 2\,[\,(Sm\,A / w)^2 - (r\,E / Sm)^2\,]^{0.5}$$

where Sm is the tensile stress at failure (1.5 Smy ≈ 1.5 times the specified minimum yield stress), A is the pipeline cross section area, and E is the modulus of elasticity of the pipeline steel.

The soil lateral loading imposed on a fully embedded pipeline in cohesive soil can be expressed as:

$$w = Kb\ Su'\ D$$

where Kb is the lateral bearing coefficient (Audibert, Lai, Bea, 1978, 1980), Su' is the soil shear strength at the time of the soil movements (Esrig, et al, 1975), and D is the pipeline diameter. Based on field measurements on pipelines embedded in soft clays (Bea, 1985; Bea, Aurora, 1981), the lateral bearing coefficient is defined as:

$$Kb = 10$$

The general effective undrained shear strength of the soil in the deformable sea floor areas of the Bay of Campeche and near the Mississippi River Delta is evaluated to be 50 to 100 pounds per square foot at the time of the soil movements (evaluated to be equal to the remolded shear strength, the soil at critical state, a soil with a static undisturbed shear strength of 150 to 300 pounds per square foot). The lateral soil loading on the pipeline is w = 500 to 1,000 pounds per foot of pipeline diameter and per foot of pipeline.

Figure 5.8 summarizes the results of the foregoing for Schedule 40 pipelines with diameters ranging from 6 inches to 24 inches composed of X52 steel (Smu = 78,000 psi = 1.5 x 52,000 psi).

Pipelines should normally be able to span soil movement widths in the range of 1,000 feet to 4,000 feet depending on the pipeline diameter and wall thickness. These results are in excellent agreement with results from nonlinear beam- column analyses of pipelines that span mudslide areas (Bea, 1983; Arnold, 1971).

A summary of the results developed by Arnold (1971) based on nonlinear beam column analyses of the responses of pipelines to soil displacements is summarized in Figure 5.9. The results are shown as the maximum tensile stress induced in the pipeline at equilibrium (Sm, kips per square inch) versus Kp, where:

$$Kp = SF \cdot SW / 1000\, D\, t$$

where SF is the soil force per unit length of the pipeline (pounds per inch), SW is the slide width (inches), D is the pipeline diameter, and t is the pipeline wall thickness.

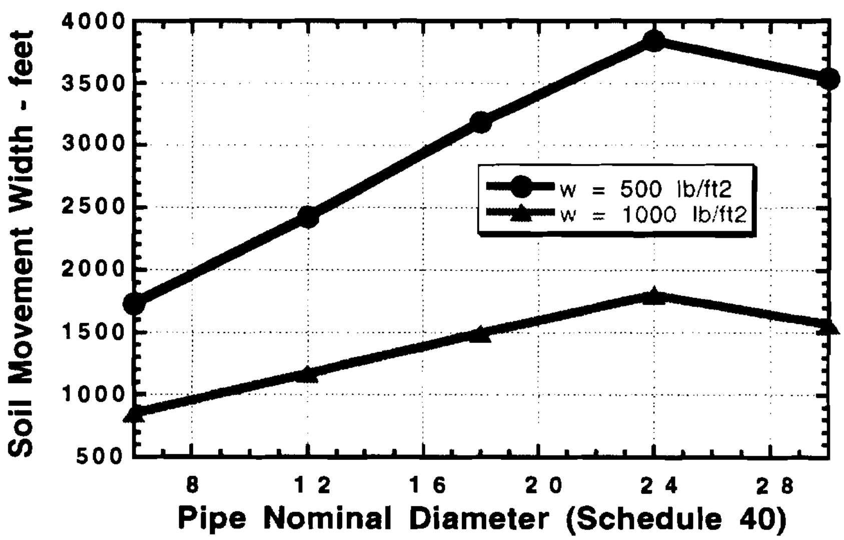

Figure 5.8 - Maximum Pipeline Span Widths for Sea Floor Displacements

Again, these results seem to be in reasonable agreement with the observed performance of pipelines in the Bay of Campeche during hurricane Roxanne and pipelines in the Mississippi River Delta during past hurricanes. It was only in a few cases that involved very large widths of implied soil movements that the pipelines were found to be displaced significant distances, and only in a few of these cases did the pipelines loose containment.

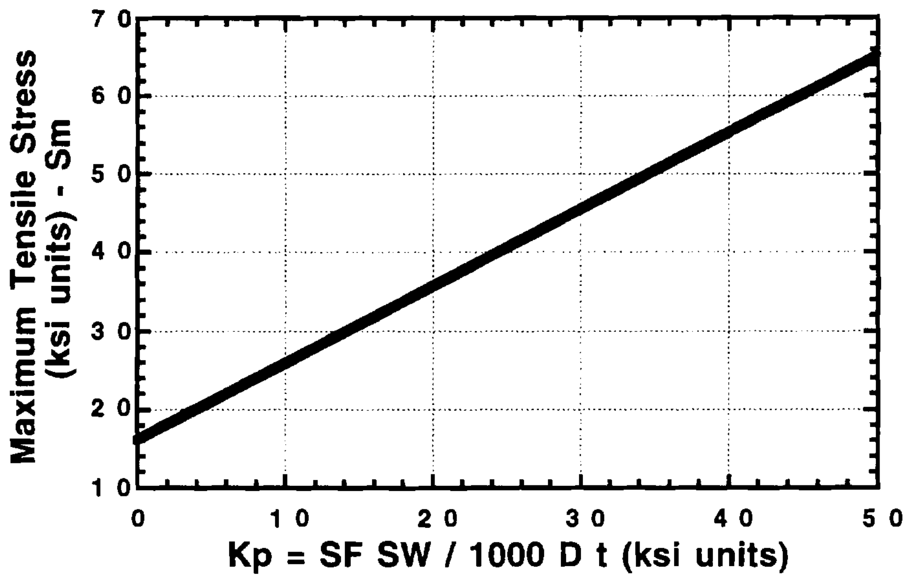

Figure 5.9 - Pipeline Tensile Stress as Function of Soil Force and Movement Width

5.5 Risk Based Criteria for Soil Movements

5.5.1 Wave Frequency Displacements

The probability of failure of a pipeline associated with wave frequency displacements can be expressed as:

$$Pf = P\ (Sm \geq Su)$$

where Sm is the maximum stress induced in the pipeline by the surface waves effects on the sea floor soils:

$$Sm = 8\ E\ r\ Hsf\ /\ L^2$$

and Su is the ultimate stress at failure of the pipeline. Alternatively, the expression for Pf could be expressed in terms of the strain or deformation demands and capacities.

The factor of safety (FS) for the 100-year wave frequency induced displacements is:

$$FS_{50} = (\ B_S\ /\ B_R\ )\ (\beta\ \sigma - 2.33\ \sigma_{Sm})$$

where B_S is the median bias in the demand or stress induced in the pipeline, B_R is the median bias in the pipeline capacity, β is the annual Safety Index for this failure mode of the pipeline, and σ is the total uncertainty in the pipeline wave loading frequency demands and capacities and σ_{S_m} is the uncertainty in the maximum stresses induced in the pipeline by the sea floor soil movements.

In this development, it is assumed that B_S is evaluated to be unity. B_R will be taken as 1.5 based on the use of the specified minimum yield stress as the reference for the pipeline design.

The uncertainty in the induced stress is a linear function of the surface wave height. The surface wave height has an annual uncertainty (standard deviation of the logarithms) of $\sigma_H = 0.30$ for shallow water waves (Bea, 1997b). As discussed earlier, the wave height induced at the sea floor is a linear function of the height of the wave at the sea surface, the wave length (or wave period) and the density and shear modulus of the sea floor soils. It will be assumed that the uncertainty in the parameter ($\alpha^2 - k^2$) is equivalent to $\sigma_{(\alpha 2 - k2)} = 0.60$. The resultant uncertainty in the demand will be taken as $\sigma_S = 0.67$.

The uncertainty in the capacity will be taken as $\sigma_{S_u} = 0.10$. The total uncertainty is thus $\sigma = 0.68$. A 'conservative' value of $\sigma = 0.70$ will be used.

The design factors of safety to be used on the wave frequency induced maximum stresses are summarized in Table 5.1 for the different SSC for new and existing pipelines. The wave frequency induced stress would be multiplied times this factor of safety and the requirement would be that the specified minimum yield stress of the pipeline steel would be equal to or greater than this design stress.

Table 5.1 - Factors of Safety for Wave Frequency Induced Stresses for Pipeline Design and Requalifications for 100-year Hurricane Conditions

SSC	β new	FS new	β existing	FS existing
1	3.72	1.9	3.54	1.7
2	3.29	1.4	3.10	1.2
3	3.10	1.2	2.87	1.0

5.5.2 Sea Floor Displacements

The probability of failure of a pipeline associated with sea floor displacements can be expressed as:

$$Pf = P\ (Tm \geq Tu)$$

where Tm is the maximum tensile force induced in the pipeline by the displaced sea floor soils and Tu is the maximum tensile force that can be sustained by the pipeline without loss of containment (failure) or rupture. As developed in Section 4.3.2, the maximum tensile force (T) is a function of the soil force developed on the pipeline (w) and the slide width (L).

The uncertainties in the soil force are functions of the uncertainties in the soil strengths and the soil bearing factor. The uncertainty in the soil force is taken to be $\sigma_w = 0.40$.

The uncertainty in the slide width is taken to be $\sigma_L = 0.40$. The total uncertainty due to the slide width is twice this value (see formulation in Section 4.3.2) or $\sigma_L = 0.80$. The resultant uncertainty in the pipeline demand is thus $\sigma_{Tm} = 0.89$.

The uncertainty in the pipeline tensile capacity is taken as $\sigma_{Tu} = 0.10$. Thus, the total resultant uncertainty in the soil displacement pipeline demands and capacities is estimated to be $\sigma = 0.90$.

Again, based on the use of the minimum tensile stress as the reference for the design stress, a bias in the pipeline capacity of $B_{Tu} = 1.5$ is assumed. The demand is assumed to have a bias of $B_{Tm} = 1.0$

The Factor of Safety (FS) based on 100-year associated soil displacement conditions is:

$$FS_{99} = (B_{Tm} / B_{Tu}) \exp(\beta \sigma - 2.33 \sigma_{Tm})$$

Table 5.2 summarizes the Factors of Safety appropriate for the different SSC for new and existing pipelines.

Table 5.2 - Factors of Safety for Soil Displacements for Pipeline Design and Requalifications for 100-year Hurricane Conditions

SSC	β new	FS new	β existing	FS existing
1	3.72	2.4	3.54	2.0
2	3.29	1.6	3.10	1.4
3	3.10	1.4	2.87	1.1

5.6 Implications for Design and Requalification of Risers and Pipeline Connections

The experience in the Bay of Campeche during hurricane Roxanne and in past hurricanes that have affected pipelines and platforms near the Mississippi River Delta have adequately demonstrated the importance of risers and pipeline connections to the integrity of pipeline systems. The movements of the sea floor soils can induce tensions in the pipelines which may not be fully supported by the non-moving soils. Thus, large tensions can be transmitted via the pipeline to risers and connections. One could argue that estimates should be made of the largest reasonable tensions that could be transmitted to the risers and connections. These tensions could approach the tensile capacity of the pipeline itself. These generally equate to very large forces that may not be reasonably supported by either connections, risers, or in some cases the platforms.

The question to be raised is: "where should engineering unbalance the design of the pipeline, its connections, and its risers to allow damage to occur in the most accessible and repairable location?" The author has designed pipelines to have intentional 'weak points' or break-away couplings to prevent very high forces from being transmitted to the pipeline risers, the platforms, or to other parts of the pipeline system. these pipeline 'fuses' were intentionally located to facilitate detection of leaks, the recovery of the pipeline, and its subsequent repair (Bea, 1981). Such a philosophy should be developed for design of the pipelines, risers, and connections in the portions of the Bay of Campeche that are subjected to significant soil displacements.

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6.0 Operating Pressures Characteristics

6.1 Analyses of Pressure Data from Oil & Gas Pipelines

For this report, data from the inlet and outlet for four gas-pipelines and one oil-pipeline obtained from a pipeline operator and were analyzed for both normal operating conditions and shutdown conditions. The behavior for the different pipelines was very different in both characteristics during shutdown and the correlation between the pressure in inlet and outlet. The oil pipeline also performs significantly different from the gas pipelines. The measurement periods are either 12 or 24 hours, with measurements taken every 5 minutes.

6.1.1 Case 1 – Oil Pipeline

The oil pipeline studied is a 16-inch diameter pipeline with a design pressure of 184 bar, and the measurement period is 12 hours. The pressure at the outlet and inlet of the pipeline is presented in Figure 6.1 and Figure 6.2. The outlet and inlet pressures have almost the same profile but at very different levels. The ratio of inlet pressure to outlet pressure has a mean value of 13.5 during normal operating conditions with a coefficient of variation of 1.8%.

This changes dramatically during the shutdown and in the period after the shutdown where the pipeline is experiencing transients at a low pressure-level. The pipeline pressure has coefficients of variation of 3.7% and 4.05% during normal operating conditions at the inlet and the outlet of the pipeline respectively. Only at the start of the pipeline can one find pressures close to the defined design pressure. The mean ratio of operating pressure over design pressure is here 0.79. The coefficient of variation for this ratio will of course be the same as for the pressure itself, namely 3.7%. During the part of the startup period included in the data, the coefficients of variation for the operating pressures are 29.2% and 100.1% at the outlet and inlet respectively. The pipeline is shut down completely at the outlet in no more than 10 minutes, and no pressure increase is measured during this shutdown.

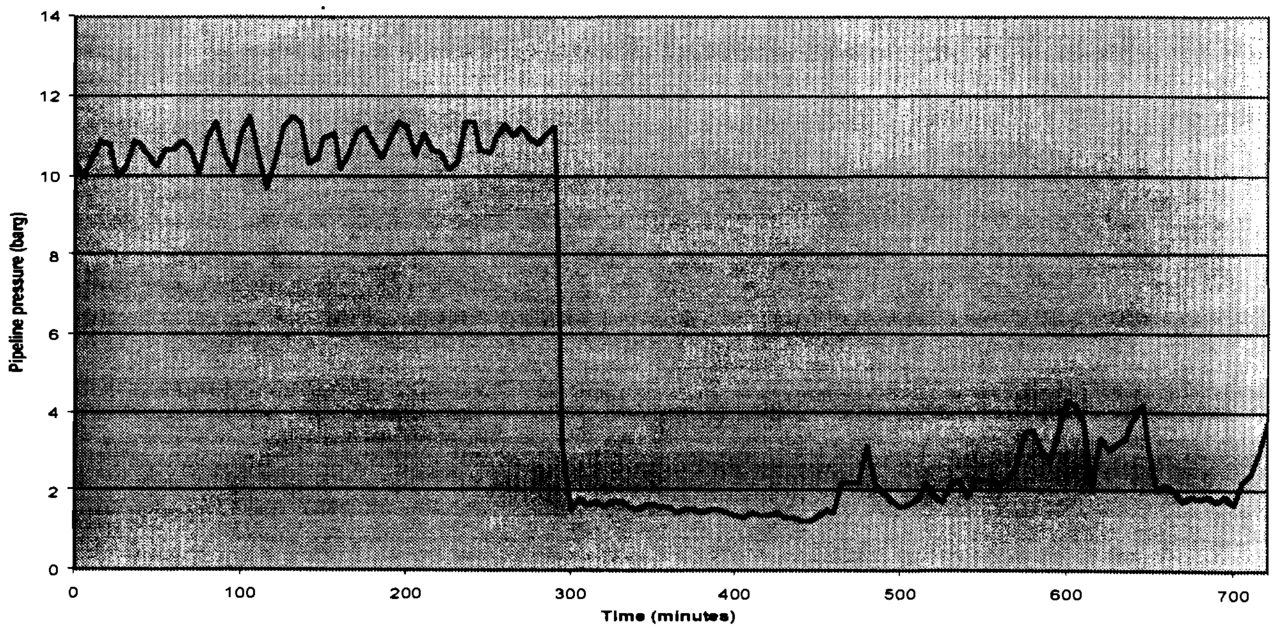

Figure 6.1 - Pressure at outlet of oil pipeline

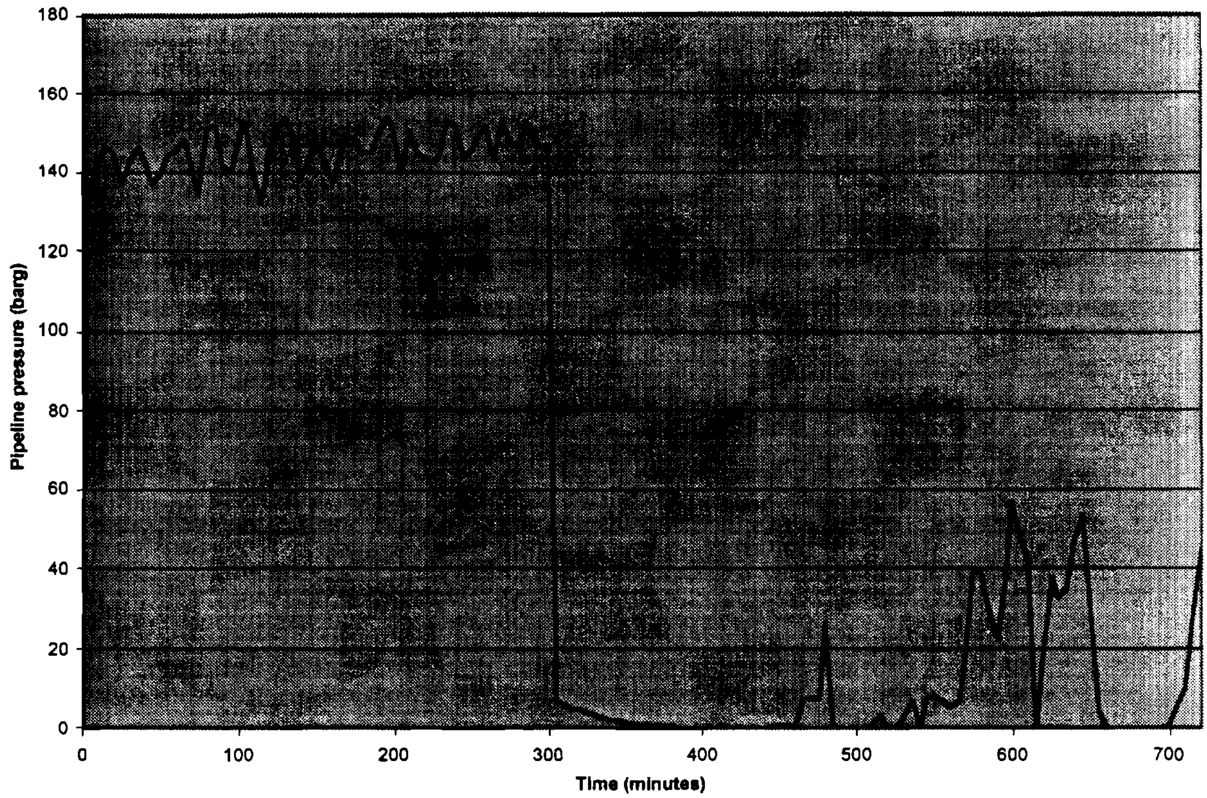

Figure 6.2 - Pressure at inlet of oil pipeline

6.1.2 Case 2 Gas Pipeline

This case is a 40-inch diameter gas pipeline with a design pressure of 156.8 bar. The measurement period is 24 hours. The pressure and flow at the outlet of the pipeline are presented in Figure 6.3, and the pressure at the inlet is presented in Figure 6.4.. During normal operating conditions, the ratio of inlet pressure to outlet pressure has a mean value of 1.7 with a coefficient of variation of 0.33%. The operating pressure has a coefficient of variation of 0.28% during normal operating conditions (before shutdown) at the outlet of the pipeline. At the inlet, the coefficient of variation in the operating pressure is 0.09% before the shutdown and 0.18% during the whole 24-hour period. At the inlet of the pipeline, the average ratio of operating pressure to design pressure is 0.93.

A partial shutdown was performed at the outlet of the pipeline during the 24-hour measuring period. The shutdown is impossible to detect at the inlet of the pipeline at the time of the shutdown, but about 4 hours later some transients can bee seen. Nothing in the flow indicates any other cause of the pressure increase, so it is possible that this is caused by the partial shutdown. Note that these transients are very small. A sharp transient at the outlet clearly indicates the shutdown with a sudden increase in the pressure from 86 to 92 bar (7% increase).

