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# Report of a Workshop on Requalification of Tubular Steel Joints in Offshore Structures

Andrew W. Taylor, editor

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Report of a Workshop on
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## Abstract

This report is a summary of a workshop titled "Requalification of Tubular Steel Joints in Offshore Structures," held September 5 and 6, 1995 in Houston, Texas. The workshop was sponsored by the U.S. Minerals Management Service, and the National Institute of Standards and Technology. This report contains the papers presented at the workshop, a summary of the workshop discussions, and the conclusions reached by the workshop participants. The major issues discussed at the workshop included tubular joint characterization, computational methods, tubular joint failure definition/condition, condition assessment, and code requirements/technology transfer. Needed technology developments included the following: improved methods for describing analytically the monotonic and cyclic behavior of joints, possibly through joint macro-models; improved methods for predicting the tensile fracture failure mode of joints; a coordinated effort to assess the body of available experimental data on tubular joints; investigation of elastic-plastic fracture mechanics applications to joints; cost/benefit studies prior to development of new analytical tools; a survey to determine the most important failure modes of joints; development of probabilistic approaches to condition assessment of joints; methods for characterizing the condition of new joints; improved methods for detecting flaws in existing joints; studies of the necessary scope and frequency of inspections of joints; improved code provisions for rating the ultimate strength of joints; improved code provisions for the use of actual steel strengths in evaluating existing joints; code provisions for evaluation of damaged joints; a definitive study of can lengths in K-joints; improved classification schemes for joints in terms of the ovalizing parameter $\alpha$; and methods of incorporating fracture mechanics into code provisions.

## Acknowledgments

A number of people and organizations were essential contributors to the success of this workshop. The workshop was funded by the U.S. Minerals Management Service (MMS). MMS support was coordinated by Charles E. Smith, Research Program Manager, Offshore Minerals Management, MMS. Meeting facilities were provided by Exxon Petroleum Research (EPR). The helpful suggestions of Nick Zettlemoyer of EPR in formulating the workshop program are also acknowledged. The workshop participants, particularly those traveling from overseas, and those who prepared and presented papers, are all thanked for contributing their time and ideas to the workshop discussions. Finally, thanks are due to the NIST researchers and staff who assisted in organizing the workshop. These are H.S. Lew, Dat Duthinh, John Gross, Shirley Taylor, and Nancy Fleegle.

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# 1. Introduction

*Workshop Purpose*

On September 5 and 6 a workshop was held in Houston to discuss the problems associated with the requalification of tubular steel joints in offshore structures. The objective of the workshop was to assess the current state-of-the-art, and the future technological challenges, related to the evaluation of steel tubular joints in existing offshore structures. Thirty-two people participated in the workshop, representing 26 different companies and organizations, as shown in Appendix A. This report is a summary of the papers presented at the workshop, the workshop discussions, and the conclusions reached by the workshop participants.

*Background:*

There are about 3800 offshore platforms operating in the Gulf of Mexico. Many of these are located in shallow water, and were constructed 20 to 30 years ago. Most of these platforms are still productive. However, the code-specified design level wave heights have increased substantially since the time when the platforms were designed and constructed, and code-specified. capacity resistance equations have changed. Consequently, these older platforms, designed under previous API standards, may not meet current requirements. More recently, several major platforms have been constructed off the West Coast of the United States. These platforms are located in zones of high seismicity, and may be subjected to severe earthquake loads. Very little is known about the cyclic inelastic behavior of joints in offshore platforms subjected to either extreme storms or to earthquakes.

When under-designed platforms are exposed to an extreme event, such as a storm or earthquake, they can undergo monotonic deformation in the inelastic range, as well as cycles of inelastic deformation. These deformations may not be large enough to cause immediate failure of a joint or frame, but the cumulative effect of plastic deformations caused by a series of storms or earthquakes over a number of years will result in reduced capacity, and may ultimately lead to failure. The question which needs to be answered, then, is what is the remaining service life of a tubular steel offshore structure which has experienced one or more damaging storms or earthquakes? Put another way, what will be the consequences of the next extreme event, in terms of the damage caused by both monotonic and cyclic loading?

Recently implemented industry and government standards have prompted re-examination of many existing offshore platforms. Under Minerals Management Service policies, offshore platforms must periodically be re-certified as to their safety and capacity. The petroleum industry has increasingly found it difficult to re-certify some older platforms because there is a lack of laboratory test data and analytical methods available to verify platform capacity under both inelastic monotonic loading and inelastic cyclic loading. Without reliable methods for evaluating joints exposed to many years of service, it becomes extremely difficult to re-assess older platforms.

*Organization of the Workshop*

The agenda of the two day workshop is shown in Appendix B. The workshop was structured so that the end product of the discussions would be a set of implementable recommendations for needed technology developments that will advance the state-of-practice of evaluation of tubular steel joints, especially as related to requalification of existing platforms.

Day 1: On the first day of the workshop, presentations were made by nine invited speakers. The purpose of these presentations was to give an overview of the current state-of-the-art with respect to requalification of tubular joints, and to set the stage for subsequent workshop discussions. The speakers reviewed research on requalification of tubular joints in England, Norway, Japan and the United States. Recent activities of the American Petroleum Institute (API) and the International Standards Organization (ISO) relating to tubular joints were also summarized. The behavior of multiplanar tubular joints was discussed, and applications of elasto-plastic fracture mechanics to assessment of tubular joints were proposed. Speakers also described the areas where they felt technology developments are most needed to improve requalification methods for existing platforms. Several authors submitted written versions of their presentations. These are reproduced in Appendix C.

The presentations were followed by an open forum, the purpose of which was to establish topics for discussion on the second day of the workshop. A list of all suggested discussion topics was compiled, but no priorities were assigned to the topics at that time. In the evening of the first day, the suggested discussion topics were reviewed by NIST staff, and organized into five main themes:

A. Joint Characterization
B. Computational Methods
C. Failure Definition/Condition
D. Condition Assessment
E. Code Requirements/Technology Transfer

Within each theme, sub-themes were listed, but in no particular order. The complete set of suggested discussion topics is shown below.

A. JOINT CHARACTERIZATION

1. Effect of frame continuity
2. Bias and uncertainty in strength. For example, elastic strength formulation based on true mean strength: $F_y = 1.167$ times the nominal $F_y$ (36 and 42 ksi steels only)
3. Static (monotonic loading) behavior
    a. Joint stiffness
    b. Joint ductility
    c. Evaluation of ungrouted joints: e.g. MSL's P-$\Delta$ and M-$\Theta$ formulas
    d. Other P-$\Delta$ and M-$\Theta$ relationships
4. Cyclic behavior
    a. Hysteresis behavior
    b. Multi-cycle ultimate strength degradation (earthquake loading)
    c. Plastic deformation cyclic damage
5. Evaluation and of grouted joints
6. Evaluation of damaged joints: formulations to account for dents, cracks, and corrosion
7. Evaluation of repaired joints: formulations for non-grouted joints needed
8. Understanding of (ductile) strength reduction in joints containing cracks
9. Strength reduction in grind repaired joints
10. Can length de-rating

B. COMPUTATIONAL METHODS

1. Validation of finite element methods for high strains and progressive cracking (qualify candidate software packages)
2. Interactive non-linear response algorithms
3. Determine benefits of a generalized elasto-plastic fracture mechanics methodology for tubular steel joints
4. Develop industry-oriented PC-based elasto-plastic fracture mechanics methodology
5. System analysis, i.e., pushover analyses including elements reflecting joint behavior

C. FAILURE DEFINITION / CONDITION

1. Develop understanding of crack growth in joints under high-amplitude, low cycle fatigue.
2. Development of tensile strain-based failure criteria
3. Effects of weld defects in high strain zones
4. Size effect on fracture
5. Summation of cycles from different types of loads (tracking damage at a specific point), for both low and high cycle fatigue.
6. Strength criteria for failures under tensile loads, applicable in non-linear finite element analysis.

D. CONDITION ASSESSMENT

1. Inspection methods of existing joints
   a. Optimization of inspection schedules
   b. Fatigue pre-damage (what to assume if inspection reveals nothing)
   c. Scope of inspection program (before and after re-assessment)
   d. Prioritizing
2. Assessment based on reliability analysis, including prior history
3. Size and distribution of weld defects, including spatial distribution

E. CODE REQUIREMENTS / TECHNOLOGY TRANSFER

1. US/Euro/IIW criteria
2. Ultimate strength formulation based on current test results: multiplier factors
3. Adoption of reduced K factors based on Hurricane Andrew JIP
4. $Q_u$
5. $Q_f$
6. Classification of joints

Day 2: The second day of the workshop began with presentation and general discussion of the lists of technology development needs. The workshop participants then formed working groups to discuss the five major areas. Each working group was charged with discussing the topics within their assigned theme, adding or deleting topics as necessary, then identifying the most important issues and prioritizing the topics where possible. "Important" topics were defined as those needing immediate development to improve the state-of-practice.

Following the group discussions, the entire workshop group re-convened, and each working group presented its findings. These findings are summarized in Chapter 2. The workshop participants then discussed how the workshop findings could be implemented. Specifically, the discussion centered on what steps should be taken to enact technology development programs in the most important areas identified during the workshop. A summary of these discussions is presented in Chapter 3.

## 2. Working Group Discussions

Following the working group discussions, each group presented its findings to the assembled workshop participants. These findings were reported in various formats, and are reproduced here exactly as they were presented by each group. A summary of the working group findings and the conclusions reached by the workshop are found in Chapter 4. The time available for working group discussion was limited, and most groups reported that they had insufficient time to thoroughly discuss all topics on their list. Nonetheless, at a minimum each group was able to identify the most important issues in their area, and some groups were able to prioritize these issues.

### Working Group A: JOINT CHARACTERIZATION

Group A presented an outline of technology development needs. The outline was divided into three categories: Joint Strength, Load-Displacement Behavior (monotonic), and Hysteretic Behavior. Items which were added during the working group discussions are shown in italics. Items requiring the highest priority are surrounded by a box.

A. Joint Strength
- 1. Simple joints
  - a. Frame effects
  - b. Bias and uncertainty in strength
  - c. Can length de-rating
  - d. *Multiplanar effects*
  - e. *Joint classification*
  - f. Tensile fracture failure mode
- 2. Complex Joints
  - a. Grouted joints
  - b. *Multiplanar effects*
  - c. *Overlap joints*
- 3. Damaged joints
  - a. Dented joints
  - b. Cracked joints
  - c. Corroded joints
  - d. Strength of grind-repaired joints

B. Load-displacement behavior (monotonic)
- -Joint flexibility/ductility: P-$\delta$, and M-$\Theta$

C. Hysteretic behavior
- 1. Cyclic P-$\delta$, M-$\Theta$ stiffness
- 2. Multi-cycle ultimate strength
- 3. Plastic deformation cyclic damage

Working Group B: COMPUTATIONAL METHODS

Group B presented a list of needed technology developments, ordered from most to least important. The group also made the general observation that there is the need to "Assess the benefits vs. the costs of development and use of computational methods."

1. Do data exist to validate computer codes? - Identify gaps.
1A. What test results are specifically required?

2. Assessment of applicability of established and emerging computational tools for analysis of tubular steel joints and systems.

3. Determine benefits of generalized EPFM (Elastic-Plastic Fracture Mechanics) methodology for tubular steel joints.

4. Develop industry-oriented EPFM methodology and tools.

Working Group C: FAILURE DEFINITION / CONDITION

Group C presented a list of needed technology developments, ordered from most to least important.

1. Assessment of important failure modes and types of damage

2. Development of computational tools

3. Validation with testing

4. Development of field-usable tools for practicing engineers

Working Group D: CONDITION ASSESSMENT

In the opinion of working group D, establishment of an overall probabilistic framework for condition assessment is a research priority. The elements of this framework are in various stages of development or usage.

1. Pre-damage assessment serves as a benchmark of comparison for future inspections.
Ideally it would also provide probability distributions of initial, pre-service flaws. Construction inspection typically would only provide maximum acceptable flaw sizes. For existing structures, industry practice is to define the service life as the actual number of years in service, rather than attempting to calculate an "adjusted service life" which would account for the fatigue damage resulting from the known load history experienced by the structure.

2. Probability of flaw detection of various inspection tools.
There is concern that in-situ flaw detection capability of existing tools is insufficient to reliably update the residual strength of an offshore structure. Also, there is a need for clearer definitions of what size flaw to look for, e.g., what is the critical crack size at a given joint for a 100 year storm?

3. Scope and frequency of inspection.
The scope and frequency of inspection can be established from the criticality of a joint (provided by global structural analysis) and a uniform standard of reliability for the entire structure. Bayesian updating is a useful tool in this context, but is hampered by uncertainties in

the stress history of joints and the low probability of detection afforded by current inspection tools.

## Working Group E: CODE REQUIREMENTS / TECHNOLOGY TRANSFER

Group E presented a list of needed technology developments, ordered from most to least important.

1. Ultimate strength guidelines
   - K factor
   - $F_y$ mean
   - Formulation

2. Grouted joints guidelines

3. Damaged joints
   - Dents
   - Cracks/grinding guidance

4. Comparison of codes
   - Graduate student with direction - possible joint industry project

5. K-joint can length reduction, if real, must be addressed as soon as possible

6. White paper
   - Classification vs. $\alpha$

7. Fracture mechanics

## 3. Summary and Conclusions

Needed Technology Developments:

Based on the findings of the five working groups, and the workshop discussions which followed, high priority areas for technology development were identified. Only the most urgent issues are listed here. Other important, but less critical, issues were identified by the working groups, and are reported in Chapter 3.

*Joint Characterization:*

Methods for describing analytically both the monotonic and cyclic behavior of joints need to be improved. There is still inadequate experimental data, (or in some cases, simply a lack of analysis of existing data) to enable reliable prediction of the load-deflection behavior of many tubular joint configurations. This is particularly true for joints subject to cyclic loads. To date, even the most sophisticated nonlinear finite element analyses cannot adequately model the degradation of stiffness, strength and energy dissipation capacity of joints subject to multiple inelastic load reversals. The most practical approach to this problem appears to be the development of joint macro-models, capable of approximating the hysteretic behavior exhibited by joints. Such models could be calibrated based on high-amplitude cyclic load tests of selected joint configurations.

Another area requiring immediate attention is the tensile fracture failure mode of tubular joints. Current methods are inadequate for estimating the susceptibility to fracture, or the remaining life to fracture, of a given joint. Prediction of tensile fracture is closely related inspection methods, which are discussed later.

*Computational Methods:*

There should be a coordinated effort to assess the body of available experimental data on tubular joints, and to determine what additional data is required to calibrate the current generation and next generation of computer analysis codes.

Investigation of Elastic-Plastic Fracture Mechanics methods may lead to more efficient computational techniques.

Prior to further extensive development of analytical tools, an effort should be made to assess the cost vs. benefits of developing specific tools. Although such a study would in itself be costly, the cost would be more than offset by savings in software development expenses and associated laboratory tests required for calibration.

*Failure Definition/Condition:*

There is not yet a clear understanding of the most important failure modes and types of damage most commonly observed in tubular joints that have undergone an extreme event. This issue is fundamental. Until it is resolved, focused efforts on improving assessment methods for tubular joints should not proceed.

The need to develop improved computational models and experimental data for model calibration is discussed above. A guiding principle in the development of new analytical tools should be that the tools must be field-usable by practicing engineers.

*Condition Assessment:*

A probabilistic approach to condition assessment promises to maximize both the effectiveness and economy of requalification programs. However, presently there are significant barriers to probabilistic evaluation of tubular joints. First, insufficient information is available on the condition of joints when they are first placed in service. Methods for characterizing the condition of new joints are needed. Second, current methods of in-situ flaw detection are not sensitive enough to provide adequate information on the condition of aged joints. Field inspection methods must be improved before reliable assessments of joint condition in-situ can be achieved.

Third, the necessary scope and frequency of inspections should be studied. That is, the need for timely and sufficient information about the condition of joints must be balanced against the cost of conducting inspections.

*Code Requirements/Technology Transfer*

The most important developments required in design codes are improved provisions for rating ultimate strength of joints, provisions for bias on the mean yield strength of steel, especially when steel records from the original fabrication are not available, and standardized formulations for ultimate strength prediction. Provisions are also required for assessing the remaining capacity of damaged joints. A definitive investigation of can length in K-joints is needed, to determine if reductions of can length are possible. Joint classification schemes, formulated in terms of the ovalizing parameter $\alpha$, need to be re-examined and clarified. Finally, methods of incorporating fracture mechanics concepts into code provisions need to be explored.

## Achieving Technology Developments:

Following presentation of the working group findings, the workshop participants discussed ways to achieve the needed technological developments. Several options were proposed.

Cooperative arrangements between private industry participants, i.e. Joint Industry Projects (JIPs), have traditionally been the most expedient method of investigating specific topics. JIPs are mutually beneficial to the participants involved because costs are shared. However, the findings of JIPs are usually available only to the paying participants, so the studies do not benefit the industry as a whole, unless there is broad participation. A real concern expressed by workshop participants is that research budgets at the major petroleum companies are dwindling. As a result, less in-house research and development is being conducted, and fewer JIPs are being funded. Third party contractors cannot normally afford to independently fund research and development.

In some areas of technology development identified by the working groups, it may be possible for industry to pool information which is already available but proprietary, with little or no exchange of funds. For example, under "Computational Methods" the working group determined that there is a need to take a broad look at the available experimental data, to determine what additional data is needed to calibrate current and future computer codes. If experimental data from proprietary sources was pooled, the aggregate data could be used to improve the calibration of computer codes, to the general benefit of the industry. While some proprietary data has already been pooled, there could be further coordination in this area.

Research assistance from agencies of the federal government was also discussed. In the current budget-reduction climate, extensive research funding from federal agencies is less common. Rather, most agencies are attempting to leverage their budgets by providing initiation funds for key programs, and encouraging substantial participation from private industry. Such public-private partnerships have become common in recent years, and have the advantage of maximizing the effectiveness of public funds, while insuring that the research topics addressed are strongly supported by industry.

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## Appendix B
## Workshop Agenda

### Workshop on Re-qualification of Tubular Steel Joints in Offshore Structures, Houston, September 5 and 6, 1995

Meeting place: Exxon Production Research, Annex 3, Room 3143

### Tuesday, September 5

11:30 Registration. Informal lunch served at the meeting room.

12:50 Welcome and introduction of participants - Andy Taylor, NIST

1:00 Opening remarks - H.S. Lew, Chief, Structures Division, NIST

1:05 Opening remarks - Charles Smith, Research Program Manager, Offshore Minerals Management, Minerals Management Service

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1:10 Presentations - chaired by Andy Taylor, NIST

1:15 Peter Marshall
MHP Systems Engineering
"Overview of Tubular Connection Issues in Offshore Platform Requalification"

1:40 Joe Kallaby/Kris Digre
Offshore Structures, Inc./Shell Offshore, Inc.
"API RP 2A, Section 17, Assessment of Existing Platforms, Current Criteria and Future Needs"

1:50 Adrian Dier
MSL Engineering Limited
"Summary of MSL Activities Related to Requalification of Tubular Joints"

2:15 Minaz Lalani
MSL Engineering Limited
"Summary of Current ISO Provision for Requalification of Tubular Joints, and Future ISO Directions"

2:25 Helen Bolt
BOMEL Engineering Consultants
"R&D Priorities for Tubular Joints"

2:50 Break

3:00 Yoshiaki Kurobane
Kumamoto University
"Ultimate Behaviour and Design of Multi-Planar Tubular Joints"

3:25 Einar T. Moe
Det Norske Veritas
"Summary of Past and Current Research on Tubular Joints at DNV"

3:50 Mel Kanninen
MFK Consulting Services
"An Assessment of Elastic-Plastic Fracture Mechanics for Requalifying Tubular Steel Joints in Offshore Structures"

4:15 William Mohr/Roger Thomas
Edison Welding Institute/Phillips Petroleum Company
"API Offshore Tubular Joint Research Center"

4:30 Preliminary discussions to establish major themes for following day:
- Workshop participants suggest areas of needed technology development related to requalification of tubular steel joints in offshore structures.
- A preliminary list of the suggested topics will be organized and presented the following morning.

5:15 Workshop adjourns for the day.

6:30 Dinner at Los Andes restaurant. Cost approximately $20.00, plus bar.

**Wednesday, September 6**

9:00 AM Workshop re-convenes
Moderated by Andy Taylor, NIST

9:10 Present preliminary list of needed technology developments related to requalification of tubular steel joints in offshore structures, suggested at the end of the previous day.

9:20 Working group discussions to refine the list of needed technology developments. Define major areas and sub-areas.

10:45 Working group discussions to prioritize major areas of needed technology development.

11:30 Finish working group discussions on needed technology developments and prioritization. Reconvene entire workshop group.
Begin working lunch.

11:40 Discuss steps to be taken to enact technology development programs in the most important areas identified above

12:45 Summary and concluding remarks.
- Charles Smith, MMS
- H.S. Lew, NIST
- Andy Taylor, NIST

1:00 Workshop adjourns.

## Appendix C
## Invited Papers

Nine papers were presented on the first day of the workshop:

Peter Marshall, MHP Systems Engineering
"Overview of Tubular Connection Issues in Offshore Platform Requalification"*

Joseph Kallaby/Kris Digre, Offshore Structures, Inc./Shell Offshore, Inc.
"API RP 2A, Section 17, Assessment of Existing Platforms, Current Criteria and Future Needs"*

Adrian Dier, MSL Engineering Limited
"Summary of MSL Activities Related to Requalification of Tubular Joints"

Minaz Lalani, MSL Engineering Limited
"Summary of Current ISO Provision for Requalification of Tubular Joints, and Future ISO Directions"

Helen Bolt, BOMEL Engineering Consultants
"R&D Priorities for Tubular Joints"*

Yoshiaki Kurobane, Kumamoto University
"Ultimate Behaviour and Design of Multi-Planar Tubular Joints"*

Einar T. Moe, Det Norske Veritas
"Summary of Past and Current Research on Tubular Joints at DNV"*

Mel Kanninen, MFK Consulting Services
"An Assessment of Elastic-Plastic Fracture Mechanics for Requalifying Tubular Steel Joints in Offshore Structures"*

William Mohr/Roger Thomas, Edison Welding Institute/Phillips Petroleum Company
"API Offshore Tubular Joint Research Center"*

Several of the speakers provided written versions of their papers, or copies of presentation materials, for inclusion in the workshop proceedings. These are indicated by an asterisk in the list above, and are presented in the remainder of this appendix.

# OVERVIEW OF TUBULAR CONNECTION ISSUES IN OFFSHORE PLATFORM REQUALIFICATION

by Peter W. Marshall
Moonshine Hill Pty.
Houston, Texas

## INTRODUCTION

As part of the ongoing requalification of offshore platforms, their welded tubular connections (commonly referred to as tubular joints) also come under scrutiny.

High-cycle fatigue aspects of this problem have already been worked in the North Sea (e.g. by Veritec). Inspections have focused on finding small fatigue cracks and require joint cleaning and magnetic particle testing. Since fatigue tests of tubular connections typically show them spending more than half their lives growing visible fatigue cracks, an inspection which shows no cracks can be used as evidence that the remaining life of the joint will be roughly equal to the life to date. Joints with short calculated fatigue lives can be given a new lease on life by this process. Supporting research has dealt with the residual strength of cracked joints, and on the beneficial effects of deep grinding to remove incipient cracks.

Fly-by inspections as practiced in the Gulf of Mexico would only find larger cracks, with perhaps only 10% of the joint's fatigue life remaining. When these inspections are performed at intervals of 5 years (with calculated lives over 50 years) they still provide assurance of structural integrity. When the platform has a high degree of structural redundancy, and is evacuated for severe storm load events, then an additional period of grace (in relation to calculated or previously demonstrated fatigue life) applies before the platform is no longer fail-safe while manned. The notion that platforms must be abandoned when fatigue calculations indicate a cumulative damage ratio of 1.0 no longer applies.

Thus, we are left with overload failure as the principal area requiring discussion. We are interested in the behavior of tubular joints, and their role in overall structural response and capacity with respect to:

reserve strength ratio (RSR) under wave loading,
ductility and residual strength under earthquake loading, and
cyclic loading characteristics for both applications.

## CONTEXT OF MEMBER AND FRAME BEHAVIOR

Tubular beam-column struts are the primary lateral load resisting elements of offshore jackets. They are subjected to axial loads due to global spaceframe action, and bending moments due to local transverse loads. Portal action of beam columns occurs in laterally loaded piles, unbraced deck legs, and in the jacket legs as part of the final collapse mechanism (after the failure of cross bracing).

Tubular struts have considerable reserve strength beyond traditional first yield criteria (Fig. 1). While axial capacity is still affected by column buckling, it can benefit from more realistic (rather than conservative) assessment of KL/R. Bending (lateral capacity) benefits from the Z/S plastic shape factor, and redistribution of moments. The failure envelope benefits from the favorable arc-sine interaction of fully plastic sections. This behavior has been confirmed by Sherman's tests (Fig. 2), but with some caveats regarding high strength steels and local buckling.

The post-buckling behavior of tubular struts has also been studied extensively. Analytical models (e.g. USFOS) consider the effects of yielding, local buckling, and even prior damage, in deriving behavior of the collapse mechanism from the properties of the plastic hinges (Fig. 3).

The hysteretic behavior under cyclic loading is more difficult to analyze, and phenomenological models have been developed from extensive series of tests. This has been done for both struts (Figs. 4 and 5) and portal beam-columns (Figs. 6 and 7). As the number of necessary empirical parameters began to grow beyond reason, more fundamental and robust models based on flow rules in the plastic hinges have been developed.

Global tools such as for inelastic structural analysis of soil-pile-structure systems have been developed, e.g. INTRA (Fig. 8) and its successors. Some of these have been calibrated against monotonic tests (BOMEL) and cyclic tests (Figs. 9 and 10). Early applications were for designing new structures for earthquake overload (Fig. 11); here the problems of tubular joints were taken care of by a "strong joints" philosophy, in which the joints were designed not to fail first, so that plasticity and load redistribution would take place in the members. They have also been used for after-the-fact analysis of platforms which have been exposed to wave forces well in excess of what they were designed for (Fig. 12). As these tools are coming into wider use for requalification of existing platforms, the need for better modeling of weak joints has become apparent.

