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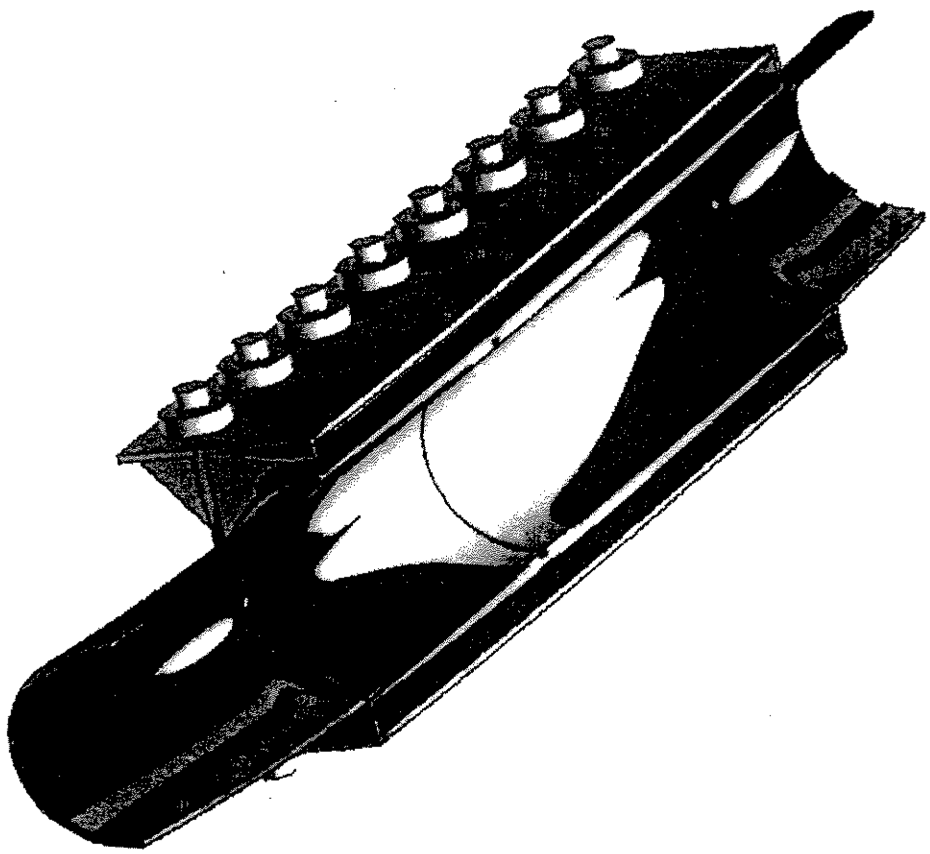
STRENGTHENING, MODIFICATION AND REPAIR OF OFFSHORE INSTALLATIONS



FINAL REPORT FOR A JOINT INDUSTRY PROJECT

CONTROLLED DOCUMENT

Document Ref. C11100R243

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Purpose of Issue	Rev	Date of Issue	Author	Agreed	Approved
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CONTROLLED DOCUMENT

STRENGTHENING, MODIFICATION AND REPAIR OF OFFSHORE INSTALLATIONS

DOC REF C11100R243 Rev 0 NOVEMBER 1995

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NUMBER	DETAILS OF REVISION
0	Final Report, November 1995

FOREWORD

This document has been prepared by MSL Engineering Limited for thirteen sponsoring organisations:

Amoco (UK) Exploration Company
British Gas plc
Chevron UK Limited
Elf Enterprise Caledonia Limited
Exxon Production Research Company
Health and Safety Executive (HSE)
Mineral Management Service (MMS)
Mobil North Sea Limited
National Energy Board (Canada)
Phillips Petroleum Company Norway
Shell UK Exploration and Production
Statoil
Texaco Britain Limited

The document has been prepared for the guidance of suitably qualified engineers to enable them to address the strengthening, modification and repair (SMR) of steel offshore installations. It is not in any way intended to have a legal status or absolve the engineer of responsibility to ensure that a safe and functional structure results.

The document is in seven parts:-

- Part I - Summary
  Executive summary, structure and usage of document, project description, outline contents of other parts.

- Part II - Assessment Engineering and Technique Selection
  A guide to the choices to be made when faced with decisions about strengthening or repairs, including legislative aspects.

- Part III - Design Recommendations
  A detailed description of the design techniques used in the formulation of a SMR scheme.

- Part IV - Background Data and Assessments
  The research, databases and appraisals which form the supporting evidence to the design recommendations in Part III.

- Part V - Clamp Studbolt Load Variation
  New project-generated data, both experimental and numerical, which rationalise this aspect of clamp design.

- Part VI - Diverless Implementation Studies
  Various feasibility studies and an actual case history on implementing repairs without the use of divers.

- Part VII - Bibliography
  Containing a detailed listing of a literature search.

Each part is written as a stand alone report making minimal reference to other parts. Nevertheless, it may prove necessary to refer to other parts, ie. to undertake preliminary concept design work (Part III) before finally selecting a particular SMR technique (Part II).

A project steering committee including representatives of the sponsoring organisations oversaw the work and contributed to the development of this document. During the life of the project the following individuals served on the committee:

Mr M Bærheim
Dr J Buitrago
Mr D Choat
Dr A F Dier
Mr D Galbraith (Chairman)
Mr M Lalani
Mr T McIntyre
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Mr N Nichols
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Mr W C Yu

The Project Manager at MSL Engineering was Dr A F Dier who carried out the work under the direction of M Lalani and with technical assistance from a number of MSL Engineering staff.

ACKNOWLEDGEMENTS

The provision of previously confidential data by Amoco (UK) Exploration Company, Chevron UK Limited, Exxon Production Research Company, HSE, MMS, Mobil North Sea Limited, and Shell UK Exploration and Production assisted in the successful outcome of the project. Acknowledgement to various authors of technical papers, who clarified and supplemented the information originally presented, is also made.

CONTENTS

Purpose of Issue	Rev	Date of Issue	Author	Agreed	Approved
Issued for comment	0	April 1993	Various	AFD	ML
Issued for Further Comment	1	September 1993	Various	AFD	AFD
Final Report	2	November 1995			

CONTROLLED DOCUMENT

STRENGTHENING, MODIFICATION AND REPAIR OF OFFSHORE INSTALLATIONS

PART I - SUMMARY

DOC REF C11100R225 Rev 2 NOVEMBER 1995

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NUMBER	DETAILS OF REVISION
0	Issued for Comment, April 1993
1	Issued for Final Comment, incorporating major revisions to Revision 0 issue. These have entailed moving Legislation chapter into Part II; Bibliography into a new Part VII; and moving of contents listing of Parts II to VI into an Appendix. All other text in chapters other than the first is new.
2	Final Report, November 1995

STRENGTHENING, MODIFICATION AND REPAIR OF OFFSHORE INSTALLATIONS

PART I - SUMMARY

STRENGTHENING, MODIFICATION AND REPAIR OF OFFSHORE INSTALLATIONS

PART I - SUMMARY

CONTENTS

I 1 EXECUTIVE SUMMARY

A joint industry project has been undertaken to examine the Strengthening, Modification and Repair (hereafter SMR) techniques for offshore structures. The project has sought to bring together in one document all the information necessary for the planning and execution of offshore SMR work.

Repair or strengthening work will probably be required at some time in the working life of an installation. The need for repair or strengthening work may arise for a number of reasons including the following:

- Fabrication or design defects
- Updates in code requirements
- Revised environmental criteria
- Damage in the installation phase
- Accidental damage due to ship impact or dropped objects
- Fatigue damage in service due to wave and current actions
- Corrosion and abrasion of steelwork in service.

Details of the scope and organisation of work are to be found in this Part of the document. Although the project has considered the repair of the topsides' structure, the main emphasis has been on the submerged portion of the installation where the greatest concentration of care and engineering judgement is required to design and install an effective repair.

Individual repair options which have been covered in detail are:

- Welding : one atmosphere, wet welding and hyperbaric techniques
- Weld improvement : grinding, peening and toe dressing
- Clamps : unstressed grouted, stress grouted, mechanical and elastomer-lined
- Grout filling : members and joints
- Member removal as a stand alone repair technique
- Mechanical repair system such as bolts, swaging and gripper locks.
- The use of adhesives and epoxy resins in repairs.

Throughout the work, consideration has been given to the practicality of the methods proposed. The theme of practicality has been extended to include the acceptance of a design or installation technique by certification and regulatory organisations. Such an approach is encapsulated by the extensive design and research advice given in Parts III and IV of the document, respectively.

The report forms an authoritative work of reference for SMR of offshore platforms. A number of hitherto unpublished design methods are presented in detail for grouted and stressed mechanical clamp design, and these, in part, have been based on new experimental and numerical data generated under this project. Design data of this ilk is of significant value to the engineering of repair solutions and it is vital that new data of this type can be brought into the domain of design technology through the aegis of such a joint industry project.

Aspects of operational philosophy which fall under the umbrella of "Health and Safety at work" are not directly considered in the report, although reference is made to the relevant governing legislation in Part II. Matters such as safe diving practices, use of potentially dangerous tools and equipment, and occupational health should be referred elsewhere. However this report does recognise that underwater SMR may be undertaken more safely in the future by the use of diverless intervention techniques.

The project work was undertaken by MSL Engineering Limited of Ascot, UK, subject to the control of the project steering committee which included representatives of each of the thirteen sponsor organisations.

I 2 DOCUMENT SCOPE AND DESCRIPTION

I 2.1 SCOPE OF WHOLE DOCUMENT

The scope of this and other parts of this document is to describe engineering techniques and good practices relating to the structural strengthening, modification and repair (SMR) of steel offshore installations including topsides and substructures. Pipelines, pipework, process equipment and foundations are not included in the scope. Non-structural repairs, eg sealing a corrosion hole in a seawater lift caisson with a glass-reinforced plastic (GRP) cover patch, are also outside the scope.

The following areas relating to SMR are covered:

- Scenarios leading to a requirement for SMR
- Guidance on assessment procedures to reduce the SMR effort
- Data to aid selection of SMR techniques
- Design recommendation for SMR techniques
- Full supporting background data and assessments for recommended design practices
- Diverless implementation studies.

The principal SMR techniques considered comprise: welding, weld improvement (including remedial grinding), clamp technology, internal grout filling (members and joints), bolting, member removal, the use of adhesives, mechanical repair systems and swaged connections. The design practices recommended in this document for these SMR techniques are usually outside the scope of recognised codes of practices, standards and regulations.

I 2.2 DESCRIPTION OF DOCUMENT

I 2.2.1 <u>Structure of Document</u>

This document is presented in seven parts as follows:

PART I - SUMMARY (this part)

The first part of the document (ie. this part) forms a precis of the entire document and provides a general summary of the main findings of the report. Detailed sections include the Executive Summary, the scope of work, and individual subject summaries.

PART II - ASSESSMENT AND TECHNIQUE SELECTION

The primary objective of Part II of the document is to identify appropriate strengthening, modification and repair schemes for a particular repair or strengthening scenario. Formalised assessment procedures are used to guide the engineer through the selection process. Supporting information is presented for the SMR selection process, enabling the engineer to initiate the appropriate engineering studies required for SMR work.

PART III - DESIGN RECOMMENDATIONS

Part III of the document presents detailed recommendations for the design and application of various offshore repair techniques. It can be used as an aid to the selection process or for the production of detailed design calculations. Topics covered include welding, weld improvement, clamp technology, grout filling and bolting. Practical issues are also addressed.

