RAM PIPE REQUAL
Pipeline Requalification Guidelines Project
Report 1

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

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Ocean Engineering Graduate Program

Marine Technology & Management Group

University of California at Berkeley

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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:

1) 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,
2) Definition of target reliabilities for different SSC of marine risers and pipelines,
3) Guidelines for assessment of pressure containment given corrosion and local damage including guidelines for evaluation of corrosion of non-piggable pipelines,
4) Guidelines for assessment of local, propagating, and global buckling of pipelines given corrosion and local damage,
5) Guidelines for assessment of hydrodynamic stability in extreme condition hurricanes, and
6) 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:

1) Review of the criteria and guidelines by an international panel of consulting engineers,

2) 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,

3) Provision of a scholarship to fund the work of graduate student reserarchers that assisted in performing this project, and

4) 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:

1) Serviceability and Safety Classifications (SSC) of pipelines and risers,

2) Guidelines for analysis of in-place pipeline loadings (demands) and capacities (resistances), and

3) 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.

The first international workshop was held in Mexico City on September 10, 1998. This workshop was attended by approximately 300 engineers, managers, operators, consultants, and contractors. The program included presentations on the background, validation, and content of the first Transitory Criteria for the Design and Evaluation of Submarine Pipelines in the Bay of Campeche. Presentations and discussions were made during this conference by Dr. Charles Smith and Mr. Alex Alvarado of the U. S. Minerals Management Service. Proceedings documenting the workshop presentations were issued by PEMEX and IMP.

The second international workshop was held in Cuidad del Carmen, Campeche, Mexico on October 21 and 22, 1998. This workshop was attended by approximately 300 engineers, managers, operators, consultants, and contractors. The program included presentations on the background, validation, and content of the first Transitory Criteria for the Design and Evaluation of Submarine Pipellines in the Bay of Campeche. These presentations emphasized operating issues of importance to the PEMEX field operations and engineering personnel that attended this conference. In addtiion, the program included presentations on the background of offshore platform operations in the northern Gulf of Mexico, offshore California, and in the Bay of Campeche. These presentations emphasized deep water development challenges and issues, advanced technology and research work on offshore pipeline systems by PETROBRAS, and decommissioning offshore platforms. Presentations and discussions were made during this conference by Mr. Felix Dyhrkopp, Mr. B. J. Kruse, and Ms. S. Buffington of the U. S. Minerals Management Service. Proceedings documenting the presentations at this workshop were issued by PEMEX and IMP.

The U. S. Minerals Management Service and several industrial organizations sponsored a two-day international workshop on Risk Assessment & Management of Marine Pipeline Systems on November 5 and 6, 1998, in Houston, Texas. This workshop was hosted by Amoco Corporation and was attended by 70 invited participants and speakers from the U. S., Canada, Norway, and the United Kingdom. The workshop included presentations on risk based management of onshore pipeline systems in the U. S. and Canada, risk based inspections of refinery pipeline systems, and offshore platform and pipeline systems. Specialized presentations were made on qualitative, quantitative, and mixed methods to assess the risks associated with pipeline systems, and on corrosion, third party damage, and other hazards to pipeline systems. This workshop included presentations and discussions by Dr. Charles Smith and Mr. Alex Alvarado of the U. S. Minerals Management Service.

Of particular importrance to this project, was a workshop concluding panel discussion lead by Mr. Ray Ayres of Shell Pipeline Company. This panel was comprised of industry and government representatives. The panel addressed issues associated with data collection, the value of information, trade-offs in mitigation of pipeline risks, how to best use the technologies developed for onshore pipeline systems, high priority additional studies that need to be conducted to advance the technology associated with risk assessment and management of marine pipelines, and development of demonstration programs associated with risk based management of marine pipelines.

This panel and workshop concluded that it would be in the interests of the industry and government to develop guidelines and criteria for the requalification and reassessment of marine pipelines. The panel advanced the need to form a committee to define the problems and issues that need to be addressed, development of industry leadership to carry forward the results from technology development projects (e.g. American Petroleum Institute), and development of a management 'case for action.'

The management 'case for action' was cited as being particularly important to allow advancement of criteria and guidelines for the requalification and reassessment of marine pipeline systems. This case for action would need to address the economic benefits of better understanding and management of pipeline performance, a statement of action being taken by other industries to improve risk assessment and management of pipeline systems (e.g. Department of Transportration, American Petroleum Institute, Classification Societies), a statement of objective to develop performance based regulations, and a statement of the need to provide improved safety and environmental protection for aging pipeline infrastructures. Proceedings documenting the presentations and panel deliberations from this conference will be issued early in 1999.

These three workshops provided excellent international forums to discuss the technical and political aspects associated with development of requalification and reassessment processes for marine pipeline systems. These workshops provided a wealth of information that was and will be utilized in this project.

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

7) 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 will be completed during July, 1999. 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:

8) **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).

9) **Report 2 –** Pipeline characteristics, Hydrodynamic Stability, Geotechnical Stability, Guidelines for Assessing Capacities of Defective and Damaged Pipelines.

10) **Report 3 –** Guidelines for Assessing Pipeline Stability (Hydrodynamic, Geotechnical), Preliminary Requalification Guidelines.

11) **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:

1) **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.
2) 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.
3) 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 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.

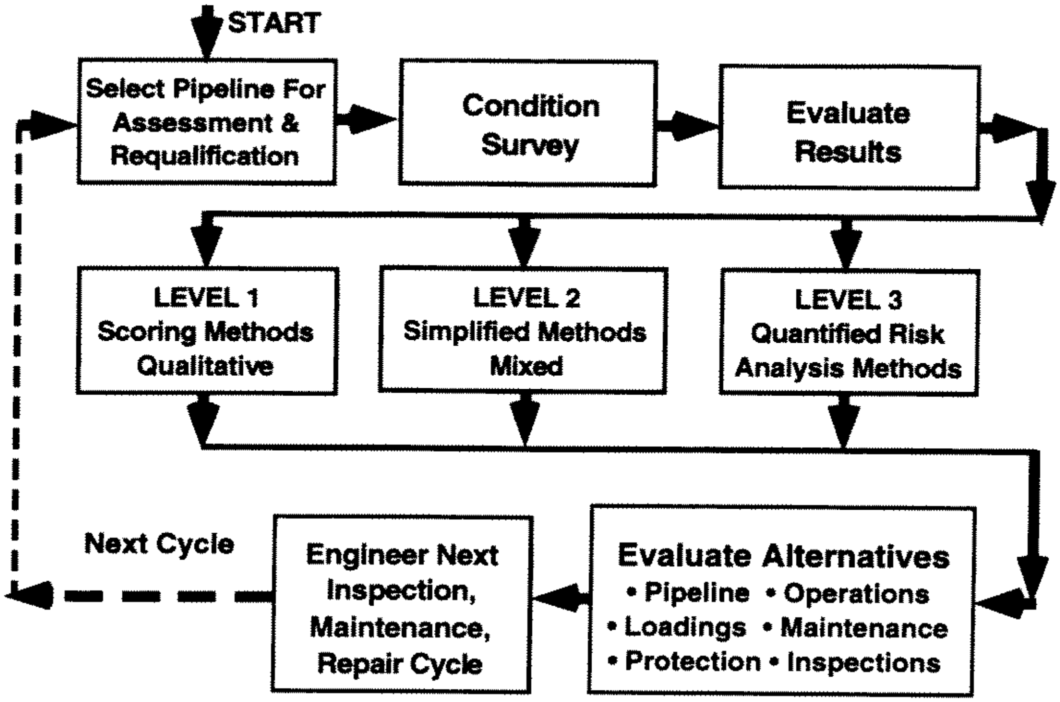

Figure 2.1 – RAM PIPE Approach

- **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 differentes between design of anew pipeoline and the reassessment of an existing pipeline
- Apply a decision-theoretic framework and sound engineering judgement
- Utilize an approachin 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 RAM Background

3.1 Working Stress Format

A traditional format for design guidelines has been the Working Stress Design (WSD) format. This format utilizes a nominal 'static' loading to define the serviceability response characteristics and strength of the structure. Linear elastic analyses are used to describe the structure response characteristics for the given nominal design loadings. Based on characterization of the demands and capacities as being Lognormally distributed, the traditional factor-of-safety (FS) in working stress design can be expressed as:

$$FS = Fe\ (B_S / B_R) \exp [(\beta\, \sigma) - (2.33\, \sigma_S)]$$

where Fe is a factor that incorporates the interactive effects of dynamic - transient loadings and the nonlinear behavior of the system (Bea, 1996), B_S is the median Bias in the maximum demand (loading), B_R is the median Bias in the capacity of the element, β is the annual Safety Index (Reliability target or goal), σ is the total uncertainty in the demands and capacities (standard deviation of the logarithms of R and S), and σ_S is the uncertainty in the annual expected maximum loadings.

The 2.33 refers to 2.33 standard deviations from the mean value, or the 99th percentile. This is equivalent to the reference of the design loading to an average annual return period of 100 years. In the case of seismic loadings, a 200-year return period is often used (99.5 percentile) and a value of 2.57 would be used in the foregoing expression. In case of installation conditions defined on the basis of a 10-year return period condition, a value of 1.28 would be used (90-th percentile).

The Safety Index β is related approximately to the Probability of Failure Pf as:

$$Pf \approx 0.475 \exp -(\beta)^{1.6}$$

$$Pf \approx\approx 10^{-\beta}$$

The Safety Index can be thought of as a type of Factor of Safety; as b gets bigger, the system gets more reliable, or Pf gets smaller.

The total uncertainty in the demands S and capacities R is determined from:

$$\sigma = \sqrt{\sigma_S^2 + \sigma_R^2}$$

where σ_S is the uncertainty (standard deviation of the logarithms) in the annual maximum demands and σ_R is the uncertainty in the capacities of the elements.

The FS is the ratio of the design capacity of a structure element (R_D) to the design or reference demand (S_D):

$$FS = R_D / S_D$$

In the WSD format, the design equation is formulated as:

$$R_D \geq FS\ S_D$$

or:

$$R_D / FS \geq S_D$$

Alternatively, the factor of safety could be referenced to the capacity of the entire structural system and thus reflect the aggregated effects of the elements that comprise the structural system. The Reserve Strength Ratio (RSR) is now generally used for this purpose and:

$$RSR = Fe\,(B_S / B_R) \exp[(\beta\,\sigma) - (2.33\,\sigma_S)]$$

where all of the parameters refer to the global demands and capacities developed on and in the structural system. As before, in this expression the design loading S_D is defined at a 100-year return period.

The transient / dynamic loading - nonlinear performance factor, Fe, is dependent on the ductility (strain - deformation capacity), residual strength (load - stress capacity beyond yield), and hysteretic (cyclic load - deformation - damping behavior) characteristics of the structure (Bea, Young, 1993; Bea, 1996). It is also dependent on the transient / dynamic loading characteristics including the duration of the imposed or induced loadings, the periodicity, and the force-time characteristics of the loadings.

Generally, the 'true' ultimate capacity of the element (Ru) is not used in the design process, and another 'nominal' or design capacity (R_D) is used. The capacity Bias is introduced to recognize this difference:

$$B_R = Ru / R_D$$

In a similar manner, the loading Bias is introduced to recognize the difference between the design or nominal demand (S_D) and the 'true' maximum demand (S_M):

$$B_S = S_M / S_D$$

The results of the foregoing developments are summarized in Figure 3.1 for Biases in the demands and capacities of unity and for a 100-year design loading condition. In developing these results it has been assumed that the total uncertainty is equal to the loading uncertainty. This is equivalent to assuming that the resistance or capacity uncertainty is negligible compared with the demand uncertainties (generally, this is a very good approximation).

Some interesting trends are indicated in Figure 3.1. For low Safety Indices ($\beta \leq 2.5$), the FS and RSR are about unity and there is little significant variation in these parameters with uncertainty in the demands and capacities. For high Safety Indices ($\beta \geq 3.5$), the FS and RSR are very sensitive to the uncertainties.

For low uncertainties ($\sigma \leq 0.2$), there is little change in the FS and RSR as a function of the target reliability expressed in the Safety Indices. This will be important for pipelines, because the uncertainties associated with their demands and capacities generally fall in the range of $\sigma \leq 0.2$. A primary exception to this generalization will be for the cases associated with damaged (dented, corroded) pipelines. The uncertainties associated with these cases will generally exceed $\sigma = 0.2$ because of the additional uncertainties due to the high variabilities associated with corrosion and denting processes and with the effects of these

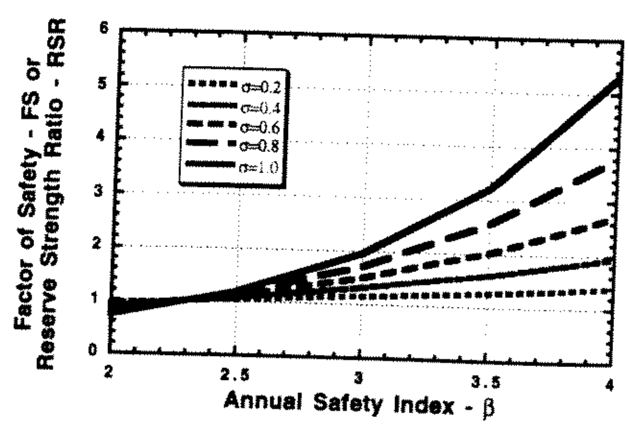

Figure 3.1 - Element Factor of Safety or System Reserve Strength Ratio as Function of Total Uncertainties for Demand and Capacity Biases of Unity

highly variable processes on the pipeline capacities.

3.2 Load and Resistance Factor Design

Another format that is being used to design pipeline systems is known as the Load and Resistance Factor Design (LRFD) format. This format utilizes a load factor (γ, generally greater than unity), and a resistance factor (ϕ, generally less than unity) as follows:

$$\phi R_D > \gamma S_D$$

Thus, the design loading S_D is factored up, and the design resistance R_D is factored down. Generally, the factoring is done such that the design engineer is still able to use linear elastic analysis methods in design computations.

To allow the load and resistance factors to be proportioned according to the uncertainties in the loading and resistance, the following approximation can be used:

$$c = \sqrt{a2 + b2} \approx 0.8\,(a + b)$$

Based on this approximation, the RAM approach can be used to determine the loading and resistance factors:

$$\gamma = Fe\, B_S \exp\,(0.8\, \beta\, \sigma_S - 2.33\, \sigma_S)$$

$$\phi = B_R \exp\,(-0.8\, \beta\, \sigma_R)$$

If the design loading, S_D, were composed of two components: Sd_D (for dead loading) and Ss_D (for storm loading), then:

$$\phi R_D > \gamma_d\, Sd_D + \gamma_s\, Ss_D$$

$$\gamma_d = B_d \exp\,(0.8\, \beta\, \varepsilon\, \sigma_d)$$

$$\gamma_s = B_S \exp\,(0.8\, \beta\, \varepsilon\, \sigma_s)$$

where the "splitting coefficient", ε, can be determined from:

$$\varepsilon = \frac{\sqrt{\sigma_d^2 + \sigma_s^2}}{(\sigma_d + \sigma_s)}$$

The results of the foregoing developments are summarized in Figure 3.2 and Figure 3.3 for the loading and resistance factors, respectively. As for the WSD illustration, it has been assumed that the Biases in the loading and resistance elements are unity, and that a 100-year design loading has been referenced.

Some interesting observations can be developed from these developments. High uncertainties and high Safety Indices imply large loading factors and small resistance factors. The 'switch-over' in the loading factors for Safety Indices less than about 2.9 is due to the use of the 100-year return period design reference loading. For example, for a loading uncertainty of 0.8 and a Safety Index of 2.5, a loading factor of 0.77 results from this formulation. However, if a 50-year return period design reference loading is used for the same loading uncertainty and Safety Index, a loading factor of 0.96 is developed. The reference return period used to define the design loading condition has an important influence on the load and resistance factors and the factors of safety.

These results could be verified as follows. For a Safety Index of 2.5, a 100-year return period design loading, and a loading uncertainty of 0.8, a loading factor of 0.77 is found. For the same Safety Index and an uncertainty in the resistance of 0.3, a resistance factor of 0.55 is found. This indicates a total uncertainty of σ = 0.85. The Factor of Safety is indicated to be 1.3 for these values.

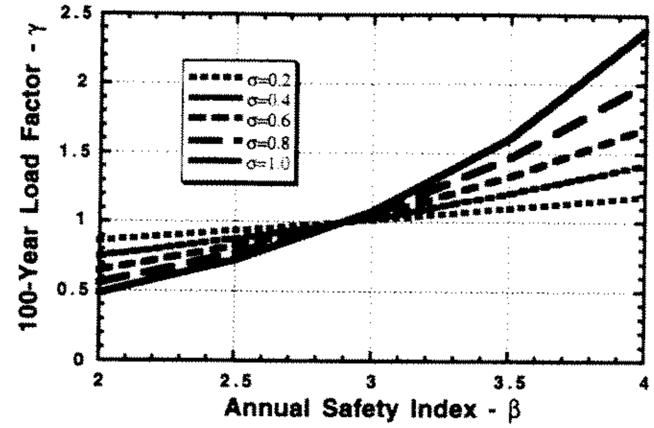

Figure 3.2- Loading Factor for 100-Year Design Loading As Function of Annual Safety Index and Loading Uncertainties

For the same conditions (100-year design loading, Biases of unity, annual Safety Index), the Factor of Safety is related to the Load and Resistance Factors as follows:

$$FS = \gamma / \phi$$

The results from the foregoing example indicate FS = 1.3 versus γ / ϕ= 1.4. The difference in result is due to the inaccuracies introduced by the splitting factor of 0.8. For these uncertainties, the use of a splitting factor of 0.777 would result in identical factors of safety.

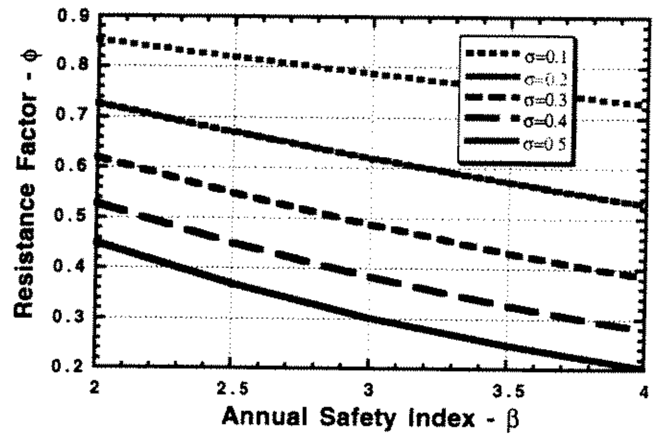

Figure 3.3 - Resistance Factor As Function of Annual Safety Index and Loading Uncertainties

Code 'calibrations' make very good use of the foregoing expression in development of LRFD codes. A 'reasonable' load factor is chosen based on judgment or 'precedent' or determined from analysis, then the resistance factor is determined based on the factor of safety contained or implied in the WSD. Alternatively, the Safety Indices implied in the WSD code are determined based on assessments of the Biases and uncertainties in the WSD element resistance formulations. 'Sticky' problems develop when the factors of safety are not explicit or the Biases are not the same in the two code formats and formulations.

3.3 Damaged and Defective Systems

One objective of LRFD developments has been to 'balance' the Safety Indices of the elements that comprise the system. The author does not advocate such a 'balance.' The needs to incorporate 'robustness' or damage and defect tolerance into structure system justify 'imbalances' in the Safety Indices of some of the elements.

While 'robustness' does not appear to be critical for an intact or otherwise 'perfect' structure system, when the inevitable defects and damage develops in important load carrying structure elements, robustness can pay dividends in maintaining the quality of the structure system at acceptable levels(Figure 3.4).

Robustness is achieved through a combination of:

- **redundancy** (excess load carrying paths),
- **ductility** (ability to absorb plastic strains without failure, ability to shift loads to other paths), and

- **excess capacity** (capacity in excess of that 'normally' needed for the intact system elements so that when loadings are shifted from damaged or defective members, that the other members are able to carry the necessary loadings).

It is precisely at this point that there can be major differences between 'robust' and 'minimum' structure systems. Both systems can have identical intact design strengths. The minimum structure will likely have a lower initial cost. However, when damage and defects are developed, there are major and sometimes catastrophic consequences in quality attributes of the minimum structures. Savings that would realized in the initial costs are quickly surrendered to the future costs associated with the loss of the quality attributes of the structure. This is one of the primary reasons that the RAM process is focused on the four attributes of quality and on the life-cycle of a structure system. There needs to be appropriate balances in the quality attributes and in the reliability with which these quality attributes are achieved throughout the life cycle of a structure system.

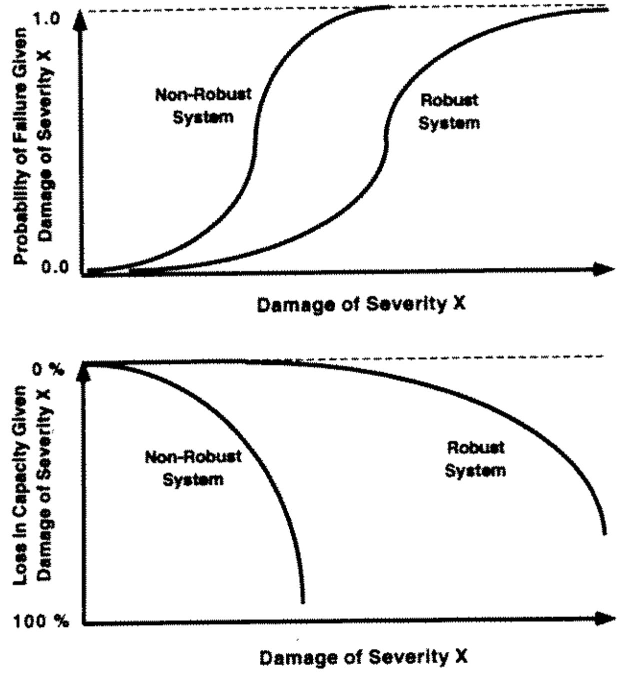

Figure 3.4 – Characteristics of non-robust and robust structure – pipeline – riser systems

This implies that the RAM design formulation that has been developed thus far has addressed only a part of the challenge of achieving safety in a structure - pipeline system.

3.4 Systems and Elements

In this development, references have been made to structure 'elements' and 'systems'. The design format developments were primarily focused on the elements that comprise a structure system (exception the Reserve Strength Ratio). The risk and reliability developments were focused on the performance characteristics of the structural system. This raises the question regarding the differences in the reliability formulation between the reliability of elements and the reliability of systems. The answer to this question is firmly embedded in the issues of how structures are designed and the correlations among the elements that comprise systems (Melchers, 1987).

A pipeline structure system can be decomposed into sub-systems of series and parallel elements (Thoft-Christensen, Baker, 1982; Melchers, 1987). A series sub-system is one in which the failure of one of the elements leads to the failure of the system. Examples of a series sub-system would be a pipeline that is comprised of joints of pipe (welded connections).

A parallel system is one in which the failure of the system only occurs when all of the elements have failed. An example of a parallel system would be several pipelines that serve the same distribution point, given that the pipelines can all carry the production required.

3.4.1 Parallel Systems

A parallel (redundant) system fails when all of the elements have failed. In probabilistic terms, the probability of failure of parallel element system can be expressed in terms of the probabilities of failure of its N elements as:

$$Pf_{system} = (Pf_1) \text{ and } (Pf_2) \text{ and } \ldots (Pf_N)$$

The strength characteristics of a parallel system are dependent on the ductility (deformation or strain capacity) and residual strength (load or stress capacity after the yield strength has been developed) characteristics of the elements that comprise the system. A ductile - high residual strength element is one which can continue to carry a major portion of its capacity when it reaches its yield-capacity strain (stress or load carrying ability). A ductile element is able to develop large plastic deformations without 'failure' (fracture, rupture). A perfectly brittle element ceases to carry any load as soon as its yield strain is reached.

The foregoing discussion can be visualized by examining the capacity of system comprised of perfectly elastic - plastic elements. The capacity of the system will be the sum of the capacities of the individual elements. This assessment is dependent on the ductility or strain capacity of the elements. They must be ductile enough to allow the loads to be redistributed to the other non-yielded elements after yield of the first element. In a similar way, given sufficient ductility, a system comprised of elements that have residual strengths that are less than the yield strength has a strength that is equal to the sum of the residual strengths. The residual strength and ductility of elements have major influences on the behavior of a system comprised of these elements (Cornell, 1987, Guenard, 1984).

A parallel system with N perfectly ductile elements, the expected capacity, **R**, of this system is determined by the sum of the expected capacities of the elements (i = 1 to N):

$$\mathbf{R} = \sum_{i=1}^{N} \mathbf{R}_i$$

If the capacities of the paired elements (i, j) are independent ($\rho_{ij} = 0.0$) and Normally distributed, the standard deviation of the system capacity, σ_R, can be expressed in terms of the standard deviations of the capacities of the elements, σ_i, as :

$$\sigma_R^2 = \sum_{i=1}^{N} \sigma_i^2$$

If the capacities of the elements are positively correlated, the standard deviation of the capacity of the system will increase, and the probability of failure of the system will increase.

An important conclusion that can be reached from these results is that if the degree of correlation between the capacities or the probabilities of failure of the parallel elements is high, then the probability of failure of the system is will be approximately the probability of failure of a single element. This is because, now the uncertainty associated with the capacity of the system can be determined from:

$$\sigma_R^2 = \sum_{i=1}^{N} \sigma^2_i + \rho ij \sigma i \sigma j$$

For high correlations between elements, as elements are added to the system, additional uncertainty is added at the same rate as the elements. Thus, for the case of high correlation, the probability of failure of the system becomes equal to the probability of failure of the most likely to failure (MLTF) element in the system.

3.4.2 Correlations

There can be a variety of ways in which correlations can be developed in elements and between the elements that comprise a system that have important ramifications on the performance characteristics of the elements, and consequently on the characteristics of the system itself. Important sources of 'correlations' include:

- **capacity and demand correlations**
- **element to element characteristics correlations**, and
- **failure mode correlations.**

The "correlation coefficient", ρ, expresses how strongly two variables, X and Y, are related to each other. It measures the strength of association between the magnitude of two variables. The correlation coefficient ranges between positive and negative unity ($-1 \leq \rho \leq +1$). If $\rho = 1$, they are perfectly correlated, so that knowing X allows one to make perfect predictions of Y. If $\rho = 0$, they have no correlation, or are 'independent,' so that the occurrence of X has no affect on the occurrence of Y and the magnitude of X is not related to the magnitude of Y. Independent random variables are uncorrelated, but uncorrelated random variables (magnitudes not related) are not in general independent (their occurrences can be related).

The correlation coefficient can be computed from data in which the results of n samples of X and Y are developed ($\underline{X}$ and $\underline{Y}$ are the mean values of the variables X and Y):

$$\rho = \frac{\sum XY - n\underline{X}\,\underline{Y}}{\sqrt{(\sum X^2 - n\underline{X}^2)(\sum Y^2 - n\underline{Y}^2)}}$$

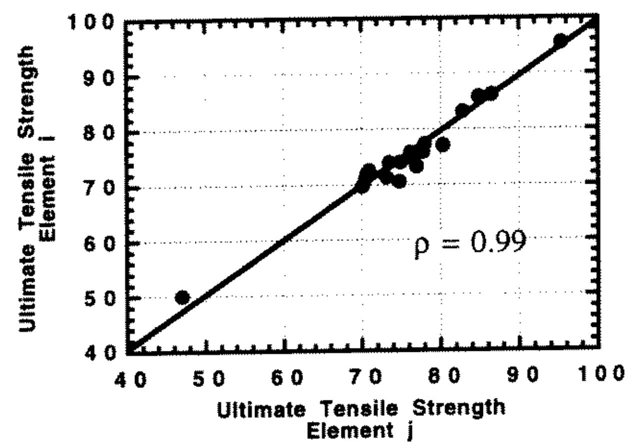

Figure 3.5 – Correlation of measured ultimate tensile strengths of paired pipeline steel samples

The term in the numerator of this expression is the covariance (COV) between the two marginal distributions, X and Y. The terms in the denominator are the standard deviations of the two distributions, X and Y.

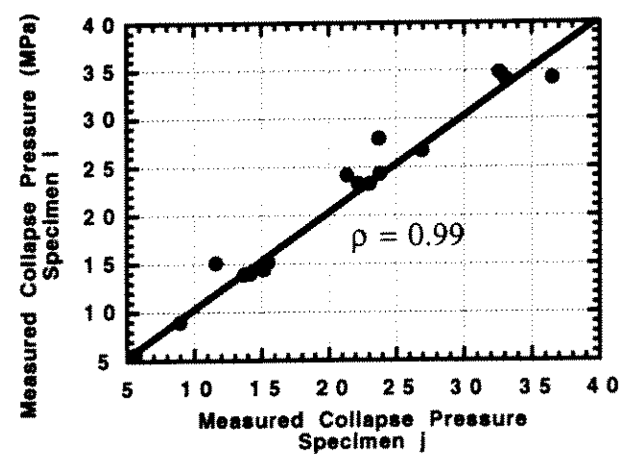

Figure 3.6 – Correlation of measured collapse strengths of paired steel pipeline samples

Frequently, the correlation coefficient can be quickly and accurately estimated by plotting the variables on a scattergram that shows the results of measurements or analyses of the magnitudes of the two variables (Figures 3.5, 3.6, 3.7). Two strongly positively correlated variables will plot with data points that closely lie along a line that indicates as one variable increases the other variable increases. Two strongly negatively correlated variables will plot with data points that closely lie along a line that indicates as one variable increases, the other variable decreases. If the plot does not indicate any systematic variation in the variables, the general conclusion is that the correlation is very low or close to zero.

3.4.3 Demand – Capacity Correlations

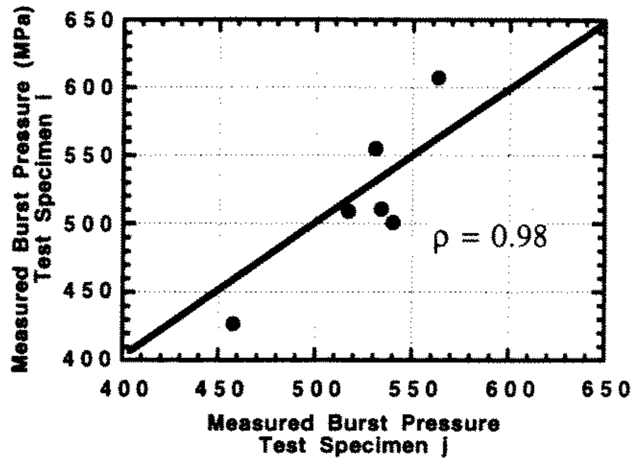

Figure 3.7 – Correlation of measured burst strengths of paired steel pipeline samples

There can be a correlation between the strength of the elements and the loadings imposed on or induced on the elements. Frequently, this source of correlation is either ignored in the reliability characterization (through the assumption of independent – non-correlated demands and capacities), or introduced through an evaluation in which the performance characteristic/s of the element are evaluated conditionally on the occurrence of the demands. 'Low-cycle' fatigue effects of extreme loadings on the load carrying capacities of some elements is an example of this type of correlation. The correlation between the buckling capacity of a pipeline and the lateral loading imposed on the pipeline is an other example.

For the case of Lognormally distributed correlated demands and capacities, The Safety Index is determined as follows:

$$\beta = \frac{\ln(R_{50} / S_{50})}{\sqrt{\sigma_{\ln R}^{2} + \sigma_{\ln S}^{2} - 2\rho_{RS}\sigma_{\ln R}\sigma_{\ln S}}}$$

Positive correlation of the demands (S) and capacities (R) results in an effective decrease in the total uncertainty and an increase in the Safety Index. Positive correlation implies that as the loadings increase, the capacities increase. This is a beneficial effect. However, for negative correlation between R and S, there is the opposite effect; the Safety Index decreases. In this case ignoring the demand - capacity correlation would result in 'unconservative' estimates of the Safety Index.

3.4.4 Series Systems

The probability of failure of a series system can be expressed as the union of the n element failure events of failure of its n elements as (Melchers, 1987):

$$Pf_{system} = P\,[\,f_1 \cup f_2 \cup f_3 \ldots \cup f_n]$$

where the element failure events are f_1, f_2, …fi. Expressed as the intersections of the element failure events:

$$Pfsystem = \sum_{i=1}^{n} Pfi - \sum_{i}\sum_{j>i} P(fi \cap fj) + \sum_{i}\sum_{j>i}\sum_{k>j} P(fi \cap fj \cap fk) - \ldots$$

The intersection failure events fi $\cap$ fj represent events in which the capacities of both elements (Ri, Rj) are less than the demand, S. These intersections represent the effects of correlations of the failure events. This equation can be further developed by ordering the failures from most probable (.i = 1) to least probable(.i = n) and dividing the right side by P(f1):

$$Pfsystem = P(f1)\left[1 + \sum_{i=2}^{n}\frac{P(fi)}{P(f1)} - \sum_{i}\sum_{j>i}\frac{P(fi \cap fj)}{P(f1)} + \ldots\right]$$

P(f1) is the most likely to fail element in the system. The term in brackets modifies the most likely to fail element probability by two factors. The first is the summation of the remaining probabilities of failure divided by the probability of failure of the most likely to fail element. The second is double summation of the probabilities of the intersection events divided by the probability of the most likely to fail element. If there were no intersections, only the first term would be important and the probability of failure of the system would be greater than the probability of failure of the most likely to fail element in the system. The effective upper bound of this term is n. However, including the probabilities of the joint events reflected in their failure correlations, there is a subtractive term. Given perfect correlation of the marginal failure events, the probability of failure of the system becomes equal to the probability of failure of the most likely to fail element.

If the n elements have the same strengths and the failures of the elements are independent ($\rho_{ij} = 0$), then the probability of failure of the system can be expressed as:

$$Pf_{system} = 1 - (1 - Pf_i)^n$$

If the elements (independent) have different failure probabilities:

$$Pf_{system} = 1 - \prod_{i=1}^{n}(1 - Pfi)$$

If the elements are perfectly correlated then:

$$Pf_{system} = \text{maximum}\,(Pf_i)$$

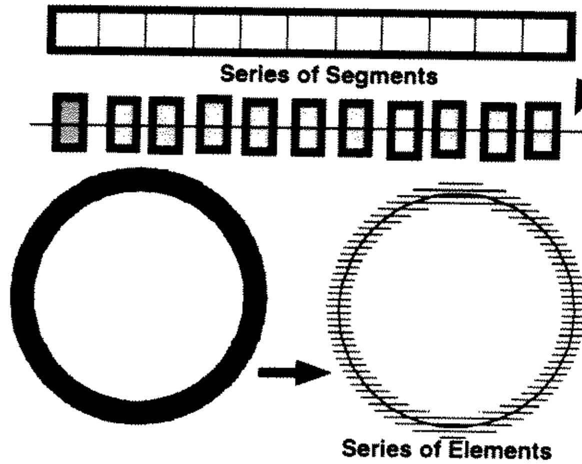

Figure 3.8 – Pipeline system as a series of segments with each segment composed of a series of elements

In general, neither perfect dependence or perfect independence of the member capacities, Ri, or failure events ($fi = (Ri - S) \leq 0$) exist. The relationship between the system probability of failure and the element probabilities of failure will depend on the correlations of the element capacities, the correlations among the safety margin variables, the magnitude of the element failure probabilities, and the type of probability distributions. Various approaches and approximations have been developed to improve the bounds for series systems having intermediate element to element and failure mode correlations (Melchers, 1987; Grigoriu, Tukstra, 1979; Thoft-Christensen, Sorensen, 1982; Ahmed, S., 1990).

For the case in which the standard deviations of the capacities are the same, the correlation coefficient between any two member safety margins ($Mi = Ri - S$) would be:

$$\rho_{ij} = (\rho'_{ij}\,\sigma_R^2) + \sigma^2_S / (\sigma_R^2 + \sigma^2_S)$$

where ρ'_{ij} is the correlation coefficient between the capacities of paired elements i and j in the system, σ_R^2 is the variance of the capacities, and σ^2_S is the variance of the demand. If the capacities and demands are characterized with Lognormal distributions, then $Mi = \ln(Ri / S)$ and:

$$\rho_{ij} = [(\rho'_{ij}\,\sigma_{Ri}\,\sigma_{Rj}) + \sigma^2_S] / [(\sigma_{Ri}^2 + \sigma^2_S)^{0.5} + (\sigma_{Rj}^2 + \sigma^2_S)^{0.5}]$$

or approximately:

$$\rho_{ij} \approx (\rho'_{ij}\,Vi\,Vj + V^2_S) / (ViVj + V^2_S)$$

where Vi and Vj are the Coefficients of Variation of the resistances of thei and j elements and Vs is the coefficient of variation of the demand. The correlation between the safety margins is dependent on the coefficients of variation of the capacities and demand and the correlation between the capacities. This clearly indicates that the relationship between the system probability of failure will be dependent on two factors: 1) the correlations between the capacities of the elements, and 2) the correlations between the failure modes.

If the elements are perfectly correlated then:

$$Pf_{system} = \text{maximum } (Pf_i)$$

Application of the foregoing developments can be illustrated as shown in Figure 3.8. The pipeline can be regarded as a series of interconnected 'segments'. The pipeline 'segments' can be regarded as a series of interconnected 'elements.' Due to the effects of design, manufacture (construction), operations, maintenance, these elements have high positive segment to segment and element to element correlations. Due to the uncertainties associated with the demands and capacities, these elements have high positive segment to segment and element to element correlations.

This conclusion has extremely important effects on the interpertations of the probabilities of failure associated with a pipeline system. Due to the multiple types of correlations that can be developed, due to the expected high degrees of correlations, the pipeline system probability of failure will be approximately the same as that of the MLTF element in the pipeline. This conclusion is contrary to the majority of interpretations that have been applied in development of either pipeline design or requalification criteria. Most of these interpretations have chosen to assume that the pipeline elements and segments are uncorrelated – independent. This is appears to be because this assumption leads to a 'conservative' relationship between the elements and segments that comprise a pipeline system. However, it is observed that such an assumption can also lead to unreasonably and unrealistically high probabiliiies of failure for long pipelines that can many hundreds if not thousands of flaws and defects.

3.5 Pipeline Inspection, Maintenance, and Repair (IMR) Programs

Pipeline Inspection, Maintenance, and Repair (IMR) programs should be an essential part of the design and life-cycle management of pipelines (Bea, Xu, 1997; Nordland, Bai, Damslet, 1997). However, many pipelines are simply designed and it is 'assumed' that the pipeline will be 'adequately' maintained. In these criteria, it is apparent that a pipeline IMR program is an explicit part of requalification of a pipeline (Figure 2.1).

For example, the wall thickness of a riser or pipeline is directly influenced by the IMR program that will be implemented to control internal and external corrosion. Similarly, the weight-coating that determines the on-bottom stability of a pipeline must be inspected and maintained to maintain the design on-bottom weight (Valdez, et al, 1997). Riser clamps and cathodic protection must be maintained to prevent premature wear and corrosion.

Pipeline inspections have two fundamental purposes (Bea, Xu, 1997):

- **confirm what is 'thought'** - validate the projections that were made at the time of design concerning the pipeline demands and capacities, and
- **disclose what is not 'thought'** - bring to light what is not known about the condition and characteristics of the pipeline - these conditions and characteristics are fundamentally 'unknowable' at the time of design of the pipeline.

Analyses can help define IMR programs to address the first purpose. These are essentially 'deductive' process that proceed from 'givens' to determine 'finds' based on analytical models. Deductive analyses provide little information for the second purpose. Rather, to disclose what is not known or knowable about the pipeline requires 'inductive' methods. These methods proceed from 'observations' and deduce what are the associated causes and effects. inspections following accidents and hurricanes are an example of the second approach that should be a part of a pipeline IMR program.

For development of pipeline IMR programs, it suggested that pipeline operators utilize an RCM (Reliability Centered Maintenance) approach. This approach was developed initially for the commercial airframe industry. The RCM approach has found applications offshore in developing maintenance programs for equipment, piping, and production control systems. The fundamental aspects of the RCM approach are outlined in Figures 3.9 and 3.10.

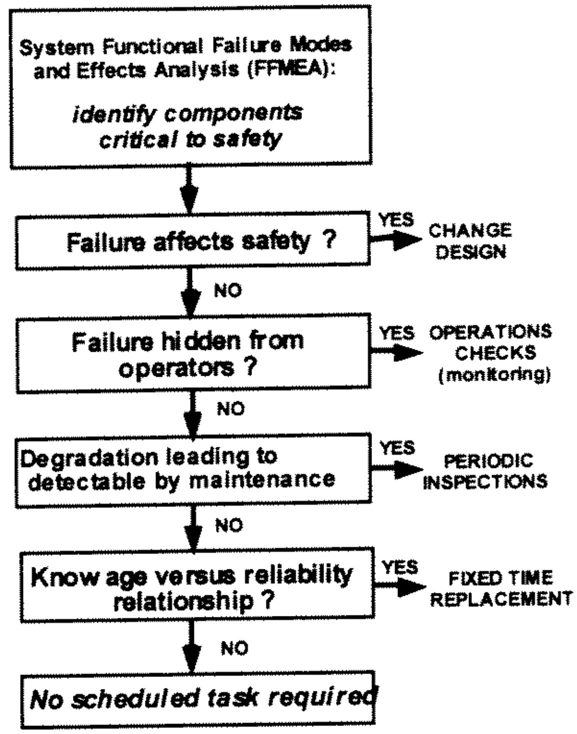

Figure 3.9 - RCM Approach to Development of IMR Programs

The RCM approach is used to define the following types of IMR programs (Figure 3.10):

- **Period based (verification),**
- **Time based (scheduled),**
- **Re-engineering based,**
- **Condition based,**
- **Break-down based (benign neglect).**

The authors have applied the RCM approach to development of IMR programs for offshore platforms and commercial tank ships (Bea, 1992; Bea, Xu, 1997). The authors adopted this approach in combination with a probability based analytical approach when platform and ship maintenance personnel discovered the major deficiencies that surfaced when only a probability based analytical approach was used.

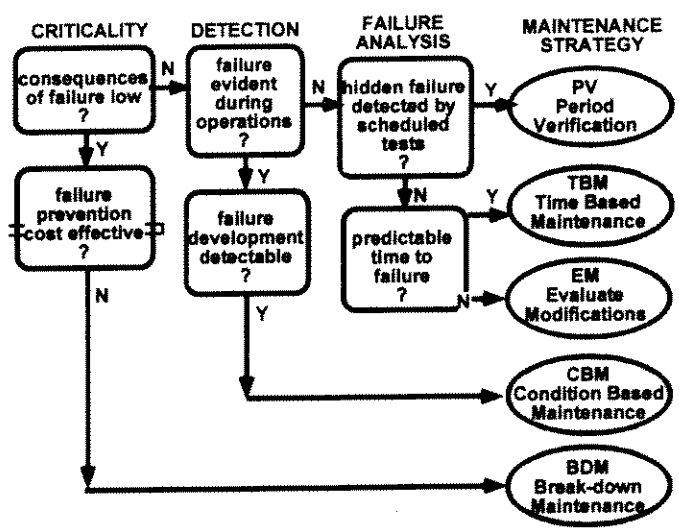

Figure 3.10 - RCM Approach to Definition of Alternative IMR Programs

The probability based analytical approach is premised on the predictability of damage and defects that can lead to degradation in the capacity of a 'system.' Experience with both onshore and offshore systems has clearly shown that only a portion of the damage and defects are 'predictable.' Many flaws and damages are essentially unpredictable and definitely unknowable in any definitive way at the time of design. Further, due to 'deficiencies' in the analytical models that are employed in the predictive analyses, they too leave much to be desired when it comes to developing realistic IMR programs for offshore systems.

Note in the development of IMR programs, that there are two fundamental 'constraints' that must be met. The first is economic. The objective is to find the most cost effective IMR program. Cost effectiveness is not only defined by the direct costs associated with implementation of the IMR program, but as well by the costs associated with sustaining and disrupting the associated operations. This requires a 'full-scope' - 'life-cycle' approach to developing evaluations of alternative IMR programs.

An 'economically optimum' (minimizes the sum total of expected initial and future costs) IMR program can be defined as the IMR program that will maintain the system so that it has a specified 'minimum' annual reliability (Pso) (Figure 3.11):

$$Pso = 1 - Pfo \approx 1 - 0.4348 / CR\,L = 1 - 0.4348 / CM$$

where CR is a Cost Ratio. CR is the ratio of the total costs associated with loss of quality (safety, durability, serviceability, compatibility) of the pipeline system to the initial costs associated with implementation of a given IMR program that will lower the probability of failure (loss of expected quality) by a factor of 10. L is the projected life (years) of the system. CM is a 'consequence' measure. Note that CM is expressed in years.

As the CR is increased due either to high costs associated with loss of serviceability or low costs associated with a given IMR program, the Pso is increased, and vice versa (Figure 3.11). As the life of the IMR program or pipeline system life increases, Pso is increased, and vice versa. This is common sense, but this common sense is frequently lost in the generally 'evolutionary processes' used to develop IMR programs (Bea, Xu, 1997; Bea, 1994, 1992).

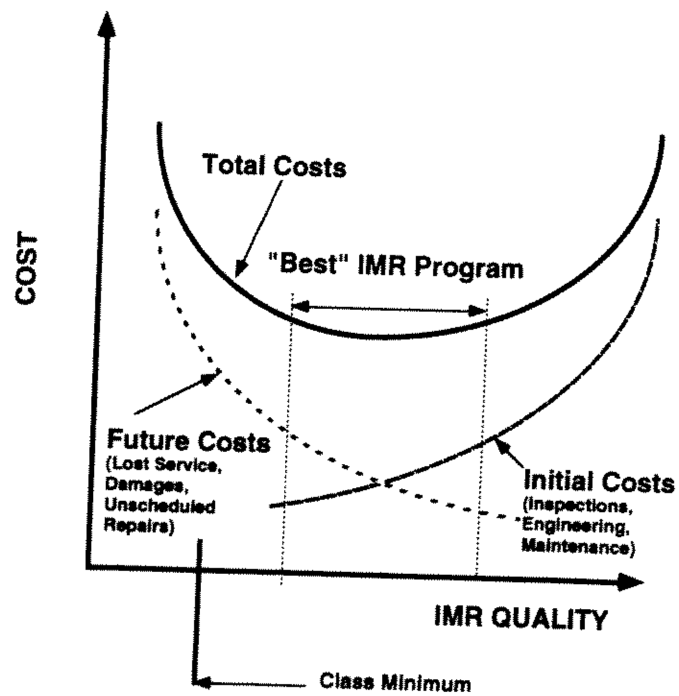

Figure 3.11 - Cost Optimized Pipeline IMR Programs Defined by Minimum Total Costs and Regulatory (Class) Minimums

The second constraint is reliability and the associated 'quality' attribute of safety. In this context, reliability is defined as the likelihood or probability that the desired quality of an offshore pipeline system will be developed during the life-time of the system. Quality is characterized with four attributes:

1) Safety,
2) Durability,
3) Serviceability, and
4) Compatibility.

Safety is freedom from undue exposure to harm or injury. Durability is freedom from unexpected degradation in the performance characteristics of a system. Serviceability is the ability of the system to meet stated performance and availability requirements. Compatibility is the ability of the system to satisfy economic, schedule, political, and environmental requirements. In this context, reliability goes far beyond safety.

It is important to note that this second constraint can also involve regulatory or code specified 'minimum' reliabilities. Generally, code specified minimum reliabilities address only safety and do not address the other equally important aspects that determine the 'quality' of an offshore pipeline system.

IMR programs for pipelines should be part of a comprehensive 'system' that is intended to maintain vigilance and reliability of a pipeline system (Bea, Schulte-Strathaus, Dry, 1995). If IMR programs are 'evolved' or 'tacked on' to existing systems, it is very likely that the system will be more costly than necessary, will not be effective, and eventually will be dropped. 'Re-Engineering' of the entire IMR process is generally required to develop an 'optimum' IMR program and system.

A critical component of an IMR program is the development, maintenance, and utilization of a pipeline performance database to facilitate archiving, analyzing, and reporting pipeline inspections, maintenance, operations, and losses of containment (normal and accidental) and other important operating information. A component of the database needs to be developed to facilitate archiving, analyzing, and reporting pipeline operations information including inspections, maintenance, repairs, operating pressures, temperatures, and information on the oil and gas characteristics (including corrosivity). A Pipeline Inspection, Maintenance, Performance, Information System (PIMPIS) (Figure 3.12) has been proposed to interface with databases that contain information on pipeline characteristics, operations, incidents, inspections, and loadings (Jaio, Bea, 1994). The system was designed to interface with the pipeline performance database developed by the MMS for Gulf of Mexico Offshore Continental Shelf Operations (does not include State waters) (Alvarado, 1998). The information and data from such an industry wide system could have important impacts on the development of criteria and on the improved management of pipeline maintenance and repairs.

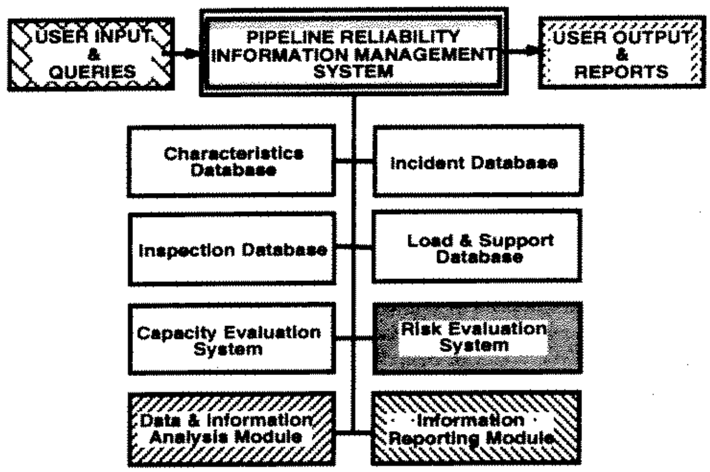

Figure 3.12 – Pipeline Inspection, Maintenance, Performance, Information System (PIMPIS)

3.6 Reliability Updating

Every structural integrity assessment procedure must take provisions for the updating of reliability or safety level on the basis of additional information gathered from inspection, monitoring, maintenance and repair (IMMR) activities. A framework for updating the RAM criteria developed herein or for re-adjusting partial factors based on new information would be valuable. A strategy should also be developed for quantitatively incorporating the effects of repair action.

The systematic application of Bayesian updating methods could prove to be valuable in future developments of the Transitory Criteria for pipelines. Bayes theorem results in the following relationship for the probability distribution of a random variable, X:

$$p[X''] = p\,[X]\,L\,[X]\,K$$

p[X"] is the updated or posterior probability distribution (density function) of X based on an observation. p [X'] is the prior probability distribution of X. L [X] is the likelihood function which is the probability of obtaining the observation for a given value of X (i.e., P [observation | x]). K is a normalization constant such that p[X"] is a proper density function.

Let the true capacity of the pipelines = R_T. Let the calculated capacity of the pipelines = R_C. Then the "bias" in the capacity, X, can be expressed as:

$$X = R_T / R_C$$

The bias will be characterized with the first two moments of its probability distribution: a mean value, $\boldsymbol{X}$, and a standard deviation, σ_x . Let the event of the observed non-failures (Successes) of pipelines be designated as S. The posterior mean of the bias, X'' ,can be shown to be:

$$X'' = \frac{E\,[\,X\,P\,[\,S \mid X]]}{E\,[P\,[S \mid X]]}$$

E [.] is the expected or mean value of the quantity [.]. P [.] is the probability of the quantity [.] and P [S | X] is read as the probability of survival, S, given the bias, X.

The mean value of the function, f(X), can be determined to good approximation as:

$$E\,[f(X)] \approx \frac{f(X + \sigma_x) + f(X - \sigma_x)}{2}$$

Using this approximation:

$$X'' = \frac{(X' + \sigma_{x'})\,P[S|X + \sigma_{x'}] + (X' - \sigma_{x'})\,P[S|X - \sigma_{x'}]}{P[S|\,X + \sigma_{x'}] + P[S|X - \sigma_{x'}]}$$

and,

$$(\sigma_{x''})^2 = \frac{E\,[\,X^2\,P[S|X]]}{E[P[S|X]]} - \mathbf{X}'^2$$

The probability of success (non-failure of the pipelines) given a value of the bias, P[S|X], can be evaluated using the probability of failure formulation summarized earlier where:

$P[S|X] = 1 - Pf|X$

The uncertainties in the loadings and the capacities are integrated into the calculation of Pf. The prior mean bias, X', and standard deviation of the bias, $\sigma_{X'}$, are integrated into the Bayesian updating formulation to determine the posterior mean bias, X'', and posterior standard deviation in the bias, $\sigma_{X''}$.