## FOCUS ON TUBULAR CONNECTIONS

We begin with some useful definitions of the geometric properties of tubular connections (Fig, 13). Tau and gamma define the importance of joint-can thickness, while beta, eta, theta, and zeta define the topology of the connection.

Various modes of failure will now be illustrated: We begin with local failure, also known to designers as punching shear (Figs. 14 and 15). Various modes of general collapse include chord ovalizing, beam bending, beam shear, etc. (Fig. 16). Unzipping refers to progressive failure of welds which do not match the strength of sections joined (Fig. 17). Lamellar tearing is a problem with highly restrained welds on susceptible material, but has also been observed in severe overloads (Fig. 18). Failure by brittle fracture (Fig. 19) is also rare, due to widespread use of notch tough materials and simple joint-can reinforcement over the last 20 years. Low-cycle fatigue failure can be illustrated by a cross joint which failed after five cycles of overload at ductility ratios up to four.

While the elastic behavior of tubular joints is well predicted by shell theory and finite element analysis, there is considerable reserve strength beyond theoretical yielding, due to triaxiality, plasticity, large deflection effects, and load redistribution (Fig. 20). Practical design criteria make use of this reserve strength, placing considerable demands upon the notch toughness of joint-can materials. Through joint classification (API) or an ovalizing parameter (AWS), they incorporate elements of general collapse as well as local failure. The resulting criteria may be compared against the supporting data base of test results to ferret out bias and uncertainty as measures of structural reliability (Fig. 21). Here K, T/Y, and X joints in compression show a bias on the safe side of 1.35 beyond the nominal safety factor, and tension joints appear to show a larger bias of 2.85; however, this reduces to 2.05 for joints over 0.12-in, and 1.22 over 0.5-in, suggesting a possible size effect for tests which end in fracture.

For overload analysis, we need not only the ultimate strength, but also the load-deflection behavior. Early tests showed ultimate deflections of 0.03 to 0.07 chord diameters, giving a typical ductility of 0.10 diameters for a brace with weak joints at both ends. As more different types of joints were tested, a wider variety of load-deflection behaviors emerged (Fig. 23), making generalization more tenuous.

Cyclic behavior raises additional considerations. One issue is whether the joint will experience a ratcheting or progressive collapse failure, or will achieve stable behavior with plasticity contained at local hotspots, a process called 'shakedown' (Fig. 24) as in shakedown cruise. While tubular connections have withstood 60 to several hundred repetitions of load in excess of their nominal capacity, a conservative analytical treatment is to consider that the cumulative plastic deformation or energy absorption to failure remains constant, with sudden removal once this is exhausted (Fig. 25).

While most recent offshore fatigue research has focused on the high cycle regime, low cycle fatigue has been studied and tested in Japan as relevant to earthquakes. Below about 1000 cycles, the fatigue strength can be expressed in relation to ultimate strength, with cycle ratio (min/max) as a variable (Figs. 26 and 27). Above 1000 cycles, these results merge with AWS/API hotspot criteria, with higher strength steels taking a bigger hit (in relation to ultimate) from fatigue. Proposed S-N curves with a log-log slope of -3.0 are potentially unconservative here.

## JOINTS IN FRAMES

When tubular joints and members are incorporated into a spaceframe, the question arises as to whether computed bending moments are primary (i.e. necessary for structural stability, as in a sidesway portal situation, and must be designed for) or secondary (i.e. an unwanted side effect of deflection which may be safely ignored or reduced). When proportional loading is imposed, with both axial load and bending moment being maintained regardless of deflection, the joint simply fails when it reaches its failure envelope (Fig. 28a). However, when moments are due to imposed lateral deflection, and then axial load is imposed, the load path skirts along the failure envelope, shedding the moment and sustaining further increases in axial load (Fig. 28b).

Another area of interaction between joint behavior and frame action is the influence of brace bending/rotation on the strength of gap K-connections. If rotation is prevented, bending moments develop which permit the gap region to transfer additional load (Fig. 29a). If the loads remain strictly axial, rotation occurs in the absence of restraining moments, and a lower joint capacity is found (Fig. 29b). These problems arise for circular tubes as well as box connections, and a recent trend has been to conduct joint-in-frame tests to achieve a realistic balance between the two limiting conditions shown. Loads which maintain their original direction (as in an inelastic finite element analysis), or worse yet follow the deflection (as in testing arrangements with a two-hinge jack), result in a plastic instability of the compression brace stub which grossly understates the actual joint strength. Existing data bases may need to be screened for this problem.

In phenomenological modeling, strut parameters can be modified to account for the tubular joints (Fig. 30). If the joint is stronger than the member, we need only to increase the elastic flexibility to include that of the joints at each end. If the joints are weaker than a tension member, similar elastoplastic-fracture behavior will be found, with increased elastic flexibility but only the plastic deformation capacity of the joints. If the joints are weaker than a compression member (not shown), the degrading postbuckling behavior of the strut is replaced with the more favorable elastoplastic behavior of the joints.

A more rational (and expensive) approach would be to model every joint as a substructure in the spaceframe (Fig. 31). Nonlinear finite element modeling of the joints would need to include cyclic plasticity and large deflection effects, and needs to be calibrated against benchmark physical tests (Fig. 32).

The softening of tubular joints as they approach their ultimate load should have a beneficial effect on load redistribution within spaceframes (Fig. 33). Failure to represent this with simple elasto-plastic modeling may be why analysis tends to overpredict the stiffness and brittleness of structures (e.g. Fig. 10).

## CONCLUSION: RESEARCH NEEDS

Based upon the foregoing, a very preliminary list of research needs have been identified as follows:

- bias and uncertainty in strength
- joint stiffness and load deflection characteristics
- joint ductility under monotonic and cyclic loading
- hysteretic behavior, e.g. capacity softening
- low cycle fatigue failure
- U.S. *vs.* European *vs.* IIW criteria differences
- tensile/fracture size effect
- effect of corrosion damage to branch, weld, HAZ, or joint-can

During this workshop, it is expected that the above list will be modified, expanded, and prioritized. That is why we are here.

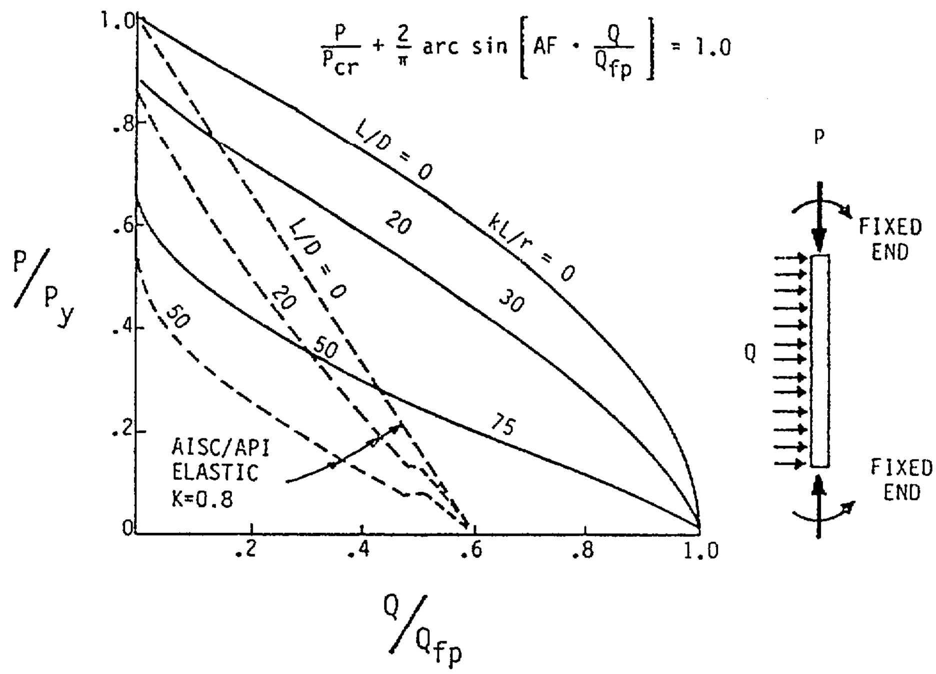



Figure 1: Beam-column interaction of tubular struts - analysis

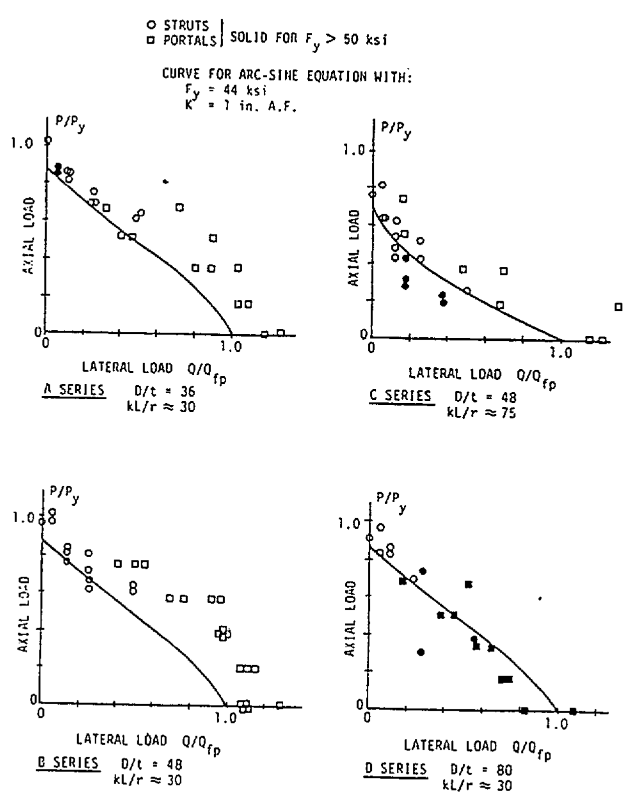



Figure 2: Beam-column interaction of tubular struts and portals - tests

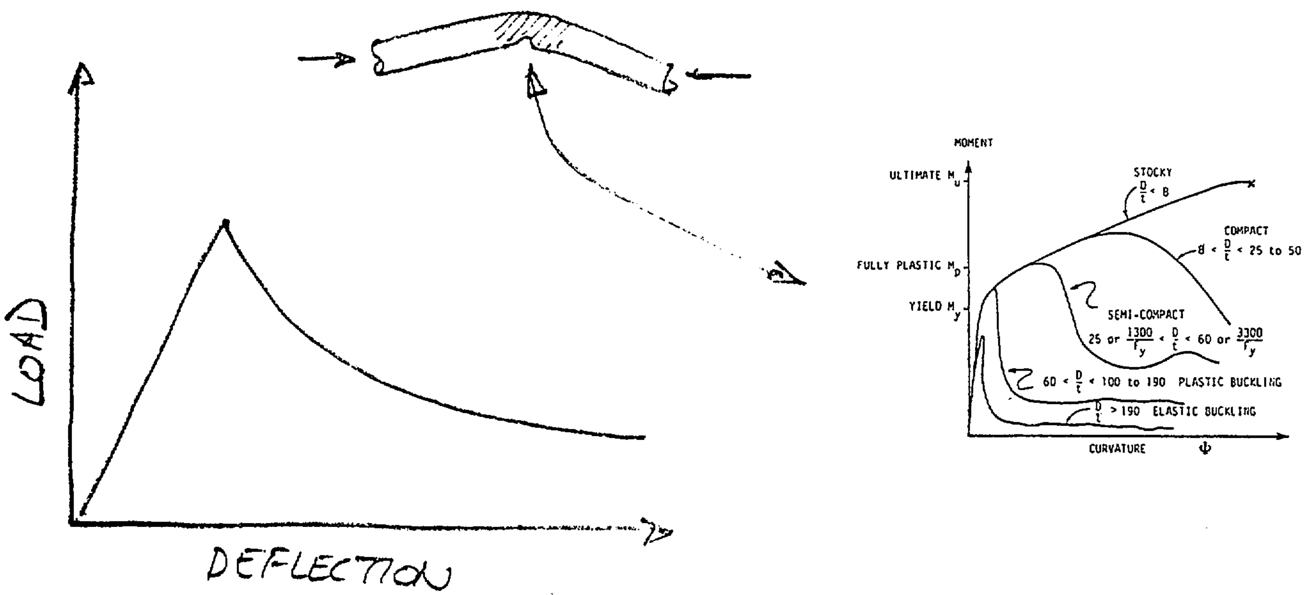



Figure 3: Inelastic behavior of tubular section with different D/t ratios

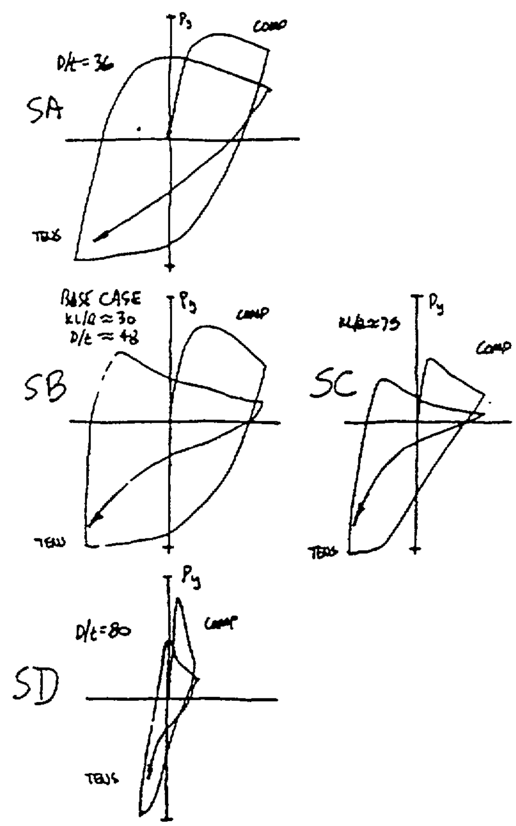
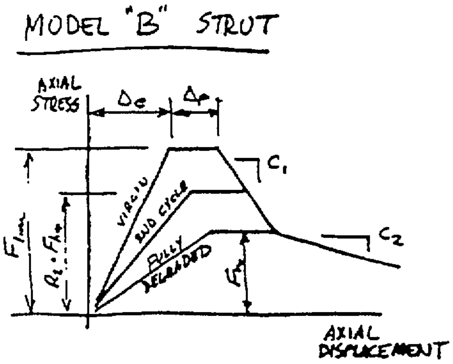



Figure 4: Axial load vs. axial displacement at largest ductility ratio tested




| SERIES | 5A | 5B | 5C | 5D |
|---|---|---|---|---|
| APPROX KL/R | 30 | 30 | 75 | 30 |
| APPROX D/t | 36 | 48 | 48 | 80 |
| PLATEAU $\Delta p/\Delta e$ | 1.6 | 1.1 | 0.5 | 0.4 |
| DEGRADATION RATIO $R_2$ | | | | |
| FOR $\mu=2$ | | .77 | .83 | .44 |
| FOR $\mu=4$ | .86 | .81 | .69 | |
| FOR $\mu=8$ | .75 | .74 | .68 | |

Figure 5: Model "B" strut

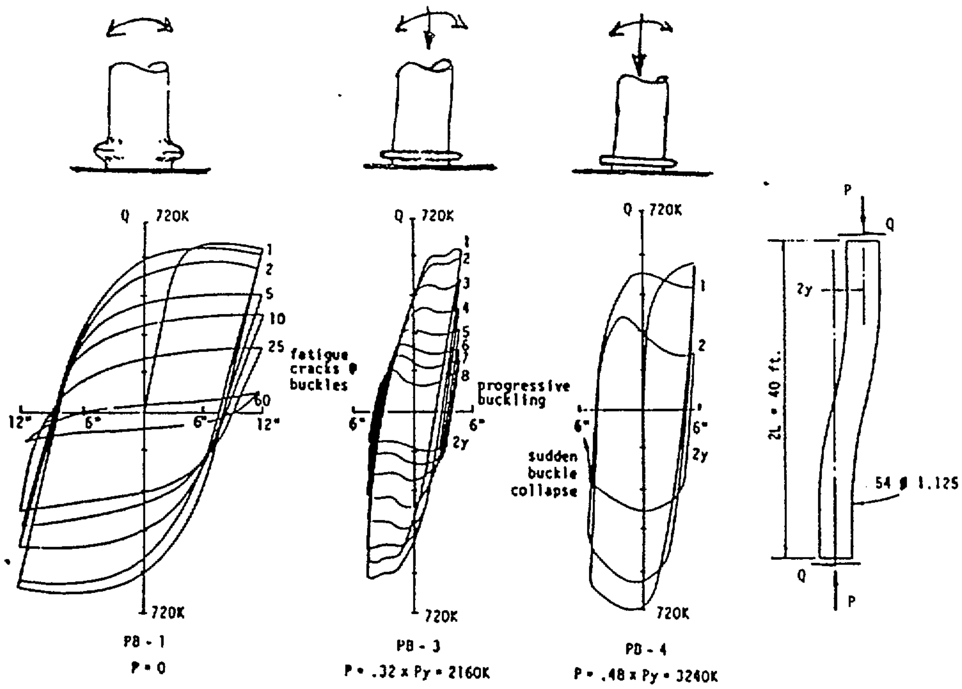



Figure 6: Cyclic shear for prototype portal beam column

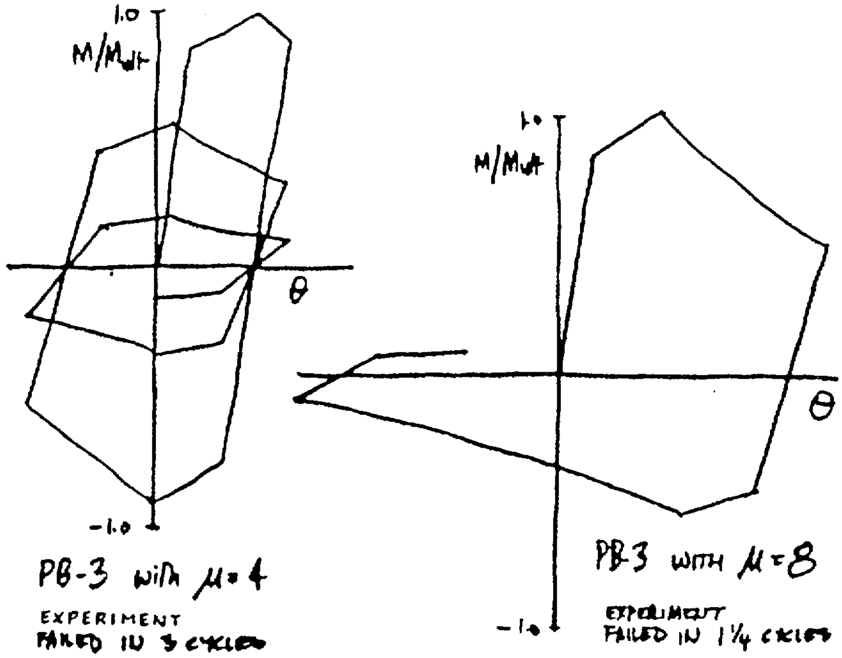



Figure 7: Modified NBEM with exponential decay

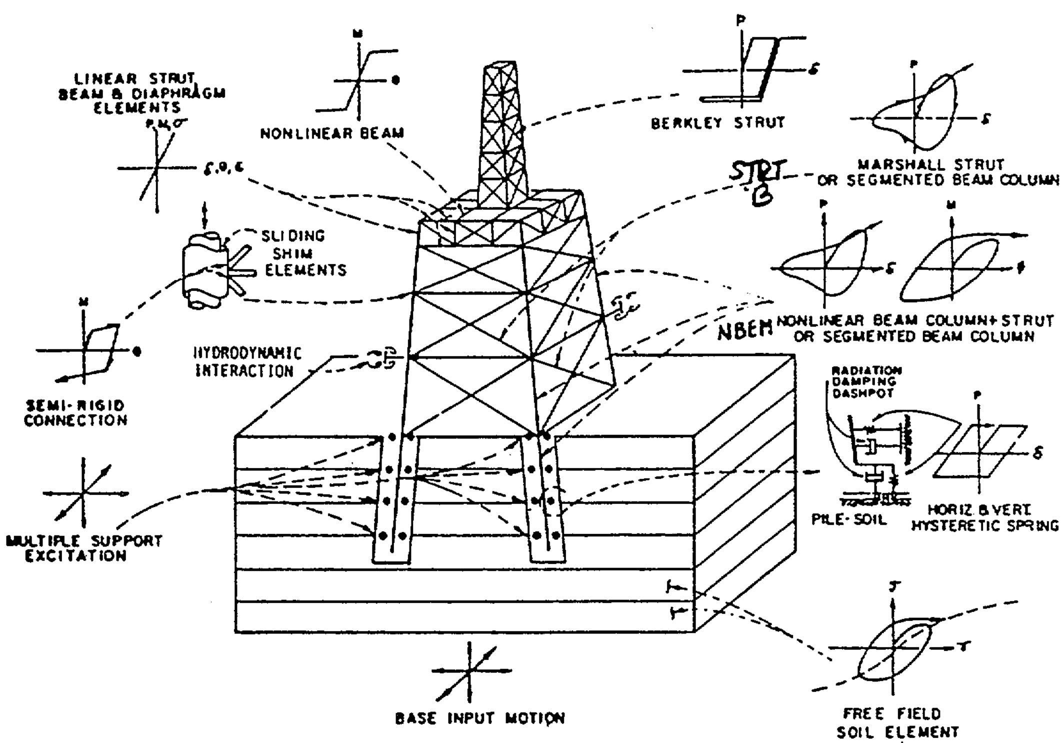



Figure 8: Finite element model of soil-pile-structure system

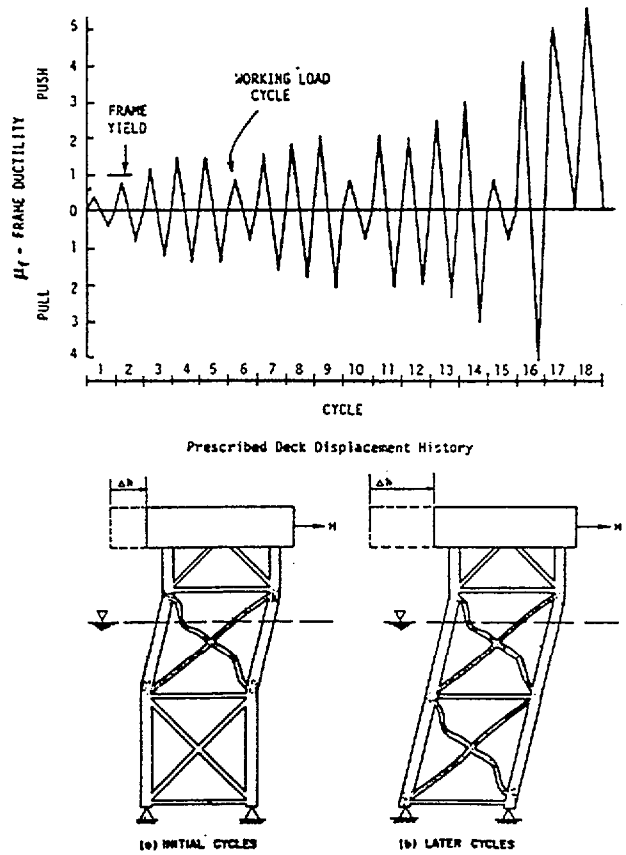



Figure 9: Results of cyclic frame test

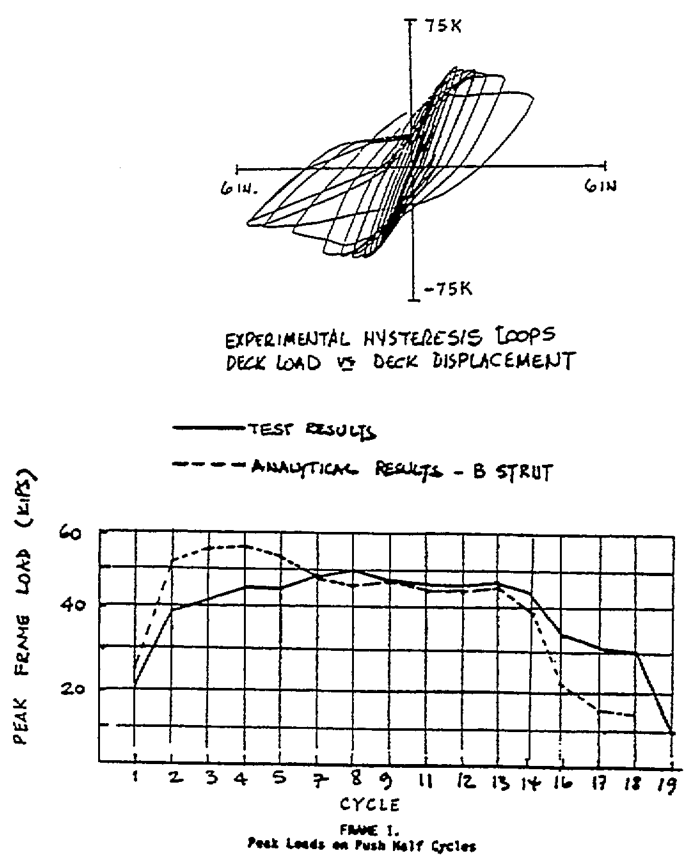



Figure 10: Results of cyclic frame test

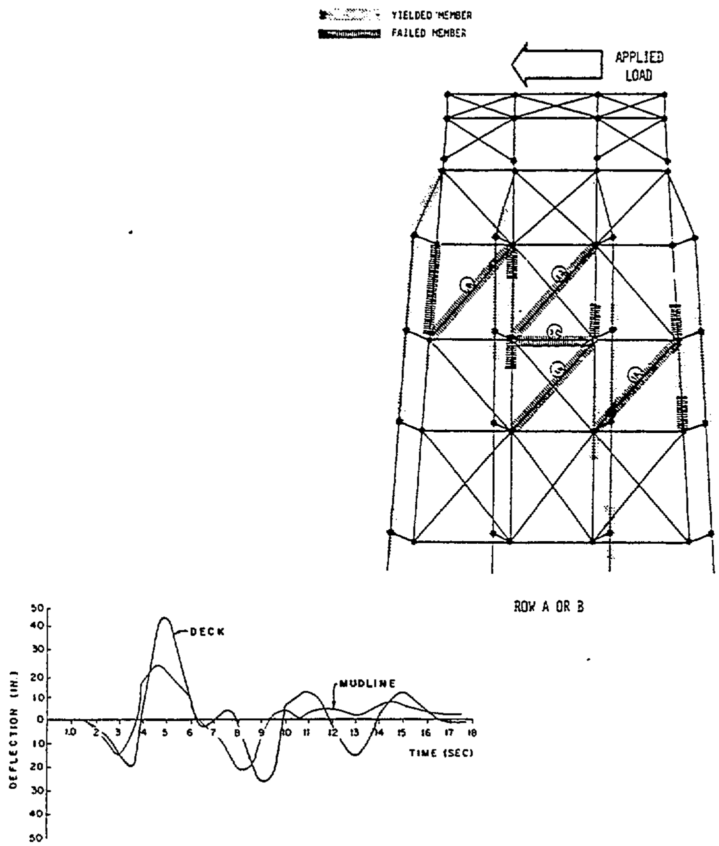



Figure 11: Seismic analysis

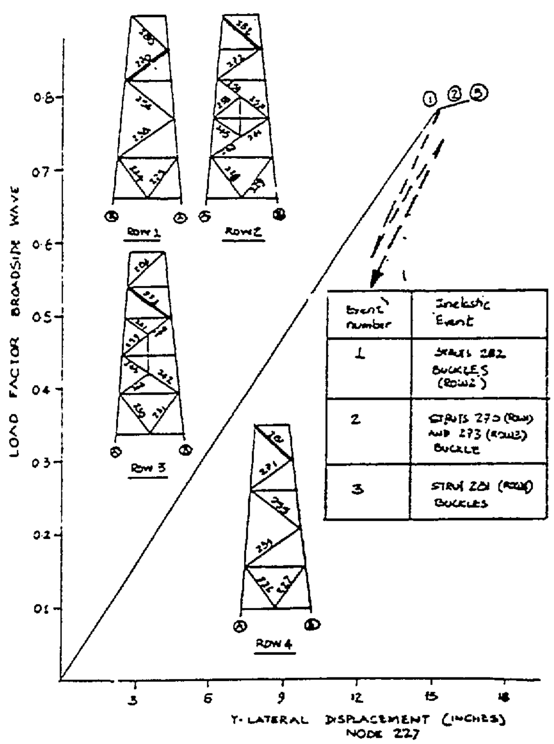
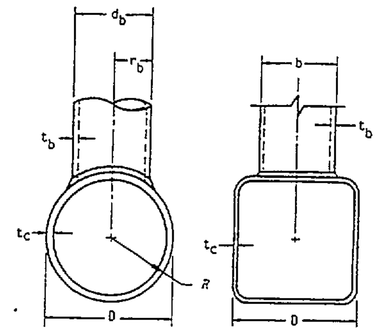