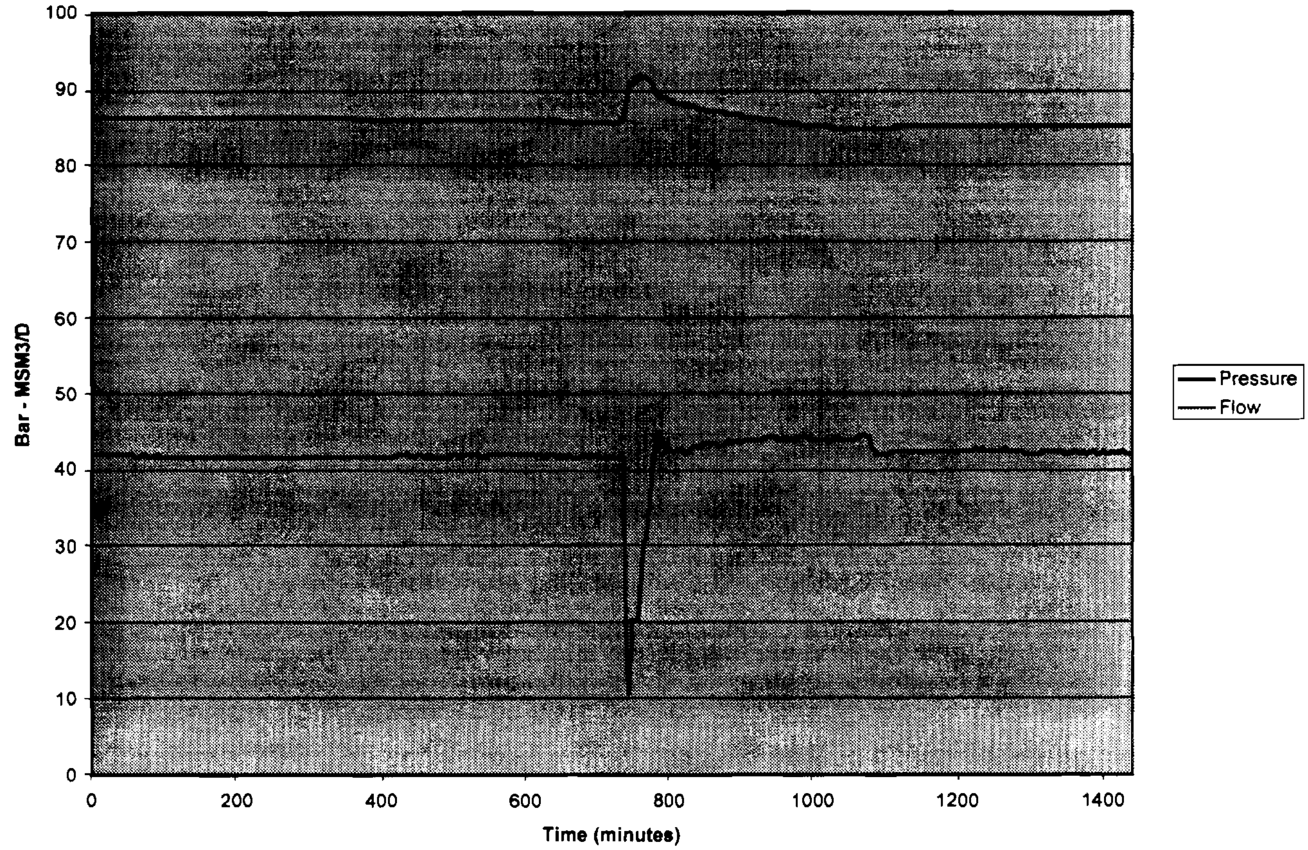

Figure 6.3 - Pressure and flow at outlet of gas pipeline

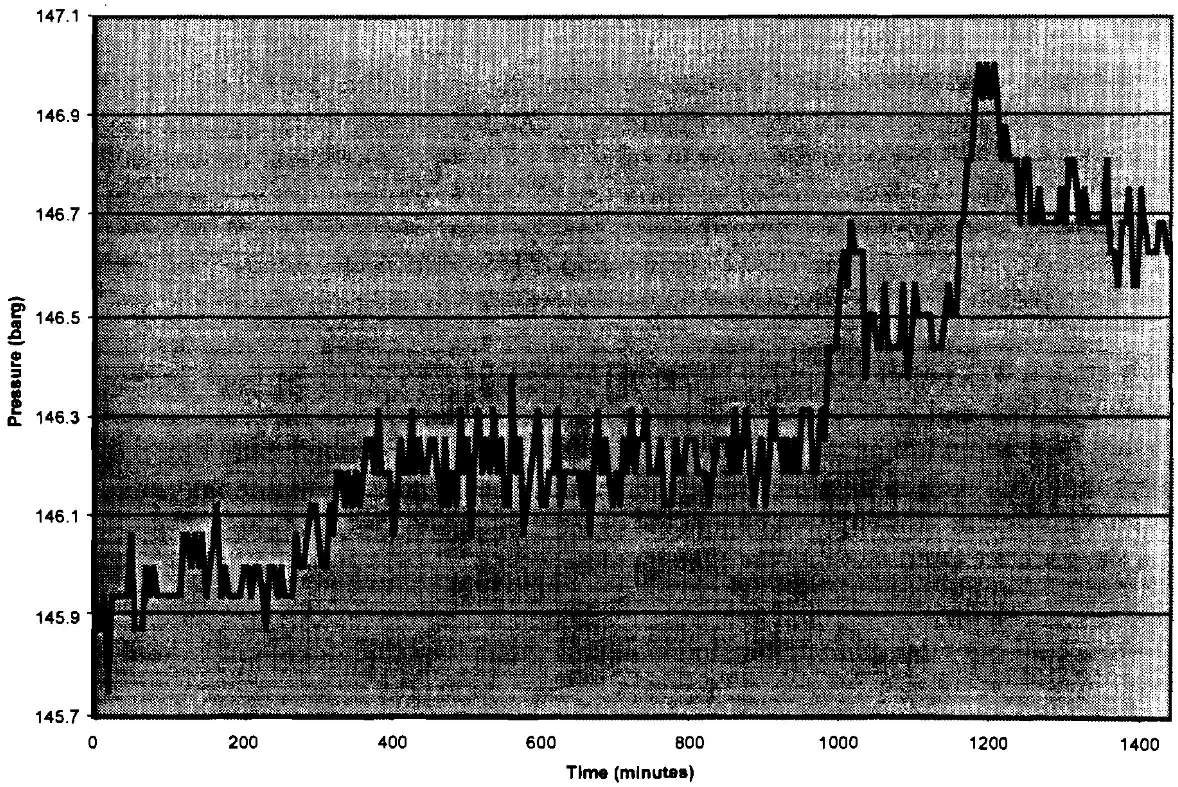

Figure 6.4 - Pressure at inlet of pipeline

6.1.3 Cases 3 and 4 – Gas Pipelines

These are also 40-inch diameter gas pipelines, but with a design pressure of 191 bar. The measurement period is 24 hours. In Figures 6.5 and 6.6, there is a clear correlation between the inlet and outlet pressure and a clear but not very large head loss from the inlet and the outlet. The mean ratio of inlet pressure to outlet pressure for Case 3 is 1.08 with a coefficient of variation of 2%. For case 4, the same numbers are 1.09 and 3.2%. All numbers are for the whole measurement period. For Case 3, in the period with normal operating pressure, the coefficients of variations were 0.13% and 0.11% for the inlet and the outlet respectively. For case 4, the same numbers were 0.17% and 0.18%. The shutdown is performed at the inlet of the pipeline, and there is a 30-40 minute time lag for the pressure reduction at the outlet of the pipeline. No increase in the pipeline pressure is experienced as a result of the shutdown.

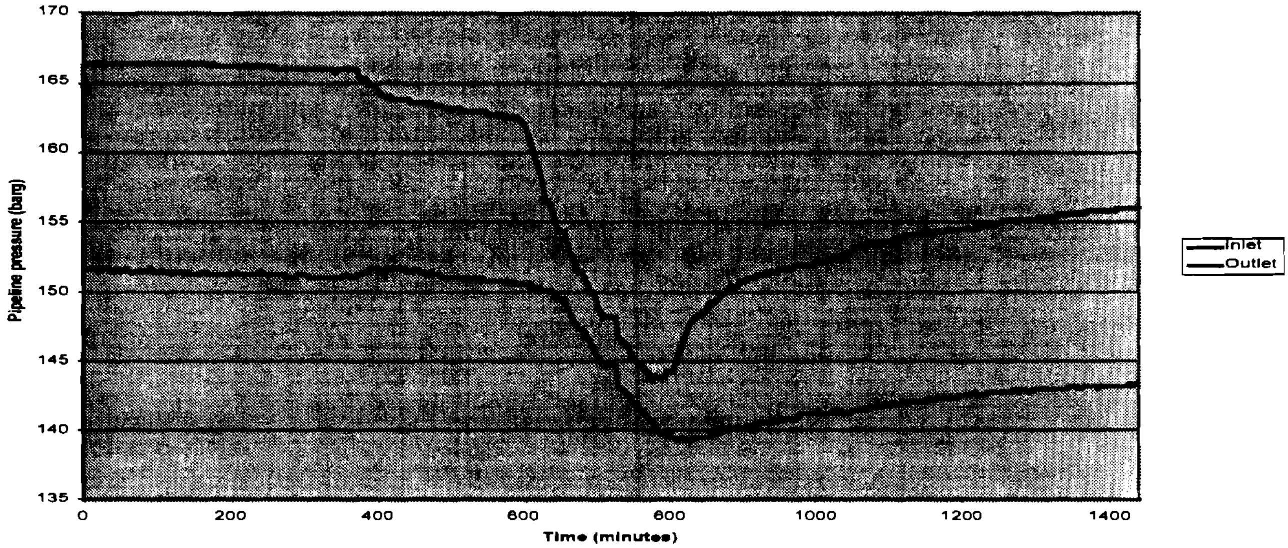

Figure 6.5 - Pressure at inlet and outlet of gas pipeline, Case 3

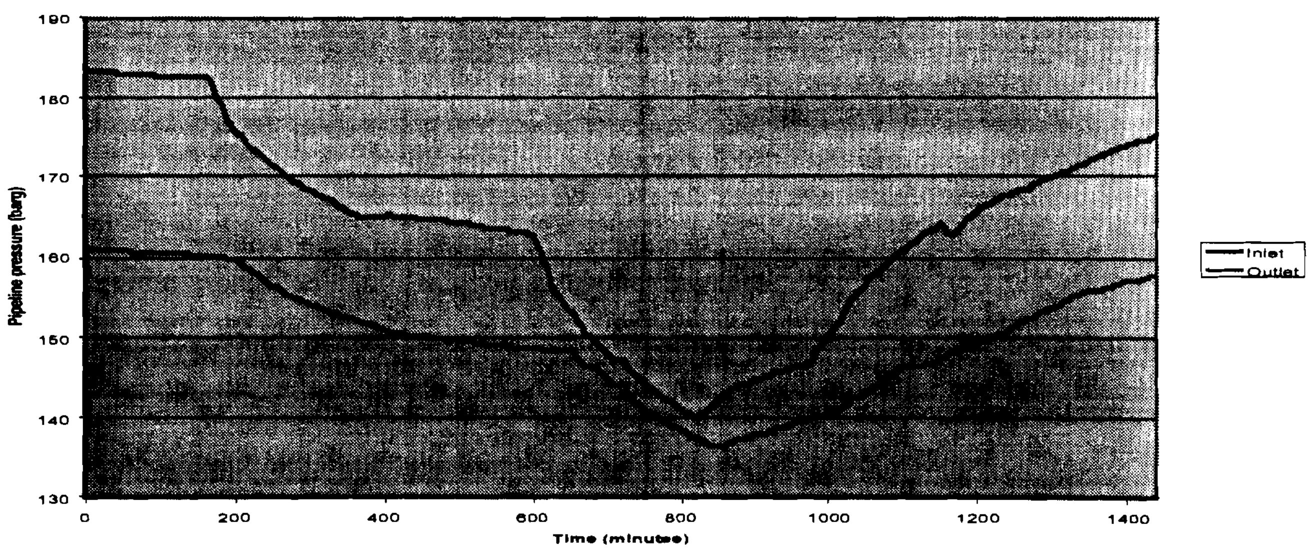

Figure 6.6 - Pressure at inlet and outlet of gas pipeline, Case 4

6.1.4 Case 5 – Gas Pipeline

The pipeline is a 16-inch gas-pipeline with a design pressure of 190 bar. The measurement period is 12 hours. There is little correlation between the inlet and outlet pressures. As the only case studied here, the inlet pressure drops below the outlet pressure. A peak is found in the inlet pressure, where the peak pressure is approximately 6% higher than the previous and following pressures. The coefficients of variations in the normal operating period are 0.07% and 0.11% for the inlet and outlet respectively.

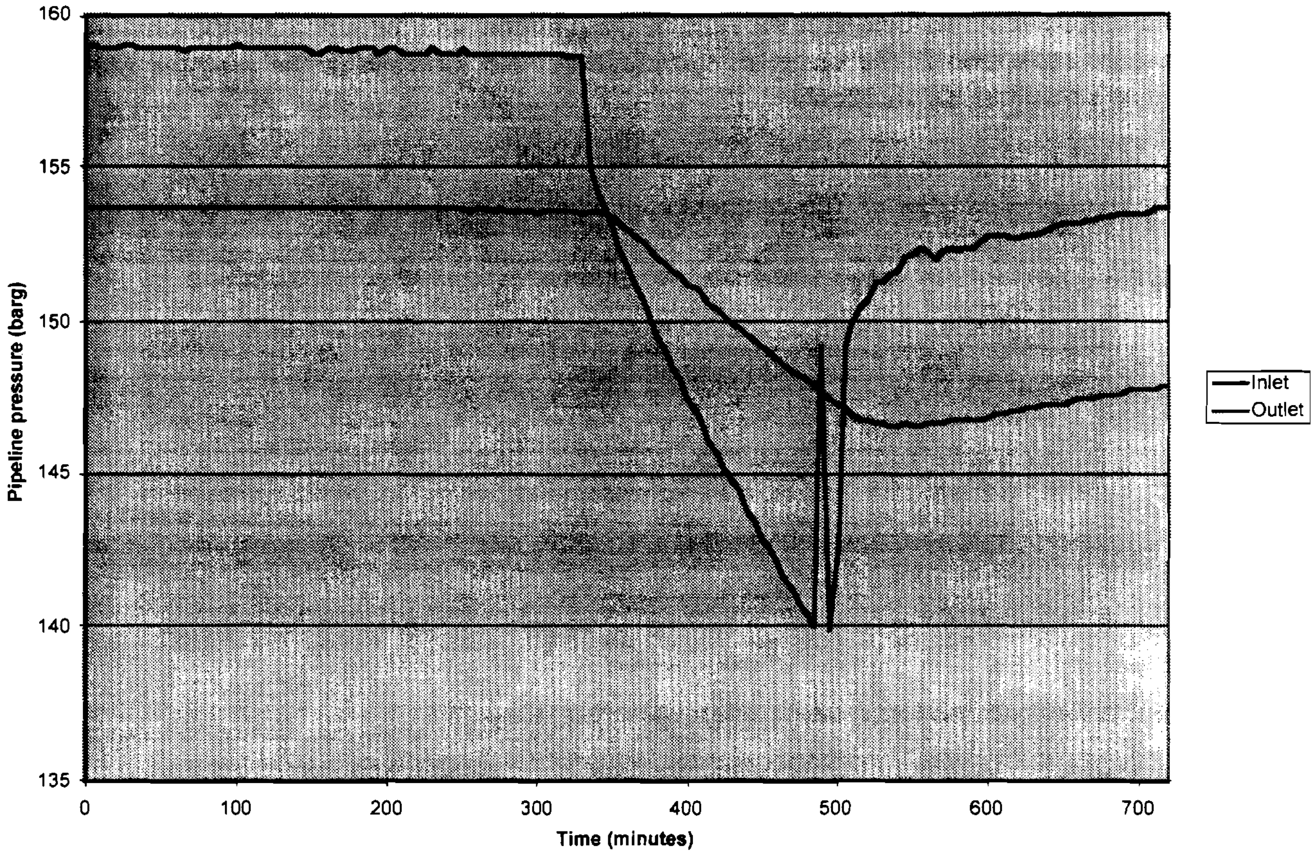

Figure 6.7 - Pressure at inlet and outlet of gas pipeline

6.2 Analytical Approach to Predict Peak Pressures

In the case of sudden shutdowns of a pipeline, the well-described phenomenon called water hammer will result in sudden increases in the pipeline pressure. Several aspects of the water hammer effect are interesting with respect to requalification of a pipeline. The most interesting point is basically how much the pressure will increase during a sudden shutdown. This pressure increase is given as (Franzini, Finnemore, 1997):

$$\Delta p = -\rho \cdot c_p \cdot \Delta V$$

where:

ρ = mass density of liquid or gas in pipeline

c_p = celerity, = $\sqrt{\dfrac{E_V}{\rho}}$

E_V = bulk modulus of liquid in pipeline (225,000 psi for oil and 145,000 psi for gasoline)

ΔV = velocity change of flow in pipeline

Use of the celerity of the fluid in this formulation is limited to the case where the pipeline is inelastic. For thin-walled pipelines, this should be taken into consideration, and c_p in the equation for maximum surge pressure should be replaced with the wave propagating velocity a, given by (Gulf Publishing Co., 1979):

$$a = \sqrt{\frac{E_V}{\rho \cdot [1 + E_V \cdot D/E \cdot t]}}$$

This is for the case of a rapid closure of the pipeline. A rapid closure is defined by the closure time t_c:

$$t_c < T_r$$

where:

T_r = $2 \cdot L/c_p$

L = length of the pipeline

For the case of a slower closure of the pipeline, the pressure increase is defined by (Franzini, Finnemore, 1997):

$$\Delta p_{slow} = 2 \cdot L \cdot \rho \cdot \Delta V/t_c$$

Since the velocity of the fluid in the pipeline is a nonlinear function of the port closure percentage, closure of the first three quarters of the pipeline has a small effect on the fluid velocity (Gulf Publishing Co., 1979). From that point on, the velocity is greatly affected, and the maximum pressure increase can be calculated by:

$$\Delta p_{slow} = \Delta p/(t_{c1/4}/2 \cdot L)^{0.5}$$

where:

$t_{c1/4}$ = Time to close the last quarter of the valve

As a rule of thumb, the following equation can be used to estimate the maximum increase in pressure (in pounds per square inch) (Marks, 1980):

$$\Delta p \approx 0.8 \cdot \rho \cdot g \cdot \Delta V$$

g is the acceleration of gravity (ft / s^2). Figure 6.8 summarizes the results from this approximation. The maximum surge pressure due to a rapid shut-in is a maximum of 1.6 times the flowing pressure.

This is an impulse dynamic loading that will be a function of the duration of the pressure pulse to the natural period of the pipeline.

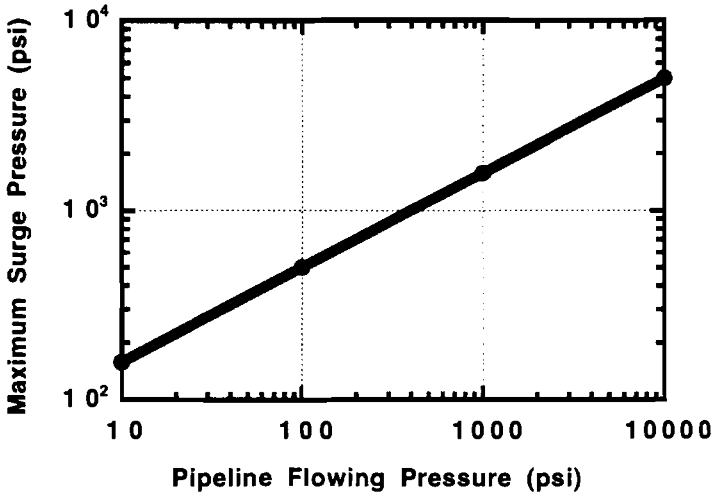

Figure 6.8 – Pressure Due to Rapid Shut-In of Flowing Fluid in Pipeline

It is clear that this transient in pressure will decrease from the shut-off point throughout the pipeline. This decrease in pressure is due to damping in the pipeline. This damping is caused by friction, compressibility of the fluid and elasticity of the pipeline. For short pipelines, the decrease in the pressure transient would be negligible. For a very long pipeline, the pressure transient will have vanished before it reaches the other end of the pipeline. It is clear that this transient will decrease as it travels through the pipelines, but none of the pressure data available for shutdowns at the pipeline was able to give information on how the transient will decrease. The oil pipeline case studied has a shutdown at the outlet of the pipeline. It is clear that pressure relief systems have eliminated the transient expected in this case. For case studies of the decrease of the transient, data from cases where the pressure relief system is failing must be studied. In the literature, this decrease is widely recognized, but no analytical model has been developed.