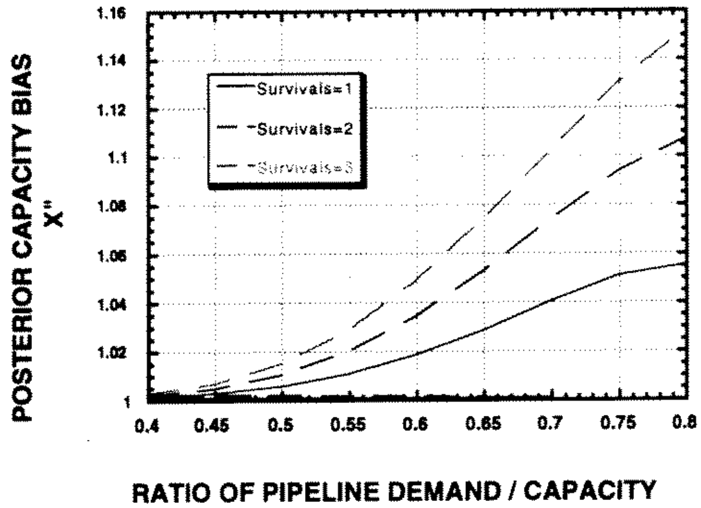

Figure 3.13 – Updating effects on capacity bias

Example results of the updating of the capacity mean bias and capacity bias standard deviation are summarized in Figures 3.13 and 3.14. The updating is shown as a function of the ratio of the expected value of the maximum demand (internal pressure, external pressure, bending, axial force) developed on and in the pipeline to the capacity of the pipeline to resist these demands. The updating is shown as a function of the number of "trials" (observed survivals) for given ratios of demands to capacities.

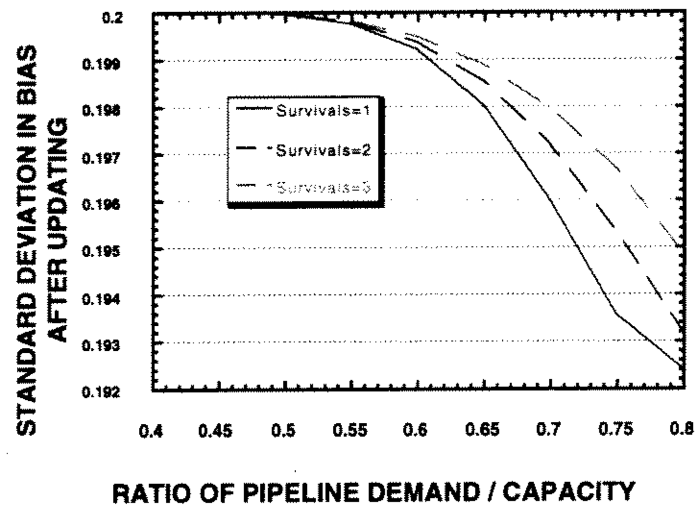

Figure 3.14 – Updating effects on capacity uncertainty

The updated pipeline capacities and uncertainties could be used to help evaluate the suitability of the analytical models used to determine the pipeline capacities. The updated pipeline uncertainties could be used to reduce the capacity uncertainties that were utilized in development of the design and reassessment criteria. Thus, continuing development of information on the pipeline 'demands' and 'capacities' and the performance of the pipelines for observed demands and capacities can directly impact or change the design and reassessment criteria through the changes in the central tendency (mean, median) and uncertainty measures utilized in development of the criteria.

3.7 Hydrotesting – Proof Testing to Improve Reliability

None of these developments have taken account of the effects of pressure testing the in-place pipeline. Based on API guidelines (API, 1993), the pipeline is tested to 1.25 times the maximum design pressure (MDP) for oil pipelines and 1.5 times the MDP for gas pipelines. The maximum operating pressure (MOP) generally is set at 90 % of the MDP.

The effect of pressure testing is to effectively 'truncate' the probability distribution of the pipeline burst pressure capacity below the test pressure (Figure 19). Pressure testing is a form of 'proof testing' that can result in an effective increase in the reliability of the pipeline.

There can be a similar effect on the operating pressure 'demands' if there are pressure relief or control mechanisms maintained in the pipeline. Such pressure relief or control equipment can act to effectively truncate or limit the probabilities of developing very high unanticipated operating pressures (due to surges, slugging, or blockage of the pipeline).

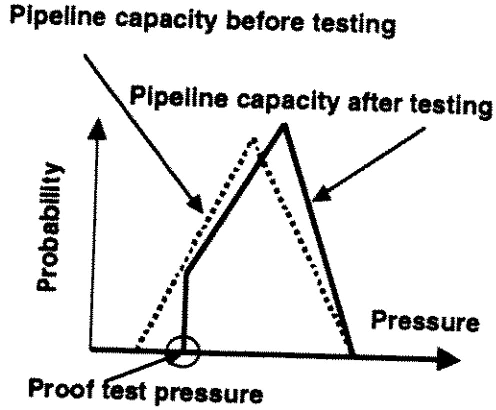

Figure 3.15 – Effects of proof testing on pipeline capacity distribution

This raises the issues associated with pressure testing and pressure controls on the required factors of safety or load and resistance factors (Hall, 1988; Grigoriu, Hall, 1984; Grigoriu, Lind, 1982). Figure 3.16 summarizes the results of pipeline proof testing on the pipeline Safety Index as a function of the 'level' of the proof testing pressure factor, K:

$$K = \ln (Xp / p_b) / \sigma_{\ln pb}$$

where Xp / p_b is the ratio of the test pressure to the median burst pressure capacity of the pipeline (test pressure deterministic, burst pressure capacity Lognormally distributed) and is the standard deviation of the Logarithms of the pipeline burst pressure capacities. These results have been generated for the case where the uncertainty associated with the maximum operating / incidental pressures is equal to the uncertainty of the pipeline burst pressures and for Safety Indices in the range of $\beta = 3$ to $\beta = 4.5$ (Fujino, Lind, 1977).

For example, if the median burst pressure of the pipeline were 2,000 psi and this had a Coefficient of Variation of 10 % ($\sigma_{\ln pb} = 0.10$), there was a factor of safety on this burst pressure of 2 (f = 0.5) (maximum operating pressure = 1,000 psi), and the pipeline was tested to a pressure of 1.25 times the maximum operating pressure (Xp = 1,250 psi), the proof testing factor K = -4.7. The results in Figure 3.15, indicate that this level of proof testing is not effective in changing the pipeline reliability. Even if the pipeline were tested to a pressure that was 1.5 times the operating pressure, the change in the Safety Index would be less than 5 %.

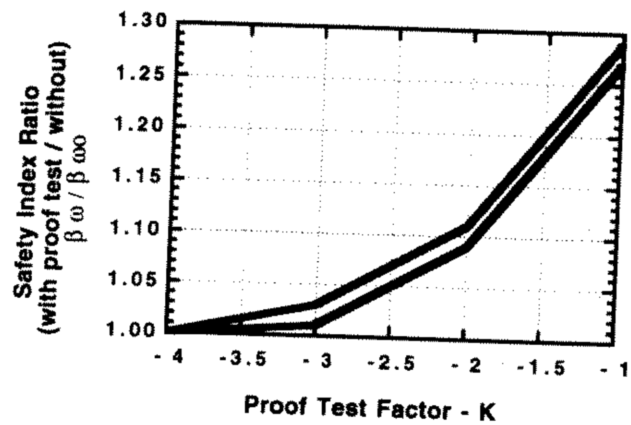

Figure 3.16 - Effects of hydrotesting or proof testing on pipeline reliability

If the test pressure were increased to 75 % of the median burst pressure, the Safety Index would be increased by about 25 %. For a Safety Index of $\beta = 3.0$ (Pf = 1E-3), these results indicate a $\beta = 3.75$ (Pf = 1E-4) after proof testing. Very high levels of proof testing are required before there is any substantial improvement in the pipeline reliability.

These results indicate that conventional pressure testing may not be very effective at increasing the burst pressure reliability characteristics. Such testing may be effective at disclosing accidental flaws incorporated into the pipeline due to human and organizational factors (e.g. poor welding). However, some studies indicate that such testing can be effective at developing and propagating what would be otherwise unimportant flaws, cracks, and other similar defects. In some instances, hydrotesting can prove to be damaging to a serviceable pipeline. Additional studies areneeded to further define the effects of pressure testing and operating pressure controls on the required factors of safety for both new and existing pipelines.

3.8 Risk and Risk Management

Risk is defined in this report as the product of the likelihood that adequate or acceptable quality is not achieved and the consequences associated with the lack of achieved quality.

Risk results from uncertainties. Uncertainties result from inherent variabilities (aleatory), "professional" or "technical" sources (analytical, modeling, parameter, epistemic), and "human" sources (individuals, teams, organizations, societies). Some uncertainties are random (aleatory) and some are systematic (epistemic). Some uncertainties can be managed (information sensitive, epistemic) and some uncertainties can not be managed (information insensitive, aleatory). Some uncertainties are essentially "static" (unchanging in time) and others are essentially "dynamic." Some uncertainties can be identified and quantified and some uncertainties can not be identified and quantified.

Consequences result from unrealized expectations and unanticipated lack of sufficient quality. Consequences can be expressed in terms of their frequency, their severity, their impacts (on site and off site), and their predictability.

Consequences can be expressed in a variety of ways and with a variety of metrics. Monetary costs are one way to measure and express consequences. Time (schedule, availability), injuries to humans, and injuries to the environment are other ways to express and measure consequences.

Some consequences can be managed or controlled (hazard mitigation measures). Some consequences can not be managed or controlled. Some consequences can be evaluated objectively and quantitatively and some consequences can not be evaluated objectively and quantitatively.

Generally, there are significant uncertainties associated with the results of evaluations of consequences. This is particularly so as one projects the consequences of insufficient or unacceptable quality far into the future.

Evaluations of consequences are difficult to make and express. Evaluations of consequences are very susceptible to the values, views, and "biases" of the evaluators. Some consequences are essentially "static." They do not change significantly in time. Other consequences are very "dynamic" in that they change markedly in time.

An identified risk is a management problem. A faulty or bad definition of a risk will breed additional risk and result in bad management of quality. A risk management framework is based on intelligent and perceptive risk identification, classification, analysis, evaluation, and response.

Risks have "sources", are translated to reality with "events", and are felt with "effects." There are initiating events (direct causes), contributing events (background causes), and compounding events (propagating or escalating causes). Risk management attempts to identify causes, detect potential and evolving events, and control effects.

Risks are independent and dependent. Risks can have partial dependence. If the occurrence of one risk does not influence the occurrence of another risk, then it is independent. If the magnitude of one risk is related to the magnitude of another risk then these two risks are correlated. Independence and correlation are critical issues in risk management.

Risks are controllable and uncontrollable. Controllable risks are those that are within the control of those that own, operate, design, classify, regulate, and build marine systems. Uncontrollable risks are those that are not within the control of the groups cited. Risk management is concerned primarily with controllable risks. Inherent risk and uncontrollable risk must be recognized and evaluated in the process of making decisions regarding the activities and ventures associated with marine systems.

A risk management system should be practical, realistic, and must be cost effective. Risk management need not be complicated nor require the collection of vast amounts of data, that in most cases of marine systems, does not exist. Excellent risk management is a combination of uncommon "common sense", qualified experience, judgment, knowledge, wisdom, intuition, and integrity. Mostly it is a willingness to operate in a caring and disciplined manner in approaching the critical features of any activity in which risk can be generated.

The purpose of a risk management system should be to enable and empower those that design, build, and operate marine systems. The purpose is to assist those groups to take the "right" risks and to achieve "acceptable" quality. *To try to eliminate risk is futile. To try to manage risks is the essence of man's activities in the sea.*

Risk analysis is the attempt to define and evaluate the sources, effects, and consequences of risks. Risk analysis can be qualitative and it can be quantitative. These are complimentary forms of risk analysis and they should be used to support each other.

Quantitative risk analysis can involve probability analysis, sensitivity analysis, scenario analysis, situation analysis, and correlation analysis. Quantitative risk analysis can be objective and / or subjective.

Qualitative risk analysis will involve the use of direct judgment, generally involves ranking and comparing attributes and options, and a descriptive analysis and evaluation.

The purpose of developing qualitative and quantitative models of risks is to provide information for making good decisions regarding management of these risks. The development of a decision model to help solve problems is outlined in Figure 3.15.

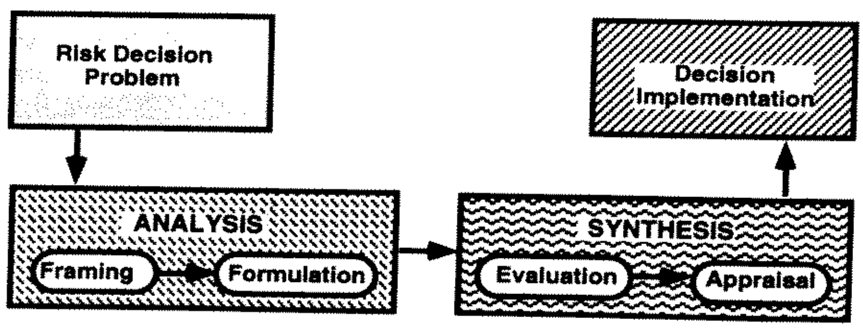

Figure 3.15- Risk decision analysis

The decision analysis process is divided into two primary parts:

- Analysis, and
- Synthesis.

Analysis involves framing and formulation. These involve decomposition of the problem into its parts. The subsequent evaluation and appraisal involve synthesis in which the parts are combined into a whole to establish the attributes of each possible solution.

The purpose of framing is to avoid working the wrong problem. The purpose of framing is to state the precise nature of a problem and the objectives to be pursued. Framing is structuring and re-stating the problem. One objective of framing is to surface the "unspoken agendas" that are generally present in a risk decision problem.

Formulation is a formal model based upon the problem. It is based on a decision process composed of three parts:

- The alternatives available to the decision maker to achieve the particular goal,
- The information that describes the relationship between the decisions and possible outcomes, and
- The preferences of the decision maker.

Information includes any form of model, forecast or probability assignment which indicates the possible outcome of the decision. Preferences express the values of the decision makers regarding the principal outcomes (e.g. which is more important, schedule or cost?).

A good decision is an action that is logically consistent with the alternatives available, the information available, and the preferences. Good decisions do not always result in good outcomes. Table 3.1 lists the attributes associated with good risk decision making processes [Flanagan, Norman, 1993]).

Table 3.1 - Attributes of good risk decision making processes

Framing	Surveys the full range of objectives to be fulfilled and the values implied by the choices
Alternatives	Thoroughly canvasses a wide range of alternative courses of action; possibility thinking
Information	Carefully weighs knowledge about the costs and risks of negative consequences as well as the positive consequences that could flow from each option. Intensely searches for new information relevant to further evaluation of the options. Keeps an open mind.
Evaluation	Correctly assimilates and takes account of any expert judgment and risk exposure, even when the judgment does not support the course of action initially preferred. Re-examines the positive and negative consequences of all known alternatives, including those originally regarded as unacceptable, before forming the final choice.
Implementation	Makes detailed provisions for implementing or executing the chosen course of action, with special attention to contingency plans that might be required if various known and unknown risks were to materialize.

Quality is freedom from unanticipated defects. Quality is fitness for purpose. Quality is meeting the requirements of those that own, operate, design, construct, and regulate marine structures. These requirements include those of serviceability, safety, compatibility, and durability.

Reliability is defined as the probability that a given level of quality will be achieved during the design, construction, and operating life-cycle phases of a marine pipeline system. Reliability is the likelihood that the system will perform in an acceptable manner. Acceptable performance means that the system has desirable serviceability, safety, compatibility, and durability.

Reliability can be expressed as the probability that the demands placed on a marine pipeline system can be supplied by that system. The probability of failure or unreliability is the compliment of the reliability and is the likelihood of undesirable or unacceptable performance of a marine pipeline system.

Risk represents the product of the likelihood of an event and the consequences associated with that event. Risks pervade all activities. Not all risks can be defined and quantified. A primary objective is to manage those risks that we can define and quantify and defend against those that we can not define and quantify.
Decisions involve framing and analysis. There are good decision making processes and the attributes of such processes have been defined. A good decision is an action that is logically consistent with the alternatives available, the information available, and the preferences. Good decisions do not always result in good outcomes.

3.9 Traditional and RAM Based Pipeline Criteria

Whether Allowable Stress Design (ASD) or Load and Resistance Factor Design (LRFD) is considered (these are only different formats for engineering design guidelines), most existing offshore pipeline codes continue to address code development with a deterministic – experience based approach. The reluctance in using a proabilistic approach may be due to lack of understanding of the approach, a belief that sufficient data is not available for a fully valid reliability approach, that there are certain unknown variables which can not be quantified, or a combination of these.

At the outset, it must be recognized that the result of the pipeline engineering process is deterministic: a certain diameter of pipeline with a certain wall thickness must be evaluated. This is true no matter how the criteria have been derived.

It also must be recognized that a sound reliability approach is founded on the same principles as a sound deterministic approach. There must be a firm foundation of deterministic understanding of the physics and mechanics that underly the important processes that determine the reliability of a pipeline. The traditional deterministic approach and the RAM approach should be complimentary. One builds on the foundation provided by the other.

The same problems that confront a reliability approach also confront a deterministic approach. A probabilistic approach needs no more or no less data and information than a deterministic approach. If there are unknown variables, then these same unknown variables pervade the deterministic and probabilistic approaches. The need for qualified judgement and experience is present in both approaches. One of the biggest dangers to both approaches are 'number crunchers' that loose sight of the need for such qualified judgement and experience to act as a filter to provide adequate understanding and insight to make good engineering decisions.

Then, what does a RAM approach bring to the process of developing good engineering design and requalification guidelines that are not brought by a traditional deterministic approach? RAM provides a disciplined and structured approach to help recognize and incorporate **explicit evaluation and assessment of uncertainties** in an engineering design, decision and management process. These uncertainties come from natural variabilities in pipline properties, capacities, and 'loadings' (external and internal) and the changes in these with time. They also come from limitations in the models, parameters, and knowledge used to evaluate the pipeline loadings and capacities. The most daunting source of these uncertainties are the future actions and inactions of people that influence the design, construction, operations, and maintenance of a pipeline during its lifetime. Once the presence of these uncertainties is recognized, then deterministic approaches must be modified – expanded to recognize and manage their effects.

One of the traditional methods used by engineers to manage uncertainties in a deterministic approach was to be 'conservative.' But, problems develop when multiple conservatisms are implicitly introduced in a sequential engineering process such as is represented by engineering design guidelines and codes (certainly most uncoordinated design codes that tend to evolve with time). Similarly when 'new information' indicates that what was thought to be conservative, no longer is conservative; the introduction of even more conservatism is the traditional response. The only active bounding influence on such conservatism is economics and feasibility.

The other active bound on the deterministic approach is legal – social - political. The deterministic approach can not recognize that there is a finite probability that a pipeline will fail. But, we know that there is such a probability (history clearly shows this). No pipeline can be designed so that there is a zero probability of failure. The deterministic approach itself has encouraged such unrealizable expectations. The disappointment and disillusionment associated with such unrealized expectations encourages the legal – social – political responses. The deterministic approach shields the real decision makers (managers, regulators that represent corporate interests) from developing an adequate understanding of the risks and then making the decisions regarding what these risks should be. The deterministic approach has encouraged engineers to make such decisions.

By its very nature, a RAM approach must be very interdisciplinary. To be effective, a RAM approach must facilitate communications between very diverse fields and viewpoints. A RAM approach must involve both 'management' and 'engineering.' A RAM approach must consider not only the details of elements, but as well, the details of entire systems. Thus, if properly used, a RAM approach can provide another set of important dimensions to the traditional deterministic approach of developing engineering codes and guidelines.

An important aspect that a RAM approach brings to the process of developing engineering design codes and guidelines, is the aspect of requiring direct recognition and treatment of **risk mitigation and management**. The determinstic approach tends to shield these aspects from the main stream of the code and guideline development. RAM recognizes that there are knowable and unknowable risks, and both must be mitigated and managed during the lifetime of a pipeline. These strategies and measures must be incorporated explicitly in engineering design guidelines.

The single biggest impediment to implementation of the RAM approach regards **education of engineers**. Engineers must learn the fundamentals of statistics and probability and how these fundamentals can be applied to help identify and solve engineering problems. As often presented, the RAM approach appears to be extremely complex, involve new (and unproven) principles and methods, and highly mathematical. This does not have to be the presentation. The complexities can be reduced to terms that can be understood and used by practicing engineers and managers. This is one of the primary objectives of this project.

4.0 Reliability Goals

4.1 Approaches

Three approaches were used to develop reliability goals for reassessment of pipelines and risers:

- **Historic** – based on actuarial probabilities of failure of pipelines in the Gulf of Mexico and North Sea,
- **Standards of Practice** – based on explicit or implicit reliabilities integrated into current design and reassessment guidelines and codes, and
- **Economics** – based on considerations of initial and future costs associated with alternative probabilities of failure.

4.2 Historic Approach

4.2.1 Gulf of Mexico Experience

As a result of a Joint Industry Project titled PIMPIS (Pipeline Inspection, Maintenance, and Performance Information System), an extensive database developed by the U. S. Minerals Management Service (MMS) has been analyzed to determine the historic performance characteristics of Gulf of Mexico pipelines (Elsayed, Bea, 1997). Work to develop this data base was initiated in 1989 (Woodson, Bea, 1990). The database has been revised, expanded, and maintained by the MMS (Marine Board, 1994). The database includes pipeline failure data for the period 1967 through 1997. The data were provided by the MMS for Offshore Continental Shelf (OCS) waters, and by the U. S. Coast Guard for State waters. The data covers approximately 15,000 miles of offshore pipelines.

Figure 4.1 summarizes the causes of pipeline failures in the OCS waters of the Gulf of Mexico. This summary includes 2,332 failures for 10,553 pipelines. Failure is defined as a loss of containment resulting in a substantial loss of hydrocarbons from the pipeline or riser). The primary cause of failure is corrosion; about 50 % of the failures are due to corrosion. Hurricanes (natural hazards) are responsible for about 25 % of the failures. The remaining 25 % of the failures can be attributed to Human and Organizational Factors (HOF) (Bea, 1994).

Based on the same database, Figures 4.2 and 4.3 shows the causes of pipeline failures for oil and gas pipelines, respectively. The distribution of the causes of failures is about the same for both oil and gas pipelines. Corrosion again accounts for about half of the failures. Most surprising was the large proportion of gas pipelines that fail due to corrosion. Improvements in gas dehydration could help reduce this source of failures.

The database contains information on the distribution of failures caused by external and internal corrosion. As summarized in Figure 4.4, in the case of risers, external corrosion accounts for about 85 % of the corrosion related failures. The vast majority of this corrosion is located at and above the mean sea level.

In the case of submerged pipelines, internal corrosion accounts for about 75 % of the corrosion related failures The database did not indicate any significant differences between the failure rates for small and large diameter pipelines.

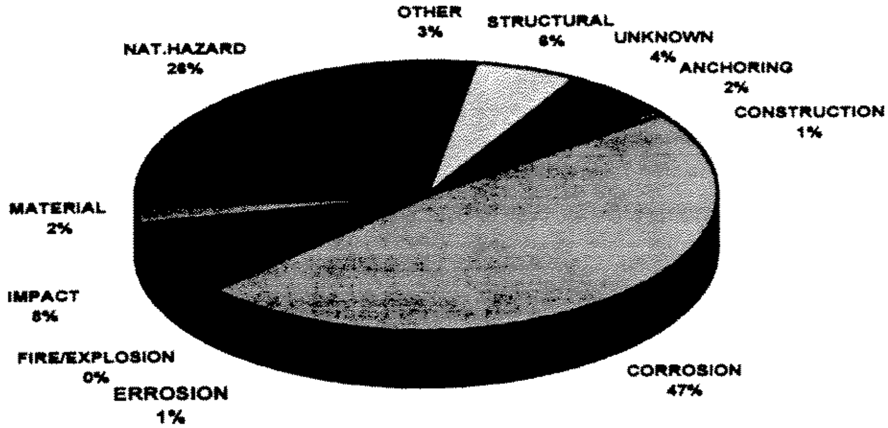

Figure 4.1 - Causes of failures of OCS pipelines in the Gulf of Mexico

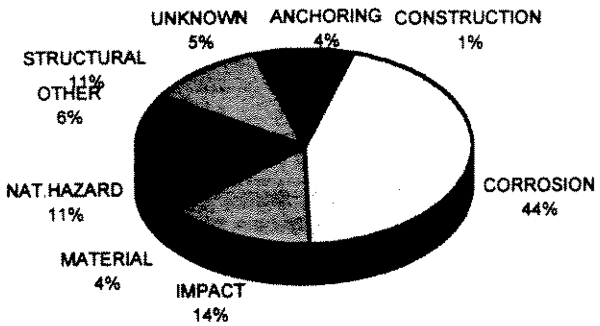

Figure 4.2 - Causes of failures in oil pipelines

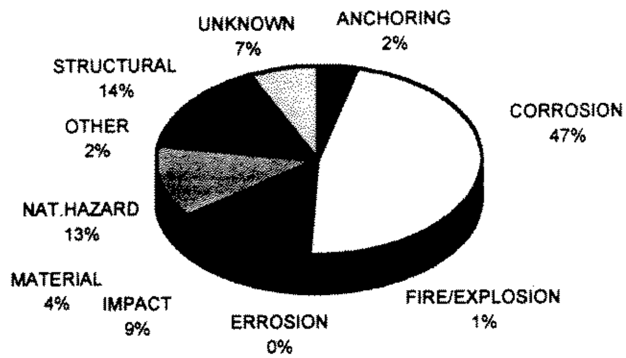

Figure 4.3 - Causes of failures in gas pipelines

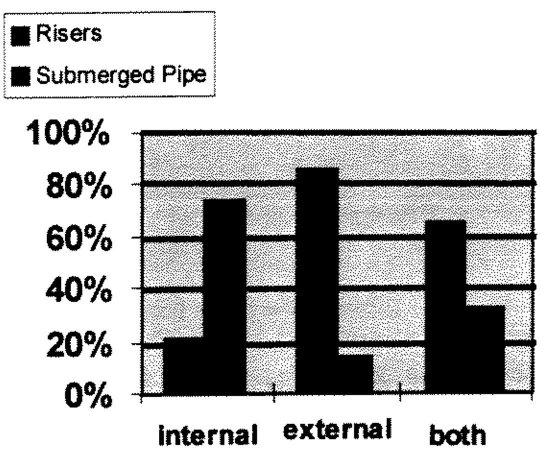

Figure 4.4 - Distribution of pipeline and riser failures due to internal and external corrosion

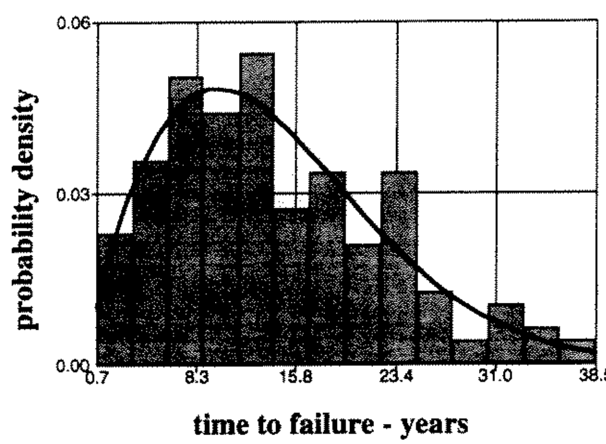

Figure 4.5 - Distribution of times to failure of gas pipelines due to internal corrosion

Figure 4.5 shows the distribution of times to corrosion failures for gas pipelines. The data includes gas pipelines with and without gas dehydration. The mean time to failure is about 10 years. The Coefficient of Variation (ratio of standard deviation to mean, COV) of the time to failure is about COV $\approx$ 100 %. This very large COV is due primarily to the natural or inherent variability in the corrosion rates and the differences in the dehydration of the gas carried by these pipelines.

Figure 4.6 summarizes the pipeline failure rate in the Gulf of Mexico OCS region during the period 1967-1997 due to all causes. The dramatic increase in the failure rate in 1992 was due primarily to hurricane Andrew.

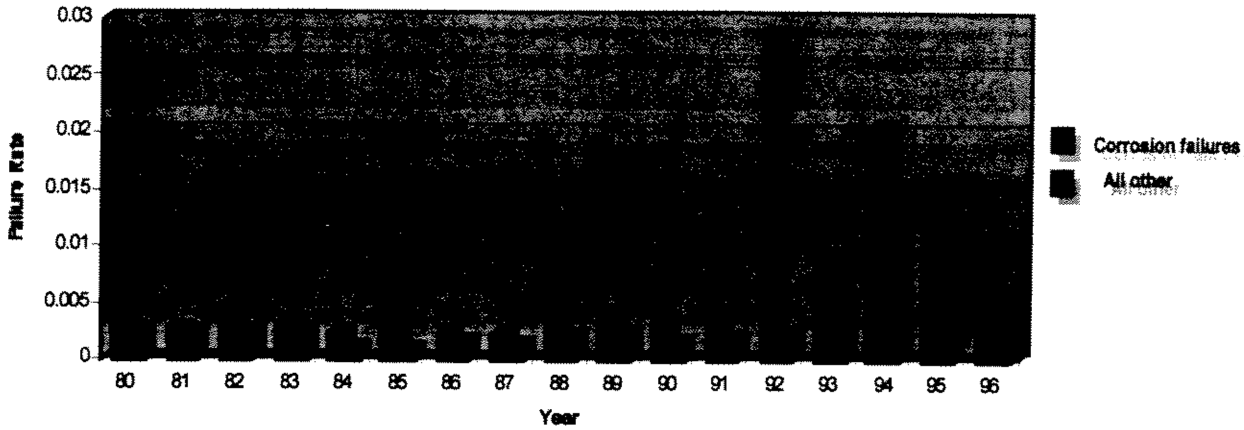

Figure 4.6 - Historic rate of failure (number of failures per mile - year) of Gulf of Mexico pipelines (OCS area, 1967-1997)

The failures that developed during the '100-year' hurricane Andrew (Figure 4.7) affected more than 10,000 'segments' of pipelines and resulted in 485 pipeline failures (Mandke, Wu, Marlow, 1995). The single largest cause of pipeline failures (52 %) was damage to the pipelines and risers caused by the platforms: movements and loss of support caused by failure of the platforms. Riser damage was chiefly due to loss of clamps and support for the riser. The non-platform related pipeline damage was due primary to hydrodynamic forces. (Collins, 1995). Mobile Offshore Drilling Units (MODUs) dragging anchors and skipping along the sea floor accounted for some damage. One MODU barely missed snagging the 36-inch diameter LOOP oil pipeline.

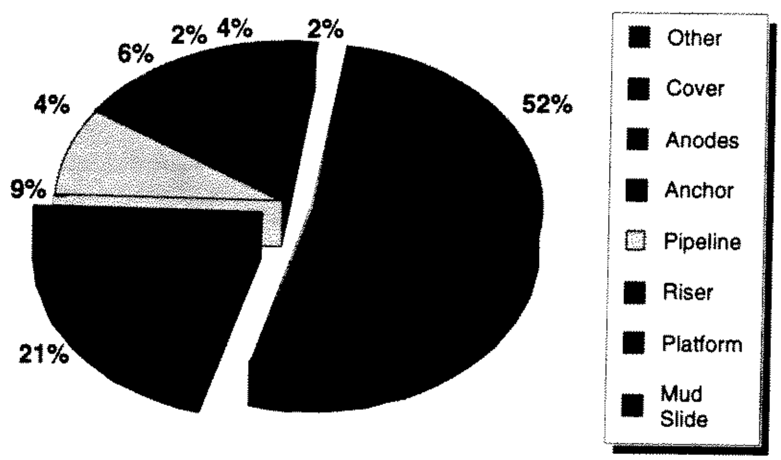

Figure 4.7 - Causes of 485 pipeline failures during hurricane Andrew (1992)

Since about 1992, there has been a dramatic increase in the number of pipeline failures due to corrosion. This is believed by some operators to be due primarily to cut-backs in pipeline maintenance budgets and efforts in the 1980's. The increase in failure rates has been noted, and the industry has taken effective measures to reduce the rates since 1994.

The failure rate has ranged from about 5 E-3 per mile-year to 2 E-2 (0.02) per mile-year. Given an 'average' pipeline in the Gulf of Mexico of 10 miles, this failure rate equates to about Pf = 5 E-2 per year to Pf = 2 E-1 per year for a 'typical' pipeline. Current operations indicate a total failure rate of about Pf = 0.01 per mile - year, or Pf = 1 E-1 = 0.1 per pipeline year.

The reference of the failure rate per pipeline per year will be discussed in the context of experience in the North Sea and current standards of practice (Sotberg, 1990). It is important to note that this failure rate has been accepted by industry, government, and public alike in the U. S. A failure rate is 'acceptable' when it has been accepted.

Figure 4.8 summarizes the historic rate of failure of Gulf of Mexico oil and gas pipelines. Oil pipelines generally have had a higher rate of failure, due chiefly to corrosion caused failures. Gas pipelines had a higher rate of failure in 1992 due to the effects of hurricane Andrew.

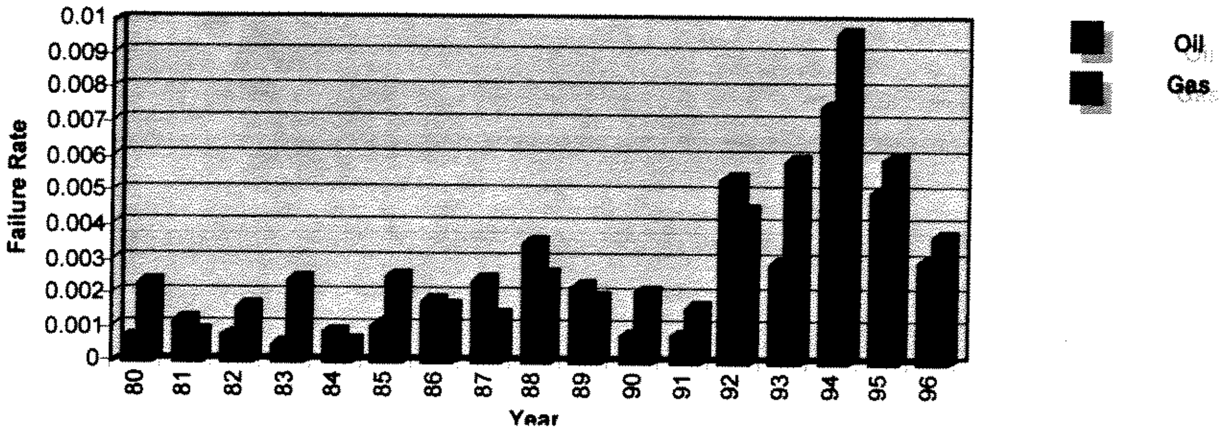

Figure 4.8 - Historic Rate of Failure of Gulf of Mexico Oil and Gas Pipelines (OCS area, 1967-1997)

4.2.2 North Sea Experience

Comparable studies have been performed of failure rates of pipelines in the North Sea (SINTEF, 1989, Sotberg, 1990; Advanced Mechanics & Engineering Ltd., 1991, 1993, 1995). The Norwegian Petroleum Directorate (NPD) study (SINTEF, 1989) indicated a pipeline failure rate in the Norwegian Sector of about Pf = 3 E-3 per km year. Given an 'average' pipeline length of 10 km, this data would indicated Pf = 3 E-2 per year. Half of this failure rate was due to spanning problems with pipelines.

The NPD study indicated a much higher rate of failure for risers. Pf was estimated to be 2 per year!. Corrosion accounted for about 1/4 of this rate of 0.5 per year. Mechanical damage (clamps, boats, dropped objects) accounted for the remaining failure rate.

Recently, the NPD has issued additional information on the rates and causes of pipeline failures in the Norwegian Sector of the North Sea (Thomsen, Leonhardsen 1998). The NPD reported that there were 2,159 major incidents and 20 loss of containment incidents during this time period.

As shown in Figure 4.9, 30 % of these incidents were due to corrosion. This rate of corrosion related failures was about the same for both oil and gas pipelines. The loss of containment incidents were confined to risers and segments of pipelines within 500 m of the platforms.

The loss of containment events equated to a failure rate of 1.7 E-2 per year. The major incident events equated to a rage of 11.1 E-2 per year. The 'Other' category of causes of pipeline incidents (31 %) relates primarily to HOF related incidents such as anchor snagging and dropped objects. These causes of failures are comparable with those in the Gulf of Mexico.

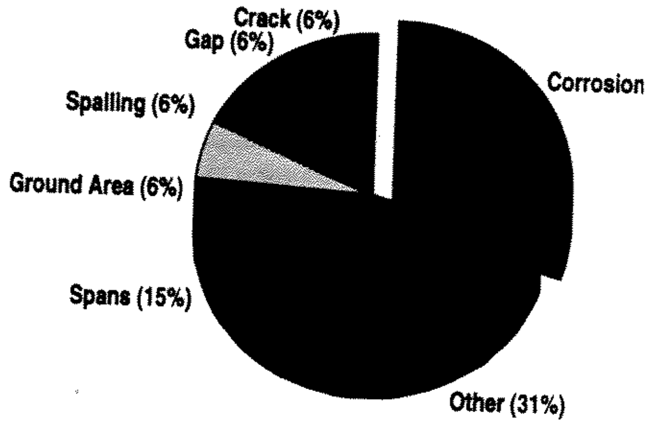

Figure 4.9 - Norwegian Sector of North Sea causes of pipeline major incidents 1975 – 1996

Figure 4.10 summarizes the loss of containment failures of pipelines and risers in the Norwegian Sector of the North Sea. It is interesting to note that the same rise in the failure rate experienced in the Gulf of Mexico in the early to mid 1990's was experienced in the Norwegian Sector. Some North Sea pipeline operators have speculated that just as in the Gulf of Mexico, when ecnomic 'hard times' hit the North Sea in the early 1980's, there were cut-backs in maintenance work and programs that were reflected in the increase in pipeline failures 10 years later.

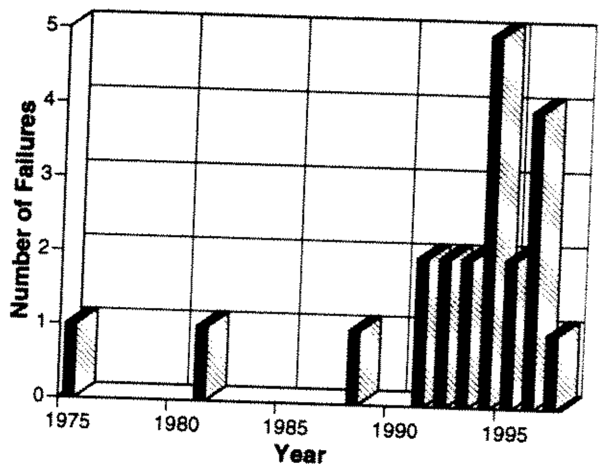

Figure 4.10 - Loss of containment pipeline and riser failures in Norwegian Sector of North Sea

The loss of containment rate of 1.7 E-2 per year in the Norwegian Sector of the North Sea compares with a rate of about 5 E-2 per year in the Gulf of Mexico. The rate of corrosion related failures is somewhat higher in the Gulf of Mexico; about 30 % in the North Sea compared with 50 % in the Gulf of Mexico.

Studies have been conducted in the UK Sector of the North Sea (E&P Forum, 1984). The study conducted by the E&P Forum indicated a failure rate of about Pf = 0.03 = 3 E-2 per year for a pipeline. The failure rate for different types and diameters of pipelines ranged between Pf = 2 E-2 and Pf = 4 E-2 per year. Failures were attributed to three causes: equipment (flanges, valves), pipeline material, and Human and Organizational Factors. Pipeline failures due to material (corrosion, welding) accounted for 10 % (gas lines) to 25% (oil lines). HOF accounted for about 40% (gas) to 70% (oil) of the pipeline failures. There were no major differences in the failure rates as a function of the pipeline diameter (a measure of the importance of the pipeline). Thus, one could conclude that this information indicates that the failure rate associated with the pipeline design has been about Pf = 3 E-3 to Pf = 1 E-2 per year per pipeline.

The E&P forum study indicated that the failure rate associated with risers was about Pf = 3 E-2 per year. The majority of the failures were due to mechanical damage and HOF related damage.

General studies of offshore pipeline accidents and loss of containment performed in the UK Sector (Canon, Lewis, 1987; Simpson, 1983) indicated that the probability of pipeline failure in recent times has been about 1 to 3 E-3 per km - year. If a 'typical' pipeline were defined as having a length of 10 km, this would indicate Pf = 1 E-2 to 3 E-2 per year. Risers were indicated to have failure rates of about 0.8 E-3 per year. A comparison with Gulf of Mexico risers indicated a failure rate in the Gulf of about 2.1 E-3 per year.

A comprehensive study of pipeline accident and loss of containment databases for the North Sea has recently been performed by AME Ltd. for the Health and Safety Executive in the UK (AME, 1995). The study involved 930 pipelines more than 17,000 km in length accounting for an operating experience of 160,000 km-years. Of 401 incidents (accidents), 73 % occurred to operating lines and 27 % to lines under construction. Of the accidents to operating lines, 40 % occurred in fittings and connections and the remaining 60 % in the pipeline or riser. Of the damage to the pipelines, 33 % resulted in loss of containment. Of the loss of containment accidents, 53 % were due to corrosion, 28 % due to human errors (impacts, dropped objects), and the remaining 19 % due to other causes, chiefly environmental loadings due to waves and currents.

Of the loss of containment events in pipelines, 36 % occurred in the Safety Zone surrounding the platform (within 500 m), 18 % to the riser, and the remaining 33 % in the mid-line portion of the pipeline. There were no shore zone failures.

The loss of containment (failure) events for risers had a frequency per year that ranged from 3.5 E-3 to 7.2 E-4 per year. Flexible risers had a failure rate that was in the range of 3.5E-3 to 1.1 E-2 per year; about an order of magnitude greater than for steel pipelines.

The loss of containment events for pipelines had a frequency per year than ranged from 1.6 E-4 per km to 8.1 E-6 per km. Given an typical pipeline length of 5 to 10 km, this would equate to about 2 E-3 to 4 E-5 per year per pipeline. The accident statistics indicate lower failure rates for large pipelines. The larger pipelines have a failure rate that is about a factor of 10 lower than the smaller lines (reflecting the consequences of failures of these pipelines).

All of the foregoing information indicates that industry, government, and the general public have accepted pipeline failure rates in the range of Pf = 1 E-3 to 1 E-4 per year per pipeline in the North Sea and Pf = 1 to 2 E-3 per year per pipeline in the Gulf of Mexico.

4.3 Standards of Practice

The DNV guidelines (1996, Sotberg, Leira, 1994; Sotberg, et al, 1996; Sotberg, et al, 1997) define three Safety Classes that are designated as Low, Normal and High. The Low Safety Classes implies no risk of human injury and minor environmental and economic consequences. The Normal Safety Class applies to normal operations or a classification for temporary conditions where failure implies risk of human injury, significant environmental pollution or very high economic or political consequences. The High Safety Class applies where failure of the pipeline or riser under normal operating conditions implies risk of human injury, significant environmental pollution, or very high economic or political consequences.

The DNV guidelines (Sotberg, et al, 1997) define the following probabilities of failure for these Safety Classes for Ultimate Limit State (ULS = burst, loss of containment, annual per pipeline or riser) and Accidental Limit State (ALS = annual, per km). Table 4.1 summarizes the annual probabilities of failure that have backgrounded the DNV guidelines:

Table 4.1 - DNV annual failure probabilities (Pf) and Safety Indices (β) for pipeline Safety Classes

Safety Class	Pf ULS	β ULS	Pf ALS	β ALS
High	1 E-4 to E-5	3.8 to 4.4	1 E-5 to E-6	4.4 to 5.0
Normal	1 E-3 to E-4	3.1 to 3.8	1 E-4 to E-5	3.8 to 4.4
Low	1 E-2 to E-3	2.3 to 3.1	1 E-3 to E-4	3.1 to 3.8

The DNV guidelines do not make any explicit provisions for the differences in target reliabilities or probabilities failure between new and existing pipelines or risers. The only differences are incorporated in definition of damage, degradation, defects that may have developed in an existing pipeline (Sotberg, et al, 1996, 1997).

The DNV guidelines indicate probabilities of failure that are substantially lower than has traditionally accepted by industry: one to two orders of magnitude lower. Recent 'history' indicates Pf = 5 E-2 to E-3 per year for engineering design (corrosion, environmental, ULS). The DNV guidelines indicate Pf = 1 E-2 to E-5 per year. The DNV failure rates are comparable with those defined for design of the platforms that connect and support the pipelines. This would appear to be an attempt to 'balance' the Pf's for these two primary components in the 'production system' (Sotberg, et al, 1997). Although not explicitly stated, it would appear from the magnitudes of the DNV guidelines that these guidelines do not include Type II (epistemic, modeling, parametric) uncertainties. Given the incluldsion of such uncertainties, these target reliabilities could be expected to increase about a factor of 10.

Moan (1995a) has considered target safety levels for reassessment of North Sea offshore platforms. This evaluation resulted in recommendations of annual target failure probabilities of 4 E-4 when the consequences are severe in terms of fatalities, significant environmental damage and economic losses, and 1 E-3 when the consequences are small, i.e. for unmanned platforms with limited likelihood of poollution, given structural failure. Moan cautiones that the failure probabilities which are compared to these targets should be calculated by the same methodology as used to establish the targets. The targets cited by Moan were based primarily on historical data and implied failure probabilities implicit in current codes. The historical data does not include Type II uncertainties. The assessment of reliabilities implicit in current codes did include assessment of Type II uncertainties.

In an extension of the study cited earlier, Moan (1995b) considered safety levels across different types of structural forms and materials implicit in codes for offshore structures. Moan assessed fixed platforms in the North Sea and U. S. waters to have annual failure probabilities in the range of 8 E-4 to 5 E-5 (given satisfactory air gaps for the platform decks). Moan found that the failure probability implied in current codes established for production vessels in the North Sea to be approximately 1 E-4 per year. Results for buoyant platforms and mooring systems were found to be in the range of 1 E-4 to 1 E-5 per year. A similar range was found for concrete gravity platforms.

4.4 Economics Approach

The assessment of desirable pipeline reliability can be evaluated from an economics standpoint and can be expressed as follows. The 'optimum' probability of failure (Pfo) is the probability of failure that produces the lowest total cost during the life of the pipeline. The optimum probability of failure can be used to define the acceptable or desirable Pf for design of new pipelines.

The pipeline total cost is the sum of the initial costs and the future costs. The initial costs are all of the costs associated with the design, construction, and commissioning of the pipeline. The future costs are all of the costs associated with the operation, maintenance, and loss of serviceability (failure). Given that the initial cost and future cost vary linearly with the logarithm of the pipeline probability of failure, the optimum probability of failure is:

$$Pfo = 0.4348 / (CF / \Delta Ci)\ PVF$$

CF are the future costs associated with loss of serviceability or quality of the pipeline (= 'failure'). ΔCi are the costs that are required to lower the probability of failure of the pipeline by a factor of 10 (incremental costs of achieving improved reliability). PVF is a present value function that brings to present day terms the future costs associated with the pipeline loss of quality or failure. Based on current industry guidelines, the PVF has been assumed to be equal to 10 in these developments. This PVF is associated with a 'long-life' system (L ≥ 25 years). For a 'short-life' system (L ≤ 5 years), PVF ≈ L.

The ratio of CF to ΔCi can be expressed as a 'Cost Ratio' (CR). Note that the CR is a dimensionless measure of the future costs of loss of serviceability to the increment in initial costs that are required to lower the probability of failure by a factor of 10. The product of the CR and PVF can be expressed as a Consequence Measure (CM) whose units are years. Thus:

$$Pfo = 0.4348 / CM$$

The 'marginal' probability of failure (Pfm) can be expressed as the total cost at which there is an equal tradeoff between cost spent on safety and cost saved by that investment. The marginal probability of failure is used to define the acceptable probability of failure for existing pipelines. This results in:

$$Pfm = 2\ Pfo$$

This result can be developed by assuming that it costs twice as much to lower the probability of failure of the pipeline by a factor of 10 after it is in place (existing) compared with achieving the same result during the design phase (new). If specific information is available on the costs associated with rehabilitation of existing pipelines, then these costs can be entered into this type of analysis and pipeline-specific economic guidelines developed.

The results of this development are summarized in Figure 4.11. A change in the product of the Cost Ratio and PVF by a factor of 10 results in a change in the Pf by about the same factor. As the future costs associated with a pipeline go up, the economics tells one to reduce the probability of failure. As the initial costs associated with reducing the probabilities of failure go up, the economics tells one to increase the probability failure. Alternatively, if it is cheap or lost cost to achieve safety, the economics guideline tells one to reduce the probability of failure.

Pipeline probabilities of failure could be defined by determining various Cost Ratio ranges and then defining the Pf's for new pipelines (Pfo) and existing pipelines (Pfm) based on these Cost Ratio ranges.

Ditlevesen (1996) has published an economics model to determine the optimal reliabilities for development of probabilistic design codes. His economics model expresses the optimum annual Safety Index as:

$$\beta o \approx \sqrt{2\log(\frac{Cf}{Cr\sqrt{2\pi}})}$$

βo is the optimum annual Safety Index, Cf are all of the costs that are associated with failure of the system, and Cr are the costs associated with changing the Safety Index by a factor of one). Ditlevesen observes that the Cf costs do not vary proportionaly with realistic monetary compensation values and that the cost of failure can only be interpreted as a socio-economic value.

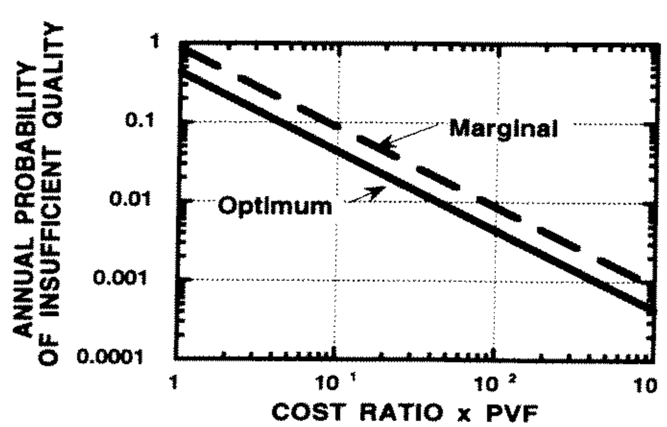

Figure 4.11 - Economics based probability of failure guidelines

Results from application of this model are summarized in Figure 4.12. Results from this model are consistent with those developed earlier in this section.

4.5 Reliability Goals

A sample of 37 existing oil and gas pipelines and risers in the Bay of Campeche were evaluated by Lara et al. (1998). Table 4.2 summarizes results from this study. The mean CM and Coefficient of Variation (COV) of the CM for the groups of pipelines and risers are indicated for both design of new pipelines and risers and reassessment of existing pipelines and risers. The oil pipelines have mean CM that are substantially greater than those for gas and mixtures of oil and gas, primarily because of the costs associated with loss of production associated with the oil pipelines. The gas risers have CM that are substantially greater than those of either oil pipelines or risers, primarily because of the costs associated with potential fires and explosions that could damage or destroy the platforms that support these risers.

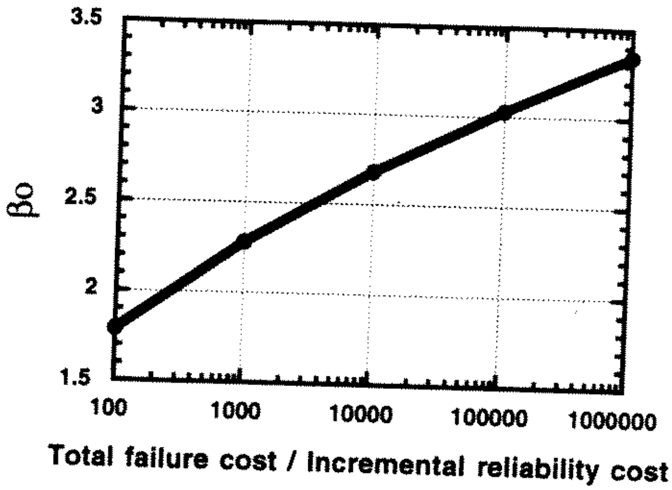

Figure 4.12 – Ditlevensen economic model to define optimum Safety Index for structural reliability codes

Two primary hazards to the Ultimate Limit Strength (ULS) of a pipeline or riser were considered in development of these criteria (Bai, Damsleth, 1997):

• **Accidents** - unanticipated defects and damage to the pipeline or riser caused by human and organizational errors, and

• **Operating** - anticipated challenges to the strength (loss of containment) of a pipeline or riser due to internal and external pressures (burst capacity), and on-bottom stability.