Figure 12: Response results, broadside wave




(M) Geometric parameters

| Parameter | Circular Sections | Box sections |
|---|---|---|
| $\beta$ | $r_b/R$ or $d_b/D$ | $b/D$ |
| $\eta$ | — | $a_x/D$ |
| $\gamma$ | $R/t_c$ | $D/2t_c$ |
| $\tau$ | $t_b/t_c$ | $t_b/t_c$ |
| $\theta$ | Angle between member center lines | |
| $\psi$ | Local dihedral angle at given point on welded joint | |
| $\zeta$ | g/D in X-connections (see below) | |

Note: Relevant gap is between braces whose loads are essentially balanced

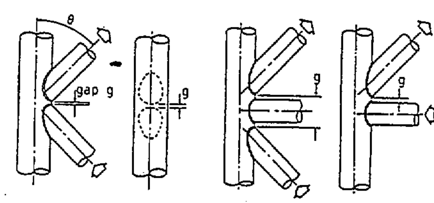



Figure 13: Non-dimensional parameters of a tubular connection

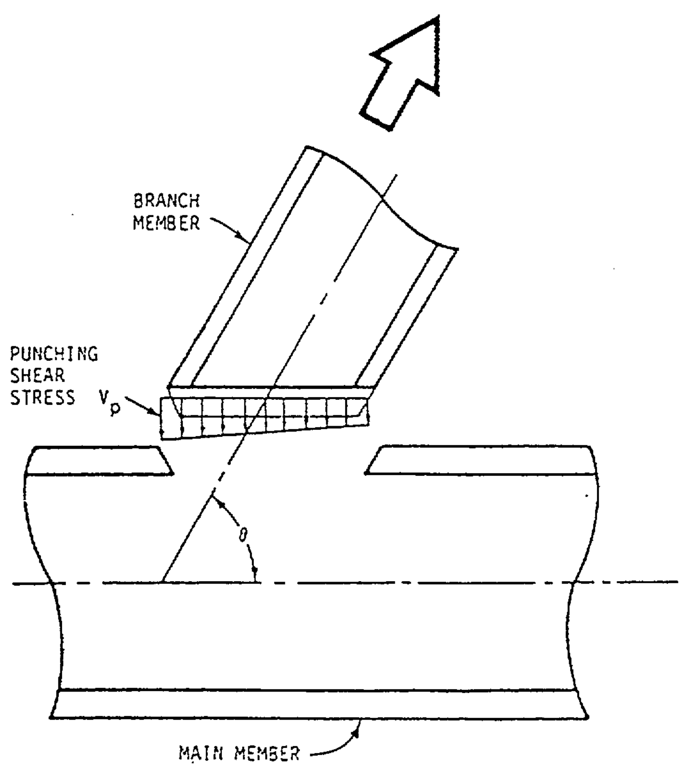



Figure 14: Simplified concept of punching shear

Figure 15: Example of local failure in service

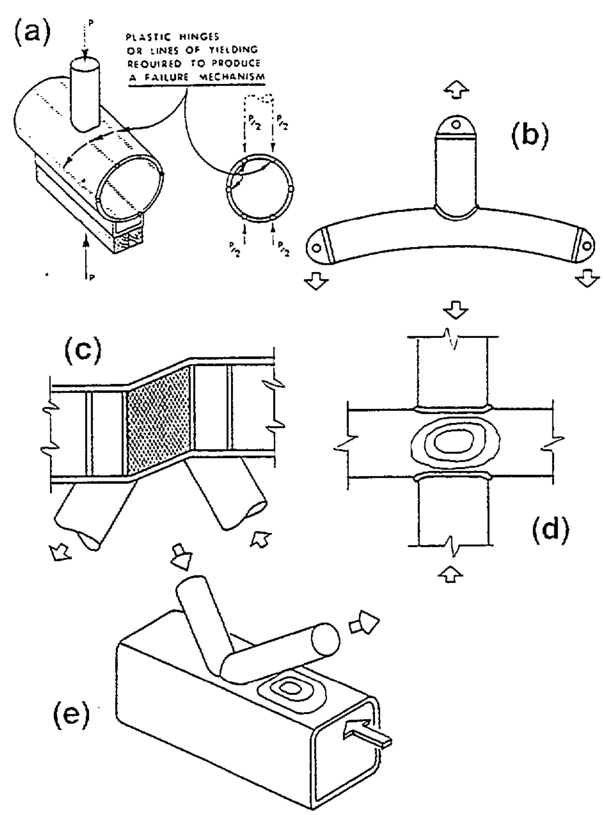



Figure 16: General collapse failure modes. (a) Ovalizing. (b) Beam bending. (c) Beam shear. (d) Web crippling. (e) Longitudinal distress.

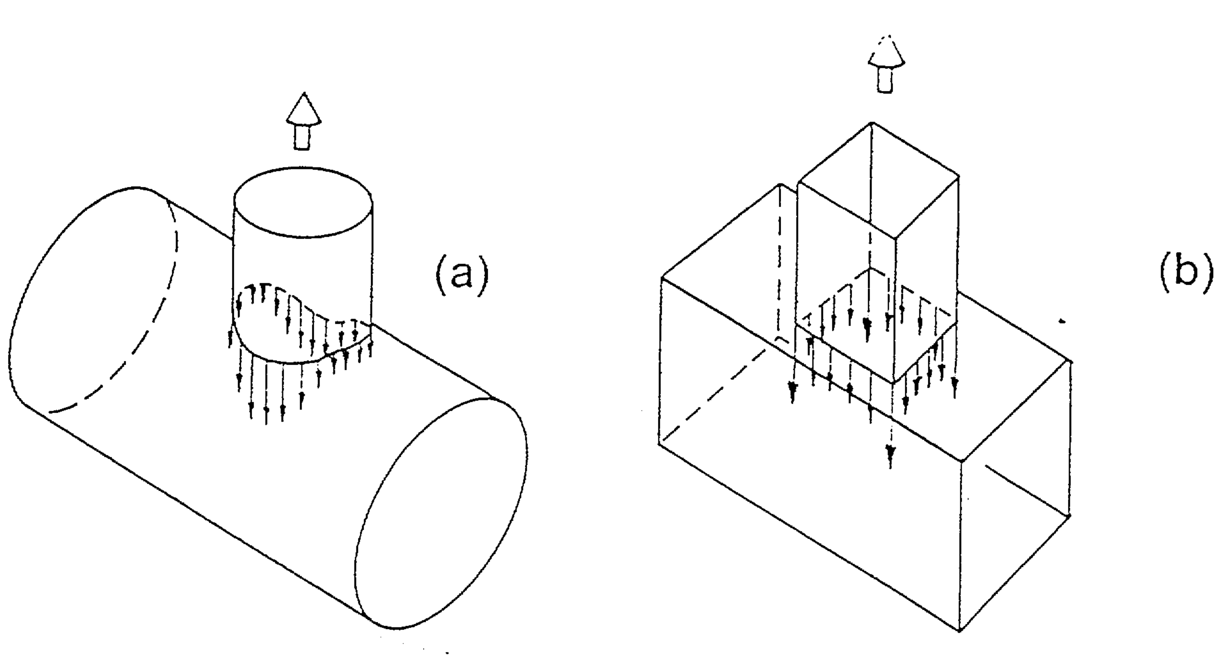



Figure 17: Uneven distribution of load. (a) Circular sections. (b) Box sections

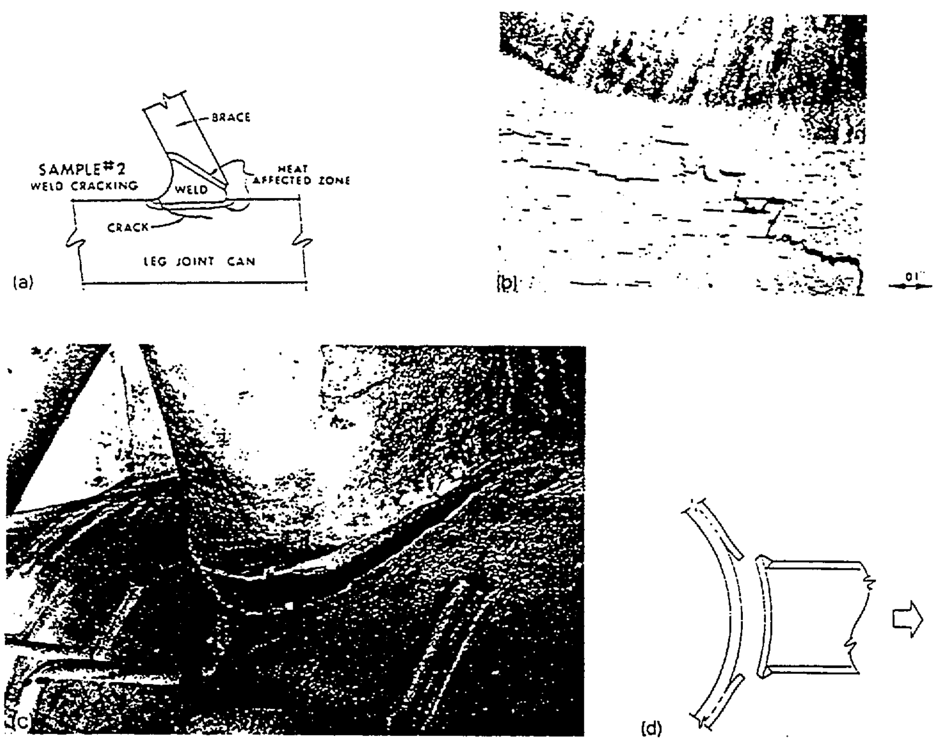
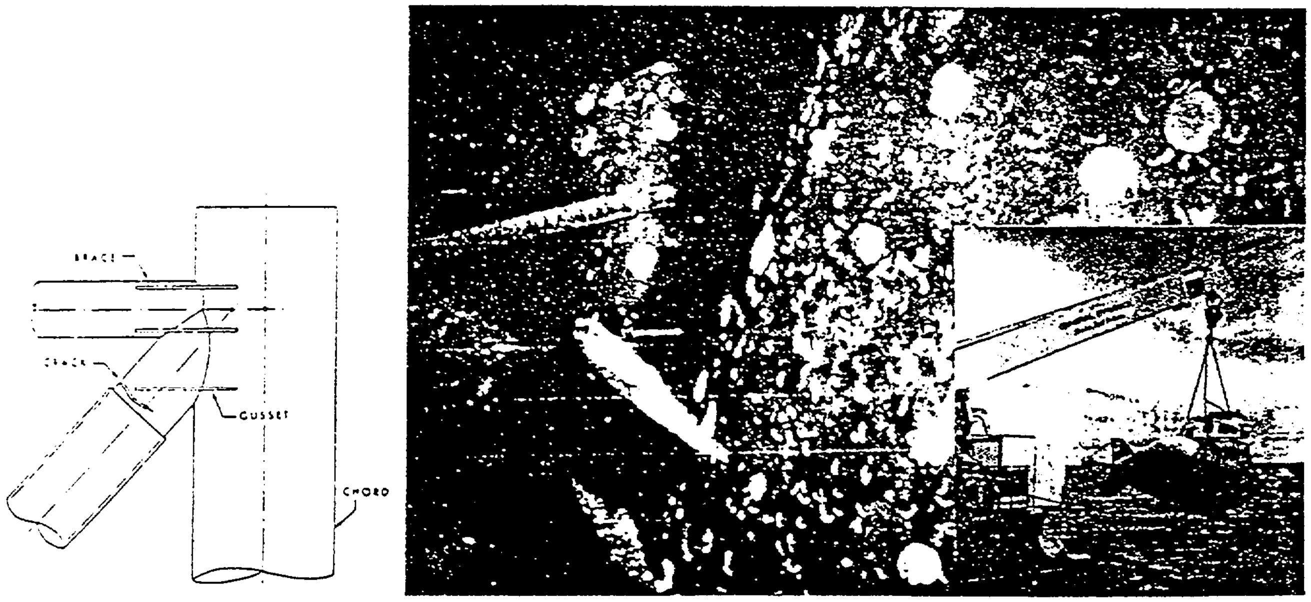



Figure 18: Lamellar tearing failures. (a) Failure during fabrications. (b) Metallographic examination. (c) Failure in service. collision overload. (d) Schematic of failure mode due to large delamination.



(a) (b)

Figure 19: Gusseted overlapping connection. (a) Brittle fracture of topside connection exposed to low air temperature. (b) Connection intact. brace severed after collision; inset shows submarine used for underwater inspection.

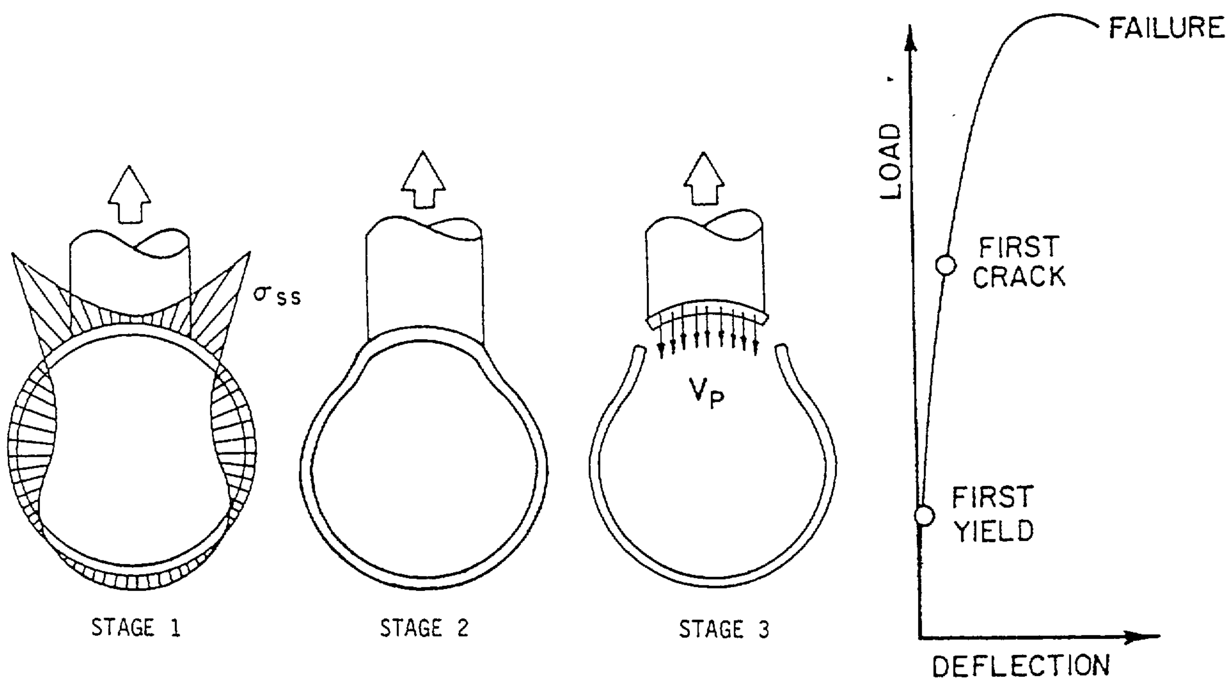



Figure 20: Stages in the overload behavior of a simple tubular connection

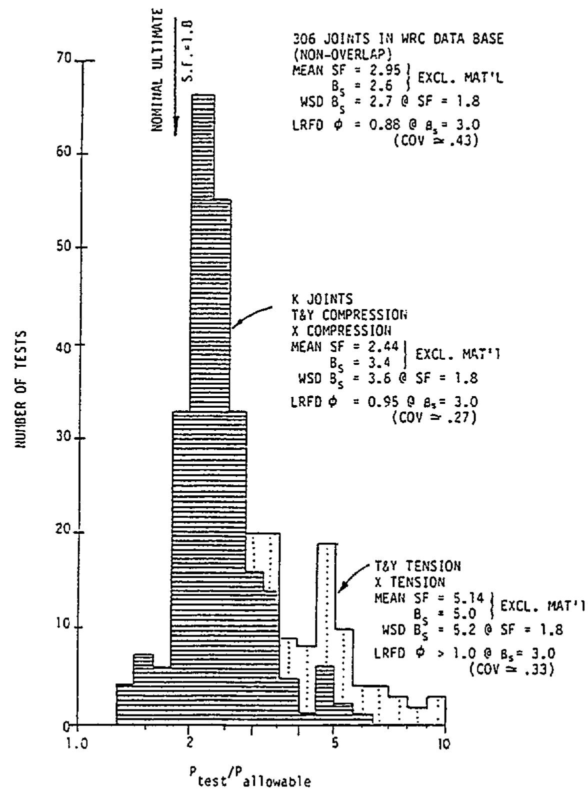



Figure 21: Correlation of AWS-84 criteria with WRC data base

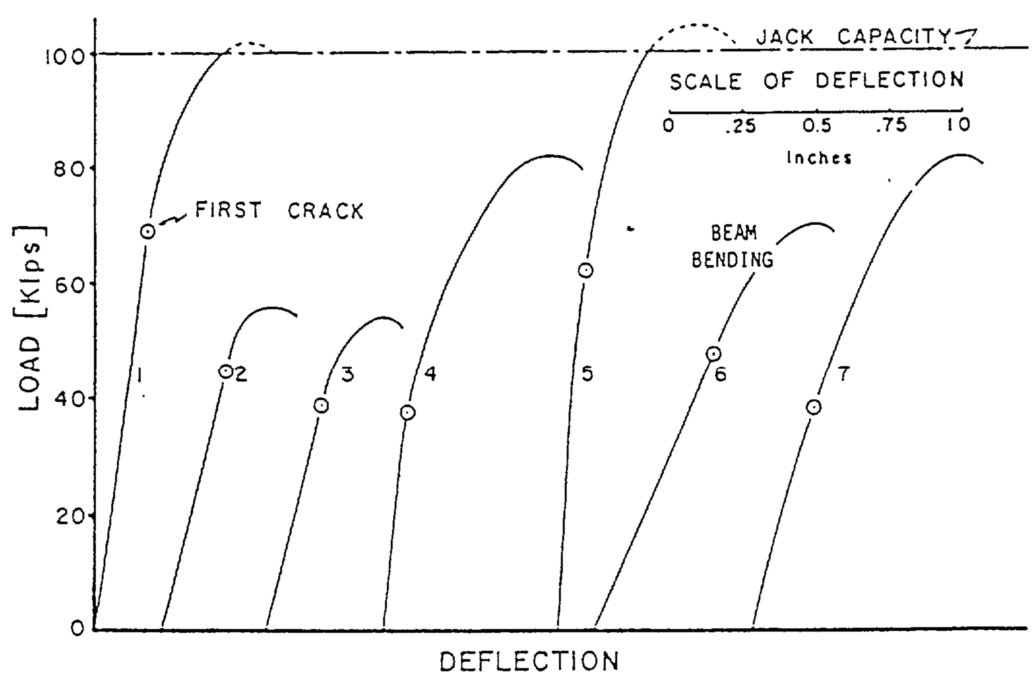



Figure 22: Load deflection plots for Toprac's tests

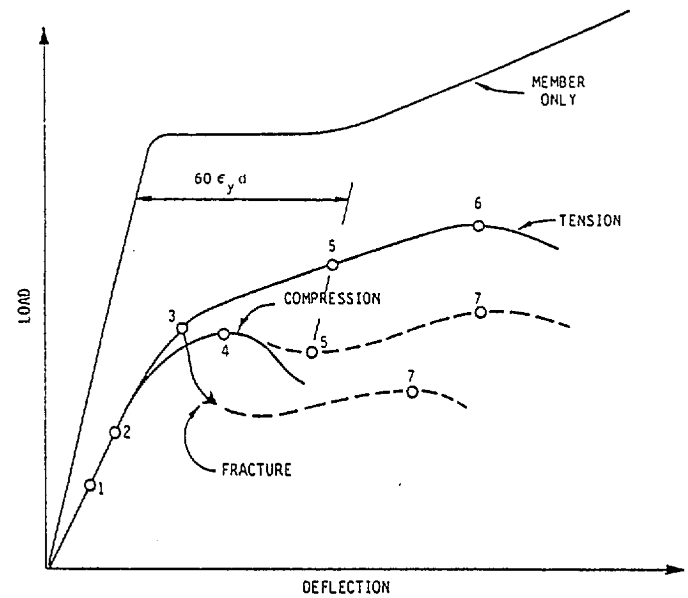



Figure 23: Load-deflection behavior and failure definitions. (1) Luder's yield lines appear. (2) Non-linear load-deflection. (3) First crack. (4) Maximum load in compression. (5) Deflection limit. (6) Maximum load in tension. (7) Second maximum.