PART IV - BACKGROUND DATA AND ASSESSMENTS

This part of the document contains all the background data and screened databases. Details of the assessment of those data leading to the recommendations given in Part III are expanded, giving scope for the inclusion of future research work. Adhesives and swaging is not considered in this part of the document because of the sparsity of data relating to the method. Similarly the detailed background to member removal is not addressed.

PART V - CLAMP STUDBOLT LOAD VARIATION

In order to reduce the hitherto undue conservation in those clamp designs for which the governing criterion is that preventing the clamp halves being pried apart, laboratory tests and numerical analyses have been undertaken and more appropriate guidance has been formulated. Part V reports on the tests and analyses, and presents a design methodology which is conservative but not unduly so.

PART VI - DIVERLESS IMPLEMENTATION STUDIES

Part VI of the document presents various studies on the implementation of repairs without using divers. The studies encompass the deployment of heavy steelwork using currently available ROVs, an examination of various strengthening/repair systems and scenarios, and a feasibility study of inserting a new brace member into a structure using enhanced clamp technology. In addition, details are given of an actual diverless repair project which has demonstrated the cost and safety benefits that can accrue as well as confirming the feasibility of diverless implementation.

PART VII - BIBLIOGRAPHY

This final part of the document identifies further reading matter, enabling the specialist to refer to a larger database of information and to obtain detailed sources.

Layout of each part:

The pagination is such that any one page can be identified to the document part and chapter; the last number referring to the page's sequential position within the chapter. Figures and tables are located immediately following where they are first referenced in the text. A three-part numbering system has been used for both figures and tables: the first two numbers refer to the chapter and (second level) section numbers, the third to the sequential number within that section. The databases, which are in Part IV only, are collated at the ends of each relevant chapter. A common nomenclature has been used for Parts III and IV.

I 2.2.2 <u>Use of Document</u>

Each part of the document has been prepared to be self-contained as far as is possible. However, recourse to more than one part will usually be necessary during the totality of any one SMR event. The parts have been structured to follow logical phases that are undertaken when considering SMR. Decision-making thus follows a rational and sequential order. Initial appraisal of a problem may be undertaken using Part II alone. The specification of a design will require reference to Part III. If there are problems with the design or if the appropriate authority requires back-up data then this can be obtained from Part IV of the document.

Where appropriate, each part and section of a part are arranged so that the reader can extract sufficient information to the depth required as quickly as possible.

I 2.2.3 <u>Need for Document</u>

The need for such a document has arisen because of a number of factors including:

- There is an increasing requirement for SMR worldwide as the number of installations grows and the existing installations age with the attendant higher incidence of fatigue or corrosion damage. The demands of increased safety also give rise to increased levels of SMR.

- Presently, there is little or no guidance given for aspects relating to SMR techniques. What there is appears in diverse documents. Therefore, the collation of information into a single document is advantageous not only from a convenience point of view, but also it facilitates a direct comparison between different SMR techniques.

- Currently no guidance is available for:

    - assessment procedures
    - selection processes
    - applicability of particular SMR techniques to given scenarios.

- It is timely to review and update the existing guidelines using the benefit of experience gained in applying SMR. There are now, for some techniques, a larger database available to which modern statistical approaches can be applied. These statistical approaches can also be applied to databases which have not altered.

- All underwater repair work to date has been diver-assisted. Increased safety requirements in shallow waters and the positioning of platforms in water depths beyond diver access are leading to an increased recognition by the industry for diverless implementation systems.

In recognition of these and other aspects, a joint industry project was instigated by MSL Engineering. A brief description of the project is given in Appendix A and the results of the work undertaken is presented in this project document. The contents listing for all parts of this document, down to second-level headings, is given in Appendix B.

I 3 SUMMARY OF PART II - ASSESSMENT ENGINEERING AND TECHNIQUE SELECTION

Assessment engineering and technique selection form the foundation for SMR work and are advised as the first port of call for the engineer charged with a repair or strengthening project.

Part II covers three main aspects which perform the following functions:

- Scene setting (Scenarios)
- Analysis methods (Assessment procedures)
- Choosing the repair or strengthening method (Selection)

Sufficient detail is given in Part II to enable the reader to develop the engineering brief for preliminary design of the SMR. However it is recognised that additional work will be required to finalise any but the most trivial of offshore repair schemes, and the reader is referred to Parts III and IV of this document for specific technical and design data.

I 3.1 SCENARIOS

The review of scenarios gives a useful insight into problems which other organisations have faced, and emphasises the importance of correct identification of the cause of a problem. The diagnosis and contextual appreciation of an offshore structural problem will be improved by reference to this section of Part II.

The section is further reinforced by details of previous applications and case histories which are presented in Appendix A to Part II.

I 3.2 ASSESSMENT PROCEDURES

The Part II section entitled "Assessment Engineering" is primarily concerned with the engineering studies needed to develop an SMR project. The hierarchical structure of analysis techniques is described through comprehensively annotated flowcharts. Typical problems which are dealt with in this section of the document are:

- The use of nonlinear analysis methods.
- Combining fracture mechanics, fatigue calculations and structural performance checks in an SMR project environment.

- How to deal with "overstressed" elements

- Integrating physical research into a design method

The section explains how to tackle the systematic investigation of failed or distressed structural arrangements before making the choice of the repair or strengthening technique.

The investigation will initially focus on three spearhead activities - **evaluation** of structural adequacy, **identification** of the extent of repair and **optimisation** of the inspection programme. These activities may need to be scheduled in parallel when there is an urgent requirement to repair a defective structure. The adequacy of the existing structure needs to be assessed by analysis. The extent and type of repair needs to be identified and an offshore inspection will need to be mounted to clarify the condition of affected elements.

Depending on the outcome of these three spearhead activities, a policy for action will be developed. The policy may take one of a number of possible forms, ranging from "do nothing but document incident" to "remove and rebuild structure". Guidance is offered for the many other alternatives which lie between these two extremes.

I 3.3 SELECTION

The selection of a repair or strengthening solution is addressed within the broadest possible context. Figure 3.3.1 shows the extent of possible SMR techniques. It is argued that there are basically four approaches to SMR, which are:

- Remove damage

- Reduce the loadings

- Undertake a localised strengthening or repair

- Undertake a global strengthening or repair

Where physical repair options are deemed necessary, charts are provided to give initial guidance on the efficacy of any particular repair option. The charts are reproduced in Tables 3.3.1 and 3.3.2. Data presented includes a qualitative comparison chart, a scenario applicability chart and an overview of repair techniques. Additional guidance on the selection of a particular repair scheme may be obtained by reference to the later chapters in Part II which describe the various techniques in detail.

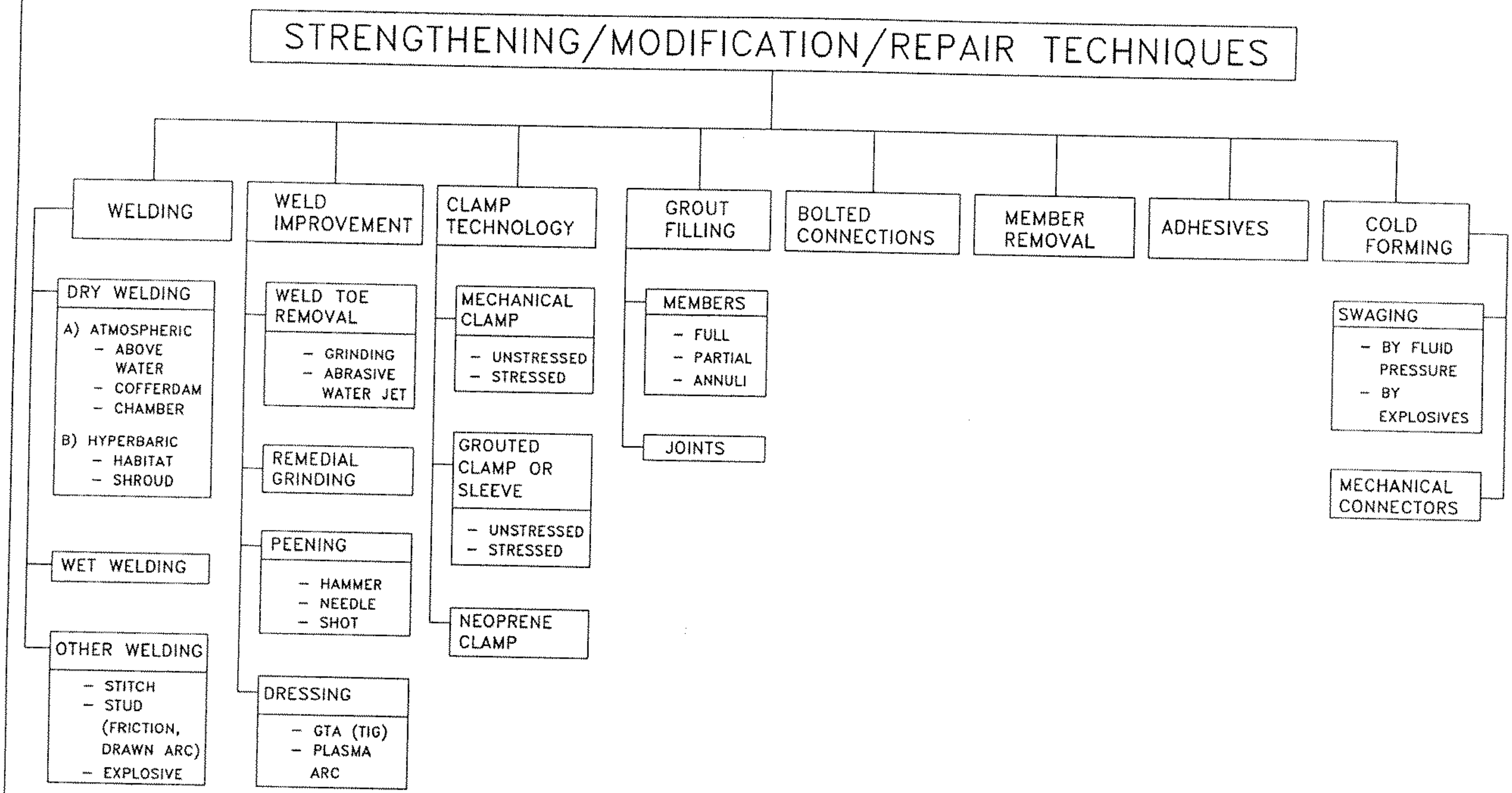

Figure 3.3.1: **Overview of techniques used in strengthening, modification and repair**

Technique	Used offshore	Data available for		Equipment needs	Offshore installation timescales	Onshore fabrication costs	Load penalties		Relative post installation inspection requirements	Design guidance available
		Static strength	Fatigue strength				Weight	Wave load		
Dry welding	yes	yes	yes	heavy	very slow	high for habitat	none	none	low	yes
Wet welding	yes	yes	yes	moderate	quick	none	none	none	low	yes
Toe grinding	yes	N/A	yes	low	moderate	none	none	none	moderate	yes
Remedial grinding	yes	yes	yes	low	moderate	none	none	none	moderate	yes
Hammer peening	yes	N/A	yes (for plates)	low	quick	none	none	none	moderate	no
Stressed mechanical clamps	yes	yes	yes	moderate	moderate	high	moderate	high	high	yes
Unstressed grouted connections	yes	yes	yes	moderate	moderate	low	low	low	low	yes
Unstressed grouted clamps without shear keys	yes	yes	yes	moderate	moderate	moderate	moderate	moderate	moderate	yes
Unstressed grouted clamps with shear keys	yes	yes	yes	heavy	slow	moderate	moderate	moderate	moderate	yes
Stressed grouted clamps	yes	yes	yes	moderate	slow	high	moderate	high	high	yes
Elastomer-lined clamps	yes	yes	yes	moderate	moderate	high	moderate	high	high	yes
Pressurised connections	no	yes	no	light	slow	moderate	low	low	low	no
Grout filling	yes	yes	no	light	quick	low	high	none	low	no
Bolting	yes	yes	yes	light	moderate	low	low	low	moderate	yes
Member removal	yes	N/A	N/A	moderate	quick	none	none	none	none	N/A
Adhesives	yes	yes	yes	light	quick	low	low	low	low	no
Swaging	yes	yes	yes	moderate	quick	moderate	low	none	low	yes