The bust capacity of the pipeline or riser could be influenced by internal corrosion, external corrosion, or both. The corrosion would be dependent on the protective measures provided to ameliorate corrosion (e.g. inhibitors, coatings) and the projected life of the pipeline or riser. The on-bottom stability of the pipeline or riser could be influenced by lift and drag forces induced by hurricane waves and currents and by movements of the sea floor (scour, bottom soil relative movements).

Table 4.2 - Mean and Coefficient of Variation of consequence measures and target annual Safety Indices for pipelines and risers

Type	Mean CM	COV CM %	Mean Design β	Mean Assessment β
oil pipeline	1228	10.2	3.4	3.2
gas pipeline	326	22.5	3.0	2.8
mixed pipeline	452	21.3	3.1	2.9
oil riser	1883	9.0	3.6	3.4
gas riser	7829	1.5	4.0	3.8
mixed riser	8064	1.6	4.0	3.8

The total probability of failure (Pft) could be expressed as:

$$Pft = Pfa + Pfo$$

where Pfa is the probability of failure due to accidents and Pfo is the probability of failure due to operating conditions.

In development of these criteria, based on current analyses of the causes of pipeline failures in the Gulf of Mexico, it was evaluated that Pfa = 0.50 Pft for pipelines and Pfa = 0.75 Pft for risers. Thus, Pfo = 0.50 Pft for pipelines and Pfo = 0.25 Pft for risers.

The probability of failure due to operating conditions could be expressed as:

$$Pfo = Pfp + Pfs$$

where Pfp is the probability of failure due to loss of containment in the pipeline, Pfs is the probability of failure due to hydrodynamic - geotechnical conditions.

Pfp is composed of the probabilities of loss of containment failure due to burst pressure, collapse pressure, longitudinal and transverse loadings, fatigue (high and log cycle), and combinations of these modes of loadings. Because of the correlation of these failure modes due to element-to-element correlation and failure mode correlation, the Pfp will be the mode that has the greatest probability of occurrence. Hence, for the loss of contaiment modes of failure, the design and reassessment target reliabilities will be based on Pfp.

Pfs is composed of the probabilities of loss of stability failure due to hydrodynamic and geotechnical conditions. Because of the correlation of these failure modes due to element-to-element correlation and failure mode correlation, the Pfs will be the mode that has the greatest probability of occurrence. Hence, for the loss of stability modes of failure, the design and reassessment target reliabilities will be based on Pfs.

Pfa is composed of the probabilities of failure due to accidental conditions, generally founded in human and organizational errors, including collapse due to damage inflicted on the pipelines by dropped objects (denting), dragging anchors (denting and gouging), ignored maintenace (corrosion, loss of weight coating), and other similar accidents.

4.5.1 In-Place Operating Conditions

The PEMEX - IMP assessment of the production and CM associated with the Bay of Campeche pipelines and risers (Lara, et al. 1998) indicted that the pipelines and risers could best be organized into the seven categories indicated in Table 4.3 and Table 4.4. Pipelines and risers that transmit both oil and gas are categorized as mixed lines with production given in terms of thousands of barrels equivalent per day (mbepd). Oil production is expressed in terms of thousands of barrels of production per day (mbpd) and gas production is expressed in terms of millions of cubic feet of gas production per day (mmcfpd). Both moderate and high production gas and gas - oil risers were assigned to the same category (1 GR).

These target annual Safety Indices for ULS are comparable with those incorporated into the DNV guidelines and summarized in Table 4.1. However, they are substantially lower than indicated by current experience in the Gulf of Mexico and North Sea. A risk is acceptable when it has been accepted. There are no indications that the current risks associated with pipeline operations in the Gulf of Mexico or North Sea are unacceptable to industry, government, or the publics they represent. Perhaps, the economics based target Safety Indices in Tables 4.2, 4.3, and 4.4 are too conservative due either to an over-estimate of the consequences associated with failure and / or due to an under-estimate in the costs required to achieve reliability for the in-place conditions.

Table 4.3 – Categorization of pipelines and risers and target annual Safety Indices for IN-PLACE CONDITIONS

Contents	Production	Pipelines			Risers		
		category	β new	β exist	category	β new	β exist
oil	600-1200 mbpd	1 OP	3.6	3.4	1 OR	3.8	3.6
oil	0 - 600 mbpd	2 OP	3.5	3.3	2 OR	3.7	3.5
gas, mixed	600 - 1500 mmcfd or 100 - 300 mbepd	1 GP	3.3	3.1	1 GR	4.0	3.8
gas, mixed	0 - 600 mmcfd or 0 - 100 mbepd	2 GP	3.2	3.0	1 GR	4.0	3.8

Table 4.4 - Categorization of pipelines and risers and target annual Safety Indices for ACCIDENTAL HAZARDS

Contents	Production	Pipelines			Risers		
		category	β new	β exist	category	β new	β exist
oil	600-1200 mbpd	**1 OP**	3.9	3.7	**1 OR**	3.8	3.6
oil	0 - 600 mbpd	**2 OP**	3.8	3.6	**2 OR**	3.7	3.5
gas, mixed	600 - 1500 mmcfd or 100 - 300 mbepd	**1 GP**	3.6	3.4	**1 GR**	4.0	3.8
gas, mixed	0 - 600 mmcfd or 0 - 100 mbepd	**2 GP**	3.5	3.3	**1 GR**	4.0	3.8

4.5.2 Accidental Limit State Conditions

The Accidental Limit State (ALS) is comprised of two occurrences:

1) occurrence of an accident that can cause or initiate failure of the pipeline or riser, and

2) occurrence of stresses greater than the pipeline or riser can sustain.

In a probability framework, the probability of an accident caused failure is expressed as follows:

$$Pfa = Pf_P \cap P_A = [Pf_P | A]\ [P_A]$$

Pf_P is the probability of a failure given damage to the pipeline caused by an accident. P_A is the probability that such an accident occurs.

For in-place pipelines, some useful information is available on the frequency of occurrence of damage to pipelines due to anchors, trawls, and dropped objects. The U. S. Minerals Management Service database for the northern Gulf of Mexico indicates that about 25 % of the failures of in-place pipelines are due to impact related accidents. The total failure rate is about 0.001 per pipeline. This would indicate a probability of such an incident to be $P_A \approx 4$ E-4 per year. This is comparable with data from the North Sea on the rates of accidents to in-place pipelines.

Given a total target reliability for in-place accidental conditions of 1 E-4 to 3 E-5 per year, and $P_A \approx$ 4 E-4 per year for initiating accidents would indicate a probability of failure due to ALS of $Pf_{Pp} = 2.5$ E-1 to 8 E-2 per year. A conservative value of $Pf_P = 4$ E-2 per year or $\beta = 1.7$ will be used to develop the in-place operating accidental incident criteria for reassessment of existing pipelines.

5.0 Pipeline Requalification Formulations & Criteria

The following tables summarize the pipeline requalification criteria developed during the first phase of this project for in-place operating and accidental condtions. 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, only one SSC has been identified for requalification criteria. This SSC represents the highest reliability requirements for the SSC discussed in Section 4.

Table 5.1 – 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 5.2 – 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 5.3 – 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 5.4 –In-Place Reassessment Working Stress Factors

[redacted]	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
[redacted]				
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 5.5 – In-Place Reassessment Loading Factors

LRFD Style In-Place Loadings	Demand	Demand	In-Place Pipelines	In-Place Risers
[redacted]	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
[redacted]				
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 5.6 – In-Place Reassessment Resistance Factors

[illegible]	Capacity Median Bias	Capacity Uncertainty V	Pipelines LRFD - ϕ	Risers 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
[illegible]				
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 5.7 –Analysis Equations References

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 4) 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 5.8 – Capacity Database References

Loading States (1)	Database	Capacity Analysis Equations References (3)
		Single - Reassessment
Longitudinal • Tension - **Td**	**1.1**	Taby, J., et al (1981)
• Compression - **Cd** • 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 5.9 – Formulations for Single Loading States

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

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

Table 5.10 – 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

6.0 Reassessment Formulations

6.1 Reassessment Longitudinal – Tension - Td

The following formulation will be used to evaluate the tension capacities of damaged – defective pipelines:

$$Td = 1.1SMYS(A - \Delta)$$

Δ is the cross sectional area that has been removed by corrosion. No data is available to quantify the capacity bias and uncertainty of defective – damaged pipelines subjected to tension.

In development of these criteria, a median Bias of B_{50} = 1.0 and a Coefficient of Variation of the Bias of V_B = 0.25 will be used. This high uncertainty is based on the variability of potential damage mechanisms such as corrosion, denting, and gouging in affecting the pipeline tensile capacity. Test data needs to be developed to allow improved quantification of the analytical model Bias and Coefficient of Variation.

6.2 Reassessment Compression – Local Buckling - Cld

6.2.1 Review of Design Criteria

Local buckling may become a problem in thinner tubes with D/t > 60. For long tubes, L/D > 1.0, the local buckling stress depends primarily on the D/t ratio and material strength and becomes independent of the length. The classic local elastic critical stress, σ_{xkL} , for thin tubes, that buckle in a periodic mode with 'diamond' shape buckles is expressed as:

$$\sigma_{xkL} = 2C_{xL}E\frac{t}{D}$$

where for a perfect shell, $C_{xL} = 0.605$. However, as a result of the imperfection-sensitivity associated with these shells, experimental buckling stresses as low as 20% of the classic critical stress can be observed for thin tubes. The imperfection sensitivity makes interpretation and screening of the tests results difficult and the uncertainty analysis of the design equations essential.

Rewriting the foregoing equation in the form:

$$\frac{\sigma_{xkL}}{\sigma_y} = 2C_{xL} \bullet \frac{E}{\sigma_y} \bullet \frac{t}{D})$$

leads to a non-dimensional buckling parameter, α_{xL}, given by:

$$\alpha_{xL} = \frac{E}{\sigma_y} \bullet \frac{t}{D}$$

The parameter α_{xL} has been derived from theory, and is analogous to λ_{xL} for column buckling.

There are a number of design codes for local buckling. The API/AISC codes recommend that local buckling be determined from:

$$C_{cl} = F_y \quad \text{for } \frac{D}{t} \leq 60$$

$$C_{cl} = \left(1.64 - 0.23(D/t)^{1/4}\right) \bullet F_y \quad \text{for} \frac{D}{t} > 60$$

where, C_{cl} = nominal inelastic local buckling strength in stress units

The equation developed during the RAM PIPE project is expressed as:

$$C_{cn} = SMYS\left[2.0 - 0.28\left(\frac{D}{t}\right)^{0.25}\right] \quad \text{for all } D/t$$

6.2.2 Review of Test Data

The screening of the local buckling test data was the same as that of the column buckling data. The major data source is from Ostapenko's experimental studies (1977, 1978, and 1980). Ostapenko, A. et al conducted a series of local buckling tests. The test specimens were made of ASTM-36 steel. The wall thickness varied from 0.26 to 0.38 in and D/t ratio from 59 to 233. Ostapenko et al (1979) concluded that the initial geometric imperfections and longitudinal welding residual stress appeared to have no influence on either the location or the pattern of the local buckles.

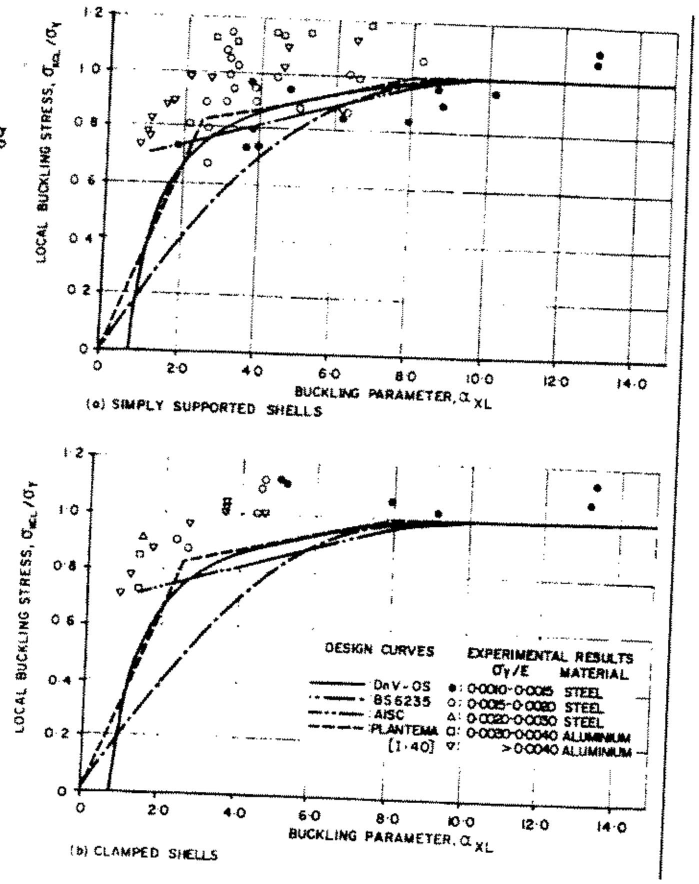

Figure 6.2.1 Comparison of Design Codes and Test Data for Local Buckling of Cylinders, Depending on Parameter α_{xkL}

6.2.3 Uncertainty Model

The uncertainty model is developed by analyzing the test data. Figures 6.2.1 illustrates the comparisons between existing design code prediction and test results. In Figure 6.2.1, the local buckling stress, σ_{xkL}, normalized by the yield stress, σ_y, has been plotted as a function of the local buckling parameter, α_{xkL}. It is apparent in Figure 6.2.1 that there is the substantial disagreement between the various existing design recommendations especially for $\alpha_{xkL} < 7$. The test data is presented according to the value of the σ_y / E ratio.

In the case of simply supported shells, shown in Figure 6.2.1 (a), the correlation between design curves and tests results is very poor for low σ_y / E ratios but improves with increasing σ_y / E with most curves providing lower bounds to tests when $\sigma_y / E > 0.002$. In the case of clamped supported shells in Figure A.2-3(b) shows that, for the small number of tests results available, all design equations form lower bounds. In both figures there is also a general trend for test results to segregate in bands according to the σ_y / E ratio. This prompted to suggest that there is a separate or different σ_y / E effect on the local buckling stress which serves the basis of the two piece API design equation.

Figure 6.2.2 illustrates the uncertainty analysis of the new local buckling design equation developed in the RAM PIPE project. The bias and COV are 1.1 and 7.03%.

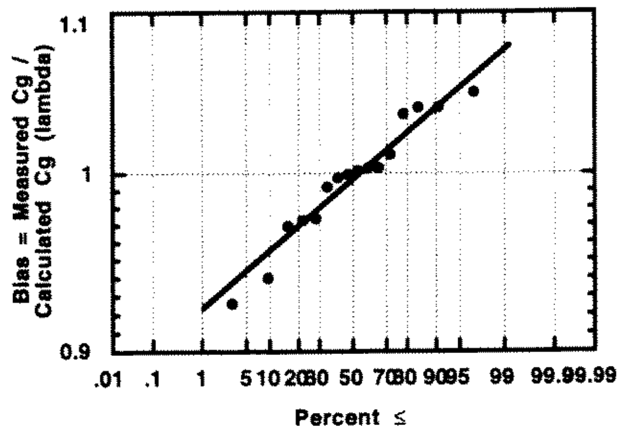

Figure 6.2.2 Uncertainty Analysis of the RAM PIPE Local Buckling Equation

6.2.4 RAM PIPE Local Buckling Equation

Given the foregoing discussion, the RAM PIPE REQUAL local buckling criteria for undamaged – defective pipelines is:

$$C_{cn} = 1.1SMYS\left[2.0 - 0.28\left(\frac{D}{t}\right)^{0.25}\right] \quad \text{for all D/t}$$

This formulation has a median Bias of 1.0 and Coefficient of Variation of the Bias of 6.13%.

The following formulation will be used to evaluate the compression local buckling of damaged – defective pipelines:

$$Cl = 1.1 \bullet SMYS\left(2.0 - 0.28\left(D/t_{\min}\right)^{1/4}\right) \bullet A \bullet Kd$$

$$Kd = 1 + 3\,fd\,(D/t)$$

No data was developed during Phase 1 to quantify the capacity bias and uncertainty of defective – damaged pipelines subjected to compression resulting in local buckling.

In development of these criteria, a median Bias of B_{50} = 1.0 and a Coefficient of Variation of the Bias of V_B = 0.25 will be used. This high uncertainty is based on the variability of potential damage mechanisms such as corrosion, denting, and gouging in affecting the pipeline local buckling capacity. Test data needs to be developed to allow improved quantification of the analytical model Bias and Coefficient of Variation.

6.3 Reassessment Compression – Global Buckling – Cgd

6.3.1 Review of Design Criteria

Long tubes with reduced slenderness ratios, $\lambda_{xE} > 1.5$, usually fail by elastic buckling at the 'Euler' stress, σ_{xkE}, defined by

$$\frac{\sigma_{xke}}{\sigma_y} = \frac{1}{\lambda_{xE}^2}$$

where, $\lambda_{xE} = \frac{KL}{r_i} \bullet \frac{1}{\pi}\sqrt{\frac{\sigma_y}{E}}$

Here the column length is L, its radius of gyration, $r_i \left(r_i = \sqrt{I/A}\right)$ and the effective length factor, which depends on the boundary conditions, is K.

The failure of short pipe is normally governed by material strength, is inelastic and strongly influenced by initial imperfections, residual stresses and boundary conditions.

Tube buckling under pure compression has been investigated for several decades. The results of this extensive research culminated in the simple, mainly semi-empirical formulations given in different design codes. On of the leading standards is the American Petroleum Institute Standards.

The column buckling criteria in API and AISC codes are:

$$C_{cn} = \left(1.0 - 0.25\lambda^2\right) \bullet F_y \quad \text{for } \lambda < \sqrt{2}$$

$$C_{cn} = \frac{1}{\lambda^2} F_y \quad \text{for } \lambda \geq \sqrt{2}$$

$$\lambda = \frac{KL}{\pi r}\left[\frac{F_y}{E}\right]^{0.5}$$

The foregoing analytial model provides a lower bound fit to the experimental data. Its mean value is expressed as:

$$C_{cn} = (1.03 - 0.24\lambda^2) \bullet F_y$$

A new design equation was developed in the RAM PIPE project (Bea, et al, 1998). The equation is expressed as:

$$C_{cn} = 1.1(1.2 - 0.25\lambda^2) \bullet F_y$$
$$F_y = SMYS \bullet A$$

6.3.2 Review of Test Data

Starting with the first detailed experimental test on the axial compressive buckling of circular tube sections by Robertson (1929) in 1929, a large number of tests were conducted. Chen, W.F., et al (1978) conducted 10 long fabricated tube column buckling tests. In Chen's tests, a number of stub column tests were made along with the measurements of residual stress in addition to the investigation of the behavior, strength and failure modes. Ellinas, C. P., et al (1986) collected 180 column buckling tests.

Only the most reliable test data were used in the RAM PIPE project. The test data were screened based on:

- Manufacture methods (fabricated, seamless welding)
- The specimen geometry (length, diameter, thickness, D/t)
- Material properties, (yield stress, ultimate stress, elastic modulus)
- Boundary conditions , effective length factors

- Applied loading
- Experimental collapse load

6.3.3 Uncertainty Model

Given the discussion of the design criteria, and tests data, a uncertainty model is developed for the design criteria. Figures 6.3-1 and 6.3-2 illustrate the uncertainty analysis of the equation (6.3-4), the equation developed by the RAM pipe. It is concluded that the median Bias and COV of the equation (6.3-4) is 1.0 and 7.8%.

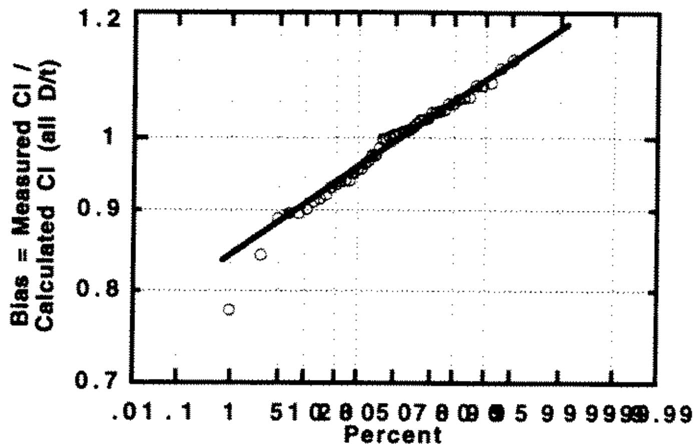

Figure 6.3-1 The Uncertainty Analysis of the RAM PIPE Equation

6.3.4 RAM PIPE Column Buckling Formulations

Given the discussion of the design criteria, review of the test data, and analysis of the uncertainty associated with the design equations, the RAM PIPE column buckling equation is recommended in the proposed RAM PIPEcriteria with 1.0 and 7.8% of the median bias and COV, respectively.

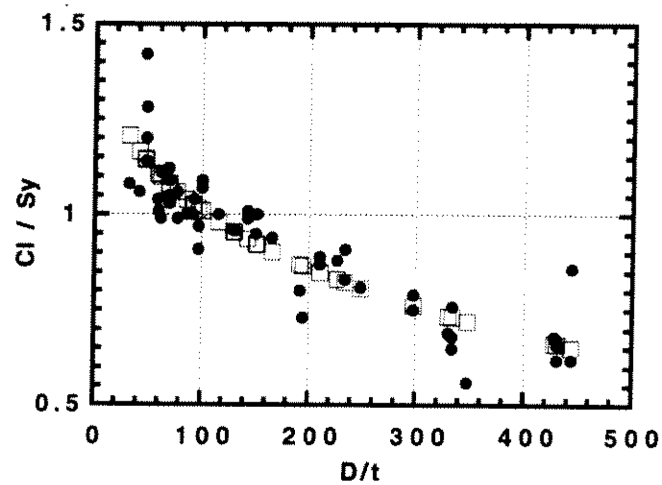

Figure 6.3-2 The RAM PIPE Column Buckling Equation

The following formulation will be used to evaluate the compression global buckling of damaged – defective pipelines:

$$Cg = 1.1SMYS\left(1.2 - 0.25\lambda^2\right) \bullet A$$

where: $\lambda = \frac{KL}{\pi r}\left[\frac{SMYS}{E}\right]^{0.5}$

$$\frac{P_{crd}}{P_{crdo}} + \frac{P_{crd}\Delta Y}{\left(1 - \frac{P_{crd}}{P_{Ed}}\right)M_{ud}} \leq 1.0$$

$$\lambda_d = \left(P_{ud} / P_{ed}\right)^{0.5}$$

$$P_{ud} = P_u \frac{A_d}{A_o} = P_u \exp\left(-0.08\frac{\Delta}{t}\right)$$

Dent Damage

Dent-damaged tubular bracing members have been analytically studied since late 70's. The analytical methods of strength prediction developed so far can be classified into three categories (Ricles, 1993):

- Beam-column analysis (Ellinas, 1984, Ricles et al, 1992, Loh, 1993)
- Numerical integration methods (Kim, 1992)

- Nonlinear finite element (FE) methods

Beam-column analysis is based on formulation of equilibrium of the damaged member in its deformed shape. The P-δ effects including the effects of out-of-straightness are considered in the equilibrium equations. The effect of dent depth is taken into account by modifying the cross-sectional properties. Numerical integration methods use empirical moment-axial load-curvature relationships to iteratively solve the differential equation of axially loaded damaged member. The empirical M-P-Φ relationship is usually based on experimental test results or finite element studies of dented tubular segments. Nonlinear FE analyses represent the most general and rigorous method of analysis. However, their accuracy and efficiency require evaluation and they are expensive and time consuming to perform.

Developed at Exxon Production Research Company, BCDENT is a general computer program that uses M-P-Φ approach to evaluate the full behavior of dented member (Loh, 1993). The behavior of the dent section is treated phenomenologically using a set of M-P-Φ expressions. Compared with the experimental results, BCDENT gives mean strength predictions for both dented and undented members. Based on BCDENT results, Loh (1993) presented a set of new unity check equations for evaluating the residual strength of dented tubular members. The unity check equations have been calibrated to the lower bound of all existing test data. The equations cover axial compression and tension loading, in combination with multi-directional bending with respect to dent orientation. When the dent depth approaches zero, the recommended equations are identical to API RP 2A equation for undamaged members(API, 1993b). Loh's equations for dent damaged members and those with global bending damage have been integrated into the development of the RAM PIPE REQUAL formulations, criteria, and guidelines.

Based on a comparison between the experimental ultimate capacities and the corresponding predicted capacities of dented tubulars using different methods of analysis, Ricles (1993) concluded that Ellinas' formulation, which is based on first yield in the dent saddle, is overly conservative. In general, it has been found that Ellinas' approach can be either conservative or unconservative depending on the dent depth, member slenderness, and out-of-straightness. Ricles further concluded that DENTA (a computer program developed by Taby (1988)), Loh's interaction equations, numerical integration based on M-P-Φ relationships, and the nonlinear FEM are able to predict the capacity of the test members reasonably well.

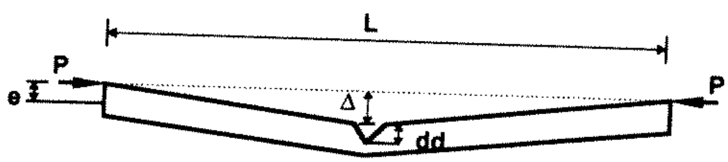

Figure 6.3.3- Definition Sketch for a Dent Damaged Pipeline

A joint industry project on testing and evaluation of damaged jacket braces was performed by PMB Engineering and Texas A&M University (1990). Twenty salvaged braces were tested and their strength behavior compared with results gained from analyses using finite element beam column models of damaged braces. It was found that on average the analyses would overpredict the capacities by 21%. The agreement in this case is not as good as that presented by other investigators. Use of new and artificially damaged braces in other investigations may explain this inconsistency. Generally, corrosion is found to add large uncertainties to the properties of the entire member. Figure 6.3.3 shows the definition sketch of a dent-damaged member with global out-of-straightness.

Using ultimate capacity equations formulated by Ellinas (1984) and Loh (1993), the ratio of damaged compressive capacity over intact buckling capacity was estimated for ten tubular braces. The intact buckling capacity of a tubular brace was taken to be that given by API (1993b). The capacity ratios are plotted for two separate cases. Figure 6.3.4 shows the results for no dent damage and varying global out-of-straightness, whereas Figure 6.3.5 shows the results for no global bending damage and varying dent depth. In case of global bending damage, the two sets of results are in close agreement indicating that the second-order P-δ effects are captured coherently by both sets of formulations.

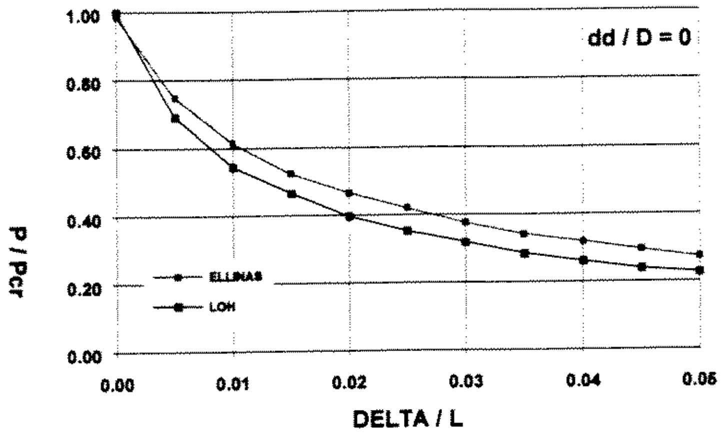

Figure 6.3.4 Comparison of Capacity Predictions for Tubulars with Global Bending Damage

In case of dent-damaged tubulars, however, the results indicate significant differences in capacity predictions by the two sets of formulations. These results confirm those previously published in the literature regarding the level of conservatism of capacity equations developed by Ellinas. An attempt was made to compare the results of different theoretical approaches to predict the compressive capacities of damaged tubulars. Nine specimen were selected from a database that represents all of the test results currently in the public domain (Loh, et al., 1992). Table 6.1 contains the member sizes and material and damage properties. The test results are compared with those gained from the programs BCDENT (Loh, 1993), UC-DENT (Ricles et al., 1992), and capacity equations given by Ellinas (1983) and Loh (1993). The numerical results are given in Table 6.2 and plotted in Figure 6.3.6. The results indicate that for the data points presented, BCDENT capacity predictions are unbiased. Loh's formulations lead to capacity predictions that are close lower bounds of test results. Ellinas' formulation is in most cases overly conservative. UC-DENT predicts capacities that are close approximations of test results.

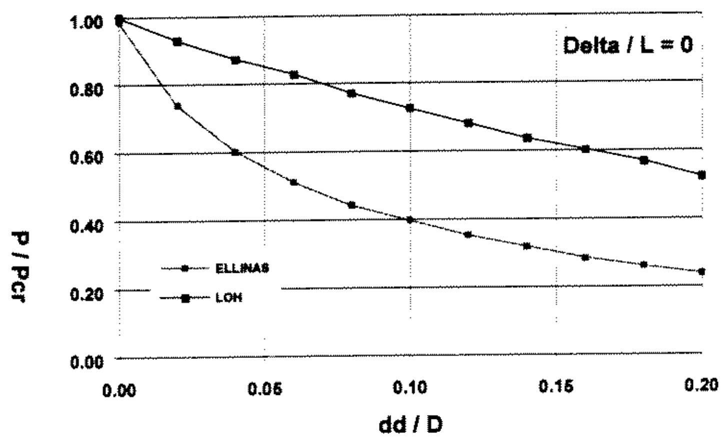

Figure 6.3.5 - Comparison of Capacity Predictions for Tubulars with Dent Damage

Based on experimental test results and parametric studies using different analytical methods, the following observations are developed:

- The residual strength decreases significantly as the dent depth increases.
- For a given dent depth, the analyses show a decrease in residual strength for members with higher D/t ratio.
- The axial compression capacity decreases as the out-of-straightness increases, but the impact on ultimate moment is negligible.
- A mid-length dent location can be assumed for any dent within the middle-half section of members effective length.
- Accounting for strain hardening has only a small effect on the maximum predicted capacity. Lateral loadings, such as those caused by wave forces, can significantly affect dented brace capacity.
- The behavior of members with multiple forms of damage are generally dominated by one damage site.

Table 6.1 - Test Specimen Properties

Test	D [IN]	t [IN]	L [IN]	Sy [KSI]	E [KSI]	dd/D [%]	delta/L [%]	e/L [%]
A1	2.50	0.08	84.63	33.06	29145		0.02	
A2	2.50	0.08	84.63	33.21	30160		0.03	0.46
A3	2.50	0.08	84.63	32.77	28710	0.046	0.55	
B3	3.13	0.07	84.63	28.71	31030	0.08	0.5	
C1	4.00	0.07	84.63	30.60	29145		0.05	
C2	4.00	0.07	84.63	41.18	29870		0.05	0.46
C3	4.00	0.07	84.63	33.79	28565	0.034	0.04	
F1	16.02	0.39	305.24	44.23	28710		0.07	
F2	15.98	0.39	305.24	42.49	31030	0.124	0.18	

Table 6.2 - Experimental and Theoretical Capacities of Damaged Tubulars

Test	dd/D	delta/L [%]	e/L [%]	Ptest [KN]	BCDENT [KN]	LOH [KN]	ELLINAS [KN]	RICLES [KN]
A1		0.02		78.10	76.50	63.46	60.94	
A2		0.03	0.46	46.00	41.60	38.86	41.88	
A3	0.05	0.55		44.20	43.80	33.68	28.23	
B3	0.08	0.50		43.30	41.50	35.97	25.04	43.96
C1		0.05		121.00	104.80	95.37	96.48	119.66
C2		0.05	0.46	89.40	97.10	90.66	97.84	
C3	0.03	0.04		95.70	101.90	93.61	68.85	86.00
F1		0.07		3238.70	3509.90	3160.30	3192.10	3862.30
F2	0.12	0.18		2056.90	2031.70	1962.40	1068.00	2051.40

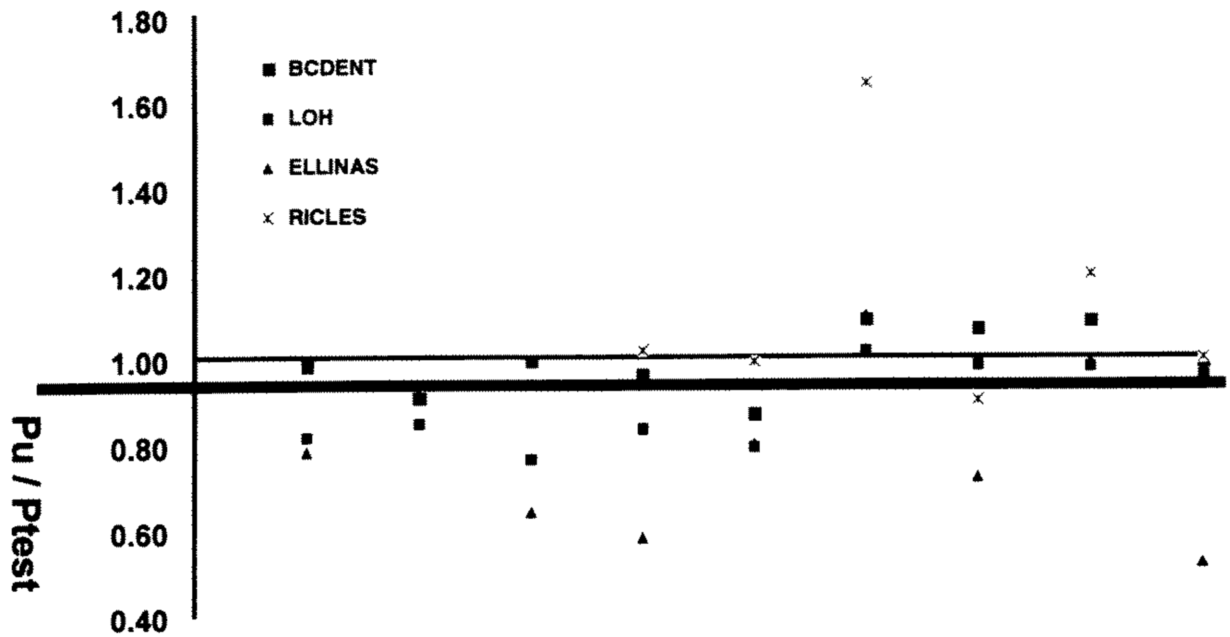

Figure 6.3.6 - Comparison of Capacity Predictions for Tubulars with Dents and Global Bending Damage

Loh's Interaction Equations for Dent-Damaged Tubular Members

Undamaged Cross Sectional Capacities

$$P_u = F_y A_o \qquad \text{for} \qquad \frac{D}{t} \leq 60$$

$$P_u = F_y A_o \left[1.64 - 0.23 \left(\frac{D}{t} \right)^{0.25} \right] \qquad \text{for} \qquad \frac{D}{t} \geq 60$$

$$\frac{M_u}{M_P} = 1.0 \qquad \text{for} \qquad 0 \leq \frac{F_y D}{t} \leq 1500(ksi)$$

$$\frac{M_u}{M_P} = 1.13 - 2.58 \frac{F_y D}{Et} \qquad \text{for} \qquad 1500 \leq \frac{F_y D}{t} \leq 3000$$

$$\frac{M_u}{M_P} = 0.94 - 0.76 \frac{F_y D}{Et} \qquad \text{for}$$

Dent-Section Properties

$$\frac{P_{ud}}{P_u} = \frac{A_d}{A_o} = \exp\left(-0.08\frac{dd}{t}\right) \geq 0.45$$

$$\frac{M_{ud}}{M_u} = \frac{I_d}{I_o} = \exp\left(-0.06\frac{dd}{t}\right) \geq 0.55$$

Strength Check

$$UC = \frac{P}{P_{ud}} + \sqrt{\left(\frac{M-}{M_{ud}}\right)^{\alpha} + \left(\frac{M*}{M_u}\right)^2} \leq 1.0$$

$$UC = \frac{P}{P_{ud}} + \sqrt{\left(\frac{M+}{M_u}\right)^2 + \left(\frac{M*}{M_u}\right)^2} \leq 1.0$$

Stability Check

$$UC = \frac{P}{P_{crd}} + \sqrt{\left(\frac{M-}{\left(1-\frac{P}{P_{Ed}}\right)M_{ud}}\right)^{\alpha} + \left(\frac{M*}{\left(1-\frac{P}{P_E}\right)M_u}\right)^2} \leq 1.0$$

$$UC = \frac{P}{P_{crd}} + \sqrt{\left(\frac{M+}{\left(1-\frac{P}{P_{Ed}}\right)M_{ud}}\right)^2 + \left(\frac{M*}{\left(1-\frac{P}{P_E}\right)M_u}\right)^2} \leq 1.0$$

$\alpha = 2 - 3\,dd / D$

Critical Buckling Capacities

$$P_{crdo} = P_{ud}\left[1 - 0.25\,\lambda_d^2\right] \quad \text{for} \quad \lambda_d \leq \sqrt{2}$$

$$P_{crdo} = P_{ud}\frac{1}{\lambda_d^2} = P_{ED} \quad \text{for} \quad \lambda_d \geq \sqrt{2}$$

$$\frac{P_{crd}}{P_{crd0}} + \frac{P_{crd}\,\Delta Y}{\left(1 - \dfrac{P_{crd}}{P_{Ed}}\right)M_{ud}} = 1.0$$

General and Pitting Corrosion

Based on parametric analytical studies performed by Ricles et al. (1995), it was determined that tubular brace capacity decreases linearly as the extent of corrosive damage increases (Figure 6.3.7). A significant reduction in capacity occurs due to the initiation of local buckling in the corrosion patches. This local buckling is attributed to the effect of a reduced wall thickness. For the purposes of RAM PIPE REQUAL guidelines, general corrosion was assumed to be uniform. Therefore, this damage was simulated by reducing the wall thickness of the pipeline.

Summary

Limited data was developed during Phase 1 to quantify the capacity bias and uncertainty of defective – damaged pipelines subjected to compression resulting in global buckling.

In development of these criteria, a median Bias of B_{50} = 1.0 and a Coefficient of Variation of the Bias of V_B = 0.25 will be used. This high uncertainty is based on the variability of potential damage mechanisms such as corrosion, denting, and gouging in affecting the pipeline global buckling capacity. Test data needs to be developed to allow improved quantification of the analytical model Bias and Coefficient of Variation.

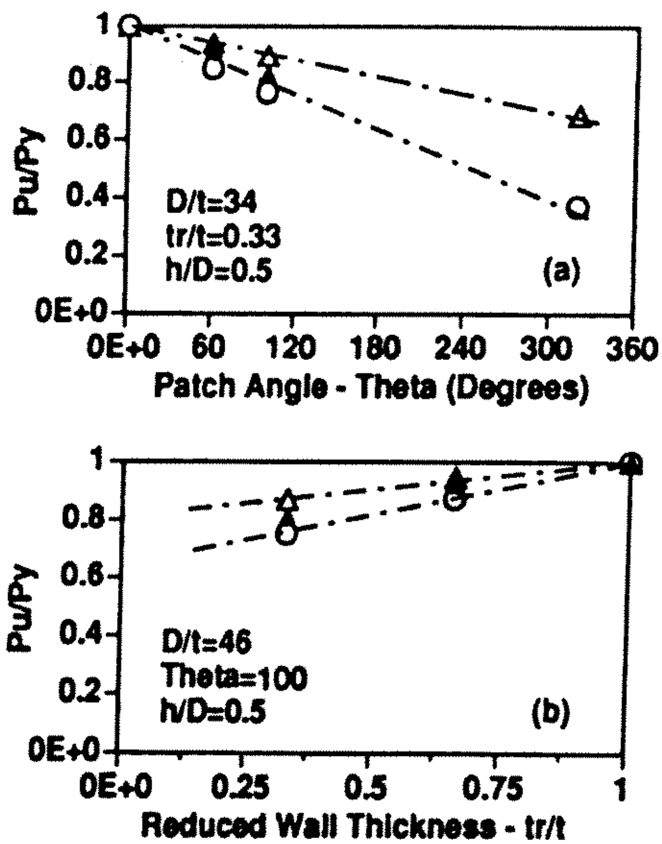

Figure 6.3.7 - Effects of General Corrosion on Axial Compressive Load Capacity

Notation

A_d	effective cross-sectional area of dent section
A_0	cross-sectional area of undamaged member
A_{st}	cross-sectional area of the steel
A_s	cross-sectional area of the soil plug in pile
D	outside diameter of tubular member
dd	dent depth
ΔY	primary out-of-straightness of a dented member
ΔY_0	$=0.001\,L$
E	Young's modulus
f_y	yield stress
I_d	effective moment of inertia of dent cross-section
I_0	moment of inertia of undamaged cross-section
K_0	effective length factor of undamaged member
K	effective buckling length factor
L	unbraced member length
λ	slenderness ratio
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_p	plastic moment capacity of undamaged member
M_{ud}	ultimate negative moment capacity of dent section
$M-$	negative moment for dent section
$M+$	positive moment for dent section
M^*	neutral moment for 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_u	axial compression capacity
P_{ud}	axial compression capacity of a short dented member
P_{crl}	axial local buckling capacity
P_{cr}	axial column buckling capacity
P_y	tensile capacity
r	radius of gyration
t	member wall thickness
UC	unity check

6.4 Reasssessment Transverse Bending – Md

Figure 6.4-1 illustrates the typical behavior of a pure bending load applied to pipe as described by Murphey and Langner (Murphey, et al 1985). As bending moment is applied to the pipe, curvature is induced. The moment-strain relationship is initially linear. As the bending moment is increased, the outermost fibers of the pipe begin to reach the proportional limit stress (point A), plastic deformation initiates and the moment-strain relationship becomes increasingly nonlinear and ovalization of the pipe occurs. Although permanent curvatures are induced, the geometry remains stable due to strain-hardening of the material.

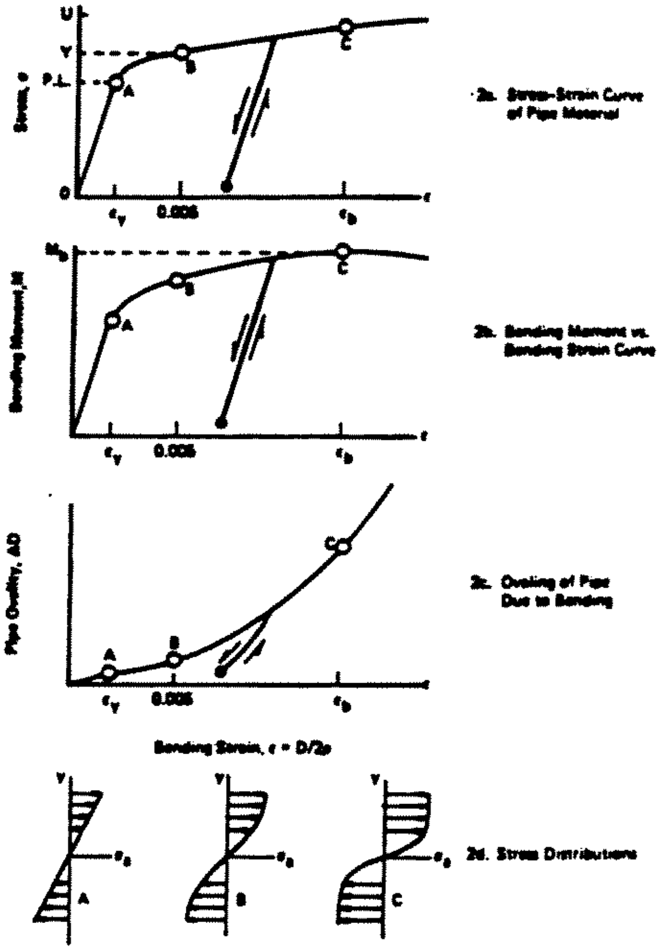

Figure 7.4-1 Mechanical behavior of Pipe in Pure Bending

Beyond yield level strains, ovalization of the pipe wall increases rapidly, further reducing the slope of the moment strain diagram. Eventually the slope becomes zero and the maximum moment is achieved when ovalization effects overcome strain-hardening and buckling is imminent. At the point of maximum moment the pipe has no more reserve stiffness to resist buckling. If the applied loads are not reduced at this point, severe deformation occurs. In a displacement controlled condition, buckling initiates and the pipe deforms to the imposed displacement. In a load controlled condition, failure occurs.

The bending strains and curvatures at the point of maximum moment are defined as the critical buckling strains and curvatures. For thin wall pipe (high D/t ratio) a small amplitude wave or wrinkle may become visiable (Figure 6.4-2) prior to achieving the maximum moment capacity of the pipe. The pipe bending behavior remains stable, however, until the maximum moment is reached.

The diagram in Figure 6.4-1 are representative of behavior for unpressurized pipe with D/t less than approximately 35. For higher D/t's (i.e., thinner wall pipe), a point of instability and inward buckle may occur before the point of zero slope on the moment curvature diagram. Figure 6.4-3 compares examples of moment bending strain diagrams for three D/t ratios.

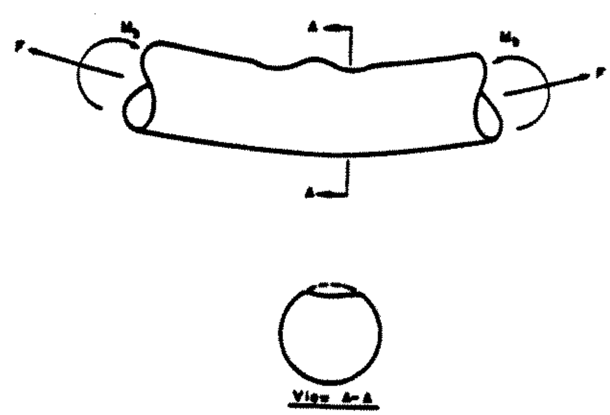

Figure 6.4-2 Exaggerated View of Small Amplitude Buckle in Thin Wall Pipe

6.4.1 Review of Design Criteria

Strain Based Criteria. A critical strain equation for pipes under pure bending is provided by BSI 8010 where:

$$\varepsilon_{cr} = 15 \bullet \left(\frac{t_{nom}}{D_0} \right)^2$$

DNV 96 assumes that critical bending strain is a linear function of t_{nom}/D. API RP 1111 states that ε_{cr} is a linear function of $t_{nom}/2D$.

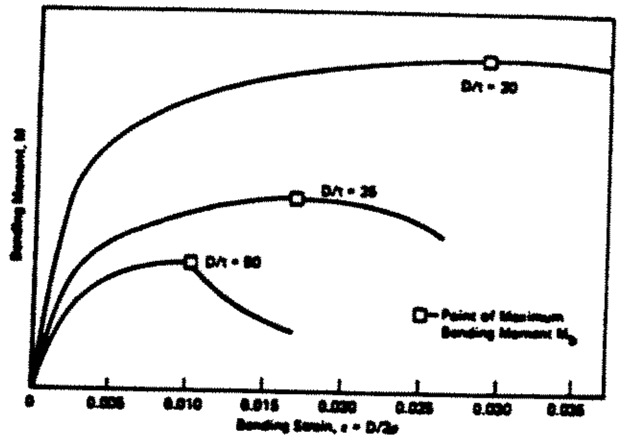

Figure 6.4-3 Moment versus Strain Curves for Constant Diameter and Yield Stress But Variable Wall Thickness

Bai et al (1994) developed that the critical bending strain corresponding to the maximum moment capacity point is:

$$\varepsilon_{cr} = 0.6275 \bullet f_1(n) \bullet f_2(n) \bullet \left(\frac{t_{nom}}{D} \right)$$

where $f_1(n)$ is a function of hardening parameter n in the Ramberg-Osgood equation:

$$f_1(n) = 0.5 + \frac{8.79}{n} - \frac{21.128}{n^2}$$

where $f_1(S)$ is a function of yield anisotropy parameter S which is yield stress in the hoop direction divided by yield stress in the longitudinal direction:

$$f_2(S) = -0.185 + 1.19S$$

The Bai's equations for critical bending strain have been developed based on extensive finite element simulation using ABAQUS where the accuracy of the equations were validated using experimental data. In other words, the parameter equations were based on experiments with functions defined from analytical and numerical studies. Because the strain hardening parameter n and anisotropy S are available, it is easy to apply the equations in pipeline design.

The BSI 8010 equations assume that critical strain is proportional to $(t_{nom}/D_0)^2$ is not based on analytical considerations. It is based on the experimental test data where some tests were not conducted for pipeline material. Therefore, the BSI equations obtained from fitting curve to experimental results are not strictly applicable to pipelines since material properties significantly affect the critical strain. In some cases, the critical strain obtained from experiments corresponds to the ultimate moment point and in other cases to the local buckling point. It is difficult to define the local buckling point since a sudden reduction of load-carrying capacity is not obvious for thick walled tube. In addition, because of the large and localized strain, the measurement of critical strain is not accurate. This is why the BSI equation overestimates critical bending strain of pipes of small t_{nom}/D_0.

Theoretically, the critical bending strains for deformation controlled situations and load controlled situations correspond to the local buckling point and the maximum moment point, respectively. For typical pipeline materials, local buckling occurs after the maximum moment point for diameter to thickness ratio D/t larger than 35. Since the allowable strain is higher for D/t far less 30, our main interest is to accurately calculate the critical strain for D/t between 30 and 35. Within this region, the critical strains due to local buckling and maximum moment due not deviate significantly. It is also not recommended to use the margin between the local buckling point and the maximum moment point for pipes under combined external pressure and bending because a small amount of the strain is load controlled, even though the situation is categorized as deformation controlled.

Stress Based Criteria. In terms of stress based criteria, SUPERB and DNV 96 recommend that the bending moment capacity is:

$$M_u = M_p = \pi D_0^2 t_{nom} \sigma_y$$

The RAM PIPE project proposed the following three equations for the pure bending capacity of pipelines:

$$M_u = 1.1\pi D_0^2 t_{nom} \sigma_y \left(1 - 0.001\frac{D_0}{t_{nom}}\right)$$

or

$$M_u = 1.1\pi D_0^2 t_{nom} \sigma_y \left(1 - 0.002\frac{D_0}{t_{nom}}\right)$$

or

$$M_u = 1.13 M_p \exp(-X)$$
$$M_p = \sigma_y D_0^2 t_{nom}$$
$$X = \frac{\sigma_y D_0}{E t_{nom}}$$

6.4.2 Review of Test Data

The amount of test data for tubes under pure longitudinal bending, are relatively restricted. Sherman (1976, 1984) presents a review of tests on fabricated pipes with geometrical and material characteristics of cylindrical members in offshore structures. Attempts have been made to establish bending limit state criteria based on the full capacity of the tubular cross section. Schilling's tests (Schilling, 1965) derived the following parameter α which characterizes the plastic moment:

$$\alpha = \frac{E/\sigma_y}{D/t}$$

Scilling (1965) indicated a lower limit of 8.8 for the parameter α.

Uncertainties about the extrapolation of the tubular test results to long pipes, as far as the plastic moment capacity is concerned, led to the testing programs of large scale pipe beams (Jirsa, et al 1972, Sherman, et al 1976, and Korol, 1979). The results indicated that some tubular sections with α greater than 8.8 did not achieve the plastic moment which implied that the limit for a compact section, was higher than anticipated up to this time, generating an inelastic local buckling criteria for longitudinal bending that is different from axial compression.

Jirsa et al (1972) reported six tests of pipe under pure bending, with diameter varying from 10 to 20 in and D/t from 30 to 78.

Sherman (1976) presents the experimental tests data of the tubes under pure bending. The tubes are with outside diameter of 10.75 in and D/t ratios from 18 to 102. Sherman (1976) concluded that the members with D/t of 35 or less can develop a fully plastic moment and sustain sufficient rotation to fully redistribute the moments in fixed end beams. This conclusion was demonstrated for pipe spans up to 22 diameters. In addition, Sherman (1976) concluded that tubes made by Electric Resistance Welded (ERW) could not develop the full plastic moment at as large a D/t as that proposed by Schilling (1965).

Korol (1979) performed a series of nine tests on single span circular hollow tubular beams with D/t ratios from 28.9 to 80.0. Korol (1979) concluded that the buckling strain was found to be inversely proportional to yield stress raised to an exponent factor between 0.5 and 1.0 for ductile materials that possess an essentially bilinear stress-strain curve and a small degree of strain hardening. This exponent factor tends to be 1.0 for elastic-perfectly plastic materials. For a high tangent modulus and small D/t pipe, it tends towards zero.

Sherman (1986) reviewed six experimental research programs which contain test on cylinders with unstiffened constant-moment regions. A total of 53 tests were included in the review. The test specimens were hot formed seamless pipe , electric resistance welded tubes and fabricated pipes. The diameters ranged from 4 to 60 inches. However, in most cases the diameters were between 10 and 24 inches.

Two tests of the test series conducted by Sternmann et al (1989) for beam columns were included in the tests database development. These tests were for tubulars with nominal D/t ratio of 42, the outside diameter of 6.625 in and L/D of 24.9 and 17.3. These models were made from X-42 steel ERW pipe.