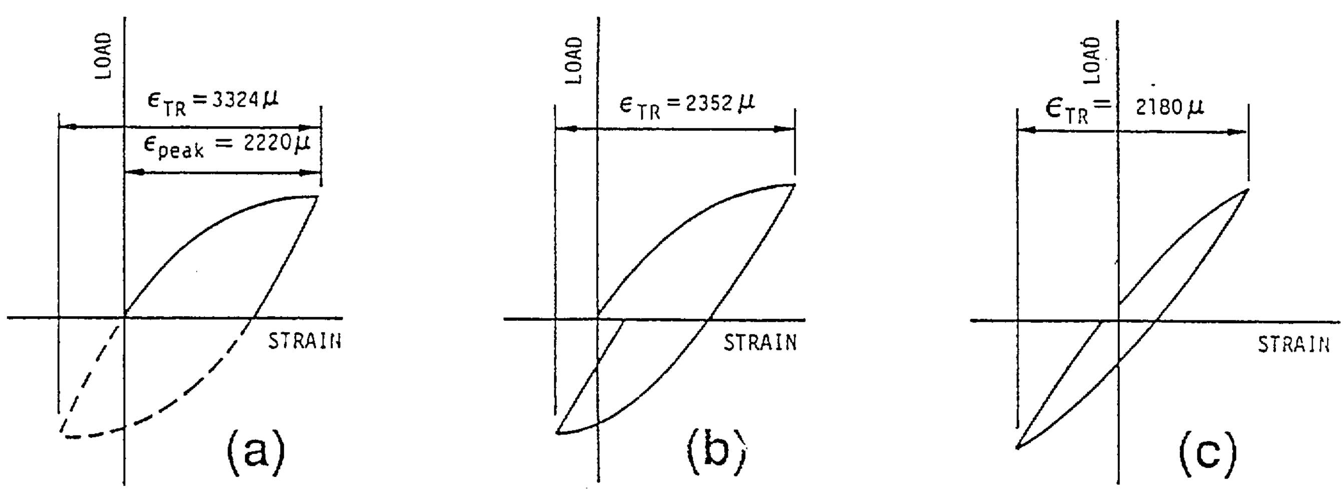



Figure 24: Three estimates of hot spot strain range from the same model tubular connection, Bouwkamp Phase II, showing the importance of "Shakedown". (a) Initial loading and unloading, as used in Bouwkamp Phase I; dashed line shows assumed symmetry about strain axis. (b) Actual full cycle shows lower strain range. (c) 70th cycle shows stabilized, closed, symmetric hysteresis loop

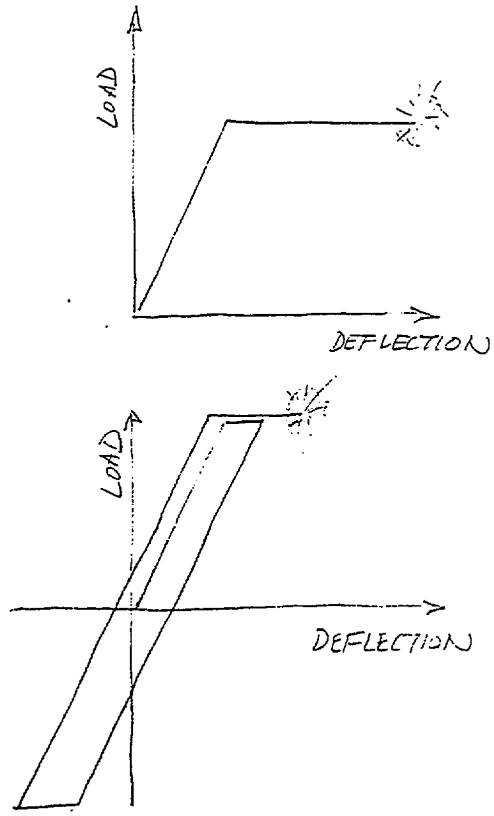



Figure 25: Failure criterion based on accumulated ductility or energy

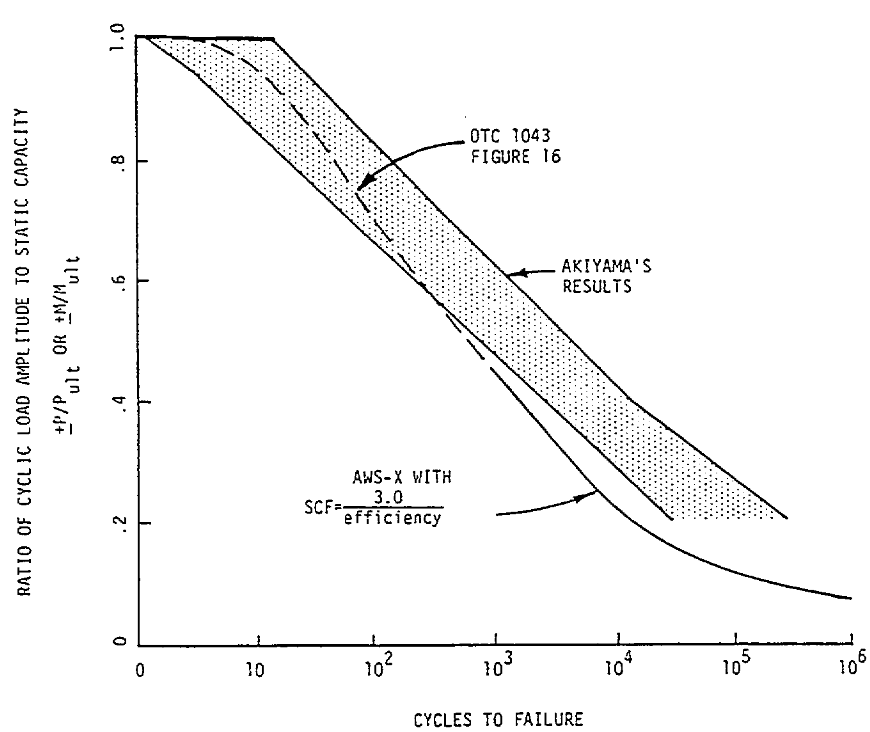



Figure 26: Fatigue criteria based on static strength efficiency

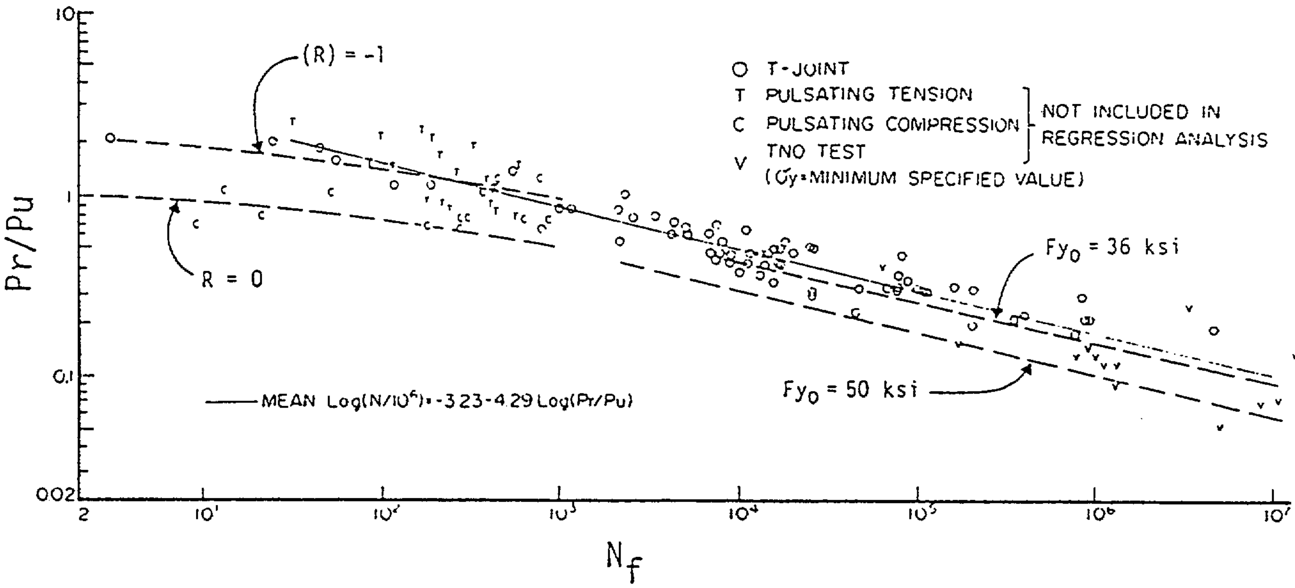



Figure 27: Fatigue test results for tubular T-joints. Fatigue strength is represented as a ratio to static strength according to Japanese rules, similar to IIW. The dashed lines represent the Marshall criteria.

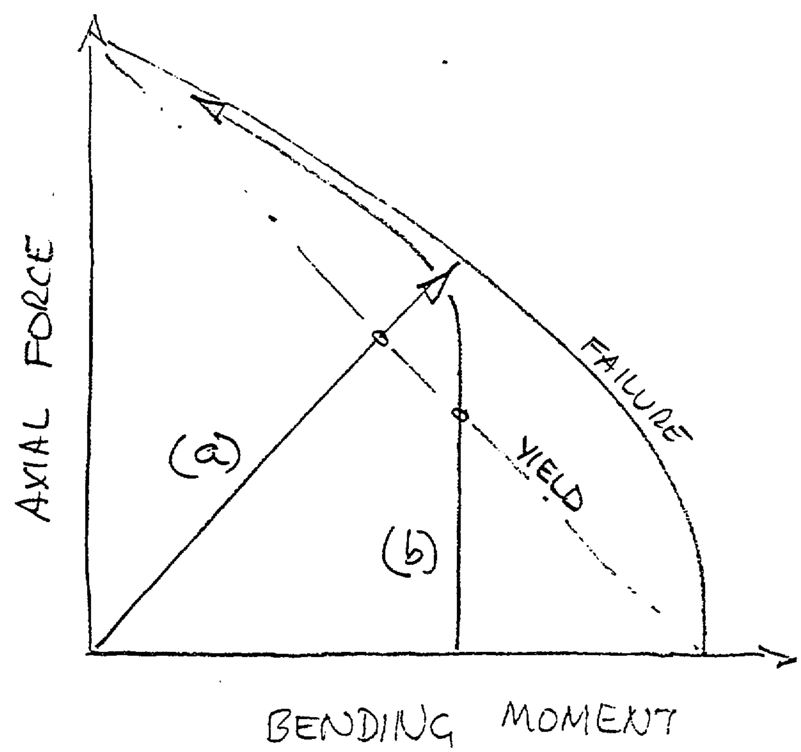



Figure 28: (a) Proportional loading, primary moments (b) Displacement induced moment, then axial load -> secondary moment is relieved

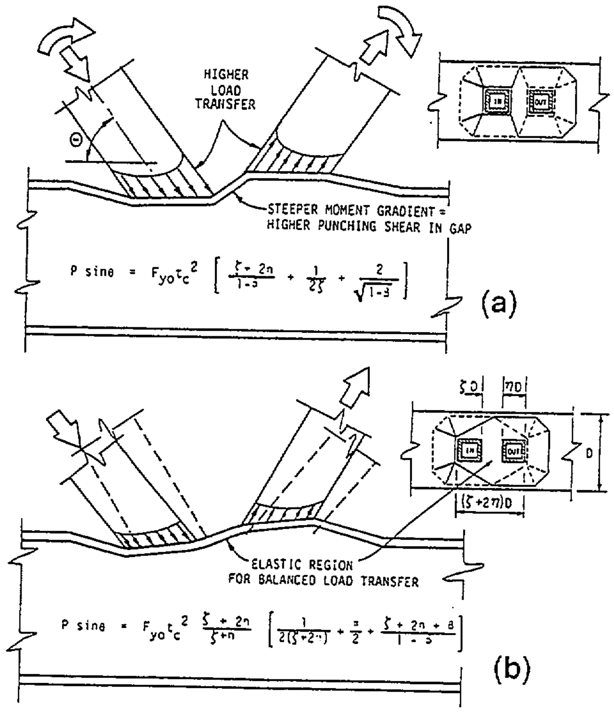



Figure 29: Alternative modes of behavior for K-connection with small gap and small β. (a) Pure axial deflection causes moments in short rigid braces. (b) Pure axial load causes rotation in long flexible braces

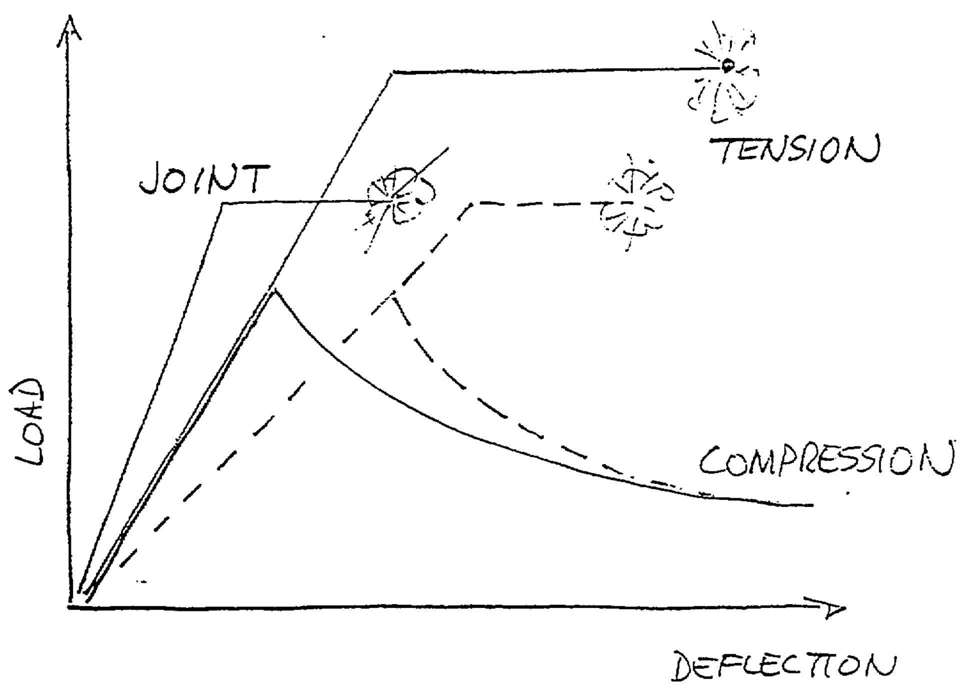



Figure 30: Effect of joints on member behavior

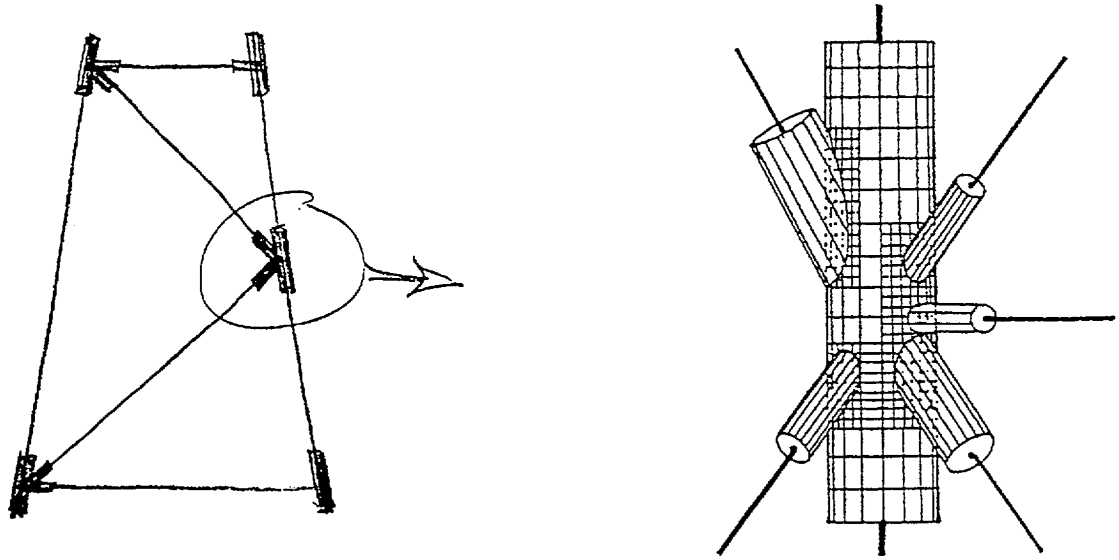


Figure 31: Joints as substructures in spaceframe model

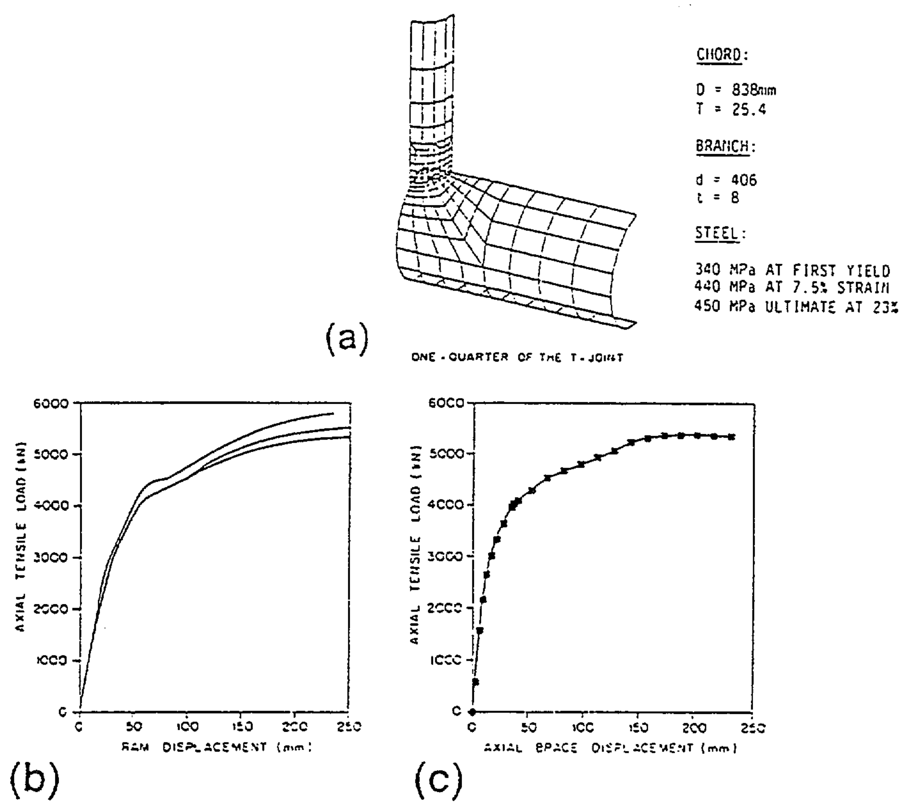



Figure 32: Inelastic finite element analysis of T-joint. (a) Mesh, MARC thin shell element type 72. (b) Three “identical” benchmark experiments. (c) Comparable analyses.

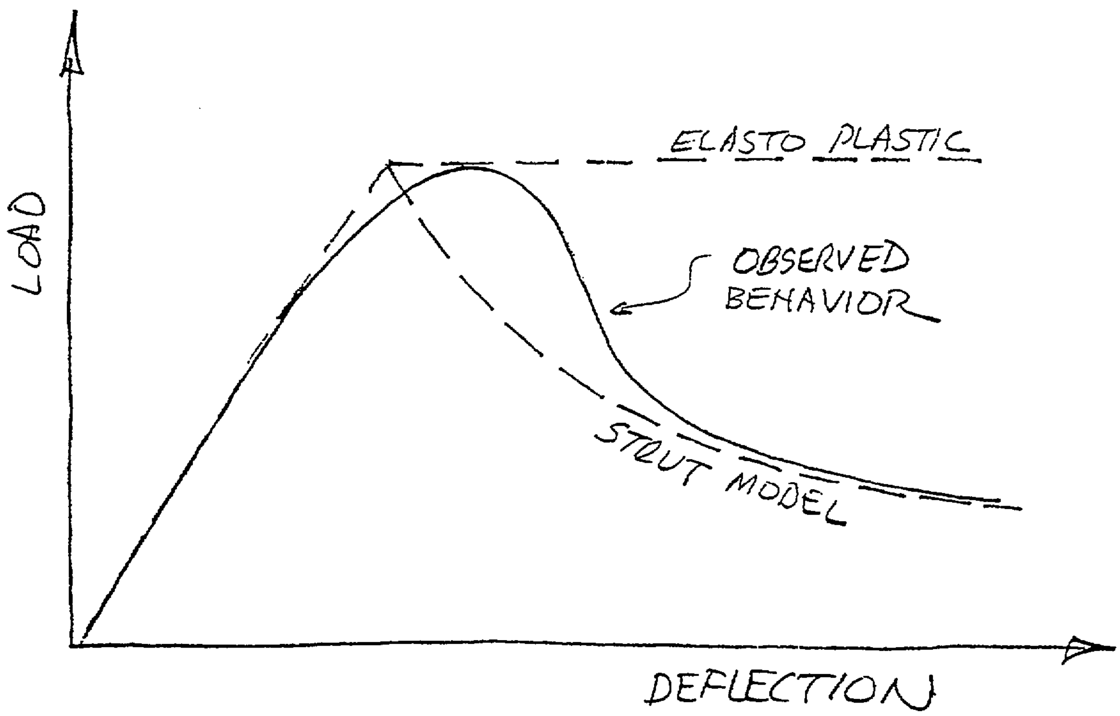



Figure 33: Softening of tubular joints as they approach their ultimate load

**API RP 2A**

**SECTION 17**

**ASSESSMENT OF EXISTING PLATFORMS**

# CURRENT CRITERIA

# &

# FUTURE NEEDS

**JOSEPH KALLABY OFFSHORE STRUCTURES, INC.**

**KRIS DIGRE SHELL OFFSHORE, INC.**

## PAPER OUTLINE

1. GENERAL
2. CURRENT SECTION 17 GUIDELINES
3. FUTURE NEEDS

## EXISTING TUBULAR STEEL JOINTS

### 1. GENERAL

**A. DESIGN BASIS** *** PRE-RP2A - DESIGNER BASED**

*** RP 2A - EDITION BASED**

**B. CONDITION** *** UNDAMAGED**

*** DAMAGED - NO REPAIR DONE**

*** DAMAGED - REPAIRED**

*** TYPE OF DAMAGE - DENTS, CRACKS**

**C. LOCATION** *** LOW TEMPERATURE STEEL DEMAND**

**D. FACTORS AFFECTING CALCULATED JOINT STRESS**

*** EFFECTIVE LENGTH "K" FACTORS**

**3.3.1 d may be too conservative, especially for Ultimate Strength Analysis**

*** ASSUMPTIONS MADE for damaged non-repaired, and damaged repaired joints or members.**

## 2. CURRENT SECTION 17 GUIDELINES

**A. DESIGN BASIS CHECK**

* **9TH EDITION ( 1977 ) OR LATER**

  **OK IF DESIGN BASIS CHECK IS MET, OTHERWISE ANALYZE TO CURRENT SECTION 4 REQUIREMENTS.**

* **PRE 9TH EDITION : ANALYZE TO CURRENT SECT 4 REQUIREMENTS**

**B. DESIGN LEVEL**

* **NO 50% MEMBER STRENGTH REQ.**

**C. ULTIMATE STRENGTH ALLOWS :**

* **ALL FACTORS OF SAFETY = 1**

* **STRENGTH BASED ON MEAN INSTEAD OF LOWER BOUND EQS OF SECT 4,BUT NO NUMERICAL GUIDANCE GIVEN**

* **ACTUAL (COUPON TEST) OR MEAN EXPECTED FY INSTEAD OF NOMINAL ALLOWED, BUT NO NUMERICAL GUID ANCE GIVEN**

* **LOWER "K" FACTORS ALLOWED (NO GUIDANCE GIVEN)**

## 3. FUTURE NEEDS

A. ELASTIC STRENGTH FORMULATION BASED ON 2D 7 3D TEST RESULTS - CURRENT API EQS CONSIDERED TOO CONSERVATIVE . SUGGEST USING MEAN FY=1.167 NOMINAL FY ( USE 42 KSI FOR 36, AND 49 KSI FOR 42, ETC BASED ON AIM)

B. ULTIMATE STRENGTH FORMULATION BASED ON CURRENT TEST RESULTS TO BE INCORPORATED INTO SECT 17. THIS CAN BE AS SIMPLE AS MULTIPLIER FACTORS (BOMEL TESTS)

C. ADOPTION OF REDUCED "K" FACTORS: ANDREW JIP USED "K" FACTORS (NODE TO NODE) FOR ULTIMATE STRENGTH :

K=0.65 DIAGONAL & K BRACED SYSTEMS
K=0.55 X BRACED SYSTEMS

D. EVALUATION OF GROUTED JOINTS : NEED FORMULATIONS AND PROCEDURES. ANDREW JIP USED API WITH SF=1, AND EQUIVALENT WALL THICKNESS BASED ON LEG, GROUT, AND PILE STRENGTHS.

E. EVALUATION OF UNGROUTED JOINTS : NEED FORMULATIONS AND PROCEDURES. MSL ENG'G DEVELOPED EQS FOR P-d AND M-Θ FOR K AND X JOINTS.

F. EVALUATIONS OF DAMAGED JOINTS : FORMULATIONS TO ACCOUNT FOR DENTS AND CRACKS.

G. EVALUATION OF REPAIRED JOINTS : FORMULATIONS AND PROCEDURES FOR NON-GROUTED JOINTS NEEDED.

WORKSHOP ON RE-QUALIFICATION OF TUBULAR JOINTS IN OFFSHORE STRUCTURES
Houston, September 5/6 1995

# R&D PRIORITIES FOR TUBULAR JOINTS

**by Dr Helen M Bolt, Director, BOMEL**

## ABSTRACT

This paper presents BOMEL's views on R&D priorities for tubular steel joints in offshore structures. The paper briefly reviews the significant changes in offshore design and requalification technology currently in progress. Of these, probably the most significant is the increasing role that non-linear pushover analysis is assuming in platform requalification and potentially in design. The limitations of current data, the requirements and research needs to support systems approaches to ultimate load analysis and the work at BOMEL on these topics are described.

R&D priorities identified include the development of tensile strain based failure criteria, validation of analytical methods at large strain, effect of frame continuity and high strain low cycle fatigue.

## INTRODUCTION

This brief paper is intended to catalogue some of the priorities for research and technology development in the field of steel tubular joints as appropriate to offshore structures. The basis for the suggested topics is BOMEL's work and involvement in the following activities:

- Tubular Joints Group - BOMEL is the technical contractor and manager on behalf of some 35 industry members. A new Tubular Joints Design Guide is nearing completion and a new initiative on benchmarking finite element methods for tubular joint analysis is underway.
- Tubular Frames Project - JIP investigating frame system behaviour with a heavy emphasis on the behaviour of joints within the continuity of a structural frame.
- Design and reassessment projects involving strength, flexibility, fatigue and fracture of tubular joints.
- Involvement in developing ISO recommendations for reassessment. (Dr Helen Bolt chairs the ISO subcommittee on this topic).
- Development and application of in-house non-linear pushover analysis software package, SAFJAC.

These various activities have provided a broad insight into the current state of technology in tubular joints from both design and assessment viewpoints. At the present time several important changes in offshore structural technology are taking place which are likely to have a major influence on future research needs and priorities. These are:

- The development of the ISO code.
- The transfer from WSD to LRFD code formats.
- The potential savings in new designs offered by system approaches to extreme loading events.
- Admissibility of non-linear pushover analysis in the assessment process.
- Development of probabilistic approaches.

Until recently the design, and to a lesser extent, the assessment processes have consisted of comparing the capacity of components (e.g. members or joints) as calculated from code formulae with the applied loads obtained from an elastic global analysis of the structure. A large effort has been expended on testing joints in order to understand their behaviour. In most cases the recorded information has been obtained from individual joint tests and has provided a load/deflection relationship to failure with, in some cases, detailed SCF measurements. Fatigue performance has also been studied by testing. The results are highly dependent on material strength, geometrical parameters, loading and support conditions and therefore a large number of tests has been necessary to develop parametric equations suitable for design. It is hoped that the review being carried out in connection with the ISO code development will make the best of the available information to present sets of comprehensive equations for ultimate capacity, SCFs and fatigue S-N curves. Detailed non-linear finite element analysis methods are becoming more widespread to extend the range of test data e.g. for multiplanar and other complex geometries. However, as described below, several limitations have been identified with the current database which impact on the new design and assessment processes now coming into use.