This calculated increase in pressure occurs at the shutdown point of the pipeline, e.g. the outlet. In this part of the pipeline, the pressure may have dropped from its initial level at the inlet because of friction in the pipeline. Again, for a short pipeline, the pressure drop will be negligible. For a long pipeline, the pressure drop is dramatic (see Case 1). The transient will therefore have less influence at the outlet than they will have at the inlet of the pipeline.

6.4 Discussion

The analyzed data give some very interesting information about the pressure characteristics in a pipeline during shutdown and startup of the pipeline. For the pipelines analyzed there is a significant head loss through the length of the pipeline. This means that the pipeline pressure is way below the design pressure at the end of the pipeline, and that transients or peaks in the pressure will most likely not be a problem at this end of the pipeline. The partial shutdown at the outlet in case 2 indicates a peak pressure at this location of 1.07 times the normal pressure before the shutdown. If we transfer this case to all the other cases we can see that none of the outlet pressures will be close to encountering the design pressure. Case 2 also shows that the shutdown at the outlet of the pipeline does not affect the pressure at the inlet at all. This is however a very long pipeline, and for shorter pipelines the transient may transfer to the inlet with the consequences that will have. The cases studied indicate that for shutdowns at the inlet of the pipeline, pressure transients will not be a problem. No pressure increase is measured in any of the cases where the shutdown is performed at the inlet of the pipeline.

Case 1 with the oil-pipeline, where transients are much larger than for gas pipeline, the head loss from the inlet to the outlet is much more significant. For normal operating conditions, the factor of inlet pressure over outlet pressure has a mean value of 13.5. Very large transients must then be present if the design pressure is to be exceeded at the outlet of the pipeline. There is however some fluctuations appearing after startup of the pipeline, and these are analyzed. For the oil-pipeline this seems to be where problems may occur. At the inlet, the coefficient of variation is 100.1% during the part of the startup period included in the measurements. If these fluctuations continue as the pipeline pressure rise, problems may very well occur at the startup.

Determination of probable maximum pressure during a pipeline's lifetime is as mentioned not trivial. As a background for a calculation like this, normal operating pressures with variations can be measured. Pressure in situations like shutdowns and startups should also be used because of the increased risk of peaks in the operating pressure during such operations. Where do we meet the problems in this approach? Pipelines will logically have some kind of pressure relief system that will be activated at a certain pressure level, and it is obvious that this level is set some place below the failure pressure of the pipeline. The probability of occurrence of such a pressure will then be dependent on the probability of failure of the pressure relief system, and can hence not be described by the lower pressure levels at all. These two situations will be more or less independent, and a good quantification of the probability of occurrence of the burst pressure will be very difficult to obtain. In order to do this, a large amount of data on situations where pressure relief systems fail will have to be collected, something that will be both very time demanding and expensive.

6.5 Conclusions and Recommendations

The studies of pressure data developed during this project gave some valuable information on when and where problems can be expected to occur. An increase in pipeline pressure of 7% appeared in one of the cases where a shutdown was done at the outlet of the pipeline. No pressure increase was measured at the oil pipeline where the shutdown also was done at the outlet. Large fluctuations in the pressure during startup of the pipeline may cause problems, but the available data studied in this thesis did not give enough information on this. The characteristics of operating pressure in pipelines still need a lot of work, both on normal operating conditions and during shutdowns and startups. Some indications of variations and peaks in the operating pressure were obtained, but a lot more data is needed in order to obtain good quantitative results on this problem.

6.6 References

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7.0 Capacity of Corroded Pipelines

7.1 MMS Pipeline Failure Database Analysis

As a part of the Unites States Department of the Interior, the Minerals Management Service, MMS, collects basic pipeline information as well as pipeline damage data and failure data for the United States offshore pipeline systems located on the Offshore Continental Shelf (OCS) lands. The following analyses are based on MMS database information available as of April 1999. This information can be accessed from the MMS web site:

http://www.gomr.mms.gov/homepg/pubinfo/freeasci/freedesc.html

7.1.1 Causes of Failure

Through analysis of the MMS Database, eight basic causes of failure (loss of reportable containment) can be identified. These causes of failure include corrosion, impact, material flaws, natural hazards (environmental attack), structural damage, anchor trawling, and construction damage. Figure 7.1 shows the distribution of these causes of failure.

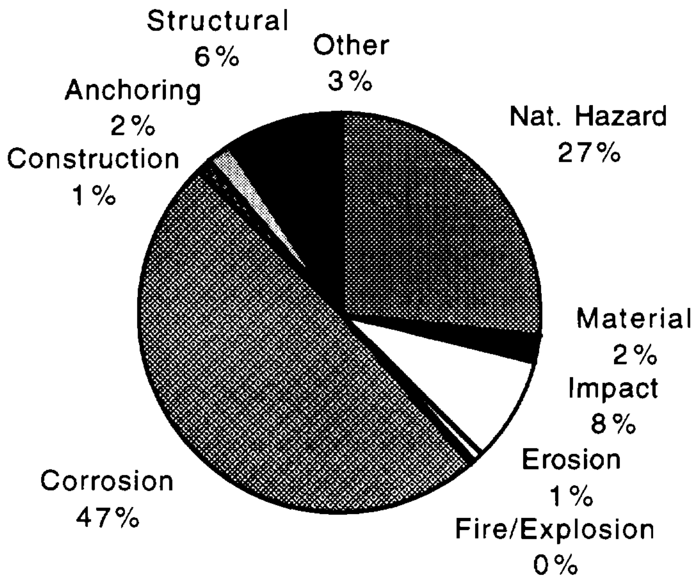


Figure 7.1 – Causes of failure of OCS pipelines

The primary cause of failure was determined to be corrosion. Corrosion resulted in 47% of the total failures. This conclusion is validated by previous studies which also find corrosion to be the leading cause of pipeline failures. The second leading cause is that of natural hazards (wave and current loadings, mudslides). Third party activities are responsible for about 14 % of the pipeline failures.

7.1.2 Corrosion Failures of Oil and Gas Pipelines

Of the total failures, 76% of the failures were in oil pipelines and 24% of failures were in gas pipelines. Therefore, it can be concluded that oil pipelines have a much higher failure rate than do gas pipelines. However, when looking at the respective causes of failure for oil and gas pipelines, many of the causes of failure occur in the same proportion. Corrosion falls into this category. For oil pipelines, corrosion accounted for 48% of the failures, while in gas pipelines, the percentage was 46%.

The MMS database uses production codes to identify what type of product is carried in each pipeline system. The production codes for gas pipeline systems are summarized in Table 7.1.

Table 7.1 – Gas production code summary

Production Code	Definition
Gas	gas transported after first processing
Lift	gas lift
INJ	gas injection
G/C	gas/condensate service
FLG	flare gas

Oil is divided into two categories: Oil and BLKO. Oil is defined as oil transported after first processing, and BLKO is defined as full well stream production from oil wells prior to processing.

One thousand random data points were analyzed to determine the proportion of constituents in both oil and gas pipeline systems. Out of these 1000 pipeline systems, 252 were damaged. Table 7.2 summarizes the products that these 252 damaged pipeline systems carry:

Table 7.2 – Products carried by damaged pipelines

Product	# of pipelines carrying product
OIL	41
BLKO	62
TEST	4
Gas/Oil	38
GAS	30
BLKG	8
INJ	1
Product	**# of pipelines carrying product**
Gas/Condensate	3
FLG	2
Water	5
No data	25
Lift	33

The gas production codes including Lift, Gas, FLG, G/C, INJ, and BLKG were then separated from the table. A total of 77 pipeline systems that failed contained gas in one of these forms. Figure 7.2 shows the distribution of failed pipelines in terms of gas production codes.

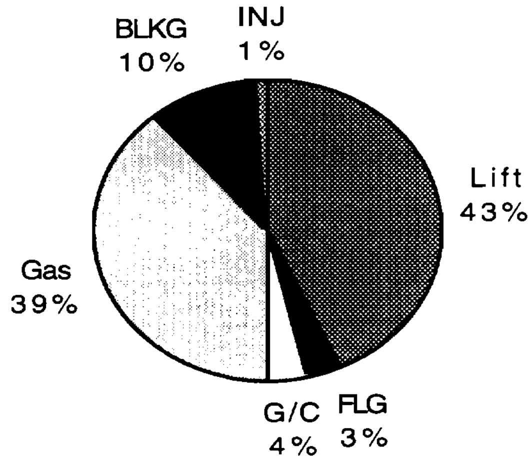


Figure 7.2 – Types of gas production associated with failed pipelines

The primary production code included in gas is Lift at 43%, which is closely followed by Gas (after first processing) at 39%. The next important step consisted of determining the corrosivity of the these major constituents including Gas, Lift, and BLKG.

The rate of corrosivity of each of the major constituents was determined by referring back to the 1000 random pipeline systems. Of the 252 damaged pipelines from the random sampling, 77 transported one of the gas products listed in Table 7.1. Table 7.3 summarizes the percentage of failures caused by corrosion for each of the major gas production codes, while Table 7.4 summarizes the percentage of failures caused by corrosion for both of the oil production codes.

Table 7.3 – Gas production corrosion failures

Gas Product	Corrosion-causing failure (%)
Gas	53%
Lift	60%
BLKG	50%

Table 7.4 – Oil production corrosion failures

Oil Product	Failures Caused by Corrosion(%)
Oil	58%
BLKO	61%

It appears from the data summarized in Table 7.3 that there is no significant difference between the three major components: Gas, Lift, BLKG. As indicated by the data summarized in Table 7.4, the same is true of Oil and BLKO, the two oil components. The trend between oil and gas appears to be the same as that indicated earlier: gas and oil have very similar corrosion failure rates with gas being slightly lower.

7.2 Effects of Corrosion Area on Burst Pressure Capacities

The Level 2 formulation developed during the first phase of the RAM PIPE REQUAL project to evaluate the median burst pressure capacities (Pb) of corroded pipelines is:

$$Pb = 2.4\ t\ SMTS\ /\ D\ SCF$$

$$SCF = 1 + 2\ (d/R)^{0.5}$$

where t is the pipeline original nominal wall thickness, SMTS is the specified minimum tensile yield strength (-3 σ from mean SMTS), D is the nominal mean pipeline diameter (= Dn – t), and SCF is the stress concentration factor that is due to the most severe corrosion defect in the pipeline. The SCF is a function of the maximum depth of corrosion, d, and the pipeline mean radius, R (= D/2).

Note that this formulation does not include any explicit recognition of the length (along the pipeline axis) and width (normal to the pipeline axis) characteristics of the corrosion feature. There is an implicit recognition of the width contained in the formulation for the SCF; the radius of the corrosion feature is characterized as the mean pipeline radius. Based on an analysis of the 151 physical burst tests that were assembled during the first phase of this project (summarized in Appendix A of Report 1), this formulation produced an unbiased estimate of the burst capacities (median ratio of measured to predicted burst pressure was unity) with a Coefficient of Variation of 22 %.

The corroded pipeline burst test database was analyzed to determine the effects of the corrosion lengths and areas on the physical test burst pressures. The results are summarized in Figures 7.3 and 7.4. The measured burst pressures have been normalized by the burst pressures for the uncorroded pipelines based on the nominal SMTS hoop stress formulation:

$$Pb = 2\ t\ SMTS\ /\ D$$

The data also were analyzed based on the ASME B31 G formulation Folias bulging factor that is a function of the square root of the square of the corrosion length divided by the product of the pipeline diameter and thickness (sq rt L^2 / D t). The results are summarized in Figure 7.5.

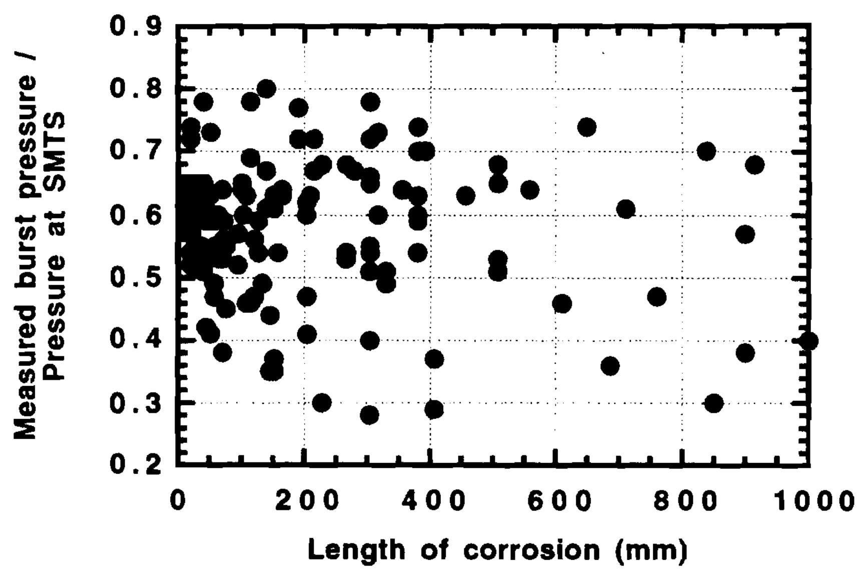


Figure 7.3 – Effects of corrosion length (along pipe axis) on measured burst pressures

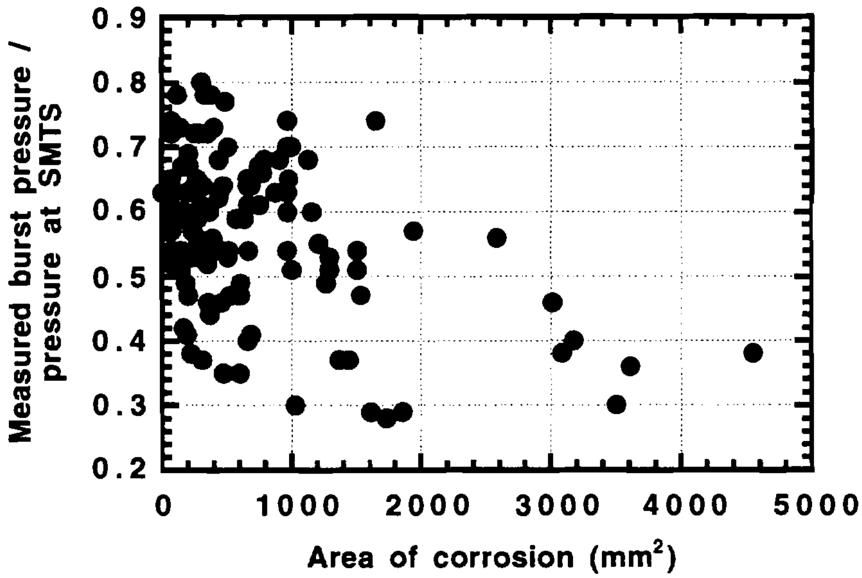


Figure 7.4 – Effects of corrosion area on measured burst pressures

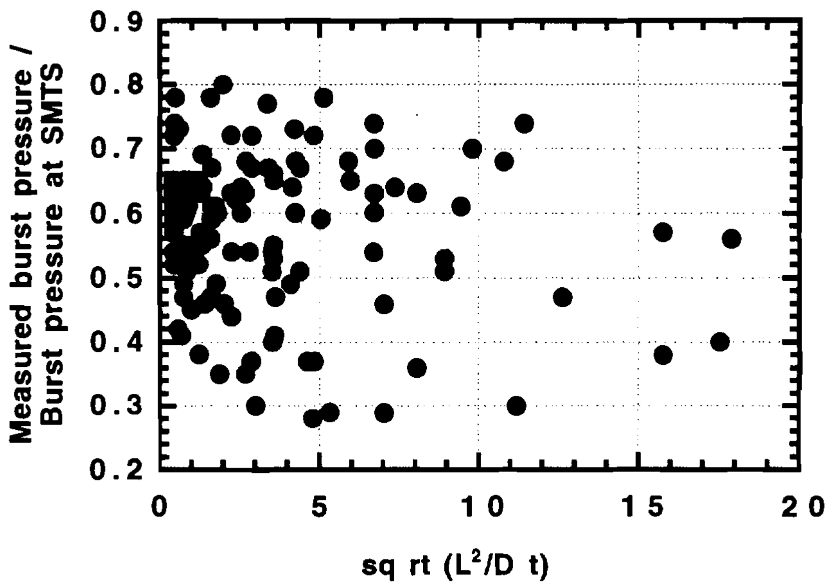


Figure 7.5 – Effects of Folias bulging factor function on measured burst pressures

Surprisingly, the physical test data do not indicate that there is any obvious dependency of the measured burst pressures on the length or area characteristics of the corrosion.

These results seem to be very strange because most of the accepted formulations to determine the burst pressure capacities of corroded pipelines involve very complex formulations that are based on the area – width – length (plan) characteristics of the corrosion features. The work published by Bai, Xu, and Bea (1997) involved development of a formulation that would improve on the B31 G formulation. This complex formulation resulted in a mean Bias (measured pressure / predicted pressure) of 1.1 with a Coefficient of Variation of the Bias of 18 %. Given the results summarized in Figures 7.3 – 7.5, one can begin to understand why the very sophisticated formulation is only able to reduce the prediction variability by 18%.

7.3 Burst Pressure of Corroded Pipelines

Experiences with pipelines and risers in use today in both the North Sea and the Gulf of Mexico indicates that corrosion is probably the most important operating hazard to the integrity of these. For pipelines, the primary concern is internal corrosion, when for risers, external corrosion is considered the primary hazard. In the re-qualification of pipelines, there are three main approaches to evaluating corrosion effects:

- Use of instrumentation and inspections to detect and quantify corrosion defects
- Use of corrosion coupons to quantify corrosion rates
- Use of indirect indicators of corrosivity and corrosion rates

Corrosion is an extremely complex process and is dependent on many variables concerning both the pipeline, what is transported in the pipeline and what is surrounding the exterior of the pipeline. The process will over time degrade the properties of the pipeline such as thickness and strength. The primary parameters determining the corrosion rate are (Bea, Xu, 1999):

- Temperature
- Water composition
- Product composition
- Operational parameters such as flow rates, regime, pressures and oil-water wetting
- Steel quality and weld properties including both macro and micro structure, alloying elements and consumables
- Sulphate reducing bacterial count and types
- Deposits and coatings on the steel surfaces
- Steel cracking
- Erosion due to the transportation of solids
- Stray currents associated with electrical operating equipment and other metals that can come into or are placed in contact with the pipeline

All these parameters can be expected to change during the life of the pipeline because the sources of oil, water and gas transported through the pipeline as well as the external environmental and operational conditions are continuously changing.

7.3.1 ANSI/ASME B31G

Several different methods have been used for strength assessment of both corroded and non-corroded pipelines. Until now, the most commonly used criterion for corrosion damage assessment of pipelines has been the ANSI/ASME B31G criterion. As pointed out in several recent publications, this criterion is not in harmony with modern design philosophies. The ANSI/ASME B31G criterion is based on the NG-18 equation adjusted to account for available experimental data. The equation is:

$$P = \frac{\sigma_{flow} \cdot 2 \cdot t}{D} \cdot \left[\frac{1 - \dfrac{A}{A_0}}{1 - \dfrac{A}{A_0} \cdot \dfrac{1}{M}} \right]$$

where:

A_0	$= d \cdot t$
M	= Folias bulging factor, accounting for effect of stress concentration at notch
P	= Failure pressure
σ_{flow}	= Flow stress
D	= Pipe outer diameter
A	= Projected corroded area
t	= Pipe nominal wall thickness
d	= Maximum corrosion depth

Here, the projected corrosion area is assumed to be parabolic, and hence the projected corroded area is $2/3 \cdot d \cdot t$. For long defects, this assumption will over-predict the pipeline's capacity, and a rectangular shape is assumed. The flow stress has an upper envelope 10% higher than the specified minimum yield stress (SMYS). The B31G burst equation for safe maximum pressure P' is then defined as:

$$P' = 1.1 \cdot P \cdot \left[\frac{1 - \frac{2}{3} \cdot \frac{d}{t}}{1 - \frac{2}{3} \cdot \frac{d}{t} \cdot \frac{1}{M}} \right] \qquad \text{for } \sqrt{0.8} \cdot X \leq 4.0$$

$$P' = 1.1 \cdot P \cdot \left[1 - \frac{d}{t} \right] \qquad \text{for } \sqrt{0.8} \cdot X > 4.0$$

where:

$$P = \frac{SMYS \cdot 2 \cdot t}{D} \cdot F$$

$$M = \sqrt{1 + 0.8 \cdot X^2} \qquad \text{Folias bulging factor}$$

$$X = \frac{L}{\sqrt{D \cdot t}} \qquad \text{Characteristic corrosion length}$$

and F is the design factor usually equal to 0.72. A limitation to this is that P' must not exceed P which is the maximum allowable design pressure for a non-corroded pipe. Corrosion above 80% of the wall thickness is not accepted, and corrosion less then 10% needs no further evaluation.