In addition, tests conducted by Kyriakides, et al (1987), Fowler, et al (1990) and Battelle (1983) for the longitudinal bending alone of the combined loading tests program were included in the database.

6.4.3 Uncertainty Model

Figure 6.4-4 illustrates the uncertainty analysis of the first RAM PIPE formulation based on the available test data. The median Bias and Coefficient of Variation of the Bias are 1.1 and 10.8%. Figure 6.4-5 illustrates the uncertainty analysis of the third RAM PIPE formulation. The median Bias and Coefficient of Variation of the Bias are 1.0 and 10.8%, respectively.

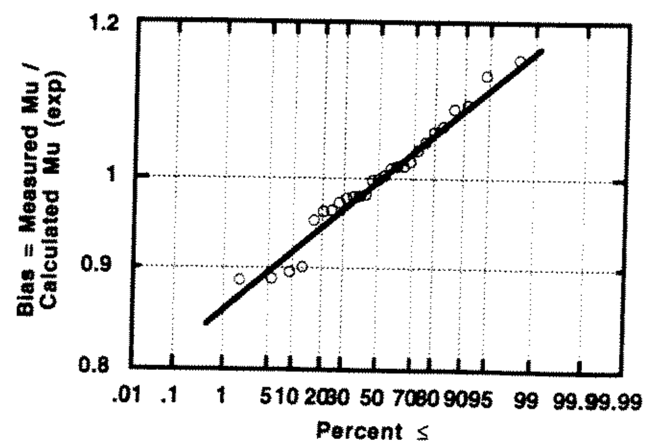

Figure 6.4-4 The Ratio Between the Measured Moment Capacity and the Predicted Moment Capacity based on First RAM PIPE Formulation

6.4.4 RAM PIPE REQUAL Bending Equation

The following formulation will be used to evaluate the transverse bending capacity of damaged – defective pipelines:

$$\frac{M_d}{M_u} = \exp\left(-0.06\frac{\Delta}{t}\right)$$

Only limited data was developed during Phase 1 to quantify the capacity bias and uncertainty of defective – damaged pipelines subjected to transverse bending.

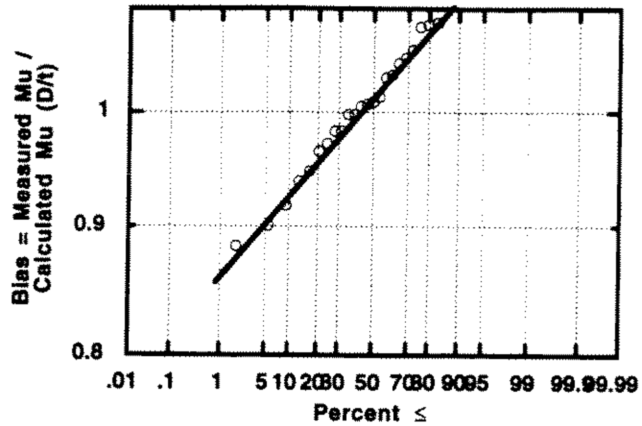

Figure 6.4-5 The Ratio Between the Measured Moment Capacity and the Predicted Moment Capacity Based on Third RAM PIPE Formulationj

In development of these criteria, a median Bias of B_{50} = 1.0 and a Coefficient of Variation of the Bias of V_B = 0.25 will be used. This high uncertainty is based on the variability of potential damage mechanisms such as corrosion, denting, and gouging in affecting the pipeline bending capacity. Test data needs to be developed to allow improved quantification of the analytical model Bias and Coefficient of Variation.

6.5 Pressure - Burst - Corroded

6.5.1 Review of Existing Criteria

The common criterion for assessment of corroded pipes, ANSI/ASME B31G (ASME 1991) was initially developed based on the NG-18 equation (Maxey, et al 1971). The NG-18 equation is defined as:

$$P = \frac{\sigma_0 \bullet 2 \bullet t_{nom}}{D_{nom}}\left[\frac{1-\dfrac{A_d}{A_0}}{1-\dfrac{A_d}{A_0}\bullet\dfrac{1}{M}}\right]$$

where, $A_0 = L \bullet t$

- M : Folias bulging factor, accounting for effect of stress concentration at notch
- P : failure pressure
- σ_0 : flow stress
- D : pipe outer diameter
- A_d : projected corroded area
- d : maximum corrosion length

In the ANSI/ASME B31G criterion the projected corrosion area is assumed to be parabolic, and hence the projected corroded area is $2d \bullet t/3$. However, for long defects this assumption will obviously overpredict the capacity and a rectangular shape is assumed. The flow stress is limited to 10% higher than the specified minimum yield stress (SMYS). The ANSI/ASME B31 G burst strength equation for Safe Maximum Pressure P′ is defined as:

$$P' = 1.1P\left[\frac{1-\frac{2}{3}\left(\frac{d}{t_{nom}}\right)}{1-\frac{2}{3}\left(\frac{d}{t_{nom} \bullet M}\right)}\right] \qquad ;P' \leq P\text{, }A \leq 4$$

$$P' = 1.1P\left(1-\frac{d}{t}\right) \qquad ;P' \leq P\text{, }A > 4$$

where

$$A = 0.893\left(\frac{L}{\sqrt{Dt}}\right)$$

$$P = \frac{SMYS \bullet 2t_{nom}}{D_0}F$$

$$M = \sqrt{A^2+1} = \text{Folias bulging factor}$$

and F is the design factor normally taken as equal to 0.72. P' may, however, not exceed P (maximum allowable design pressure for uncorroded pipe).

Modifications have been proposed to improve the NG18/B31G criterion in order to better predict the actual burst failure pressure. The modifications have, however, mainly been based on modification changes of the equation parameters. AN overview of some of the modifications to the flow stress, the bulging factor M and the procedure for estimating the projected corrosion area A is given below (Denys, 1995).

Flow Stress. Variations of proposed flow stress:

$$\sigma_0 = 1.1 \cdot SMYS$$
$$\sigma_0 = 1.15 \cdot SMYS$$
$$\sigma_0 = 0.5(\mathrm{SMYS} + \mathrm{SMTS})$$
$$\sigma_0 = \mathrm{SMYS} + 10\mathrm{ksi}$$
$$\sigma_0 = a \cdot SMTS$$

where a=0.90, 1.0 pr 1.1 and SMTS is the Specified Minimum Tensile Strength

Projected Corrosion Area. Variations of proposed projected corrosion area definition:

$A = d \cdot l$	(rectangular)
$A = 2d \cdot l/3$	(parabolic)
$A = 0.85d \cdot l$	(approx. average of retangular and parabolic)
A ="exact" calculation	

Bulging Factor. Variations of proposed definition of bulging factor:

- Maxey et al (1971)
  $$M = \sqrt{1+6.28 \cdot 10^{-1} \cdot X^2 - 3.38 \cdot 10^{-3} \cdot X^4}$$
- Kiefner (1974)
  $$M = \sqrt{1+0.8 \cdot X^2}$$

- Kiefner and Vieth (1989)

$$M = \begin{cases} \sqrt{1+6.28\cdot 10^{-1}\cdot X^2 - 3.38\cdot 10^{-3}\cdot X^4} & X \le \sqrt{50} \\ 3.3+3.2\cdot 10^{-3}\cdot X^2 & X > \sqrt{50} \end{cases}$$

The problems with modifying one or more parameters in the B31G criterion to obtain a better adaption to existing and newer results, is that it will most likely result in a negative effect for other design cases (geometries and corrosion configurations).

Bai, et al (1997) developed the improved B31G criterion based on the extensively calibration and analysis of the burst test data. The expression is:

$$P_b = \gamma P' = \frac{2\sigma_0 t_{nom}}{D}\frac{1-QA_d/A_0}{1-M^{-1}A_d/A_0}$$

where: $M = \sqrt{1+0.8\dfrac{L^2}{Dt_{nom}}}$

Q	Spiral Correction Factor
P'	Safe Maximum Pressure
γ	design factor

DNV 96 recommended that the burst capacity expression be:

$$P = P_0\cdot\frac{1-\dfrac{d'}{t_{nom}}}{1-\dfrac{d'}{t_{nom}}\cdot\dfrac{1}{M}}\cdot H$$

where P_0 is a normalising tensile strength hoop pressure:

$$P_0 = \frac{2\cdot t_{nom}}{D_0}\cdot\sigma_u$$

and

σ_u :	ultimate tensile strength
d' :	equivalent corrosion depth, accounting for the corrosion shape
t_{nom} :	pipe wall nominal thickness
D_0 :	nominal pipe diameter
M :	bulging factor accounting for the length and the projected shape of the corrosion
H :	adjustment factor accounting for the influence of combined loading, the yield sstress and tensile strength ratio. The adjustment for the ratio also dependent on the length and projected shape of the corrosion

RAM PIPE developed the burst equation for the corroded pipe as:

$$P_{bd} = \frac{3.2t_{nom}\cdot SMTS}{D_0\cdot SCF}$$

where, t_{nom} is the minimum wall thickness, and

$$SCF = 1 + 2\left(\frac{d}{R}\right)^{0.5}$$

The Stress Concentration Factor (SCF) due to the corrosion defects is a function of the maximum depth of corrosion d and the pipeline radius R. This is the SCF (maxmum hoop stress/nominal hoop stress) that is due to a notch of depth d in the pipeline cross section that has radius R.

The uncertainties affecting the determination of the acceptable annual maximum operating pressure for a corroded pipeline is typically uncertainty associated with the determination of :

- The extent of corrosion (depth, length and shape)
- The pipe material characteristics (actual yield stress and tensile strength)
- The pipe geometry (e.g. pipe thickness)
- The operation pressure (daily variations and efficiency of pressure control systems),
- The goodness of the applied capacity prediction models.

6.5.2 Review of the Test Data

AGA. The Pipeline Research Committee of the American Gas Association published a report on the research to reduce the excessive conservatism of the B31G criterion (Kiefner and Vieth 1989). 86 test data were included in the AGA test data. The first 47 were used to develop the B31G criterion, and were full scale tests conducted at Battle. The rest of the 86 tests were also full scale and were tests on pipe sections removed from service and containing real corrosion.

NOVA. Two series of burst tests of large diameter pipelines were conducted by NOVA during 1986 and 1988 to investigate the applicability of the B31G criterion to long longitudinal corrosion defects and long spiral corrosion defects. These pipes were made of grade 414 (X60) steel with an outside diameter of 160 in and a wall thickness of 2 inch. Longitudinal and spiral corrosion defects were simulated with machined grooves on the outside of the pipe.

The first series of tests, a total of 13 pipes, were burst. The simulated corrosion defects were 8 inch wide and 0.8 in deep producing a width to thickness ratio (W/t) of 4 and a depth to thickness ratio (d/t) of 0.4. Various lengths and orientations of the grooves were studied. Angles of 20, 30, 45 and 90 degrees from the circumferential direction, referred to as the spiral angle, were used. In some tests, two adjacent grooves were used to indicate interaction effects.

The second series of tests, a total of seven pipes, were burst. The defect geometries tested were longitudinal defects, circumferential defects, and corrosion patches of varying W/t and d/t. A corrosion patch refers to a region where the corrosion covers a relatively large area of pipe and the longitudinal and circumferential dimensions were comparable. In some of the pipes, two defects of different sizes were introduced and kept far enough apart to eliminate any interaction.
British Gas. Hopkins and Jones (1992) conducted five vessel burst tests and four pipe ring tests. The pipe diameter were 20 inch. The wall thickness was 4 inch. The pipe was made of X52. The defect depth was 40% of the wall thickness.

Jones et al (1992) also conducted nine pressurized ring tests. Seven of the nine were machined internally over 20% of the circumference, the reduced wall thickness simulating smooth corrosion. All specimens were cut from a single pipe of Grade API 5L X60 with the diameter of 299 in and wall thickness of 7 in.

Waterloo. 13 burst tests of pipes containing internal corrosion pits were reported by Chouchaoui et al (1992). In addition, Chouchaoui et al (1992a, 1992b) reported the 8 burst tests of pipes containing circumferentially aligned pits and the 8 burst tests of pipes containing longitudinally aligned pits.

6.5.3 Development of Uncertainty Model

Given the test data and design standards, the uncertainty analysis was conduct to evaluate the bias and uncertainties of the existing design equations. Figure 6.5.1 illustrates the uncertainty analysis results. Bias 1 is the results of the ASME/ANSI B31 G criteria. Bias 2 is the results of the NG-18 criteria. Bias 3 is the results of the modified ANSI/ASME B31 G criteria based on the effective area. Bias 4 is the results of the modified ANSI/ASME B31 G Criteria based on the 0.85dL area. The Bias 5 is the results of the Bai, Xu, Bea criteria. Note that the bias and COV of the existing ANSI/ASME B31 G criteria is 1.71 and 0.51. The Bias and COV of the Bia, Xu, and Bea (1996) equation is 1.07 and 0.18.

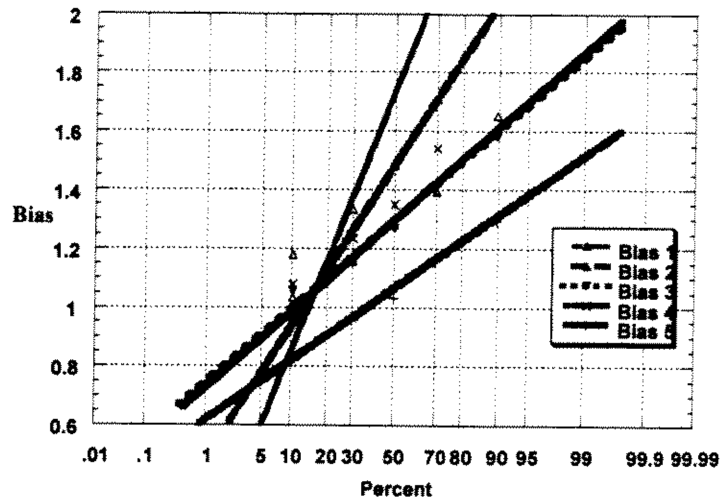

Figure 6.5.1 - Analysis of corroded pipe burst pressure Bias

Figure 6.5.2 summarizes results from analysis of the RAM PIPE REQUAL database on the burst pressures associated with corroded pipelines. This database is summarized in Appendix A. The formulation used to assess the burst capacities was:

$$Pbd = 2\,t\,Su / D\,SCF$$

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

$$Pbd = 3.2\,t\,SMYS / D\,SCF$$

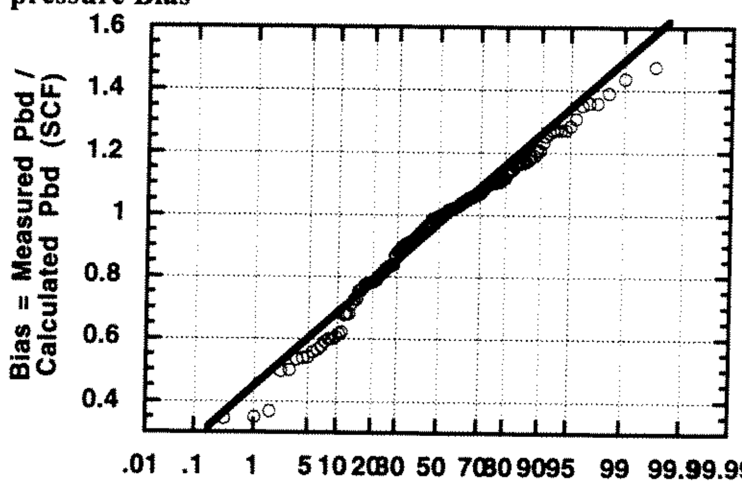

Figure 6.5.2 – Analysis of corroded pipe burst pressure Bias based on RAM PIPE formulation

where Pbd is the burst pressure capacity of the corroded pipeline, t is the nominal wall thickness (including the corrosion allowance), Su is the ultimate tensile strength (transverse) of the pipeline steel, D is the pipeline diameter, SMYS is the specified minimum yield strength, and SCF is a stress concentration factor that is due to the corrosion effects. The SCF is a function of the depth of corrosion, d ($d \leq t$), and the pipeline radius, R. This is the SCF (maximum hoop stress / nominal hoop stress) that is due to a notch of depth d in the pipeline cross section that has a radius R.

This formulation has a median bias of unity, $B_{50} = 1.0$, and a coefficient of variation of the bias of V_B = 24.1 %.

This test data provided information to assess the pipeline specimen tensile yield and ultimate strengths. The results are summarized in Figures 6.5.3, 6.5.4, and 6.5.5. Figure 6.5.3 summarizes the statistical analysis of the ratio of the measured yield strength to the specified minimum yield strength. The data indicates a median ratio of 1.2 and a Coefficient of Variation of 10.7 %.

Figure 6.5.4 summarizes the statistical analysis of the ratio of the measured ultimate tensile strength to the specified minimum yield strength. The data indicates a median ratio of 1.6 and a Coefficient of Variation of 7.3 %.

Figure 6.5.5 summarizes the statistical analysis of the ratio of the measured ultimate tensile strength to the specified minimum tensile strength. The data indicates a median ratio of 1.2 and a Coefficient of Variation of 5.9 %.

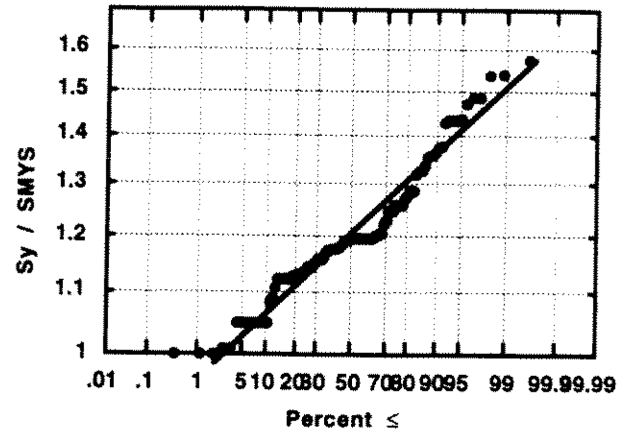

Figure 6.5.3 – Analysis of ratio of measured yield strengths to SMYS

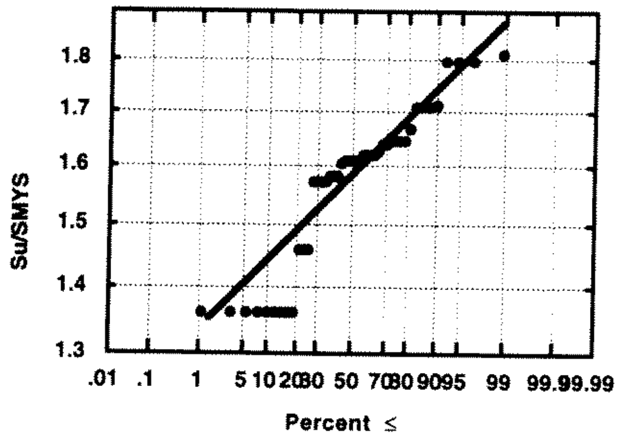

Figure 6.5.4– Analysis of ratio of measured ultimate tensile strength to SMYS

The RAM PIPE database test data was analyzed to determine the biases and uncertainties associated with the formulation based on the specified minimum tensile strengths (SMTS):

$$Pbd = 2\ t\ SMTS / D\ SCF$$

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

Analysis of the test data (151 tests, Appendix A) based on the SMTS (Figure 6.5.6) indicated a median Bias of 1.2 and a Coefficient of Variationof the Bias of 21 %. Analysis of the test data based on 1.2 times the SMTS (Figure 6.5.6) indicated a median bias of 1.0 and a Coefficient of Variation of the Bias of 22 %.

The design formulation for burst pressure for corroded pipelines will be based on the formulation developed in this Section, the SMTS, and the criteria developed based on a median Bias of 1.2 and a Coefficient of Variation of the Bias of 23 %. Given that the SMTS is defined at minus three standard deviations, and the coefficient of variation of the Suts is about 10 %, the median bias of 1.2 for the design equation referenced to SMTS is reasonable. This contention is further justified by analysis of the test data summarized in Figure 6.5.5.

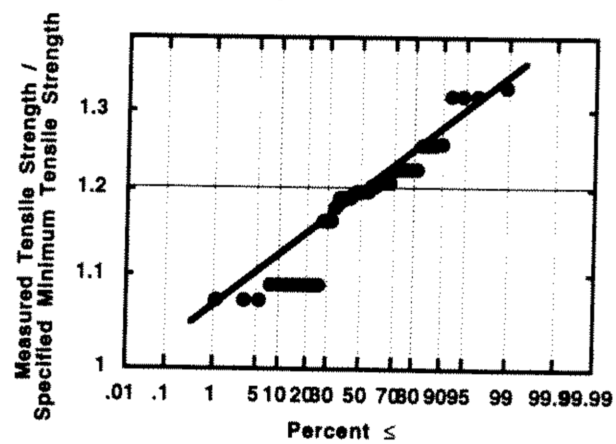

Figure 6.5.5 – Analysis of ratio of measured ultimate tensile strength to SMTS

Figure 6.5.7 summarizes the test data summarized in Appendix A in the form of the measured burst pressure for the corroded condition divided by the burst pressure for no corrosion (Pbm / Pbo) versus the corrosion depth divided by the nominal wall thickness of the pipeline specimens (d/t). There is a large scatter in the Pbm / Pbo for larger values of d/t. Note that even at values of d/t ≥ 0.7, the pipeline can sustain as much as 80 % of the uncorroded burst pressure (and as low as 40 %).

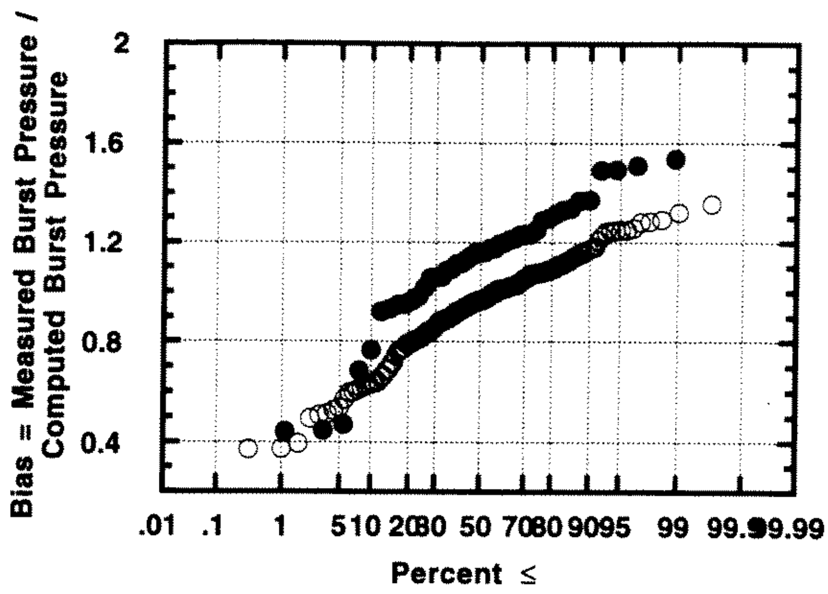

Figure 6.5.6 – Bias in design formulations based on SMTS

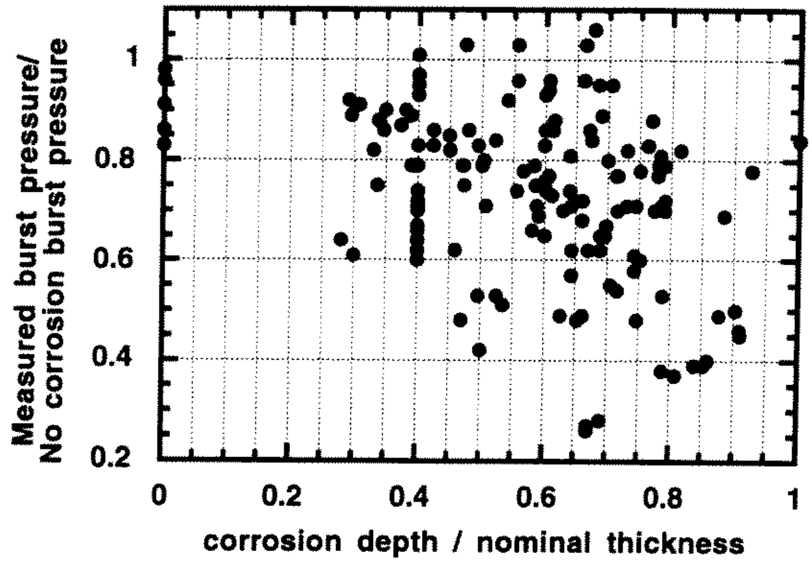

Figure 6.5.7 – Variation in burst pressure ratio with d/t

6.6 Pressure - Burst - Dented Pipes

Denting is a damage state in that it represents plastic deformation of a pipeline. Denting typically results from highly localized reaction loads that can occur during installation or as a result of outside "third-party" induced loads during service such as dropped objects. When the local stresses induced by such loads exceed yield strength, a dent results, permanently deform the pipe.

The limit states associated with denting are clearly those associated with failure of the pipeline and those that affect the serviceability, such as those that would stop a pipeline rig. No systematic studies have been carried out on serviceability limit states for denting.

A number of research projects in the past 30 years have addressed the strength of pipelines containing various types of defects, including dents. The following discussions summarizes the research related to denting limit states to develop the RAM PIPE reassessment equations.

6.6.1 Background

Three general types of defects have been investigated to evaluate their impact on pipeline integrity: stress concentrations, plain dents, and combination of the two. This classification was developed according to the behavior of the defects under internal pressure.

Stress concentrations consists of V-notches, weld cracks, stress-corrosion cracks, and gouges in pipe that have not been dented. The distingulishing characteristic of this type is that they reduce the pipe wall thickness without changing the curvature of the pipe wall. Considerable research has been conducted into the behavior of this type of defect, and formations are available to predict the severity of reduction in normal burst strength of pipe caused by the presence of this type of defect. Consideration of the damage and limit states due to this type of defect, because they are not associated with dents, was beyond the scope of this section. The reader is directed toward next sections for a detailed discussion and the corresponding RAM PIPE reassessment equations.

Plain dents are distinguished by a change in curvature of the pipe wall without any reduction in the pipe wall thickness. The behavior of this type of defect has been extensively investigated.

The third defect type represents a combination of the first two and is a dent with a stress concentration. This type of defect represents the mechanical damage that is one of the leading causes of leaks and failures in gas distribution and transmissing piping. Research into the behavior of this type of flaw has indicated that it is the most serious of the three types. Recent research has led to the development of the RAM PIPE equations to predict the severity of mechanical damage defects.

Plain Dent

Research on the severity and behavior of plain dents in line pipe is summarized in a Pipeline Research Committee report (Eiber, et al 1981). The report reviews and discusses results of conducting capped end pipe burst tests over a range of temperatures (-24 to 90F) on 44 dents of the configurations shown in Figure 15. The majority of these were placed in the body of the pipe away from the longitudinal weld. In all of these tests involving dents remote from the longitudinal weld, the pipes failed at their ultimate strengths and the fractures were at points remote from the dent locations. No effect of temperature was noted in the experimental results.

The results of strain gage readings taken during pressurization of the dents have indicated that the principal effect of plain dents is to introduce high localized longitudinal and circumferential bending stresses in the pipe wall. These add to the nominal pressure stresses at some locations in the dent and subtract from them at other locations. Thus, yielding occurs at local locations in the dent at much lower pressure levels than it occurs in the remainder of the pipe body. Without a sharp stress cocnentrations, however, the yielding occurs over large enough areas that the pipe has sufficeint ductility to yield and accept the plastic flow without failure.

It was found that when dents occur near or on the longitudinal weld, failures can result at low pressures because of cracks that develop in or adjacent to the welds. The cracks probably develop because of the stress concentration associated with weld, and because the various weld zone may exhibit less ductility than the base metal. It was concluded that dents that disturb the curvature of the longitudinal and/or circumferential weld are potentially hazardous defects as they may contain cracks.

The final conclusion of the report is that plain dents whose depth is up to 8 percent of the pipe diameter did not decrease the burst pressure of the section of line pipe. Hence plain dents without stress concentrations represent damage in that the pipe has been plastically deformed but they do not represent an ultimate limit state. This conclusion has also been substantiated by the related work of Belonos and Ryan (1958) on continuous dents.

Gouge-in-Dent

More recent research has focused on assessing the severity of mechanical damage defects and the effect of fracture toughness in resisting the failure of these defect types. The conclusion of this research is that fracture toughness does play a singificant role in the failure pressure of mechanical damage (gouge-in-dent) defects.

Nearly all research on this subject has been conducted at two laboratories; Battelle, and British Gas Corporation research facilities, Newcastle-Upon-Tyne, Great Britain. British Gas conducted most of their research using ring cut from pipe, damaged, and then tested on a ring yield-test-machine, using the same type of ring tester used by pipe mills to determine pipe yield strength. British Gas also conducted tests on pipe that was damaged while not pressurized and later on pipe that was pressurized and then damaged. Battelle's efforts involved testing both pipes that were damaged and then pressurized to failure and pipes damaged while pressurized. It is believed that gouge length is a strong influencing parameter, and it is not possible to use gouge length as a variable in the ring test method.

The research is handicapped by the large number of variables that affect the failure pressure of the damaged pipe. Variables examined to date are gouge depth, gouge length, dent depth, pipe size, pipe thoughness (as indicated by Charpy Upper Shelf Energy), and pipe yield strength.

Other variables that are known to affect dent and gouge severity and consequently pipe failur epressure are coatings and the shape and size of the gouging agent. Both hard and soft coatings reduce the amount of gouging but appear to do so in different ways. The softer coatings appear to act as lubricants, allowing the tool to glide over the pipe surface, causing more indentation and less gouging. Tougher coatings are more easily gouged, but in turn protect the pipe underneath, which is gouged less. If the tool is sharp and small, it will gouge more (dent less) than if it is wide and blunt. Real damage done in service is random. Damage can be positioned at any orientation relative to the pipe. Damage also varies depending on the type of equipment being used and the persistence of the equipment operator. It would be impractical to investigate all these variables independently. The research doen on this subject did not examine many variables independently, nor all of the variables, such as tool variation, construction equipment variation, defect orientation, and coating types (generally no coating was tested).

Figure 6.6.1 represents the initial research results for the burst stress for pipe, which was notched and dented without internal pressure, representative of damage that may occur during handling and installation of a pipeline. The data are presented in terms of normalized failure stress and a Q parameter, developed by Battelle (Stephenes, et al 1991) to illustrate the synergistic effects of material toughness (CVN), gouge depth (d), gouge length (2c), and dent depth (D) on failure stress of mechanically damaged pipe. The failure stress is normalized by the flow stress (approximated for line pipe ssteels as $\sigma_y + 10ksi$). The data noted as "dented-then-notched pipes" is that developed by British Gas in which gouges were machnied into the bottom of an already indented pipe. The data noted as "notched-then-dented pipes" were developed by Battelle by machining a V-shaped notch in the pipe wall, and then pressing a round bar, laid over the notch, into the pipe to form a dent.

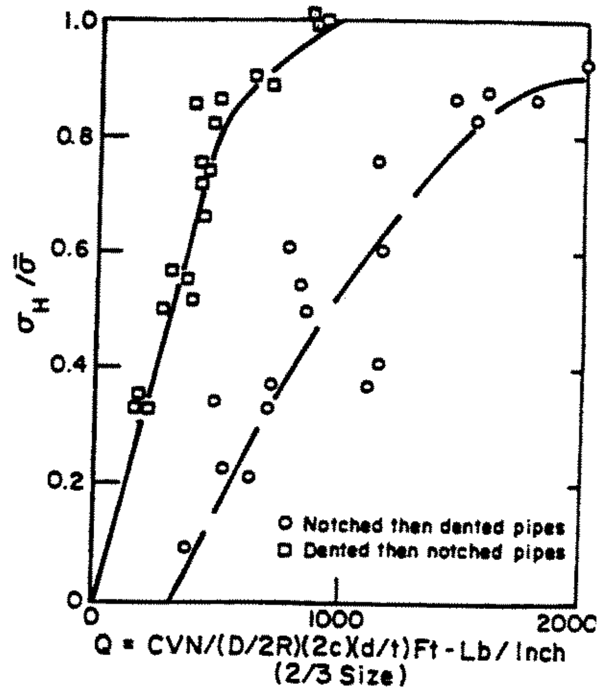

Figure 6.6.1 – Burst stresses for notched and dented pipe without pressure during the damages

More recent research on mechanical damage was aimed at damage doe to pressurized pipe to simulate in-service failures from mechanical damage. A special purpose mechanical damage machine was constructed to reproduce the create desired defect geometries. The device for imparting damage to the pipe specimen is a blunt tool piece that is loaded by two hydraulic cylinders, one horizontal and one vertical. The damaging process occurs dynamically, at a rate that can be controlledd between 6 and 48 in./sec. The specimens were completely water-filled and pressurized to the desired level with water.

Figure 6.62 presents the results of experiments where mechanical damage flaws failed dduring the time that damage was being done. Comparison of Figures 6.6.1 and 6.6.2 shows that the same Q parameterization is also appropriate for pressurized samples. The limit state curve for notched-tehn-dented specimens provides a conservative approximation for the nominal hoop stress at ultimate failure of a pipeline containing a gouge-in-dent.

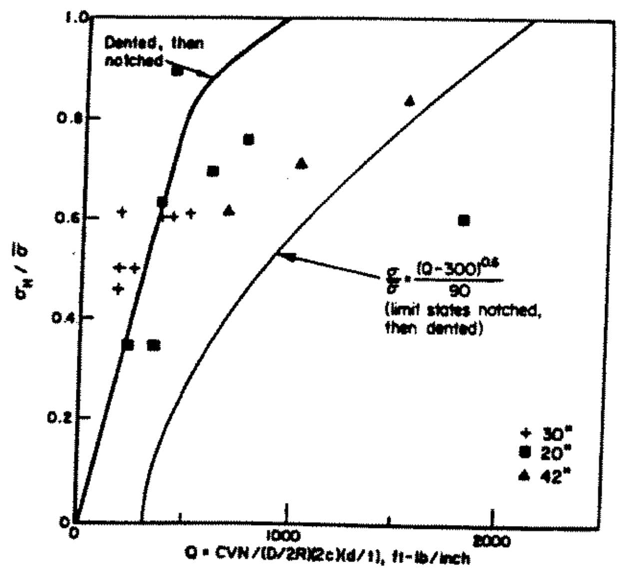

Figure 6.6.2 – Burst Pressure for dented and notched specimens with pressure during the damages

It was observed that if a defect is not a severe enough to cause dynamic failure, the indentation remaining has been partially removed as a result of the internal pressure. If a defect such as this were found in the field, the only observable and measurable dent depth would be that of the partially removed dent. Measurements of remaining dent depth were recorded on defects that did not fail during the damage process. These data were used to calculate the ratio of remaining dent to maximum dynamic dent depth as plotted in Figure 6.6.3. An empirical curve has been fitted to the experimental data and an equation for this curve is shown in Figure 6.6.3.

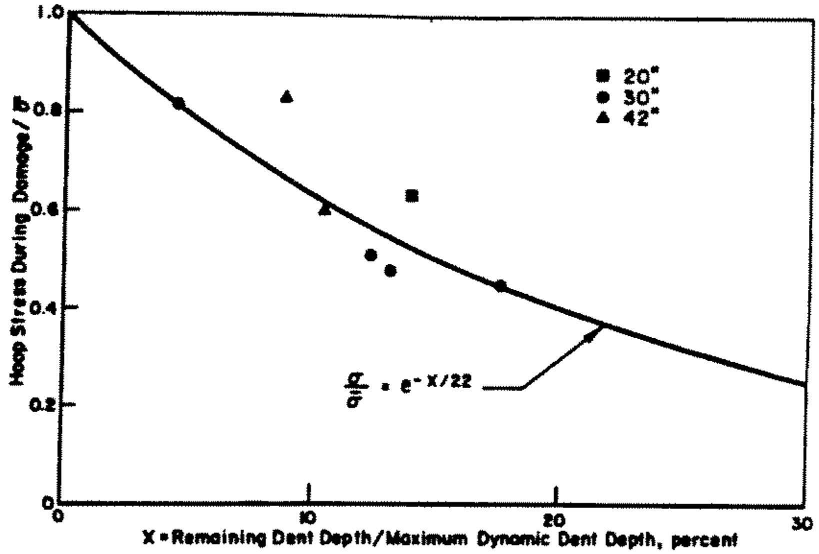

Figure 6.6.3 – Dent removal by pressurization of pipeline

6.6.2 Summary of Test Data

Appendix B contains a database on the dented and gouged pipeline tests assembled during the RAM PIPE REQUAL project. This database is organized by the sequence of denting and gouging and type of test performed. Tests 1 – 41, 42 – 99, and DTZ 1R, 2R are ring (short sections of pipelines) tests performed by the British Gas Corporation (Jones, 1990-1991). Tests 99-117 and DTZ 1P, 2P are tests performed on prototype sections of pipelines by the British Gas Corporation (Jones 1990-1991).

Study of this test data indicates the following primary conclusions:

- Plain denting with smooth shoulders has no significant effect on burst pressures. Smooth shoulderd denting is not accompanied by macro or microcracking and the dent is re-formed under increasing internal pressures.
- Denting with sharp shoulders can cause macro and micro cracking which can have some effects on burst pressures and on fatigue life (if there are significant sources of cyclic pressures – straining (Hopkins, 1990). The degree of macro and micro cracking will be a function of the depth of gouging. Generally, given pressure formed gouging, there will be distortion of the metal and cracking below the primary gouge that is about one half of the depth of the primary gouge.
- Gouging can cause macro and micro cracking in addition to the visible gouging and these can have significant effects on burst pressures (Hopkins, 1990). In laboratory tests, frequently gouging has been simulated by cutting grooves in the pipe (Jones, 1990-1991). These grooves can be expected to have less macro and micro cracking beneath the test gouge feature.
- The combination of gouging and denting can have very significant effects on burst pressures and on fatigue life.
- The effects of combined gouging and denting is very dependent on the history of how the gouging and denting have been developed. Different combinations have been used in developing laboratory data. In some cases, the pipe is gouged, dented, and pressured to failure. In other cases, the pipe is dented and gouged simultaneously, and then pressured to failure. In a few cases, the pipe is gouged, pressured, and then dented until the pipeline looses containment. These different histories of denting and gouging have important effects on the propagation of macro

and micro cracks developed during the gouging and denting. It will be very difficult for a single formulation to be able to adequately address all of the possible combinations of histories and types of gouging and denting.

- Gouging is normally accompanied by denting a pipeline under pressure. If the pipeline does not loose containment, the reassessment issue is one of determining what the reliability of the pipeline segment is given the observed denting and gouging (does it need to be repaired or the pipeline pressures reduced?). Addressing this issue requires an understanding of how the pipeline would be expected to perform under increasing pressure demands (loss of containment due to pressure) or under continuing cyclic strains (introduced by external or internal sources). In the case of loss of containment due to pressure, the dent is re-formed under the increasing pressure and the gouge is propagated during the re-forming. Cracks developed on the shoulders of the dents can also be expected to propagate during the re-forming.

6.6.3 Stress Concentration Factors Due to Denting

Some information is available on the Stress Concentration Factors (SCF) that are associated with sharp shouldered denting (Fowler, 1993). The theoretical aspects of dent and gouge associated SCF have been studied by Svoboda and Gajdos (1994). This information indicates for longitudinal un-reformed dents the SCF is a function of the location around the circumference of the pipeline, the diameter to thickness ratio (D/t), and the depth of the dent relative to the pipe thickness or pipe diameter (H/t, H/D).

Fowler's results are summarized in Figure 6.6.4 for the maximum SCF (located at the pipe crown – the point of maximum denting). Fowler's results are based on the results of extensive nonlinear finite element analysis (FEA) studies. The study by Svoboda and Gajdos indicates that the SCF for denting is:

$$SCF = 1 + 6\ H/t$$

where H is the depth of the dent and t is the pipeline wall thickness. This relationship indicates SCF's that are much greater than those developed by Fowler. Given the extensive nonlinear behavior found in the FEA studies, it is not unexpected that a linear elastic based SCF could be expected to over-predict the SCF's.

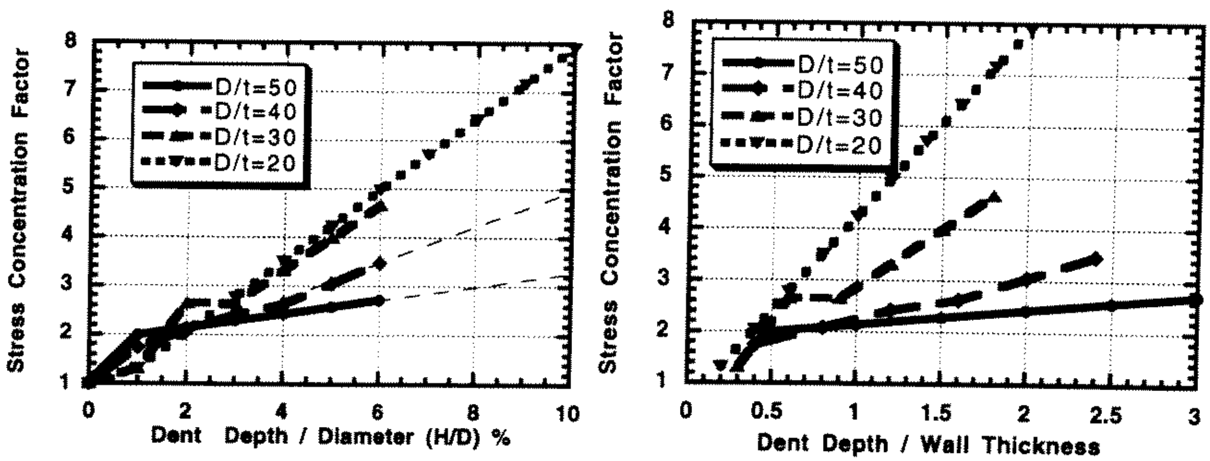

Figure 6.6.4 – Un-reformed longitudinal dent SCF's at pipe crown (location of maximum denting)

As a result of study of the test data and results from analyses of flawed pipelines (Fowler, 1991; Miller, 1988) the following relationships were developed to evaluate the SCF's associated with plain longitudinal denting without gouging (e.g. due to dropped objects) and the effects of the SCF on burst pressures.

Smooth denting and transverse denting ($H/R \leq 25\%$)

$$SCF_D = 1$$

Sharp longitudinal denting

$$SCF_D = (1 - H/R)^{-1}$$

$$SCF_D = 1 + 2\pi\,(H/R)^{0.5}$$

For dents associated with gouging (e.g. due to anchor dragging, jet sled damage) the following SCF was developed.

$$SCF_D = 1 + 0.2\,(H/t)3$$

6.6.3 Stress Concentration Factors Due to Gouging

Extensive information is available on the Stress Concentration Factors (SCF) that are associated with gouging – cracking (Miller, 1988). All those reviewed during this project were based on elastic analysis methods. Figure 6.6.5 summarizes results from several of the analyses that were identified as being most appropriate for the denting – gouging SCF effects on pipeline burst pressure capacities. The two SCF relationships indentified as MisPSdeep and MisPS shallow are those based on a Mises Plain Stress analysis of tensile induced cracking in which as the crack is developed bending stresses are introduced (Miller,1988).

The SCF relationship identified as 1/1-d/t is based on a plane stress Tresca and Mises, and plane strain Tresca analysis of cracking for pure tension (no bending stresses as cracking is developed). It is interesting to note that the crack root radius and flank angle (angle that describes the inclination of the sides of the crack) have not effect on this SCF. The SCF identified as edge shallow and deep is based on a tension loaded edge crack that has a root radius equal to the plate thickness. The deep relationship recognizes the increased tensile stress as the crack propagates through the plate thickness.

Based on an analysis of the applicability of these different analytical models, therelationship adopted in this study to define the SCF associated with plain gouging was:

$$SCF = (1 - de/t)^{-1}$$

This relationship does not take account of the interactions with denting – either during the denting proces or during the dent re-forming under pressure process. To recognize these interactive effects, two contributions to the gouge – crack depth were identified:

- gouge – crack depth associated with denting = de
- propagation of gouge –crack depth associated with reforming the dent under pressure = dt

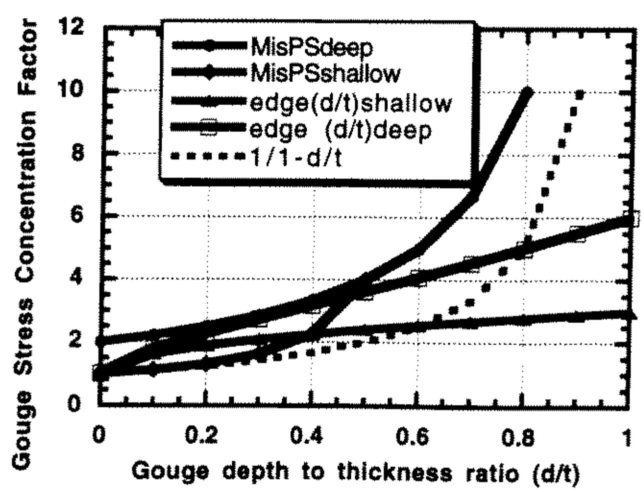

Figure 6.6.5 – Stress Concentration Factors due to gouging

$$d = de + dt$$

The effective gouge depth is:

$$de = Kd - dg$$

where K is the factor to recognize the mechanism used to introduce the gouging effect in extending the depth of the gouge below the gouge root. dg is introduced to enable recognition of the effect of grinding the gouge (a repair method).

The results provided by Hopkins (1990) in a study of the depth of cracking associated with the denting – gouging process when the pipeline is under pressure are summarized in Figure 6.6.6. The data on maximum depth of cracking (determined using MPI) normalized by the pipeline wall thickness, a/t, associated with a gouge depth normalized by the pipeline wall thickness, d/t, is summarized in Figure 6.6.6. It is apparent that there can be significant cracking developed below the root of the gouge during the denting – gouging process. K could range from 1.5 to 2.0.

If the gouge was introduced by a mechanical sawing or grinding process, K could be expected to be near unity. This indicates that the test data must be carefully regarded concerning how the gouge is introduced into the test specimens.

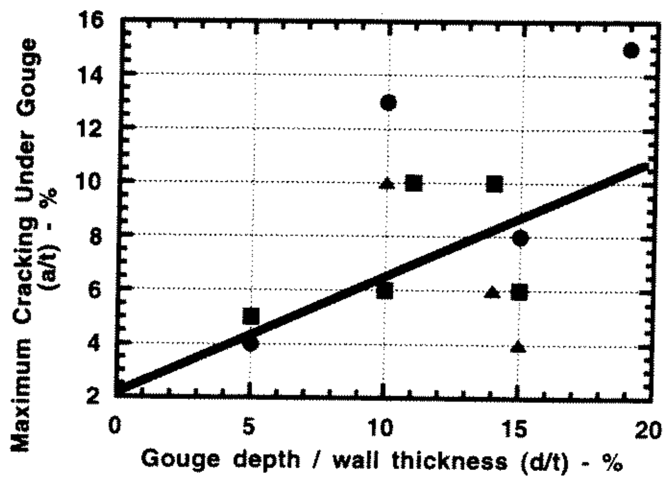

Figure 6.6.6 – Cracking developed below the gouge when the pipeline is dented and gouged under pressure

Propagation of the gouge under pressure reformation of the dent is difficult to determine. Initially, fracture mechanics was used to estimate how much the combined gouge – dent associated cracking might be propagated during the reformation of the dent (Barsom, Rolfe, 1987; Almar-Naess, 1985). The propagating stresses for the Type I stress conditions (tensile prying) developed stress intensity factors that generally fell in Region III or the rapidly propagating cracking region. These results indicated that the reforming cracking was most likely a ductile tearing mechanism rather than a cyclic strain – progressive cracking mechanism (Region II – moderate Stress Intensity Factor region) (Barson, Rolfe, 1987).

Given this insight, the crack propagation was related to the bending strains introduced during the pressure reformation process. The analysis indicated that the reformation bending strains could be expected to be a function of 16 times the dent depth to pipe radius (H/D):

$$\varepsilon_{rf} = 16\ H/D$$

These strains were evaluated to be responsible for the additional ductile tearing of the remaining gouged – dented cross section during pressure reforming:

$$dt = (16\ H/R)\ (t\text{-}de)$$

These studies resulted in two methods to evaluate the SCF associated with gouging and denting. The first method was based on separate SCF's for the gouging and the dent reformation propagation:

$$SCF_G = (1 - d/t)^{-1}$$

$$SCF_D = 1 + 0.2\ (H/t)^3$$

$$SCF_{DG} = SCF_G \times SCF_D$$

The second method was based on a single SCF that incorporated the gouge formation and propagation:

$$SCF_{DG} = \{[1 - (d/t) - [16\ H/D(1-d/t)]\}^{-1}$$

6.6.2 Analysis of test data

Figure 6.6.4 summarizes results from analysis of tests performed by British Gas Corporation (Jones, 1981) on 30-inch diameter X52 pipelines (Table 6.6.1). The pipelines were first dented. The dent depths (H) to diameter ratios were in the range H/D = 1.0 % to 3.6 %.

A defect simulating a gouge was machined into the center of the dent parallel to the axis of the pipeline. The defects had depths (h) to wall thickness ratios that were h/t = 25%.

The analyses were based on the measured ultimate tensile strengths of the pipeline steel specimens.