## LIMITATIONS WITH CURRENT INFORMATION

Limitations with current research data on tubular joints arise both due to the way in which the tests have been performed and due to the test objective which may have been limited in relation to current needs. These limitations can be addressed in terms of boundary conditions, loads, scale, values measured and extent of the test as follows:

### Boundary conditions

Most tests have been carried out on isolated joints with simple boundary conditions. The Frames Project has shown that frame continuity often imposes different boundary conditions (deformations, loads, constraints) which can seriously affect both failure mode and capacity. In some cases (e.g. gapped K joints) these boundary conditions convert a ductile compression yielding to a sudden tensile tearing.

### Loads

Most tests have comprised simple joints subjected to either axial or single axis bending loads. Chord loading is a rarity. In practice chord loads will be present and many joints will be subjected to significant moments in addition to axial loads.

### Scale

Most tests have been performed at relatively small scale. Where tearing or fracture are significant to failure mode and capacity, scale is likely to affect the results.

### Values measured

In many cases a single objective for a particular test has resulted in lack of information on other parameters which would now be useful (e.g. joint flexibility, strain distribution, fracture properties).

### Extent of test

The majority of tests to determine ultimate capacity have not proceeded much beyond reaching this capacity and therefore little information is available about post peak ductility, softening and tearing. This is often due to limitations in the testing arrangement (e.g. actuator stroke, inability to control brace direction, limitations of instrumentation). Furthermore there is almost no information on repeated loading into the highly non-linear range either pre or post peak. The Frames Project has shown that large joint deformations can occur at relatively small overall frame deformations and that repeated loading at high capacity (i.e. very high local strains) rapidly result in cracking. This is in the very high strain zone of the high stress-low cycle fatigue region.

## WHAT INFORMATION IS REQUIRED?

The computational capability now exists to explore the behaviour of structures into the heavily non-linear range. In order to fully account for the effects of joint non-linearity a full description of this is required accounting for load interaction, boundary conditions, yielding, strain hardening, softening, fracture and tensile rupture. This needs to cover the full range of deformations up to complete collapse of the chord or separation of the brace.

In this region the tensile behaviour becomes very important and the various limit states (ultimate capacity, fracture and fatigue) interact. Near ultimate capacity, very high strains (>20%) may be calculated or observed. Methods need to be established and validated to deal with fracture and fatigue crack growth within zones of very high strain.

## RESEARCH NEEDS

Based on the above, the following research priorities can be identified:

- Validation of non-linear finite element methods for calculating strain distributions with very large local strains.
- Development of tensile strain based failure criteria.
- Examination of current defect acceptance criteria for validity in large strain zones.
- Understanding of the effects of frame continuity on tubular joint failure modes and capacities.
- Development of non-linear joint response algorithms with allowance for multiplanar effects, load combinations, load history, frame continuity (boundary conditions).
- Generation of high strain low cycle fatigue data and fatigue and fracture assessment methodology.

## WORK ONGOING AND PLANNED

BOMEL has several initiatives underway in the priority R&D areas described above. These are:

- Tubular Joints Group - Phase IVA. Following preparation of the new Tubular Joints Design Guide in Phase IV, Phase IVA concentrates on validation and best practice for finite element methods applied to tubular joint analysis (SCFs, joint flexibility, static strength, SIF's, high strain problems).

- Frames III. This major JIP has, as a principal objective the study of multiplanar K, KT, Y and YT joints within a large frame structure. The main frame test, scheduled for 1996, will enable the study of joint boundary conditions and multiplanar behaviour. Separate joint tests will enable isolation of individual parameters.

- Ultiguide Phase I. This is a joint BOMEL/SINTEF/DnV initiative to produce a best practice for conducting non-linear pushover analysis including the effects of joint non-linearity and failures.

- Tensile strain based failure criteria. This project, recently started with the aid of UK funding, is aimed at investigating tensile failure of joints with high local strain. The project will include large scale X and K joint tests to investigate the effect of scale on fracture and rupture.

- High stress-low cycle fatigue. BOMEL's Kuala Lumpur office is developing a project on this topic with a group of South East Asian testing organisations and universities. Project definition is well advanced with joint industry funding being sought during the final quarter of 1995.

- Improvement of joint interaction expressions within SAFJAC. Current expressions limit the capacity of the principal load component in relation to the other load components. This is being improved to include full loading history interaction and post peak softening.

## CONCLUSION

This paper outlines what BOMEL believes are the research priorities in tubular joint behaviour. It is not an exhaustive list but concentrates on providing further data and validation to improve and increase acceptance of system approaches to design and assessment. Clearly there are gaps in our current knowledge of isolated joint behaviour (strength and fatigue) but the added value from significant new work is seen to have greatest potential in addressing the contribution from joints to the full structural system. Already the Frames Project has demonstrated the differences, some major, between joints within and without the system continuity.

# ULTIMATE BEHAVIOUR AND DESIGN OF MULTI-PLANAR TUBULAR JOINTS


## By Yoshiaki Kurobane[1]



**Abstract:** Multi-planar tubular joints are classified in 4 large groups by ultimate behaviour of joints. In the light of the latest findings, tentative design formulae for these 4 types of joint are devised following the Eurocode 3 format. The AWS ultimate strength equation is compared with existing test results for planar and multi-planar K-joints, which shows that the AWS equation errs on the unsafe side as chords become heavier, owing to the thickness squared strength formulation in it. Recent research developments in the behavior and design of multi-planar joints under axial brace loading are summarized.


### 1 Introduction

Multi-planar joints are frequently used in tubular structures like large span space frames, towers and offshore jacket structures. In contrast to uniplanar joints, multi-planar joints have braces in multiple planes. Marshall's book (1989) shows an example of a joint with 16 braces along with computer coding for classifying joints, demonstrating that there should exist many varieties of multi-planar joints. However, basic multi-planar joints may be classified in 4 large groups as shown in Fig. 1. From the behaviour of these joints one can learn about interactions between loads in different planes. This paper will mainly discuss the ultimate limit state design of these 4 joints in which braces are welded directly to main members. The design of more complex joints will also be discussed briefly.

Further, this paper will limit its scope to joints under axial brace loading. Although study on multi-planar joints under bending loads is progressing, there still remain many points to be resolved. For example, bending moments vary with the stiffnesses of members and joints at the ultimate limit state. No definite design criterion has yet been established for the magnitude of permissible bending moments to be carried by joints. However, most space frames are designed to carry external loads mainly by axial forces in the members. Multi-planar joints under axial loads are more important than those under bending loads.

Eccentricities in joints induce primary bending moments. The effects of these bending moments on joint capacities are incorporated in existing capacity prediction formulae, providing these eccentricities are within certain limits. Although these in-plane moments should be taken into account in the design of members, they are not effective for the design of planar trusses whose ultimate limit states are governed by out-of-plane buckling of members. However, bending moments due to joint eccentricities are influential on the stability of members in space trusses. Designing multi-planar joints with the smallest possible eccentricities is frequently profitable.

The last part of this paper outlines the state-of-the-art on existing investigations into the ultimate behaviour of multi-planar joints. Since background data is insufficient, all the design rules for multi-planar joints are still in an early stage of development. It is useful to examine background research papers to correctly apply recommended design rules to practice.

### 2 Eurocode 3 Approach

Since provisions for tubular joints included in Eurocode 3 (1990) are in principle identical to those in the IIW Recommendations (1989) and Cidect Design Guide (Wardenier et al. 1991), the following descriptions are extracted from the Cidect Guide, which is the easiest to read. Design equations for multi-planar joints in Eurocode 3 have a format of uniplanar joint strength multiplied by correction terms. This format is used also in more accurate design formulae shown later. The correction terms for the TT, XX and KK-joints are summarized in Fig. 2. These correction terms are discussed in some detail here.

Multi-planar joints shown in Fig. 1 fall into 3 groups of the uniplanar X, T and K-joints, because ultimate behaviour of each multi-planar joint is similar to the uniplanar counterpart.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The XX-joint under the compressive loads $N_1$ has out-of-plane braces carrying the axial loads $N_2$. When the loads $N_2$ increase in compression, the capacity expressed in terms of $N_1$ also increases because the chord wall deflection (frequently called the chord ovalization) is suppressed. On the contrary the out-of-plane loads $N_2$ in tension have an adverse effect of increasing chord wall deflections and the capacity decreases. Similarly, when the inplane braces are under the tensile loads $N_1$, tensile loads in the out-of-plane braces give a beneficial effect, while compressive loads in the out-of-plane braces give an adverse effect, on the joint capacity. These effects of out-of-plane braces on the multi-planar joint capacity, frequently called multi-planar effects, are discussed in detail in Marshall's book (1989). The formula shown in Fig. 2 roughly represents these multi-planar effects.

The TT-joint under compression can carry a load comparable to the planar T-joint capacity at each end of the compression braces and, therefore, a correction factor of 1.0 is recommended. The TX-joint is not included in Fig. 2 and will be referred to later.

Planar X and T-joints usually show a greater resistance when braces are under tension than under compression. It is to be noted, however, that design rules in Eurocode 3 do not take advantage of this increased resistance because of possible premature tensile cracks.

In the KK-joint the beneficial effect of suppressed ovalizing is less pronounced and is more involved than in the XX or TT-joint because part of the load is transmitted through the gap region of the chord wall between the tension and compression braces. KK-joints showed capacities even smaller than those of uniplanar K-joints as the transverse gap $g_t$ was increased. A simple constant value of 0.9 is recommended as the correction factor for the KK-joint, which is applicable to either of symmetrical or anti-symmetrical axial brace loading patterns.

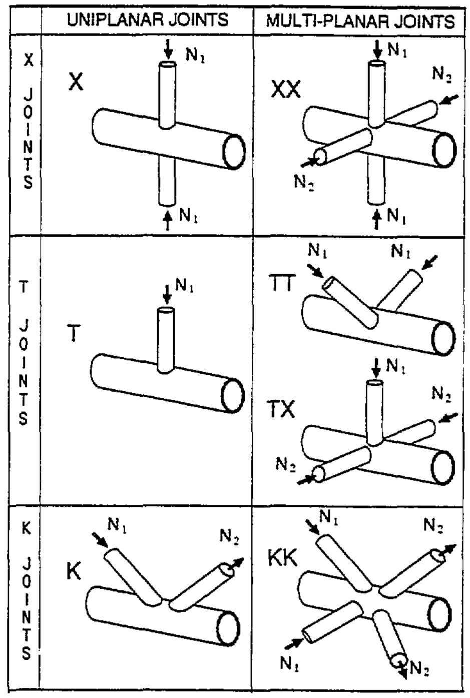



Fig. 1 Basic Types of Multi-Planar Joints

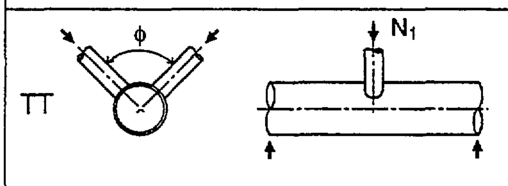
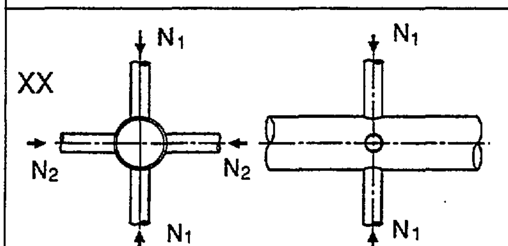
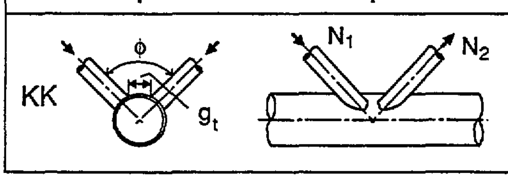
| type of joint | correction factor to uniplanar joint $60^\circ \leqq \phi \leqq 120^\circ$ |
|---|---|
|   | 1.0 |
|   | $1 + 0.33\, N_2/N_1$<br>Note: take account of the sign of $N_2$ and $N_1$ ($N_1 \geqq N_2$) |
|   | 0.9 |

Fig. 2 Correction Factors for Multi-Planar Joints

Although the correction terms given in Fig. 2 are less accurate than those described in the following Section, they roughly correspond to the average values of those given by more accurate prediction equations. They can be used as an approximate measure for the joint design.

## 3 Accurate Approach

The design rules in Eurocode 3 were based on the data base obtained by Makino et al. (1984), Scola et al. (1990) and Paul et al. (1989) as of 1990. Since then extensive investigations on the XX, TT, TX and KK-joints have been performed. The ultimate capacity equations recently developed fit well with existing test and numerical analysis results, although these equations have fairly complex forms and require a further simplification process and reliability analysis to be used as design equations. The following part of this Section shows correction terms derived from these new ultimate capacity equations that are considered reliable and applicable to a wide range of geometrical variables.

As a result of extensive numerical analyses van der Vegte (1995) proposed ultimate limit state equations for X, T; XX and TX-joints under axial, inplane bending and out-of-plane bending brace loads. The experimental verification of numerical results was performed using three X and nine XX-joints (van der Vegte et al. 1991a). The capacity equations for XX and TX-joints take a form of a product of influencing factors. For example, the capacity of XX-joints is given as the capacity of XX-joints with non-load carrying out-of-plane braces multiplied by a correction factor to account for the effect of out-of-plane brace loads. The capacity of XX-joints is increased by the stiffening effect due to the out-of-plane braces even if they are free from any load. This stiffening effect may conservatively be ignored so that the capacity of XX-joints is given as the uniplanar X joint capacity multiplied by a correction factor to allow for the multi-planar effect. An additional correction factor is that to take account of the effect of chord length. An influential correction factor in TX-joint equations is that to account for the effect of overall chord flexure on joint capacity.

Although a more thorough verification of adopted numerical models as well as an investigation into effects of material properties is desired, the proposed capacity equations for uniplanar X and T-joints under axial brace loads fit best with the Kurobane empirical equations (Kurobane et al. 1984), which are applicable to X-joints with sufficiently long chords ($\alpha$=16) and also to T-joints having chords simply supported with a variable span. Since the Kurobane X and T-joint equations formed the basis of Eurocode 3 equations, the following correction terms according to van der Vegte (1995) may be combined with Eurocode 3 design formulae.

The design equation for XX-joints under axial brace loads and the correction factor $C_{XX}$ are given as:

$$N^*_{1,XX} = C_{XX} \cdot N^*_{1,X} \qquad (1a)$$

and

$$C_{XX} = \frac{1}{1-(1.6\beta-1.2\beta^2)J+(1.5\beta-2.5\beta^2)J^2} \qquad (1b)$$

where

$$J = N_2 / N_1$$

In the above equation $N^*_{1,X}$ and $N^*_{1,XX}$ denote the Eurocode 3 design strengths of X and XX-joints respectively, expressed in terms of the axial load $N_1$. The same notation rules are used throughout this paper. The values of the correction factor are compared with those calculated by the Eurocode 3 formula in Fig. 3. The design equations for TX-joints under axial brace loads and the correction factor $C_{TX}$ are given by one or the other of the two sets of formulae,

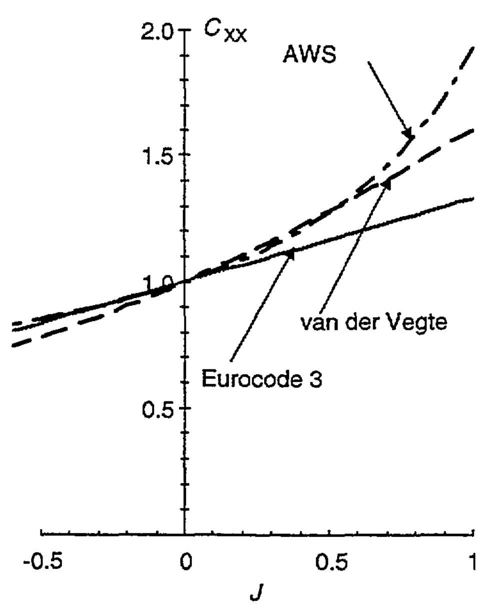



Fig. 3 Correction Factors for XX-Joints ($\beta$=0.5)

$$N^*_{1,TX} = C_{TX} \cdot N^*_{1,T} \qquad (2a)$$

and

$$C_{TX} = \frac{1}{1-1.9\beta^2 J-(0.5-3.5\beta+3.7\beta^2)J^2} \qquad (2b)$$

$$N^*_{2,\mathrm{TX}} = C_{\mathrm{TX}} \cdot N^*_{2,\mathrm{X}} \tag{3a}$$

and

$$C_{\mathrm{TX}} = \frac{1}{1 - (0.23\beta + 0.4\beta^2)(1/J)} \tag{3b}$$

TX-joints sustain local failure in the chord walls under the inplane braces when $J$ is small (Eq. 2 applies), while they fail owing to local deflection of the chord walls under the out-of-plane braces when $J$ is large (Eq. 3 applies). The capacity of TX-joints is given as the smaller of loads determined by these two sets of formulae.

Equations 1, 2 and 3 are applicable for the range $1.0 > J = N_2/N_1 > -0.6$. Note that negative loads represent tension, while positive loads represent compression. The above limitation is due to the fact that the finite element method used in van der Vegte's analyses was not modeling crack initiation and extension processes.

Prediction equations for TT-joints were also proposed by van der Vegte (1995) based on an extensive numerical analysis. These equations have a format different from those for XX and TX-joints: TT-joints were subdivided into two groups of the X and T-joint, depending on the failure mode, each with the corrected $\beta$ ratio. The proposed equations reproduce the test results well, in addition to the empirical equations reported by Paul et al. (1994). Since the latter empirical equations follow the same format as for Eqs. 1, 2 and 3 and are simpler than the van der Vegte equations, the empirical equations will be shown below.

The TT or KK-joint consists of two planar T or K-joints with a common chord as seen in Fig. 4. Additional geometrical variables required to define TT and KK configurations include: $g_t$, which is called the transverse gap or out-of-plane gap, $\phi$ showing the out-of-plane angle between the planes where two braces lie, and $d_1'$ that signifies the transverse distance between the outsides of two compression braces (See Fig. 4). Not all of these variables are independent.

Both the TT and KK-joints demonstrate two types of failure mode as shown in Figs. 5 (a) and (b). The first failure pattern, which is called failure Type 1 and is

Table 1 Relations between Geometrical Variables

| | |
|---|---|
| $\dfrac{g_t}{d_0} = \dfrac{-B + \sqrt{B^2 - 4AC}}{A}$ | $\dfrac{d_1'}{d_0} = \dfrac{-B + \sqrt{B^2 - 4AC}}{A}$ |
| $A = \left(\dfrac{\cos\frac{\phi}{2}}{\sin\frac{\phi}{2}}\right)^2 + 1$ | $A = 4\left\{\left(\dfrac{\cos\frac{\phi_1}{2}}{\sin\frac{\phi_1}{2}} \sin\theta_1'\right)^2 + 1\right\}$ |
| $B = \beta \cdot \dfrac{\cos\frac{\phi}{2}}{\sin^2\frac{\phi}{2}}$ | $B = -\dfrac{4\beta\cos\frac{\phi_1}{2}\sin^2\theta_1'}{\sin^2\frac{\phi_1}{2}}$ |
| $C = \dfrac{1}{4}\left\{\left(\dfrac{\beta}{\sin\frac{\phi}{2}}\right)^2 - 1\right\}$ | $C = \dfrac{\beta^2\sin^2\theta_1'}{\sin^2\frac{\phi_1}{2}} - 1$ |

Note:

$d_1'$: transverse distance between outsides of two compression braces

$g_t$: transverse gap

$\phi$: out-of-plane angle between planes in which braces lie

$\theta_1$: inplane angle between chord and compression braces

$\phi_1 = 2\tan^{-1}\left(\sin\theta_1' \cdot \tan\frac{\phi}{2}\right)$ : angle between two compression braces

$\theta_1' = \tan^{-1}\left(\tan\theta_1 \cdot \cos\frac{\phi}{2}\right)$ : angle between chord axis and plane in which two compression braces lie

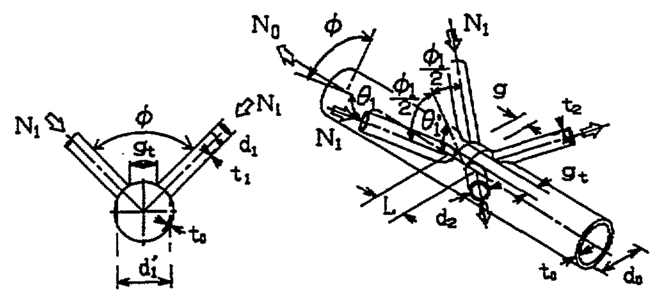



Fig. 4 Typical Warren Type KK-Joints with Definition of Symbols

identified as the failure mode of T-joints in van der Vegte's paper, shows no local deflection in the chord wall in the region between the two compression braces. Namely, the two compression braces act as one member and penetrate the chord wall together. The second failure pattern, which is called failure Type 2 and identified as the failure mode of X-joints in van der Vegte's paper, shows radial deflection of the chord wall in the region between the compression braces, eventually creating a fold between them. The Type 1 failure mode occurs when $g_t$ or $\phi$ is small. As $g_t$ or $\phi$ increases, a Type 2 failure starts to appear.

When a Type 1 failure occurs, the ultimate capacity of the TT or KK-joint is best represented by the resultant of axial loads in the two compression braces, which is denoted by $N_{11,TT}$ or $N_{11,KK}$, and can be given as the ultimate capacity of the T or K-joints, $N_{11,T}$ or $N_{11,K}$, with the brace diameter $d'_1$ and the angle $\theta'_1$ between the compression brace and the chord. Utilization of such a model for a regression analysis led to the ultimate capacity equations for TT and KK-joints written as Eqs. 4 and 6. When a Type 2 failure occurs, the uniplanar T or K-joint included in the TT or KK-joint behaves more independently. The ultimate capacities are best predicted simply by using the regression model on the uniplanar joint capacity multiplied by correction terms. Thus Eqs. 5 and 7 follow. Again the Kurobane equations (1984) were used for the uniplanar joint capacities. Therefore, the following correction terms may be combined with Eurocode 3 design formulae.

In summary, for TT-joints failing in the Type 1 mode,

$$N^*_{11,TT} = C'_{TT} \cdot N^*_{11,T} \qquad (4a)$$

and

$$C'_{TT} = 0.747(1 + 0.586\beta) \qquad (4b)$$

and for TT-joints failing in the Type 2 mode,

$$N^*_{1,TT} = C_{TT} \cdot N^*_{1,T} \qquad (5a)$$

and

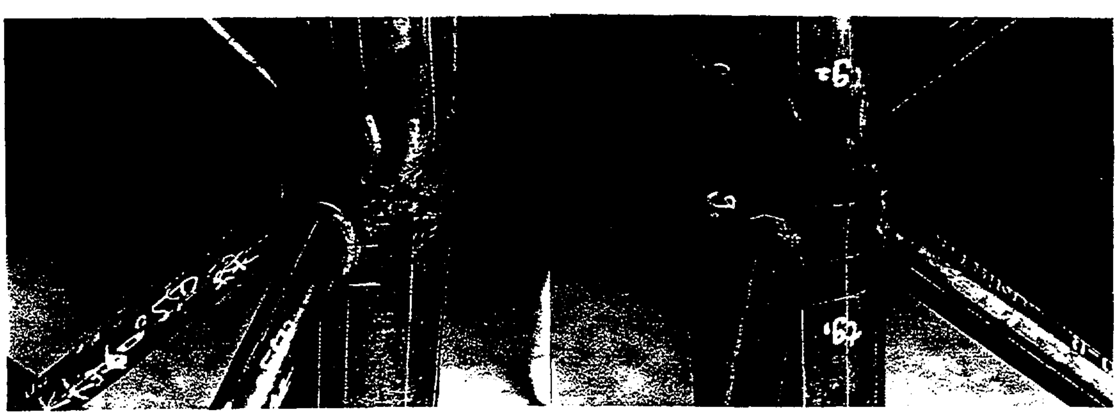
$$C_{TT} = 1.33\left(1 - 0.336\frac{g_t}{d_0}\right) \qquad (5b)$$

For KK-joints failing in the Type 1 mode,

$$N^*_{11,KK} = C'_{KK} \cdot N^*_{11,K} \qquad (6a)$$

and

$$C'_{KK} = 0.746(1 + 0.693\beta)\left(1 + 0.741\frac{g}{d_0}\right) \qquad (6b)$$



(a) Failure Type 1　　(b) Failure Type 2

Fig. 5 Failure Modes for KK-Joints under Symmetrical Axial Loads and for KK-joints failing in the Type 2 mode,

$$N^*_{1,KK} = C_{KK} \cdot N^*_{1,K} \tag{7a}$$

and

$$C_{KK} = 0.798(1+0.808\beta)\left(1-0.410\frac{g_t}{d_0}\right)\left(1+0.423\frac{g}{d_0}\right) \tag{7b}$$

In both TT and KK-joints the real capacities are given by the smaller of the capacities governed by either the failure modes 1 or 2.

The variables $g_t$ and $d_1'$ are not always easy to calculate. Further, since the joint capacity is given as the resultant of two compression brace loads when the Type 1 failure is governing, the capacity in terms of axial load in each brace may be required. Relationships between the geometrical variables are shown in Table 1 to facilitate these calculations.

The design of KK-joints under balanced loads in general can be performed in the following ways. Assume an inverted delta truss with three chords as shown in Fig. 6. Under the horizontal shear load $H$ the KK-joint sustains axial brace loads anti-symmetrical about the vertical system plane as shown in Fig. 6. The loads in the braces are symmetrical when the vertical shear load $V$ acts on the KK-joint.