Several modifications have been proposed to improve this criterion in order to give better predictions of the actual failure pressure. These are mainly changes of the equation parameters such as the flow stress σ_{flow}, the bulging factor M and the definition of the projected corrosion area A. The disadvantage with modifying one or more of these parameters in the B31G equation based on test results, is that it will most likely result in a negative effect for other design cases, e.g. other geometric and corrosion configurations.

7.3.2 DNV Recommended Practice

The new DNV guidelines are still under development, but the version issued in December 1998 will be treated here (DNV, 1998). The guidelines provide recommended practice for assessing corrosion defects subjected to:

- Internal pressure loading only
- Internal pressure loading combined with longitudinal compressive stresses

The guidelines describe two alternative approaches to the assessment of corrosion damage, Load and Resistance Factor Design (LRFD) and Allowable Stress Design (ASD). The LRFD approach is based on the safety philosophy in the DNV Offshore Standard OS-F101, Submarine Pipeline Systems, which is reviewed earlier in this thesis. The following types of defects can be assessed using these guidelines:

- Internal corrosion of base material
- External corrosion of base material
- Corrosion in seam welds
- Corrosion in girth welds
- Colonies of interacting corrosion defects
- Metal loss due to grind repairs

Internal pressure loading case can be considered for the following defects:

- A single defect, which does not interact with one ore more neighboring defects. The failure pressure of a single defect is independent of other defects in the pipeline.
- Interacting defects, which means defects interacting in either an axial or a circumferential direction. The failure pressure of interacting defects is lower than if the interacting defects were considered as single.
- A complex shaped defect, which is a defect that results from combining colonies of interacting defects, or a single defect for which a profile is available.

Internal pressure combined with longitudinal compressive stresses can only be considered for single defects. The only failure mode considered is plastic collapse, and the guidelines are not recommended for applications where fracture is likely to occur.

7.3.2.1 Load and Resistance Factor Design

As mentioned, this approach is based on the safety philosophy in the DNV Offshore Standard OS-F101, Submarine Pipeline systems. Here, partial safety factors are given for two general inspection methods based on relative or absolute measurements, four different levels of inspection accuracy and three different reliability levels corresponding to the Safety Class classification of DNV OS-F101. The following safety classes are considered:

Safety class	Indicating a target annual failure probability of:
High	$< 10^{-5}$
Normal	$< 10^{-4}$
Low	$< 10^{-3}$

Safety class High is used for risers and parts of the pipeline close to platforms or in areas with frequent human activity. Safety class Normal is used for oil and gas pipelines where no frequent human activity is anticipated. Safety class Low can be considered for e.g. water pipelines.

The partial safety factors will depend on the inspection sizing accuracy, which is given relative to the wall thickness and for a specified confidence level. The confidence level is the portion of the measurements that will fall within a given sizing accuracy. The rules are assuming a normal distribution of d/t, and then the standard deviations of d/t can be estimated:

Relative sizing accuracy	Confidence level	
	80%	90%
Exact	StD[d/t] = 0.00	StD[d/t] = 0.00
± 5% of t	StD[d/t] = 0.04	StD[d/t] = 0.03
± 10% of t	StD[d/t] = 0.08	StD[d/t] = 0.06
± 20% of t	StD[d/t] = 0.16	StD[d/t] = 0.12

The partial safety factors are then given as functions of the sizing accuracy for both inspection methods. The two partial safety factors and corresponding fractile levels for the characteristic values used are:

γ_m	= Partial safety factor for model prediction
γ_d	= Partial safety factor for corrosion depth
ε_d	= Factor defining a fractile value for the corrosion depth
StD[d/t]	= Standard deviation of the measured d/t ratio

The values of the partial safety factors also depend on the material quality level. The rules state that the material quality level is to be taken as II or III unless it can be documented that the material is of quality level I. The partial safety factors are also influenced by the method of depth measurement used, either relative depth measurements or absolute depth measurements. For relative depth measurements, the values for γ_m and γ_d are given in Table 7.5 and Table 7.6:

Table 7.5 Partial safety factors for model prediction

Material quality level	Safety class		
	Low	Normal	High
II and III	$\gamma_m = 0.79$	$\gamma_m = 0.74$	$\gamma_m = 0.70$
I	$\gamma_m = 0.82$	$\gamma_m = 0.77$	$\gamma_m = 0.73$

Table 7.6 – Partial safety factors for corrosion depth

Inspection sizing accuracy, StD[d/t]	ε_d	Safety class		
		Low	Normal	High
0.00 (exact)	0.0	$\gamma_d = 1.00$	$\gamma_d = 1.00$	$\gamma_d = 1.00$
0.04	0.0	$\gamma_d = 1.16$	$\gamma_d = 1.16$	$\gamma_d = 1.16$
0.08	1.0	$\gamma_d = 1.20$	$\gamma_d = 1.28$	$\gamma_d = 1.32$
0.16	2.0	$\gamma_d = 1.20$	$\gamma_d = 1.38$	$\gamma_d = 1.58$

For absolute depth measurements, the following values are used for γ_m:

Table 7.7 – Partial safety factors for model prediction

Material quality level	Safety class		
	Low	Normal	High
II and III	$\gamma_m = 0.82$	$\gamma_m = 0.77$	$\gamma_m = 0.72$
I	$\gamma_m = 0.85$	$\gamma_m = 0.80$	$\gamma_m = 0.75$

The values of γ_m and ε_d are the same as those for relative depth measurements. For circumferential corrosion defects, the following values apply for γ_{mc} and η:

Table 7.8 – Partial safety factors for model prediction

Material quality level	Safety class		
	Low	Normal	High
II and III	$\gamma_{mc} = 0.81$	$\gamma_{mc} = 0.76$	$\gamma_{mc} = 0.71$
I	$\gamma_{mc} = 0.85$	$\gamma_{mc} = 0.80$	$\gamma_{mc} = 0.75$

Table 7.9 – Partial safety factors for corrosion depths

Material quality level	Safety class		
	Low	Normal	High
II and III	$\eta = 0.96$	$\eta = 0.87$	$\eta = 0.77$
I	$\eta = 1.00$	$\eta = 0.90$	$\eta = 0.80$

The usage factors for longitudinal stress are given in Table 7.10:

Table 7.10 – Longitudinal stress usage factors

Safety class	Usage factor ξ
Low	0.90
Normal	0.85
High	0.80

For a pipeline with a large number of corrosion defects, the system effects must be accounted for when determining the reliability level of the pipeline. The rules suggest adding the failure probability of each defect as a conservative way to assess the system effect.

The safety factors are then used with a number of equations to assess the allowable pressure in corroded pipelines for various types of defects. The guidelines are defining assessment of single defects, interacting defects and complex shaped defects.

7.3.2.2 Single defect

A defect is in the DNV guidelines treated as a single defect if any of the following conditions are specified:

1. The depth of the defect, $\gamma_d(d/t)^*$ is less than 20%
2. The circumferential angular spacing between adjacent defects, ϕ, is larger than:

$$\phi > 360 \cdot \frac{3}{\pi} \cdot \sqrt{\frac{t}{D}} \qquad \text{(degrees)}$$

3. The axial spacing between adjacent defects , s, is larger than:

$$s > 2.0 \cdot \sqrt{D \cdot t}$$

If the pipeline is subject to internal pressure loading only, the allowable pressure is given by the following equation:

$$p_{corr} = \gamma_m \cdot \frac{2 \cdot t \cdot SMTS}{D - t} \cdot \frac{[1 - \gamma_d \cdot (d/t)^*]}{\left[1 - \frac{\gamma_d \cdot (d/t)^*}{Q}\right]}$$

where:

$$Q = \sqrt{1 + 0.31 \cdot \left(\frac{1}{\sqrt{D \cdot t}}\right)^2}$$

$$(d/t)^* = (d/t)_{meas} + \varepsilon_d \cdot StD[d/t]$$

If $\gamma_d \cdot (d/t)^* \geq 1$ then $p_{corr} = 0$, and p_{corr} is not allowed to exceed p_{mao}. The rules also state that measured defects depths exceeding 85% are not accepted. For longitudinal corrosion defects with internal pressure and superimposed longitudinal compressive stresses, the following method applies:

Step 1 Determine longitudinal stress from external loads and calculate the nominal longitudinal elastic stresses in the pipe, based on the nominal wall thickness:

$$\sigma_A = \frac{F_x}{\pi \cdot (D - t) \cdot t}$$

$$\sigma_B = \frac{4 \cdot M_x}{\pi \cdot (D - t)^2 \cdot t}$$

The combined nominal longitudinal stress is then:

$$\sigma_L = \sigma_A + \sigma_B$$

Step 2 If the combined longitudinal stress is compressive, the allowable pipe pressure is given by:

$$p_{corr} = \gamma_m \cdot \frac{2 \cdot t \cdot SMTS}{D - t} \cdot \frac{[1 - \gamma_d \cdot (d/t)^*]}{\left[1 - \frac{\gamma_d \cdot (d/t)^*}{Q}\right]} \cdot H_1$$

where:

$$H_1 = \frac{1 + \frac{\sigma_L}{\xi \cdot SMTS} \cdot \frac{1}{A_r}}{1 - \frac{\gamma_m}{2 \cdot \xi \cdot A_r} \cdot \frac{1 - \gamma_d \cdot (d/t)^*}{1 - \frac{\gamma_d \cdot (d/t)^*}{Q}}}$$

$$A_r = 1 - \frac{d}{t} \cdot \theta$$

For circumferential corrosion defects with internal pressure and superimposed longitudinal compressive stresses, the following procedure is given:

Step 1 Determine longitudinal stress as in the previous case.

Step 2 If the combined longitudinal stress is compressive, the allowable pipe pressure is given by:

$$p_{corr,circ} = \min\left[\gamma_{mc} \cdot \frac{2 \cdot t \cdot SMTS}{D - t} \cdot \left(\frac{1 + \frac{\sigma_L}{\xi \cdot SMTS} \cdot \frac{1}{A_r}}{1 - \frac{\gamma_{mc}}{2 \cdot \xi} \cdot \frac{1}{A_r}} \right), \gamma_{mc} \cdot \frac{2 \cdot t \cdot SMTS}{D - t}\right]$$

where:

$$A_r = 1 - \frac{d}{t} \cdot \theta$$

The longitudinal stress in the remaining ligament is set to not exceed $\eta \cdot SMYS$ neither in tension nor compression:

$$|\sigma_L| \leq \eta \cdot SMYS \cdot (1-(d/t))$$

7.3.2.3 Interacting defects

The DNV rules on interacting defects are treating the load case including internal pressure only. A lot of information is also required for an assessment of interacting defects. The minimum information required is:

1. The angular position of each defect around the circumference of the pipe
2. The axial spacing between adjacent defects
3. Whether the defects are internal or external
4. The length of each individual defect
5. The depth of each individual defect
6. The width of each individual defect

The allowable operating pressure for a pipeline with a colony of interacting defects can then be estimated using the following procedure:

Step 1 For regions where there is background metal loss, the local wall thickness and defect depths can be used.

Step 2 The corroded section of the pipeline should be divided into sections of a minimum length of $5.0 \cdot \sqrt{D \cdot t}$, with a minimum overlap of $2.5 \cdot \sqrt{D \cdot t}$. Steps 3 to 12 should be repeated for each sectioned length to assess all possible interactions.

Step 3 Construct a series of axial projection lines with a circumferential angular spacing of:

$$Z = 360 \cdot \frac{3}{\pi} \cdot \sqrt{\frac{t}{D}} \text{ (degrees)}$$

Step 4 Consider each projection line in turn. If defects lie within ±Z they should be projected onto the current projection line

Step 5 Where defects overlap, they should be combined to form a composite defect. Taking the combined length and the depth of the deepest defect forms this. If the composite defect consists of an overlapping internal and external defect, then the depth of the composite defect is the sum of the maximum depth of the internal and external defects.

Step 6 Calculate the allowable pipeline pressure ($p_1, p_2 \dots p_N$) of each defect to the N^{th} defect, treating each defect or composite defect as a single defect:

$$p_i = \gamma_m \cdot \frac{2 \cdot t \cdot SMTS}{D - t} \cdot \frac{\left[1 - \gamma_d \cdot (d_i / t)^*\right]}{\left[1 - \frac{\gamma_d \cdot (d_i / t)^*}{Q}\right]} \qquad i = 1 \dots N$$

Where variables are as given in the assessment of a single defect.

Step 7 Calculate the combined length of all combinations of adjacent defects. For defects n to m the total length is given by:

$$l_{nm} = l_m + \sum_{i=n}^{i=m-1} (l_i + s_i) \quad n,m = 1 \dots N$$

Step 8 Calculate the effective depth of combined defect formed from all of the interacting defects from m to n, as follows:

$$d_{n,m} = \frac{\sum_{i=n}^{i=m} d_i \cdot l_i}{l_{nm}}$$

Step 9 Calculate the allowable pipeline pressure of the combined defect from n to m (p_{nm}), using l_{nm} and d_{nm} in the single defect equation:

$$p_{nm} = \gamma_m \cdot \frac{2 \cdot t \cdot SMTS}{D - t} \cdot \frac{\left[1 - \gamma_d \cdot (d_{nm} / t)^*\right]}{\left[1 - \frac{\gamma_d \cdot (d_{nm} / t)^*}{Q_{nm}}\right]} \quad n,m = 1 \dots N$$

where the variables are defined as for a single defect. Here, the definition of the standard deviation of d_{nm}/t is dependent on whether or not the depth measurements are correlated. For fully correlated depth measurements, the rules specify:

$$StD[d_{nm}/t] = StD[d/t]$$

For uncorrelated depth measurements, DNV gives:

$$StD\left[d_{nm}/t\right] = \frac{\sqrt{\sum_{i=n}^{m} l_i^2}}{l_{nm}} \cdot StD\left[d/t\right]$$

Step 10 The allowable corroded pipe pressure for the current projection line is taken as the minimum of the failure pressures of all of the individual defects (p_1 to p_N), and of all the combinations of individual defects (p_{nm}) on the current projection line.

$p_{corr} = \min (p_1, p_2, \ldots p_N, p_{nm})$

p_{corr} is not allowed to exceed p_{mao}.

Step 11 The allowable corroded pipe pressure for the section of the corroded pipe is taken as the minimum of the allowable corroded pipe pressures calculated for eeach of the projection lines around the circumference.

Step 12 Repeat steps 3 to 12 for the next section of the corroded pipe.

As the reader can see, this is a long and rather time-demanding procedure. The assessment of complex shaped defects is even longer. The reader should refer to the DNV guidelines for a complete description.

7.3.2.4 Allowable Stress Design

As mentioned, the DNV rules have a second approach to assessment of corroded pipelines, i.e. the Allowable Stress Format. In this method, the failure pressure or capacity of the pipeline with the corrosion defect is calculated, and this failure pressure is multiplied by a single safety factor based on the original design factor. Here, the ultimate tensile strength is used (UTS), but if it not known, the rules specify that SMTS should be used. The total usage factor is specified as:

$$F = F_1 \cdot F_2$$

where:

$F_1 =$ 0.9 (Modeling factor)

$F_2 =$ Operational Usage Factor, which is introduced to ensure a safe margin between the operating pressure and the failure pressure of the corrosion defect (normally equal to the Design Factor)

The safe working pressure of a single defect subject to internal pressure loading only is given by the following procedure:

Step 1 Calculate the failure pressure of the corroded pipe (P_f):

$$P_f = \frac{2 \cdot t \cdot UTS}{D - t} \cdot \frac{1 - \frac{d}{t}}{1 - \frac{d}{t \cdot Q}}$$

where:

$$Q = \sqrt{1 + 0.31 \cdot \left(\frac{1}{\sqrt{D \cdot t}}\right)^2}$$

Step 2 Calculate the safe working pressure of the corroded pipe (P_{sw}):

$$P_{sw} = F \cdot P_f$$

The rules clearly specify that due consideration should be given to the measurement uncertainty of the defect dimension and the pipeline geometry, which is not accounted for in the equations. This method also assesses the cases of internal pressure and combined compressive loading, interactive defects and complex shaped defects.

7.3.3 RAM PIPE REQUAL Project

In the RAM PIPE REQUAL project two approaches are taken to evaluate corrosion rates. The first is a 'qualitative' model based on scoring or ranking methods to develop general indicators of the rates and extents of corrosion. The second model is termed 'quantitative' and is based on measurements of pipeline wall losses, either internal or external.

The RAM PIPE REQUAL project has also resulted in a suggested burst pressure equation for corroded pipelines and descriptions of the time dependent reliability of a corroded pipeline.

7.3.3.1 Qualitative Model of Corrosion Rates

This model is meant to give general indicators of corrosion loss in a pipeline, and can be used for an overall evaluation of a pipeline. The loss of pipeline or riser wall thickness due to corrosion (t_c) is expressed as:

$$t_c = t_{ci} + t_{ce}$$

where:

t_{ci} = Loss of wall thickness due to internal corrosion
t_{ce} = Loss of wall thickness due to external corrosion

The loss of wall thickness due to internal/external corrosion ($t_{ci/e}$) is then formulated as follows:

$$t_{ci/e} = \alpha_{i/e} \cdot v_{i/e} \cdot (L_s - L_{pi/e})$$

where:

$\alpha_{i/e}$ = Effectiveness of the inhibitor or protection
$v_{i/e}$ = Average (mean during service life) corrosion rate

L_s = Service life of the pipeline or riser (in years)
$L_{pi/e}$ = 'Life' of initial protection provided to the pipeline

This model assumes that there are no inspections and repairs performed during the service life of pipeline or riser to maintain the strength integrity of the pipeline to carry pressure. Maintenance is required to preserve the protective management measures of employed (e.g. renew coatings, cathodic protection and inhibitors). The corrosion management is 'built-in' to the pipeline or riser at the start of the service period. Inspections and maintenance are performed to disclose unanticipated or unknowable defects and damage (due to accidents). Stated another way, when an existing pipeline is re-qualified for service, inspections should be performed to disclose the condition of the pipeline and riser, and then an assessment performed to determine if under the 'present' condition of the pipeline that it is fit for the proposed service. Alternative management of the pipeline could be to de-rate it (reduce allowable operating pressures), protect it (inhibitors, cathodic protection), repair it (doublers, wraps) or replace it.

For design and re-qualification, the corrosion rate is based on the owner or operators evaluation of the corrosivity of the fluids and/or gases transported inside the pipeline and on the corrosivity of the external environment conditional on the application of the protection or 'inhibition' program. Table 7.11 gives the suggested median corrosion rates, their variabilities or standard deviation of the logarithms of the corrosion rates (approximately the coefficient of variation of the corrosion rates) and the linguistic variables used to describe these corrosion rates.

Table 7.11 – Descriptors of corrosion rates

Descriptor	Corrosion rate (mm/year)	Variability (%)
Very low	0.001	10
Low	0.01	20
Moderate	0.1	30
High	1.0	40
Very high	10.0	50

As an example, a dehydrated sweet gas would have a low to very low corrosion rate, particularly if inhibitors were used to protect the steel. A normally dehydrated sweet oil without inhibitors would have a moderate corrosion rate. A pipeline transporting high temperature salt water or sour wet gas could have a corrosion rate that would be high to very high. An unprotected pipeline could be expected to have a moderate external corrosion rate.

In this corrosion formulation, the effectiveness of corrosion management is expressed with two parameters, the inhibitor efficiency, $\alpha_{i/e}$, and the life of the protection, $L_{pi/e}$. If the inhibitor were perfect, $\alpha_{i/e}$ would equal 1.0. Otherwise, the inhibitor efficiency is expressed as in Table 7.12.

Table 7.12 – Descriptors of inhibitor efficiency

Descriptor	$\alpha_{i/e}$
Very low	10.0
Low	8.0
Moderate	5.0
High	2.0
Very High	1.0

The life of the protection, $L_{pi/e}$, reflects the operator's decision regarding how long the protection that will be provided will be effective at preventing corrosion. Table 7.13 defines the general categories of the life of protective systems. It is also used to specify the expected service life of the pipeline.

Table 7.13 – Descriptors of protection life

Descriptor	$L_{pi/e}$ or L_s
Very short	1
Short	5
Moderate	10
Long	15
Very Long	≥ 20

For example, the life of high quality external coatings in the absence of mechanical damage can be 10 years (moderate), or the life of low quality external coatings with mechanical damage can be 1 year or less (very short).

Given this information, the pipeline operator could define the expected life of the pipeline, the life of the protective system and the effectiveness of this system, and then based on the transported product and environment of the pipeline estimate the internal and external corrosion rates. Formula X then determines the corrosion allowance.