Note: N/A = not applicable

Table 3.3.1: Comparison of SMR Techniques

Technique	Defect								
	Fatigue crack	Non-fatigue crack	Dent	Corrosion	Inadequate static strength		Inadequate fatigue strength		Understrength topsides plating
					member	joint	high loads	fabr. fault	
Dry welding	yes(1)	yes	yes(3)	yes(3)	yes(1)	yes(1)	no	yes	yes
Wet welding	no(2)	yes	yes(3)	yes(3)	yes(1)	yes(1)	no	yes	no
Toe grinding	no	no	no	no	no	no	yes	no	yes
Remedial grinding	yes	yes(1)	no	no	no	no	no	no	no
Hammer peening	no	no	no	no	no	no	yes	no	yes
Stressed mechanical clamp	yes	yes	no	yes	yes	no	yes	yes	no
Unstressed grouted connection	yes	yes	yes	yes	yes	yes	yes	yes	no
Unstressed grouted clamp	yes	yes	yes	yes	yes	yes	yes	yes	no
Stressed grouted clamp	yes	yes	yes	yes	yes	yes	yes	yes	no
Stressed elastomer-lined clamp	no	yes	no	yes	yes(4)	no	no	no	no
Pressurised connections	yes	yes	yes	yes	yes	yes	yes	yes	no
Grout-filling	no	no	yes	no	yes	yes(4)	yes(4)	no	no
Bolting	no	yes	no	no	no	no	no	no	yes
Member removal	yes(5)	yes(5)	yes(5)	yes(5)	no	no	yes(5)	yes(5)	no
Adhesives	yes(6)	yes(6)	yes(3 or 6)	yes(3 or 6)	yes(3 or 6)	yes(6)	yes(3 or 6)	yes(3 or 6)	yes(3)
Swaging	yes(7)	yes(7)	no	yes(7)	no	no	no	no	no

Notes:
(1) Usually in conjunction with additional strengthening measures
(2) Except to apply weld beads in unstressed grouted connection/clamp repairs
(3) To apply patch plates
(4) Applicability depends on type and sense of loading
(5) If member is redundant
(6) Used as epoxy grout in clamps
(7) If damage can be by-passed

Table 3.3.2: SMR Techniques directly applying to various scenarios

I 4 SUMMARY OF TECHNIQUES

The following subsections are very brief summaries of the status of the various strengthening, modification and repair (SMR) techniques that are presently available. Naturally, much more detail is given in Parts II to IV of this document.

I 4.1 WELDING

There are many instances where welding has been successfully used and it will always remain a popular technique. For underwater repairs, the welding options available are:

- Wet welding (there are great regional differences as to the acceptability of this option)
- One atmosphere welding (for topsides' repairs, and for the submerged structure in cofferdams or pressure resisting enclosures)
- Hyperbaric welding where the welding atmosphere pressure is equal to the water pressure head at the bottom of the habitat.

The various advantages of each type of welding scheme are discussed in detail in other parts of this document. The individual welding processes and the operations required to design, build and operate underwater welding enclosures are also considered.

Currently, much work is being conducted at the Colorado School of Mines on wet welding.

I 4.2 WELD IMPROVEMENT

Weld improvement techniques are beneficial in alleviating fatigue problems, but not in addressing static strength issues. Indeed the techniques may even be detrimental in the latter case.

There are three main divisions of weld improvement techniques which may be considered: toe grinding, remedial grinding and hammer peening. The first two are extremely important in that they are often undertaken during the course of routine inspections. The third technique, hammer peening, looks very promising but has not yet been widely applied offshore.

Toe grinding is the deliberate removal of weld and base metal from the point at which the weld merges into the plate. The process is widely acknowledged as one of the principal techniques for the improvement of the fatigue life of welded nodes. Rotary burr grinding is more efficient that disc grinding and restores the fatigue life to 2.2 times that of the original, as welded, life.

Remedial grinding involves chasing out a crack in a welded joint and finishing the deep groove with a smooth profile. Here again, the fatigue life is restored to 2.2 times the as welded life. In some cases, the ground out weld can be left for the remaining life of the installation. Whereas the extent of toe grinding should not be subject to any particular limits, the position with respect to remedial grinding is completely reversed. Remedial grinding should only proceed as far along and into the depth of the weld as is necessary to chase out the crack. Design guidance is given in Part III to enable check calculations to be made on the static strength of remedially ground joints.

Hammer peening involves the introduction of a groove at the weld toe by plastically deforming the metal using a pneumatic or hydraulic 'percussion' tool. Available results indicate that the resulting fatigue life is a factor of 10 on the as welded life.

There are a number of other weld improvement techniques available which include TIG dressing, plasma arc dressing and stress relieving which are not discussed at length in the document, primarily because of their limited applicability for SMR operations. However, it is accepted that several of the other techniques may have future or specialised applications.

I 4.3 CLAMP TECHNOLOGY

Repair clamps constitute a most versatile SMR technique. They are normally constructed from low carbon steel and consist of a reinforcing sleeve which may include brace attachments and be split or hinged for ease of installation. Whether used for member or for node repairs, a clamp will typically take on the same general appearance as the part of the structure it is to reinforce. Clamps typically weigh between 0.5 and 10 tonnes and their fixture may require grouting, bolting or some other structural bonding technique.

Clamps may be used to repair a member, to install an additional brace, or to make a connection to a riser or caisson if there has been loss of support to such an appurtenance. However, the principal use of steel repair clamps is in repairing primary structural nodes. Four main design divisions are identified for steel repair clamps. The classification is based upon installation and fixing method rather than in terms of usage. The four clamp types are:

- The stressed mechanical (friction) clamp which uses long studbolts to produce a friction grip on the repaired structural elements.

- The unstressed grouted repair clamp which relies upon grout shear strength to effect load transference, working in a similar way to the ubiquitous grouted pile sleeve connection.

- The stressed grouted clamp is a hybrid of the above two clamp types with load being transferred partly by grout bond but mainly by friction. It is popular for its ability to tolerate dimensional problems whilst achieving load transfer in a reasonable sleeve length.

- The stressed elastomer-lined clamp which is similar to a mechanical clamp but for an elastomer (usually neoprene) liner that lays between the member and the clamp. Such clamps are often selected for repairs to caissons and other secondary structural elements.

The selection criteria for the various clamp types are discussed in sufficient detail to enable the reader to make a preliminary design choice. However, the practical issues are often governed by local design considerations, often involving a trade-off between structural joint complexity, accuracy of dimensional data and the space available for load transference.

The basis of any clamp design must be the establishment of the forces in the structural elements under calm and storm loading conditions. It is normal to evaluate such forces for the undamaged condition and then design the clamp so that load transfer is accomplished within the body of the repair clamp.

In the case of the unstressed grouted clamp, the relatively weak load transfer mechanism associated with the grout requires a significant length of clamp to transfer loads. This often places a limit on the use of this type of clamp.

The load transfer mechanism for stressed clamps, on the other hand, relies predominately upon friction to carry forces and moments from the damaged element into the repair clamp. This mechanism is efficient and therefore leads to short, light-weight, clamps. Stressed clamps, therefore, are the most popular choice for SMR.

The technology surrounding clamp installation is highly developed and a number of specialist companies are able to provide the necessary equipment and materials on a contract basis.

I 1.4 MEMBER GROUTING

Grout filling of structural members is a cheap and effective solution to several SMR problems. It is most beneficial for compressively loaded dent-damaged elements where the grout prevents any further disruption of the tubular section. In such cases the grout filling need only be extended to the immediate region of the damage.

If there is no damage to the member but an increase in axial load-carrying capacity is sought, then a few preliminary calculations will indicate if there is likely to be an appreciable increase in member capacity. If the member is short and has a low kl/r ratio then the axial load capacity in compression will be increased if the element is completely filled and load transference can be made from the node capping plate to the grout body. If the element has a high D/T ratio then there may be an improvement in load carrying capacity due to the prevention of local instability.

No benefit accrues under the following conditions:

- Tensile loads in element (unless there is a problem with hydrostatic / tension interaction collapse).

- Unchanging compressive or bending loads in element (because a repair can only carry a load which is applied after the grout has set).

- Partially filled compressive elements - because there is no guaranteed load transfer mechanism between the grout and the steel.

Practical issues include grout mix design, heat of hydration effects, grout shrinkage, nozzle location and ensuring complete element filling if deemed essential.

In general, grout filled elements and joints will be stiffer than their pure steel counterparts and as such they may attract more load in a statically indeterminate frame. In seismic zones, the additional mass associated with grout filling may need to be considered in the structural analysis. On any structure the additional weight imposed by grout may constitute a significant load, particularly if the grouted joint or element lies in a horizontal plane, as is the case with conductor bracing.

I 4.5 JOINT GROUTING

Grout filling of joints is considered to be a repair option which may be increasingly relied upon to bridge the gap between structural expectations and performance for older steel platforms.

Grout filling of tubular chord elements is used to improve the static strength of the joint and to extend the fatigue life of the connections made at the joint. The repair method has the advantage of introducing no additional wave and current loads on the platform but the increase in local deadloads can be appreciable.

Only grout filled joints have been considered in this document. It is therefore assumed that the material is cementitious and not reinforced. If the joint is simply pumped full of grout then a simple grout filled joint is created. In some instances there will be a concentric pile within the joint (as is the case with many leg sections), and the resulting construction is then termed **double skinned**. The grout is assumed to completely fill the available annulus in the joint.

In some cases the node can diameter is purposefully increased to admit the jointing of several different brace elements. Internal ring stiffening is sometimes used at the joint to increase the resistance of the chord wall to applied member forces. Both of these details, ie. enlarged can diameter and internal stiffening, will cause problems in ensuring complete grout filling.

The presence of the grout greatly reduces the amount of radial deformation (eg. ovalisation) of the chord wall. The recommendations in Part III present joint strength coefficients (Q_u terms) so that the static strength of grouted joints may be treated in a similar manner as for conventional unreinforced joints. The reduced chord wall bending also leads to lower SCFs and hence increased fatigue lives. The recommended approach for fatigue assessment follows an effective thickness methodology. Alternatively, either numerical modelling or laboratory testing may be used to verify both static and fatigue (SCF) behaviour. A joint industry project is currently underway at MSL Engineering which is generating SCF and ultimate strength laboratory data to put this technique on a firmer footing.

I 4.6 BOLTING

Bolts (including studbolts) are not extensively used to repair offshore structures by themselves because of the complicated details which need to be employed to join tubular elements with bolts. Plated and rolled steel sections used above the waterline present an opportunity for their use, however.

Bolts are an integral part of underwater steel repair clamps, and are found in riser and other pipe supports throughout the platform. They are also used to repair topsides' steelwork because a bolted joint can often be made in a hazardous area without the need to shut down the main process or to take extensive precautions against the threat of fire or explosion which may be caused by welding.

A bolt is normally tightened to produce a predetermined tension. The tightening up of bolts is largely achieved by:

- Application of torque using a wrench, or spanner. Flogging spanners are specially designed to allow a large hammer to be used to apply the torque. Hydraulic wrenches can be used both above and below the waterline.