In the KK-joint under anti-symmetrical brace loads each of the K-joints behaves rather independently and no Type 1 failure mode appears (See Fig. 7). The ultimate capacity of KK-joints under anti-symmetrical loads can simply be predicted by the capacity equation for K-joints with correction terms as shown below:

when $g_t / d_0 \geq 0.16$

$$C_{KK} = 0.858 \tag{8}$$

and when $g_t / d_0 < 0.16$

$$C_{KK} = \left(1.36 - 3.17\frac{g_t}{d_0}\right) \tag{9}$$

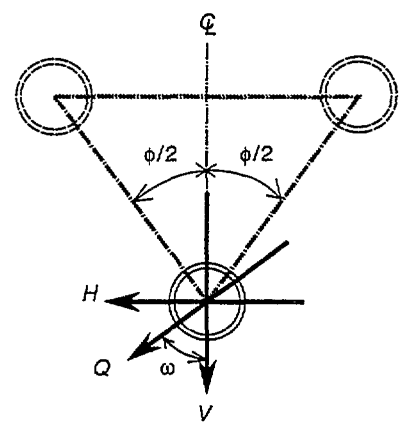



Fig. 6 Inverted Delta Truss

Equations 8 and 9 are based on test results for 8 joints plus FE analysis results for 2 joints filling voids of experimental data (Makino et al. 1995). Especially for a small transverse gap size (Eq. 9), cracks are likely to control the capacity of joints. More data are needed to enhance the reliability of these two equations.

The capacity of KK-joints under the shear load Q with an incline of $\omega$ (See Fig. 6) can be predicted by using the following interpolation technique. When $\omega=\phi/2$, the braces in one plane carry the shear load and the braces in the other plane are free from any axial load. The capacity of KK-joints with $\omega=\phi/2$ can be given as that of planar K-joints. The capacity of general KK-joints can be represented by straight lines linking the points at $\omega=0$ (symmetrical load), $\omega=\phi/2$ (uniplanar K-joint) and $\omega=90$ degrees (anti-symmetrical load) as shown in Fig. 8. In this figure the vertical axis shows the KK-joint capacity non-dimensionalized by the capacity of planar K-joints, expressed in terms of shear loads.

The Eurocode 3 design formulae are common to design formulae in many national codes not only in Europe but also in Australia, Canada and Japan. Therefore, an attempt was herein made to devise more reliable design equations following the Eurocode 3 format.

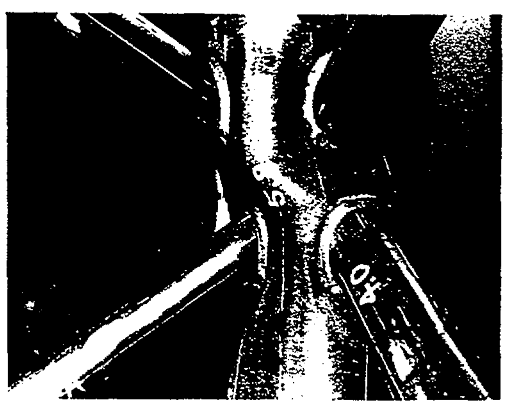


Fig. 7 Failure Mode for KK-Joints under Anti-Symmetrical Axial Loads

## 4 AWS Approach

The AWS code (1994) is one code that shows definite design criteria for multi-planar tubular joints. Further, the AWS design equation is the only exception that proposes general design criteria applicable to any type of non-overlapping multi-planar joints without a need of joint classification. The AWS equation in ultimate strength format is shown below.

$$P_u = 6\pi\beta\left(\frac{1.7}{\alpha} + \frac{0.18}{\beta}\right) Q_\beta^{0.7(\alpha-1)}\, Q_f\, \frac{F_y t_0^2}{\sin\theta} \tag{10a}$$

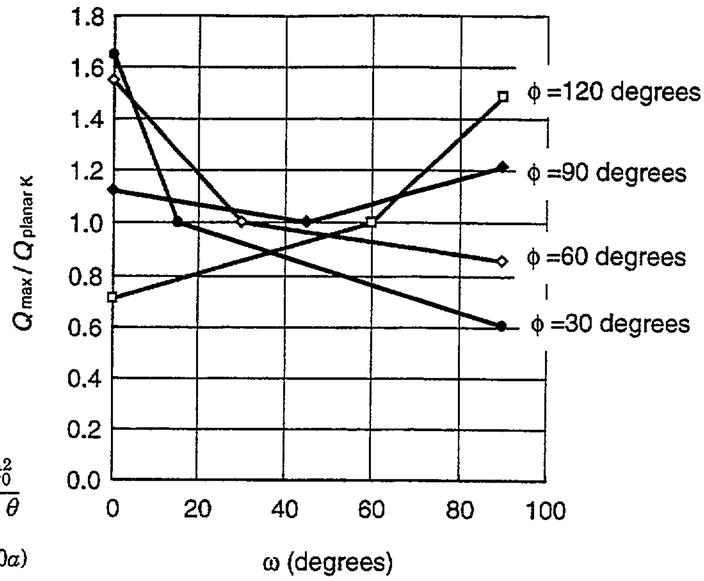



Fig. 8 Capacity of KK-Joints under Balanced Loads in General

with

$$\alpha = 1 + 0.7\frac{\sum\limits_{\substack{\text{all braces}\\ \text{at a joint}}} P\sin\theta\cdot\cos 2\phi\cdot\exp\left(-\frac{z}{0.6\gamma}\right)}{\left[P\sin\theta\right]_{\substack{\text{reference brace for}\\ \text{which } \alpha \text{ applies}}}} \tag{10b}$$

where

$z = L / \sqrt{d_0 t_0/2}$

The parameter $\alpha$ in the above equation plays a role of incorporating not only a multi-planar effect due to chord wall ovalizing (circumferential bending) but also a membrane shell effect due to loads at positions $L$ distant from the reference brace. The value of $\alpha$ is evaluated separately for each brace for which the ultimate limit state capacity is checked (the reference brace), with the summation being taken over all braces present at the node for each load case. The spacing $L$ is measured longitudinally between the centres of foot prints of two braces.

The correction factors, namely the ratios of multi-planar to uniplanar joint capacities, are compared between the AWS and accurate equations. Figure 9 shows the correction factors for KK-joints under symmetrical axial brace loading calculated from Eq. 10 and Paul's formula (1994). Note that the capacity of KK-joints is converted to the axial load in one of the compression braces when the capacity is given as the resultant of two brace loads (See Eq. 6) and that the K-joint capacities for the AWS and Paul equations are computed by Eq. 10 and the Kurobane formula (1984), respectively. The correction factor for XX-joints according to the AWS equation is shown in Fig. 3. Both Figs. 3 and 9 show correction factors according to the Eurcode 3 formulae as well.

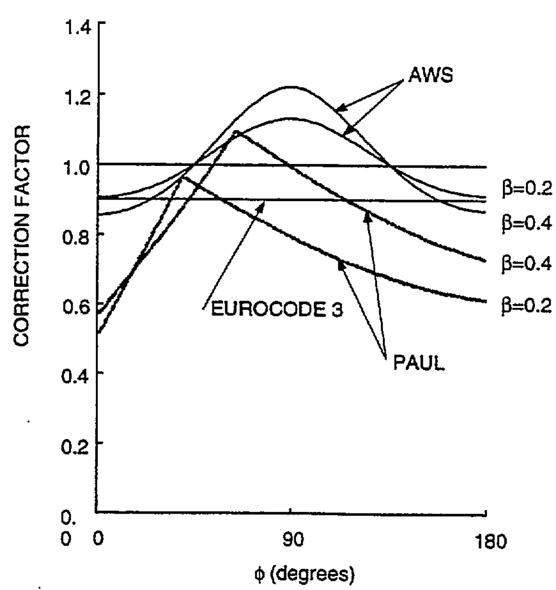



Fig. 9 Correction Factors for KK-Joints (Note: $g/t_0$=10, $\theta'$=45 degrees)

As seen in Figs. 3 and 9, the AWS chord ovalizing parameter captures well a general trend of the multi-planar to uniplanar capacity ratio. Figure 9 shows that the correction factors according to the AWS and Paul formulae first increase with $\phi$ and then start to decrease at a certain value of $\phi$. Actually some differences are found between the two formulae when one looks into details. The correction factor becomes greater than 1.0 in the range where $\phi$ varies from 45 to 135 degrees according to the AWS equation, whereas it essentially becomes lower than 1.0. This is because a different failure mode starts to appear as $\phi$ increases: the chord wall sustains local deflection between the two compression braces (Type 2 failure) and the capacity declines, in contrast to the AWS prediction that speculated that the chord wall ovalization was suppressed as $\phi$ increased showing the maximum resistance at $\phi$=90 degrees. In a wide range of geometrical variables the simple Eurocode 3 correction factors even give more precise predictions than the AWS correction factors.

The test results for KK-joints obtained by Makino and Paul (Makino et al. 1984, Paul et al. 1994, Makino et al. 1994) are compared with predictions in Fig. 10, which shows that the reliability of Paul's equations is excellent. The predictions by the AWS equation scatter widely. This scatter is mainly caused by errors in the evaluation of the K-joint capacity rather than those errors in the correction factor, which were found to be within a limited range in Figs. 3 and 9. To show this, test to prediction ratios for all non-overlapping K-joints (See Ochi et al. 1984 for the database) are plotted against $d_0/t_0$ in Fig. 11. This figure shows that the AWS predictions, although they scatter widely, have a tendency to allow less margin of safety as $d_0/t_0$ decreases. This is due to the thickness squared strength formulation in the AWS equation, whereas strength actually varies as the 1.7 to 1.8 power of thickness (Marshall 1989). Since the AWS equation gives lower bound predictions and contains the resistance factor of 0.74, the AWS predictions should be compared with the 1/0.74 line to see how safe they are, which shows that many of the AWS predictions for K-joints tend to err on the unsafe side as chords become heavier. The predictions according to the Kurobane K-joint equation cluster tightly around the mean line over the whole range of $d_0/t_0$ with the mean equal to 1.00 and the coefficient of variation equal to 0.094.

The systematic errors in the AWS equation, unless the existing equation is altered, have to be corrected by applying a constant bias factor besides the resistance factor, which would make the AWS rules more uneconomical.

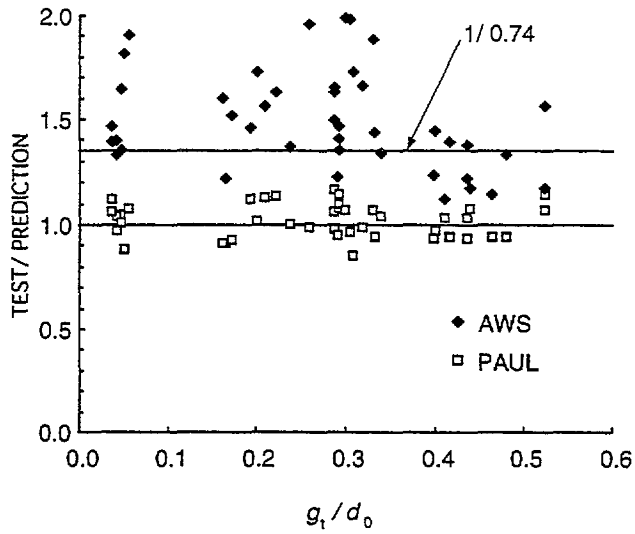



Fig. 10 Ultimate Capacities of KK-Joints: Test to Prediction Ratios

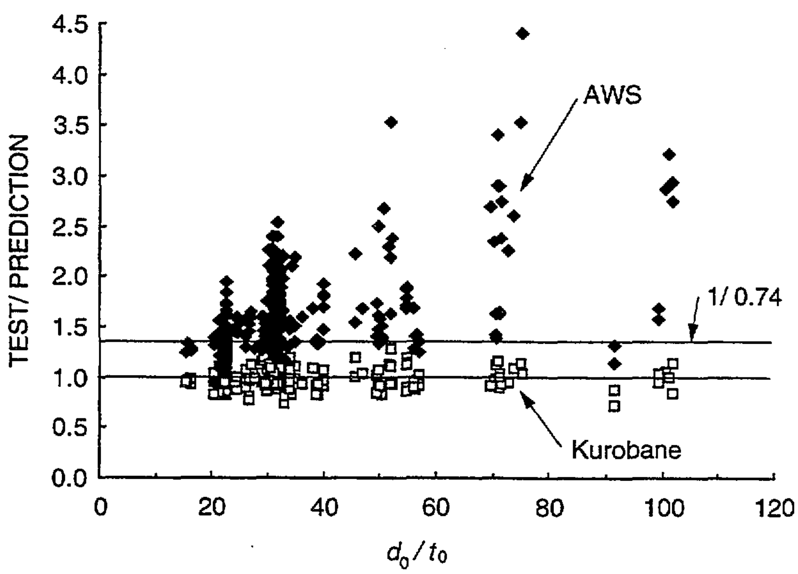



Fig. 11 Ultimate Capacities of Planar K-Joints: Test to Prediction Ratios

## 5 Recent Research Developments

The developments of design criteria for multi-planar joints previously discussed in the "Accurate Approach" Section were promoted by two major groups of researchers.

Makino et al. (1984) proposed the first ultimate capacity equations for KK-joints based on a series of tests in 1984. Although two types of failure modes were found even in this early study, further studies on TT-joints by Scola et al. (1990) and TT and KK-joints by Paul et al. (1991, 1992) were required until these failure modes were used for building mathematical regression models to draw more reliable prediction equations for KK and TT-joints (Paul 1992, Paul et al. 1994). Wilmshurst et al. (1993, 1993a) performed a series of FE analyses on 30 KK-joints. They validated numerical models against the Makino tests and drew versatile conclusions about mesh layout, numerical models for welds and boundary and loading conditions. The fact that the AWS equation tends to overpredict the capacity of KK-joints with heavy chords was also recognized by researchers at University College of Swansea. Wilmshurst (1993a) showed that the AWS equations gave unsafe estimates when $d_0/t_0$ became less than 24. Their numerical results were combined with test results to formulate ultimate capacity equations for KK-joints under anti-symmetrical brace loading, as mentioned earlier (Makino et al. (1995).

Mouty et al. (1992) performed an extensive series of tests on KK-joints. However, because of a large difference in boundary conditions, their test results were not directly applicable to the ultimate capacity equations referred to here.

Paul et al. (1989) started an investigation on XX-joints by means of FE analysis in 1989 and showed strength interactions between inplane and out-of-plane brace loads, which were discussed in the "Eurocode 3 Approach" Section. This work was succeeded by van der Vegte et al. (1991, 1995), in which ultimate capacity equations were determined by regressing extensive numerical results on mathematical models derived from closed ring analyses. Their numerical models were calibrated against a series of tests on both X and XX-joints (van der Vegte et al. 1991a). The same methodology was extended successfully to predict ultimate behaviour of TX and TT-joints (van der Vegte et al. 1995, 1995a). In these works the above methodology made it possible to identify the effects of overall chord flexure on the ultimate capacity governed by chord ovalization in T, TX and TT-joints (van der Vegte et al. 1994, 1995). All the works on XX, TX and TT-joints are discussed in detail in van der Vegte's Ph. D. Thesis (1995).

Somewhat independently of the studies described above Ward et al. (1992) and Davies et al. (1994) reported numerical investigations into strength interactions between multi-planar braces.

## 6 Summary and Conclusions

All the existing proposals for the design of multi-planar tubular joints under axial brace loading were discussed along with their background investigations. The capacities of multi-planar joints can best be predicted by those capacities of relevant uniplanar joints multiplied by correction factors, for which reliable capacity prediction equations are available. The AWS equation, which has a unique format different from the others, is attractive from a practical viewpoint because the capacity of multi-planar joints is automatically computed by inputting load and location data for the braces framing into each node. However, large errors containing systematic components due to AWS prediction may lead to an unsafe design.

The design of multi-planar joints is still in an early stage of development. Any attempt to improve the existing proposals is in prime need.

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August 1995

# SUMMARY OF PAST AND CURRENT RESEARCH ON TUBULAR JOINTS AT DNV


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Presented at

Workshop on Requlification of Tubular Steel Joints in Offshore Structures, Houston, September 5 and 6, 1995.



## Abstract

This paper presents the background for DNV's research activities in tubular joint technology. The results of major research projects are presented, and with focus on the technologies applicable to reassessment and extended life analysis of structures based on knowledge on strength and behaviour of tubular joints.

Finally, important needs for further research and technology developmetn are indicated, with particular attention to reassessment of existing platforms.



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INDUSTRY AS

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This paper presents the background for DNV's research activities in tubular joint technology. The results of major research projects are presented, and with focus on the technologies applicable to reassessment and extended life analysis of structures based on knowledge on strength and behaviour of tubular joints.

Finally, important needs for further research and technology development are indicated, with particular attention to reassessment of existing platforms.

## 1. GENERAL RESARCH ON TUBULAR JOINTS

### 1.1 Introduction

Research into the complex problem of strength of tubular joints in offshore applications was started in Norway as early as 1972. These activities were concentrated at Det norske Veritas, which, as a classification society, has been concerned with problems related to fatigue, corrosion fatigue, fracture strength of ships for over 130 years.

A major challenge, when oil was discovered in the North Sea in the late 1960's, was to develop technologies for design of fixed steel structures that could sustain the hostile environment. Furthermore, in view of the great human, ecological and material risks involved, due considerations had to be made with respect to reliability and safety requirements.

With respect to tubular joints in particular, two problems were the main subjects, i.e.

i) Static Strength

ii) Fatigue Strength

### 1.2 Aspects on static strength

In the late 1960's former knowledge and design criteria for offshore steel structures belonged to the "Mexican Gulf" era, and the governing code was the API "Recommended practice for planning designing and construction fixed offshore platforms".

DNV's introduction to this field was to carry out a series of 12 static strength tests of X-joints exposed to axial tension and compression, and 19 static strength tests on T-joints exposed to in-plane bending loads, Refs. /1/ and /2/. These tests, combined with other available published data resulted in capacity formulations on the static strength of simple tubular joints, which formed a basis for DNVs formulas for static strength of tubular joints in the 1974-rules, Ref. /3/.

The limitations of those capacity formulations (and the situation is mainly the same today) are that they are based on simple uni-plane nodes tested under single load situations. When developments in computer technology made available advanced non-linear analysis software in combination with very fast, high capacity computers, DNV launched the research project "Static Strength of Tubular Joints". The results of this project, which was undertaken in two phases in the period from 1985 to 1991, Refs. /4/, /5/, /6/ and /26/ have in reality opened for the possibility to carry out far more detailed strength assessments. A large number of computer analyses using the 1981-verision of ADINA, and later the 1986 -version of SOLVIA, were compared with 8 tubular joint tests that were controlled to the extent that the tubulars were machined to extremely close tolerances to eliminate sources of inaccuracy. On the basis of this work it was concluded that static strength of complex tubular joints reliably can be predicted by non-linear FE-analysis, provided the failure mode is of large compressive strains and chord ovalization or brace buckling, Fig. 1.

### 1.3 Aspects on fatigue strength

In the late 1960's, the former knowledge with tubular joints origined from areas where metal fatigue was not a major concern. It was therefore a great need to develop a better basis for fatigue prediction of tubular joints being exposed to North Sea conditions, i.e. $5 \times 10^6$ wave cycles per year, and with maximum wave heights of more than 30 m amplitude.

For this reason fatigue was the focus of major research efforts in Norway in the 10-year period 1975 to 1985.

For the period 1981-1985, the research was organized as the National 5 Year Program for Fatigue of Offshore Steel Structures in Norway, and DNV had the responsibility for tubular joints. The program was also coordinated with similar programs organized by the European Coal and Steel Community (ECSC).

A large number of linear FE-analyses compared with photo-elastic measurements and tests (Refs. /7/ to /12/, and in conjunction with the information obtained within the ECSC program, contributed to the acquisition of fundamental knowledge being used today related to, Fig. 2:

- determination of the fatigue relevant hot spot strains and SCF definition
- establishment of rational definitions for crack initiation
- establishment of the validity of present S-N curves
- establishment of the applicability of thin shell FE calculations for reliable determination of the SCF in tubular joints.
- establishment of the applicability and reliability of the alternating current potential drop, (ACPD), method for the purpose of measuring crack sizes in the in-thickness directions.
- disclosing that the surface crack growth rate along the intersection of tubular joints, chord side, is a constant and linear function of the load cycle number N.
- disclosing and confirming that the in-thickness crack growth rate is also a linear function of N.
- providing significant indications of the efficiency of cathodic protection against corrosion for fatigue of tubular joints.

## 2. TUBULAR JOINT TECHNOLOGY RELEVANT FOR REASSESSMENT CURRENT STATE OF THE ART

### 2.1 Introduction

Research at DNV has focused on the following subjects with respect to reassessment of tubular joints.

* The effect of Double Skin Grout Reinforced Joints
* The use of grinding as a repair method
* The development of refined fatigue analysis
* The risk of unstable fracture

Main elements and references related to the above subjects are presented in the following.

### 2.2 Double skin grout reinforced joints

A double skin grout reinforced joint (DSGR) is a tubular joint with an internal tubular inside the chord, and where the annulus between the two tubulars are filled with grout, Fig. 3. DSGR joints have significantly improved static- and fatigue strength properties as compared with ordinary welded joints.

DSGR joints are relevant in the context of reassesment of structures for the following reasons:

* Many existing structures have tubular piles driven through and grouted into the legs of a structure and thus creating a large number of DSGR joints. Typically, the positive effects have not been incorporated in initial design, and a reassessment based on actual conditions may verify considerable reserve capacities.
* Grouting, either as DSGR, or full grouting of a chord member represent an efficient and inexpensive method for reinforcing tubular joints.

  Substanitial research and development work into DSGR joints has been going on in DNV since 1978 with the object of gaining detailed knowledge of DSGR-joint's ultimate strength and fatigue strength behaviour, Ref. /13/ and /14/. A large number of stress analyses and fatigue tests were carried out on X-, T-, Y-, K- and overlapping K-joints. Furthermore, the FEM approach has been applied with considerable success for the numerical analysis of the tested joints.

The following general comments can be made:

- The grout reinforcement provides remarkable increases in the static strength of the connections, which may amount to four fold or higher, even under tensile axial brace loads. Under compressive axial loads the DSGR-chords are virtually undestructible.
- Remarkable SCF reductions (versus ungrouted joints) are obtainable. SCF reductions to the order of 1/3 are readily obtainable. Even larger reductions can be obtained.
- The stress concentration factor of grouted joints is load dependent. The dependency is non-linear.

- The SCF reductions (versus ungrouted state) under compressive loads are larger than those under tensile loads.
- Grouted joints can be evaluated for fatigue strength using SN-curves applicable to ungrouted joints.
- DSGR joints can be stress analyzed numerically. This requires use of non-linear FEM calculations. Otherwise the SCF must be determined by laboratory tests.

### 2.3 Grind repairs of tubular joints

During the life of a welded structure it is likely that situations may occur where repairs of the welded connections have to be carried out. Previous experience had indicated that repair of surface cracks could be done by just removing them by grinding. On this basis, a major research effort was launched in 1984 involving structural testing of 72 narrow specimens, 36 wide plates and more than 10 tubular joints in order to develop operational justififcation and guidance for grind repairs of welded connections in offshore structures as a means of permanent repair. The results are reported in Refs. /15/ to /18/.

The project has provided detailed documentation with respect to how a repair situation could be approached, including fully documented SN-curves that are applicable to various connections, such as tubular joints, Fig. 4.

One of the most remarkable conclusions from the project is that it is possible to obtain significantly improved fatigue life of locally grind repaired connections, even for repairs as deep as 50% of the thickness. The reason for this is that when removing a crack by grinding, the hot spot is shifted from the traditional location in the HAZ at the weld toe, and into parent metal in the groove, Fig. 5.

The improvement depends, however, on the following main parameters:

* Stress concentration factor in the groove
* The shape of the longterm load distribution
* The original design life of the repaired node

It was found that the fatigue life of a grind repaired joint depends strongly on the magnitude of the maximum stress range in the repair groove. This stress range depends also on the SCF in the as-welded node, and the shape of the repair in terms of groove radius, its length along the weld toe, and the repair depth. To be able to determine this stress range in a real situation, a preprocessor for generation of a FE analysis model of a groove in the SESAM software package was developed at DNV, and comparisons with the tests have proven that groove stresses can be calculated accurately povided proper geometrical representation using thick-shell elements, Fig. 6.

Also the ultimate strength capacity of grind repaired tubular joints was checked. A slight reduction in capacity was noted as compared with the as-welded condition. Conservatively, the reduction is assumed to be proportional to the extent of puching shear cross sectional area removed by the repair.

### 2.4 Refined fatigue analysis

In traditional fatigue analyses of tubular joints, the nominal stresses in the braces are combined with stress concentration factors, which either are derived from parametric equations developed from simpel joints, or from detailed FE-analyses.

Major weaknesses of this simplifcation are:

* interaction effects from other brace-members are neglected
* the stress distributions along the weld toes are not assessed accurately
* the SCF's are normally based on the maximum principal stress, and not the strain normal to the weld toe, which is the correct basis for the SN-curves.

The principle behind refined fatigue analysis is that a detailed, 3-dimensional FE-model of the node in question is being analysed for a unit-displacement at the end of each brace for all possible degrees of freedom (DOFs); one DOF at the time. Based on this, influence matrices can be developed, defining the correlation between a DOF and a distribution of strains normal to the weld toe. Through superposition, the absolute true strain can be calculated for any actual combination of DOFs which may be available from a traditional jacket analysis, Fig. 7.

DNV has developed this concept and used it for a number of applications, Refs. /19/ and /20/, and it has been possible to demonstrate substantially longer fatigue lives of tubular joints as compared with traditional analyses.

### 2.5 Unstable fracture

The problem of sudden, unstable fracture initiating from possible pre-existing cracks during a storm situation is a subject of continuous concern.

In the period from 1980-1988, DNV tested 9 T-joints and 4K-joints in large scale, all joints were deliberately tested at temperatures giving low material toughness, and with pre-existing fatigue cracks of depth 3 to 6 mm relative to a wall thickness of 30 mm, Refs. /21/, /22/ and /23/.