7.3.3.2 Quantitative Model of Corrosion Rates

The RAM PIPE REQUAL project defines the following quantitative model to predict and evaluate pipeline corrosion losses.

$$\text{CorrosionLoss} = [1 + e^{(1-N\cdot t)}]\cdot \log(1 + t)^{P}\cdot [1 + 1/(1+t)\cdot t^{1/3}]$$

where:

N, P = Shaping parameters depending on pipeline's environment
t = Time, measured in years

To calibrate the corrosion loss equation, several sources of data on corrosion on atmospheric conditions were used to supply corrosion loss data. For the corrosion loss data, the foregoing expression was applied and a fit of the curve for the values provided was accomplished to produce the best estimate (unbiased) results. For the corrosion rate data, the same approach was used, but with the equation for the corrosion rate. The corrosion rate equation is then the derivative of the corrosion loss equation.

The mean values and variability of P and N for different pipeline metals are presented in Table 7.14. Note that there is a very large variability associated with the corrosion parameters.

Table 7.14 – Values for corrosion loss parameters P and N

	Iron	Carbon steels	Low alloy steels	Stainless steels
Mean P	7.48	15.03	9.38	0.47
Mean N	3.00	3.48	1.90	~
COV P	32%	103%	81%	67%
COV N	94%	124%	75%	~

The general effect of the parameters P and N is that with increasing values of P, the corrosion loss or rate increases. The value of N does not influence the corrosion rate or loss for large values of t, which means that this parameter does not play an important role in long-term analyses. It is however important for short-term analyses. P and N are dependent on the parameters describing the fluid or gas transported in the pipeline and the surrounding environment where the corrosion loss is being calculated. Among others these can be bio-corrosion, pH of fluid or gas, temperature of fluid or gas and the flow regime. The flow regime is relevant for multiphase carrying pipes, where the flow can be difficult to classify. A multiphase pipeline may carry a certain percentage of oil, gas and water, each of which has a different viscosity and density and therefore tends to move with a different velocity in the pipe. The rate of corrosion is directly related to the velocity of the media in the pipeline.

7.3.3.3 Burst Pressure Capacity

RAM PIPE REQUAL project developed the following burst equation for a corroded pipe as:

$$p_{bd} = \frac{2.2 \cdot t_{nom} \cdot SMTS}{D_0 \cdot SCF}$$

where:

t_{nom} = Minimum wall thickness (original wall thickness, t – corrosion depth, d)
D_0 = Mean pipeline diameter, = D – t
SCF = Stress concentration factor defined by:

$$SCF = 1 + 2 \cdot (d/R)^{0.5}$$

The defined stress concentration factor is the ratio of maximum hoop stress over nominal hoop stress due to a notch of depth d in the pipeline cross section that has a radius R.

7.3.3.4 Time Dependent Reliability

The RAM PIPE REQUAL approach accounts for the fact that the reliability of a pipeline is a time dependent function dependent on the corroded thickness of the pipeline. The corroded thickness is dependent on the average rate of corrosion and the time that the pipeline or riser is exposed to corrosion. This project defines the time dependency with the following equation:

$$\beta = \ln (K_p \cdot t - K_p \cdot t_{ci/e}) / \sigma_{\ln P/R}$$

where:

K_p = $(2.2 \cdot SMTS / D \cdot SCF \cdot P_0)$
β = Safety index in the standard normal distribution, $P_f = 1 - \phi(\beta)$
$Kp \cdot t = FS_{50}$

where FS_{50} is the median factor of safety in the burst capacity of the pipeline or riser. The final expression for β is then:

$$\beta = \ln (FS_{50} - FS_{50} \cdot (t_{ci/e}/t)) / \sigma_{\ln P/R}$$

As the pipeline corrodes, the reduction in the pipeline wall thickness leads to a reduction in the median factor of safety that in turn leads to a reduction in the safety index β and an increase in the probability of failure. In addition, there is an increase in the uncertainties associated with the corrosion rates and their effects on the burst capacity of a pipeline or riser. An analytical model for the increase in total uncertainties as a function of the corrosion is expressed as:

$$\sigma_{\ln P/R}|t = \sigma_{\ln P/R}|t_0 \cdot (1 - t_{ci/e}/t)^{-1}$$

where:

$\sigma_{\ln P/R}|t$ = Uncertainty at a given time t
$\sigma_{\ln P/R}|t_0$ = Uncertainty at t = 0
$t_{ci/e}$ = Corroded thickness
t = Initial thickness

As an example, from no corrosion to a measured corrosion equal to half the wall thickness ($t_{ci/e}/t$ = 0.5), the uncertainty will increase by a factor of 2.

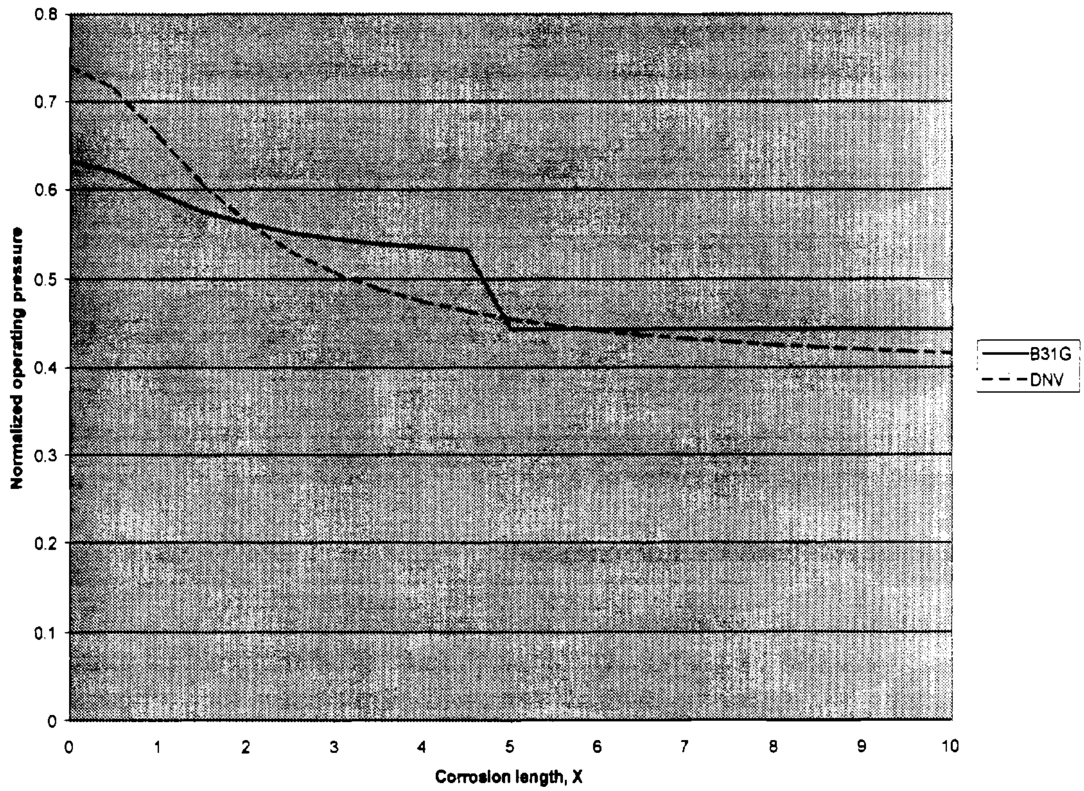
7.3.4 Comparison of Burst Pressure Formulations

To get a better understanding of the properties of the different guidelines reviewed, a quantified comparison of the methods will be useful. Different properties of the pipeline and the corrosion define the safe operating pressure for the methods reviewed. This makes the comparison somewhat intricate if continuous ranges of any of these variables are to be studied. The B31G equation and the DNV code are easily compared as in [8], but this is not the case for the equations developed in the RAM PIPE project. The burst pressure data are therefore presented in different figures. Figures for corrosion depths, d/t equal to 30%, 50% and 70% are presented. For the DNV case, a target annual failure probability of 10^{-4} is selected with the corresponding safety class Normal. According to Bai (1998), current technology on MFL (magnetic flux leakage) methods is able to give a sizing accuracy of ±10% of t with a confidence level of 80%. This level is therefore chosen for the calculations together with the partial safety factors for relative depth measurements. The characteristic corrosion length, X, is used as the free parameter for the B31G equation and the DNV method. Since the RAM PIPE method gives the safe operating pressure as a function of d/R independent of the corrosion length, d/R is used as the free variable here.

The first case studied is for d/t = 30 % and is presented in Figure 7.6. For increasing corrosion lengths (X equals 0 to 10), the normalized operating pressure calculated by DNV varies from 0.74 to 0.42. Using B31G, the normalized operating pressure varies from 0.64 to 45. For short corrosions, B31G give more conservative results than DNV while the opposite is the case for longer corrosion. For increasing d/R ratios, RAM PIPE gives normalized operating pressures ranging from 0.54 to 0.42, which is in the same range as the pressures using DNV and B31G for corrosion lengths X > 2.

Figure 7.7 presents the case of d/t = 50 %. Here, the DNV method gives more conservative results than B31G for corrosion lengths X > 1. The normalized operating pressures are ranging from 0.74 to 0.23 and from 0.64 to 32 respectively. The RAM PIPE equation gives normalized operating pressures ranging from 0.37 to 0.28 for increasing d/R. As seen on the figure, these values are in the same range as the values from DNV for X > 2, and the values from B31G for X > 4.5.

For the last case shown in Figure 7.8, d/t = 70%, the operating pressure calculated by DNV drops to very low values already at X = 0.5. B31G gives much less conservative results with normalized operating pressures ranging from 0.64 to 0.18 for increasing values of X. RAM PIPE gives values ranging from 0.18 to 0.23 for increasing values of d/R.



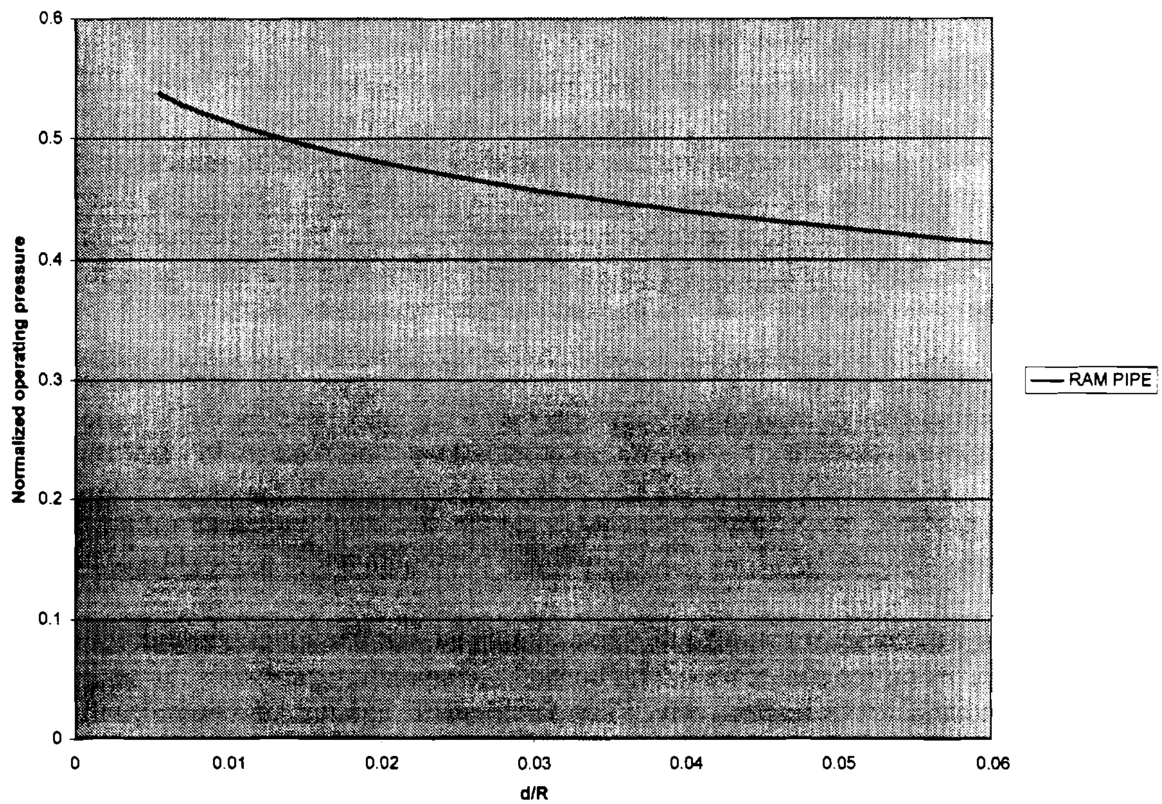


Figure 7.6 - Safe operating pressures, d/t = 30%

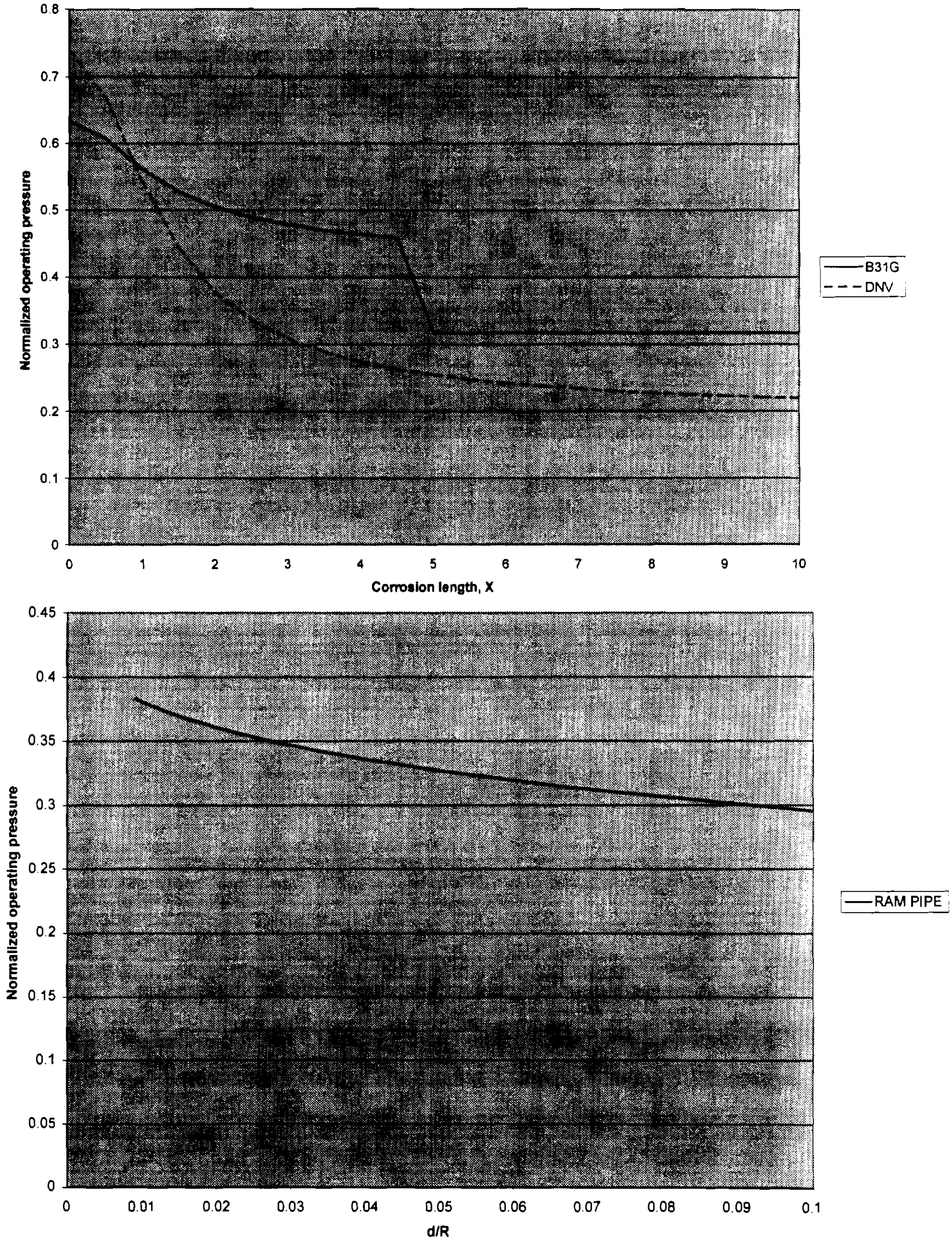


Figure 7.7 - Safe operating pressure, d/t = 50%

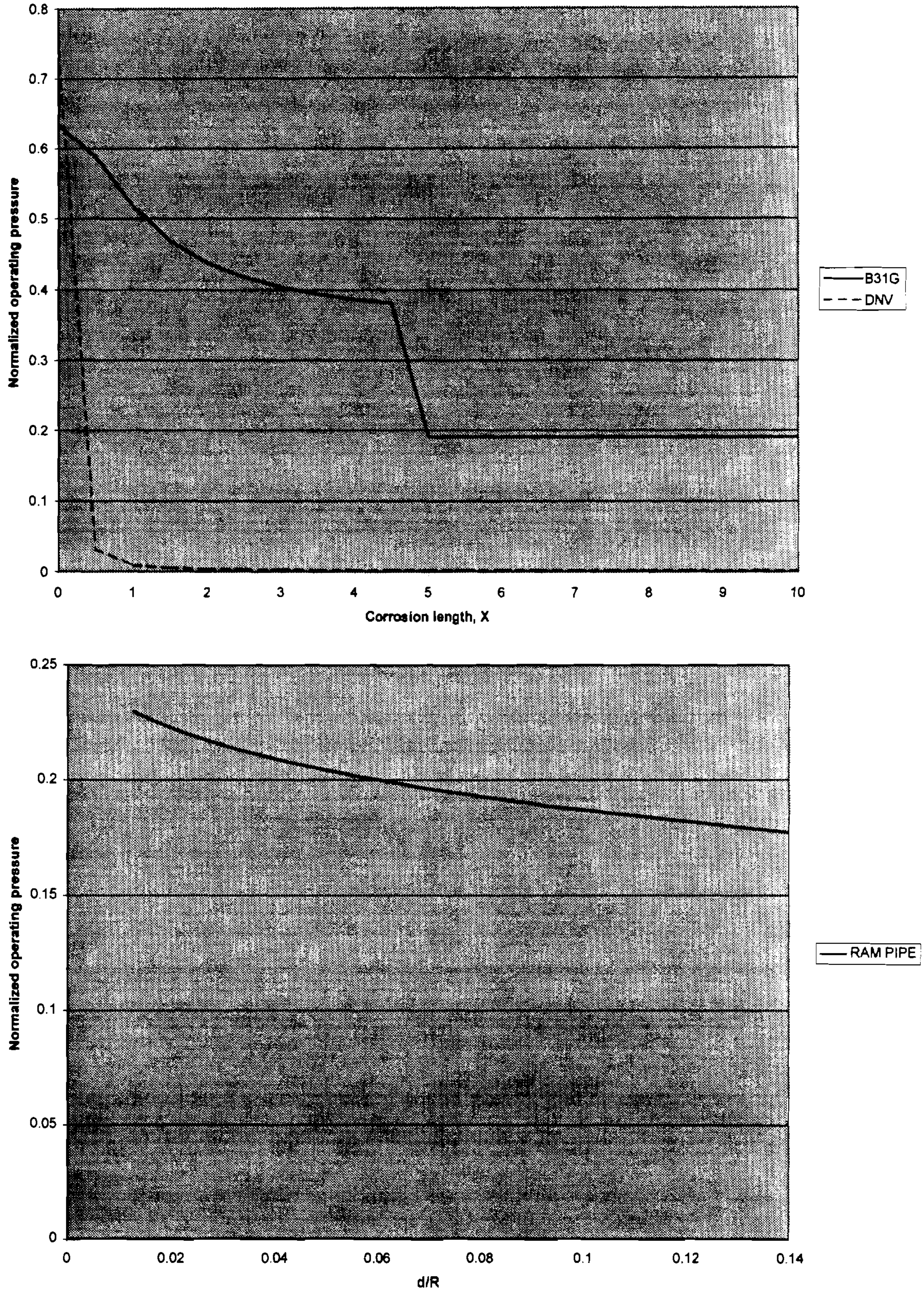


Figure 7.8 - Safe operating pressure, d/t = 70 %

7.3.5 Comparisons of Burst Pressure Formulations With Test Data

The three methods described are compared with laboratory burst pressure data from the 151 tests summarized and presented in RAM PIPE REQUAL Report 1. For the B31G equation and the DNV method, all safety factors are set to unity to compare the burst pressures, not the safe operating pressures. The results are presented in Table 7.15.

Table 7.15 – Comparisons of Burst Pressure Formulations with Laboratory Test Data

Method	B31G	DNV	RAM PIPE
Mean Bias	1.71	1.47	1.00
Standard Deviation	0.92	0.81	0.22
COV	0.54	0.55	0.22

Compared with these physical burst pressure tests, the method developed in the RAM PIPE REQUAL project performs much better than B31G and the DNV formulations. A mean bias of 1.00 is excellent and the coefficient of variation is also low with 22%. Surprisingly, even though the DNV formulation is much more complex and based on extensive data developed from numerical simulations, this formulation is indicated to be significantly biased with a mean bias of 1.47 and with a large scatter in the bias reflected in the bias coefficient of variation of 55 %.