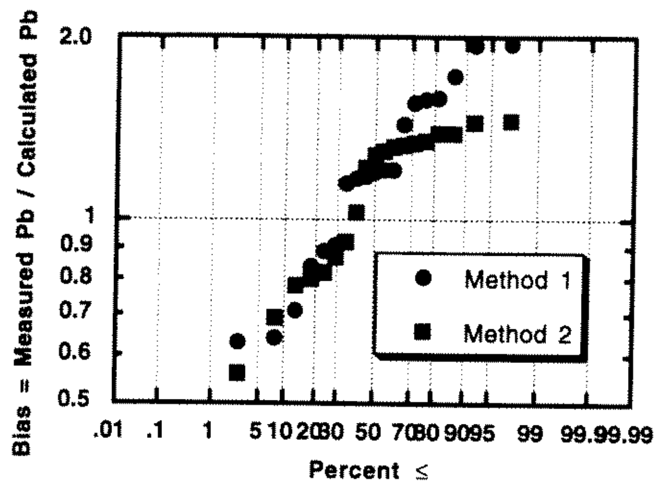

Figure 6.6.4 – British Gas Tests on 30-inch diameter pipelines with dents and gouges

The Method 1 reassessment formulation used to analyze these data is based on stress concentration factors for the dent of:

$$SCF_H = 1 + 0.2(H/t)^3$$

The reassessment formulation used to analyze these data is based on stress concentration factors for the gouges of:

$$SCF_h = (1 - d/t)^{-1}$$

Table 6.6.1 – Pipeline dent – gouge test data and analytical results

D	t	Sy	Su	H	H/D	H/t	h	h/t	Smc	Pbm	Pbc	SCF	SCF	SCF	SCF	Pbc	B	SCF	Pbc	B
in	in	ksi	ksi	in			in		ksi	ksi	ND	c	H	h	t	DD		t	DD	
30	0.469	56.1	83.9	0.404	0.013	0.861	0.122	0.260	64.50	2.017	2.665	1.32	1.128	1.352	1.524	1.748	1.15	1.723	1.55	1.30
30	0.469	56.1	83.9	0.388	0.013	0.827	0.122	0.260	66.35	2.075	2.665	1.28	1.113	1.352	1.505	1.771	1.17	1.704	1.56	1.33
30	0.469	56.1	83.9	0.398	0.013	0.849	0.125	0.267	65.80	2.057	2.665	1.30	1.122	1.363	1.530	1.742	1.18	1.731	1.54	1.34
30	0.469	56.1	83.9	0.758	0.025	1.616	0.129	0.275	53.40	1.670	2.665	1.60	1.844	1.379	2.544	1.047	1.59	2.315	1.15	1.45
30	0.469	56.1	83.9	0.783	0.026	1.670	0.129	0.275	50.00	1.563	2.665	1.70	1.931	1.379	2.663	1.001	1.56	2.368	1.13	1.39
30	0.469	56.1	83.9	0.827	0.028	1.763	0.123	0.262	47.30	1.479	2.665	1.80	2.097	1.355	2.842	0.938	1.58	2.425	1.10	1.35
30	0.469	56.1	83.9	0.894	0.030	1.906	0.130	0.277	44.80	1.401	2.665	1.90	2.385	1.383	3.300	0.808	1.73	2.644	1.01	1.39
30	0.469	56.1	83.9	1.090	0.036	2.324	0.128	0.273	34.30	1.072	2.665	2.48	3.511	1.375	4.828	0.552	1.94	3.285	0.81	1.32
30	0.469	56.1	83.9	1.197	0.040	2.552	0.123	0.262	20.90	0.653	2.665	4.08	4.325	1.355	5.863	0.455	1.44	3.749	0.71	0.92
30	0.469	56.1	83.9	0.570	0.019	1.215	0.117	0.249	56.80	1.776	2.665	1.50	1.359	1.332	1.811	1.472	1.21	1.914	1.39	1.28
30	0.469	56.1	83.9	0.640	0.021	1.365	0.117	0.249	51.50	1.610	2.665	1.65	1.508	1.332	2.010	1.326	1.21	2.023	1.32	1.22
30	0.469	56.1	83.9	0.970	0.032	2.068	0.117	0.249	45.00	1.407	2.665	1.89	2.769	1.332	3.690	0.722	1.95	2.760	0.97	1.46
30	0.469	56.1	83.9	0.410	0.014	0.874	0.117	0.249	40.10	1.254	2.665	2.13	1.134	1.332	1.510	1.764	0.71	1.705	1.56	0.80
30	0.469	56.1	83.9	0.780	0.026	1.663	0.117	0.249	20.90	0.653	2.665	4.08	1.920	1.332	2.558	1.042	0.63	2.281	1.17	0.56
30	0.469	56.1	83.9	0.300	0.010	0.640	0.117	0.249	55.20	1.726	2.665	1.54	1.052	1.332	1.402	1.901	0.91	1.586	1.68	1.03
30	0.469	56.1	83.9	0.670	0.022	1.429	0.117	0.249	35.90	1.122	2.665	2.37	1.583	1.332	2.109	1.263	0.89	2.073	1.29	0.87
30	0.469	56.1	83.9	0.520	0.017	1.109	0.117	0.249	32.00	1.001	2.665	2.66	1.273	1.332	1.696	1.572	0.64	1.844	1.45	0.69
30	0.469	56.1	83.9	0.740	0.025	1.578	0.117	0.249	30.20	0.944	2.665	2.82	1.786	1.332	2.379	1.120	0.84	2.201	1.21	0.78
30	0.469	56.1	83.9	1.080	0.036	2.303	0.117	0.249	22.30	0.697	2.665	3.82	3.442	1.332	4.586	0.581	1.20	3.142	0.85	0.82

The resultant stress concentration factor is:

$$SCF_{H,h} = SCF_H \times SCF_h$$

The Method 2 was based on the combined denting – gouging SCF:

$$SCF_{DG} = \{[1 - (d/t) - [16\ H/D(1-d/t)]\}^{-1}$$

The reassessment damaged pipeline burst capacity was based on:

$$Pbd = (2\ Suts / SCF_{H,h})\ (t / D)$$

Results of the analyses indicate Method 1 has a median Bias of B_{50} = 1.2 and a Coefficient of Variation of the Bias of 33 %. Method 2 has a median Bias of B_{50} = 1.3 and a Coefficient of Variation of the Bias of 26 %. The bias was removed by introducing the specified minimum tensile stress of the pipeline steels in the reassessment formulation:

$$Pbd = (2\ SMTS / SCF_{H,h})\ (t / D)$$

Table 6.6.2 summarizes test data on dented and gouged pipelines performed by Det Norske Veritas Oberg, Rengard, Wiik, 1982). The first three tests involved only denting; the second three tests denting and gouging; and the last four tests denting, gouging, and griding of the gouges (gouge depth and 0.2 to 1.5 mm beyond the gouge depth). These tests were published and analyzed by Song and Bai (1998). Song and Bai developed a modification of the Maxey - Kiefner – Battelle approach for determining the effects of longitudinal flaws on burst pressure capacities (Kiefner, Maxey, Eiber, Duffy, 1973). The modification utilized a Bilby-Cottrell-Swiden Dislocation Model, a modification for pipe steel toughness, and a Newman – Raju Stress Intensity Factor. The Song-Bai model did not account for the effects of denting. The test data indicate that longitudinal denting alone can have important effects on burst strength. Two identical pipeline specimens when dented, can develop very different burst capacities.

The combination of denting and gouging results in even more significant reductions in the burst strength. Grinding the gouges improves the burst strength, however, there is still a significant effect of the gouging even though the gouges are removed by grinding and by over-grinding in an attempt to remove the cracks under the root of the gouges.

Table 6.6.2 includes an evaluation of the pipeline burst capacities based on the formulations developed during the RAM PIPE REQUAL project. The formulation for the dented pipeline results in a reasonably unbiased results (mean bias B = 0.95). The formulation for the dented and gouged pipelines results in similarly unbiased resutls (mean bias B = 1.0). However, the dented – gouged – ground results are much more scattered. The analyses summarized in Table 6.6.2 were based on a crack depth below the gouge equal to K = 1.5 times the gouge depth. It is apparent that this resulted in an under-estimate of the crack depths below the gouge roots in three of the five cases. Additional work is needed to be able to model cracking beneath the root of the gouges and on the propagation of cracking that accompanies re-forming the dents in gouged-dented pipelines.

One of the parts of the fracture mechanics based study of crack behavior during the denting, gouging, and re-forming processes was the evaluation of the pressures required to re-form the dents. The plastic analysis formulation resulted in the following analytical model to predict the pressures required to re-form the dents (P_F), and the stresses associated with the reforming:

$$P_F = [2\ t / R\ (1 + 4\ H/t)]\ \sigma_y$$

$$\sigma_F / \sigma_y = 2 / (1 + 4\ H/t)$$

Figure 6.6.5 summarizes the results from this analytical model compared with three of the pipe reformation pressure – deformations provided by Oberg, et al, (1982).

There is excellent agreement between the plastic analysis based formulation and the test data. Very little pressure – stress is required to reform dents in pipelines. The pipeline can be returned to a dent depth equal to twice the wall thickness with about 20 % of the yield stress or pressure required to cause yielding in the pipeline.

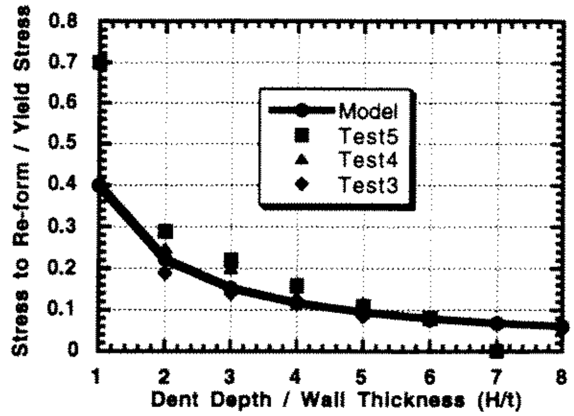

Figure 6.6.5 – Evaluation of dent re-forming stresses compared with experimental results

Table 6.6.2 – DNV dented – gouged pipeline tests

D mm	t mm	t/D	D/t	Suts	H/D	h/t	Pbm	PbND	SCF	SCF	SCF	SCF	Pb	Bias
Dented				Mpa			Mpa	Mpa	c	H	d	t	DD	
273	10	0.0366	27.32	630	0	0.000	46.0	46.12	1.00	1.000	1.00	1.00	46.1	1.00
273	10	0.0366	27.32	630	0.2	0.000	30.0	46.12	1.54	1.667	1.00	1.67	27.7	1.08
273	10	0.0366	27.32	630	0.2	0.000	21.2	46.12	2.18	1.667	1.00	1.67	27.7	0.77
D/gouged														
814	20	0.0246	40.70	675	0.18	0.125	4.5	33.17	7.37	1.563	1.23	8.16	4.1	1.11
814	20	0.0246	40.70	675	0.12	0.125	7.4	33.17	4.48	1.316	1.23	4.01	8.3	0.90
814	20	0.0246	40.70	675	0.05	0.125	16.2	33.17	2.05	1.111	1.23	2.23	14.9	1.09
D/G/grd														
814	20	0.0213	47.06	675	0.18	0.063	7.4	28.69	3.88	1.563	1.10	4.62	6.2	1.19
814	20	0.0206	48.54	675	0.18	0.028	27.0	27.81	1.03	1.563	1.04	3.72	7.5	3.61
814	20	0.0203	49.31	675	0.12	0.023	7.7	27.38	3.56	1.316	1.03	2.48	11.0	0.70
814	20	0.0203	49.33	675	0.12	0.013	6.8	27.36	4.02	1.316	1.02	2.39	11.4	0.59
814	20	0.0197	50.88	675	0.12	0.000	6.0	26.54	4.42	1.316	1.00	2.29	11.6	0.52

6.7 Pressure – Collapse – Pcd

6.7.1 Pressure – Collapse – corrosion

The following formulation will be used to reassess the pressure collapse capacities of pipelines that have high ovalities ($f_{50} = 1.0$ %):

$$P_c = 0.5\left\{P'_{ud} + P_{ed}K_d - \left[\left(P'_{ud} + P_{ed}K_d\right)^2 - 4P'_{ud}P_{ed}K_d\right]^{0.5}\right\}$$

The following formulation will be used to reassess the pressure collapse capacities of pipelines that have low ovalities ($f_{50} = 0.1$ %):

$$P_c = 0.5\left\{P_{ud} + P_{ed}K_d - \left[\left(P_{ud} + P_{ed}K_d\right)^2 - 4P_{ud}P_{ed}K_d\right]^{0.5}\right\}$$

where,

$$P_u' = 5.1\frac{\sigma_u t_d}{D_0}$$

$$P_E = \frac{2E}{1-v^2}\left(\frac{t_d}{D_0}\right)^3$$

$$K_d = 1+3f\left(\frac{D_0}{t_{nom}}\right)$$

$$P_{ud} = \frac{2SMTSt_{\min}}{D_0}$$

$$f = (D_{max} - D_{min}) / (D_{max} + D_{min})$$

No data is available to quantify the capacity bias and uncertainty of corrosion damaged pipelines subjected to collapse pressures

In development of these criteria, a median Bias of B_{50} = 1.0 and a Coefficient of Variation of the Bias of V_B = 0.25 will be used. This high uncertainty is based on the variability of potential damage due to corrosion in affecting the pipeline collapse capacity. Test data needs to be developed to allow improved quantification of the analytical model Bias and Coefficient of Variation.

6.7.2 Pressure – Collapse – denting

Kyriakidies and Yeh (1985) developed test data on the burst pipelines that was intentionally 'dented' resulting in large ovalization of the pipeline cross section. A summary of their results is given in Figure 6.7.1. There is a linear decrease in the collapse pressure with the pipeline diameter to thickness ratio, and the rate and amount of decrease is a function of the ovality of the pipeline.

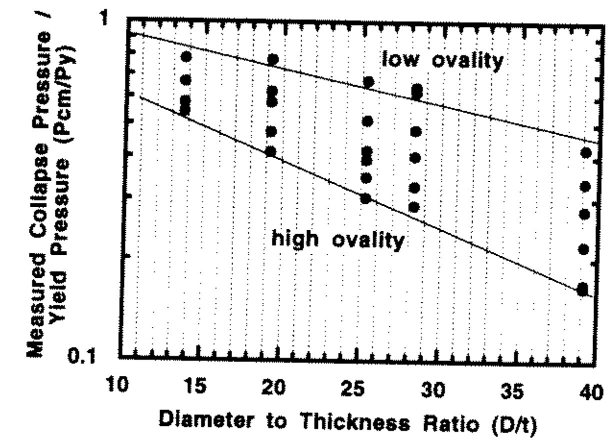

Figure 6.7.1 – Effect of ovality / denting on pipeline collapse pressures

Analysis of this test data to determine the bias and uncertainty in the Timoshenko yield and ultimate (2 hinge model) is summarized in Figure 6.7.2. The median Bias associated with the Timoshenko yield formulation is B_{50} = 1.2. The Coefficient of Variation of the Bias is V_B = 23 %. The median Bias associated with the Timoshenko ultimate (2-hinge) formulation is B_{50} = 0.9. The Coefficient of Variation of the Bias is V_B = 20 %.

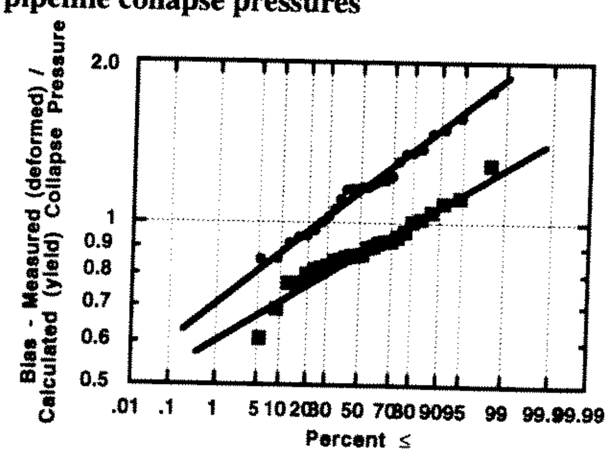

Figure 6.7.2 – Bias in computed collapse pressures of dented pipelines based on Timoshenko Elastic and Ultimate formulations

Kyriakides and Yeh (1985) also provided test data on different diameter to thickness ratios for varying lengths of the denting – ovalization relative to the pipe diameter. The length / diameter ratios ranged from 1 to 12. Figure 6.7.3 summarizes the bias and uncertainty for pipe having a diameter to thickness ratio of D/t = 13.45. The median Bias based on the Timoshenko yield and ultimate formulations is B_{50} = 1.5 and 1.0, respectively. The Coefficient of Variation of the Bias based on the Timoshenko yield and ultimate formulations is V_B = 16 % for both formulations.

Figure 6.7.4 summarizes the bias and uncertainty for pipe having a diameter to thickness ratio of D/t = 25.3. The median Bias based on the Timoshenko yield and ultimate formulations is B_{50} = 1.3 and 1.0, respectively. The Coefficient of Variation of the Bias based on the Timoshenko yield and ultimate formulations is V_B = 23 % and V_B = 20 %, respectively.

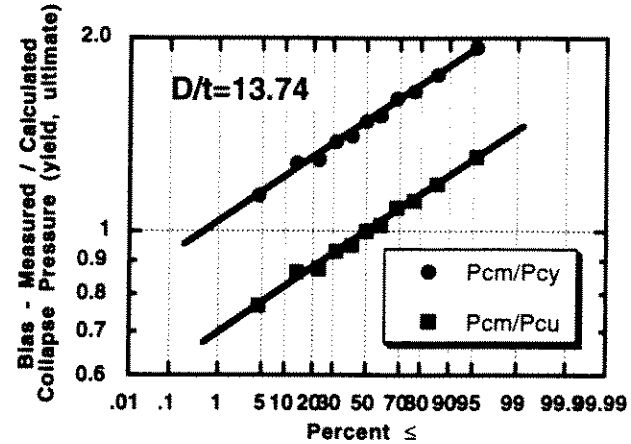

Figure 6.7.3 – Bias in computed collapse pressures based on the Timoshenko yield and ultimate formulations for D/t = 13.74 pipe

6.8 Reassessment Pressure – Propagating – Pbd (accidental limit state)

Available data on only steel pipes (Shell Pipeline, 1974) having sizes approximating those of prototype pipelines were assembled and the Bias determined. The results are summarized in Figure 6.8.1. The median Bias is indicated to be B_{50} = 1.05 and the coefficient of variation of the Bias is indicated to be V_B = 8.8 %.

In these developments, the Mesloh, et al (1976) formulation will be used as the reference design and reassessment analysis model. The experimental results indicate that this model has a median Bias B_{50} = 1.05 and a coefficient of variation of the Bias V_B = 8 %.

The experimental Bias characteristics could be validated by considering the basic pipe characteristics:

$$B_{50} = B_{Syn} (B_t / B_D)^{2.5}$$

$$B_{50} = 1.1 (1.0 / 1.0)^{2.5} = 1.1$$

$$V^2_B = V^2_{Sy} + (2.5 V_{t/D})^2$$

$$V_B = 0.05^2 + (2.5 \text{ x } 0.02)^2 = 0.07$$

These results are very close to those based on the experimental results.

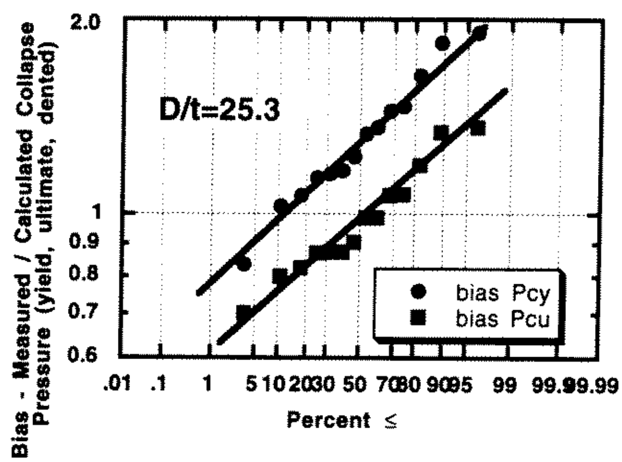

Figure 6.7.4 – Bias in computed collapse pressures based on the Timoshenko yield and

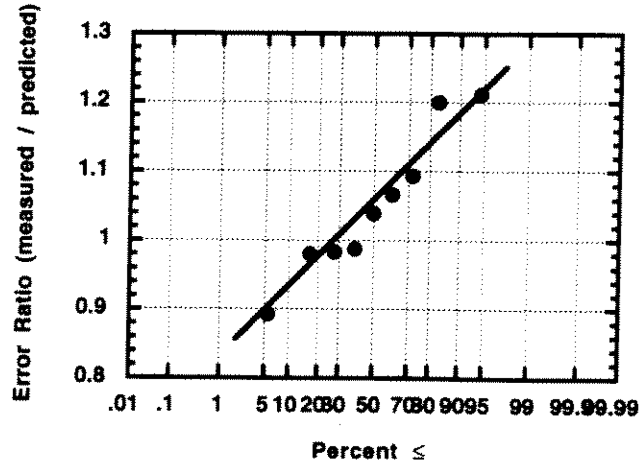

Figure 6.8.1 – Bias in predicted propagation pressures based on Mesloh, et al (1976) and Shell Pipeline (1974) test data

The design and reassessment formulation will be expressed as:

$$Pp / Sym = B_{Sy}\, 34\, (t/D)^{2.5}$$

where Sym is the specified minimum yield strength, and B_{Sy} is the median Bias introduced by using the specified minimum yield strength and the Mesloh, et al formulation. Based on the API specification based data cited earlier (B_{Sy} = 1.1 x 1.05 = 1.16), the design and reassessment analysis propagating pressure formula becomes:

$$Pp / Sym = 39\, (t/D)^{2.5}$$

Figure 6.8.2 summarizes results from 43 tests of model tubes (SS-304) that defines the measured and calculated propagation pressures as a function of the diameter to thickness ratio. There is a linear relationship between the propagation pressure and the diameter to thickness ratio. There is good agreement between the calculated (based on the design formulation) and measured propagation pressures.

Figure 6.8.3 summarizes the statistical analysis of the bias in the calculated propagation pressures. The median bias is B_{50} = 1.09 and the Coefficient of Variation of the Bias is V_B = 35%.

Figure 6.8.4 summarizes results from 12 tests of small scale (4-inch diameter) pipelines fabricated from X-42 and X-65 steel that defines the measured and calculated propagation pressures as a function of the diameter to thickness ratio. There is a linear relationship between the propagation pressure and the diameter to thickness ratio. There is good agreement between the calculated (based on the design formulation) and measured propagation pressures.

Figure 6.8.5 summarizes the statistical analysis of the bias in the calculated propagation pressures. The median bias is B_{50} = 0.92 and the Coefficient of Variation of the Bias is V_B = 11.5 %.

The design and reassessment formulation was expressed as:

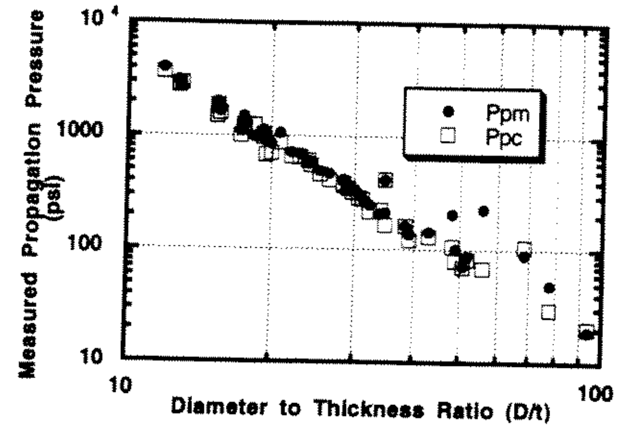

Figure 6.8.2 – Measured & calculated propagation pressures (model tubes)

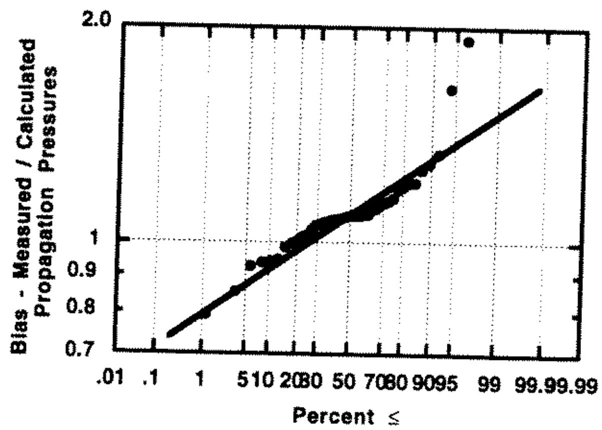

Figure 6.8.3 – Bias in measured to calculated propagation pressures for model tubes

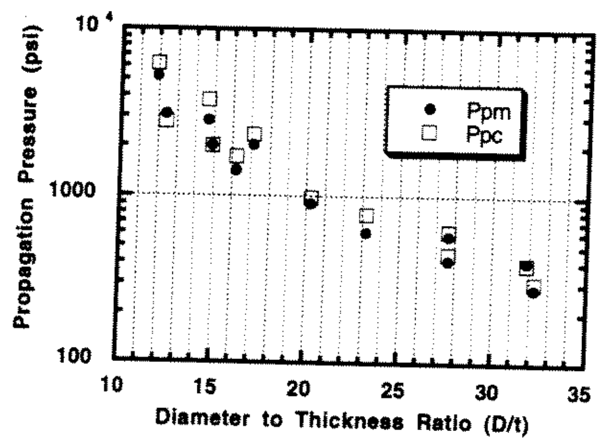

Figure 6.8.4 – Measured and calculated propagation pressures for X42 and X65 piplines

$$Pp / Sym = B_{Sy}\, 34\, (t/D)^{2.5}$$

where Sym is the specified minimum yield strength, and B_{Sy} is the median Bias introduced by using the specified minimum yield strength and the Mesloh, et al formulation. Based on the API specification based data cited earlier (B_{Sy} = 1.1 x 0.92 = 1.01), the design and reassessment analysis propagating pressure formula becomes:

$$Pp / Sym = 34 (t/D)^{2.5}$$

The median Bias associated with this formulation is $B_{50} = 1.0$ and the coefficient of variation of this bias is $_{B} = 12\%$

Figure 6.8.6 summarizes the test data on the effects of concrete weight coating on the collapse and propagating pressures as a function of the thickness of the weight coating to the steel thickness (Langner 1974,;Mesloh et al 1976). The pipelines tested had diameter to thickness ratios in the range of 51 to 111.

The concrete coating has the effect of increasing both the initiating or collapse pressure and the propagating pressure by substantial amounts. For a thickness ratio of 10, both the collapse and propagating pressures are increased by a factor of 2. As the thickness of the concrete coating relative to the pipeline wall thickness increases, there is a continued increase in the initiating and propagating pressures. The increase in the propagating pressures could be expressed as:

$$Rpc = (tc/ts) / 5$$

where Rpc is the ratio of the propagating pressures with the concrete cover to the propagating pressures without the concrete cover, tc is the thickness of the concrete cover, and ts is the thickness of the pipeline steel.

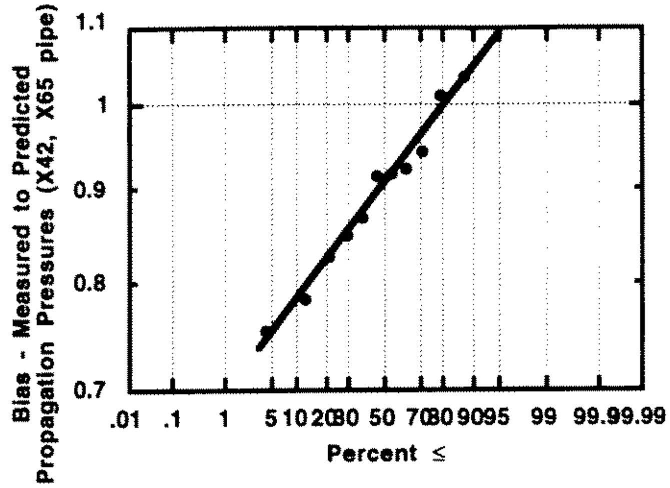

Figure 6.8.5 – Bias measured / calculated propagation pressures for X42 and X65 pipelines

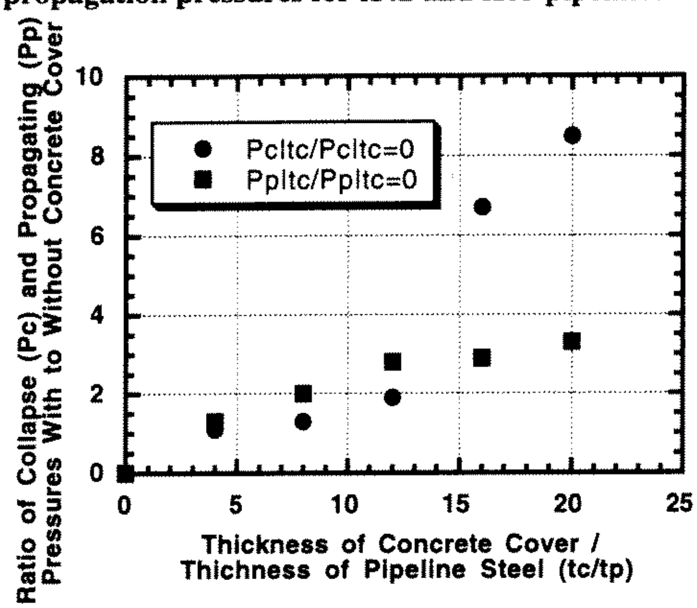

Figure 6.8.6 – Effect of concrete cover on collapse pressures and propagating pressures

7.0 Operating Pressure Conditions

To develop pipeline requalification criteria and guidelines, the variabilities, uncertainties, and biases associated with the operating conditions must be evaluated and quantified. For requalification of pipelines, this will involve two conditions:

1) normal operating and,

2) accidental.

7.1 External Collapse Pressures

The external collapse pressure, Po, is comprised of two components:

$$Po = Ph + Ps$$

Ph is the hydrostatic pressure associated with the maximum normal water depth, and Ps is the additional pressure associated with storm conditions (expected maximum wave amplitude and storm water depth). Based on a first order, second moment approximation (or the algebra of Normal functions), for the small range of uncertainties in these variables, the median effective pressure can be expressed as:

$$Po_{50} = Ph_{50} + Ps_{50}$$

The Standard Deviation of the effective external collapse pressure can be expressed as:

$$\sigma^2_{Po} = \sigma^2_{lPh} + \sigma^2_{lPs}$$

where σ_{lPh} is the Standard Deviation in the hydrostatic pressure and $\sigma_{lPs}$is the Standard Deviation in the storm associated pressures.

The characterization of the external pressure will be developed for two conditions: accidental (10-year storm), and extreme operating (100-year hurricanes). Both conditions probability characterizations will be developed for a water depth of 150 feet.

7.1.1 Accidental In-Place Operating Condtions - Collapse

The normal conditions probability characterization will be based on a median sea water density of γ = 63.5 pounds per cubic foot (pcf) with a Coefficient of Variation of $V\gamma$= 1.5%. The normal conditions effective pressure should be associated with the mean maximum water depth and storm surge which for this example would be 153.6 feet.

The accidental conditions median hydrostatic pressure on the pipeline at the seafloor will be:

$$P_{hy50} = (\ 63.5 \text{ pcf }) (153.6 \text{ ft}) = 9754 \text{ pounds per square foot (psf)}$$

The hydrodynamic pressures at the sea floor (Δp)will be based on:

$$\Delta p = \ \gamma_w (v\ H_{10yr}) / \cosh kd$$

where k is the wave number (2 π / L), v is the percentage of the wave height that is above the mean water depth, γw is the sea water density (63.5 pcf), d is the water depth (153.6 ft), and L is the wave length. It should be noted that this is the pressure that is developed on a non-deformable sea floor. A deformable sea floor will develop smaller pressures due to the motions of the sea floor. The median Bias associated with the non-deformable sea floor condition will be taken to be unity. This evaluation is based on the premise that this conservatism will be compensated by errors associated with determination of the water depth and pipeline elevation.

The 10-year return period wave height will be taken to be H_{10yr} = 26 feet with a period of 12 seconds (wave height to length ratio of H / L = 1/25). The wave length will be taken to be L = 650 feet. The wave number is k = 0.01 ft^{-1} and kd = 1.48. The crest height to wave height ratio will be taken to v = 0.7. The crest height above still water will be 0.7 x 26 feet = 18.2 feet.

The 10-year return period differential bottom pressure can be computed to be:

$$\Delta p = 63.5 \text{ pcf} \times 18.2 \text{ ft} / \cosh(1.48) = 500 \text{ psf}$$

The 10-year return period accidental design pressure would be 500 psf + 9754 psf = 10254 psf.

The uncertainty associated with the annual expected maximum wave heights is $V_H = \sigma_{lnH} = 0.4$. The 10-year wave height associated hydrodynamic bottom pressures will have a Bias (median Δp / 10 year Δp) of:

$$B_{\Delta p 10\,yr} = \exp\text{-}(1.28\ \sigma_H) = \exp\text{-}(1.28 \times 0.4) = 0.60$$

The median annual maximum differential bottom pressure can be computed to be 500 psf x 0.6 = 300 psf. The median resultant pressure at the seafloor will be 9754 psf + 300 psf = 10054 psf. The Bias in the 10-year accidental pressure is thus B_{P10yr} = 10054 / 10254 = 0.98.

The uncertainty in the sea floor resultant pressure expressed as the standard deviation in the annual maximum sea floor pressure can be computed from the quadrature of the standard deviations in the hydrostatic and hydrodynamic pressures. The standard deviation in the hydrostatic pressure is σ = 0.015 x 9754 psf = 146 psf. The standard deviation in the hydrodynamic pressure is σ = 0.4 x 300 psf = 120 psf. The resultant standard deviation in the pressure at the sea floor will be the result of the quadrature of these two pressures:

$$\sigma = [\ 1462 \text{ psf} + 1202 \text{ psf}\ ]^{0.5} = 189 \text{ psf}$$

The resultant Coefficient of Variation of the sea floor pressures will be:

$$V_P = 189 \text{ psf} / 10054 \text{ psf} = 2\ \% = \sigma_{lnP}$$

The Bias and Coefficient of Variation appropriate for accidental in-place conditions based on a return period of 10-years will depend on the internal pressure condition that controls the pipeline performance:

$$Pcp = Po - Pi$$

where Po is the combined hydrostatic and hydrodynamic pressure, and Pi is the pipeline internal pressure.

In the case of oil pipelines, the minimum accidental internal pressure would be 0.9 x hydrostatic (static head of fluid contents, 0.9 Specific Gravity of fluid contents). The net pressure to collapse the pipeline would be 0.1 times the hydrostatic pressure plus the hydrodynamic pressure: 975 psf + 500 psf = 1475 psf. The median net collapsing pressure would be: 975 psf + 300 psf = 1275 psf. The accidental conditions for the oil pipelines would have a Bias of 0.86. The resultant Coefficient of Variation of the collapse pressures would be 10 %.

As the pipeline external pressures becomes dominated by hydrostatic pressure (deeper water), the foregoing Bias will approach unity and the Coefficient of Variation will become smaller (approach 2 %). Similarly, as the pipeline external pressures become dominated by hydrodynamic pressure (shallow water), the Bias will approach 0.60 and the Coefficient of Variation will approach 40 %.

In the case of gas pipelines, the minimum internal accidental pressure could be atmospheric (compression or well-head pressure shut off and pressure at other end of pipeline allowed to go to atmospheric). The net pressure to collapse the gas pipeline would be the hydrostatic pressure plus the hydrodynamic pressure. This combined pressure would have a Bias of 0.98 and a Coefficient of Variation of 2 %.

Design criteria for oil and gas pipelines and pipelines carrying mixtures of oil and gas will be developed for accidental conditions based on a Bias of 0.98 and Coefficient of Variation of the collapse pressure of 2 % (this produces the most conservative design and requalification factors).

It is specified as a basis of these criteria that the 'normal' minimum operating pressure will not be less than the hydrostatic pressure. This minimum pressure is important in determining the collapse pressure performance characteristics of the pipeline during extreme conditions.

Given repair operations on the pipelines, it is specified that the internal pressures in the pipeline will not be lower than hydrostatic. Thus, the external and internal pressures will be in equilibrium during repair operations.

Given inspection operations on the pipelines, it is specified that the normal minimum operating pressures will not be compromised (pigging operations will be conducted with the specified minimum operating pressure in the pipeline).

This analysis indicates that propagating buckling is of primary concern for accidental installation conditions (for pipelines installed without pressure or at atmospheric pressure). This is because it is during this time that there are the maximum collapse pressures. An accident is needed to cause local buckling of the pipeline, which when the pipeline is lowered to the sea floor, the local buckle can be propagated by the external hydrostatic pressures. Design of the installation procedures, design of the pipeline to withstand the expected installation stresses, and the installation Quality Assurance and Quality Control (QA/QC) measures will normally preclude this concern.

Given that propagating buckling could be possible (very deep water, installation conditions with a high likelihood of inducing buckles in the pipeline), then rather than making the entire pipeline able to resist such buckling, it would be preferable to use buckle arrestors. Design guidelines and criteria for such arrestors have been developed and will be summarized later in this report. Given the use of buckle arrestors, the propagating buckling would be contained between two of the arrestors. The spacing of the arrestors would be based on economics considerations (cost of arrestors versus cost of repairs). Given a propagating buckle in the pipeline, the pipeline installation would be interrupted and repairs made before the pipeline installation were completed.

Given the foregoing developments concerning the minimum internal pressures to be maintained in the pipelines during in-place operations, propagating buckling does not appear to be a governing condition for in-place design criteria.

7.1.2 Operating In-Place Extreme Conditions

The extreme (design 100-year hurricane) conditions probability characterization will be based on a median sea water density of $\gamma = 63.5$ pcf with a Coefficient of Variation of $V\gamma = 1.5\%$ and a water depth that is the sum of the mean maximum water depth (152.5 feet) and a median storm tide of 3.5 feet: total of 156 feet. There would be a hydrostatic pressure of 9906 psf.

The 100-year (annual 99 %tile) expected maximum wave height will be taken as 46 feet. The expected wave height associated with the deformable sea floor conditions has been estimated to be 0.8 times the wave height for a non-deformable sea floor. The expected wave height to wave length ratio will be taken to be 1:16. Thus, the wave length will be 736 feet and the wave number will be $k = 0.0085\ ft^{-1}$ and $kd = 1.33$.

A wave amplitude of 37 feet will be used in this development (based on crest elevation to wave height ratio of $\nu = 0.8$). This would result in a sea floor hydrodynamic pressure of:

$$\Delta p = (63.5 \text{ pcf}) (37 \text{ ft}) / \cosh (1.33) = 1162 \text{ psf}$$

The 100-year hurricane conditions hydrostatic pressure would be 9906 psf. This pressure would have an uncertainty of $V_{Pe} = 1.5\ \% = \sigma_{lnPe}$ ($\sigma = 148.6$ psf). The 100-year design total pressure would be 9900 psf + 1162 psf = 11068 psf.

The uncertainty associated with the annual expected maximum wave heights is $V_H = \sigma_{lnH} = 0.4$. The 100-year wave height associated hydrodynamic bottom pressures will have a Bias (median Δp / 100 year Δp) of:

$$B_{\Delta p\ 100yr} = \exp\text{-}(2.33\ \sigma_H) = \exp\text{-}(2.33 \times 0.4) = 0.4$$

The median extreme conditions hydrodynamic pressure would be 0.4 x 1162 psf = 465 psf. The median extreme conditions external pressure would be 9906 psf + 465 psf = 10371 psf. The 100-year design pressure would thus have a Bias of B_{P100yr} = 10371 psf / 11068 psf = 0.94.

If the median extreme conditions pressures were determined by the hydrodynamic pressures, the median Bias would be = 0.4. The hydrostatic component would have a standard deviation of σ = 148.6 psf. The wave height hydrodynamic component would have a standard deviation of σ = 0.4 x 1162 psf = 465 psf (the long-term distribution of expected annual maximum wave heights has a Coefficient of Variation of approximately 40 %). Combining the two components in quadrature would give a resulting standard deviation of σ = 488 psf. This indicates a resulting Coefficient of Variation in the extreme conditions pressures of V_Pe = 488 psf / 10371 psf = 4.7 % = σ_{lnPe}.

For the extreme 100-year hurricane design external pressure conditions, the Bias will be $B_{Pe50} = 0.61$ and the uncertainty equal to $\sigma_{lnPe} = 11\ \%$. This Bias and uncertainty is based on the combination of hydrodynamic and hydrostatic external pressures. If the effective pressure were based only on the external hydrodynamic pressure (because the internal pressure was limited to a minimum of hydrostatic), then the Bias will be $B_{Pe50} = 0.40$, and the uncertainty equal to $\sigma_{lnPe} = 40\ \%$.

The Bias and Coefficient of Variation appropriate for design in-place conditions based on a return period of 100-years will depend on the internal pressure condition that controls the pipeline performance during extreme conditions. In the case of oil pipelines, the minimum shut-in internal pressure would be 0.9 x hydrostatic (static head of fluid contents, 0.9 Specific Gravity of oil contents). The net pressure to collapse the pipeline would be 0.1 x hydrostatic pressure plus the hydrodynamic pressure: 990 psf + 1160 psf = 2150 psf. The median net collapsing pressure would be: 990 psf + 465 psf = 1455 psf. The design conditions for the oil pipelines would have a median Bias of 0.68. The resultant Coefficient of Variation of the collapse pressures would be 32 %.

In the case of gas pipelines, the minimum internal extreme conditions pressure is generally controlled to be greater than the hydrostatic pressure (compression or well-head pressure shut off and pressure and valves at other end shut in afterward). Generally, operating personnel report that the minimum internal pressure in the gas pipelines is more than 70 % of the normal operating pressures (IMP, 1998). In a worst case operating condition (not accidental), the net pressure to collapse the gas pipeline would be the hydrodynamic pressure. This pressure based on 100-year conditions would have a median Bias of 0.40 and a Coefficient of Variation of 40 %.

7.2 In-Place Operating Internal Burst Pressures

The effective internal burst pressure, Pbp, is comprised of two components:

$$Pbp = Pi - Po$$

Po is the external pressure (hydrostatic + hydrodynamic + geostatic, if buried), and Pi is the internal pressure.

During operating extreme conditions (100-year return period design), with the pipelines shut-in, the internal pressure generally will be equal to or greater than about 70 % of the normal operating pressures (IMP, 1998). The study performed by IMP (1998) indicates that the mean operating pressures of the pipelines are 61 % of the maximum design pressures. These operating pressures have a Coefficient of Variation of 34 %. Table 7.1 lists minimum, normal, and maximum operating pressures of a sample of Bay of Campeche pipelines.

Table 7.1 – Operating pressures in Campeche Bay Pipelines

Line No.	Service	Diameter (inches)	Wall Thickness (inches)	Temperature °F	Operating Pressures (psi)		
					Minimum	Normal	Maximum
93.14	Gas	8	0.312	68	782	1000	1100
93.06	Gas	8	0.312	68	782	1000	1100
93	Gas	20	0.500	68	782	1000	1100
93.04	Gas	8	0.312	68	782	1000	1100
127	Gas	8	0.375	68	782	1000	1100
24	Oil	36	0.625	147	227	470	500
77	Gas	36	0.875	130		924	1000
36	Gas	36	0.750	130		965	1000
27	Gas	36	0.750	68	800	1000	1200
107	Gas	24	0.562	170	85	100	
138	Gas	8	0.500	130	782	1000	1100
137	Gas	8	0.500	130	782	1000	1100
93.13	Gas	8	0.312	68	782	1000	1100
41	Oil	48	0.625	86	240	360	600
64	Oil	36	0.875	140	560	640	711
109	Oil-Gas	20	0.625	176/208/230	176	208	230
112	Oil-Gas	20	0.500	176/208/230	170	425	1000
90	Oil-Gas	36	0.750	176/208/230	170	425	1000
68	Oil	36	0.750	140	560	640	711
71	Gas	36	0.875	130/145		924	1000
66	Oil	36	0.750	140	5	25	711
25	Oil	24	0.688	165	426	500	570
75	Oil	36	0.875	140	560	640	711
1	Oil	36	0.625	140	560	640	711

The IMP (1998) study indicates that the average ratio of maximum design pressures to hydrostatic pressures for near future pipelines in the Bay of Campeche is 15.5. The minimum ratio of maximum design pressure to hydrostatic pressure is 2.2. The Coefficient of Variation of the ratio maximum design pressures to hydrostatic pressures is 61 %. Given that these pipelines will be operated at pressures that are similar to those of the existing pipelines in the Bay of Campeche, then the pipelines generally are operating at pressures that are about 10 times the external hydrostatic pressures, and only in very rare instances would a pipeline be operating at a pressure that is approximately equal to the external hydrostatic pressure.

The IMP (1998) study indicates that the average ratio of maximum design pressures to hydrostatic pressures for existing pipelines in the Bay of Campeche is in the range of 15 to 16. The variations of the MOP are reported by PEMEX operating personnel to be approximately 10 %. The MDP is not exceeded in any case (IMP, 1998). Figure 7.1 summarizes the maximum operating pressures that were recorded on a 24-inch diameter gas pipeline (pipeline no. 030) during a one month period. The mean maximum operating pressure is 912 psi. The Coefficient of Variation of the maximum operating pressure is 8 %. The ratio of the mean maximum operating pressure to the maximum design pressure is 80 %.

Load uncertainty characterization is an important part of the limit state functions used for all the failure modes considered. During the RAM PIPE REQUAL project, it was recognized that there was much more information that related to the capacities of pipelines than there was that related to the demands – pressures – stresses imposed and induced in the pipelines. It is a very high priority future effort that operational pressure and temperature data be gathered, stored, and analyzed on a continuous basis (Figure 7.2, Figure 7.3). Indications from the available data on gas pipelines, indicates that the variabilities of maximum operating pressures are reasonably well understood and modeled in development of these criteria. However, available information on oil and mixed (oil and gas) service pipelines is not so definitive.

In development of these guidelines and criteria, it is specified that the maximum operating pressure (MOP) is equal to the maximum design pressure (MDP). This maximum pressure is important in determining the burst pressure performance characteristics of the pipeline. Criteria for normal in-place operating conditions will be based on the specified MOP minus the external hydrostatic pressure as the median maximum operating pressure. The maximum net operating pressure will have a Coefficient of Variation of 10 %.

7.3 Pressure, Thermal, Pre-tension, and Curvature Longitudinal Stresses

The longitudinal stress in the pipeline results from the summation of longitudinal stresses due to external and internal pressure, thermal expansion related forces (for pipelines transporting hot oil or gas), pre-tension forces due to laying operations, and tensile forces due to suspended segments of the pipeline. In this development, it will be assumed that the primary longitudinal stresses important to the development of requalification criteria

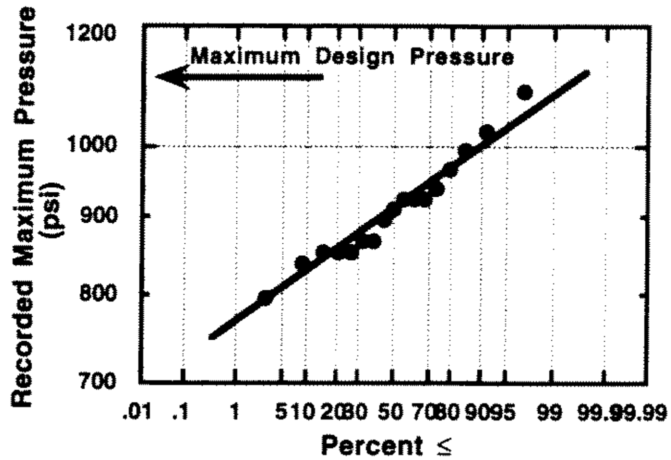

Figure 7.1 – Recorded maximum operating pressures on a 24-inch diameter gas pipeline during a one month period

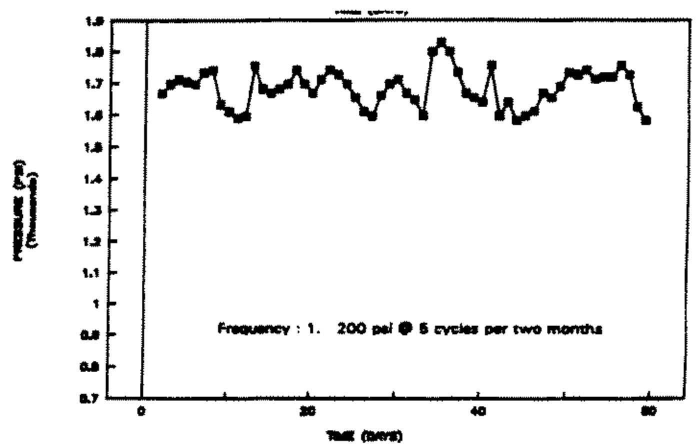

Figure 7.2 – Operating pressures on gas pipeline during 2 month period (note maximum operating pressure for pipeline is 1,800 psi)

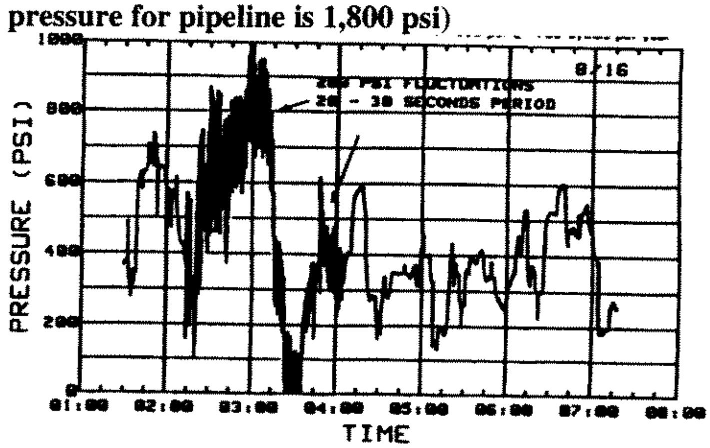

Figure 7.3 – Operating pressures in oil pipeline during 6 hour period (note maximum operating pressure for pipeline is 1,000 psi)

will be the result of pressure and thermal stresses.

The longitudinal force induced in the pipeline by the internal pressure, Pi, is:

$$Ti = Pi\ Ai$$

where Ai is the internal cross sectional area of the pipeline (based on internal diameter).

The longitudinal force induced in the pipeline by external pressure, Po, is:

$$To = Po\ Ao$$

where Ao is the external cross sectional area (based on external diameter).

The effective longitudinal tensile force, Te, induced in the pipeline by Pi and Po is:

$$Te = Ta - Ti + To$$

where Ta is the axial tension in the pipeline due to laying and unsupported portions of the pipeline.

The axial compression force, $C_{\Delta T}$, generated by the differential temperature between the pipeline contents and the sea water, ΔT, is for fully restrained end conditions and no intermediate restraints:

$$C_{\Delta TIR} = As\ E\ \Delta T\ \alpha$$

$C_{\Delta TIR}$ is the compressive force in the pipeline for fully end restrained conditions, As is the area of the steel, E the modulus of elasticity, and α is the thermal expansion coefficient (11 E-6 per C. degree, 6.1 E-6 per F. degree differentials).

For no end (very flexible) or intermediate restraints (very weak soils), the axial compression forces and stresses due to the thermal expansion would be zero (free expansion).

For intermediate axial restraint provided by the soils, Fs, the axial force for full end restraints would be:

$$C_{\Delta T} = C_{\Delta TIR} - Fs$$

Stronger soils would lower the thermal expansion forces and weaker soils would increase the thermal expansion forces. In general, the surface soils in most of the Gulf of Mexico are weak cohesive soils. With or without burial of the pipelines, one would expect that these soil forces would be very low due to the weak soils and wave – soil interaction motions that would tend to relieve soil restraining forces.

During this study, IMP provided information on in-place design of 17 pipelines (IMP, 1998). The design characteristics included soil shear strengths, design pressures and temperatures, product densities, flow rates, and preliminary design stresses. A pipeline – soil – thermal interaction model was employed by IMP to compute the longitudinal thermal expansion stresses associated with these pipelines. These stresses range between about 25 % to 50 % of the pipe steel SMYS. Further analysis by IMP has indicated that as the soil strengths are decreased (e.g. due to creep, wave-soil movements), the thermal stresses in the pipelines increased, and vice versa. The analyses indicated that as the stiffness of the boundary end restraints (locations of maximum thermal compressive stresses) were decreased, the maximum stresses also decreased. Figure 7.4 summarizes the results of the IMP analyses. The maximum stresses in the pipeline are relatively insensitive to the soil shear strength. Two different analytical models were used and these models produced results that were within 10 % of each other.

Based on analyses performed by IMP (1998), axial stresses in a variety of Bay of Campeche pipelines were computed based on thermal and hoop stress calculations as specified by API B31.4 guidelines (1996):

$$S_L = E\,\alpha\,(T_2 - T_1) - \nu\,S_h$$

where S_L is the longitudinal thermal stress, E is the modulus of elasticity of the pipeline steel, α is the linear coefficient of thermal expansion, ν is Poisson's ratio, T_1 is the installation temperature, T_2 is the maximum operating temperature, and S_h is the hoop stress due to pressure. The thermal longitudinal stresses range from 15 % to more than 30 % of the SMTS.

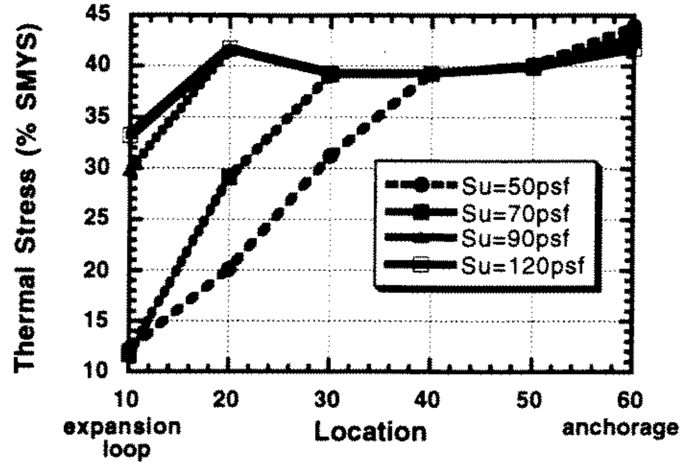

Figure 7.4 – Longitudinal thermal stresses in example pipeline

The foregoing results indicate that the primary variable that determines the the variability of the maximum thermal stresses is the variation in the operating temperatures. Figure 7.5 summarizes the variations in the operating temperatures in a 24-inch diameter gas pipeline (line no. 030) during the period of one month. The Coefficient of Variation of the operating temperatures is 7 %.

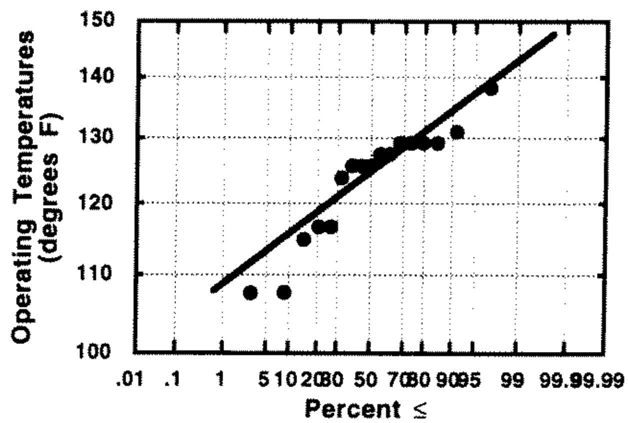

Figure 7.5 – Statistical distribution of operating temperatures in 24-inch diameter gas pipeline

Results from in-place loading and strength analyses of Bay of Campeche pipelines presented by Serpas (1998) and Matias (1998) are summarized in Figures 7.6 and 7.7, the primary in-place stresses in the pipelines studied (20-inch and 8-inch gas pipelines) were due to the bending stresses due to the radius of curvature of the pipeline along its route, the hoop stresses due to external and internal pressures, and the axial – longitudinal pressure stresses. The hoop stresses were in the range of 30 % to 60 % of SMYS. The longitudinal pressure stresses were in the range of 15 % to 30 % of SMYS.