In the course of this project it became clear that the problem of unstable fracture in tubular connections is a very complicated issue. For both geometries it was possible to establish a unique correlation between the crack depth, the applied load, and the physical crack opening at the crack tip; CTOD;

$$\frac{CTOD * E}{a * \sigma_y} = 2.5\left(\frac{\sigma_{max}}{\sigma_y}\right)^2$$

By this simple expression it was hoped that by introducing the value of the critical CTOD of the material, it would be possible to calculate the maximum stress and corresponding maximum load.

Based on the 8 tests that failed in an unstable mode it became apparent, however, that it was necessary to introduce a correction factor, $\eta$, that was equal to 0.7 for the K-joints as well as for the T-joints.

$$\frac{CTOD * E}{a * \sigma_y} \geq 2.5 * \eta * \left(\frac{\sigma_{max}}{\sigma_y}\right)^2$$

## 3. VIEWS ON THE NEED FOR FURTHER RESEARCH AND DEVELOPMENT IN THE CONTEXT OF REASSESSMENT

The application of new technolgies developed in DNV is, today still, to some extent hindered because of the following reasons, or combinations of them:

* the technologies are not generally known to the industry
* the technology is not user-friendly and easily available
* further questions must be investigated before the technology can be used effectively

On this basis, the various projects discussed in Section 1 and 2 are discussed in the focus of future needs.

Static Strength of Tubular Joints

DNV is of the opinion, that the cost-effective way in the future for detail assessment and reassessment of the static strength of a complex, multiplane joint exposed to loads in all degrees of freedom, is to use non-linear FE-analysis.

The state-of-the-art today is that load and deformation (strain) patterns can be well estimated based on the inclusion of material-, and geometrical non-linearities.

The solution to the problem of ultimate strength capacity using FE-analysis is established for cases which either results in buckling failures of braces, or a maximum plateau due to chord ovalization is reached for compressive brace loads. There is, however, a need for a faliuire criteria for loads in tension. DNV has observed in a number of tests, that a failure criteria based on appr. 10% maximum strain in a FE-analysis can give close prediction. However, to establish a broader basis for acceptance, this needs to be studied further. Further research should therefore focus on tensile strength criteria to be used in non-linear FE-analysis for determination of the ultimate strength.

Double Skin Grout Reinforced Joints

The technology shows extreme potential. The knowledge can be expanded to cover locally fully grouted members as means of strengthening and repair.

Both testing techniques and numerical analysis techniques are well understood.

Further research should focus on:

* Guidelines on practical repair aspects
* Guidelines on feasibility of repair
* Practical capacity formulations for ultimate strength
* Practical SCF-formulations
* Verification of numerical analysis tool

Grind Repairs of Tubular Joints

The technology is in principle ready to use and fatigue assessments can be made. However, more sophisticated assessment of ultimate strength reduction aspects are needed today.

Further research aspect should focus on:

* Ultimate Strength

Refined Fatigue Analysis

The technology is ready to use, but available software is not commerically developed. Further development should focus on:

* Development of Software

Unstable Fracture

One observation from DNVs research project Ref. /21/, was that significant ductile tearing may take place from fatigue cracks during a limited number of high load cycles that would occur during extreme weather conditions.

Large welded structures will often contain crack-like defects (fabrication induced or service induced). Fracture mechanics concepts are used to quantitatively assess the integrity of such structures.

The most common procedures used today for assessment of severity of cracks in the elastic-plastic regime are level 1, 2 and 3 assessments described in PD 6493:1991, Ref. /24/ or the R6 failure assessment method, Ref. /25/. The experimental verification for these methods for tubular joints is, however, quite limited. DNV is currently doing theoretical studies on this subject, and 3 large-scale T-joints will be tested in this program. DNV considers this issue to be of significant importance in the context of requalification of offshore structures.

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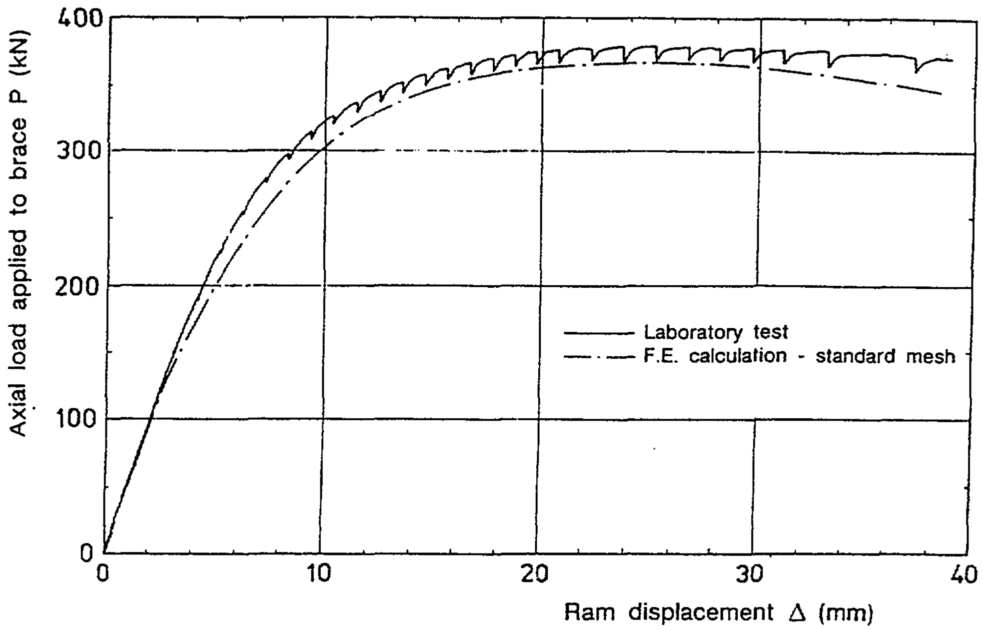



Fig. 1: Example of experimental and numerical FE-analysis of ultimate axial capacity of X-joint in compression

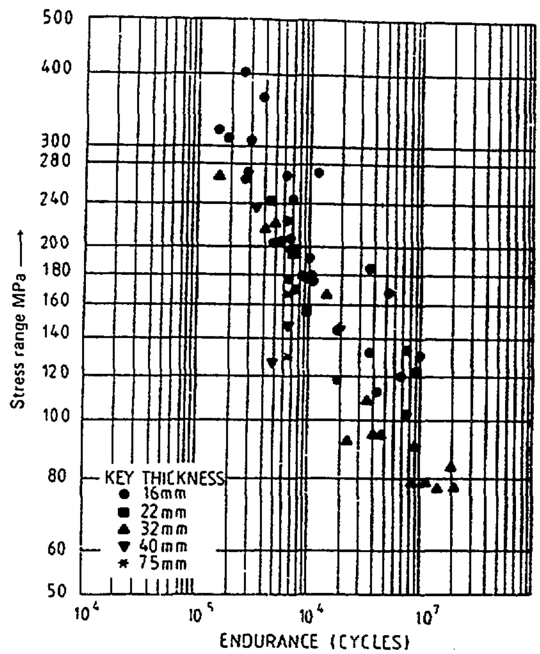



Fig. 2: Experimental results expressed in terms of the number of cycles to through-wall cracking from "Background to new design guidance for steel welded joints in offshore structures" Dept. of Energy 1982

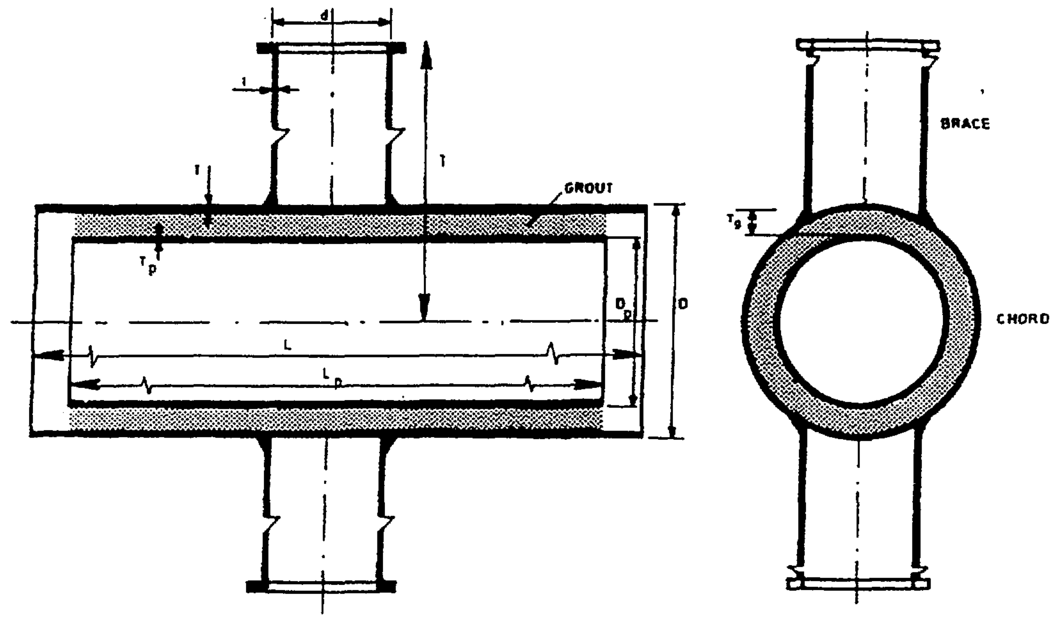



Fig. 3: A double skin grout reinforced X-joint

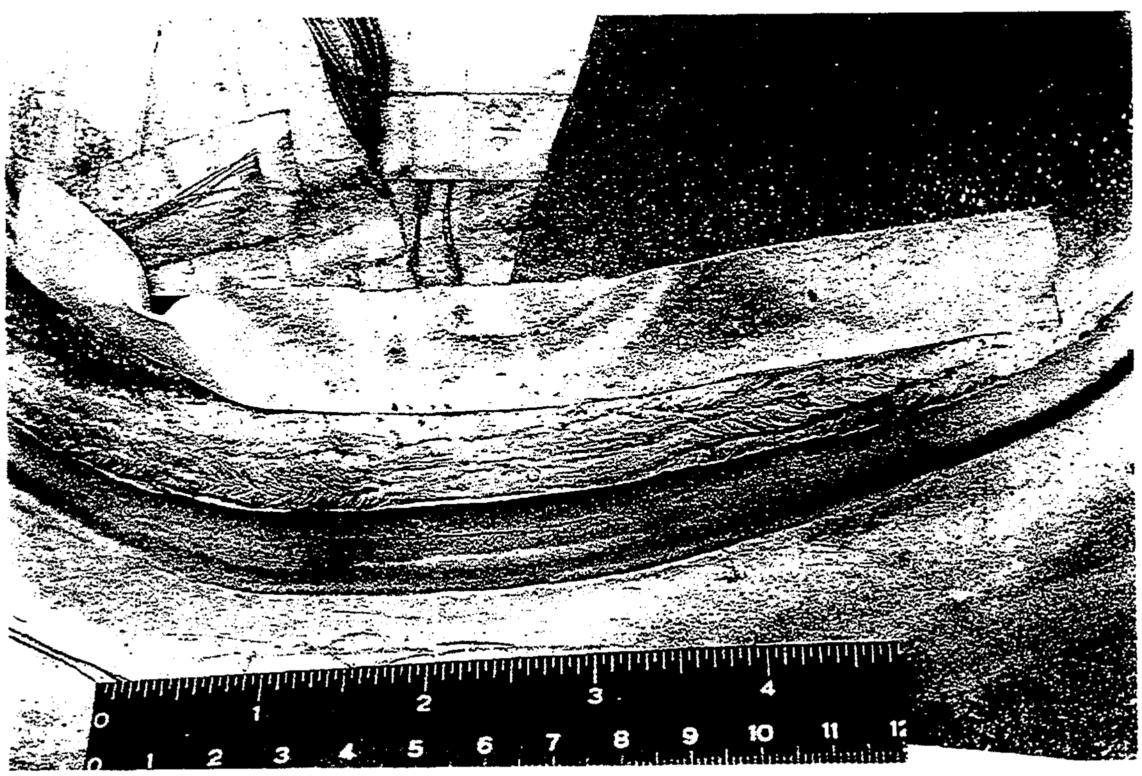
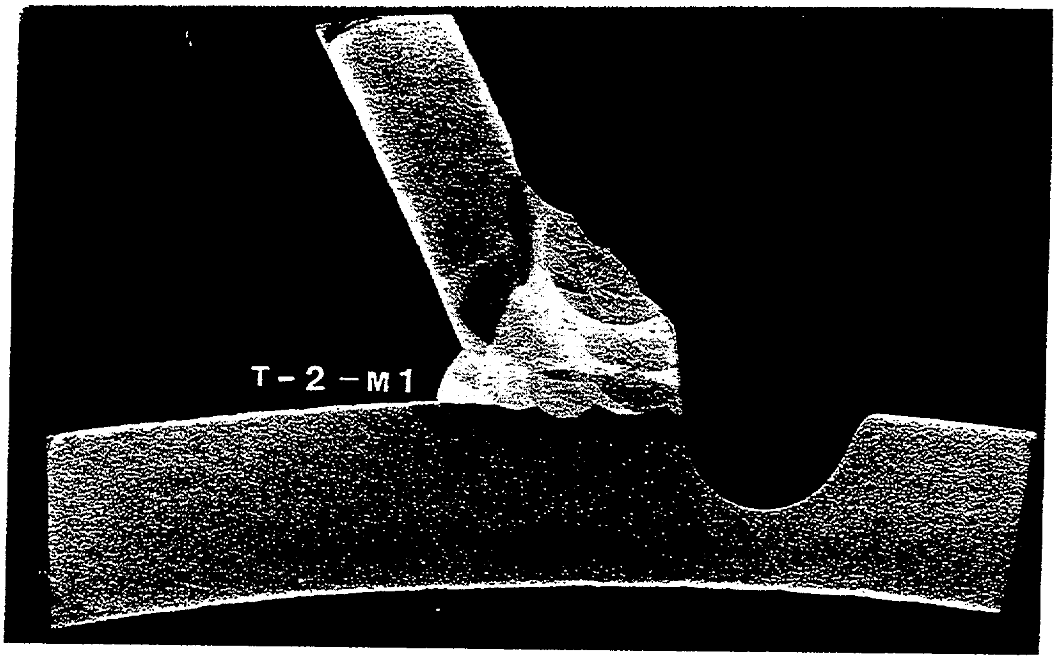


Fig. 4: Grind-repaired tubular joint



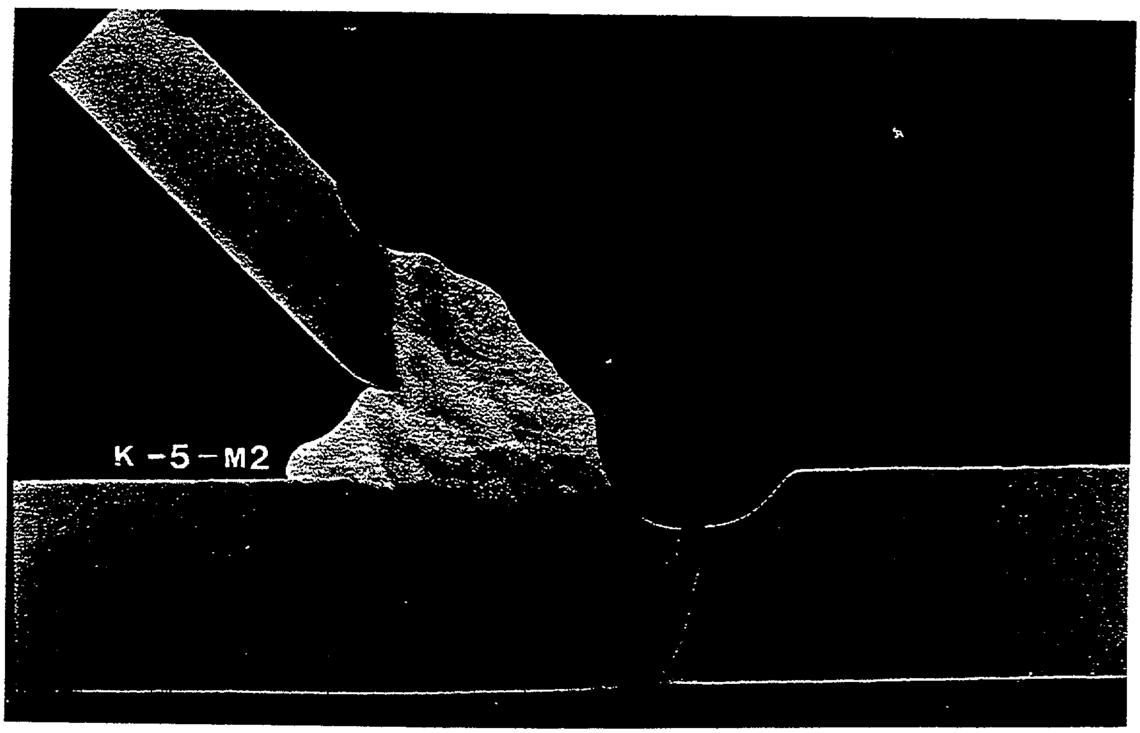
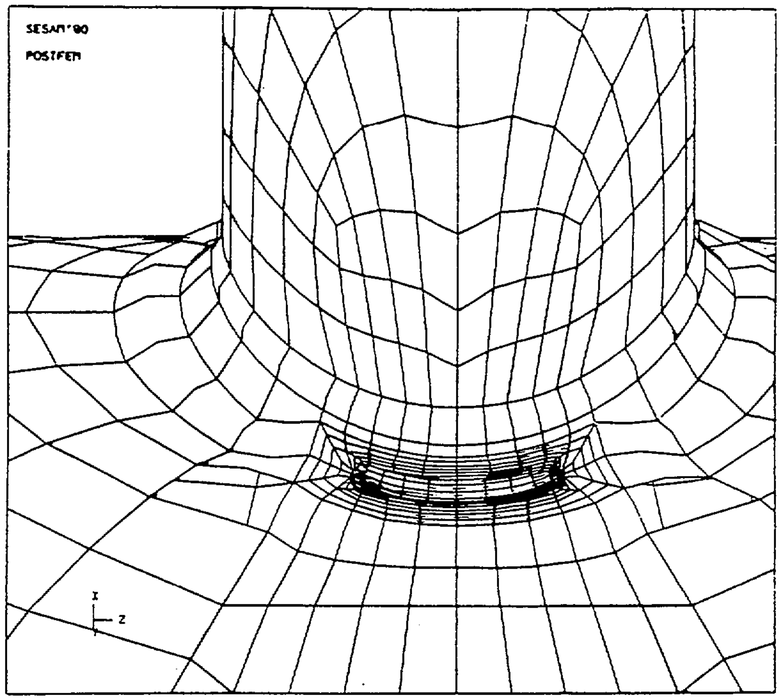




Fig. 5: Cross-section of Repair Grooves.



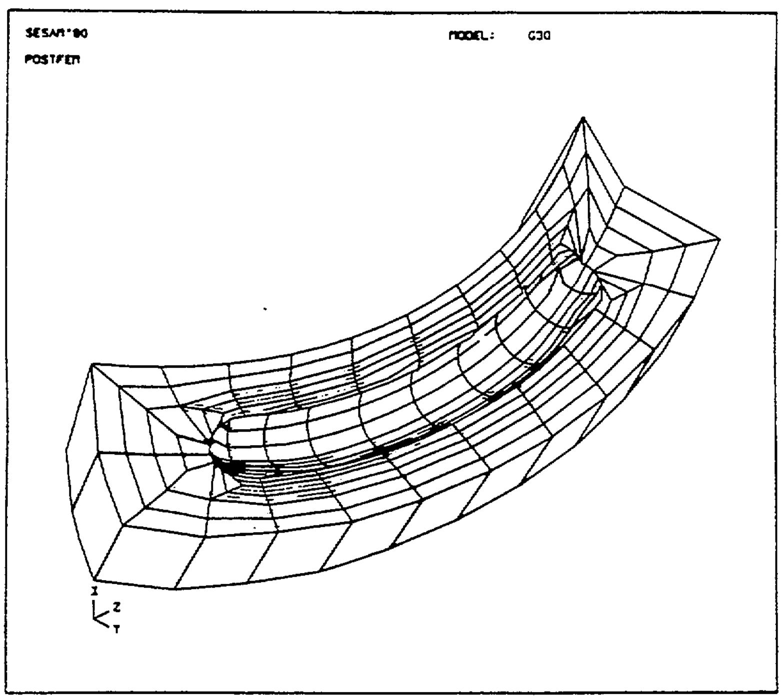




Fig. 6: FE-model representation of a repair groove in SESAM-80

# REFINED FATIGUE ANALYSIS

Shortcomings and approximations in traditional fatigue analyses of steel offshore structures are overcome using DNVI's Refined Fatigue Analysis Technology.

Refined fatigue analysis takes accurately into account;

* Local stresses and their angle relative to welds
* Load interaction from adjacent structural members
* Node flexibility

Both options of stochastic and deterministic analyses are available.

Refined fatigue analyses may typically be recommended in the following situations:

- for particular important structural elements, i.e. critical nodes, pile-sleeve connections etc.
- if traditional analyses gives inadequate fatigue strength
- for extended life prediction
- for optimization of inspection and maintenance routines

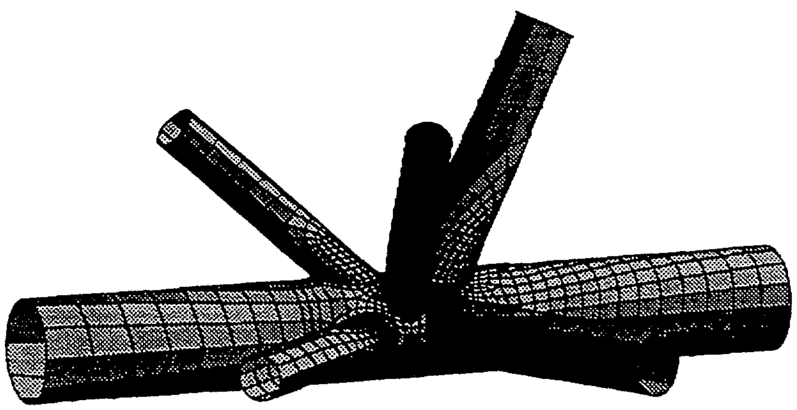


Fig. 7: The principles of refined fatigue analysis

# AN ASSESSMENT OF ELASTIC-PLASTIC FRACTURE MECHANICS FOR REQUALIFYING TUBULAR STEEL JOINTS IN OFFSHORE STRUCTURES

by

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Workshop on Re-qualification of Tubular Steel Joints in Offshore Structures

Houston, Texas

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## Summary

Any process for re-qualifying tubular steel joints in offshore structures will surely encounter components having crack-like damage. This will require the use of fracture mechanics technology. While this technology has been found to be useful for many different engineering applications, unfortunately, the joints in offshore structures present a demanding set of conditions that exceed those in which fracture mechanics applications have so far been successful. These conditions include combinations of three-dimensional geometries, nonstraight crack fronts, nonplanar crack surfaces, residual stresses, and heat-transformed mechanical and fracture parameters. To help scope the research that is needed to overcome these barriers, this paper reviews current uses of fracture mechanics in other engineering systems. The objective is to identify a viable starting point for attacking the unresolved issues that currently prevent the full and effective use of this technology in re-qualifying joints in offshore structures.

# AN ASSESSMENT OF ELASTIC-PLASTIC FRACTURE MECHANICS FOR REQUALIFYING TUBULAR STEEL JOINTS IN OFFSHORE STRUCTURES

## 1. Introduction

The re-qualification of an engineering structure may be required for one or more of several reasons. One is that new analysis techniques, NDE methodologies, and/or more complete databases may have come into existence subsequent to the original qualification. Another is that the structure of interest may have been subjected to external forces and/or environments significantly more severe and prolonged than were originally anticipated. A third reason is that it would simply be too expensive to retire the structure at its nominal design life. For offshore structures, it is not unlikely that all three of these reasons are motivating interest in structural re-qualification in general, and of joints in particular.

Conservative detail designs and material selection, and a substantial amount of structural redundancy, has kept all but a few failures from occurring in offshore structures. But, as these structures age, and when their service history includes particularly violent storm loadings such as Hurricane Andrew, attention must be given to the possibility that widespread and significant damage could accrue that would negate the protection of structural redundancy. For damage that is in the form of crack-like flaws and defects, it is imperative that fracture mechanics technology be called upon. Unfortunately, the currently available techniques are not entirely adequate for such applications.

While a broad range of issues exist that bear on the subject of this workshop - see references [1-3] - this paper focuses on one particular area: the need for improved fracture mechanics treatments that can meet the specific challenges posed by the operating conditions, structural configurations, material selections and joining processes in offshore structures. This will be done by reviewing the state-of-the-art of elastic-plastic fracture mechanics in order to identify the research that is needed for this technology to meet the demands for integrity and durability of joints in offshore engineering applications such as those shown in Figure 1.

## 2. Fracture Mechanics Concepts

Credit for establishing fracture mechanics as an engineering methodology has conventionally been bestowed upon A. A. Griffith and G. R. Irwin for their pioneering research in the 1920's and 1940's, respectively. Nonetheless, modern fracture mechanics and its attendant multiplicity of applications is actually less than three decades old. Contributions by Rice, Hutchinson and Paris around 1970 were instrumental in establishing the crack tip dominance point of view that has both demonstrated the unity of the subject and has allowed the various degrees of complexity beyond that of the Griffith-Irwin approach to be logically formulated [4].