- Tensioning systems - which extend the bolt, and hold the tension until the nut can be run down the thread at which time the tensioner is released and load transfer occurs. The system is normally used so that all bolts within a group can be stressed simultaneously. Tensioning systems are the preferred method for repair clamps.

The security of long term bolt tension is central to a safe bolted design. Proof of the applied tension at the time of bolt installation is the normal standard for acceptance. Bolt tension will be indicated by the pressure applied through hydraulic equipment. Good engineering practice demands that the loss of bolt tension through load transfer and elastic relaxation be calculated. Additional long term bolt tension losses can occur by creep in stressed grouted and elastomer-lined clamps.

There are physical limits placed on the bolt sizing, spacing and group number when tensioning devices are used. It is unlikely that bolt tensioners can be made to work efficiently for short bolts. If the tension is computed on the basis of applied torque then an assumption must be made about the coefficient of friction between the bolt head, washer and plate. This will usually involve underwater calibration testing.

Corrosion of bolting materials has been a problem and particular attention should be given to material selection of bolts on any part of an offshore installation.

I 4.7 MEMBER REMOVAL

An unusual approach to SMR which has become increasingly attractive with the advent of advanced analysis techniques is the removal of elements.

Structural member removal may be a staged development in a larger repair scheme or may constitute a repair in its own right. In either event the structural framework will need to be checked to ensure adequacy under the proposed loading and framing regime.

Changed loading arrangements or design rules may mean that there are reduced requirements for supporting steelwork within the structure than was originally considered necessary. In particular, conductors may be able to span a larger distance than was originally considered safe and this fact may substantiate a scheme for dismantling part or all of a fatigue damaged conductor bracing frame. There are a number of structural elements which although not strictly superfluous, may not be required once the structure has been put in place.

Choosing the right technique for cutting the member and making sure that the cuts can be made safely and accurately is a major design consideration. It may be necessary to undertake a series of structural frame analyses to determine the stability of lifted assemblies and the magnitude of spring back forces which may be released at the instant of severance.

The cutting of a brace from a structure leaves a remnant stub, often with a rough cut edge. It is conventional to grind back the stub. There are very few circumstances in which it would be admissible to leave a rough cut stub on an existing structure.

I 4.8 ADHESIVES AND EPOXY GROUTS

There are three main structural uses of resins offshore: as adhesives, as grout and as the matrix in composite materials. Whilst there are a range of structural resins, including acrylic, cyanoacrylic, polyester, vinylester and urethane products, the epoxy resins are the most commonly used and are available as one-component and two-component resins. They may have fillers added to increase the thixotropy or to improve the strength of the final joint. Various catalysts or inhibitors might be added to modify the curing behaviour.

By comparison with their long-standing successful use in the aerospace industry, the use of resins offshore is not well-established, and there is a considerable degree of suspicion by engineers concerning their reliability, especially as adhesives. Even under controlled conditions, adhesives can give unpredictable results. It is this uncertainty of the behaviour of the joints made, rather than any limitations of adhesives themselves, that has restricted their use.

However, composites represent one specialised repair and strengthening option which may see increased usage in the next decade of offshore SMR work. These two-phase materials consist of layers of long fibres (glass or carbon fibre) within a resin matrix; the layers being stacked so as to achieve, the desired properties in one or more directions. High strength and stiffness (even exceeding those of steel) can be achieved with low weight. Composites have already been successfully applied to offshore installations and ships in the context of SMR. Presently, a joint industry project is underway at MSL Engineering in conjunction with Devonport Management Limited (DML) in order that these materials can be exploited.

Use of a particular epoxy for offshore applications needs a partnership approach with client, supplier and designer taking responsibilities for various aspects of the project.

I 4.9 COLD FORMING

Two broad categories of cold forming techniques are recognised: use of mechanical connectors and swaging.

Several specialised mechanical connectors have been devised by wellhead supply companies for use in down hole and subsea installations and also by pipeline repair companies. Key elements of mechanical connectors are:

- the connection can be made quickly
- full strength is obtained immediately on installation
- suitable for permanent or temporary SMR (some connectors are reusable)

- may be modified for ROV installation.

Several systems are the subject of patent protection. Individual options may be developed in conjunction with the relevant equipment supply company.

A swaged connection between two concentric tubulars is formed when the inner one is expanded (by internal pressure) and is plastically deformed into grooves machined in the outer member. The technique has been used to make pile-sleeve connections offshore. The advantages of a swaged connection which may potentially be exploited in SMR operations are:

- the connection be made quickly
- full strength is obtained as soon as the inner member has been expanded
- clear inspection criteria for adequacy of joint formation
- amenable to ROV operation.

Details of a proprietary pile connection system have been described in the literature. This and other commercial systems may be covered by patent protection.

APPENDIX A

PROJECT DESCRIPTION

A PROJECT DESCRIPTION

A.1 INTRODUCTION

There are many reasons for SMR but they may be categorised into four main groups:

- modifications to increase the operational safety of existing installations
- repairs following damage (eg. from supply vessel collision, fatigue cracking, etc.)
- strengthening required by code changes, regulatory bodies or certifying authorities or to extend platform life
- modifications to permit increased loadings (from upgraded topsides, additional appurtenances, etc).

As platforms age, SMR is more likely to be needed. Possible reasons are a higher incidence of detected fatigue cracks or the need for platform upgrading as a result of field development. In the short to medium terms, the modification of topside structures to increase their operational safety may necessitate strengthening of the supporting components and systems. In recognition of these aspects, operators are paying greater attention to SMR so that the structural integrity of existing installations can be maintained with confidence. It is against this background that this development project was undertaken.

A.2 OBJECTIVES

The two main overall project objectives were as follows:

- To develop a comprehensive Design and Applications Guide for all present day strengthening/repair techniques.
- To extend, enhance and develop selected and appropriate techniques for application and implementation using remote-controlled operation systems. These diverless systems should be operational for water depths up to 1000m.

A.3 WORK PROGRAMME

The work required to achieve the stated project objectives was arranged in 18 work packages, each with defined objectives, input requirements and deliverables. A brief description of these work packages is given below, and the interrelationships between them are indicated in Figure A.1.

WP1 - Management and co-ordination:
Technical auditing, progress and cost control functions.

WP2 - Data capture:
Literature searches, capture of confidential data, and interviews with various types of organisations.

WP3 - Minimize SMR for new structures:
Collation of techniques for new designs.

WP4 - Assessment engineering:
Catalogue basic, refined and advanced analysis techniques to reduce SMR effort.

WP5 - SMR scenarios:
Discuss scenarios leading to a requirement for SMR for both damaged and intact structures.

WP6 - SMR techniques; appraisal:
For each SMR technique describe background, concepts, description, controlling and limiting parameters, timescales and inspection requirements.

WP7 - SMR techniques; research:
For each technique describe past, current and planned research. Describe methodology (experimental and/or theoretical), results and compile into databases.

WP8 - SMR techniques; applications:
Catalogue applications to date in terms of platform type, cause of defect, choice of technique, certifying authority involvement, etc.

WP9/WP10/WP11 - Design approach and criteria:
For each technique present design philosophy and criteria, with applicable codes and standards. Describe survey requirements. Give design equations, factors of safety, and ranges of application.

WP12 - Design document:
Collate findings from work packages WP2 to WP11 and present in a design dossier of recommended practices.

WP13 - Non ROV intervention methods:
For each of air diving, saturation diving, atmospheric diving suits and manned submersibles, catalogue scope and limitations, legislation, logistics, equipment, reliability and research and development.

WP14 - ROVs
Examine ROV technology with respect to capabilities, reliability, equipment operational constraints, track record and research and development.

WP15 - Deepwater premise:
Define scenarios, at a water depth of 1000m, complete with technical and operational criteria for SMR by remote intervention.

WP16 - Feasibility study on diverless SMR:
Appraise SMR techniques for deepwater application. Where appropriate, detail required modifications and prepare specification and drawings for ROV compatibility.

WP17 - Diverless implementation:
Feasibility study on installation schemes and procedures. Detail installation aids and identify technical and operational aspects for future development.

WP18 - Reporting:
Progress and technical reporting; presentations to project sponsors.

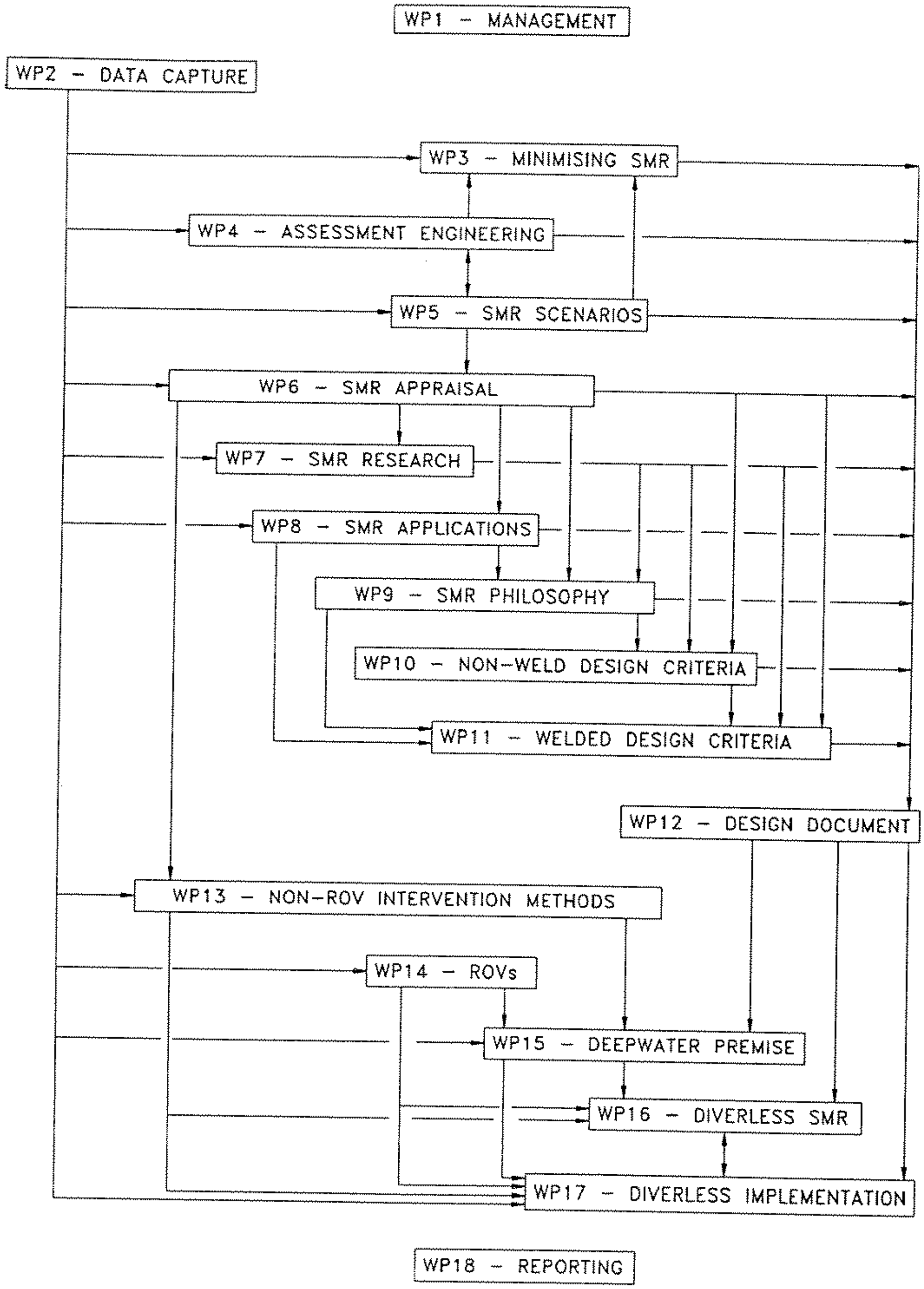

Figure A.1: Interrelationship of work packages

APPENDIX B

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BIBLIOGRAPHY

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Final Draft Issue	1	September 1993	Various	AFD	AFD
Final Report	2	November 1995			

CONTROLLED DOCUMENT

STRENGTHENING, MODIFICATION AND REPAIR OF OFFSHORE INSTALLATIONS

PART II - ASSESSMENT ENGINEERING AND TECHNIQUE SELECTION

DOC REF C11100R224 Rev 2 NOVEMBER 1995

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STRENGTHENING, MODIFICATION AND REPAIR OF OFFSHORE INSTALLATIONS

PART II - ASSESSMENT ENGINEERING AND TECHNIQUE SELECTION

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STRENGTHENING, MODIFICATION AND REPAIR OF OFFSHORE INSTALLATIONS

PART II - ASSESSMENT ENGINEERING AND TECHNIQUE SELECTION

CONTENTS

II 1 INTRODUCTION

II 1.1 GENERAL

It is the primary objective of this Part II to identify appropriate strengthening, modification and repair (SMR) schemes and techniques for a given scenario in the context of legislative requirements and present-day knowledge and experience of SMR.