7.3.6 Discussion

On basis of the different guidelines reviewed here it is hard to say anything about which one is the "best" guideline. Some of the evaluation criteria are:

- Simplicity – Ease of use and implementation.
- Versatility – The ability to handle a wide variety of real problems.
- Compatibility – Readily integrated into common engineering and operations procedures.
- Workability – The information and data required for input is available or economically attainable, and the output is understandable and can be easily communicated.
- Feasibility – Available engineering, inspection, instrumentation and maintenance tools and techniques are sufficient for application of the approach.
- Consistency – The approach can produce similar results for similar problems when used by different engineers.

This section will evaluate and discuss each formulation in view of these criteria applied to corrosion damage of pipelines. Conclusions can be made to which method will perform best in each one of these. Some of the discussion points are only relevant for one single method and will be mentioned in the chapter for each method. Other points such as instrumented vs. non-instrumented pipelines are relevant for all methods and will be discussed in a separate section.

7.3.6.1 General

Two ways of assessing corrosion damage can either be methods based on inspections of the pipelines and methods based on corrosion data from other pipelines. Methods based on inspections of pipelines will be able to give better results for the residual strength of the pipeline if good data on the biases and uncertainties of the inspection results are known. One problem is that this is not always the case, especially on specifications on biases of the corrosion depths. Another problem is the uncertainties on the length of the corrosion damage. No data is available on how well the speed on the instrumented pig can be controlled. For the other case these problems are not present, but others appear. There will be large uncertainties associated with a transfer of data from one pipeline to another. Corrosion is as mentioned dependent on very many factors related to e.g. pipeline material, environment and the oil or gas transported in the pipeline. One can even expect the corrosion rate of a pipeline to change during the life of the pipeline. Many pipelines are not ready for inspection, and the only solution will then be the latter.

7.3.6.2 ANSI/ASME B31G

The origin of the ANSI/ASME B31G equation is the NG18 equation, which was first developed in 1971. It is obvious that pipeline technology has advanced a long way since then, and even if the B31G equation is from 1991 it is still based on NG18 with some modifications to account for available experimental data. Many studies have pointed out weaknesses in this equation and it still needs a fair amount of information to be applicable. The variables needed for calculations are:

- Pipe diameter
- Nominal pipe wall thickness
- Maximum corrosion depth
- Length of corrosion
- Specified minimum yield stress (SMYS)

This equation also incorporates a design factor F, which is normally equal to 0.72. This design factor was never based on a rational assessment of operational stresses but can be tracked back to the 1935 B31 codes where the working pressure was limited to 80% of the mill test pressure which itself had a design factor up to 0.9. The total design factor is thus $0.8 \cdot 0.9 = 0.72$. This number has since been used directly in some codes such as the DNV codes of 1977 or the 1958 version of B31.8 for on-land pipelines.

The B31G equation for safe working pressure is a discontinuous function of the characteristic length of the corrosion, $X = \frac{L}{\sqrt{D \cdot t}}$. At the split value $\sqrt{0.8} \cdot X$, depending on the corrosion depth, the safe maximum pressure drops up to 50%. No analytical or logical argumentation can justify this. In [X], it is mentioned that the inherent reliability of the pipeline is extremely dependent on the length and shape of the corrosion. For a given corrosion depth, the probability of failure changes with as much as a magnitude of 4 (from 10^{-8} to 10^{-4} for $d/t = 0.5$) for a characteristic corrosion length X changing from 1 to 10. The same happens with changing corrosion depths, e.g. for long corrosion ($X = 10$), the probability of failure is about 10^{-6} for $d/t = 0.3$ and 10^{-2} for $d/t = 0.7$.

Several modifications have also been proposed to these guidelines, e.g. changes in proposed flow-stress, corrosion area definition or proposed bulging factor. The drawback has often been that modifying one or more parameters in order to obtain a better adaptation to existing and newer results have resulted in a negative effect for other design cases with a different geometry and corrosion configuration. We can take a look at how good the B31G equation performs with regards to the evaluation criteria listed to get a good perception of how good it performs.

The equation is clearly simple to use and implement. All variables go right into the equation for safe maximum pressure, and calculations are trivial. The equation is also versatile in the way that it can handle corrosion depths from 10% to 80% of the original pipe wall thickness. It does not however handle a real life problem such as system effects when more than one corrosion defect occurs. This is a very realistic problem, and it is a fact that the pipeline reliability is dependent on the number of failures found. The compatibility criterion is automatically fulfilled since the B31G equation has been the standard of practice for many years. On the other hand, the equation still needs exact data on the size of the corrosion damage. This data is only available through inspections, and a lot of pipelines are not ready for inspection. Coupled with the inconsistency in the results, the workability is not the good side of this approach. The engineering tools required for use of this equation such as instrumented pigs are good and well developed, and the method can be said to be feasible.

7.3.6.3 DNV Recommended Practice on Corroded Pipelines

These guidelines have recently been published in final form (1999). It is clear that DNV has taken a step in the right direction of good assessment of corroded pipelines with these rules. There are a lot of issues to discuss on the guidelines, but that does not necessarily mean that they don't perform well. It is basically founded in the fact that the more you know about a subject, the more will you have to say about it.

First, the DNV guidelines are fully based on results from inspections of pipelines. The advantages and drawbacks with this is discussed later in this section, but some other points with regards to this will be discussed here. The data needed for use of the guidelines are the basic dimensions of the corrosion, which are corrosion depth, length and width. For the cases of interacting defects or complex shaped defects, additional information is required. The calibration of the code is done with fully probabilistic methods and the code itself is actually a limit-state based code. The code is based on partial safety factors on which I have some comments. The partial safety factor for corrosion depth is taking account for the uncertainties associated with the sizing of the corrosion. The guidelines do however indicate that only the uncertainties in the depth sizing are taken into account. There will always be uncertainties in the length and width of the corrosion too, and these should also be included here. The same inspection sizing accuracy is given relative to the wall thickness and for a specified confidence level. Confidence levels of 80 and 90% are specified, but nothing is mentioned about a possible bias in the measurements. Studies done as part of the RAM PIPE REQUAL project indicate that inspections performed by instrumented pigs can give results with substantial under-predictions of the corrosion depth, where actual depth over predicted depth can be as large as 1.3.

These guidelines are taking system effects into account, which means that the effect several corrosion-damages have on the pipeline reliability is included. The guidelines suggest that, as a conservative estimate, the failure probabilities of each defect should be added. This means that for severe corrosion damage or a very large number of defects, this may be overly conservative. For a more correct evaluation in this situation, conditional probability methods should be used.

As mentioned, fully probabilistic methods have been used in the calibration of the partial safety factors. To fully understand the validity of this process, data on the biases and uncertainties determined from the test data are needed. No information is given on this in the guidelines, and the necessary data should be easily available from the work done on the guidelines.

A very interesting point is the way the calibration of the guideline is performed. Test data is used to calibrate results from finite element analyses, which in turn is used to calibrate the partial safety factors. The test results provided are for burst pressures up to 250 bar. The finite element analyses are however done up to 1100 bar burst pressures, which is a very significant extrapolation. When we look at pressures and structural behaviors of this degree (bursting), the performance of the pipeline material is highly non-linear. This extrapolation therefore seems very large, and this might mean that the final safety factors are not the optimal ones. Further, there are several places in the guidelines mentioned that there are limitations in for which corrosion sizes the validation have been performed. This support the fact that there are very few full-scale tests performed in the validation process.

So, how do the DNV guidelines perform with respect to the evaluation criteria? The rules are still not very complicated, even if the calculations can look somewhat time demanding on the first look. The calculations are however trivial, and the simplicity must therefore be said to be good. The rules are versatile, they treat a wide variety of problems, but one drawback is that they do not treat the problem on non-instrumented pipelines. They are based on the new DNV 'Rules for Submarine Pipeline Systems', and is therefore easily integrated into common procedures. The data amount needed can in some cases be large, but as long as the output from the instrumented pigs is good, the rules will perform well. The rules should also be consistent in view of the calibration methods used on the safety factors.

7.3.6.4 RAM PIPE REQUAL

This project is taking a somewhat different approach to the corrosion problem than most guidelines and codes in use today. An equation for burst pressure is developed on the basis of a database of full-scale tests of pipelines, and everything is very well documented. The burst pressure equation is based on corrosion depth and a stress concentration factor. Data on the depth is therefore needed, and this can either be obtained through inspections or the equation for corrosion loss developed in this project. The equation for corrosion loss includes the two variables, P and N, on which there will be large uncertainties. These variables are dependent on several factors such as steel quality, bio-corrosion, pH and temperature of the oil. One can therefore see large problems in the process of finding good and correct values for P and N. The project is also treating the problem of increased uncertainty in corrosion rates as a function of time, which is very good.
The RAM PIPE REQUAL guidelines are mostly simple and versatile. Easy calculations and an ability to treat a wide variety of real problems are key words for this project. For the engineer updated on reliability based engineering, the guidelines are also compatible. Problems can be seen in the workability and consistency because of the mentioned parameters P and N in the corrosion loss equation.

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Appendix A – Wave Load Effects on Pipelines

Wave Load Effects on Pipeline

Ahsan Kareem and Xinzhong Chen
NatHaz Modeling Laboratory
Dept. of Civil Engineering and Geological Sciences
University of Notre Dame
Notre Dame, Indiana 46556-0767
kareem@nd.edu

ABSTRACT: Wave forces on a fixed horizontal pipeline segment under short-crested seas are first investigated using the directional spectra and an alternative model considering the spatial correlation of water particle motions in terms of coherence functions is then examined.

Introduction

In the Bay of Campeche, the integrity of more than 500 km of pipeline is threatened each year by the potential of hurricane activity in the region. The performance of pipeline is not only judged by its capacity to withstand pressure and impact of corrosion, but also by its ability to withstand movements of sea floor soils. The soft sea floor soils in the Bay of Campeche are known to develop significant motions during the passage of hurricanes. The wave and current induced hydrodynamic loads and movement of the sea floor influence the stability of the pipeline. Hurricane Roxanne, the most severe hurricane to affect the Bay of Campeche during this century inflicted major damage to offshore pipelines in the region (Valdex et al., 1997; Bea et al., 1998a; Cardone et al., 1998). Morris et al. (1988) and Bea et al. (1998a) have reported Risk Assessment and Management (RAM) based criteria for the design and requalification of pipelines and risers in the Bay of Campeche. Their paper summarizes various possible loading scenarios on the pipeline, based on specifications laid down by IMP and guidelines developed by AGA.

It is noted by Bea et al. (1998a) that the specified design loads will generally result in a significant conservative bias in the computed design loads. The bias is defined in terms of the ratio between the expected maximum loading and the design maximum loading. Based on the orientation of the pipeline, Bea et al. (1998a) developed biases in terms of the ratio between the current and wave induced particle velocities. Bea et al. (1998a) correctly pointed out another bias introduced by the space-time variation in the wave kinematics, associated hydrodynamic loads, and the transient nature of these loads. The spatial variation in hydrodynamic loads may not only reduce the effective loading on a section of pipeline, but may also influence the support conditions of the pipeline by shifting the soil support, thus limiting the displacements or deformations experienced by the pipeline.

It is noted in this discussion that the space-time variability of wave kinematics and variations in the currents can have a major influence on the design and performance of a pipeline. Several studies in the literature have addressed this topic at different levels of detail and modeling sophistication (Zimmerman et al., 1986; Lambrakos, 1982; Jacobsen et al., 1989; Lambrakos et al., 1987; Lammaert et al., 1989; Borgman and Hudspeth, 1992; Collins et al., 1995; Hale et al., 1989; Grace and Zee, 1981). In light of the importance of the problem and a lack of unified information on hydrodynamic load effects, accurate quanification of these effects for risk based design standards and guidelines are needed.

This study examines the space-time variation of hydrodynamic loads acting on pipelines for a range of orientations with respect to the local bathymetry. The coherence modeling plays a central role in the analysis of structures exposed to wind, wave or seismic load effects (Kareem et al., 1997). A single point representation of the stochastic field concerning wind, wave, or seismic effects may be employed for structures smaller than a typical scale. However, partial correlation over larger or longer structures such as pipelines necessitates the use of multipoint stochastic field statistics to accurately ascertain dynamic load effects. The level of correlation is generally expressed in terms of a coherence function whose form depends on the characteristics of the stochastic field under consideration (Mitwally and Novak, 1989; Kareem and Song, 1998). Although the importance of directional distribution of ocean waves has been acknowledged in offshore engineering (Borgman, 1990), the directional wave spectra is not widely used in design practice due to its complexity and added computations.

The main feature of a directional wave system is that the total first order wave force on a structure is less than that of a unidirectional wave system having the same total energy. The lack of correlation may be accounted for by using a coherence function and cross-spectrum of the wave height and water particle velocity fluctuations between locations of interest. Therefore, associated hydrodynamic loads can be expressed in terms of this coherence function. Once loads are expressed in terms of the coherence, several simplifications in the load description can be introduced. These simplifications are similar to those for wind effects, which are very attractive for inclusion in design specifications or other guidelines.

Water Particle Velocity

In this study, a Cartesian coordinate system is used with horizontal x and y coordinate axes, a vertical z coordinate axis, and an origin at the mean water level (Fig. 1). The water depth, h, is assumed to be constant. The sea surface is denoted as $z = \xi(x, y, t)$, where t is time. Using the appropriate spatial and temporal scales (i.e., small distance from origin and limited time), the sea surface can be treated as a stationary stochastic field.

Assuming that sea water is an incompressible, inviscid fluid and that the wave motion is irrotational. the velocity potential can be represented in the form of the Fourier-Stieltjes integral which leads to the surface elevation $\xi(x, y, t)$:

$$\xi(x,y,t) = -\frac{1}{g}(\frac{\partial\Phi}{\partial t})_{z=0} = \int_{-\infty}^{\infty}\int_{-\pi}^{\pi} \exp[ik(x\cos\theta + y\sin\theta) - i\omega t]dA(\omega,\theta)$$

with the two-dimensional energy spectrum (two-side) of the surface waves $\bar{S}_{\xi\xi}(\omega,\theta)$ defined as:

$$< dA(\omega,\theta)dA^*(\omega_1,\theta_1) >= \bar{S}_{\xi\xi}(\omega,\theta)\delta(\omega-\omega_1)\delta(\theta-\theta_1)d\omega d\omega_1 d\theta d\theta_1$$

in which $\delta()$ is Dirac's delta function; * denotes the complex conjugate operator; and <> indicates the statistical averaging. The two-sided spectrum $S_{\xi\xi}(\omega,\theta)$ is related to the one side spectrum (extending from 0 to ∞) by:

$$S_{\xi\xi}(\omega,\theta) = 2\bar{S}_{\xi\xi}(\omega,\theta) \tag{1}$$

The water particle velocity components in the x, y and z-directions, $v_x(t), v_y(t)$ and $v_z(t)$, can be represented as:

$$v_x(t) = \int_{\infty}^{\infty}\int_{-\pi}^{\pi} \omega\cos\theta\frac{\cosh[k(z+h)]}{\sinh(kh)}\exp[ik(x\cos\theta + y\sin\theta) - i\omega t]dA(\omega,\theta) \tag{2}$$

$$v_y(t) = \int_{-\infty}^{\infty}\int_{-\pi}^{\pi} \omega\sin\theta\frac{\cosh[k(z+h)]}{\sinh(kh)}\exp[ik(xcos\theta + y\sin\theta) - i\omega t]dA(\omega,\theta) \tag{3}$$

$$v_z(t) = \int_{\infty}^{\infty}\int_{-\pi}^{\pi} -i\omega\frac{\sinh[k(z+h)]}{\sinh(kh)}\exp[ik(x\cos\theta + y\sin\theta) - i\omega t]dA(\omega,\theta) \tag{4}$$

Directional Surface Wave Spectrum

Within a storm-generating area, the sea surface elevation can be described by the directional spectrum in the form

$$S_{\xi\xi}(\omega,\theta) = S^u_{\xi\xi}(\omega)D(\omega,\theta) \tag{5}$$

where $S^u_{\xi\xi}(\omega)$ is the unidirectional one-sides spectrum and $D(\omega,\theta)$ is the directional spreading function which satisfies the condition:

$$\int_{-\pi}^{\pi} D(\omega,\theta)d\theta = 1 \tag{6}$$

Without the loss of generality, the Pierson-Moskowitz (1964) spectrum is initially used in this study, and it is given by:

$$S^u_{\xi\xi}(\omega) = \frac{\alpha g^2}{\omega^5} \exp\left[-0.74(\frac{\omega_p}{\omega})^4\right] \tag{7}$$

where $\alpha = 8.1 \times 10^{-3}$ is the Phillips constant; g is gravity acceleration; $\omega_p = g/U$; and U is the mean wind velocity.

The directional spreading function given by Donelan et al. (1985) is used in this study and takes the form

$$D(\omega, \theta) = \frac{1}{2}\beta \cosh^{-2}[\beta(\theta - \theta_0)] \tag{8}$$

where θ_0 is the mean wave direction.

Using high-frequency stereo photography, Banner (1990) proposed a formulation for β as:

$$\beta = \begin{cases} 2.61(\frac{\omega}{\omega_p})^{1.3} & for \quad 0.56 < \frac{\omega}{\omega_p} < 0.95 \\ 2.28(\frac{\omega}{\omega_p})^{-1.3} & for \quad 0.95 < \frac{\omega}{\omega_p} < 1.60 \\ 10^y & for \quad \frac{\omega}{\omega_p} > 1.6 \end{cases} \tag{9}$$

in which

$$y = -0.4 + 0.8393 \exp\left[-0.567 \ln(\frac{\omega}{\omega_p})^2\right] \tag{10}$$

Power Spectra of Water Particle Kinematics

To evaluate the resultant wave forces on a structure, the cross spectral densities of the velocity field are needed. The cross spectrum (one-side) of water partial velocities at Points 1 and 2 can be given as:

$$\begin{aligned} S_{v_1 v_2}(\omega) &= \omega^2 \frac{S^u_{\xi\xi}(\omega)}{\sinh^2(kh)} HYP_1[k(z_1 + h)] HYP_2[k(z_2 + h)] \\ & \int_{-\pi}^{\pi} D(\omega, \theta) C_1(\theta) C_2(\theta) \exp\{ik[(x_1 - x_2)\cos\theta + (y_1 - y_2)\sin\theta]\} d\theta \end{aligned} \tag{11}$$

in which x_1, y_1 and z_1 and x_2, y_2 and z_2 are the Cartesian coordinates of Points 1 and 2, respectively and $C_1(\theta)$ and $C_2(\theta)$ are given by

$$C_1(\theta) = \begin{cases} \cos\theta & for \quad 1//X \\ \sin\theta & for \quad 1//Y \\ 1 & for \quad 1//Z \end{cases} \tag{12}$$

in which $1//X$ indicates the velocity component at Point 1 in the x-direction, with analogous definitions for $1//Y$ and $1//Z$. $C_2(\theta)$ is similarly defined for the velocity component at Point 2. Finally,

$$HYP_1[k(z_1+h)] = \begin{cases} \cosh[k(z_1+h)] & for\ \ 1//X\ or\ 1//Y \\ -i\sinh[k(z_1+h)] & for\ \ 1//Z \end{cases} \tag{13}$$

and

$$HYP_2[k(z_2+h)] = \begin{cases} \cosh[k(z_2+h)] & for\ \ 2//X\ or\ 2//Y \\ i\sinh[k(z_2+h)] & for\ \ 2//Z \end{cases} \tag{14}$$

The power spectral density of the water particle velocities for the directional wave system, $S_{vv}(\omega)$, takes the form

$$S_{vv}(\omega) = \omega^2 \frac{S^u_{\xi\xi}(\omega)}{\sinh^2(kh)} HYP^2[k(z+h)] \int_{-\pi}^{\pi} D(\omega,\theta)C^2(\theta)d\theta \tag{15}$$

where $C(\theta)$ and HYP are defined like $C_1(\theta)$ and HYP_1.

The cross spectra of the water particle velocities for a unidirectional wave system propagating with an angle θ_0 can be given in the form

$$\begin{aligned} S^u_{v_1v_2}(\omega) &= \omega^2 \frac{S^u_{\xi\xi}(\omega)}{\sinh^2 kh} C_1(\theta_0)C_2(\theta_0)HYP_1[k(z_1+h)]HYP_2[k(z_2+h)] \\ &\quad \exp\{ik[(x_1-x_2)\cos\theta_0+(y_1-y_2)\sin\theta_0]\} \end{aligned} \tag{16}$$

in which $C_1(\theta_0)$ and $C_2(\theta_0)$ are given by Eq.12 with θ_0 substituted for θ.