The maximum bending stresses due to the pipeline route curvatures were in the range of 65 % to 70 % of SMYS. The localized bending stresses were the largest contributor to the longitudinal stresses. It was inferred that these high bending stresses were developed when the pipelines were moved laterally from their original location by hydrodynamic forces and soil movements during hurricane Roxanne (Serpas, 1998).

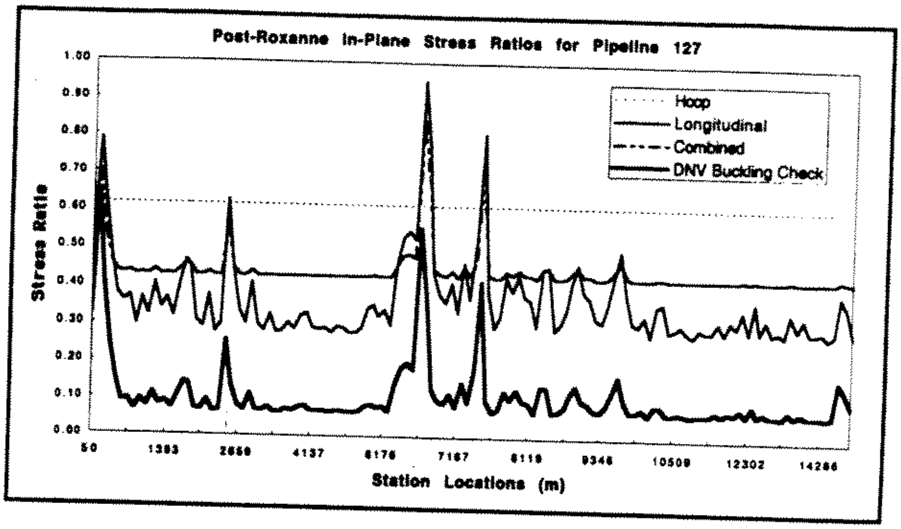

Figure 7.6 – In-place stresses in 20-inch diameter, 0.5-inch wall thickness Akal gas pipeline loop

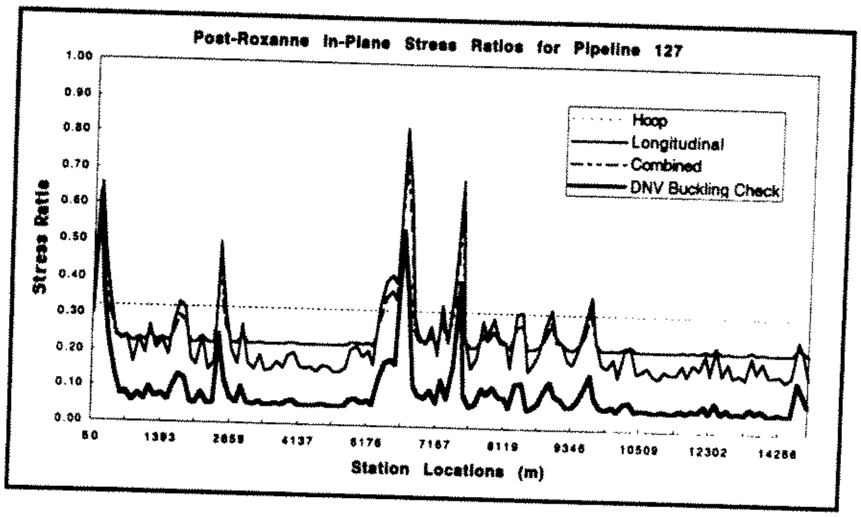

Figure 7.7 – In-place stresses in 8-inch diameter, 0.375-inch wall thickness Akal L gas pipeline

Based on the foregoing evaluations, it is concluded that for the reassessment of existing pipelines, the governing demand will depend on the specific characteristics of the pipeline (e.g. amount of corrosion, movement induced stresses) that is to be evaluated. Inspection and surveying methods can be used to determine the specific pipeline characteristics (Valdes, et al, 1997; Serpas, 1998). This will allow improved definition of the actual conditions contrasted with the projected conditions associated with design of a new pipeline. Based on this rationalle, a median Bias of unity and a Coefficient of Variation of 10 % will be used for development of reassessment criteria for existing pipelines.

Given the limited detailed information that could be developed during this project on PEMEX pipeline in-place operating 'demands' including normal and extreme condition operating pressures, longitudinal – transverse forces, a high priority effort for future developments of these criteria should be a detailed study and quantification of these 'demands.' It is apparent that the Biases and uncertainties associated with the pipeline demands can exceed those associated with the pipeline capacities. Much more information is available on the pipeline capacities than is available on the pipeline demands.

8.0 Assessment of Corrosion & Effects on Burst Pressure Capacities

8.1 Corrosion

Experience with Gulf of Mexico and North Sea pipelines and risers (oil, gas, mixed) clearly shows that the primary operating hazard to the integrity of pipelines and risers is corrosion. For pipelines, the primary hazard is internal corrosion. For risers, the primary hazard is external corrosion, particularily in the vicinity of the waterline and in the atmospheric zone immediately above the waterline.

There are three fundamental approaches to evaluating corrosion effects in the requalification of pipelines:

- use of instrumentation and inspections to detect and quantify corrosion defects,
- use of corrosion coupons to quantify corrosion rates, and
- use of indirect indicators of corrosivity and corrosion rates.

Corrosion is a complex function of what is transported in the pipeline or riser, what surrounds the exterior of the pipeline or riser, and how the corrosion is managed. Corrosion is an extremely complex electro-chemical-mechanical process in which the properties of the steel that comprises the pipeline can be degraded. Both thickness and strength can be diminished. Primary corrosion rate determining parameters include:

- Temperature,
- Water composition (pH, salinity), and concentration (collection & stagnation sectors),
- Product composition (chemical composition, pH),
- Operational parameters including flow rates, regime, pressures, and oil-water wetting,
- Steel – weld properties including macro and micro structrure, alloying elements, and consumables,
- Sulphate reducing bacterial (SRB) count and types,
- Deposits - coatings on the steel surfaces,
- Steel cracking (stress corrosion fatigue),
- Erosion due to the transport of solids, and
- 'Stray' currents associated with electrical operating equipment and, and other metals that can come into or are placed in contact with the pipeline.

Anything that can accelerate the transport of electrons from a cathode to an anode will accelerate corrosion, and vice versa. In general, all of the parameters cited can be expected to change during the life of a pipeline because the sources of oil, water, and gas transported through a pipeline are changing and because the external environmental and operational conditions are continuously changing. Similar statements can be made regarding the effects of changes in space along the length and around the perimeter of the pipeline.

Corrosion management is paramount if corrosion is to be controlled in pipelines and risers. Management processes include dehydration, inhibition, coatings, use of bactericides, pH neutralizers, inspections, inspections, instrumentation, and use of coupons to indicate corrosion rates. Once the steel has been lost and its properties degradated, they can not be restored other than by replacement. For long-life pipelines, corrosion management, or lack thereof, can pay rich dividends.

A wide variety of models have been developed to help predict or evaluate potential corrosion rates; the most popular model is that of de Waard and Milliams (1975), and de Waard et al. (1991, 1995). Enhancements have been developed to recognize the effects of multiphase slug flow (Jepson, et al, 1997), and other important factors (Papavinasam, 1997). Most of the models have been founded on laboratory studies (de Waard, et al, 1991; Jepson, et al., 1997) conducted using laboratory 'pipe runs'. Very few of these models have been verified with field pipeline experience. Because there are so many parameters that influence corrosion, there are no universal models; the models are applicable to restricted ranges and combinations of parameters. During the conduct of this review, not a single comprehensive database on corrosion could be located that would serve to define the biases and uncertainties associated with the corrosion models. At best, the general expert opinion was that the current models used by industry generally substantially over-predict the corrosion rates associated with internal and external corrosion of marine pipelines (Thill, 1998; Esaklul, 1998).

Information can be developed on corrosion through the use of 'intellegent' or 'smart' pigs. There are a wide variety of such instrumentation including Magnetic Flux Leakage, Ultra Sonic, and Eddy Current devices. In addition, there are caliper and inertia pigs that are able to detect dents / deformations of the pipe and the position of the pipe, respectively. However, the data obtained by such instruments, while extremely helpful, is fraught with problems associated with both the instrumentation and the interpertation of the data from the instrumentation. Both 'false negatives' (falws indicated not present, or over-sized) and 'false positives' (flaws not indicated or missed, or under-sized). There can be wide differences between the results developed by different types of inline instruments. There can be wide differences betweeen the interpertations of the results developed by different types of inline instruments. It is clear that additional standardization, calibration, and verification are needed before the results from these instruments can be regarded as 'unbiased' and the uncertainties associated with the results quantified. There are similar problems associated with instrumentation used to gage metal losses from outside the pipeline. More will be developed on these points later in this report.

Corrosion coupons are also extremely useful. They can be installed in pig trap and manifold areas, and periodically removed to determine the corrosion rates and indicate changes that may be developing in the corrosivity of the fluids in the pipeline. However, the coupons are also limited because they can only give general indications of corrosion conditions. This is because they can not be placed througout the pipeline to be able to sense local corrosion conditions.

The sad fact for most pipelines is that neither corrosion coupons or instrumentation results are available. In addition, most pipelines in the Gulf of Mexico, and certainly the older pipeline systems, can not be pigged. Thus, indirect methods must be used to estimate and evaluate corrosion rates and conditions. Hence, the need for analytical models.

It is important to note that when there is sufficient information available from instrumentation and coupons, then this information can be used to infer what corrosion could be expected in a given pipeline for a given set of conditions. The problem with this is that it is rare to find a sufficiently developed or detailed database on pipeline corrosion to use this approach. Note also that the failures of pipelines, could be used in a similar way. If there were sufficient information on the conditions under which a pipeline lost containment due to corrosion, then this informatoion could be used to estimate corrosion rates. However, neither the models or the data exist to perform such determinations. Thus, at the present time, a pipeline requalification and reassessment guideline and process must involve the development of models to estimate corrosion in pipelines.

There are two general types of models in use at the present time. The first can be termed 'qualitative.' These models are based on scoring or ranking methods to develop general indicators of the rates and extents of corrosion. These methods attempt to incorporate recognition of the parameters and conditions that are of dominant importance in determining the potential corrosion rates and locations.

The second type of model can be termed 'quantitative.' These models are based on measurements of pipeline wall losses, either internal or external. Smart pigs can be used to perform the internal measurements and ultrasonic devices used to perform the external measurements.

Examples of both qualitative and quantitative models will be developed in this Section.

8.2 Corrosion Rates Based on Gaugings

PEMEX and IMP have developed an extensive database on measured 'maximum' (maximum wall loss at given location) corrosion in pipelines and risers in the Bay of Campeche (Lara, et al, 1998). This database is summarized in Table 8.1.

Table 8.1 – Results from gauging inspections of Bay of Campeche Pipelines

Insp. Date	Pipe No.	D (in)	Service	Install Year	t (in)	t min (in)		Corrosion Rate (in / yr)	
						ZONE E	ZONE F	ZONE E	ZONE F
1998	001-A	36	OIL	1979	0.843	0.600	0.550	0.01279	0.01542
1998	002-B	24	OIL-GAS	1979	0.700	0.540	0.500	0.00842	0.01053
1998	003-B	20	OIL	1979	0.88	0.470	0.460	0.02158	0.02211
1998	004-B	14	OIL-GAS	1979	0.460	0.360	0.360	0.00526	0.00526
1995	005-A	16	OIL	1979	0.570	0.480	0.500	0.00563	0.00438
1996	008-A	14	GAS	1979	0.700	0.370	0.370	0.01941	0.01941
1998	008-B	14	GAS	1979	0.670	N/A	0.420	N/A	0.01316
1997	009-A	24	GAS	1979	0.840	0.662	0.680	0.00989	0.00889
1998	009-B	24	GAS	1979	0.800	0.600	0.680	0.01053	0.00632
1997	010-B	14	OIL-GAS	1979	0.585	0.370	0.350	0.01194	0.01306
1996	012-B	20	OIL-GAS	1980	0.700	0.480	0.480	0.01375	0.01375
1998	013-A	14"	OIL	1979	0.600	0.440	0.430	0.00842	0.00895
1998	014-B	14	OIL	1980	0.570	0.300	0.320	0.01500	0.01389
1996	015-A	24	OIL	1980	0.840	0.600	N/A	0.01500	N/A
1996	016-A	14	GAS	1980	0.500	0.430	0.450	0.00438	0.00313
1996	016-B	14	GAS	1980	0.540	0.410	N/A	0.00813	N/A
1998	017-A	14	OIL-GAS	1980	0.480	0.340	0.380	0.00778	0.00556
1998	017-B	14	OIL-GAS	1980	0.540	0.360	0.400	0.01000	0.00778
1996	020-B	20	OIL-GAS	1980	0.800	0.640	0.620	0.01000	0.01125
1997	021-A	36	OIL	1980	1.000	0.640	0.600	0.02118	0.02353
1998	027	36	GAS	1981	1.050	0.880	0.740	0.01000	0.01824
1997	030-A	24	GAS	1981	0.860	0.540	0.540	0.02000	0.02000
1997	030-B	24	GAS	1981	0.900	0.680	0.540	0.01375	0.02250
1998	045-A	24	GAS	1981	0.800	0.670	0.670	0.00765	0.00765
1997	045-B	24	GAS	1981	0.770	0.660	0.670	0.00688	0.00625
1997	046-A	24	GAS	1981	0.830	0.680	0.670	0.00937	0.01000
1997	046-B	24	GAS	1981	0.800	0.670	0.680	0.00813	0.00750
1998	048-A	24	GAS	1981	0.830	0.740	0.600	0.00529	0.01353

Table 8.1 – Results from gauging inspections of Bay of Campeche Pipelines

Insp. Date	Pipe No.	D (in)	Service	Install Year	t (in)	t min (in)		Corrosion Rate (in / yr)	
						ZONE E	ZONE F	ZONE E	ZONE F
1996	048-B	24	GAS	1981	0.950	0.760	0.620	0.01267	0.02200
1998	067-B	36	GAS	1981	0.860	0.600	0.630	0.01529	0.01353
1996	070-A	36	GAS	1982	0.940	0.880	0.750	0.00429	0.01357
1996	079-A	20	OIL-GAS	1985	0.746	0.620	0.600	0.01145	0.01327
1996	079-B	20	OIL-GAS	1985	0.750	0.600	0.620	0.01364	0.01182
1998	081-A	36	OIL	1985	1.200	0.771	0.981	0.03300	0.01685
1998	081-B	36	OIL-GAS	1985	1.075	0.881	1.040	0.01492	0.00269
1996	089-A	24	OIL-GAS	1986	0.860	0.800	0.800	0.00600	0.00600
1997	089-B	24	OIL-GAS	1986	0.900	0.800	0.720	0.00909	0.01636
1996	091-A	20	OIL	1986	0.900	0.610	N/A	0.02900	N/A
1998	092-A	20	OIL-GAS	1987	0.760	0.620	0.640	0.01273	0.01091
1996	093-B	20	GAS	1987	0.940	0.830	0.840	0.01222	0.01111
1998	093-01	8	GAS	1987	0.540	0.480	0.480	0.00545	0.00545
1996	093-05	8	GAS	1987	0.500	0.490	0.320	0.00111	0.02000
1998	093-06	8	GAS	1987	0.580	0.500	0.500	0.00727	0.00727
1996	093-09	8	GAS	1987	0.530	0.480	0.500	0.00556	0.00333
1998	093-10	8	GAS	1987	0.520	0.460	0.460	0.00545	0.00545
1996	093-11	8	GAS	1987	0.520	0.480	0.320	0.00444	0.02222
1996	093-12	8	GAS	1987	0.520	0.480	0.520	0.00444	0.00000
1996	093-13	8	GAS	1987	0.540	0.490	0.470	0.00556	0.00778
1996	096-A	20	OIL-GAS	1988	0.800	0.590	0.600	0.02625	0.02500
1996	096-B	20	OIL-GAS	1988	0.720	0.630	0.640	0.01125	0.01000
1998	111-B	20	OIL	1991	0.730	0.490	0.500	0.03429	0.03286
1996	117-A	24	GAS	1991	0.780	0.600	0.720	0.03600	0.01200
1995	117-B	24	GAS	1991	0.680	0.540	0.570	0.03500	0.02750
1998	118-A	24	GAS	1991	0.630	0.560	0.540	0.01000	0.01286
1996	119-A	24	OIL-GAS	1991	0.660	0.570	0.570	0.01800	0.01800
1998	121-B	20	OIL-GAS	1991	0.600	0.490	0.520	0.01571	0.01143
1997	132-A	24	GAS	1992	0.730	0.640	0.670	0.01800	0.01200
1996	132-B	24	GAS	1992	0.780	0.630	0.620	0.03750	0.04000
1996	135-A	24	GAS	1992	0.640	0.570	N/A	0.01750	N/A
1996	136.1	8	GAS	1992	0.570	0.470	N/A	0.02500	N/A
1998	137-A	8	GAS	1992	0.540	0.440	0.440	0.01667	0.01667
1998	137-B	8	GAS	1992	0.511	0.480	0.480	0.00517	0.00517
1997	138-A	8	GAS	1992	0.600	0.500	0.500	0.02000	0.02000
1997	138-B	8	GAS	1992	0.710	0.500	N/A	0.04200	N/A
1997	139-A	20	OIL	1992	0.760	0.600	0.600	0.03200	0.03200
1996	140-B	24	OIL-GAS	1992	0.830	0.700	0.640	0.03250	0.04750
1997	152-A	36	GAS	1994	1.250	N/A	1.180	N/A	0.02333

Figure 8.1 identifies the locations of Zone E and Zone F. Zone E is at the lower section of the riser. Zone F is in the expansion loop portion of the pipeline.

Figure 8.2 summarizes corrosion rate data from all of the pipelines. The median corrosion rate is 0.015 inches per year for both Zones. The corrosion rate has a Coefficient of Variation of 68 %.

Figure 8.3, Figure 8.4, and Figure 8.5 summarizes data for the oil, gas, and mixed oil & gas service as a function of the age of the pipelines. The term 'average' refers to the average of the maximum wall thickness loss over a given period of time.

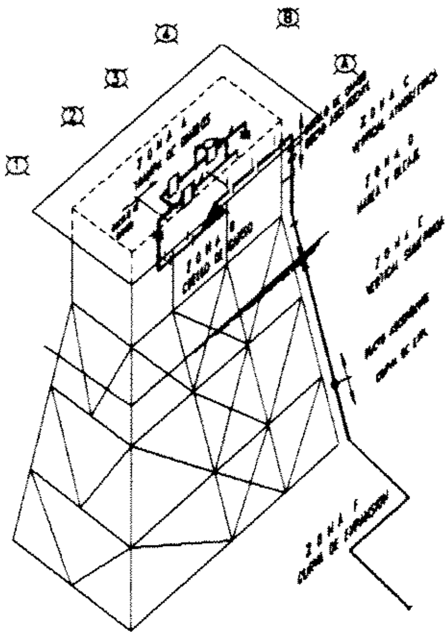

Figure 8.1 – Zones for corrosion gauging inspections

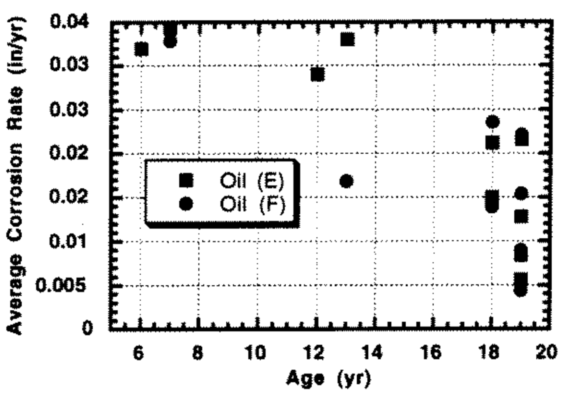

Figure 8.3 – Oil service corrosion rates as function of pipeline age

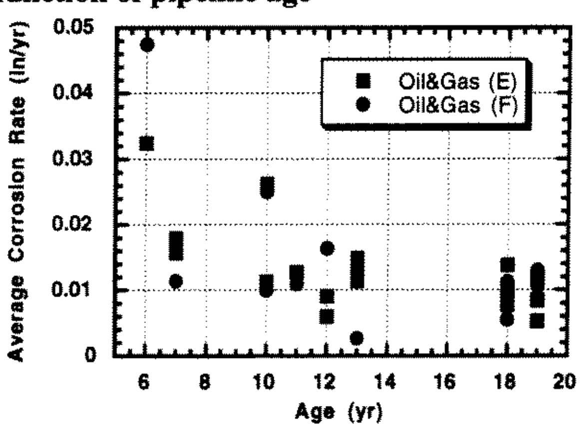

Figure 8.5 – Mixed oil and gas service corrosion rates as function of pipeline age

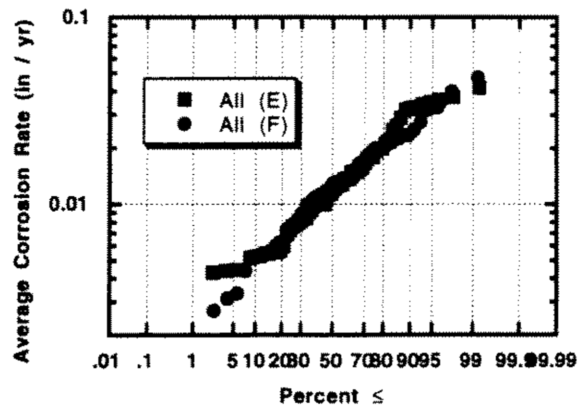

Figure 8.2 – Corrosion rate data for all pipelines (Table 8.1)

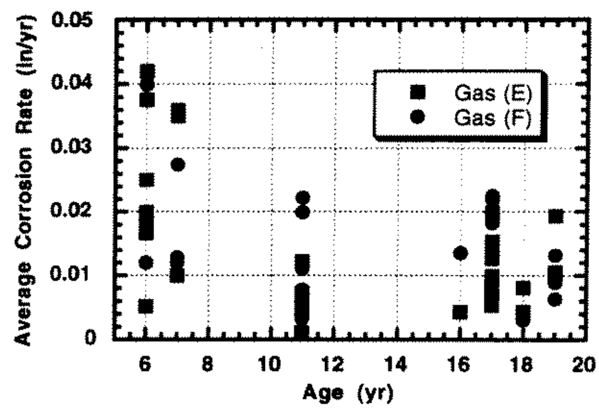

Figure 8.4 – Gas service corrosion rates as function of pipeline age

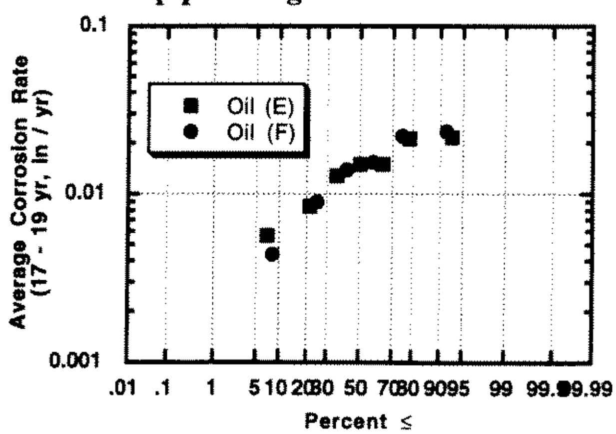

Figure 8.6 –Oil pipeline corrosion rates for ages 17 – 19 years

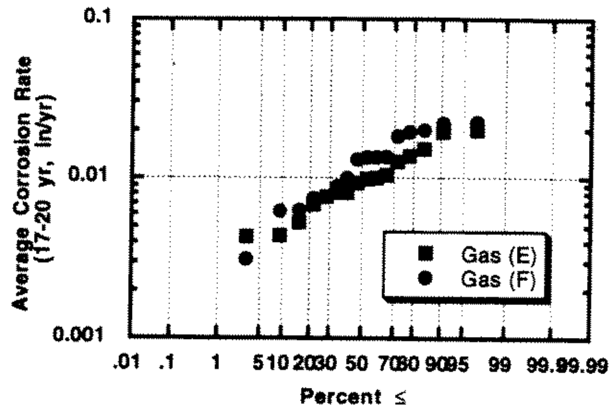

Figure 8.7 – Gas pipeline corrosion rates for 17 – 19 years

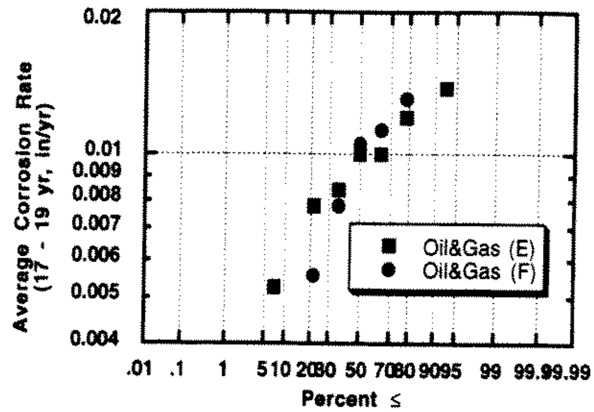

Figure 8.8 – Oil and gas pipeline corrosion rates for 17 – 19 years

There is a definite decrease in the corrosion rates with time for all services. The initial rates of corrosion for gas lines are generally about half those associated with the oil lines. The rate of corrosion is much higher early in the life of the pipeline.

Figures 8.6, 8.7, and 8.8 summarize the pipeline corrosion rates for pipelines that are 17 to 19 years old. The median rate of corrosion for oil pipelines is 0.015 in / yr (15 mpy). The oil pipelines corrosion rate has a Coefficient of Variation of 40 %. The medain rate of corrosion for gas pipelines is 0.010 in/yr (10 mpy). The gas pipelines corrosion rate has a Coefficient of Variation of 40 %. The mixed oil and gas pipelines median corrosion corrosion rate is 0.01 in / yr. The mixed pipelines corrosion rate has a Coefficient of Variation of 30 %.

Figure 8.9 summarizes results from a one year study of internal corrosion rates associated with the gas pipeline risers on the Abkatum A compression platform (Zone A, Figure 8.1). The average corrosion rate through the year is about 0.005 inches per year, however, the maximum corrosion rate can reach 0.01 to 0.015 inches per year. These are all older pipelines (10 to 18 years). There seems to be a tendency for the high corrosion rates to be associated with the winter season months (due to condensation in the pipelines).

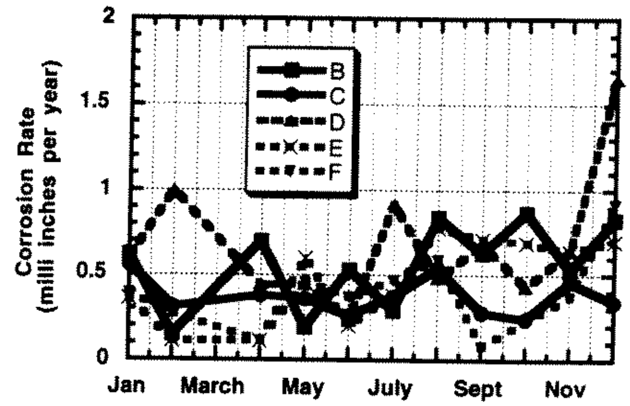

Figure 8.9 – Corrosion rate study on gas risers in Abkatum A Compression platform

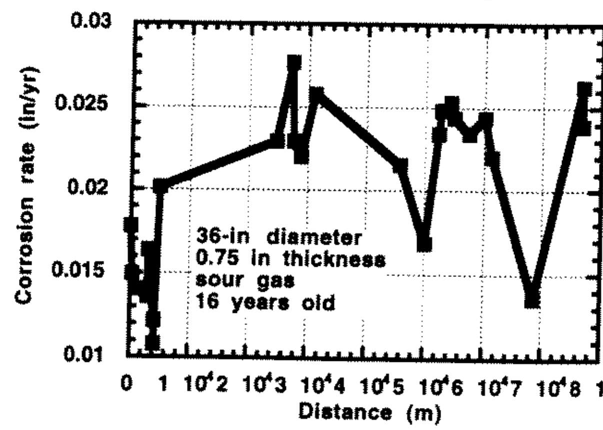

Figure 8.10 – Measured maximum corrosion rates in sour gas pipeline

8.3 Example: Corrosion Effects on Burst Pressures

Figure 8.10 shows data on measured maximum corrosion rates (based on measured maximum depths of corrosion at point on the pipeline) recently obtained on a 36-inch diameter gas pipeline (Number 27, Atasta to Nohoch A drilling) that has had 16 years of service transporting sour gas (Lara, et al.

1998). Sour gas is extremely corrosive. Thus, these corrosion rates should represent a maximum that could be expected for a long-life pipeline. The maximum corrosion rate is 0.028 in / yr (28 mpy).

Figure 8.11 summarizes this data in terms of the maximum depth of corrosion in the pipeline. In some sections, this pipeline has lost almost 60 % of the wall thickness.

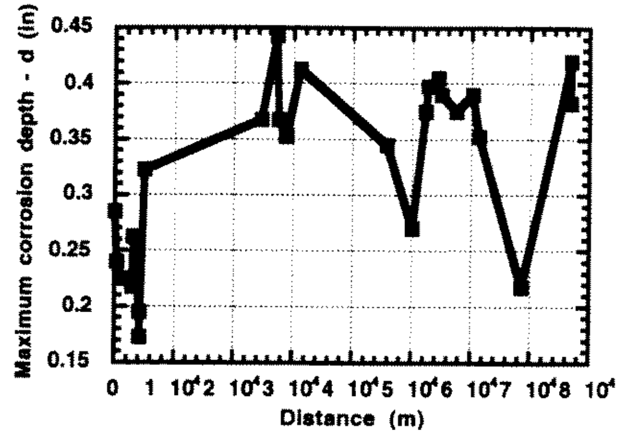

Figure 8.11 – Measured loss in wall thickness in sour gas pipeline

This data provides some important information to help calibrate or verify the analytical model developed during RAM PIPE to predict the burst pressure of corroded pipelines. This pipeline is operating at a pressure of 1,200 psi (maximum design pressure was 1320 psi). The pipeline is 36-inches in diameter, has a 0.75 inch wall thickness, and was fabricated with X60 steel (SMYS = 60 ksi; SMTS = 75 ksi). The external hydrostatic pressure is about 70 psi. Thus, the present net pressure in this pipeline is 1,130 psi. The maximum depth of corrosion measured in this pipeline is d = 0.42 inches. The reassessment burst pressure could be computed to be:

$$Pb = 2.2\ (t\text{-}d)\ (SMTS)\ /\ (D\text{-}t)\ SCF$$

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

$$SCF = 1 + 2\ (0.42 \text{ in} / 18 \text{ in})^{0.5} = 1.31$$

$$Pb = 2.2\ (0.75 \text{ in} - 0.42 \text{ in})\ (75{,}000 \text{ psi})\ /\ (36 \text{ in} - 0.75 \text{ in})\ 1.31 = 1179 \text{ psi}$$

$$Po \leq 0.8\ Pb \leq 943 \text{ psi}$$

The pipeline is being operated at a pressure that exceeds the allowable reassessment pressure. Figure 8.12 summarizes the computed, allowable, and operating pressure profile along this pipeline. The pipeline is being operated above the RAM PIPE allowable pressure along considerable portions of the pipeline.

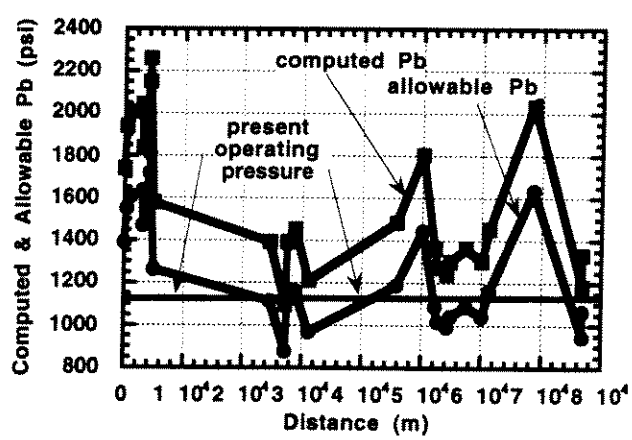

Figure 8.12 – Computed and allowable burst pressures along sour gas pipeline

The allowable reassessment pressure is based on an annual Safety Index of $\beta = 3.4$ or an annual probability of failure of Pf = 4 E-4. It would be of interest to determine what the probability of failure 'profile' looks like for this pipeline. This profile is summarized in Figure 8.13. The pipeline has a probability of failure of one chance in 10 or 10 % per year over substantial portions of the pipeline.

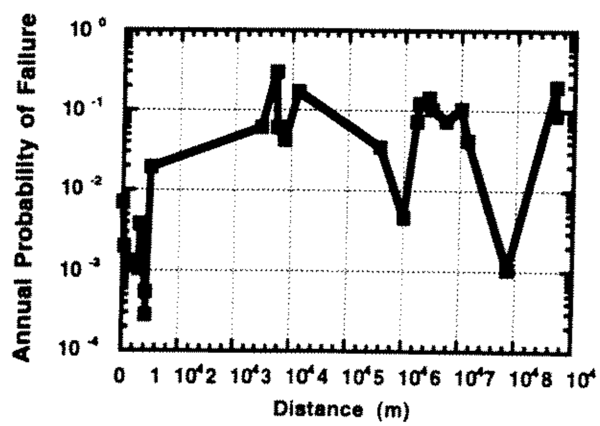

Figure 8.13 – Annual probability of failure of pipeline 27 based on measured corrosion loss

As summarized in Figure 8.14, given the present maximum rate of corrosion of about 0.025 in / yr, this pipeline could be expected to loose containment in about 2 years. In three years, there would be substantial sections of this pipeline that would have a probability of failure (loss of containment – leak) of unity. It will be intersting to watch the operating history of this pipeline. This illustration indicates how the history of pipeline leaks can be used to help 'up-date' the design and reassessment models and processes.

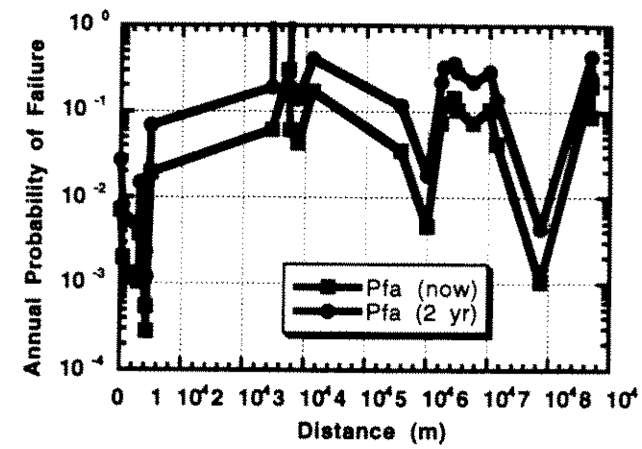

Figure 8.14 – Annual probabilities of failure of pipeline 27 in 1998 and in 2000

8.4 Corrosion Based on Inline Instrumentation

Instrumentation or 'smart pigs' can be used to help develop evaluations of corrosion rates and remaining wall thicknesses (Rosen Engineering Group, 1997). These measurements can be used to help make evaluations of corrosion in comparable pipelines that can not be instrumented. Figure 8.15 shows a probability distribution of corrosion rates determined for the Alyeska pipeline and North Sea oil pipelines based on results from smart pigs. The median values of the corrosion rates are $v = 0.06$ and $v = 0.03$ mm/year for these two sets of pipelines. This type of information can be used to help update corrosion rates based on indirect measurements and on qualitative methods such as is discussed in the next section.

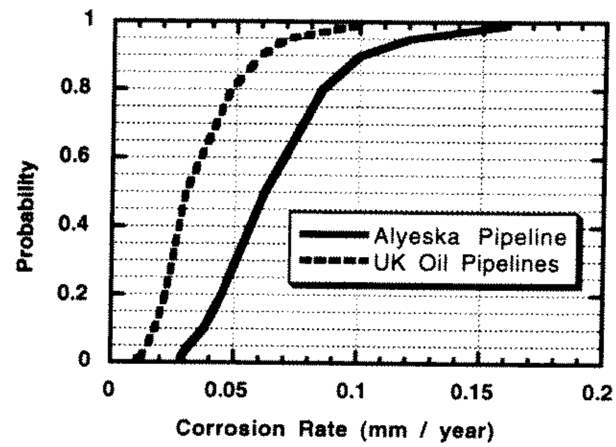

Figure 8.15 – Measured corrosion rates in Alyeska and UK oil pipelines

It is important recognize that making evaluations of corrosion rates and wall thicknesses from the recordings have significant uncertainties (Bal, Rosenmoeller, 1997). The measurements can give both 'false positives' and 'false negatives.' The pigs can miss significant defects and indicate the presence of defects that are not present. Figure 8.16 shows a comparison of the Probability of Detection (POD) of corrosion depths (in mils, 50 mils = 1.27 mm) developed by three different 'smart pigs' (Magnetic Flux Leakage, MFL, based instrumentation). This information was based on comparing measured results from sections of the Trans Alaska pipeline that were pigged and then excavated and the true corrosion depths determined (Rust, et al 1996; Vieth, et al 1996).

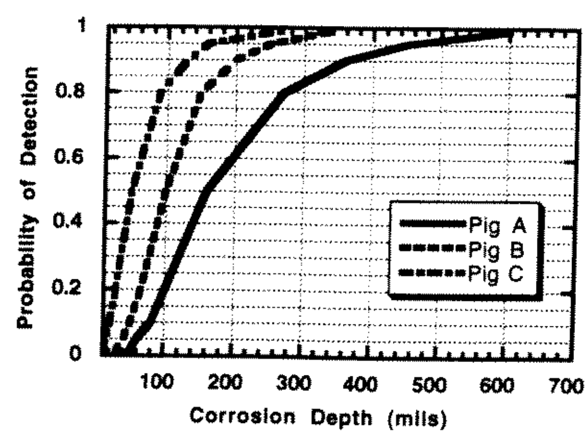

Figure 8.16 – Probability of Detection Curves for Three Smart Pigs

There is a dramatic difference in the performance characteristics of these three smart pigs. If this type of variability is to be avoided or minimized, then specifications and test runs must be developed to verify the ability of the pigs to detect corrosion damage. Specifications for intelligent pig inspections of pipelines need to be developed if consistent and repeatable results are to be realized (Shell International, 1996).

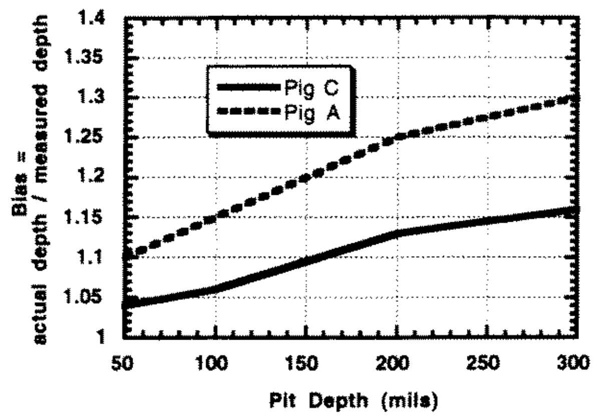

Figure 8.17 – Bias in measured corrosion depths

There are significant uncertainties in the depths of corrosion indicated by the pigs due to such factors as variable temperatures and degrees of magnetism, and the speed of movements of the pig (Bal, Rosenmoeller, 1997). Corrosion rates are naturally very variable in both space and time. Thus, if instrumentation is used to determine the wall thicknesses and corrosion rates, the uncertainties in these characteristics needs to be determined and integrated into the evaluations of the fitness for purpose of the pipeline. Figure 8.17 summarizes data for two of the smart pigs noted in Figure 8.15. Both pigs tend to under estimate the corrosion depth. The uncertainties associated with the measured depths ranged from 35 % (for 50 mils depths) to 25 % (for 200 mils depths).

For the instrumented pipelines, the expression for the probability of failure can be expressed as:

$$Pf = Pf_D + Pf_{ND}$$

where Pf_D is the probability of failure associated with the detected flaws and Pf_{ND} is the probability of failure associated with the non-detected flaws.

The detected depth of corrosion must be corrected to the median depth of corrosion (Figure 9.17):

$$t_{C50} = t_{CD} \, (B_{Dt})$$

The detected depth of corrosion has a standard deviation of the Logarithms of the corrosion depths of:

$$\sigma_{lntc} = 0.25 \text{ to } 0.35$$

The probability of failure associated with the detected depth of corrosion is:

$$Pf_D = 1 - \Phi\{ [\ln (p_{B50} / p_{O50})] / [(\sigma^2_{pB} + \sigma^2_{pO}) ^{0.5}] \}$$

where Φ is the standard cumulative Normal distribution, p_{B50} is the 50th percentile (median) burst pressure, p_{O50} is the 50th percentile maximum operating pressure, σ_{pB} is the standard deviation of the logarithms of the burst pressure, and σ_{pO} is the standard deviation of the logarithms of the maximum operating pressures.

The pipeline burst pressure is determined from:

$$p_B = 2 \, S \, (t - tc) / D$$

The median of the burst pressure is determined from the medians of the variables:

$$p_{B50} = 2\, S_{50} (t_{50} - t_{C50}) / D_{50}$$

The uncertainty in the burst pressure is determined from the standard deviations of all of the variables:

$$\sigma^2_{\ln pB50} = \sigma^2_{\ln S} + \sigma^2_{\ln t} + \sigma^2_{\ln tc} + \sigma^2_{\ln D}$$

The probability of a corrosion depth, X, exceeding a lower limit of corrosion depth detectability, xo, is:

$$P[\, X \geq xo \mid ND\,] =$$
$$P[\, X > xo\,]\, P[\, ND \mid X \geq xo\,] / P[\, ND]$$

$P[\, X \geq xo \mid ND\,]$ is the probability of no detection given $X \geq xo$. $P[\, X > xo\,]$ is the probability that the corrosion depth is greater than the lower limit of detectability (Figure 8.16). $P[\, ND \mid X \geq xo\,]$ is the probability of non detection given a flaw depth (Figure 8.16). P [ND] is the probability of non detection across the range of flaw depths (Figure 8.16) where:

$$P[ND] = 1 - P[D]$$

and:

$$P[ND] = \Sigma\, P[ND \mid X > xo]\, P[X > xo]$$

The probability of failure for non-detected flaws is the convolution of:

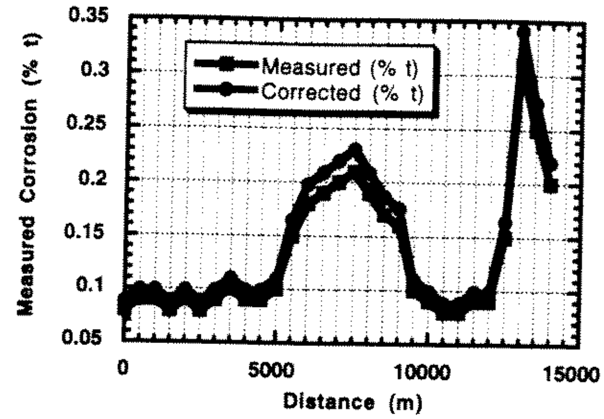

Figure 8.18 – Pig C measured and corrected corrosion readings

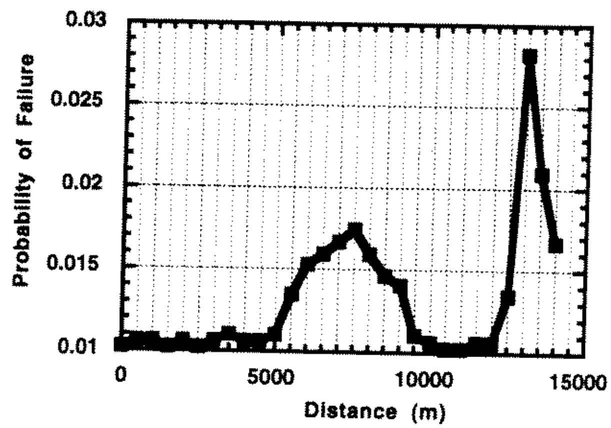

Figure 8.19 – Probabilities of burst pressure failure

$$Pf_{ND} = \Sigma\, [Pf \mid X > xo]\, P[\, X \geq xo \mid ND\,]$$

Figure 8.18 shows results from an instrumentation of a 20-inch diameter gas line based on use of Pig C The measured and corrected corrosion expressed as a percentage of the wall thickness is shown. Based these results and foregoing developments, Figure 8.19 shows the probabilities of burst failure (detected and non-detected) of the pipeline. Two sections of the pipeline would be candidates for replacement.

8.5 Corrosion Based on Qualitative Model

Experience with Gulf of Mexico pipelines and risers (oil and gas) has clearly shown that the primary operating hazard to the integrity of pipelines and risers is corrosion; primarily internal corrosion for pipelines, and external corrosion for risers (generally in the vicinity of the mean water level) (Elsayed, Bea, 1997; Marine Board, 1994; AME 1993; Mandake, 1990).

Corrosion of steel in pipelines and risers is a function of what is transported in the pipeline or riser, what surrounds the exterior of the pipeline or riser, and how the corrosion is 'managed' (ASME, 1991; Bea, 1992; 1994). A variety of techniques can be used to reduce the rates of corrosion including internal or external coatings, cathodic protection (for continuously submerged segments of pipelines), dehydration of the gas or oil, and the use of inhibitors. Marine growth tends to inhibit or reduce corrosion of risers (NACE, 1992; Kvernvold, et al, 1992).

For this approach, the loss of pipeline or riser wall thickness due to corrosion (tc) was formulated as follows:

$$tc = tci + tce$$

where tci is the loss of wall thickness due to internal corrosion and tce is the loss of wall thickness due to external corrosion.

The loss of wall thickness due to internal and/or external corrosion (tci/e) was formulated as follows (Elsayed, Bea, 1997):

$$tci/e = \alpha i/e \ vi/e \ (Ls - Lpi/e)$$

where $v_{i/e}$ is the average (mean during service life) corrosion rate, $\alpha_{i/e}$ is the effectiveness of the inhibitor or protection (1.0 is perfect protection, and 10.0 is little effective protection), Ls is the service life of the pipeline or riser (in years), and $Lp_{i/e}$ is the 'life' of the initial protection provided to the pipeline.

This model assumes that there are no inspections and repairs performed during the service life of the pipeline or riser to maintain the strength integrity of the pipeline to carry pressure. Maintenance is required to preserve the protective management measures 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 requalified for service, inspections should be performed to disclose the condition of the pipeline and riser, and then an assessment performed to determined if under the then '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 requalification , the corrosion rate is based on the owner/operators evaluation of the corrosivity of the fluids and/or gases transported inside the pipeline or riser, and of the corrosivity of the external environment conditional on the application of a certain protection or 'inhibition' program. Table 8.2 summarizes suggested median corrosion rates, their variabilities (standard deviations 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 (Elsayed, Bea, 1997; NACE, 1992).

Table 8.2 - Internal (i) and External (e) Corrosion Rates (v) and Variabilities

Descriptor	Corrosion Rate mm/year	Corrosion Rate Variability - %
Very Low	0.001	10
Low	0.01	20
Moderate	0.1	30
High	1.0	40
Very High	10.0	50

For example, a dehydrated sweet gas would generally have a low to very low corrosion rate (0.001 to 0.01 mm/year), particularly if inhibitors were used to protect the steel. A 'normally' dehydrated sweet oil without inhibitors could have a moderate corrosion rate (0.1 mm/year). A pipeline transporting high temperature salt water could have a corrosion rate that would be High to Very High (1.0 to 10.0 mm/year). Sour wet gas without any inhibitors could have similar corrosion rates (in addition to degrading the steel material properties).

A riser in the splash zone in the Gulf of Mexico without coating protection could have a corrosion rate that is High (1 mm/year). This zone would extend from mean low water to about 4 m above mean low water. Below this zone, the corrosion rate would be Moderate (0.1 mm/year), although local riser connections and other elements that could lead to local corrosion or pitting could have a corrosion rate that would be High (1.0 mm/year). An unprotected pipeline could be expected to have an external corrosion rate that would be Moderate (0.1 mm/year), unless there were other factors that could increase this rate (very high water velocities, severe erosion caused by sediment movements).

Table 8.3 - Internal (i) and External (e) Inhibitor Efficiency ($\alpha_{i/e}$)

Descriptor	Inhibitor Efficiency
Very Low	10.0
Low	8.0
Moderate	5.0
High	2.0
Very High	1.0

In this development, the effectiveness of corrosion management is expressed with two parameters, the inhibitor efficiency ($\alpha_{i/e}$) and the life of the protection ($Lp_{i/e}$). If the inhibitor (e.g. coating, dehydration, chemical inhibitor, cathodic protection) were 'perfect', then $Lp_{i/e}$ would equal 1.0. If experience had indicated otherwise, then the inhibitor efficiency could be introduced as summarized in Table 8.3.

The life of the protection reflects the operator's decision regarding how long the protection that will be provided will be effective at preventing steel corrosion. For example, the life of high quality external coatings in the absence of mechanical damage can be 10 years, where the life of low quality external coatings with mechanical damage can be 1 year or less. Another example would be cathodic protection that could be reasonably provided to protect the pipeline for a period of 10 years, but the expected life of the pipeline was 20 years. Thus, there would be 10 years of life in which the cathodic protection was not provided and the steel would be 'freely' corroding. Table 8.4 defines the general categories of the life of protective systems. This same Table can be used to specify the expected service life of the pipeline or riser (Ls).

Table 8.4 - Expected Life of the Protective System ($Lp_{i/e}$)or the Service Life of the Pipeline or Riser (Ls)

Descriptor	$Lp_{i/e}$ or Ls (years)
Very Short	1
Short	5
Moderate	10
Long	15
Very Long	≥ 20

Given this information, pipeline operators could define the expected life of the pipeline or riser (e.g. Very Long, Ls = 20 years), define the life of the protective management system that would be incorporated as a part of the pipeline or riser (e.g. Moderate, $Lp_{i/e}$ = 10 years), define the effectiveness of the protective management system (e.g. High, $\alpha_{i/e}$ = 2.0), and then based on the transported product and environment of the pipeline or riser, estimate the internal and external corrosion rates (e.g. vi = 0.1 mm/year, ve = 0.1 mm/year). The corrosion thickness allowance would then be determined as:

$$t_{ci/e} = \alpha_{i/e}\, v_{i/e}\, (Ls - Lp_{i/e}) = 2.0 \cdot 0.2 \text{ mm/y } (20 \text{ y} - 10 \text{ y}) = 4 \text{ mm}$$

This formulation could be expressed in terms of 'effective' corrosion rates ($ve_{i/e}$) and 'exposed life' ($Le_{i/e}$) as follows:

$$t_{ci/e} = ve_{i/e}\, (Le_{i/e})$$

Figure 8.20 shows the results of this development in the form of a corrosion allowance($t_{ci/e}$) as a function of the 'effective' corrosion rate (internal and external, $ve_{i/e}$) and 'exposed life' ($Le_{i/e}$). The linguistic variables identified in Table 9.1 were used in this illustration.

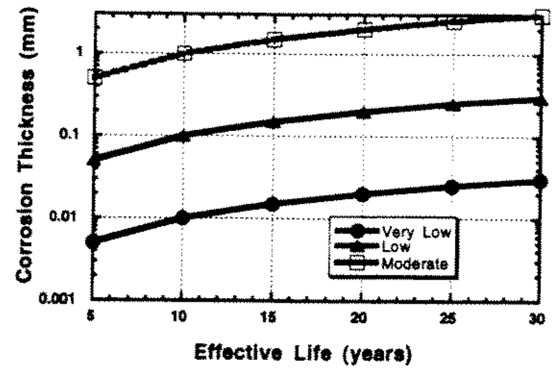

Figure 8.20 - Corrosion Thickness Allowance As Function of Exposed Life and Mean Rate of Corrosion

Corrosion thickness allowances for moderate corrosion and long lives are 1 mm to 2 mm. For highly effective corrosion management programs, the corrosion thickness allowances for low effective corrosion rates are 0.1 to 0.2 mm.

8.6 Corrosion Based on Quantitative Model

The background on pipeline corrosion developed during the PIMPIS (Pipeline Integrity, Maintenance, Performance Information System) project (Farkas, Bea, 1998) developed the following quantitative model to predict / evaluate pipeline corrosion losses:

$$CorrosionLoss = \left[1 + e^{(1-Nt)}\right]\left[\log(1+t)^{P}\right]\left[1 + \frac{1}{(1+t)}\right] t^{\frac{1}{3}}$$

In the foregoing expression, the variables N and P serve as shaping parameters, and depend upon the type of environment where the corrosion loss is being calculated. The variable t in the equation is measured in years.