Each scenario presents its own particular problems and constraints and this uniqueness does not permit a presentation of ideal solutions to all scenarios. Nevertheless, guidance is included herein to indicate which repair techniques have previously been applied successfully for different classes of problems.

Formalised assessment procedures are used to guide the engineer to a safe and economic solution. In some cases, it may prove necessary to carry out some preliminary design/sizing (eg. of a clamp) to enable a final selection to be made between two or three candidate repair schemes. In these cases, reference to Part III will be required for recommended design practices. Normally, however, the selection of a repair scheme can be made on the basis of this Part II alone and it has been prepared as a stand alone document with this is mind.

It is also recognised that operators will have different experiences with the various techniques and this will influence the final selection. Furthermore, there are important and significant regional differences, especially with respect to wet welding (which is common in American waters but not in the North Sea) and clamping (common in North Sea but not in American waters). Finally, there are statutory and operator preferences for implementation methods; there is greater emphasis on diver safety even to the extent of a tendency nowadays to remove man from the water.

II 1.2 STRUCTURE AND USE OF PART II

The use of this Part II is summarised in Figure 1.2.1.

Section II 2 summarises the various causes that can give rise to the need for strengthening, modification or repair (SMR). It is emphasised that it is vital to understand the cause of a problem to ensure a successful SMR.

Section II 3 deals with the engineering assessment of the perceived problem. It is the purpose of the section to minimise or eliminate the requirements of SMR. The techniques that can be brought to bear on tackling the problem, through analysis, are addressed. A second objective of analysis is to validate a proposed SMR scheme and this, too, is addressed within Section II 3.

Section II 4 summarises general legislative requirements relating to SMR.

Section II 5 is concerned with preliminary identification/selection of SMR techniques. It presents summary charts of the applicability of techniques for various scenarios and allows a rapid appraisal of various aspects to be made on a comparative basis.

Final selection is aided by referring to Sections II 6 to II 13, as appropriate, which deal with individual classes of techniques, their strength and weaknesses, previous applications, the extent of background research, etc.

Selected case histories are given in the Appendix. The case histories demonstrate the application of various techniques and summarise the lessons learnt.

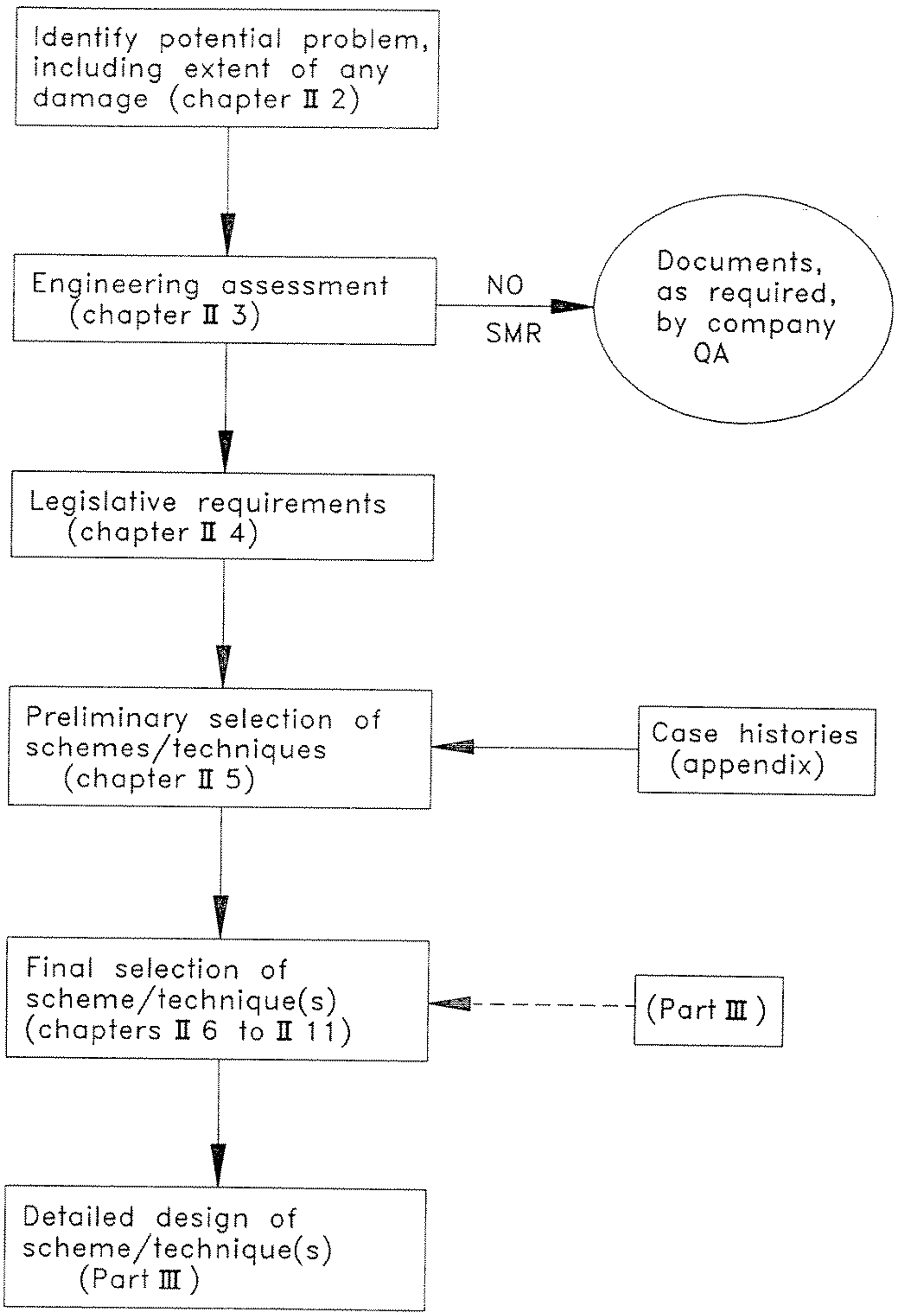

Figure 1.2.1: Flowchart demonstrating use of Part II

II 2 STRENGTHENING, MODIFICATION AND REPAIR SCENARIOS

II 2.1 INTRODUCTION

It is the purpose of this Section II 2 to catalogue the scenarios which may give rise to a need for SMR and to describe qualitatively the types of damage that may occur.

A useful distinction can be drawn between structures which have suffered some form of damage and those which are essentially intact. In the case of the former, a survey is required to map the extent of the damage (as well as the local geometry around the damage site) to enable a rational appraisal of SMR requirements to be made.

It is important to identify the root cause leading to a requirement for SMR as this may affect the selection of SMR techniques to be utilised.

II 2.2 DAMAGE SCENARIOS

There are several causes of damage quoted in the literature. It is not the purpose here to reflect on the extent and occurrence of these but rather to identify the resulting types of damage sustained as it is this that impacts on the relevance of the SMR techniques that may be adopted. The causes, and the resulting types of damage include:-

- Fatigue loads

  These, of course, may cause fatigue cracks and are a design consideration. However, an ignorance of both environmental loads and aspects of structural behaviour has led to an occurrence of fatigue cracks. They may be severe, even leading to member severance. The growth of the crack may affect other members at a joint; cracking originally in a secondary brace member may eventually grow into and affect the primary member.

- Dropped objects

  Examples of dropped objects cited in the literature comprise tubular components (eg. drillstrings and piles), a link bridge, a lifting crane and miscellaneous items such as wire ropes. A further cited example of a dropped object is a complete horizontal frame which, having failed at the connection to the leg members, dropped onto the next lower frame and caused extensive further damage.

  Dropped objects may cause damage varying from denting, bowing and holing of members, to member severance and joint tearing and deformation. In the case of wire ropes which become entangled onto horizontal members, wave action can make the wire rock to and from thereby causing the wire to chafe the member. In at least one incidence wire chafing has led to the member being cut through.

- Vessel collision

Supply boat collisions, and at least one case of submarine impact, have caused denting and bowing of members. Dragging of anchor lines across the structure has also occurred.

The relative magnitude of denting and bowing depends on the D/t ratio, the overall slenderness of the member and the degree of restraint (rotational and axial) afforded to the ends of the member by the rest of the structure. In addition to bowing and denting, deep scratches and gouges can be formed.

- Corrosion

Excessive corrosion may be relatively general, as in the case of an underdesigned, or failed, cathodic protection system, or rather localised, as in galvanic (bimetallic) corrosion of caissons housing stainless steel pump/strainer components. In either case thinning and/or perforation of the carbon steel occurs.

- Damage during installation

Most of the incidents of damage relate to pile driving operations. Piles falling through the guides damaging the guides and skirt structures can be classified as dropped objects as above. However there have been reports of extensive cracking at joints attributable to excessive vibration, resulting from the hammer blows. The fouling of a member on the installation barge during platform launch caused it to bow in one case.

- Welding and/or fabrication fault

Under this category can be included general errors such as use of inferior materials, incorrect member sizes, or incorrect member positions (and even omissions). More commonly, however, errors are of a detailed nature such as lack of penetration or excessive undercutting, and the failure to provide vent holes for intended flooded members. The latter has caused implosion of such members on installation. The former can lead to unexpected fatigue cracks.

- Explosions

  In more than one case, local enterprising fishermen have adopted an extreme method to catch the fish which tend to congregate around platforms. This consisted of detonating dynamite in close proximity to the platforms, stunning the fish which then rose to the surface. The resulting damage consisted of severe denting deformation to members (enough to give a crescent shaped cross-section).

- Ice

  In arctic waters, pack ice can impart substantial forces onto a platform. In one reported incidence, build-up of ice within the structure caused one end of a major diagonal bracing member to sever.

- Build-up of drill cuttings

  Under certain conditions, drill cuttings can accumulate and bear onto lower frames or members, thereby damaging them.

- Underdesign

  Underdesign can cause member buckling, or joint failure either statically with commensurate permanent deformation and possible tearing, or from fatigue with associated cracking.

To summarise, damage, depending on its cause, may manifest itself as dents, bows, permanent deformations, loss of thickness, gouges, cracks, tears, holes and severance of members. These forms of damage can occur singly or in combination.