The power spectra density of the water particle velocities for the unidirectional system, $S^u_{vv}(\omega)$, is given by

$$S^u_{vv}(\omega) = \omega^2 \frac{S^u_{\xi\xi}(\omega)}{sinh^2(kh)} C^2(\theta_0)HYP^2[k(z+h)] \tag{17}$$

The ratio of the power spectral density for unidirectional and directional systems can be given as

$$R = S_{vv}(\omega)/S^u_{vv}(\omega) = \int_{-\pi}^{\pi} D(\omega,\theta)C^2(\theta)d\theta/C^2(\theta_0) \tag{18}$$

It is clear that, mathematically speaking, the unidirectional wave system can be taken as a special case of the directional wave system when

$$D(\omega,\theta) = \delta(\theta-\theta_0) \tag{19}$$

Coherence Function for the Directional Wave System

For the directional wave system, the total wave energy along the mean wind direction is less than that of a unidirectional wave system having the same total energy, since part of this total energy produces lateral water partial motion. This reflects one aspect of the lack of spatial correlation. It may be accounted for using a coherence function and the cross-spectrum of the water partial velocity at two locations of interest. This can be expressed as:

$$S_{v_1 v_2}(\omega) = \sqrt{S_{v_1 v_1}(\omega) S_{v_2 v_2}(\omega)}\, coh(\Delta r, \omega) \tag{20}$$

in which the coherence function $coh(\Delta r, \omega)$ takes the form

$$coh(\Delta r, \omega) = \frac{\int_{-\pi}^{\pi} D(\omega, \theta) C_1(\theta) C_2(\theta) \exp\{ik[(x_1 - x_2)\cos\theta + (y_1 - y_2)\sin\theta]\} d\theta}{\int_{-\pi}^{\pi} D(\omega, \theta) C_1(\theta) C_2(\theta) d\theta} \tag{21}$$

The following approximated coherence function model is widely used in many studies

$$\begin{aligned} S_{v_1 v_2}(\omega) &= S^u_{v_1 v_2}(\omega) coh_0(\Delta r, \omega) \\ &= \sqrt{S^u_{v_1 v_1}(\omega) S^u_{v_2 v_2}(\omega)} \exp\{ik[(x_1 - x_2)\cos\theta_0 + (y_1 - y_2)\sin\theta_0]\} coh_0(\Delta r, \omega) \\ &= \sqrt{S^u_{v_1 v_1}(\omega) S^u_{v_2 v_2}(\omega)}\, coh^u(\Delta r, \omega) \end{aligned} \tag{22}$$

in which $coh_0(\Delta r, \omega)$ is the square root of the coherence function. It is analogous to turbulence and diminishes with the ratio of separation distance to wave length. A general expression for coherence function may be given in the form

$$coh^u(\Delta r, \omega) = \exp\Big(-\frac{1}{\lambda}\sqrt{(C_x \Delta x)^2 + (C_y \Delta y)^2 + (C_z \Delta z)^2}\Big) \tag{23}$$

in which $\Delta x = |x_1 - x_2|$, $\Delta y = |y_1 - y_2|$, and $\Delta z = |z_1 - z_2|$; C_x, C_y and C_z are the decay factors for the x, y and z-directions, respectively; $\lambda = 2\pi/k$ is the wave length; and

$$coh_u(\Delta r, \omega) = \exp\{ik[(x_1 - x_2)\cos\theta_0 + (y_1 - y_2)\sin\theta_0]\} coh_0(\Delta r, \omega) \tag{24}$$

In the same manner, the cross spectrum of sea surface elevations at any two locations in partially-correlated seas can be expressed as:

$$S_{\xi_1 \xi_2}(\omega) = \sqrt{S^u_{\xi_1 \xi_1}(\omega) S^u_{\xi_2 \xi_2}(\omega)}\, coh^u(\Delta r, \omega) \tag{25}$$

Wave Force on a Horizontal Pipeline

Consider a horizontal element of length L which is parallel to the X-axis. For any two locations on the element, $y_1 = y_2$ and $z_1 = z_2$. The wave force per unit length can be given as

$$p(x,t) = \rho\pi\frac{d^2}{4}C_M\dot{v}(x,t) + \frac{1}{2}\rho dC_D|v(x,t)|v(x,t) \tag{26}$$

where d is the diameter of the element; C_M and C_D are the inertia and drag coefficients which are dependent upon the flow and structural response characteristics; and $v(x,t)$ and $\dot{v}(x,t)$ are the water particle velocity and acceleration components in the y-direction, respectively.

The term $v(x,t)$ in the above equation is nonlinear. A linearlization of this term is required for the spectral analysis because it employs a linear superposition over the frequencies describing the sea-state. The following linearlization form is applied in this study:

$$|v|v = \sqrt{8/\pi}\sigma_v v \tag{27}$$

in which σ_v is the root mean square velocity given by:

$$\sigma_v^2 = \int_0^\infty S_{vv}(\omega)d\omega. \tag{28}$$

Thus, the wave force per unit length can be given in the form

$$p(x,t) = \alpha\dot{v}(x,t) + \beta\sigma_v v(x,t) \tag{29}$$

in which

$$\alpha = \rho\pi\frac{d^2}{4}C_M \qquad \beta = \frac{1}{2}\rho dC_D\sqrt{8/\pi}. \tag{30}$$

The cross spectrum of the wave force at two locations can be given as

$$\begin{aligned} S_{p_1p_2}(x_1,x_2,\omega) &= \alpha^2 S_{\dot{v}_1\dot{v}_2}(x_1,x_2,\omega) + \alpha\beta\sigma_v S_{\dot{v}_1v_2}(x_1,x_2,\omega) \\ &\quad + \beta\alpha\sigma_v S_{v_1\dot{v}_2}(x_1,x_2,\omega) + \beta^2\sigma_v^2 S_{v_1v_2}(x_1,x_2,\omega) \end{aligned} \tag{31}$$

Note that

$$S_{v_1\dot{v}_2}(x_1,x_2,\omega) = i\omega S_{v_1v_2}(x_1,x_2,\omega) = S^*_{\dot{v}_1v_2}(x_1,x_2,\omega) \tag{32}$$

$$S_{\dot{v}_1\dot{v}_2}(x_1,x_2,\omega) = \omega^2 S_{v_1v_2}(x_1,x_2,\omega) \tag{33}$$

in which * denotes the complex conjugate, and

$$S_{v_1v_2}(x_1,x_2,\omega) = S_{vv}(\omega)coh(x_1,x_2,\omega). \tag{34}$$

Thus, Eq. 31 can be written in the form:

$$S_{p_1p_2}(x_1,x_2,\omega) = (\alpha^2\omega^2 + \beta^2\sigma_v^2)S_{vv}(\omega)coh(x_1,x_2,\omega) \tag{35}$$

in which

$$S_{vv}(\omega) = \omega^2 \frac{S^u_{\xi\xi}(\omega)}{\sinh^2(kh)} \cosh^2[k(z+h)] \int_{-\pi}^{\pi} D(\omega,\theta)\sin^2\theta d\theta \tag{36}$$

and

$$coh(x_1, x_2, \omega) = \frac{\int_{-\pi}^{\pi} D(\omega,\theta)\sin^2\theta \exp\{ik(x_1 - x_2)\cos\theta\} d\theta}{\int_{-\pi}^{\pi} D(\omega,\theta)\sin^2\theta d\theta} \tag{37}$$

The total load on the element, $P(t)$, is given by

$$P(t) = \int_0^L p(x,t)dx \tag{38}$$

and its power spectral density, $S_{PP}(\omega)$, is given by

$$S_{PP}(\omega) = \int_0^L \int_0^L S_{p_1 p_2}(x_1, x_2, \omega) dx_1 dx_2 = (\alpha^2\omega^2 + \beta^2\sigma_v^2) S_{vv}(\omega) J^2(\omega) L^2 \tag{39}$$

in which the $J(\omega)$ is the joint acceptance function and takes the form

$$\begin{aligned} J^2(\omega) &= \frac{1}{L^2}\int_0^L \int_0^L coh(x_1, x_2, \omega) dx_1 dx_2 \\ &= \frac{2}{(kL)^2} \frac{\int_{-\pi}^{\pi} D(\omega,\theta)\tan^2\theta[1 - \cos(kL\cos\theta)] d\theta}{\int_{-\pi}^{\pi} D(\omega,\theta)\sin^2\theta d\theta}. \end{aligned} \tag{40}$$

For the approximating coherence function model, the spectrum of the total load on the element, $S^u_{PP}(\omega)$, is given by

$$S^u_{PP}(\omega) = (\alpha^2\omega^2 + \beta^2\sigma^2_{v^u}) S^u_{vv}(\omega) J^2_u(\omega) L^2 \tag{41}$$

in which the $J_u(\omega)$ is the joint acceptance function and takes the form

$$\begin{aligned} J_u^2(\omega) &= \frac{1}{L^2}\int_0^L \int_0^L coh_u(x_1, x_2, \omega) dx_1 dx_2 \\ &= \frac{1}{Q_a^2}[\exp(Q_a) - 1 - Q_a] + \frac{1}{Q_b^2}[\exp(-Q_b) - 1 + Q_b] \end{aligned} \tag{42}$$

and

$$Q_a = (i\cos\theta_0 - \frac{C_x}{2\pi})kL \qquad Q_b = (i\cos\theta_0 + \frac{C_x}{2\pi})kL \tag{43}$$

and

$$\sigma^2_{v^u} = \int_0^\infty S^u_{vv}(\omega) d\omega. \tag{44}$$

When the mean wave is normal to the element, i.e., $\theta_0 = \pi/2$, then

$$J_u^2(\omega) = \frac{2}{Q_0^2}[exp(-Q_0) + Q_0 - 1] \tag{45}$$

in which

$$Q_0 = \frac{C_x}{2\pi} kL. \tag{46}$$

Numerical Results

The following values of the parameters are used for this calculation unless otherwise stated:

Mean wind velocity: $U = 40$ m/s
Water depth: $h = 100$ m
Mean wave direction relative to the x-axis: $\theta_0 = \pi/6, \pi/3, \pi/2$
Element position: parallel to the x-axis and $z = -100$ m
Diameter of the element: $d = 1.0$ m
Inertia and drag coefficients: $C_D = 2$ and $C_M = 1.4$
Length of the element: $L = 50$ m, 100 m, 150 m, 200 m, 250 m

The unidirectional surface wave spectra at mean wind velocity of 40 m/s is shown in Fig. 2. Figure 3 shows the relationship between the wave number k and frequency ω. The direction spreading function $G(\omega, \theta)$ when $\theta_0 = \pi/2$ is shown in Fig. 4. $D(\omega_p, \theta)$ is also given in Fig. 4.

Figures 5 and 6 show the comparison results of the power spectral density of the water particle velocity component in the y-direction for the unidirectional and directional wave at different mean wave angles (Eqs. 15 and 17). The result is also given assuming that $D(\omega, \theta) = D(\omega_p, \theta)$. It is noted that the wave energy along the mean wind direction is less than that of the unidirectional wave system (Fig. 6, $\theta_0 = \pi/2$).

Figures 7 and 8 show the coherence and joint acceptance functions obtained from the directional and unidirectional system. It is noted that for the spreading wave system, the coherence and joint acceptance functions can not be taken only as functions of the dimensionless parameter kL. They are also functions of frequency, since the spreading function is a function of frequency. When the spreading function is approximated as $D(\omega_p, \theta)$, they become functions only of the parameter kL as in the approximated coherence function model.

Figure 9 shows the joint acceptance function for the elements with lengths of 100 m and 200 m. For a given length, the joint acceptance function is only a function of frequency.

Figure 10 shows the power spectra of the wave force on the horizontal elements with lengths of 100 m and 200 m.

Figures 11 and 12 show the root mean square values of the wave velocity and wave force on the element.

Concluding Remarks

This is the current summary of our work to date. We would use the developed methodology for specific conditions in the Bay of Campeche in the coming months following a meeting with Professor Robert Bea.

Table 1: RMS of the water velocity and wave force on element

	$\sigma_v(m/s)$	$\sigma_P/L \quad (kgf/m)$				
		L=50m	L=100m	L=150m	L=200m	L=250m
Mean wave direction $\theta_0 = \pi/2$						
Unidirectional wave	1.9962	650.9	650.9	650.9	650.9	650.9
$C_x = 4$	1.9962	622.6	596.9	573.7	552.4	533.0
$C_x = 8$	1.9962	596.9	552.4	515.2	483.5	456.4
Directional wave	1.8180	516.4	514.9	512.5	509.1	505.1
Directional wave $(D(\omega_p, \theta))$	1.8629	553.6	552.2	550.0	547.0	543.2
Mean wave direction $\theta_0 = \pi/3$						
Unidirectional wave	1.7287	487.7	484.6	479.5	472.5	463.7
$C_x = 4$	1.7287	466.6	445.1	424.7	405.5	387.6
$C_x = 8$	1.7287	447.5	412.5	382.7	357.3	335.6
Directional wave	1.6277	422.1	419.8	416.2	411.3	405.2
Directional wave $(D(\omega_p, \theta))$	1.6527	442.1	439.6	435.7	430.3	423.8
Mean wave direction $\theta_0 = \pi/6$						
Unidirectional wave	0.9981	163.4	160.3	155.3	148.7	140.7
$C_x = 4$	0.9981	156.4	147.7	138.9	130.4	122.4
$C_x = 8$	0.9981	150.0	137.2	126.0	116.5	108.3
Directional wave	1.1554	225.5	223.1	219.2	214.0	207.7
Directional wave $(D(\omega_p, \theta))$	1.1194	205.0	202.6	198.7	193.5	187.2

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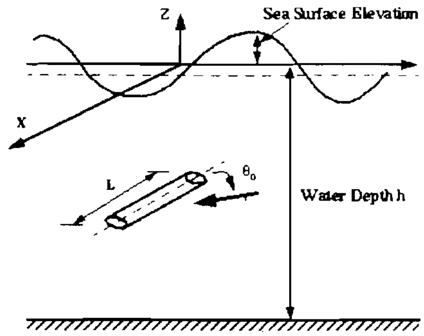


Fig.1 Cartesian coordinate system

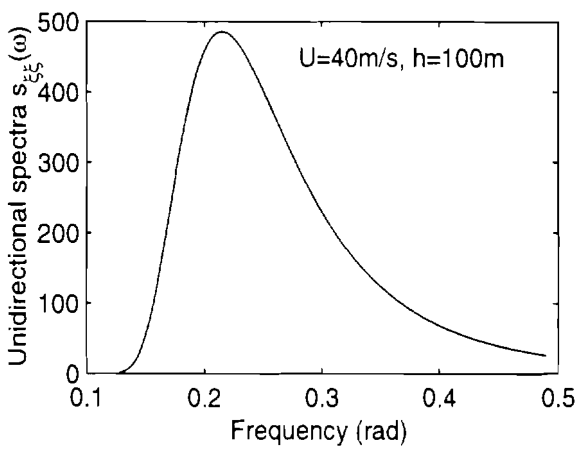


Fig.2 Unidirectional surface water spectrum

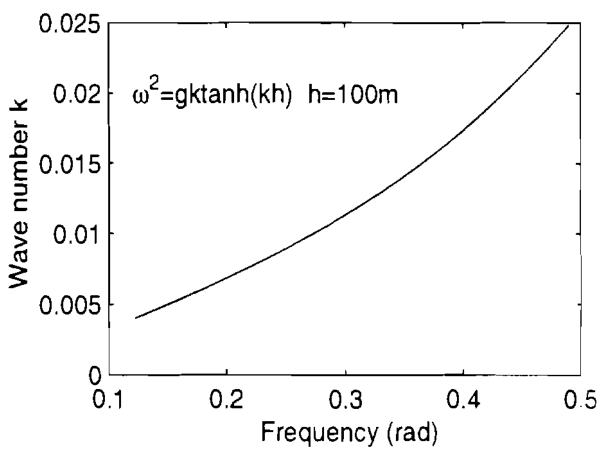
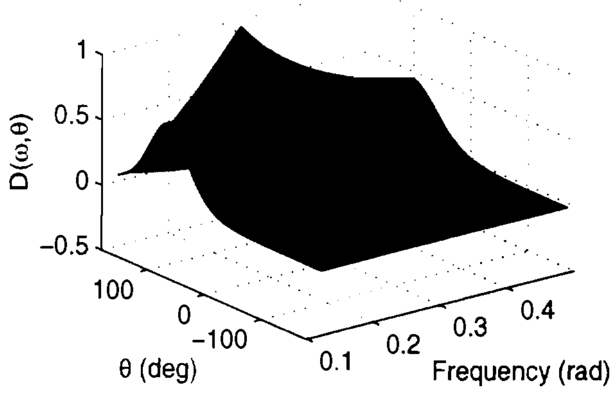