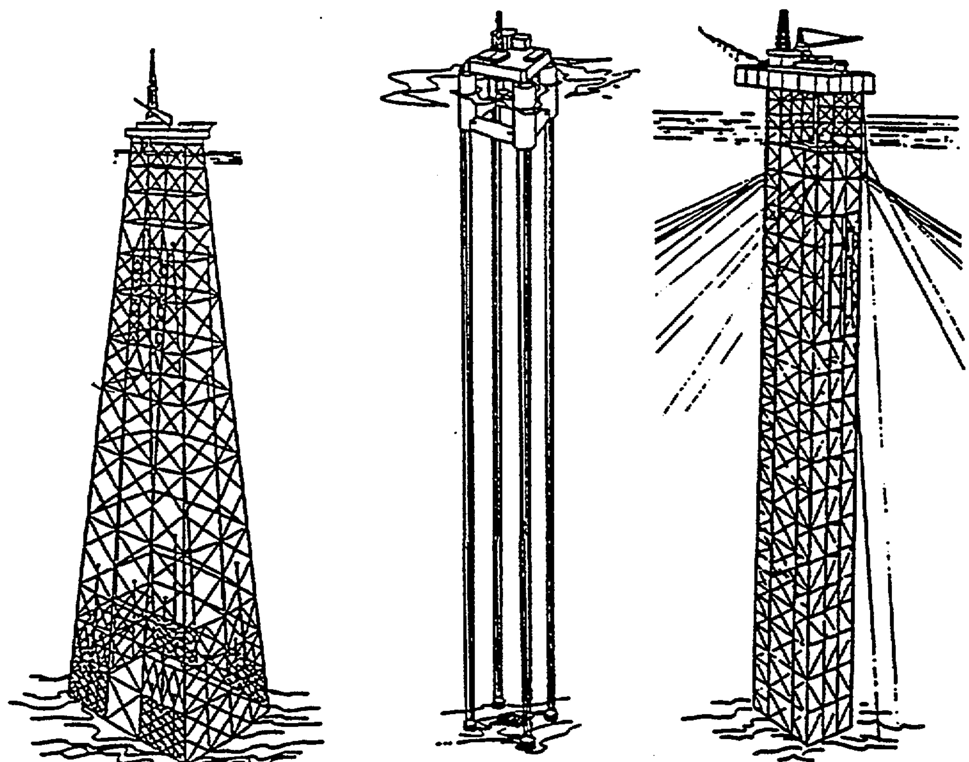


Figure 1. Typical Offshore Structures

The crucial importance of the crack tip dominance concept is that it creates a mathematical link between the factors remote from the crack tip (i.e., the crack size, shape and orientation; the applied and residual stresses; and the overall size and shape of the body), and the deformation state in the near-tip vicinity that actually controls the crack extension process. This effectively uncouples the structural analysis from the material property characterization testing, thus greatly streamlining the assessments of the strength and durability of a component that contains a crack.

At its simplest, an engineering structural component can be considered to deform in a completely linear elastic manner whereupon the technology that is appropriate is designated as linear elastic fracture mechanics (LEFM). In an LEFM approach the driving force for crack extension is given by a mathematical quantity called the stress intensity factor, commonly denoted by K. The point of unstable crack extension is simply found by equating K to its material property counterpart, the fracture toughness, $K_c$. Likewise, subcritical crack growth (i.e., when $K < K_c$) can be quantified in terms of functions of K.

For example, while there certainly are many more complex relations that can be used, the rate at which a crack grows under a cyclic loading (fatigue) is often expressed as

$$da/dN = C(\Delta K)^m \tag{1}$$

where a denotes the crack length, N is the number of load cycles, $\Delta K$ is the difference between the maximum and minimum K values in a load cycle, while C and m are empirical constants. Similar relations exist for the time-rate of environmentally-assisted crack growth. In either instance a basis exists for what is known in aircraft applications as damage tolerance analysis (DTA), and in the oil and gas industry as fitness for purpose. By estimating the time to failure of a real or postulated crack in a structure, a DTA approach allows NDE inspection targets and intervals can be established to allow continued operation with reduced risk of an in-service fracture incident.

While its concepts mirror all fracture mechanics applications, LEFM has a limited regime of applicability. The more general approach that is of interest here (of the many that are available) is elastic-plastic fracture mechanics (EPFM) in which significant metal plasticity can attend crack growth. The most used EPFM approaches employ either the J parameter or the crack opening displacement (CTOD) as the crack driving force. These two parameters are somewhat interchangeable because it has been shown that [4]

$$CTOD = d_n J / \sigma_0 \tag{2}$$

where $d_n$ is a constant of order one that depends on the strain hardening exponent n. Like K, J is a mathematically-based function of applied loads, crack size, and component geometry, but, unlike K, it is also influenced by the stress-strain behavior of the material. The CTOD, in contrast, is associated with a particular deformation state (i.e., a strip-yield zone) and material behavior ( i.e., elastic-perfectly plastic). This makes CTOD much more readily useable, but less precise and more conservative than are analyses based on J.

The J parameter, like all crack tip dominating parameters, has its material property counterpart. This is the J-resistance curve, a function of the extent of stable crack growth, which must be obtained experimentally - see Figure 2. The drawback to the J-based approach is that it assumes deformation plasticity - in essence, nonlinear elasticity - whereupon it also has limited applicability, albeit one that is broader than LEFM. Currently used generalizations of J and CTOD that include incremental plasticity are the $T^*$ and crack tip opening angle (CTOA) parameters described in the following.

## 3. Overview of Current EPFM Applications

There are many kinds of structural joints - mechanical (e.g., bolts, rivets), chemical (e.g., adhesives), and heat-induced fusion (e.g., weldments) - and a plethora of individual micromechanical failure mechanisms that can impair the integrity of each. Nonetheless, in a general sense, if failure is defined as the inability of a component to function as effectively as intended, there will be just two primary modes of failure. These are (1) gross deformation resulting from inelastic material behavior, and (2) the separation of the component into two or more pieces. A unified approach is given by the EPFM technology that has been successfully used in aircraft, transmission pipelines, and nuclear power plant structural integrity and durability applications, as described here.

### Aircraft Applications

Fracture mechanics owes much of its popularity to applications motivated by aircraft and aerospace vehicle applications. Because light weight and high strength materials are used in these applications, most of these are within an LEFM framework. However, a problem area has arisen in regard to high usage aging commercial aircraft which is giving rise to a need for direct consideration of crack tip plasticity. This is due to the instances of multiple-site damage (MSD) that have been observed - see Figure 3. The MSD scenario includes crack initiation, subcritical mixed-mode crack growth by environmentally-assisted fatigue, and the interaction and ultimate link-up of curved cracks to form a major fracture, as in fact occurred in the Aloha Airlines accident in 1988 [5]. All but the initial stages of this process occur in conditions where the validity of LEFM are not met, thus requiring use of EPFM.

While simplistic net-section yield, Irwin pseudo-yield and strip-yield models have been invoked to address the MSD problem in commercial "high time" aircraft, the rigorous computational mechanics models use either the $T^*$ or the CTOA parameters to model crack growth and link-up. Similar to Eqn (2), these can be related through

$$T^* = k\ \varepsilon\ \sigma_0\ \mathrm{CTOA} \tag{3}$$

where $\sigma_0$ is the ultimate stress, $\varepsilon$ is the size of the exclusion zone in the $T^*$ computation, and k is a dimensionless constant on the order of one. Thus, these two seemingly independent EPFM parameters are inter-linked, just as are J and CTOD.

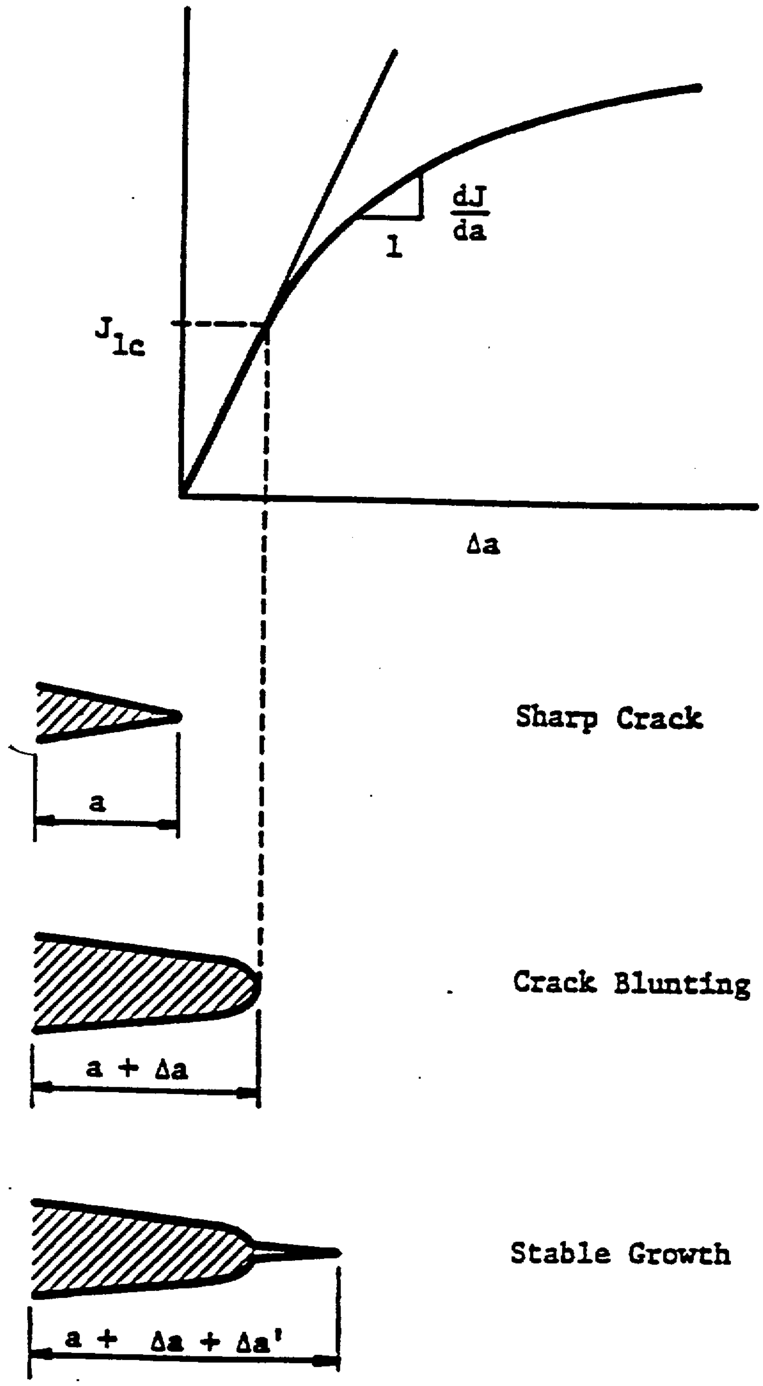


Figure 2. Physical Basis of the J-Resistance Curve and its Relation to CTOD

Figure 3. Multiple Site Damage at Rivet Holes in Commercial Aircraft Fuselage

The inter-relation between $T^*$ and CTOA makes it possible to use them in combination. While this has not been seriously done as yet, it is an attractive possibility because $T^*$ is particularly well suited for the initiation and small amounts of crack growth but, due to both computational and material property limitations, it is not readily applied for large amounts of growth. In contrast, because it quickly reaches a constant value during stable or unstable crack growth, CTOA is well suited for very large amounts of growth. But, it exhibits such severe transients immediately following initiation that it is not useful there.

**Transmission Pipelines**

Much like offshore structures, gas and oil transmission pipelines are vulnerable to environmental and third party impact damage, but do not generally employ redundancy in their design. Accordingly, the nature of the fracture mechanics applications are in either to quantify the so-called "leak-before-break" condition, or to preclude a long-running fracture propagation event - see Figure 4. Unfortunately, most work that has been performed in this application area tends to be of an ad hoc semi-empirical kind. The most rigorous work is that aimed at the prevention of a long-running pipe fracture through the use of the CTOA parameter [5].

This work utilized an elastic-plastic, large deformation, fluid-structure interaction computer model to determine the maximum crack driving force for a long running gas transmission pipeline. The material property that is then relevant is the $(CTOA)_c$ parameter which in this application represents the material resistance to fast, ductile fracture propagation. A new test procedure was evolved to measure this property of linepipe steel that is based on the hypothesis that the initiation energy can be separated from the propagation energy. Hence, by testing two specimens having different ligament lengths, the initiation energy can be eliminated, whereupon the remaining energy gives $(CTOA)_c$.

**Nuclear Piping Systems**

Just as the aerospace industry should be credited with providing the impetus for the popularization of LEFM, the nuclear power industry has been a prime motivator for developing and validating EPFM. In this industry the leak-before-break condition has played a key role in overcoming the excessive - and too often counterproductive - conservatism that has been invoked to insure the integrity of the system in the event of a seismic or other abnormal event acts upon a pipe that is degraded by fatigue, corrosion or mechanical damage.

The common approach is to utilize (1) a set of J-resistance curves generalized to include the effects of high cycle loading as would occur in a seismic event, and, (2) because of the great many different flawed pipe geometries that must be evaluated, the handbook approach to the computation of J that is illustrated in Figure 5.

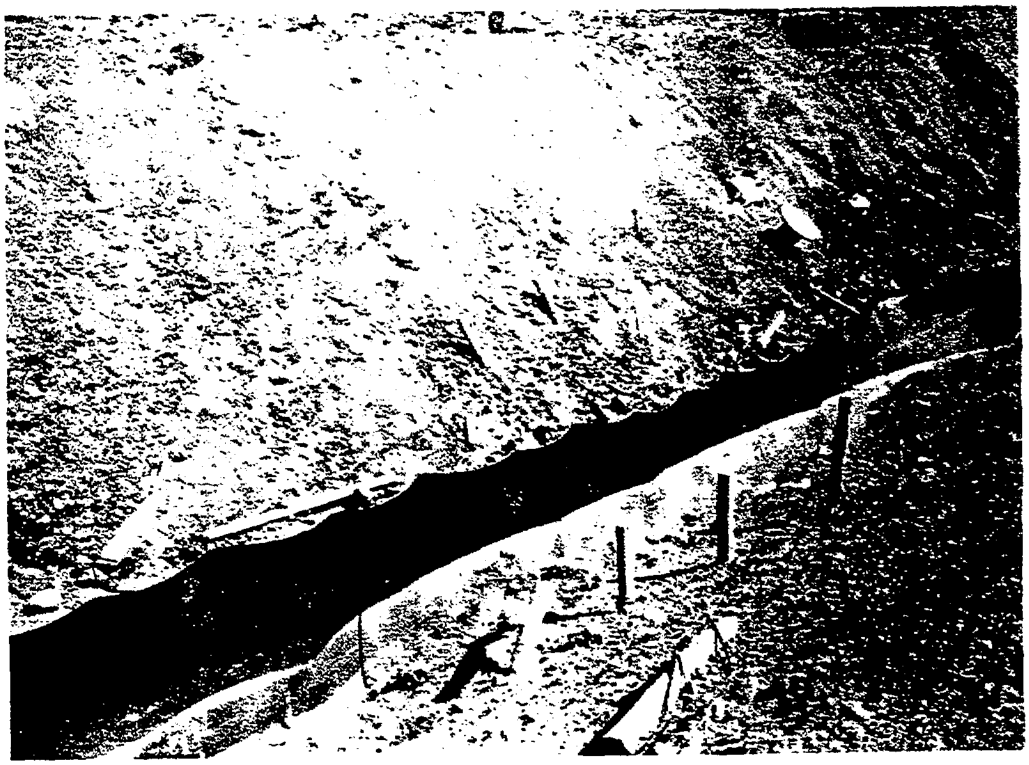

Figure 4. Large-Scale Fracture Propagation in a Gas Transmission Pipeline

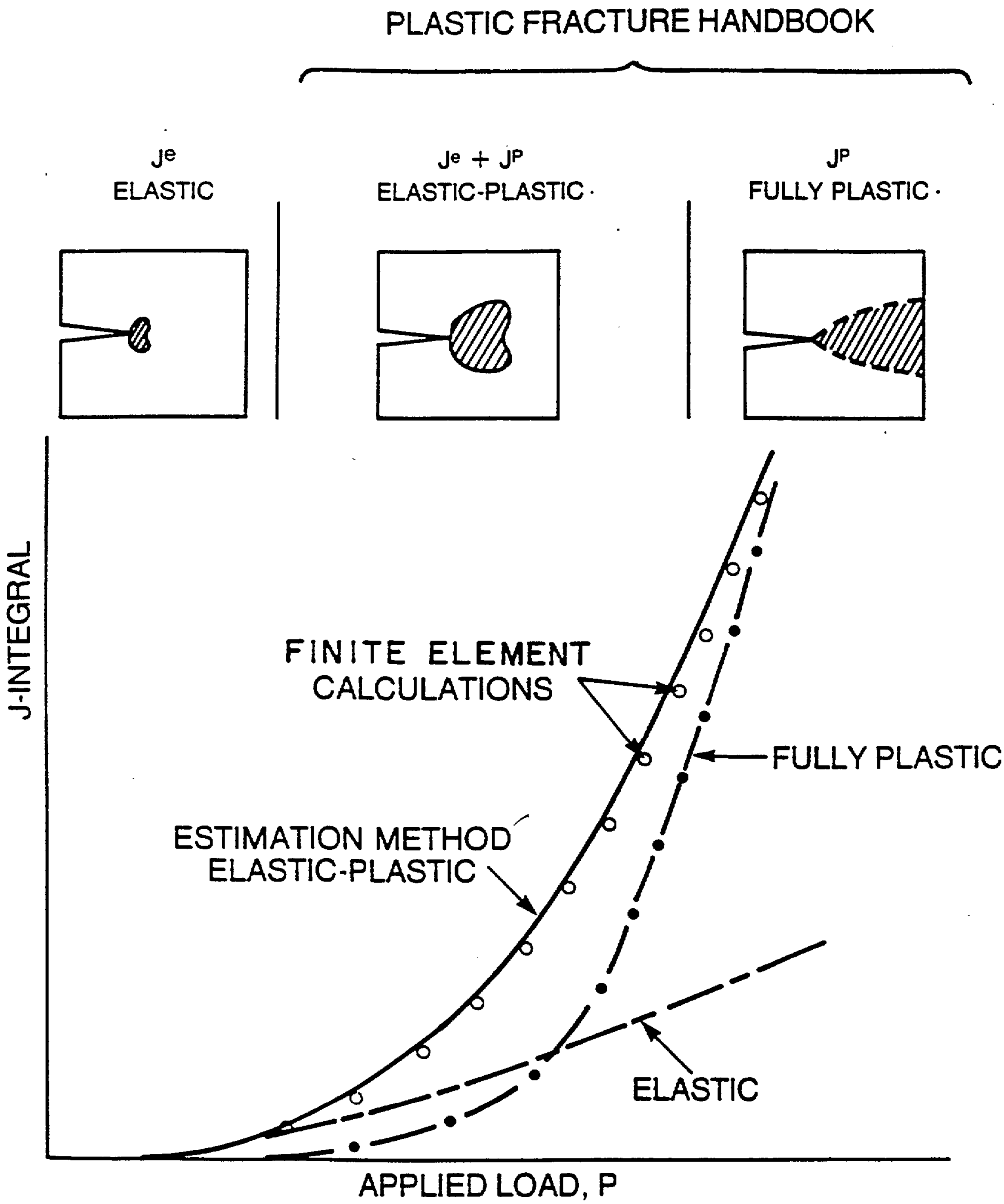


Figure 5. The Basis of the User-Oriented Handbook Approach to Determining the EPFM Crack Driving Force

Perhaps the most useful element of technology transfer that might be made from the nuclear to the offshore arena is the idea of the failure analysis diagram (FAD). The FAD, which takes into account the combined effects of elastic-plastic material behavior and of elastic-plastic crack growth, is highly useful for the very high toughness materials and weldments used in nuclear power plant piping where net section yielding is a viable failure mechanism. This important concept is illustrated in Figure 6 where the parameter $K_r$ denotes the ratio of the crack driving force to its critical value while $L_r$ denotes the ratio of the stress level to the yield stress. Note that $K_r$ can be taken in terms of J or of CTOD [7], and can be used together with resistance curve behavior to treat stable growth.

## 4. Conclusions and Recommendations

Because of conservative detail design, structural redundancies and their general newness, offshore structures have not experienced the catastrophic incidents that have impelled other industries to incorporate fracture mechanics and other structural reliability methodologies into their safety assessment procedures. Further mitigating against their use is the fact that the EPFM procedures that currently exist are incapable of treating the full complexity of crack growth and fracture in the area of highest vulnerability - the tubular steel joints. Nonetheless, if and when re-qualification action is undertaken, the industry and its regulators can expect to be confronted with identifiable crack-like damage in some existing structures.

As discussed by Rhee and Kanninen [5], there are a number of potential needs for fracture mechanics to be applied to offshore structures. The use of LEFM seems appropriate for the evaluation of fatigue crack growth, and is currently used by offshore engineers for that purpose - see reference [8]. However, to determine the strength of cracked joints where EPFM is necessary, there are intrinsic difficulties that limit the applicability of current methodologies. As illustrated in Figure 7 and 8, the difficulties include (1) three-dimensional crack/structure geometries, (2) non-straight and non-coplanar crack shapes, (3) non-homogeneous material with welding heat-induced microstructural property changes and residual stresses. Current EPFM techniques can cope with some of these issues, albeit not all at the same time.

To reduce the excessive conservatism that neglect of stable crack growth would engender, consideration of the non-self similar crack growth process that would take place in tubular joints must be made. Taking action before it is truly necessary is both wasteful of resources and, through improperly performed repairs, can cause the further damage that triggers an accident. At the same time, not taking action when it is prudent to do so risks a catastrophic event. The remedy for this dilemma is a theoretically-sound, industry-oriented elastic-plastic fracture mechanics methodology that takes into account the crucial factors that affect crack growth in tubular steel joints. The development of an appropriate EPFM methodology for offshore structures, because of the complexity of this application cannot be a modest effort, but must be one that enlists integrated contributions from computational analysis, experimentation, and material and weld characterization.

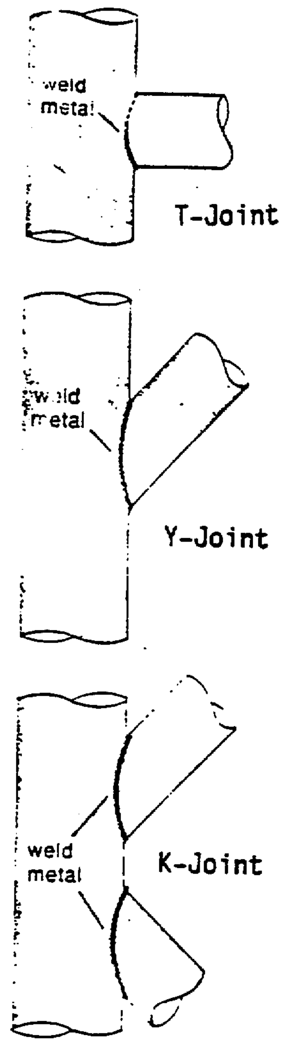


Figure 7. Common Types of Welded Tubular Steel Joints in Offshore Structures

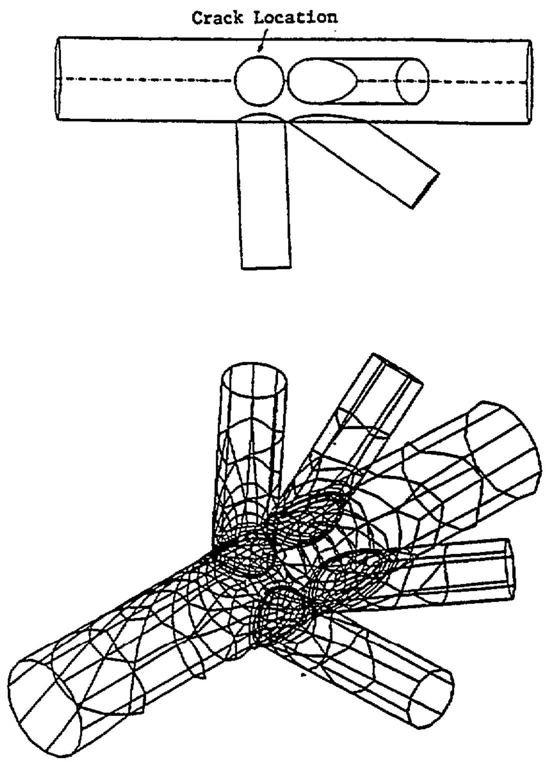


Figure 8. Typical Offshore Structural Configuration Illustrating Challenges to EPFM

The recommended research should be viewed as a two-phase process. First, three-dimensional, large deformation, elastic-plastic finite element analyses need to be performed to properly quantify the deformation states that exist during subcritical crack growth in flawed tubular steel joints (e.g., with Atluri's finite element alternating method). Also needed are small-scale material and weldment property characterization testing work (e.g., to quantify the $T^*$ / CTOA combination), and large-scale proof of principle experimentation on welded tubular joints. The second phase would utilize the resulting in-depth understanding to provide industry-oriented design and assessment guidelines

## 6. Acknowledgment

Many useful and stimulating discussions on offshore industry practices and problems, particularly in regard to the potential for the increased use of fracture mechanics in this industry, have been held with Dr. Chong Rhee, Conoco, Houston, Texas.

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# Workshop on Re-qualification of Tubular Steel Joints in Offshore Structures

## API Offshore Tubular Joint Research Center

Edison Welding Institute
University of Illinois
Ohio State University

Bill Mohr, EWI
September 5-6, 1995

EWI

requal.vg- 1

# First 4 Projects

- ✓ Stress Concentration Factors
- ✓ S-N Curves for Tubular Joints
- Profile and Post-Weld Improvements
  3-year project near completion
- Static Strength of Simple Joints
  Recently started - procedures/X joints

EWI

# Conclusions

- Modern SCF equations have significant advantages over older Kellogg type equations, particularly for T and X geometries.
- HSE guidance design lines confirmed on basis of tubular joint database.
- Selection of thickness exponent (0.25 or 0.3) is sensitive to particular choice of database elements.

EWI

# Conclusions

- Thickness effects for plates bracket those for tubular joints.
- Profile and post-weld improvement effects can be quantified and included in codes, but procedure requirements would need to be included.
- Beyond $10^7$ cycles, models of crack growth suggest a slope change from 3 to 4 for tubular joints and most transverse attachments.