The damage may or may not be important to the integrity of the platform. This depends on the severity of the damage, the loads carried by the damaged component and the degree of structural redundancy. Each situation has to be assessed individually to enable a rational decision to be made on whether repair and/or strengthening is required.

II 2.3 SCENARIOS FOR INTACT STRUCTURES

The scenarios that may give rise to the need for SMR of intact structures fall into one of three broad categories:-

i. Change in Platform Operation

   In the refurbishment of topside structures, the placement of additional equipment or the upgrading of existing equipment may lead to the imposition of increased superstructure and substructure loadings.

The development of marginal or satellite fields using sub-sea systems is finding increasing favour. This often places an additional burden on receiving 'parent' platforms through, for example, the placement of additional risers, other appurtenances or additional topside equipment.

In either case, the integrity of the platform requires reevaluating and this reassessment may indicate a need for modification or strengthening of the supporting structure.

ii. Availability of New Information

During the lifetime of a structure and as the industry experience and knowledge grows, new information (through code updating, for example) on environmental loadings and/or structural response can indicate that the platform is underdesigned. Again, a structural appraisal would confirm whether SMR is required.

This is particularly important for older installations which may be required to remain in service beyond the originally perceived service life. Within the context of this requirement, fatigue life could be a dominant consideration as fatigue is a time-dependent phenomena.

iii. Measures for Increased Safety

In recent times, a number of measures have been proposed to increase the operational safety of existing installations. For instance, the Cullun Report on the Piper Alpha disaster includes a number of safety-related recommendations, and this may lead to a requirement to strengthen or modify the superstructure and substructure.

Certification and/or regulatory requirements may stipulate the necessity for structural integrity evaluations at regular periods.

There are instances where a requirement for SMR is operator led. For example, should a reanalysis of a structure indicate that a member or joint is overstressed (statically or from a fatigue standpoint) before any damage has actually occurred, or where similar platforms have already suffered some form of damage to which the subject platform is also likely to sustain, the operator may decide on SMR measures. Even in those cases where it can be shown that the member concerned is redundant, it will usually be required to take steps (for example, placing additional ties or removal of the member) to avoid the consequences of the member potentially separating from the structure and causing further damage on its way down to the seabed.

II 3 ASSESSMENT ENGINEERING

II 3.1 INTRODUCTION

The purpose of this Section II 3 is to catalogue the techniques for assessing the structural integrity of existing offshore installations, in either intact, damaged, or repaired states.

The objectives of an engineering assessment can be divided broadly into three areas:

i. Evaluate structural adequacy and integrity of intact or damaged structure.

   An approach to global analysis is presented which follows a stepwise path of increasing refinements and enhancements. Included in the approach may be risk and reliability assessments. It is the intention, within the approach presented, to demonstrate the areas in which research work and advanced analysis can usefully be brought to bear on a problem.

ii. Identify and optimise extent of any repair or strengthening work and the associated urgency.

   Global and local SMR schemes should be considered in terms of their effects on the structure as a whole. The final design should reflect the degree of connectivity assumed in the assessment. It may be possible to delay the implementation of a SMR scheme until a more favourable time; such decisions would typically be taken on the basis of known and predicted operational criteria and a risk assessment.

iii. Optimise specification of inspection programmes.

   Inspection can be optimised by specifying an inspection programme based on the criticality of components to the integrity of the structure as a whole rather than on a member by member basis.

There are a number of possible outcomes from the assessment, as follows:-

i. No strengthening or repair

   Under more refined analysis, the structure is demonstrated to have adequate structural integrity (with sufficient redundancy) to withstand all the design load cases.

ii. <u>No strengthening or repair but inspect</u>

Under more refined analysis, defects are predicted to grow at an acceptable rate and there is no likelihood of rapid unstable growth. This situation would be subject to periodic review following inspections, whose intervals would be set within the assessment.

iii. <u>No strengthening or repair but change operating procedures</u>

Under more refined analysis, the structure is demonstrated to have adequate structural integrity to withstand most of the design load cases. The operating procedures should be changed to reduce or eliminate exposure to those design events which the structure is not able to withstand (eg. modify vessel operating procedures).

iv. <u>No strengthening or repair but set demanning restrictions</u>

Under more refined analysis, the structure is demonstrated to have adequate structural integrity to withstand most of the design load cases. Certain non-preventable design events lead to unacceptable levels of risk. Therefore, the platform operations should be made safe and should be evacuated in advance of the occurrence of these events, assuming that they can be forecast in sufficient time.

v. <u>Local strengthening or repair</u>

Following engineering assessments, it is determined that local SMR is needed to provide adequate component resistance. The repair does not change the load distribution within the structure.

vi. <u>Global and local strengthening or repair</u>

Following engineering assessments, it is determined that SMR is needed which changes the load distributions within the structure through provision of alternative load paths. Within the new arrangement, certain local component SMR may still be needed.

vii. <u>Total strengthening or repair</u>

Following engineering assessment, it is determined that total SMR is required, ie. the provision of systems external to the installation, and attached to the installation (eg. prop systems founded next to the installation). In this context, global and local SMR may also be needed.

II 3.2 BACKGROUND AND OVERALL APPROACH

The reappraisal of existing offshore installations forms an important and integral part of offshore engineering. A number of reasons give rise to the need for reappraisal as discussed in Section II 2.

An approach to assessment engineering is described herein which outlines a set of procedures which may be followed in the assessment of existing installations. Two basic approaches are outlined, as follows:-

i. An approach for the assessment of an intact structure.

ii. An approach for the assessment of a damaged structure.

Figures 3.2.1 and 3.2.2 respectively present, in flowchart form, a set of activities for the above two cases. For both intact and damaged structures, the steps involved in global analysis are discussed in more detail in Section II 3.4. The following comments apply to Figures 3.2.1 and 3.2.2:-

a. For intact structures, an updated analysis model is created which reflects the current structural condition, and code checks are carried out for static and fatigue strength. At the end of this stage, a review of the findings is carried out, and decisions in respect of the following can be made, based on relative costs, risks and benefits:-

   - further analysis
   - member removal
   - load reduction (member cleaning programme or topsides reduction)
   - inspection
   - do nothing
   - strengthening/repair
   - change operating procedures.

   Each of the subsequent phases of the assessment procedure follows a similar pattern. Review actions are set at the end of each phase when the options listed above should be evaluated.

b. For damaged structures, the methodology for assessment shows some differences from the procedure for intact structures. As damage is present, code checks alone are not sufficient, and engineering expertise must be brought to bear on the local assessment of the damaged element. The phase involving (further) refined analysis modelling contains an extra step to incorporate the damaged member response.