Fig.3 Relationship between k and ω



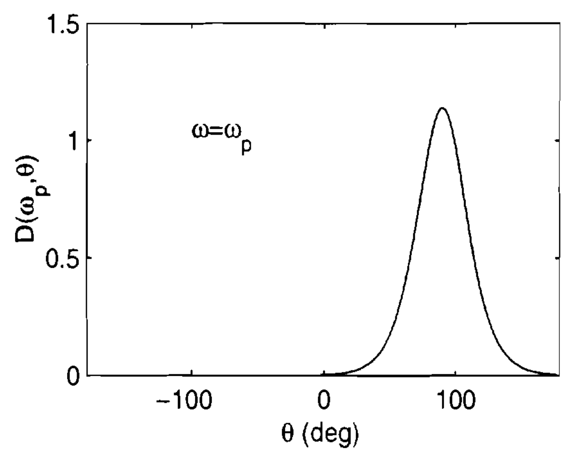


Fig.4 Directional spreading function $G(\omega, \theta)$ with $\theta_0 = \pi/2$

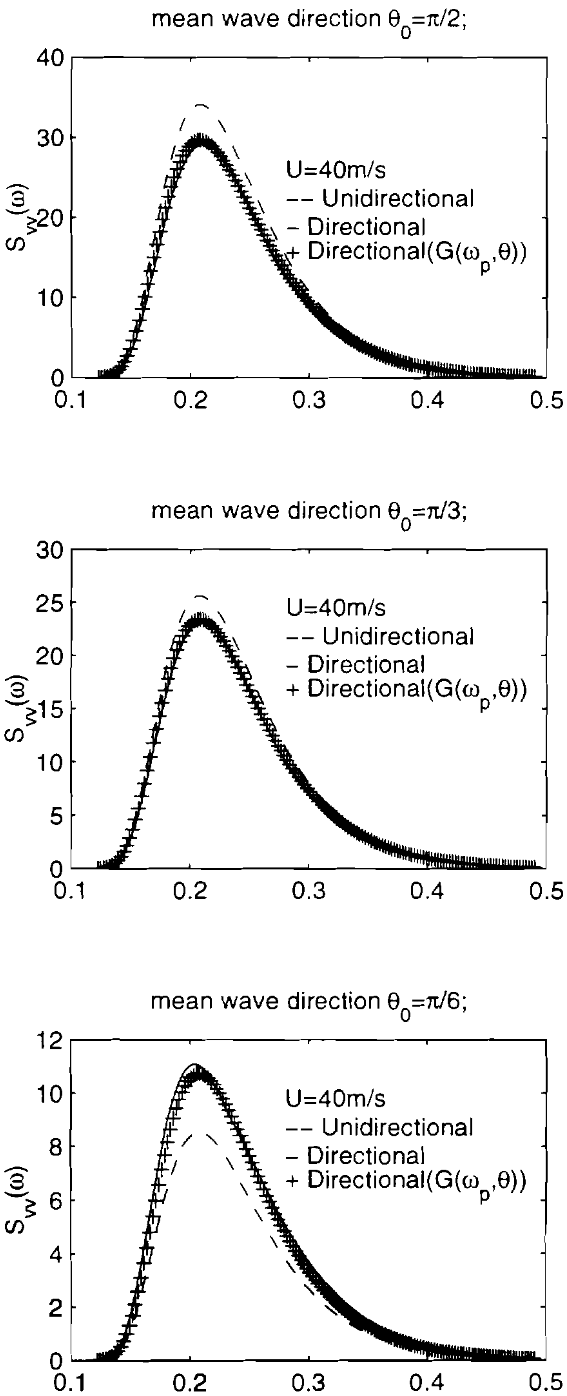


Fig.5 Power spectra of water particle velocity

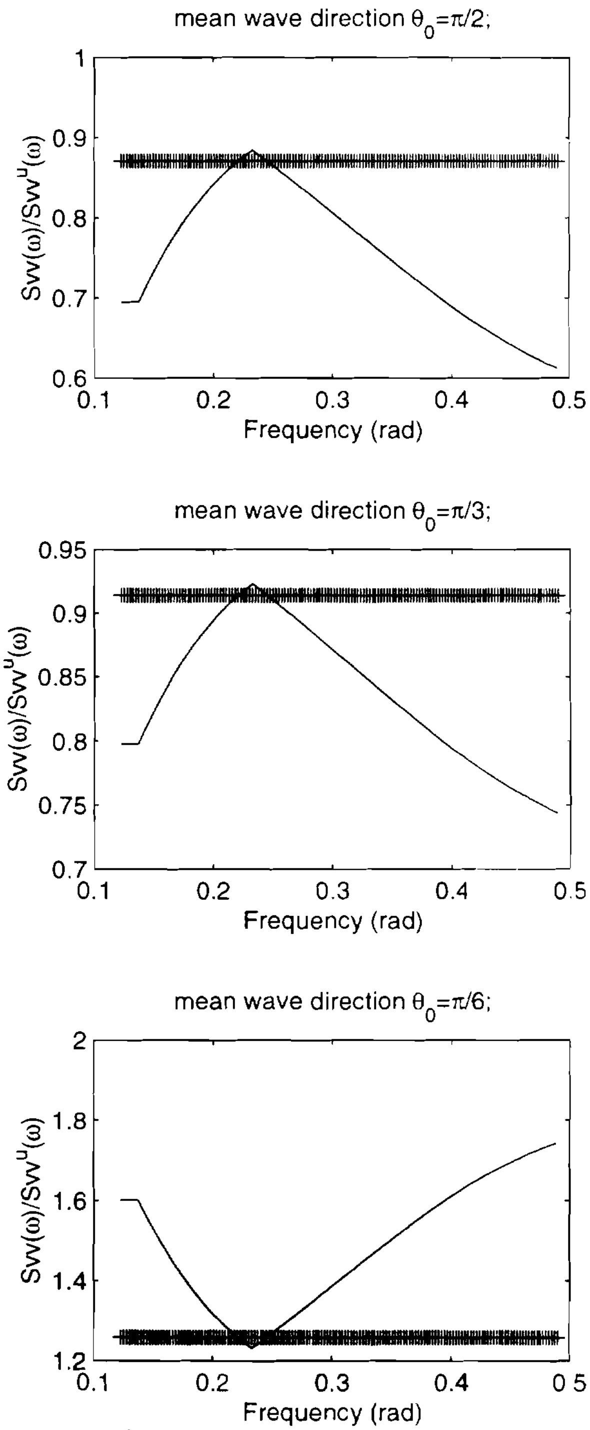


Fig.6 Ratio of directional and unidirectional spectra

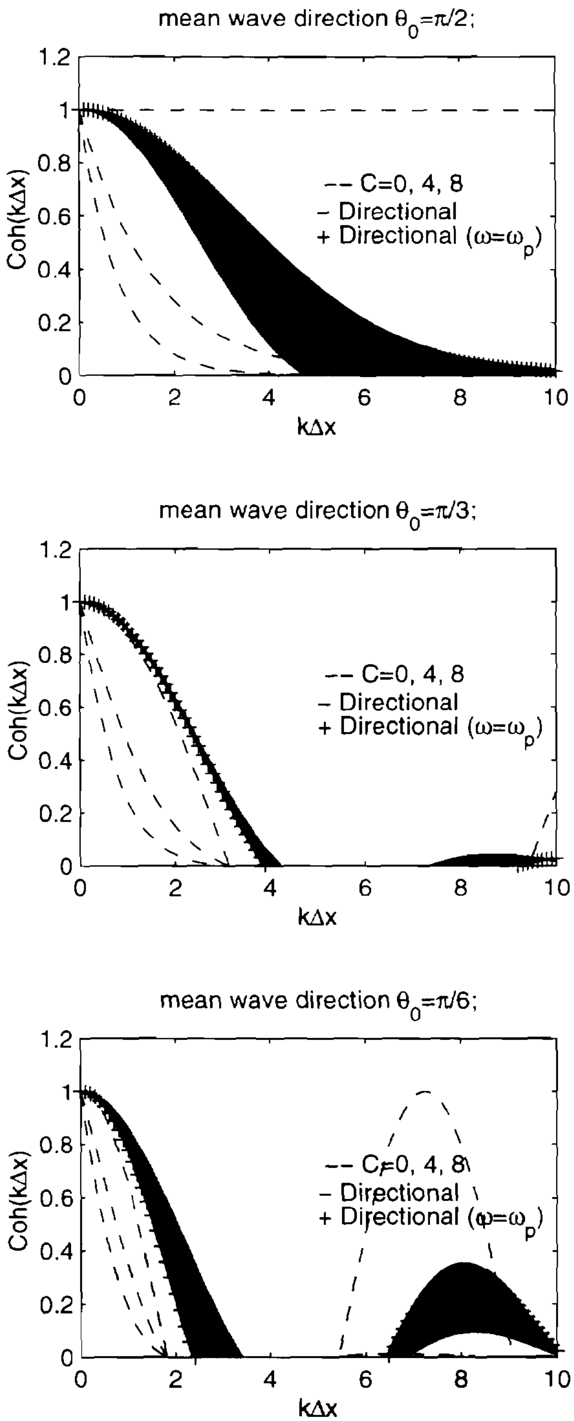


Fig.7 Coherence function at different mean wave direction

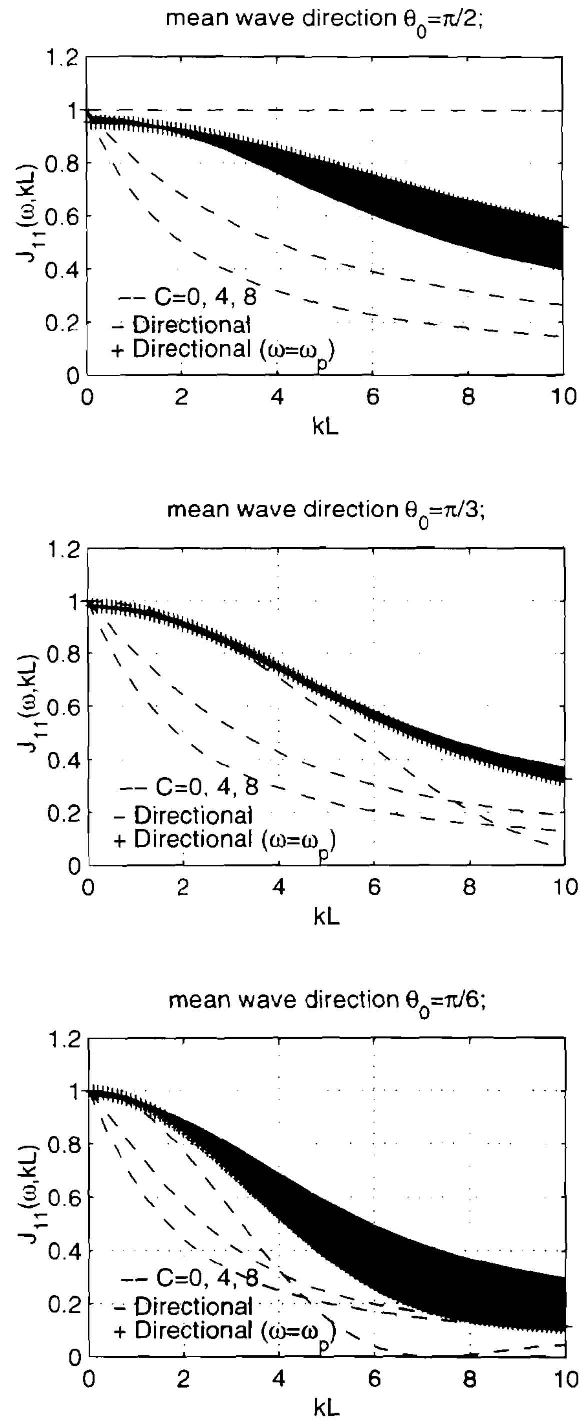


Fig.8 Joint acceptance function at different mean wave direction

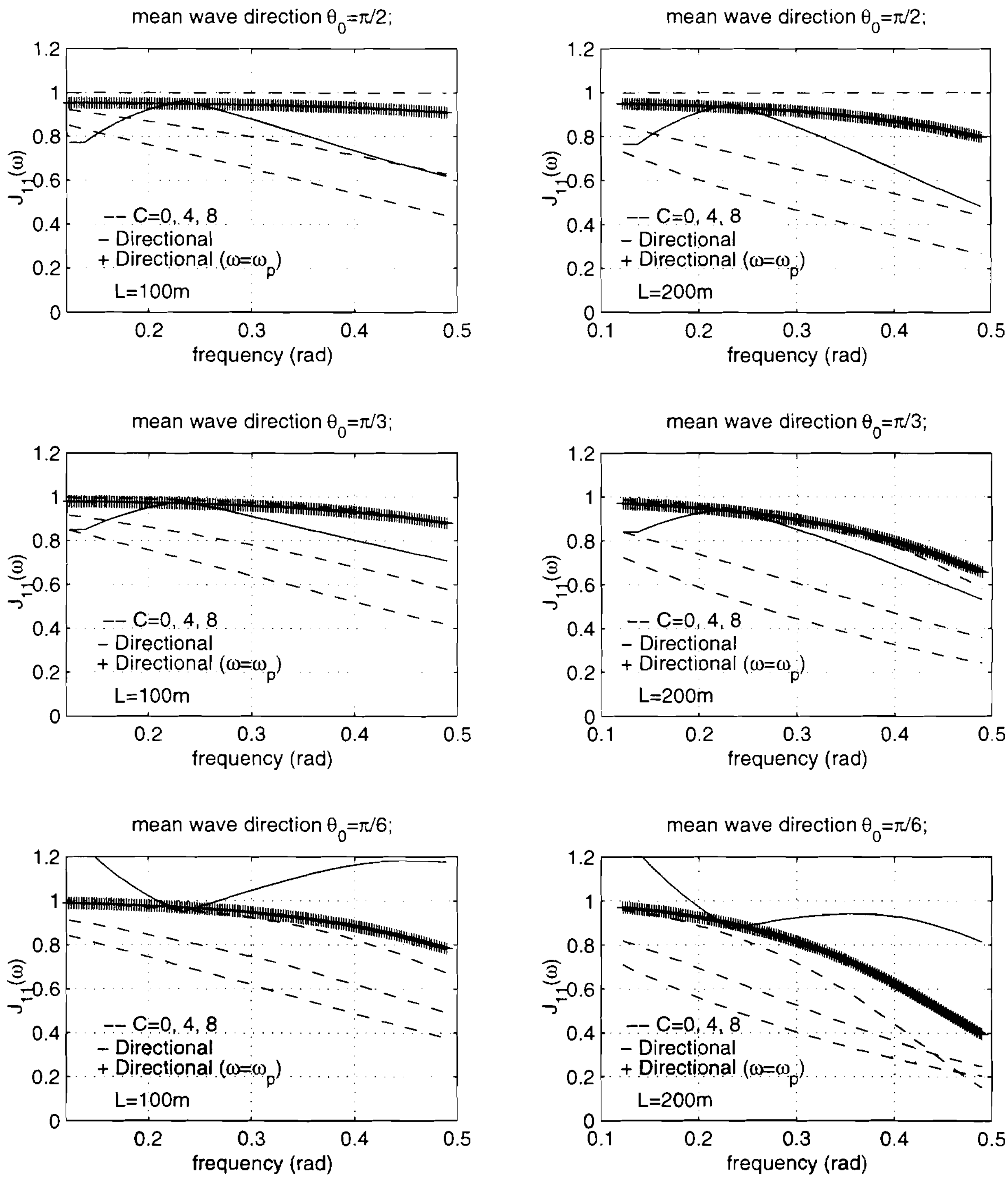


Fig.9 Joint acceptance function for the elements with different lengths of elements.

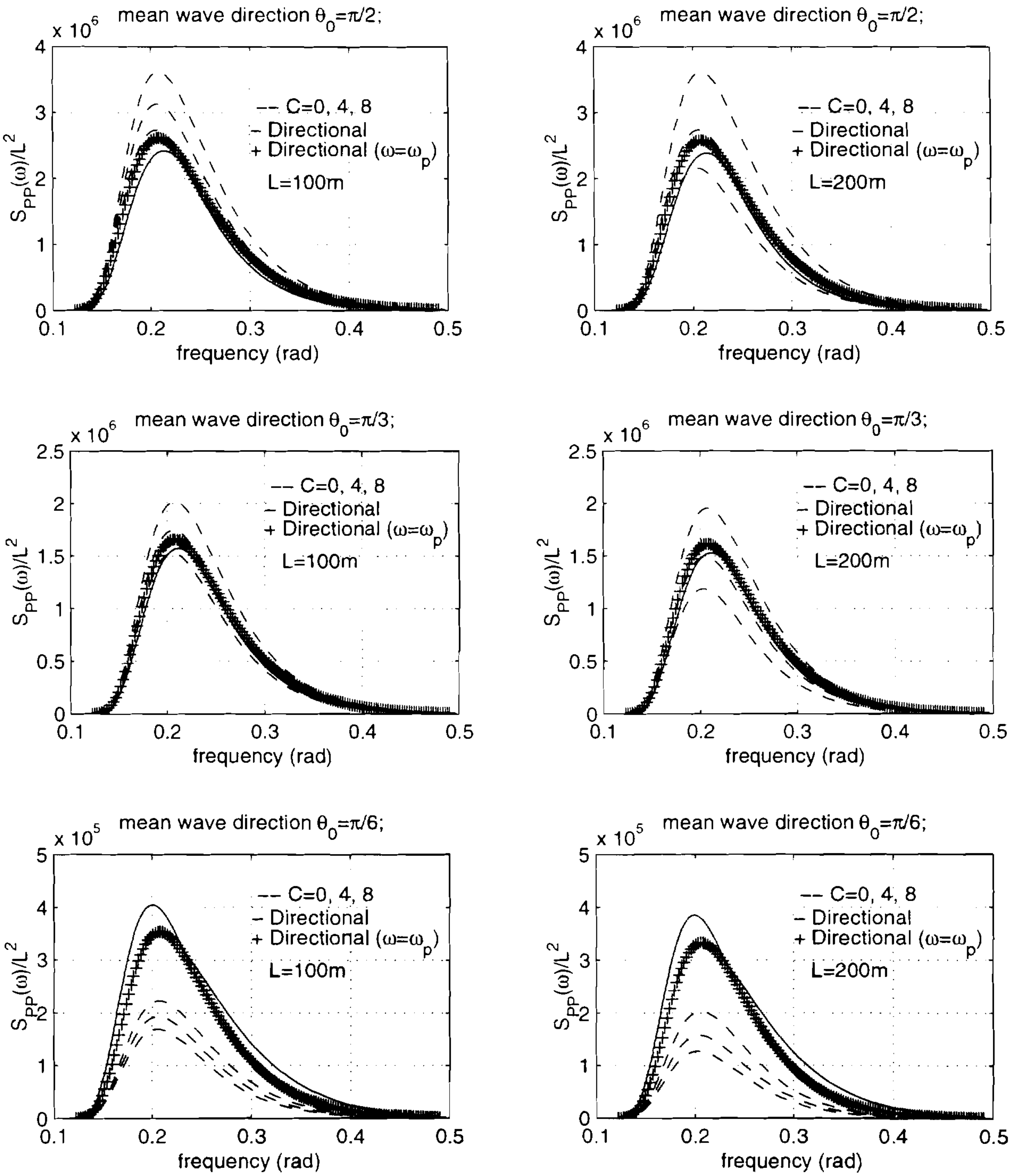


Fig.10 Power spectra of wave force on the horizontal element

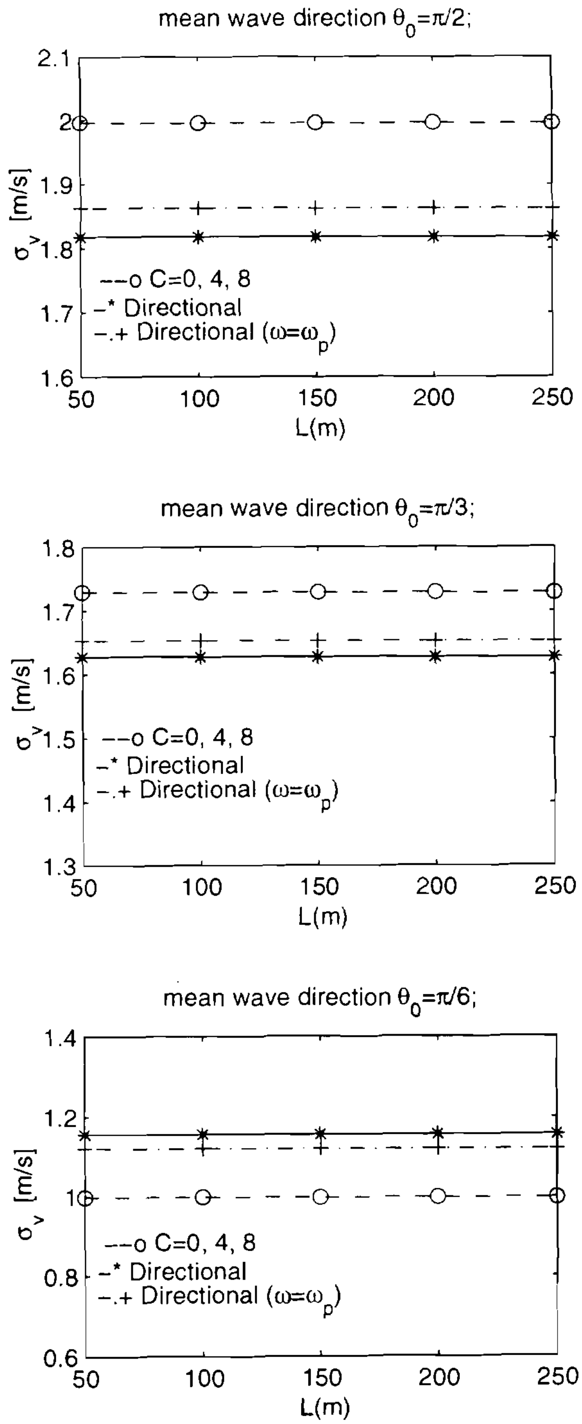


Fig.11 RMS values of wave velocity

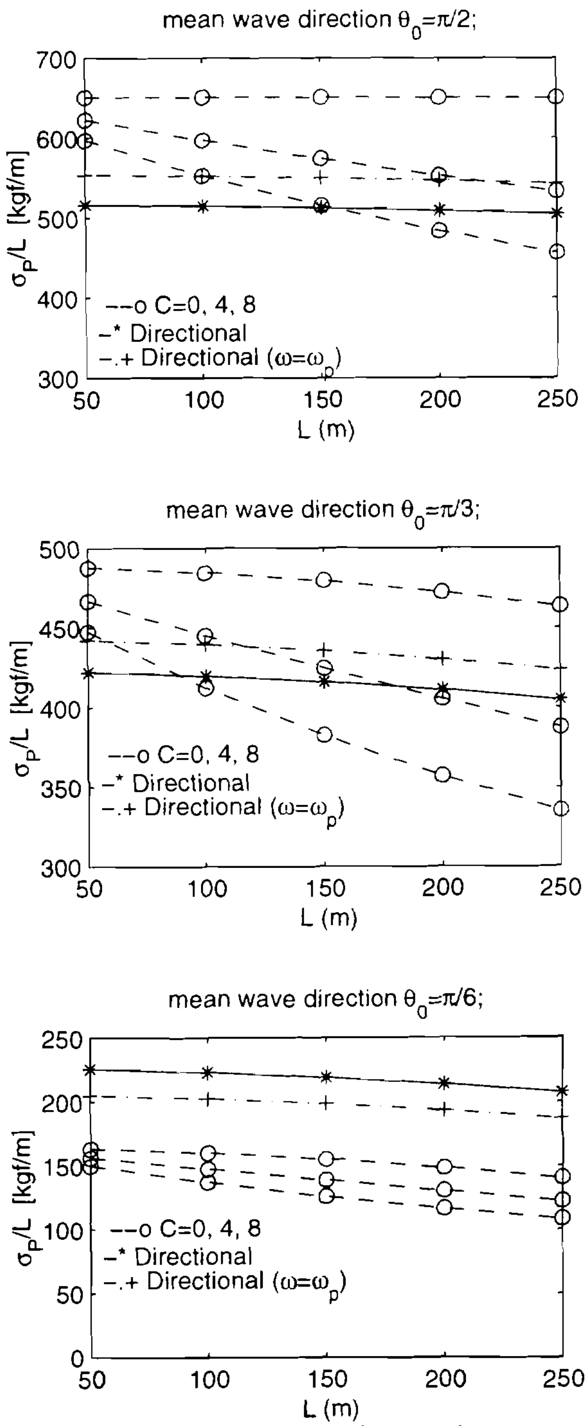


Fig.12 RMS values of wave force on the element