To calibrate the corrosion loss equation, several sources of data on corrosion in atmospheric (marine and non-marine) conditions were used to supply corrosion loss data (Fink, Boyd, 1970; Shreir, 1994). Most of the data available is for a limited number of metals, therefore the effort of calibration was focused around the type of metals on which there is considerable information. These metals include iron, mild steel or carbon steel, low alloy steels, stainless steels, and nickel iron alloys. For the corrosion loss data, the foregoing expression was applied, and a fit of the curve for the values provided was accomplished to produce best estimate (unbiased) results.

For the corrosion rate data, the same approach was used as with the corrosion loss, but first the equation for the corrosion rate was calculated. The corrosion rate equation is the derivative of the corrosion loss equation, and takes the following form:

$$CorrosionRate = \left[1 + e^{(1-Nt)}\right]\left[\log(1+t)^{P}\right]\left[1 + \frac{1}{(1+t)}\right] t^{\frac{1}{3}} \left\{\frac{1}{3t} - \frac{1}{(1+t)^2 + (1+t)} + \frac{\log(e)}{\left[\log(1+t)\right]\left[1+t\right]} - N\frac{e^{(1-Nt)}}{1 + e^{(1-Nt)}}\right\}$$

Results for the various mean values of P and N are tabulated in Table 8.5. Note the very large variabilities associated with the corrosion parameters.

Table 8.5 – Corrosion Rate – Loss Equation Parameter Values

	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 of P	32%	103%	81%	67%
COV of N	94%	124%	75%	~

With increasing values of P, the corrosion loss or rate increases. The value of N does not influence the corrosion rate or loss at large time values, therefore this parameter does not play an important part in the result of long term analysis. The value N however is important if only the short-term corrosion effects have to be calculated. Larger values of N tend to reduce the corrosion loss at the early stages of corrosion, while lower values of N result in a sharp rise in the corrosion loss. An illustration of how values of N influence corrosion loss can be seen in Figure 8.21. Figure 8.22 shows the results for the 'best estimate' values of the different types of steels.

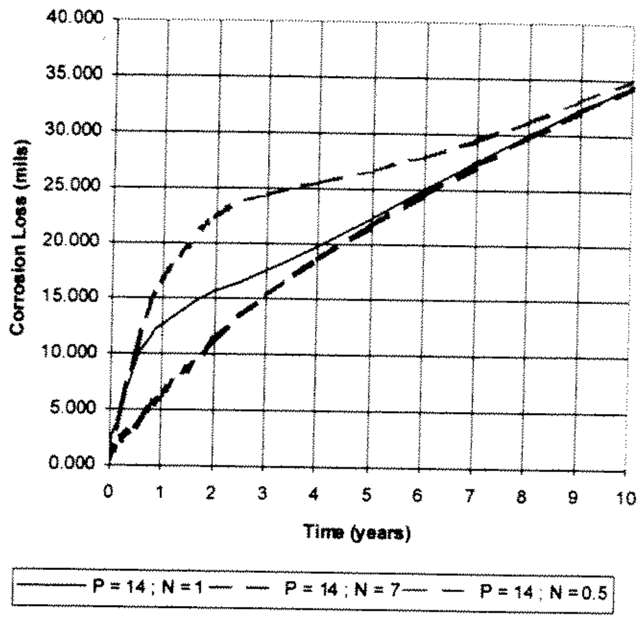

Figure 8.21 – Effect of N parameter on corrosion loss

The next step in developing the analytical model for corrosion loss was to analyze the specific environment where the model is to be applied. Since there are many specific environments where the equation can be utilized, to keep the problem reasonably simple one such environment was chosen. The specific environment chosen was that in which oil and gas are transported over long distances, and where secondary recovery techniques like pumping water into the wells are utilized.

8.6.1 Effects of Biocorrosion on P and N

Souring of the wells can be largely attributed to microbial activity, where through the aid of bacteria, hydrogen sulfide is produced (Videla, 1996). Other sulfur compounds will also be present and all these compounds react with iron or steel when contact is made. When exposed to sulfur species, iron and steel first develop a weak protective film of mackinawite (an iron sulfide rich in iron) that later changes through different chemical and electrochemical paths to more stable iron sulfides.

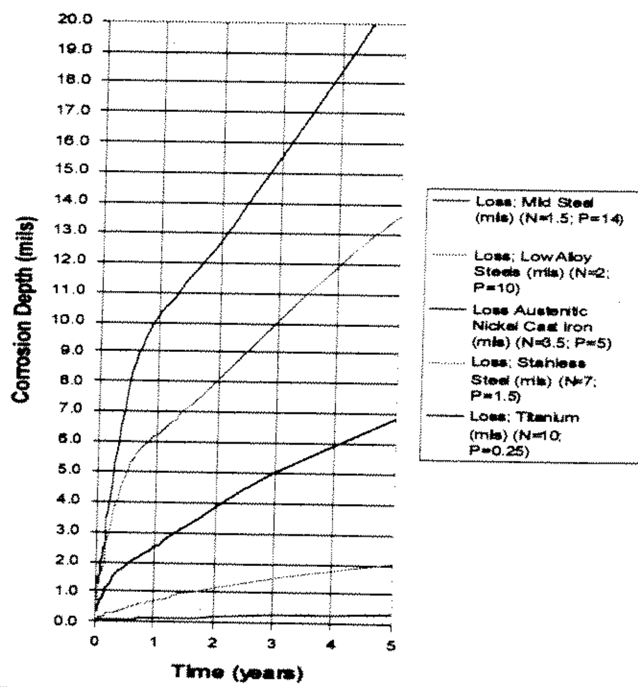

Figure 8.22 – Best estimate results for steels

In all cases iron sulfides are characterized by their marked cathodic effects on the hydrogen reduction reaction, which leads to an increase in the corrosion rate. In many cases the biocorrosion process is related to the passivity breakdown by metabolic products having aggressive characteristics which are introduced into the medium by the activity of sulfate reducing bacteria (SRB). Also, other anions able to facilitate localized corrosion are frequently present in the environment, such as the widely distributed chlorides that enhance the aggressiveness of sulfur compounds.

Biocorrosion attack can be attributed to the capacity of the bacteria to uptake hydrogen by the means of their enzymatic systems (hydrogenase), which in turn produces ferrous sulfide and ferrous hydroxide, corrosion byproducts (Videla, 1996). It has been observed that the settlement of a bacterial film on a carbon steel surface previously coated with an iron sulfide film can diminish the spalling of this film, but cannot avoid the localized corrosion hazard. Usually corrosion affects areas where there are defects in the iron sulfide film or metal matrix. Hence, the role of environmental conditions are very important in determining the chemical structure and physical form of the iron sulfides that, in turn, condition the rate and extent of the corrosion.

The rate of corrosion is also affected by the presence of oxygen, therefore the less oxygen present in the system the better are the chances of the metal not corroding. As a biofilm attaches to the surface of the metal, with time it grows and after a certain time period it becomes thick enough to prevent the efficient diffusion of oxygen to the metal-biofilm interface. When this occurs, at the bottom of the biofilm there are strictly anaerobic bacteria. The bacterial deposits therefore create a differential availability of oxygen at the metal surface. Note however, that sulfate can also act as a terminal electron acceptor, instead of oxygen, so eliminating oxygen from the system might not necessarily stop the corrosion process.

On a microscopic scale, a metal is rarely uniform and each grain will have slightly different surface characteristics and oxygen availability from its neighbors. At any time, some of the grains will be acting as anodes while others will be acting as cathodes. Later, the conditions may be reversed, and these constantly changing anodic and cathodic sites explain why a metal shows uniform rusting over its entire surface. In the case of bicorrosion however, the area under the biofilm has no access to oxygen, therefore it becomes the anode. It is evident therefore that sulfate reducing bacteria act on corrosion in an indirect way, due to their ability to produce hydrogen sulfide that could be used as a cathodic reactant (removes electrons from metal). This in turn determines whether an area on a metal surface will be anodic or cathodic.

Sulfate reducing bacteria (SRB) are prokaryotic microorganisms, which means that they lack a definite nucleus, and reproduce through binary fission. These bacteria are also heterotrophes, therefore an external source of carbon is required for their growth. Some recent studies have suggested that there is a wide range of carbon sources that these bacteria can use for their growth. Several species are able to use acetate as the sole carbon source, and in the case of marine SRB, the limiting factor for growth is not the sulfate ion but the concentration of the carbon source available in the seawater.

The pH range that is optimal for the different SRB varies between a value of 0.5 to 9. The temperature range also varies from a low of 25° C (77° F) to a high of 70° C (158° F). All SRB can be found in the marine environment, and can be responsible for the souring of oil wells, or the pitting of steel. This is particularily true when untreated sea water is used for water flooding – secondary recovery processes.

The corrosion of pipelines therefore is dependent upon what type of bacteria is present in the system. According to a study performed on the producing wells of 24 oil fields (Bernard, Connan, 1992) it was concluded that as the temperature and the salinity of a well increases, the bacterial count in the well decreases. In Figure 8.23 a plot of bacterial count versus the temperature of each well from the study is summarized.

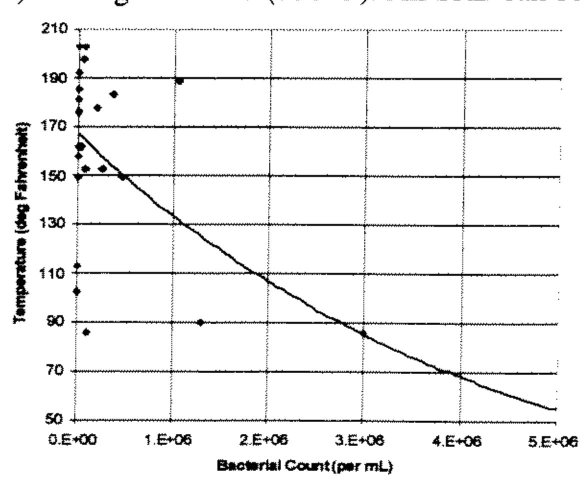

Figure 8.23 – SRB count as a function of well temperatures

The conclusion from the study is it is more likely for a low temperature wells to be sour, due to the fact that they provide a more suitable environment for bacterial growth. There are two hypotheses as to how a well can become sour. The first hypothesis states that as water is pumped into the oil wells during secondary recovery techniques, the indigenous bacteria present in the well are provided with nutrients, which in turn stimulates them to grow. The second hypothesis states that since ocean water contains many types of bacteria, these bacteria when introduced into the oil well, use the nutrients in the well and flourish.

Oil wells often contain connate water that was trapped during the geological formation of the wells, and many times the water supports indigenous bacteria (Bernard, Connan, 1992). When the connate water in oil wells are sampled, new species of bacteria are always found, especially in the lower temperature oil wells. This implies that life in the wells is able to flourish, therefore when water is pumped in from the ocean, the sulfate reducing bacteria in the water are able to flourish unimpeded.

Table 8.6, lists temperature ranges corresponding to possible localized pH ranges at the surface of the metal. At the lower temperatures, the possible pH range has lower values, while at the higher temperatures, the pH ranges are near neutral. The explanation for lower temperature ranges having lower possible pH ranges is that sulfate reducing bacteria are more likely to survive at lower temperature. Therefore the more species that survive, the more likely it is that hydrogen sulfide will be produced, and the pH will be lower.

Table 8.6 – Localized pH ranges on metal surface for various well – pipeline temperatures

Temperature Range of Well - °C (°F)	Possible pH Range
30 – 50 (86 - 122)	0.5 – 5.0+
50 – 70 (122 – 158)	2.0 – 6.0+
70 – 90 (158 – 194)	4.0 – 7.0+
90 – 110 (194 – 230)	5.0 – 8.0
110 – 140 (230 – 284)	7.0 – 9.0

It is important to note however that for localized pH values to be on the order of 1 and 2, there has to be a biofilm present on the surface of the metal, under which sulfate reducing bacteria are active. Due to the effect of shear stress at the wall of the pipe this might not be possible along certain sections of the pipe, therefore pH ranges at these pipe sections would have to be adjusted.

8.6.2 Effect of pH on P and N

As the pH of a solution decreases, the corrosion rate tends to increase exponentially (Shrier, 1994). Since P affects the corrosion process directly the following relationship was developed: Corrosion Loss is directly proportional to P. The rule for N is the opposite, where with increasing pH, N decreases. The following equations express the effects of pH on the corrosion equation loss parameters N and P. Figure 8.24 summarizes the effects of pH on the corrosion loss parameters N and P.

$$\text{Exponent}_N = \left(\frac{2.80}{pH^n}\right)^{-1} \qquad \text{Exponent}_P = \frac{2.80}{pH^n}$$

8.6.3 Effect of Flow Regime on P and N

Flow in a multiphase carrying pipe can be difficult to classify, due to several reasons. One reason is that there are at least three major types of fluids present in the pipeline. A multiphase pipeline may carry a certain percentage of oil, gas and water, each of which has a different viscosity, density, and therefore tends to move with a different velocity in the pipe. The rate of the corrosion in the pipeline is directly related however to the velocity of the media within the pipeline.

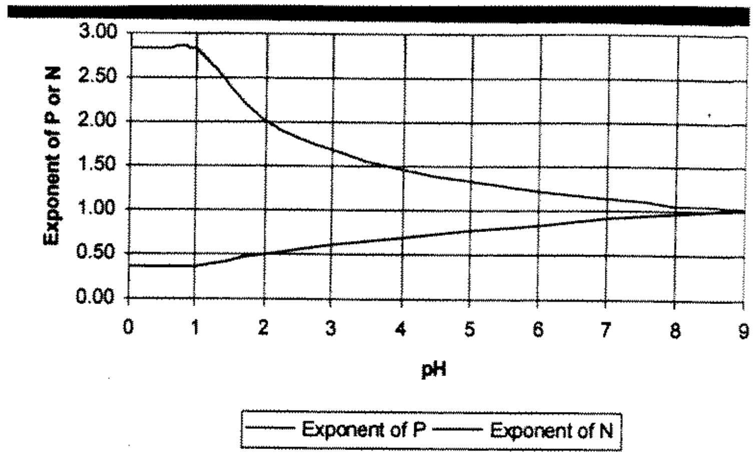

Figure 8.24 – Effects of pH on corrosion loss parameters

The corrosion processes in oil and gas production pipelines involve the interaction between metal wall and the flowing fluids. Relative motion between fluid and the metal surface will in general affect the rate of the corrosion. Three theories have been proposed as to how flow affects corrosion. The three ways in which flow can affect corrosion rate are, through convective mass transfer, phase transport, and erosion. For convective mass transfer controlled corrosion, the corrosion rate is affected by either the convective transport of corrosive material to the metal surface or the rate of dissolved corrosion products away from the surface. The phase transport corrosion depends on the wetting of the metal surface by the phase containing corrosive material. The phase distribution is strongly affected by the multiphase flow. Erosion corrosion occurs when high velocity, high turbulence fluid flow and/or flow of abrasive material prevents the formation of a protective film, allowing fresh material to be continuously exposed to the corrosive environment (Wilkins, Jepson, 1996).

Multiphase flow conditions in oil and gas pipelines are also important factors influencing the corrosion and the inhibitor effectiveness. A strong relationship has been found between field measurement of corrosion rate and flow regime (Sun, Jepson, 1992). At low liquid and gas flow rates, the three phases flow in a smooth stratified pattern. As the gas flow rate is increased, the interface between the oil and gas becomes wavy. If the liquid flows are increased, plug flow is reached.

In three-phase plug flow, the oil/water interface remains stratified while intermittent gas pockets remove the oil from the top of the pipe. If the gas flow rate is increased from plug flow, slug flow regime is reached. Characteristics of this slug flow include mixing of the oil and water layers, gas pockets of increased length, and gas bubble entrainment in the front of the slug, commonly referred to as the mixing zone. An additional increase in the gas velocity creates a flow pattern termed pseudo slug flow. Pseudo slugs have the same characteristics as slugs, but the mixing zone extends through the slug length allowing occasional gas blow through to occur. At even higher gas flow rates, annular flow is reached. Annular flow exists when the less dense fluid, the gas, flows in a core along the center of the pipe, while the more dense fluid, the oil/water mixture, flows as an annular ring around the pipe wall.

A study of multiphase flow in high-pressure horizontal and +5 degree inclined pipelines resulted in the following conclusions (Wilkins, Jepson, 1996; Jepson, et al, 1997):

- The slug frequency increases with increasing liquid flow rate, regardless of liquid composition, inclination and pressure.
- The slug frequency was not variant with pressure.
- Increasing the pressure has no effect upon the stratified/intermittent boundary.
- Increasing the pressure causes pseudo-slug flow to dominate the slug flow regime.
- Increasing the inclination forces the stratified/intermittent boundary to occur at lower liquid flow rates.
- The wall shear stress changes substantially across the front of the slug. The greatest changes occur at high Froude numbers.
- The wall shear stress is always greatest at the bottom of the pipe and decreases towards the top.
- Both the wall shear stress and turbulent intensity increase with an increase in Froude number.
- Adding the oil phase into the flow system increases the wall shear stress but decreases the turbulent intensity.

According to these conclusions, several hypotheses can be developed. One is that near the well, the velocities in the pipe are large and there is a high probability that there is a lot of turbulence, and also that the shear stress is high. As the flow is examined further down the pipeline, due to head loss in the pipe, the flow velocity decreases due to friction losses. Therefore the second hypothesis states that as the velocity in the pipe decreases the flow regime shifts away from slug flow to plug flow or to stratified flow. The conclusions then are that near the well it is more likely that erosion corrosion along with convective mass transfer corrosion are controlling, but due to the high turbulence bacterial colonies are not able to attach themselves to the pipe walls. As the flow regime changes down the line however, water separates from the oil and the flow becomes stratified. This enables the bacteria to find suitable conditions to thrive and the water at the bottom of the pipeline is where bacterial colonies tend to be found, which also explains why internal corrosion is predominantly found along the bottom of pipelines.

Based on these results, the following modification to the corrosion loss equation parameter P was developed:

$$MultiplicationFactorForP = \left(1.05 - \frac{PercentHeadLoss_{OverTotalLength}}{100}\right)\frac{PercentLength_{OfTotalLength}}{100} + \left(\frac{PercentHeadLoss_{OverTotalLength}}{100} + 0.20\right)$$

Based on this expression, the corrosion rate will depend upon how much head loss there is in the pipeline. The head loss is taken to be uniform over the length of the pipeline for. The multiplication factor for P depends upon which point along the line is being examined, and reaches a maximum value of 1.05 at the end of the pipeline. At the start of the pipeline, the multiplication factor is equal to 0.20 plus the head loss over the total length of the pipeline. The multiplication factor for N on the other hand can be ignored,

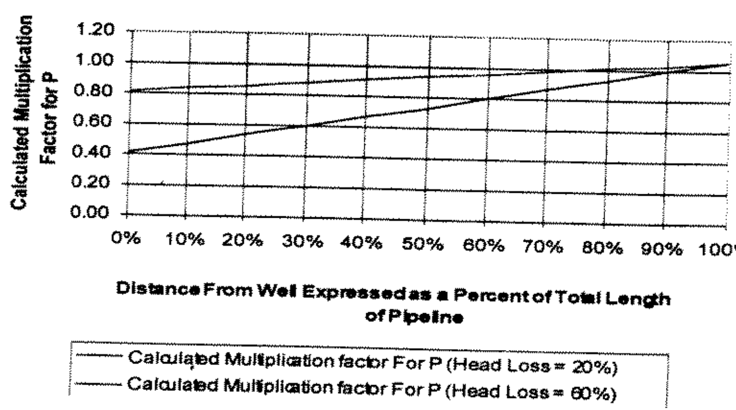

Figure 8.25 – Effects of head loss in pipelineon flow regime and on corrosion loss parameter P

because N does not have a significant role in the corrosion loss. Figure 8.25 illustrates the change in the multiplication factor for different values of head loss.

To use Figure 8.25, the head loss over the length of the pipeline must be known and the user must decide where the corrosion loss in the pipe is to be calculated: at 50% of the total length or at 75% of the total length. If the pipeline is divided into sections for analysis, then the average distance of that section from the well can be used to obtain a value from the Figure 8.25.

8.7 Corrosion – Time Dependent Reliability

Pipeline reliability is a time dependent function that is dependent on the corroded thickness of the pipeline ($t_{ci/e}$). The corroded thickness is dependent on the average rate of corrosion and the time that the pipeline or riser is exposed to corrosion. This time dependency can be clarified with the following:

$$\beta = \ln (Kp\, t - Kp\, t_{ci/e})) / \sigma_{lnp/R}$$

where:

$$Kp = (2.2\ SMTS / D\ SCF\ Po)$$

If one defines:

$$Kp\, t = FS_{50}$$

where FS_{50} is the median factor of safety in the burst capacity of the pipeline or riser. Then:

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

As the pipeline corrodes, the reduction in the pipeline or riser wall thickness leads to a reduction in the median factor of safety that in turn leads to a reduction in the Safety Index (or an increase in the probability of failure). In addition, as the pipeline corrodes, there is an increase in the total uncertainty due to the additional 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 uncertainty as a function of the corrosion could be expressed as:

$$\sigma_{lnp/R}|t = \sigma_{lnp/R}|to\ (1 - t_{ci/e} / t)^{-1}$$

where $\sigma_{lnp/R}|t$ is the uncertainty at any given time 't', $\sigma_{lnp/R}|to$ is the uncertainty at time t = 0, $t_{ci/e}$ is the corroded thickness and t is the initial thickness. When $t_{ci/e} / t = 0.5$ the initial uncertainty would be increased by a factor of 2.

Results for $\sigma_{lnp/R}|to = 0.2$ and $= 0.30$ and $FS_{50} = 2.0$ (same as median bias used previously) are summarized in Figure 8.26.

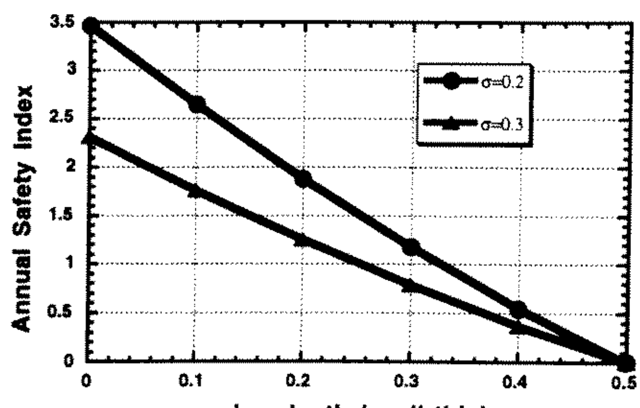

Figure 8.26 - Influence of Corrosion Depth and Uncertainty on Annual Safety Index

High quality assurance and control in the pipeline reliability management leads to lower uncertainty and higher reliability. Given corrosion, there is a decrease in the reliability of the pipeline as a function of time reflected in the depth of the corrosion normalized by the wall thickness. If the target reliabilities are defined as those that the pipeline should not be lower than during its life (Figure 8.27), then either corrosion protection must be provided to preserve the initial thickness of the pipeline or riser, or corrosion allowance must be added to the pipeline or riser initial thickness, or a combination of these two measures. For example, if an annual Safety Index of 2 during the pipeline life were desired, and the initial uncertainty associated with the pipeline demands and capacity were 20%, then the corrosion allowance would need to be 20% of the pipeline thickness. This would result in an initial annual Safety Index of 3.5. Given the projected corrosion rate for the life time of the pipeline or riser, the annual Safety Index would decrease to 2.0 by the end of the projected life.

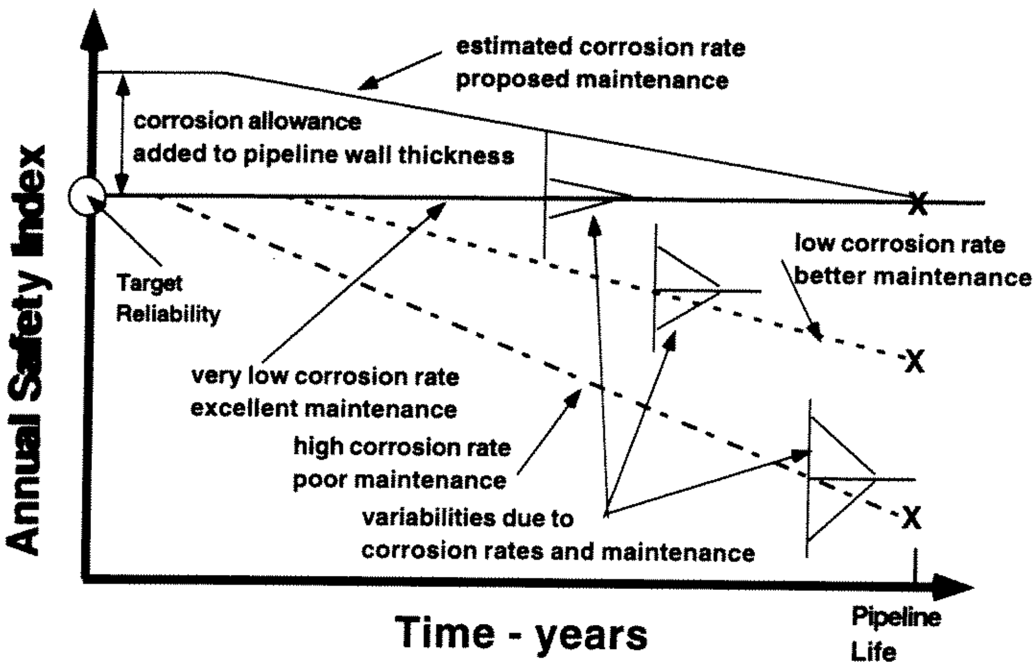

Figure 8.27 – Corrosion allowance added to pipeline wall thickness

Additional insight into the change in the uncertainty associated with the pipeline capacity associated with the loss of wall thickness due to corrosion, can be developed by the following:

$$\bar{t'} = \bar{t} - \bar{d}$$

t' is the wall thickness after the corrosion, t is the wall thickness before corrosion, and d is the maximum depth of the corrosion loss. Bars over the variables indicate mean values.

Based on First Order – Second Moment methods, the standard deviation of the wall thickness after corrosion can be expressed as:

$$\sigma_{t'} = \sqrt{\sigma_t^2 + \sigma_d^2}$$

The Coefficient of Variation (COV = V) can be expressed as:

$$V_{t'} = \frac{\sigma_{t'}}{\bar{t'}} = \frac{\sqrt{(V_t \bar{t})^2 + (V_d \bar{d})^2}}{\bar{t} - \bar{d}}$$

A representative value for the COV of t would be 2 %. A representative value for the COV of d would be Vd = 0.40. Figure 8.28 summarizes the foregoing developments for a 16-inch diameter pipeline with an initial wall thickness of Vt = 0.5 inches that has an average rate of corrosion of 10 mpy (0.010 inches per year). The dashed line shows the results for the uncertainties associated with the wall thickness. The solid line shows the results for the uncertainties that include those of the wall thickness, the prediction of the corrosion burst pressure, and the variabilities in the maximum operating pressure.

At the time of installation, the pipeline wall thickness is equal to 2 %. But, as time develops, the uncertainties associated with the wall thickness increase due to the large uncertainties associated with the corrosion rate – maximum depth of corrosion. The solid line that reflects all of the uncertainties converges with the dashed line that represents the uncertainties in the remaining wall thickness, until at a time of about 20 years, the total uncertainty is about the same as that of the remaining wall thickness (Vt-d ≈ 25 %). As more time develops, there is a dramatic increase in the remaining wall thickness and total uncertainties. These uncertainties are dominated by the uncertainties attributed to the corrosion processes.

Figure 8.29 portrays these results in terms of the corrosion loss divided by the initial nominal wall thickness. The trends and observations are the same as those developed earlier. Note at a wall thicknes of about 80 %, the uncertainties are approximately 100 %, and the probabilities of failure could be expected to be very high (near unity). This is an interesting observation because the 80 % loss in wall thickness is frequently used as the cut-off point for pipeline operations.

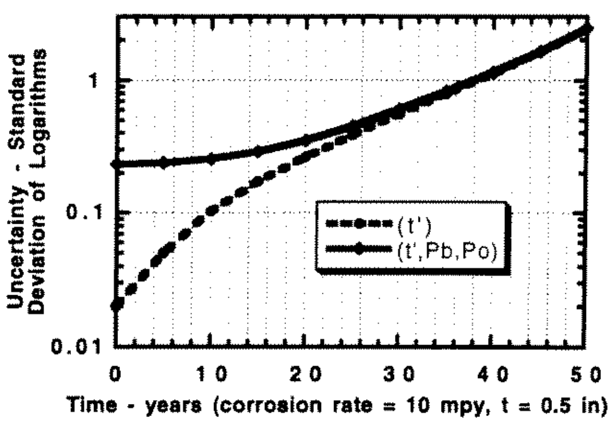

Figure 8.28 – Uncertainty in pipeline wall thickness and burst pressure capacity as a function of the pipeline exposure

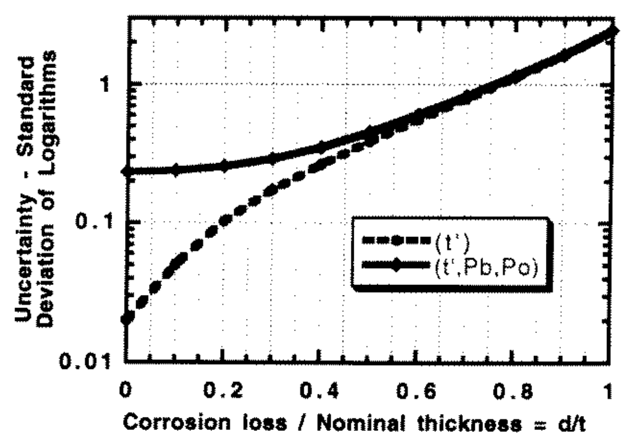

Figure 8.29 – Uncertainty in pipeline wall thickness and burst pressure capacity as a function of the normalized loss in pipeline wall thickness.

These observations have extremely important rammifications on the probabilities of failure – loss of containment of the pipeline. After the 'life' of the pipeline is exceeded (e.g. 20 to 25 years), one can expect there to be a rapid and dramatic increase in the uncertainties associated with the corrosion processes. In addition, there will be the continued losses in wall thickness. These two results combined will result in a dramatic and rapid increase in the probability of failure of a pipeline. Figure 8.30 summarizes example results for a 16-inch diameter, 0.5 inch wall thickness pipeline that has a maximum operating pressure of 5,000 psi. The pipeline is operated at the maximum pressure, and at 60 % of the maximum operating pressure for a life of 0 to 50 years (Figure 8.28). The average corrosion rate was taken as 10 mpy. For the 60 % pressured line, during the first 20 years, the annual probability of failure rises from 1E-7 to 5 E-3. After 20 years, the annual probability of failure rises very quickly to values in the range of 0.1 to 1. Perhaps, this is why the observed failure rates associated with corrosion in the Gulf of Mexico are in the range of 1 E-3 per year.

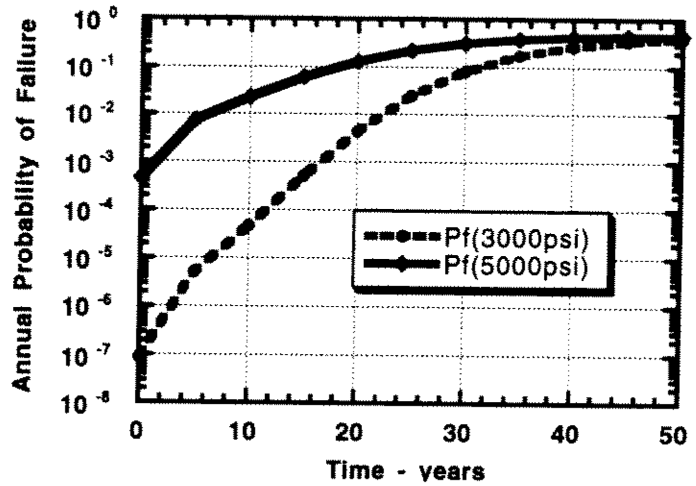

Figure 8.30 – Example pipeline failure rates as function of exposure to corrosion

Example results for a 8-inch diameter, 0.5 inch wall thickness pipeline fabricated with X42 steel, and installed in 1971 are summarized in Figure 8.30. The pipeline is an inhibited gas lift line operating at 25 % of its maximum allowable pressure. The pipeline was evaluated to have an average over life corrosion rate of 10 mpy. The probabilities of failure are negligible for about the first 25 years, then there is a rapid increase. The results indicate that the pipeline can be expected to fail in about the year 2010.

Similar example results for a 16-inch diameter, 0.5 inch wall thickness pipeline fabricated with X42 steel, and installed in 1993 are summarized in Figure 8.31. The pipeline transports oil and formation water and is operating at 10 % of its maximum allowable pressure. The pipeline was evaluated to have an average over life corrosion rate of 75 mpy. This very high corrosion rate is due to SRB in the produced fluids due to reservoir water flooding with untreated sea water. The probabilities of failure are negligible for about the first 5 years, then due to the very high corrosion rate, and the rapidly increasing uncertainties associated with the loss in wall thickness due to corrosion, the probabilities of failure increase very rapidly. The results indicate that the pipeline can be expected to fail in about the year 2000.

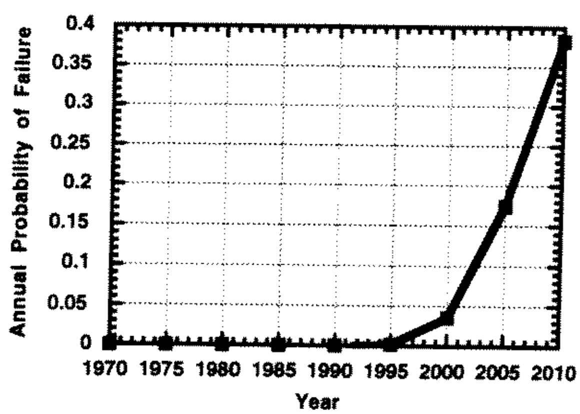

Figure 8.30 – Example probability of failure results for an 8-inch gas lift pipeline

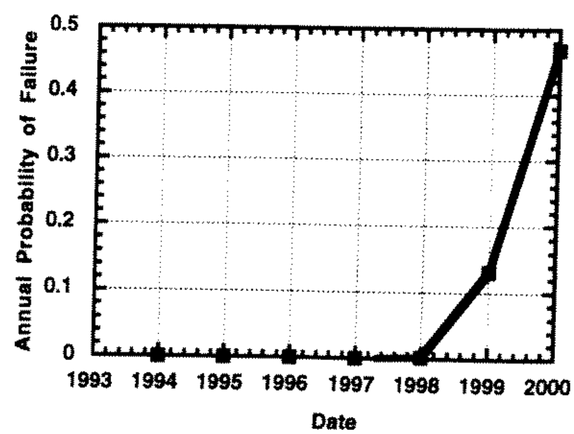

Figure 8.31 – Eample probability of failure results for a 16-inch oil-water pipeline

Another example is developed to illustrate the time dependent reliability characteristics of pipelines subjected to corrosion conditions. Figure 8.32 shows the time dependent operating pressures for a 30-inch diameter, 1-inch wall thickness, X60 pipeline that transports crude oil over a distance of 50 km (Collberg, Cramer, Bjornoy, 1996). The operating pressures decrease from a maximum of 3,000 psi (inlet) at the time of commissioning to a maximum of 2,300 psi at 20 years.

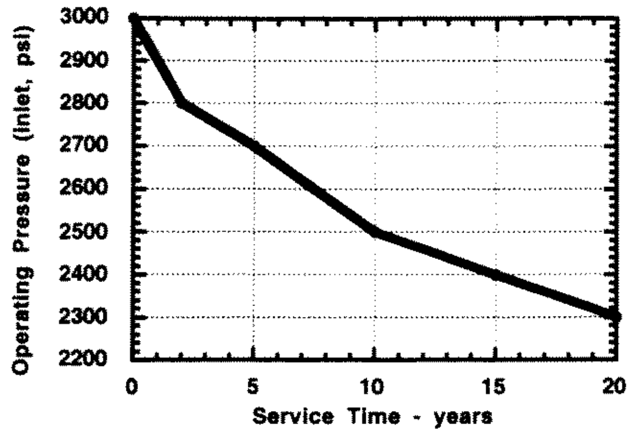

Figure 8.32 – Example pipeline service time maximum operating pressures

The corrosion in the pipeline is modeled as a time dependent process that is organized into three stages (Figure 8.33). The first stage (5 years) is when there is no significant water-cut in the oil stream. The pipeline is effectively protected by the oil wetting of the steel. The best estimate average corrosion rate during Stage 1 is estimated to be 5 mpy. The second stage (10 years) is when there is a significant (above 40 %) water cut in the oil stream. Salt water is in contact with signficant portions of the horizontal sections of the pipeline. The best estimate average corrosion rate during this stage is estimated to be 20 mpy. The third stage (5 years) is when there is significant SRB count (above 1E4) due to water flooding of the reservoir with untreated sea water. The best estimate average corrosion rate during this period is estimated to be 50 mpy.

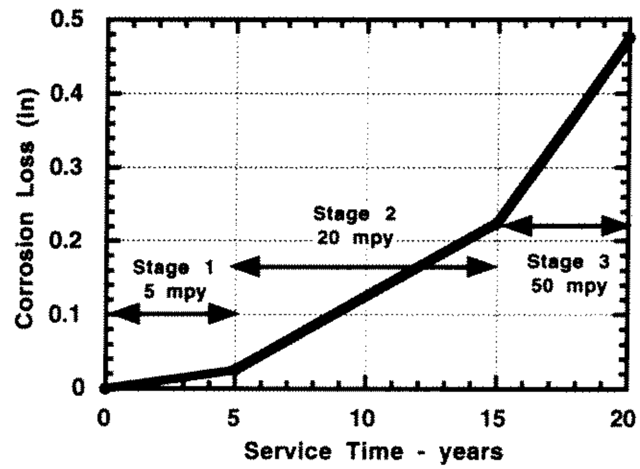

Figure 8.33 – Example pipeline loss of wall thickness due to corrosion

Figure 8.34 summarizes the result of the evaluation of the annual probabilities of loss of containment as a function of the service time. This example recognizes the time changes of the operating pressures, corrosion rates, and uncertainties associated with the corrosion losses (COVd = 40 %). In Stage 1, there is a relatively rapid rise in the probabilities of failure early in the life of the pipeline due to the increased uncertainties associated with the corrosion damage and the prediction of the burst pressure capacities of the corroded pipeline. In Stage 2, there is a leveling off of the probabilities of failure due to the compensating effects of the lowered operating pressures and the increased corrosion rate and uncertainties associated with the corrosion damage. In Stage 3, there is again a rapid increase in the probabilities of failure due to the large increase in corrosion damage associated with the SRB effects.

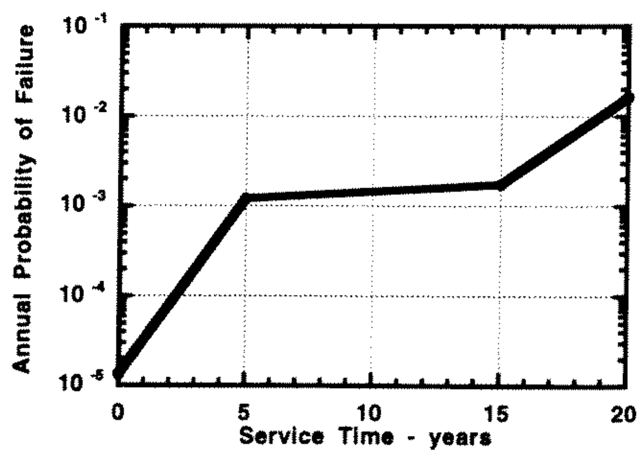

Figure 8.34 – Example pipeline service time versus annual probability of failure

8.8 Failure Data Used to Validate Analytical Models

Given that failure (loss of contaiment) data were available on a pipeline and that sufficient information was available on the characteristics of the pipeline, one could develop an assessment of the average corrosion rate that would have been active during the life of the pipeline. An estimate of the amount of wall thickness lost, d, in a given period of time, L could be estimated from:

$$\frac{d}{L} = \frac{D}{2}\left\{\left[\frac{2.6tSMTS}{Pb(D-t)} - 1\right]0.5\right\}^2 = v_{ie}$$

The 2.6 represents the result of multiplying the reassessment equation value of 2.2 times the median bias of 1.2. The pipeline diameter, wall thickness, SMTS, operating pressure at the time of the failure, Pb, and exposure period, L, would be needed to determine the average corrosion rate.

A 16-inch diameter, 0.5 inch wall thickness pipeline fabricated with X42 steel was installed in 1971. The pipeline transported gas, oil, and water until 1989 (19 years) when it lost containment due to internal corrosion (based on diver inspection report). The pipeline was being operated at a pressure of 4,000 psi at the time of the failure. Given this information, the best estimate pipeline average corrosion rate could be determined to be 25 mpy. The range of this corrosion rate could be 15 mpy to 35 mpy (Figure 6.5.7). If sufficient information could be developed on the characteristics of the product transported by the pipeline, then qualitative and quantative links could be developed between these characteristics and the average rate of corrosion for the given time period. Given a direct measurement of the depth of corrosion at the time of the failure (e.g. from measurements on the failed section of pipeline), more definitive information could be provided to help validate the corrosion condition burst pressure analytical model. In this particular example, the pipeline was repaired by clamping, and no direct measurements were taken on the corrosion losses.

Given that information as outlined above could be gathered on a long-term industry wide basis, a sufficiently large database on the performance of pipelines in the field could be developed. Such a database has been established by the MMS (Alvarado, 1998). Even more valuable data could be developed in special cases where the failed section of the pipeline were subjected to examinations and measurements of properties and characteristics. Such information would be invaluable in providing validations of the analytical models used to reassess and requalify pipelines. Such data can not reasonably be regarded as being a 'competitive advantage.' Given that the specific details of the owner / operator could be screened from the data to avoid the 'blame and shame syndrome,' it would seem to be an industry competitive economic and safety advantage to develop such a comprehensive database.

This was the original objective of the PIMPIS project (Jaio, Bea, 1995): to develop a data archiving, analysis, and communications system that would interface with the MMS pipeline database and provide information to help reassess and requalify existing pipelines. This objective is yet to be realized.

Several reasons for lack of development of such a system can be advanced. Among these are: there is insufficient industry wide support for development and maintenance of such a system. Industry support for such a system follows fluctuations in oil prices; there is no long-term commitment to developing, implementing, and maintaining such a system. Industry management has not realized that there is an economic advantage to be gained from developing, implementing, and maintaining such a system. There are fears of legal and punitive actions that could arise from such a development. The list of reasons is very long and complex but most seem to be centered in business economics.

It is important to note that none of these reasons are technological. Sufficient developments in electronic computations and communications, and sufficient engineering technology exists for such a development. Substantial strides have been made in development of components for such systems for onshore pipelines (Office of Pipeline Safety, 1997). The authors have seen similar developments started for offshore platforms and floating systems (e.g. commercial ships). But, in these parts of the marine world there does not seem to be sufficient general industry support to continue to develop and maintain these systems over long periods of time.

8.9 Burst Pressures Criteria

The RAM PIPE REQUAL based burst pressure factors non-corroded pipelines is:

$$f_{Pb} = [(Bs\ Fe\ /\ Br)\ \exp\ (\beta\ \sigma)]^{-1}$$

The median Bias in the maximum net operating pressures (internal – external) will be taken to be Bs = 1.0. The median transient loading – nonlinear response factor will be taken to be Fe = 1.0. Based on the use of the nominal ultimate tensile stress in the design formulation, the median bias in the design formulation for burst capacity would be B = 1.1. The uncertainty in the burst capacity for new (no corrosion) pipelines will be taken as σ_{Pb} = 6 %. The uncertainty in the net burst pressures (internal – external) will be taken as σ_{Pe} = 10 % (range fo 100 year conditions is 6.5 % to 9 % depending on the internal to external pressure ratio). The total uncertainty in the pipeline burst pressure demands and capacities is σ_{nPb} = 11 % (Bea, et al, 1998). Note that these uncertainties imply a very high correlation coefficient for this failure mode (ρ'_{ij} = 0.8). Thus:

$$f_{Pb} = [0.91\ \exp\ (\beta\ 0.11)]^{-1}$$

Based on the annual Safety Indices developed for design of pipelines (Lara, et al 1998), subjected to very low corrosion conditions and well maintained (dehydration, inhibition) the following RAM PIPE REQUAL based burst pressure design factors are determined for all categories of pipelines: 1) oil and gas pipelines: f_{Pb} = 0.73, and oil and gas risers: f_{Pb} = 0.71.

The comparable API RP 1111 (1998) are f_{Pb} = 0.75 x 0.80 = 0.60 for risers and are f_{Pb} = 0.90 x 0.80 = 0.72 for pipelines.

The formulation used by API (1998) to compute the burst pressures is:

$$Pb = 0.9\ (Smys + Smts)\ (t\ /\ D - t)$$

For a 24-inch diameter, 0.5 inch wall thickness X52 steel pipeline and riser, the API formulation would determine the burst pressure to be:

$$Pb = 0.9\ (52\ ksi + 66\ ksi)\ (0.5\ in\ /\ 23.5\ in) = 2,260\ psi$$

The allowable API pipeline design pressures for the example risers and pipelines would be 1,360 psi and 1,630 psi, respectively.

For the formulation developed during this project, the burst pressure would be:

$$Pb = 2\ (66\ ksi)\ (0.5\ in\ /\ 23.5\ in) = 2,810\ psi$$

The allowable RAM PIPE pipeline design pressures for the pipeline and risers would be 2,050 psi and 2,000 psi, respectively.

The DNV96 guidelines specify useage factors for pressure containment that are based on the SMTS and are 0.72, 0.67, and 0.64 for Low, Normal, and High Safety Classes. These useage factors would result in design pressures of 2,020 psi, 1,880 psi, and 1,800 psi, respectively.

The foregoing design factors do not include provision for significant corrosion. They presume a non-corrosive environment in which the maintenance (corrosion prevention) is 'perfect.'

Depending on the program of corrosion prevention and pipeline inspection, maintenance, and repair, a corrosion allowance should be developed. Given that the program will not prevent significant corrosion from developing in a given pipeline, criteria must be developed to determine the amount of corrosion that can be expected and how this corrosion will affect the burst pressure reliability characteristics.

For un-instrumented (not smart piggable) pipelines, it is suggested that the 'best estimate' (expected value) amount of corrosion be deducted from the design wall thickness (nominal thickness plus original corrosion allowance) to define a minimum wall thickness that is then utilized in the design formulation. The estimated loss in wall thickness due to corrosion could be based on information from databases on corrosion in comparable pipelines, from databases developed from instrumented pipelines in which corrosion was measured, and from corrosion analytical models.

The uncertainty introduced into the burst pressure analytical model by corrosion can be estimated to be 23 %. Thus, a reasonable estimate of the total (burst pressures and burst pressure capacities) uncertainty associated with the burst capacity of a moderately corroded pipeline would be 25 %. In these guidelines, an uncertainty in the corroded pipeline burst demand and capacity of $\sigma = 25\ \%$ was used (based on the premise that the expected corrosion loss is subtracted from the original wall thickness to define t and the use of the specified ultimate tensile strength). Thus:

$$f_{Pb} = [(1/1.2) \exp (\beta\ 0.25)]^{-1}$$

Based on the annual Safety Indices developed for design of pipelines subjected to moderate corrosive conditions (Lara, et al 1998), the following burst pressure design factors are developed for all categories of pipelines: 1) oil and gas pipelines: $f_{Pb} = 0.49$, and 2) oil and gas risers: $f_{Pb} = 0.44$.

The wall thickness that should be used with these factors is the nominal wall thickness without the corrosion allowance.

Returning to the 24-inch pipeline and riser developed earlier, based on the RAM PIPE formulation the burst pressure was computed to be 2,810 psi. The allowable RAM PIPE pipeline design pressures for the pipeline and risers would be 1,380 psi and 1,240 psi, respectively.

The allowable API pipeline design pressures for the example pipelines and risers would be 1,630 psi and 1,360 psi, respectively. The DNV96 design pressures for Low, Normal, and High Safety classes would be 2,020 psi, 1,880 psi, and 1,800 psi, respectively.

The RAM PIPE criteria for corroded pipelines result in more conservative allowable pressures for the pipelines and risers than either API (1998) or DNV (1996).

9.0 Propagating Buckling

9.1 RAM PIPE REQUAL Formulation

For design of new pipelines and reassessment of existing pipelines, the RAM based formulation for the working stress based design propagating pressure factor can be expressed as:

$$f = [(Bs\ Fe / Br) \exp (\beta\ \sigma)]^{-1}$$

Bs is the median bias in the pipeline demands (pressures), Br is the median bias in the pipeline capacities (propagating pressures), Fe is a transient loading – nonlinear response factor (close to 1.0 for propagating buckling pressure conditions), β is the designated Safety Index, and σ is the total uncertainty in the demands and capacities.

9.2 Propagating Pressure Characteristics

The propagating buckling condition an accidental limit state. An accident is required to produce a collapsed section in the pipeline that can be propagated by the effective collapse pressures. The accidental limit state assessment will be based on 10-year return period conditions.

As discussed in Section 6.1.1 of this report, the median bias in the collapse pressures for installation and operating accidental conditions (10-year storm) for oil pipelines is $B_{Pe50} = 0.86$. The uncertainties in the total external pressures and internal pressures is σ_{lnPe} = 10 %. For gas pipelines $B_{Pe50} = 0.98$. The uncertainties in the total external pressures and internal pressures is σ_{lnPe} = 2 %.

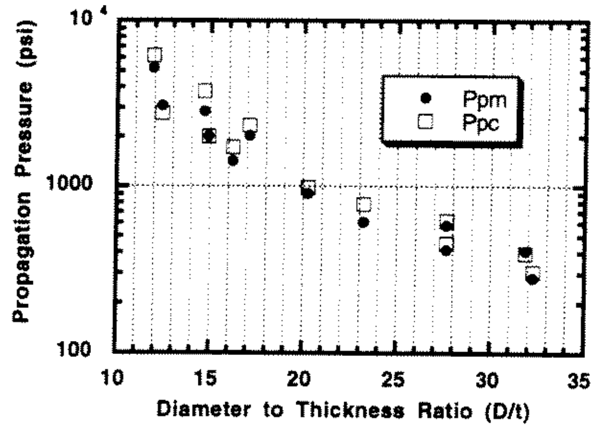

Figure 9.1 – Measured and calculated propagation pressures for X42 and X65 piplines

9.3 Propagating Buckling Capacities

Figure 9.1 summarizes results from 12 tests of small scale (4-inch diameter) pipelines fabricated from X-42 and X-65 steel that defines the measured and calculated propagation pressures as a function of the diameter to thickness ratio. There is a linear relationship between the propagation pressure and the diameter to thickness ratio. There is good agreement between the calculated (based on the design formulation) and measured propagation pressures.

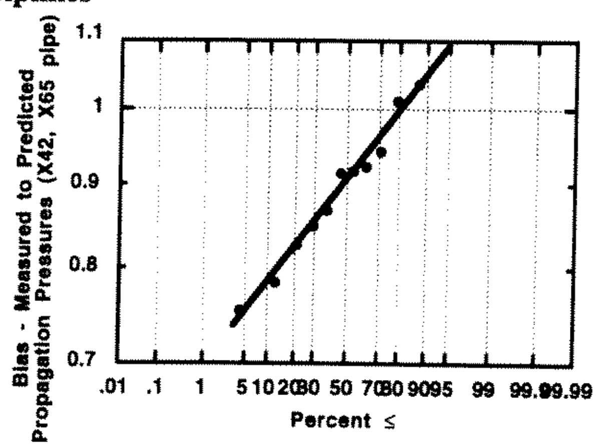

Figure 9.2 – Bias measured / calculated propagation pressures for X42 and X65 pipelines

Figure 9.2 summarizes the statistical analysis of the bias in the calculated propagation pressures. The median bias is $B_{50} = 0.92$ and the Coefficient of Variation of the Bias is $V_B = 12$ %.

The design and reassessment formulation was expressed as:

$$Pp / Sym = B_{Sy}\, 34\, (t/D)^{2.5}$$

where Sym is the specified minimum yield strength, and B_{Sy} is the median Bias introduced by using the specified minimum yield strength and the Mesloh, et al formulation. Based on the API specification based data cited earlier (B_{Sy} = 1.1 x 0.92 = 1.01), the design and reassessment analysis propagating pressure formula becomes:

$$Pp / Sym = 34\, (t/D)^{2.5}$$

The median Bias associated with this formulation is B_{50} = 1.0 and the coefficient of variation of this bias is V_B = 12 %.

9.4 Effects of Concrete Coatings

Figure 9.3 summarizes the test data on the effects of concrete weight coating on the collapse and propagating pressures as a function of the thickness of the weight coating to the steel thickness (Langner 1974; Mesloh et al 1976). The pipelines tested had diameter to thickness ratios in the range of 51 to 111.