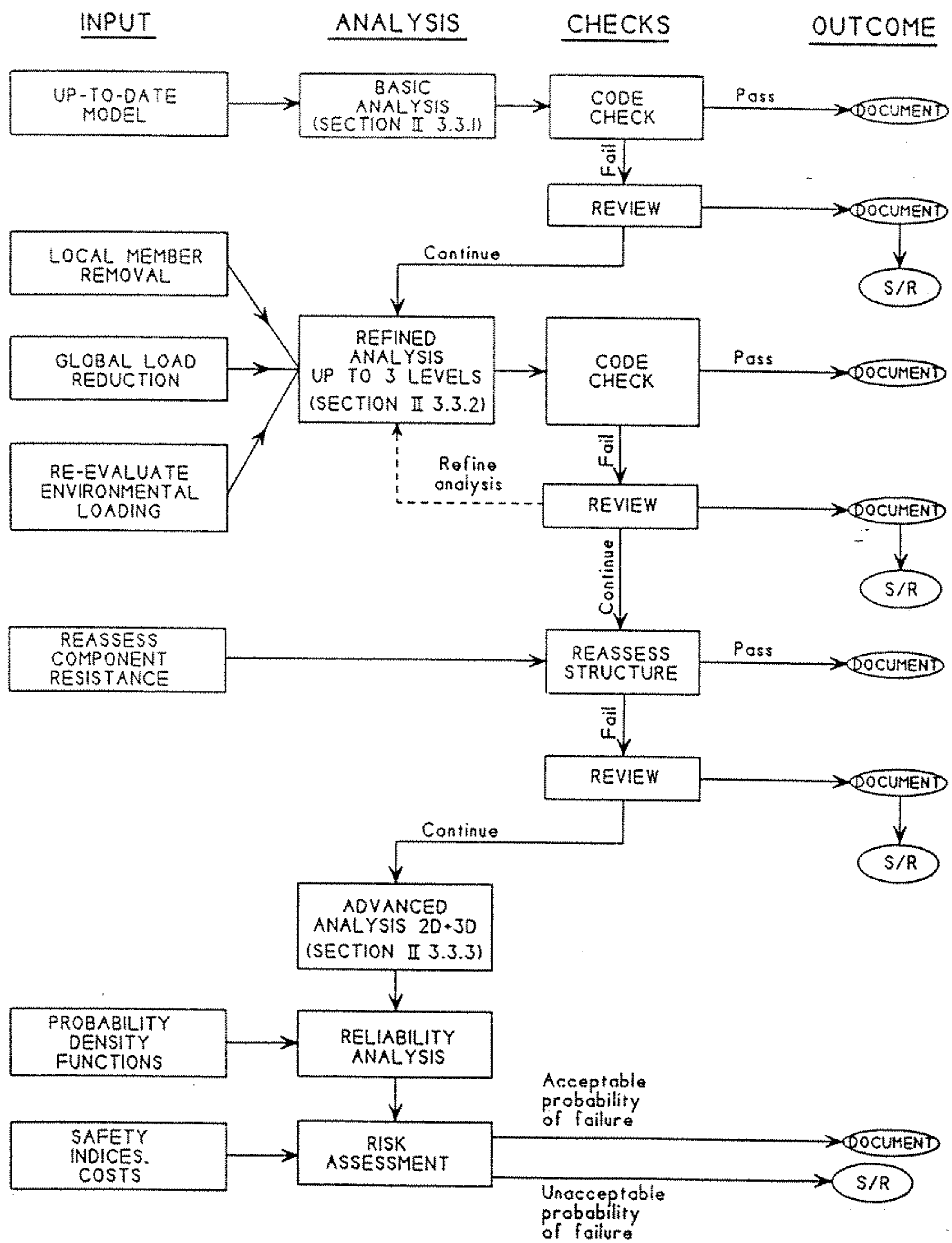

Figure 3.2.1: Assessment methodology for intact structures

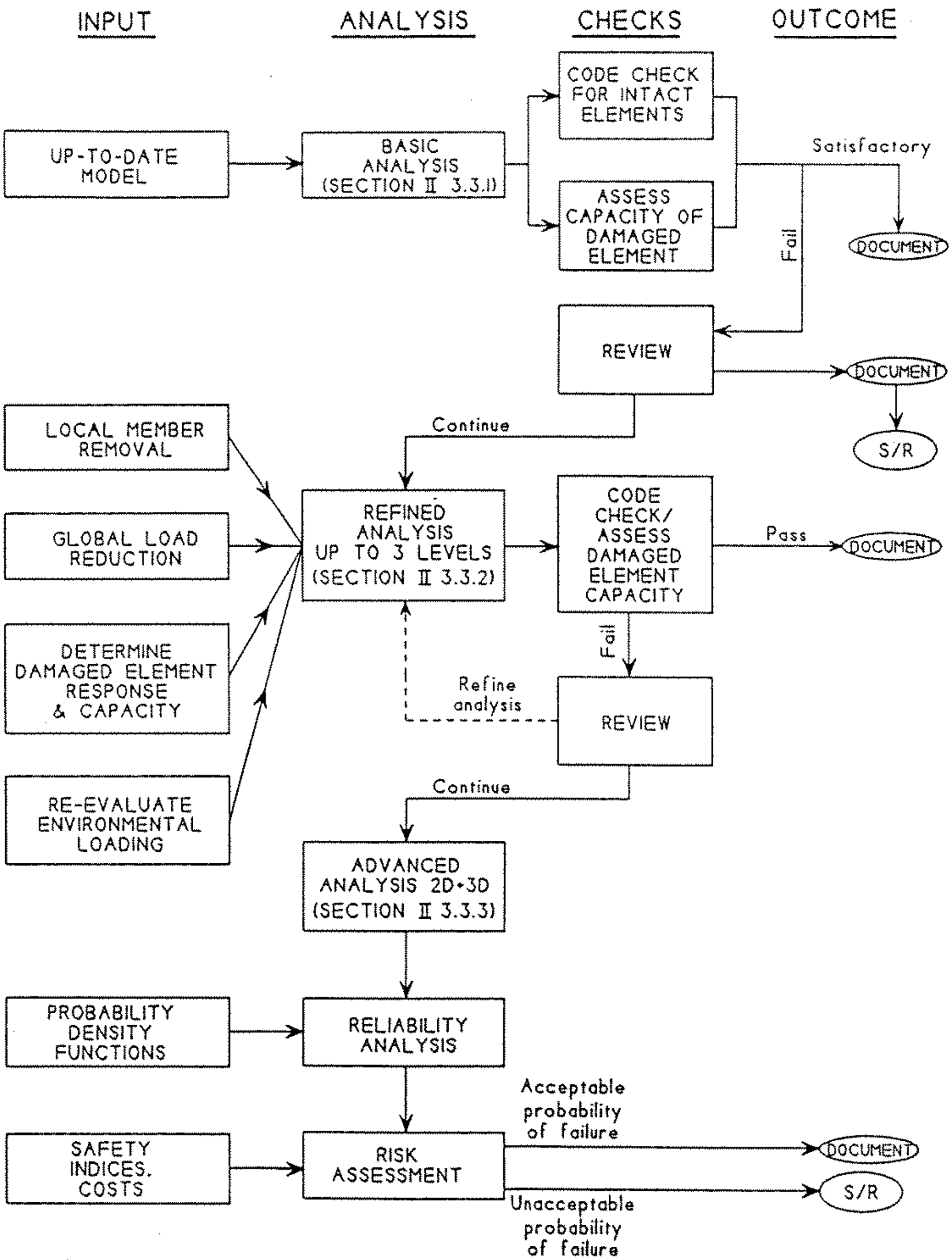

Figure 3.2.2: Assessment methodology for damaged structures

c. Within the overall context of the assessment, it is possible to make risk- and reliability-based decisions on the timing of SMR implementation. It may be beneficial to delay undertaking a single SMR operation until a more appropriate time perhaps when a number of SMR measures can be implemented together. In this manner, the unit cost of each SMR may be reduced. A third course of action may be to undertake a small emergency temporary SMR before a more substantial permanent SMR can be actioned.

d. The engineering assessment must recognise that implementation of SMR measures may alter load paths within the structure. The SMR should, therefore, not be analysed in isolation to the whole structure. SMR will also alter the total loading on the structure (eg. dead loads, increased wave loads), and this should be reflected in the engineering assessment. Temporary works (eg. cofferdams) also give rise to additional loads.

e. In certain instances, it may be possible to consider and reach decisions on remedial measures on completion of the basic analysis. An example in this category relates to members whose primary functions are either not essential or are of secondary importance to the structure once it is installed. Note that secondary members, which in themselves are not critical, may be connected to primary members and, therefore, retention of the structural integrity of the joint is necessary in order to avoid compromising the integrity of the primary member.

f. When direct action, ie. member removal, is not possible, then it may be beneficial to consider an indirect load reduction programme offshore. This entails removing all extraneous items which attract environmental loading such as redundant conductors, caissons, sling platforms, pile guides, boat loadings, launch bracing and marine growth (annually if necessary). The removal of redundant topsides equipment such as tanks may also be mentioned in this context.

g. A refinement of the resistance criteria may be carried out where there is sufficient evidence to suggest that the chosen design code recommendations underestimate the strength of the components in question. This refinement may comprise:-

- an appraisal of all available component data and information
- component linear elastic finite element analysis for stress-related considerations (eg. fatigue strength)
- component non-linear finite element analysis for strength-related considerations (eg. static strength).

There have been a number of cases where an appraisal of available data and information has had a significant impact on the understanding of the structural response of components. References 3.1, 3.2 and 3.3 serve as useful starting points for such studies.

h. In order to evaluate the costs and implications of all the combinations of actions available, it may be useful to perform a risk analysis. One such technique is the AIM approach (Assessment, Inspection, Maintenance), as described by Bea et al[3.4]. A non-linear analysis is essential to a reliability analysis as it is used to determine the damaged strength of the structure, see Lalani et al[3.5]. The application of reliability techniques within the framework of risk assessments is described by Martindale et al[3.6].

II 3.3 TYPES OF GLOBAL ANALYSIS

This section describes an approach to global structural analysis based on progressive engineering cost, effort and complexity. The benefit of this stepwise method is that the analysis and assessment can be limited to the extent necessary to demonstrate structural adequacy. Decision and review milestones are set after each stage of the procedure so that progress can be reviewed and appropriate adjustments made.

Analysis will be influenced by the nature of the damage which is being investigated and the extent of knowledge of the as-built condition of the structure and the damaged condition. However, a three stage procedure is proposed which leads to a rational build-up in engineering knowledge and complexity:-

Stage 1 - Basic analysis

Stage 2 - Refined analysis (of various complexities)

Stage 3 - Advanced analysis.

Figure 3.3.1 illustrates in flowchart form the main steps and decision milestones in the procedure.

II 3.3.1 Basic Analysis

This stage consists of undertaking a storm load linear-elastic structural analysis of the structure in question. The model would incorporate the latest information on topside loadings (including future contingencies if necessary), the actual number of conductors, geometry and modifications since installation (eg. additional risers, boat fenders, removed members). This baseline analysis acts as a simple foundation for more refined analysis later. At this stage the components (members and joints) are checked individually to code requirements (ie. unity checks), eg. API RP2A[3.7]. Later modelling refinements can be made in a stepwise fashion as the need arises, in the manner described in Section II 3.3.2.

Damage would not be explicitly modelled at this stage, nor would any members be removed. Therefore, this analysis model forms the first stage of assessment of problems related to inadequate static strength in the intact structure or to insufficient fatigue life.

II 3.3.2 **Refined Analysis**

II 3.3.2.1 Simple analysis refinements

At this stage, the results of the up-to-date basic analysis will act as a guide to identify the extent of modifications required to the model. A balance should be maintained between the required realism to be achieved in light of the extent of the problem and the design effort required. Consequently, the model complexity is built up through, initially, simple steps but then by more complex steps. Simple steps can be made without reference to other research work or detailed studies. By their very nature, more refined steps such as literature searches, finite element analysis or fracture mechanics studies, require greater effort.

A guide to simple forms of refinements to the basic analysis model which can be incorporated is given below:-

- Model member centre-line end eccentricities at joints, thereby capturing secondary effects.

- Model member offsets at joints so that double counting of wave load in both brace and chord within the joint depth is eliminated.

- Model submerged rather than in-air weights of non-generated appurtenances and secondary steelwork, eg. anodes and pile guides.

- Ensure that other loads have not been double counted, eg. where cast nodes have been used, the additional weight of the casting applied to the model should reflect the extent of chord member which it replaces.

- Where actual member section properties are known, eg. diameter and thickness, they may be input at this stage.

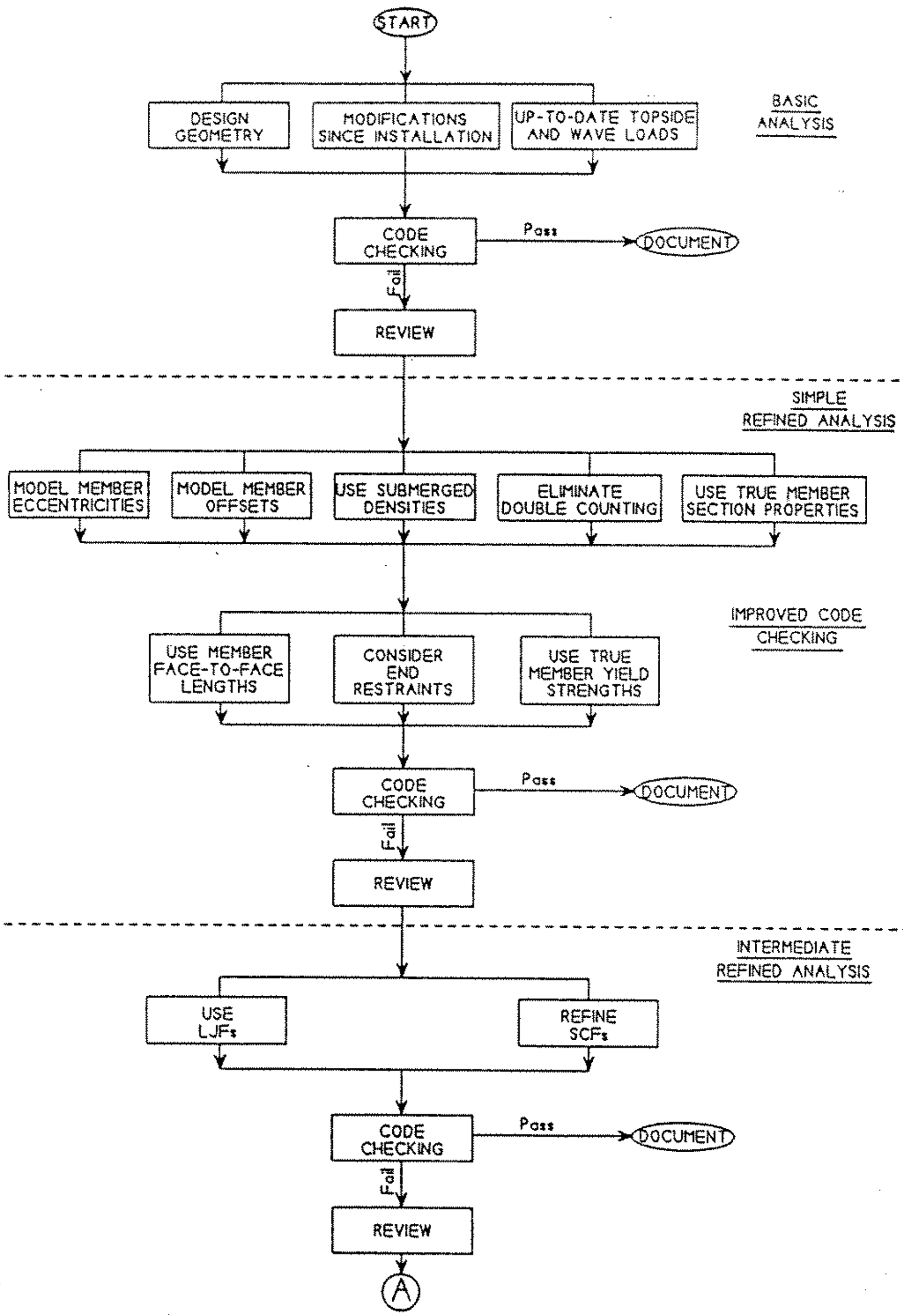

Figure 3.3.1: Global analysis (continued...)