The concrete coating has the effect of increasing both the initiating or collapse pressure and the propagating pressure by substantial amounts. For a thickness ratio of 10, both the collapse and propagating pressures are increased by a factor of 2. As the thickness of the concrete coating relative to the pipeline wall thickness increases, there is a continued increase in the initiating and propagating pressures.

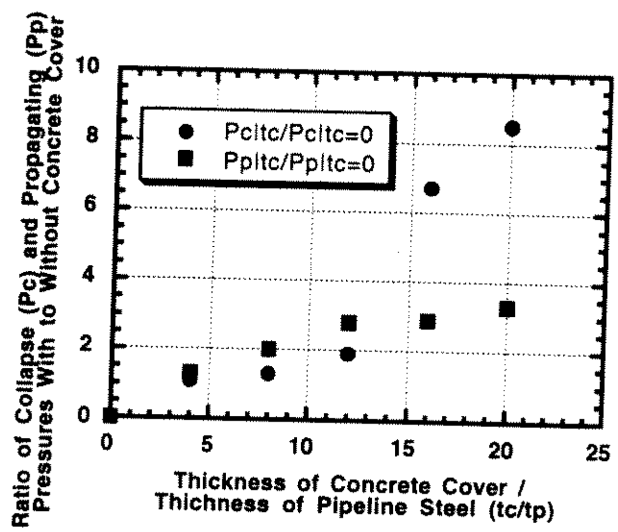

Figure 9.3 – Effect of concrete cover on collapse pressures and propagating pressures

9.5 In-Place Propagating Pressure Criteria

For accidental operating conditions for fluid filled pipelines, the resulting total uncertainty in the demand and capacity variables is $\sigma = (0.10^2 + 0.12^2)^{0.5}$ = 16 %. Given these developments, the WSD propagating buckling design factor, fpp, can be computed from:

$$fpb = [(0.86)\exp (2 \times 0.16)]^{-1} = 0.84$$

For installation and operating accidental limit state conditions evaluated with 10-year return period conditions, β = 2, and fpb = 0.80.

It will be recalled that the foregoing analysis was based on fluid filled pipeline conditions. If gas filled pipeline conditions were used (internal pressure at o spheric), the propagating buckling design factor can be computed from:

$$fpb = [0.98 \exp (2 \times 0.12)]^{-1} = 0.80$$

The gas filled pipeline conditions would control the propagating buckling design factor.

The LRFD propagating pressure design loading factor, γpb, can be computed from:

$$\gamma pb = 0.98 \exp (0.75 \times 2 \times 0.02) = 1.0$$

The LRFD propagating pressure capacity factor, ϕpb, can be computed from:

$$\phi pb = 1.0 \exp (-0.75 \times 2 \times 0.12) = 0.84$$

The formulation can be verified as follows:

$$fpb = 0.80 \text{ and } \phi pb / \gamma pb = 0.84 / 1.0 = 0.84$$

The error is due to the approximation involved in the splitting factor of 0.75.

9.6 Example Application

The design formulation for not needing buckle arrestors is:

$$fpb\, Pp \leq Pe - Pi$$

or

$$fpb\, Pp = Pe - Pi$$

or

$$Pp = (Pe - Pi) / f$$

The value included in the proposed API 1111 (1998) guidelines is f = 0.8. However, the API 1111 formulation to determine the buckling pressure is specified as:

$$Pp / Sym = fpb\ 24\ (t/D)^{2.4}$$

From this study the buckling pressure is specified as:

$$Pp / Sym = fpb\ 34\ (t/D)^{2.5}$$

For D/t = 40, the API formulation indicates Pp / Sym = 3.4 E-3 and including fpb = 0.8 indicate indicates Pp / Sym = 2.7 E-3. For 55 ksi nominal minimum yield strength steel, Ppb = 150 psi.

For the same D/t, and for the design 10-year installation conditions, the formulation developed during this study indicates Pp / Sym = 3.4 E-3. Including fpb = 0.84 and fpb = 0.80 indicates Pp / Sym = 2.8 E-3 and 2.7 E-3. For 55 ksi nominal minimum yield strength steel, Ppb = 155 psi and Pbp = 150 psi.

Based on the information developed in Section 7, for installation conditions, the design pressure for fluid filled pipelines is 1475 psf or 10.2 psi. The pressure required to propagate a dent in the fluid filled pipelines would be 155 psi. Propagating buckling would not be a problem for these pipelines.

Based on the information developed in Section 7, for installation conditions, the design pressure for gas filled pipelines is 10,254 psf or 71.2 psi. The pressure required to propagate a dent in the gas filled pipeline would be 150 psi. Propagating buckling would not be a problem for these pipelines.

Buckle arrestors for installation conditions would not be required by either API (1998) or the criteria developed during this study.

The annual Safety Index implied by the API guidelines and the information developed during this study could be computed to be:

$$\beta = -\ln (f / 0.98) / \sigma = -\ln (0.8 / 0.98) / 0.12 = 1.7$$

This value is very close to the annual Safety Index of $\beta = 2.0$ adopted for these criteria.

10.0 Burst Pressure Capacities of Dented – Gouged Pipelines

Figure 10.1 summarizes results from analysis of tests performed by British Gas Corporation (Jones, 1981) on 30-inch diameter X52 pipelines. The pipelines were first dented. The dent depths (H) to diameter ratios were in the range H/D = 1.0 % to 3.6 %.

A defect simulating a gouge was machined into the center of the dent parallel to the axis of the pipeline. The defects had depths (h) to wall thickness ratios that were h/t = 25%.

The reassessment formulation used to analyze these data is based on stress concentration factors for the dent of:

$$SCF_H = 1 + 0.2(H/t)^3$$

The reassessment formulation used to analyze these data is based on stress concentration factors for the gouges of:

$$SCF_h = 1 + 2\ (h/R)^{0.5}$$

The resultant stress concentration factor is:

$$SCF_{H,h} = SCF_H \times SCF_h$$

The reassessment damaged pipeline burst capacity was based on:

$$Pbd = (2\ SMTS / SCF_{H,h})\ (t / D)$$

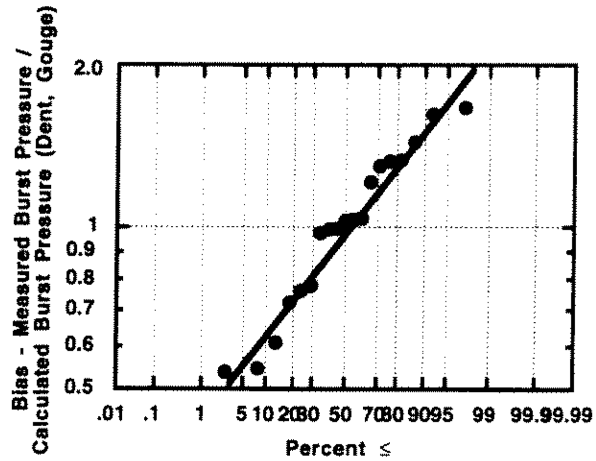

Figure 10.1 – British Gas Tests on 30-inch diameter pipelines with dents and gouges

This reassessment formulation developed a median Bias of B_{50} = 1.0 and a Coefficient of Variation of the Bias of V_B = 28 %. These values are dependent on accurately known dent depths and gouge depths. The assessment of capacity of the dented – gouged pipelines needs to reflect the uncertainties associated with determination of the dent depths and gouge depths. In addition, the test data analyzed at this point in the project have been very limited.

Additional test data are being developed and analyzed that will likely change the variability, and perhaps change the median bias associated with the proposed formulation. Until this test data can be developed and analyzed, a median bias of B_{50} = 1.0 and a Coefficient of Variation of the Bias of V_B = 30 % will be used in development of the reassessment criteria.

The uncertainty introduced into the burst pressure analytical model by denting and gouging damage can be estimated to be 30 %. Thus, a reasonable estimate of the total (burst pressures and burst pressure capacities) uncertainty associated with the burst capacity of a pipeline damaged by denting and gouging would be 32 %. Thus:

$$f_{Pb} = [\ \exp\ (\beta\ 0.32)]^{-1}$$

Based on the annual Safety Indices developed for reassessment of pipelines subjected to denting and gouging damage, the following burst pressure design factors are developed for all categories of pipelines: 1) oil and gas pipelines: f_{Pb} = 0.32, and 2) oil and gas risers: f_{Pb} = 0.28.

Returning to the 24-inch pipeline and riser developed earlier, assume that the pipeline was found to have a dent whose depth was 0.5 inch and in the center of this dent was a gouge that was 0.1 inch deep. The dent SCF would be:

$$SCF_H = 1 + 0.2(H/t)^3 = 1 + 0.2(0.5/24)^3 = 1.0$$

The gouge SCF would be:

$$SCF_h = 1 + 2\ (h/R)^{0.5} = 1 + 2\ (0.1/0.5)^{0.5} = 1.89$$

As indicated by this formulation, the dent has no important influence on the burst capacity. This observation is substantiated by test data on dented pipelines, given that the dent or process of denting does not cause cracks to develop in the wall of the pipeline (Jones, 1990). However, the gouge (crack) has an important influence on the burst capacity.

Based on the RAM PIPE REQUAL formulation for undamaged pipelines, the burst pressure for the 20-inch diameter, 0.5-inch wall thickness pipeline and riser was computed to be 2,810 psi. The RAM PIPE reassessment burst pressure for the damaged pipeline and riser would be 2,810 psi / 1.89 = 1,490 psi. The RAM PIPE allowable pressures for pipelines and risers would be 480 psi and 420 psi, respectively.

11.0 Collapse Pressure Capacities

11.1 Pipeline Collapse Conditions

The collapse pressure failure mode represents the condition when the pipeline external pressure exceeds the internal pressure at the point of failure. The net collapse pressure (internal minus external pressures) will be indicated as Pc.

During the pipeline operation, the normal minimum internal pressure will limited to be hydrostatic. The external pressure will be the hydrostatic and hydrodynamic pressures. The normal maximum collapse pressure would consequently be the external hydrodynamic pressures. During hurricanes, both oil and gas pipelines are shut-in with significant internal pressure (normally about 70 % of the normal operating pressures). These pressures would be far smaller than for installation conditions in which the internal pressure in the pipeline was atmospheric.

There could be an accidental condition for gas pipelines in which the operating pipeline internal pressure was allowed to fall to atmospheric (e.g. pipeline depressured in case of explosion). In this case, the effective collapse pressure would be the same as for the installation conditions (pipelines installed with atmospheric internal pressure).

For oil pipelines and pipelines transporting oil and gas, there could be an accidental condition in which the pipeline pump pressures were reduced to zero and the pipeline internal pressures would be that due to the hydrostatic pressure of the oil inside the pipeline. In this case, the collapse pressure would be determined by the difference in Specific Gravity of the sea water outside the pipeline and the oil inside the pipeline.

The target reliability for this accidental in-place condition would be the same as for the propagating buckling condition, $\beta = 2.0$. Given that the gas , oil, and oil-gas pipelines were evaluated for this accidental condition using 10-year return period conditions, the pipelines will be evaluated for external pressures that would have a median Bias of 0.98 and Coefficient of Variation of 2 %.

Consequently, for in-place operating design conditions, the collapse pressure limit state will be confined to an accidental limit state that is evaluated on the basis of 10-year return period conditions and for a target Safety Index of $\beta = 2.0$.

11.2 Analytical Model – Modification of Timoshenko Elastic Formulation

The fundamental analytical expression that will be used for evaluation of measured pipeline net collapse pressure will be:

$$Pc = 0.5 \{Pu + Pe\,K - [\,(Pu + Pe\,K)^2 - 4\,Pu\,Pe]^{0.5}\,\}$$

This equation will be termed the 'Timoshenko Ultimate' formulation.

A second formulation will be used as follows:

$$Pc = 0.5 \{Py + Pe\,K - [\,(Py + Pe\,K)^2 - 4\,Py\,Pe]^{0.5}\,\}$$

This is the traditional 'Timoshenko Elastic' formulation. The terms in these expressions are as follows:

$$Pu = 3\ Su\ t / D$$

$$Py = 2\ Sy\ t / D$$

$$Pe = 2\ E\ (1 - v^2)^{-1}\ (t/D)^3$$

$$K = 1 + 3\ f\ (D/t)$$

$$f = (D_{max} - D_{min}) / (D_{max} + D_{min})$$

where Pu is the ultimate pressure at collapse, Py is the yield pressure at collapse, Pe is the elastic collapse pressure, K is an imperfection factor, f is the pipe ovality factor, Su is the ultimate tensile stress (transverse), Sy is the yield tensile stress (transverse), E is Young's modulus, n is Poisson's ratio (0.3 for steel), t is the pipe wall thickness, D is the pipe diameter, D_{max} is the maximum pipe diameter, and is the minimum pipe diameter.

This 'Timoshenko Ultimate' formulation has been based on an expression for Pu that represents a modification of the traditional yield pressure at collapse, Py. This modification takes account of the additional pressure required to form two plastic hinge lines in the wall of the pipeline.

In general terms, pipelines that have D/t greater than about 25 will be controlled by the elastic buckling pressure, Pe. Pipelines that have D/t less than about 25 will be controlled by the yield or ultimate collapse pressures, Py or Pu.

11.3 Fabricated Pipe Collapse Pressures

Figure 11.1 summarizes results of analysis of the RAM PIPE collapse pressure database on fabricated pipelines (Miller, et al, 1976; Bea, et al, 1998). These pipelines has initial ovalities that range from 0.6 % to 1.6 % with a mean value of 1.0 %. The coefficient of Variation of the ovalities was 55 %.

Figure 11.1 shows the test buckling or collapse pressure, the elastic collapse pressure (Pe), and the Timoshenko Elastic formulation (noted as Timoshenko Reduced Pressure) as a function of the pipeline D/t ratio. For the D/t's greater than about 30, the Timoshenko Elastic formulation results in a dramatic under-prediction of the test collapse pressures. The elastic collapse pressure without any modification for ovality does a better job of matching the test data. But, the test data specimens have ovality and it would not be reasonable to use the elastic collapse pressure as a design formulation.

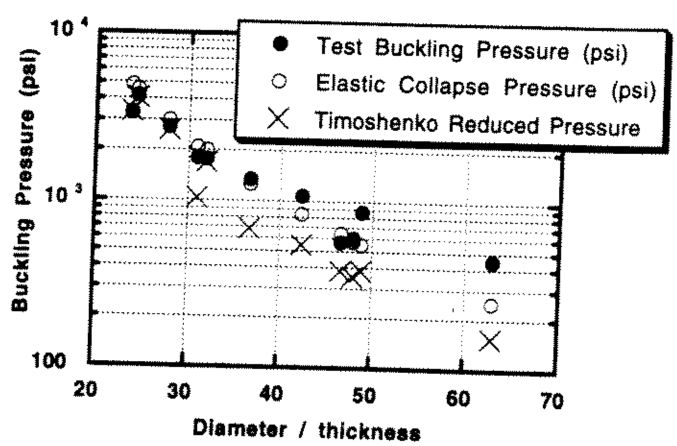

Figure 11.1 – Comparison of collapse pressure test data with Timoshenko Elastic and elastic collapse pressure formulations

Figure 11.2 summarizes a statistical analysis of these results. The Timoshenko Elastic formulation has a median Bias (measured pressure / predicted pressure) of 1.6 and has a coefficient of variation of V = 35 %. The elastic collapse pressure formulation has a median Bias of 1.0 and a coefficient of variation of V = 30 %

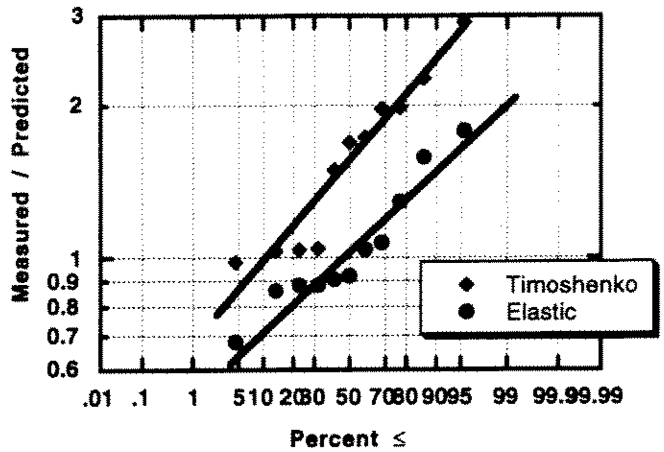

Figure 11.2 – Statistical analysis of bias in collapse pressure prediction formulations

In an attempt to develop an un-biased formulation of the collapse pressure for pipelines that have significant initial ovalization, the Timoshenko Elastic formulation was extended to a 4 hinge model (ratio of 4 hinge to 2 hinge model capacities = 1.7) in which:

$$Pu = 5.1 \ (t / D)$$

was used in the formulation:

$$Pc = 0.5 \ \{Pu + Pe \ K - [(Pu + Pe \ K)^2 - 4 \ Pu \ Pe]^{0.5}\}$$

The results are summarized in Figure 11.3. The median Bias is 1.0 and the Coefficient of Variation of the Bias is 31 %.

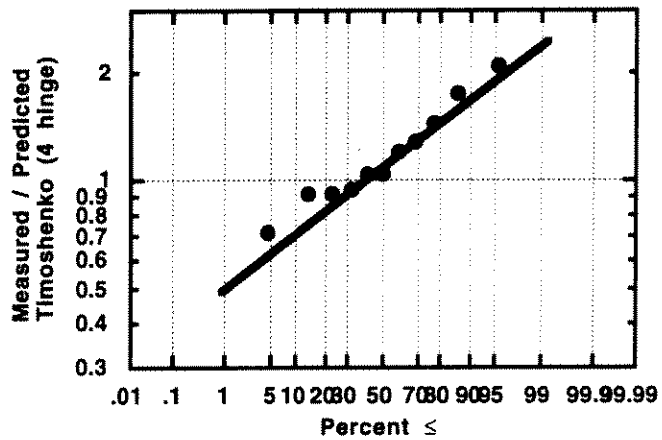

Figure 11.3 – Statistical analysis of bias in 4-hinge Timoshenko Ultimate formulation

11.4 Seamless Pipe Collapse Pressures

A database of 74 tests on seamless pipeline test specimens was assembled during the RAM PIPE project (Bea, et al, 1998). These specimens had ovalities that ranged from 0.01 % to 1.0 % with a mean value of 0.10 %. The Coefficient of Variation of the ovalities was 90 %.

The analyses were initially performed using the 4-hinge Timoshenko Ultimate formulation. The formulation substantially over-predicted the collapse pressures. The analyses were then performed using the Timoshenko Elastic formulation. The results are summarized in Figure 11.4. The median bias is B_{50} = 1.0 and the Coefficient of Variation of the Bias is V_B = 12.4 %.

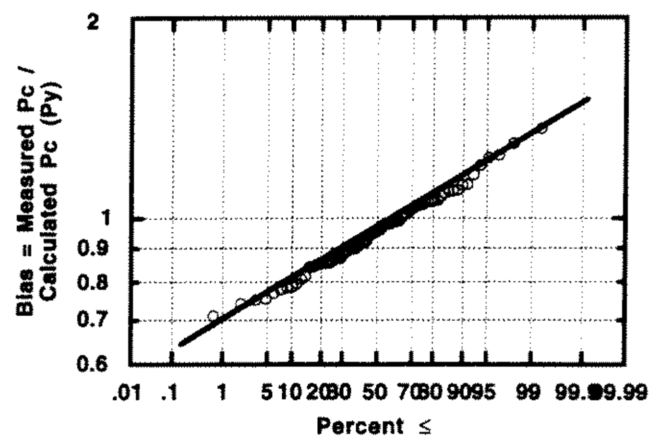

Figure 11.4 – Bias in Timoshenko Elastic formulation based on results from seamless pipe tests

The Timoshenko Elastic formulation results in an unbiased formulation of the collapse pressures for the seamless pipelines that have very low ovality. The Timoshenko Ultimate 4-hinge formulation results in an unbiased formulation of the collapse pressures for fabricated pipelines that have very high ovality. The quality assurance and control used in manufacture of the pipeline has an important influence on the pipeline ovality and hence on the appropriate design formulation.

11.5 Simulated Pipe Test Data Analysis

A database of 44 simulated 'tests' on collapse pressures of X-52 and X-77 pipe were provided by Bai, et al (1996) and Igland (1997). Figure 11.5 summarizes results from the statistical analysis of the simulated test data Bias. The Timoshenko Elastic model was used to calculate the collapse pressures. Four ovalities were used in the calculations: f = measured, f = 0.001 (e.g. high quality seamless pipe), f = 0.01 (low quality fabricated pipe), and f = 0.005.

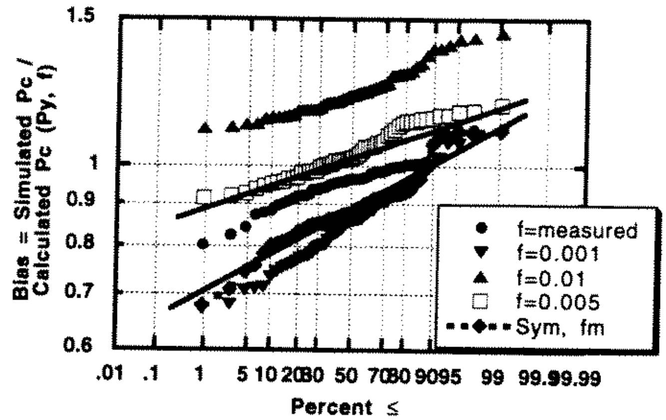

Figure 11.5 – Bias from Simulated Test Data for Various Ovalities

The formulation based on f = 0.005 produced a median Bias B_{50} = 1.0 and a coefficient of variation of the bias of V_B = 4.0 %.

The formulation based on the measured ovalities and SMYS times 1.1 produced a median Bias of B_{50} = 0.96 and Coefficient of Variation of the Bias of V_B = 4.1 %. The formulation based on the measured ovalities and the simulation model yield strengths produced a median Bias of B_{50} = 0.90 and Coefficient of Variation of the Bias of V_B = 8.7 %.

If one used the seamless pipeline test data, a median bias of B_{50} = 1.0 and Coefficient of Variation of the Bias of 11.4 %. The simulation data analyzed in the same way as the test data developed a median bias of B_{50} = 0.90 and Coefficient of Variation of the Bias of 8.7 %. The simulation median bias would have to be multiplied by a median Bias correction factor of 1.11. The simulation test data bias Coefficient of Variation would have to have an additional Coefficient of Variation of 8.8 % added to it in quadrature.

11.6 Collapse Pressure Design Factors

The median bias in the external collapse pressures for accidental operating conditions (10-year storm) B_{Pe50} = 0.98. The uncertainties in the total external pressures will be σ_{lnPe} = 2 %.

The median bias in the collapse pressure capacities for high ovality (f_{50} = 1.0 %) fabricated pipelines will be taken as B_{50} = 1.0 and a Coefficient of Variation of the Bias V_B = 31 %. The median bias in the collapse pressure capacities for low ovality (f_{50} = 0.1 %) pipelines will be taken as B_{50} = 1.0 and a Coefficient of Variation of the Bias V_B = 12 %.

For accidental operating conditions, the resulting total uncertainty in the demand and capacity variables is 31 % and 12 % for high and low ovality pipelines, respectively. The accidental conditions target annual Safety Index will be taken to be β = 2.0 (design) and β = 1.7 (reassessment).

The WSD collapse design factor for accidental operating conditions can be computed from:

$$f = [0.98 \exp (2.0 \times 0.31)]^{-1} = 0.55 \text{ (high ovality)}$$

$$f = [0.98 \exp (2.0 \times 0.12)]^{-1} = 0.80 \text{ (low ovality)}$$

The WSD collapse reassessment factor for accidental operating conditions can be computed from:

$$f = [0.98 \exp (1.7 \times 0.31)]^{-1} = 0.60 \text{ (high ovality)}$$

$$f = [0.98 \exp (1.7 \times 0.12)]^{-1} = 0.83 \text{ (low ovality)}$$

The API 1111 (1998) guidelines define WSD collapse design factors of 0.7 for seamless or ERW pipe (low ovality), and 0.6 for cold expanded pipe such as DSAW pipe (high ovality).

11.7 Collapse Pressures of Dented Pipelines

Kyriakidies and Yeh (1985) developed test data on the burst pipelines that was intentionally 'dented' resulting in large ovalization of the pipeline cross section. A summary of their results is given in Figure 11.6. There is a linear decrease in the collapse pressure with the pipeline diameter to thickness ratio, and the rate and amount of decrease is a function of the ovality of the pipeline.

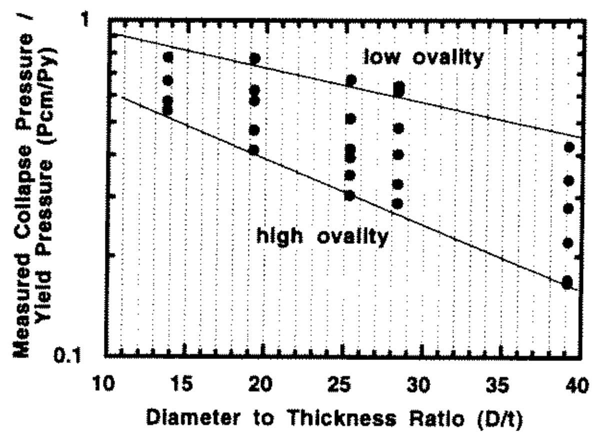

Figure 11.6 – Effect of ovality / denting on pipeline collapse pressures

Analysis of this test data to determine the bias and uncertainty in the Timoshenko Elastic (yield) and Ultimate (2 hinge model) is summarized in Figure 11.7. The median Bias associated with the Timoshenko Elastic formulation is $B_{50} = 1.2$. The Coefficient of Variation of the Bias is $V_B = 23$ %. The median Bias associated with the Timoshenko Ultimate (2-hinge) formulation is $B_{50} = 0.9$. The Coefficient of Variation of the Bias is $V_B = 20$ %.

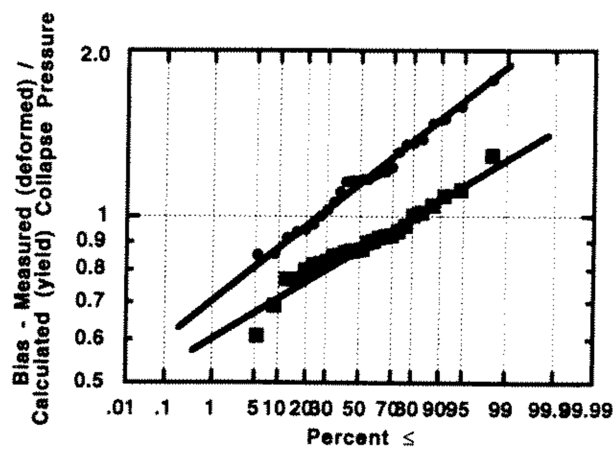

Figure 11.7 – Bias in computed collapse pressures of dented pipelines based on Timoshenko Elastic and Ultimate formulations

Kyriakides and Yeh (1985) also provided test data on different diameter to thickness ratios for varying lengths of the denting – ovalization relative to the pipe diameter. The length / diameter ratios ranged from 1 to 12. Figure 11.8 summarizes the bias and uncertainty for pipe having a diameter to thickness ratio of D/t = 13.45. The median Bias based on the Timoshenko Elastic and Ultimate formulations is $B_{50} = 1.5$ and 1.0, respectively. The Coefficient of Variation of the Bias based on the Timoshenko yield and ultimate formulations is $V_B = 16$ % for both formulations.

Figure 11.9 summarizes the bias and uncertainty for pipe having a diameter to thickness ratio of D/t = 25.3. The median Bias based on the Timoshenko Elastic and Ultimate formulations is $B_{50} = 1.3$

and 1.0, respectively. The Coefficient of Variation of the Bias based on the Timoshenko Elastic and Ultimate formulations is $V_B = 23$ % and $V_B = 20$ %, respectively.

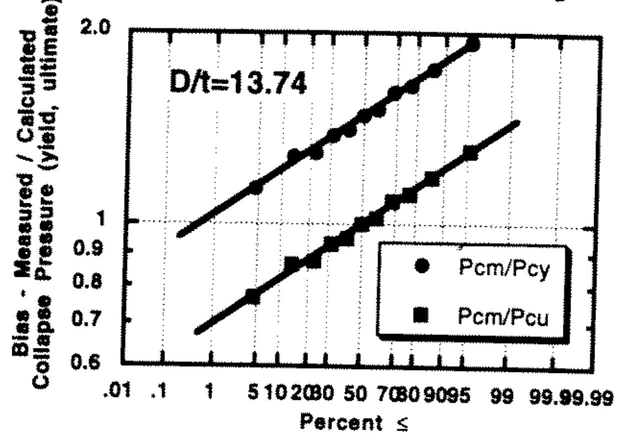

Figure 11.8 – Bias in computed collapse pressures based on the Timoshenko yield and ultimate formulations for D/t = 13.74 pipe

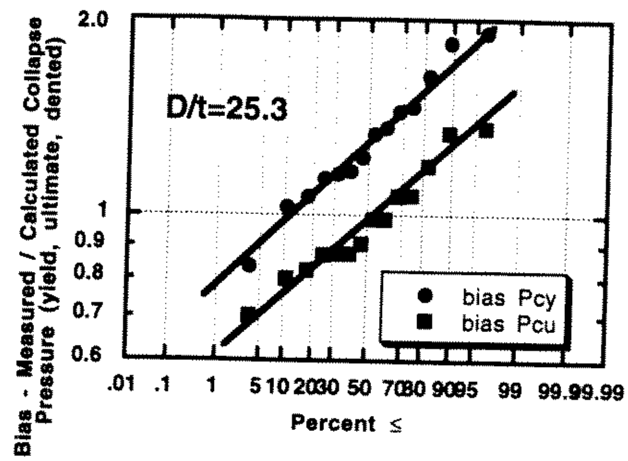

Figure 11.9 – Bias in computed collapse pressures based on the Timoshenko yield and ultimate formulations for D/t = 25.3 pipe

11.8 Collapse Pressure Reassessment Factors

The median bias in the external collapse pressures for accidental operating conditions (10-year storm) $B_{Pe50} = 0.98$. The uncertainties in the total external pressures will be $\sigma_{\ln Pe} = 2$ %.

In the case of dented pipelines, the effects of the denting are reflected in the ovality of the pipeline expressed by:

$$f = (D_{max} - D_{min}) / (D_{max} + D_{min})$$

The collapse pressure capacites are determined using the Timoshenko Ultimate (2 hinge) analysis:

$$Pc = 0.5 \{Pu + Pe\,K - [\,(Pu + Pe\,K)^2 - 4\,Pu\,Pe]^{0.5}\,\}$$

where

$$Pu = 3\,Su\,t / D$$

$$Pe = 2\,E\,(1 - v^2)^{-1}\,(t/D)^3$$

$$K = 1 + 3\,f\,(D/t)$$

The median bias in the collapse pressure capacities for dented pipelines will be taken as $B_{50} = 1.0$ and a Coefficient of Variation of the Bias $V_B = 20$ %.

For accidental operating conditions, the resulting total uncertainty in the demand and capacity variables is 20 %. The accidental reassessment conditions target annual Safety Index is $\beta = 1.7$.

The WSD collapse design factor for accidental operating conditions can be computed from:

$$f = [0.98 \exp (1.7 \times 0.20)]^{-1} = 0.73$$

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Appendix A – Corroded Pipeline Tests Burst Pressure Database

No	Dia	t	SMYS	SMTS	Sig(y)	d	d/t	Pbm
	mm	mm	MPa	Mpa	MPa	mm		Mpa
1	762	9.398	358.50	471	404.80	3.708	0.395	11.19
2	762	9.398	358.50	471	404.80	3.708	0.395	11.17
3	762	9.398	358.50	471	404.80	3.988	0.424	11.72
4	762	9.525	358.50	471	440.00	6.096	0.640	11.52
5	762	9.525	358.50	471	405.50	5.309	0.557	10.52
6	609.60	9.271	241.30	350	279.30	6.883	0.742	7.59
7	609.60	9.271	241.30	350	279.30	6.375	0.688	8.03
8	609.60	9.271	241.30	350	279.30	6.375	0.688	8.41
9	609.60	9.398	241.30	350	288.30	6.629	0.705	7.17
10	609.60	9.525	241.30	350	288.30	7.163	0.752	8.03
11	609.60	9.27	241.30	350	288.30	6.629	0.715	7.03
12	609.60	9.271	241.30	350	288.30	5.563	0.600	8.38
13	609.60	9.271	241.30	350	288.30	5.842	0.630	9.10
14	609.60	9.271	241.30	350	288.30	6.629	0.715	9.10
15	609.60	9.652	241.30	350	288.30	6.375	0.660	9.21
16	609.60	9.398	241.30	350	288.30	4.775	0.508	9.31
17	609.60	9.398	241.30	350	288.30	6.096	0.649	9.48
18	609.60	9.525	241.30	350	288.30	6.096	0.640	9.92
19	609.60	9.271	241.30	350	288.30	6.629	0.715	10.00
20	609.60	9.525	241.30	350	288.30	6.375	0.669	8.28
21	609.60	9.525	241.30	350	288.30	7.417	0.779	10.28
22	609.60	9.525	241.30	350	288.30	5.563	0.584	10.48
23	609.60	9.525	241.30	350	288.30	4.775	0.501	10.48
24	609.60	9.525	241.30	350	288.30	4.496	0.472	10.48
25	609.60	9.652	241.30	350	288.30	6.883	0.713	10.41
26	762	9.525	358.50	471	427.60	9.525	1.000	12.03
27	762	9.525	358.50	471	422.80	3.708	0.389	12.69
28	762	9.525	358.50	471	427.60	2.921	0.307	13.07
29	762	9.525	358.50	471	456.60	5.842	0.613	12.24
30	762	9.525	358.50	471	486.90	5.309	0.557	14.76
31	762	9.525	358.5	471	458.6	5.309	0.557	13.79
32	508	8.255	241.3	350	282.8	5.309	0.643	7.93
33	508	8.255	241.3	350	283.4	5.563	0.674	11.69
34	406.4	7.874	172.4	264	197.2	5.842	0.742	7.59
35	406.4	7.874	172.4	264	197.2	6.096	0.774	8.76
36	406.4	7.874	172.4	264	197.2	7.163	0.910	5.66
37	406.4	7.874	172.4	264	197.2	6.909	0.877	6.14
38	406.4	7.874	172.4	264	195.9	5.055	0.642	8.90
39	609.6	10.59	241.3	350	346.2	7.366	0.696	9.62

No	Dia	t	SMYS	SMTS	Sig(y)	d	d/t	Pbm
	mm	mm	MPa	Mpa	MPa	mm		Mpa
40	609.6	10.41	241.3	350	322.8	9.652	0.927	11.45
41	609.6	10.06	241.3	350	346.2	9.144	0.909	6.41
42	609.6	11.28	241.3	350	346.2	5.588	0.495	13.10
43	609.6	9.296	241.3	350	371.7	6.985	0.751	10.10
44	609.6	9.246	241.3	350	358.6	6.452	0.698	8.72
45	609.6	9.017	241.3	350	358.6	7.341	0.814	10.38
46	609.6	8.103	241.3	350	327.6	5.486	0.677	11.94
47	609.6	8.433	241.3	350	310.3	5.588	0.663	12.08
48	609.6	9.525	241.3	350	371	7.493	0.787	5.12
49	609.6	9.525	255.1	366	336.6	8.128	0.853	5.43
50	508	7.925	241.3	350	344.8	6.401	0.808	4.92
51	508	7.747	241.3	350	380	5.334	0.689	11.54
52	609.6	9.169	241.3	350	326.9	8.103	0.884	8.90
53	609.6	9.169	241.3	350	284.1	7.239	0.790	10.17
54	609.6	9.017	241.3	350	346.9	6.172	0.684	12.01
55	609.6	9.423	241.3	350	310.3	7.01	0.744	9.36
56	609.6	9.423	241.3	350	310.3	7.391	0.784	9.36
57	609.6	9.449	241.3	350	332.4	7.214	0.763	11.03
58	609.6	9.246	241.3	350	331.7	5.69	0.615	11.34
59	609.6	9.296	241.3	350	296.6	6.147	0.661	12.47
60	609.6	9.296	241.3	350	355.2	4.851	0.522	10.92
61	609.6	9.347	241.3	350	329	7.315	0.783	10.55
62	508	7.188	241.3	350	261.4	4.623	0.643	7.52
63	508	6.960	241.3	350	279.3	3.302	0.474	11.99
64	508	7.899	241.3	350	243.4	6.071	0.769	11.68
65	508	7.899	241.3	350	243.4	2.667	0.338	11.68
66	508	6.756	241.3	350	277.2	3.658	0.541	10.39
67	508	7.849	241.3	350	289	5.537	0.705	12.52
68	762	9.449	358.5	471	418.6	3.302	0.349	12.72
69	762	9.550	358.5	471	358.6	5.842	0.612	10.45
70	762	9.525	358.5	471	415.9	3.556	0.373	12.52
71	762	9.703	358.5	471	437.9	3.683	0.380	13.12
72	762	9.550	358.5	471	406.9	3.302	0.346	12.31
73	762	9.601	358.5	471	428.3	2.794	0.291	13.21
74	762	9.627	358.5	471	451	4.318	0.449	12.24
75	762	9.677	358.5	471	358.6	7.62	0.787	7.72
76	762	9.601	358.5	471	422.1	4.318	0.450	11.86
77	762	9.576	358.5	471	426.2	4.064	0.424	12.34
78	762	9.474	359.5	471	415.2	2.794	0.295	12.69
79	609.6	9.525	255.1	366	289.7	8.179	0.859	5.55

No	Dia	t	SMYS	SMTS	Sig(y)	d	d/t	Pbm
	mm	mm	MPa	Mpa	MPa	mm		Mpa
80	762	9.271	358.5	471	404.1	5.817	0.627	6.81
81	762	9.525	358.5	471	474.5	6.223	0.653	6.84
82	762	9.525	386.1	471	444.1	3.81	0.400	13.59
83	508	6.604	358.5	471	420.7	5.537	0.838	5.76
84	914.4	8.382	448.2	495	448.3	5.537	0.661	5.35
85	762	7.569	413.7	471	489.7	6.833	0.903	5.62
86	558.8	5.029	358.5	542	420.7	3.759	0.747	5.71
87	508	6.35	413.7	517		2.54	0.400	14.55
88	508	6.35	413.7	471		2.54	0.400	13.85
89	508	6.35	413.7	517		2.54	0.400	12.35
90	508	6.35	413.7	517		2.54	0.400	15.85
91	508	6.35	413.7	517		2.54	0.400	11.25
92	508	6.35	413.7	517		2.54	0.400	11.55
93	508	6.35	413.7	517		2.54	0.400	13.05
94	508	6.35	413.7	517		0	0.000	13.05
95	508	6.35	413.7	517		0	0.000	13.05
96	508	6.35	413.7	517		2.54	0.400	15.25
97	508	6.35	413.7	517		2.54	0.400	11.05
98	508	6.35	413.7	517		2.54	0.400	10.55
99	508	6.36	413.7	517		0	0.000	15.45
100	508	6.4	413.7	517		0	0.000	15.25
101	508	6.4	413.7	517		3.429	0.536	8.00
102	508	6.4	413.7	517		2.159	0.337	11.80
103	508	6.4	413.7	517		3.008	0.470	12.50
104	508	6.4	413.7	517		2.944	0.460	9.80
105	508	6.4	413.7	517		3.366	0.526	8.45
106	508	6.4	413.7	517		3.175	0.496	8.40
107	610	12.34	358.5	471	451	4.936	0.400	14.44
108	610	12.34	358.5	471	447	4.936	0.400	14.00
109	610	12.34	358.5	471	447	4.936	0.400	15.45
110	610	12.34	358.5	471	447	4.936	0.400	16.46
111	610	12.34	358.5	471	447	4.936	0.400	18.45
112	610	12.34	358.5	471	451	0	0.000	21.30
113	610	12.34	358.5	471	451	4.936	0.400	14.90
114	610	12.34	358.5	471	451	0	0.000	21.20
115	610	12.34	358.5	471	451	4.936	0.400	14.40
116	914	22	413.7	517	434	0	0.000	26.30
117	914	22	413.7	517	434	0	0.000	26.40
118	914	22	413.7	517	434	6.6	0.300	18.70
119	914	22	413.7	517	434	6.16	0.280	19.50

No	Dia	t	SMYS	SMTS	Sig(y)	d	d/t	Pbm
	mm	mm	MPa	Mpa	MPa	mm		Mpa
120	914	22	413.7	517	434	10.34	0.470	14.70
121	914	22	413.7	517	434	11	0.500	13.00
122	914	22	413.7	517	434	15.18	0.690	8.60
123	914	22	413.7	517	434	14.74	0.670	8.10
124	914	22	413.7	517	434	14.74	0.670	8.20
125	324	5.93	317.2	432	378	4.685	0.790	13.49
126	324	6.07	317.2	432	381	4.006	0.660	14.29
127	324	5.84	317.2	432	382	3.913	0.670	16.29
128	324	5.99	317.2	432	351	4.672	0.780	15.36
129	324	6.00	317.2	432	403	4.38	0.730	16.09
130	324	6.07	317.2	432	421	2.914	0.480	16.95
131	324	5.58	317.2	432	346	4.408	0.790	13.00
132	324	6.14	317.2	432	375	2.395	0.390	15.78
133	324	6.16	317.2	432	356	4.497	0.730	14.29
134	324	5.95	317.2	432	356	4.165	0.700	15.57
135	324	6.02	317.2	432	359	1.987	0.330	16.12
136	324	6.40	317.2	432	382	3.23	0.505	16.64
137	324	6.01	317.2	432	382	3.6	0.599	16.22
138	324	6.30	317.2	432	373	3.57	0.567	15.95
139	323	6.31	317.2	432	373	3.73	0.591	14.16
140	324	6.16	317.2	432	356	3.73	0.606	18.85
141	324	6.27	317.2	432	356	3.76	0.600	19.13
142	324	6.25	317.2	432	356	3.79	0.606	19.27
143	324	6.18	317.2	432	421	3.75	0.607	19.44
144	325	6.45	317.2	432	373	3.05	0.473	15.81
145	324	6.40	317.2	432	373	3.72	0.581	13.87
146	325	6.45	317.2	432	356	3.79	0.588	14.84
147	324	6.35	317.2	432	356	3.72	0.586	15.53
148	322	6.27	317.2	432	381	3.77	0.601	17.61
149	324	6.29	317.2	432	378	3.79	0.603	15.11
150	324	6.24	317.2	432	381	3.79	0.607	15.67
151	324	6.16	317.2	432	378	3.70	0.601	15.25

Appendix B – Dented - Gouged Pipeline Tests Burst Pressure Database

Spec. No.	D (in)	t (in)	D/t	sy (ksi)	sf(ksi)	DD(in)	d(in)
BGC-1	12.75	0.26	49.0	57.0	59.2	*0.25*	*0.026*
BGC-2	12.75	0.27	46.8	57.0	56.1	0.28	0.030
BGC-3	12.75	0.29	43.6	57.0	58.0	0.23	0.038
BGC-4	12.75	0.30	43.2	57.0	14.3	0.62	0.059
BGC-5	12.75	0.30	43.2	57.0	11.8	0.63	0.059
BGC-6	12.75	0.31	41.6	57.0	16.7	0.69	0.049
BGC-7	12.75	0.30	42.0	57.0	6.4	0.98	0.091
BGC-8	12.75	0.30	42.5	57.0	7.0	1.02	0.084
BGC-9	12.75	0.27	47.3	57.0	9.0	0.93	0.089
BGC-10	18.00	0.32	55.6	50.5	35.9	0.17	0.110
BGC-11	18.00	0.31	57.3	50.5	39.0	0.08	0.110
BGC-12	18.00	0.32	55.6	50.5	44.4	0.09	0.110
BGC-13	18.00	0.32	56.5	50.5	54.5	0.37	0.051
BGC-14	18.00	0.31	57.4	50.5	53.7	0.36	0.047
BGC-15	18.00	0.31	57.4	50.5	53.4	0.37	0.047
BGC-16	18.00	0.32	56.6	50.5	45.0	1.08	0.035
BGC-17	18.00	0.30	60.0	50.5	51.5	1.08	0.030
BGC-18	18.00	0.31	57.9	50.5	53.9	1.08	0.028
BGC-19	24.00	0.47	51.2	50.5	6.4	1.68	0.072
BGC-20	24.00	0.47	51.2	50.5	7.7	1.75	0.062
BGC-21	24.00	0.47	51.2	50.5	9.5	1.58	0.068
BGC-22	24.00	0.47	51.2	50.5	15.9	1.58	0.074
BGC-23	24.00	0.47	51.2	50.5	8.3	1.78	0.103
BGC-24	24.00	0.47	51.2	50.5	26.4	1.10	0.051
BGC-25	24.00	0.47	51.2	50.5	13.0	1.13	0.060
BGC-26	24.00	0.47	51.2	50.5	16.0	1.10	0.052
BGC-27	24.00	0.47	51.2	50.5	32.8	1.20	0.046
BGC-28	24.00	0.47	51.2	50.5	33.0	1.22	0.049
BGC-29	24.00	0.47	51.2	50.5	26.6	0.60	0.133
BGC-30	24.00	0.47	51.2	50.5	17.8	0.67	0.176
BGC-31	24.00	0.47	51.2	50.5	15.9	0.62	0.135
BGC-32	24.00	0.47	51.2	50.5	28.7	0.55	0.125
BGC-33	30.00	0.47	64.0	50.5	17.0	1.11	0.084
BGC-34	30.00	0.47	64.0	50.5	16.5	1.05	0.089
BGC-35	30.00	0.47	64.0	50.5	23.5	1.02	0.098
BGC-36	30.00	0.47	64.0	50.5	24.7	0.93	0.098
BGC-37	42.00	0.56	74.7	50.5	23.5	2.01	0.120
BGC-38	42.00	0.56	74.7	50.5	24.3	1.87	0.130
BGC-39	42.00	0.56	74.7	50.5	24.7	1.96	0.116
BGC-40	42.00	0.56	74.7	50.5	19.4	3.02	0.065
BGC-41	42.00	0.56	74.7	50.5	24.7	3.06	0.063
BGC-42	30.00	0.47	64.0	60.0	14.1	0.92	0.109

Spec. No.	D (in)	t (in)	D/t	sy (ksi)	sf(ksi)	DD(in)	d(in)
BGC-43	30.00	0.47	64.0	60.0	5.5	1.41	0.109
BGC-44	30.00	0.47	64.0	60.0	26.1	0.35	0.109
BGC-45	30.00	0.47	64.0	60.0	18.5	0.53	0.109
BGC-46	30.00	0.47	64.0	60.0	10.2	1.09	0.135
BGC-47	30.00	0.47	64.0	60.0	4.7	1.81	0.135
BGC-48	30.00	0.47	64.0	60.0	11.5	0.98	0.135
BGC-49	30.00	0.47	64.0	60.0	23.4	0.62	0.135
BGC-50	30.00	0.47	64.0	60.0	4.3	2.09	0.135
BGC-51	30.00	0.47	64.0	60.0	22.2	0.38	0.135
BGC-52	30.00	0.47	64.0	60.0	17.0	0.50	0.116
BGC-53	30.00	0.47	64.0	60.0	14.7	0.54	0.116
BGC-54	30.00	0.47	64.0	60.0	18.3	0.49	0.116
BGC-55	30.00	0.47	64.0	60.0	18.8	0.44	0.116
BGC-56	30.00	0.47	64.0	60.0	11.4	0.56	0.116
BGC-57	30.00	0.47	64.0	60.0	34.2	0.38	0.175
BGC-58	30.00	0.47	64.0	60.0	31.0	0.49	0.175
BGC-59	30.00	0.47	64.0	60.0	27.9	0.57	0.175
BGC-60	30.00	0.47	64.0	60.0	35.6	0.42	0.175
BGC-61	30.00	0.47	64.0	60.0	32.2	0.48	0.175
BGC-62	30.00	0.47	64.0	60.0	87.0	0.73	0.026
BGC-63	30.00	0.47	64.0	60.0	67.2	0.61	0.040
BGC-64	30.00	0.47	64.0	60.0	61.2	0.71	0.045
BGC-65	30.00	0.47	64.0	60.0	59.8	0.75	0.052
BGC-66	30.00	0.47	64.0	60.0	58.1	0.73	0.049
BGC-67	30.00	0.47	64.0	60.0	56.7	0.73	0.100
BGC-68	30.00	0.47	64.0	60.0	56.4	0.76	0.045
BGC-69	30.00	0.47	64.0	60.0	20.2	0.81	0.020
BGC-70	30.00	0.47	64.0	60.0	15.1	2.07	0.094
BGC-71	30.00	0.47	64.0	60.0	28.5	2.09	0.010
BGC-72	30.00	0.47	64.0	60.0	10.6	2.09	0.011
BGC-73	30.00	0.47	64.0	60.0	29.8	2.29	0.007
BGC-74	30.00	0.47	64.0	60.0	42.4	2.30	0.117
BGC-75	30.00	0.47	64.0	60.0	36.6	2.65	0.117
BGC-76	30.00	0.47	64.0	52.0	57.4	0.45	0.117
BGC-77	30.00	0.47	64.0	52.0	55.6	0.60	0.117
BGC-78	30.00	0.47	64.0	52.0	53.3	0.72	0.117
BGC-79	30.00	0.47	64.0	52.0	52.7	0.78	0.117
BGC-80	30.00	0.47	64.0	52.0	44.0	0.99	0.117
BGC-81	30.00	0.47	64.0	52.0	54.1	0.50	0.117
BGC-82	30.00	0.47	64.0	52.0	45.4	0.81	0.117
BGC-83	30.00	0.47	64.0	52.0	43.7	0.66	0.117
BGC-84	30.00	0.47	64.0	52.0	42.7	0.72	0.117
BGC-85	30.00	0.47	64.0	52.0	41.6	0.78	0.117
BGC-86	30.00	0.47	64.0	52.0	40.5	0.93	0.117

Spec. No.	D (in)	t (in)	D/t	sy (ksi)	sf(ksi)	DD(in)	d(in)
BGC-87	30.00	0.47	64.0	52.0	38.3	1.06	0.117
BGC-88	30.00	0.47	64.0	52.0	42.8	0.46	0.117
BGC-89	30.00	0.47	64.0	52.0	39.0	0.61	0.117
BGC-90	30.00	0.47	64.0	52.0	38.4	0.43	0.117
BGC-91	30.00	0.47	64.0	52.0	37.8	0.58	0.117
BGC-92	30.00	0.47	64.0	52.0	33.1	0.74	0.117
BGC-93	30.00	0.47	64.0	52.0	34.0	0.85	0.117
BGC-94	30.00	0.47	64.0	52.0	28.9	1.22	0.117
BGC-95	30.00	0.47	64.0	52.0	27.2	0.99	0.117
BGC-96	30.00	0.47	64.0	52.0	26.7	1.19	0.117
BGC-97	30.00	0.47	64.0	52.0	21.7	1.34	0.117
BGC-98	30.00	0.47	64.0	52.0	18.4	1.61	0.117
BGC-99	30.00	0.47	64.0	52.0	64.5	0.41	0.122
BGC-100	30.00	0.47	64.0	52.0	66.4	0.39	0.122
BGC-101	30.00	0.47	64.0	52.0	65.8	0.40	0.125
BGC-102	30.00	0.47	64.0	52.0	53.4	0.76	0.129
BGC-103	30.00	0.47	64.0	52.0	50.0	0.78	0.129
BGC-104	30.00	0.47	64.0	52.0	47.3	0.83	0.123
BGC-105	30.00	0.47	64.0	52.0	44.8	0.89	0.130
BGC-106	30.00	0.47	64.0	52.0	34.3	1.09	0.128
BGC-107	30.00	0.47	64.0	52.0	20.9	1.20	0.123
BGC-108	30.00	0.47	64.0	52.0	56.8	0.57	0.117
BGC-109	30.00	0.47	64.0	52.0	51.5	0.64	0.117
BGC-110	30.00	0.47	64.0	52.0	45.0	0.96	0.117
BGC-111	30.00	0.47	64.0	52.0	40.1	0.41	0.117
BGC-112	30.00	0.47	64.0	52.0	20.9	0.78	0.117
BGC-113	30.00	0.47	64.0	52.0	55.2	0.30	0.117
BGC-114	30.00	0.47	64.0	52.0	35.9	0.70	0.117
BGC-115	30.00	0.47	64.0	52.0	32.0	0.52	0.117
BGC-116	30.00	0.47	64.0	52.0	30.2	0.75	0.117
BGC-117	30.00	0.47	64.0	52.0	22.3	1.08	0.117
DTZ/1P	30.00	0.50	60.0	62.4	20.0	0.46	0.240
DTZ/2P	30.00	0.50	60.0	62.4	11.7	0.98	0.138
DTZ/1R	30.00	0.50	60.0	62.4	23.4	1.05	0.103
DTZ/2R	30.00	0.50	60.0	62.4	20.7	1.06	0.095