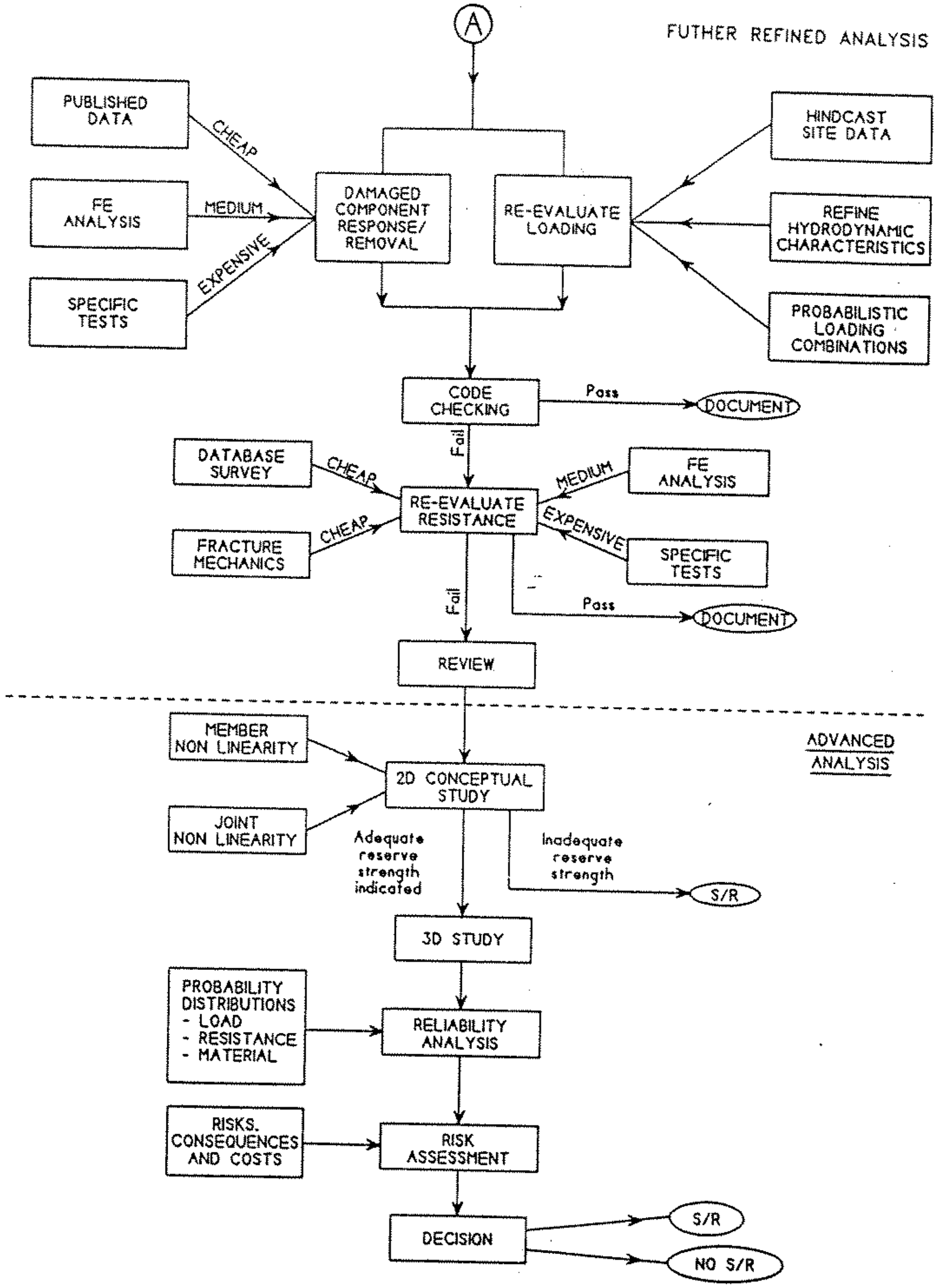

Figure 3.3.1: Global analysis (...continued)

Having determined the loading, simple refinements to code-checking can be made, as follows:-

- Use member face-to-face lengths, where appropriate, when determining member lengths for buckling. Guidance is given in API RP2A on this aspect.

- Give consideration to the degrees of restraint provided by the rest of the structure at individual member ends. This will influence the value of effective length factors, K, in the expression for slenderness ratio KL/r. Codified blanket values of K, by their nature, are upper bounds as they assume that minimum restraint is provided. The commentary to the AISC Manual of Steel Construction Specification[3.8] gives one treatment of this subject.

- Where adequate data exists, input representative member yield strengths as derived from material certificates for steel used to fabricate the structure. Extreme care should be exercised to ensure relevance of the material certificates to the structural components of the platform. It should be realised that yield strength varies even within a single steel plate or element[3.9]. Thus the material certificate may not give a truly accurate indication of the yield strength.

At the end of this stage, the analysis and assessment has not gone beyond codified methods and has not involved input of specific research work. Components (members and joints) have still been checked individually. No yielding or load distribution has yet been considered in the structure.

II 3.3.2.2 Intermediate analysis refinements

Intermediate analysis for the purposes of this discussion is taken to mean analysis which necessitates making reference to technical literature, finite element analysis or research work. It requires greater computational and manpower effort compared to the basic and simple refined analysis as well as a greater knowledge and understanding of the behaviour of offshore structures and components. Three primary modifications can be considered:-

i. Incorporate local joint flexibilities (LJFs).

Up to this point, it is assumed that the tubular joints within a structure provide rigid member end connections. LJFs provide a means of introducing the effects of the flexibility inherent in all joints into the structural analysis. Incorporating LJFs has the effect of relieving bending moments at the ends of brace members whilst increasing midspan bending moments. Brace axial loads may also be reduced. Therefore, both the static and fatigue capacity may be influenced through the introduction of LJFs.

ii. Use refined parametric equations for stress concentration factors (SCFs).

There are a number of stress concentration factor formulations available to industry. These formulations have differing ranges of applicability and accuracy of prediction depending on the joint type. Choice of parametric equations, and hence SCFs, has a significant influence on the calculated fatigue life. Detailed assessment and application of specific SCF data forms part of a more advanced analysis. There is, however, scope for improvement in the accuracy of fatigue analysis by judicious selection of SCF formulations.

iii. Use refined fatigue analysis.

The SCF concept was developed with simple uniplanar joints in mind. In consideration of multibrace and multi-planar joints, the traditional SCF approach suffers from several shortcomings and gross approximation and simplifications used in design lead to underestimates of the joints' true lives. In the refined fatigue analyses, described in Reference 3.10, the joints are incorporated in the global frame model by a substructuring technique. The stress influence matrices of the joint, along with LJFs, are thereby accounted for in the platform analyses. On one platform on which the technique was used, fatigue lives 5 to 10 times longer than those given by conventional methods were calculated[3.10].

II 3.3.2.3 Further analysis refinements

This further level of analysis represents the limits of what can be achieved by linear-elastic structural analysis. Most of the effort here involves extensive reference to research work and background data and, implicitly, extends the bounds of codified methods. Three approaches are described below:-

i. Damaged member properties

Post-damage stiffness properties can be specified in a structural model to represent the damaged members. Damage types for which this procedure is typically adopted include dented or buckled members and members suffering excessive corrosion.

Dented member properties may be determined by reference to published data (eg. Smith et al[3.11] and Moan et al[3.12]), or through finite element analysis or experimentation (see Sections II 3.4.2 and II 3.4.3, respectively). If the dent geometry is known, the deformed shape of the member can be modelled analytically to capture the P-δ effects. A well known ultimate strength and stiffness (including post-peak stiffness) prediction program is DENTA II[3.13] developed at the University of Trondheim, Norway.

ii. Re-evaluation of loading

Offshore structures are often designed using limited environmental data; data might be obtained by interpolation from the nearest monitoring points which may be many miles away. The assumptions made on the basis of measurements taken over a short period of time may lead to conservatism in the assumed extreme event scenarios. Where actual environmental data records for the installation are available, an appraisal of the environmental criteria may be possible to establish the criteria for adoption in the assessment of the installation. Examples of specific areas that might be addressed are:-

- probabilistic evaluations relating to directionality and manner of combination of wave heights with currents and windspeeds
- re-evaluation of drag and inertia coefficients for wave loading
- examination of shielding effects
- detailed hydrodynamic studies of flow around components such as conductor arrays, legs and regular/irregular attachments such as pile guides.

Barltrop et al[3.14] discuss the above aspects further.

iii. Re-evaluation of resistance

For both the ultimate and fatigue limit states, the present day practices contained in design codes and guidance documents have been established on the basis of an interpretation of experimental and numerical data, ie. empirically-derived approaches are recommended. In many instances, the codified methods contain implicit conservative extrapolations which have been necessitated by a lack of data and information, to allow both the ultimate and fatigue limit states to be checked for all components across the range of configurations, geometries and load cases which occur in practice. In assessment engineering, it is often important to obtain as best an estimate of component resistance as possible, and in many instances, this objective can only be realised through evaluations and appraisals of available data and information regarding the subject component. In many cases, specific finite element evaluations or test programmes may be commissioned in order to generate the necessary data and information. The decision to proceed ahead with numerical or experimental studies depends on the outcome of the appraisal of available data, which may indicate the need for these studies subject to the availability of sufficient evidence of the cost and technical benefits of this approach.

II 3.3.3 Advanced Analysis

Advanced analysis for the purpose of this document consists of non-linear analysis and reliability analysis.

A non-linear analysis of the complete structure is required to assess its ultimate load-bearing capacity, using reserve strength principles. In this manner, the redundancy aspects can be assessed more accurately than by the linear elastic procedures adopted in the basic and refined analysis. Further, this non-linear analysis (commonly referred to as pushover analysis) allows the ultimate limit state of the complete structural system to be appraised, as opposed to the component-based assessments described above.

The reserve strength of a structure is dependent on the conservatism in the design of individual members and joints, but ultimately on the performance of these members and joints within the frame. For tubular beam-column members, design codes such as API RP2A/AISC incorporate a nominal safety factor of 1.25 against first yield for tension and 1.25 - 1.44 for compression for storm load conditions. However, a considerable reserve beyond first yield is available and, for typical members designed to code requirements, the real safety factors for the operating load case are closer to 2.0 for the ultimate limit state. Similarly, an examination of the failure modes for tubular joints indicates a large reserve capacity beyond the point of first yield. The non-linear stiffness characteristics for joints have a significant bearing on frame behaviour for connection-dominated collapse mechanisms, and the reserve beyond first yield is dependent on the joints' configuration, geometry, load type and ductility. Ductile yielding at a member cross-section or at a joint in a statically indeterminate structure (such as typical offshore jackets) will redistribute internal forces so that larger loads can be applied. The manner of redistribution is dictated by the availability of alternative load paths and the interaction between member and joint non-linear behaviour. Loads can be increased until hinges occur at sufficient locations to create a collapse mechanism. The load at which a collapse mechanism develops is governed by a number of sources of reserve strength, and may include the following:-

- overdesign by exceeding minimum requirements
- material reserve strength
- code safety factors
- corrosion thickness allowance
- design for pre-service conditions
- structural redundancy
- foundation system redundancy.

A significant amount of work on the development and application of reserve strength technology has been carried out over the past decade[3.15]. Reserve strength calculations are highly sensitive to the strategy established for conducting pushover analysis. As significantly different reserve and residual strength factors may be calculated on the basis of different strategies, it is important to ensure that the strategy adopted enables the system behaviour to be captured in a realistic manner. The parameters which require consideration include the following:-

- Member modelling, to monitor and account for plasticity along its length and through its cross-section.

- Member modelling, to account for overall buckling, local buckling and post-buckling behaviour.

- Joint modelling, to account for coupled and decoupled translational and rotational non-linear stiffnesses, both in two and three dimensions.

- Foundation system modelling, to account for non-linear pile-soil-structure interaction.

- Method of accounting for imperfections.

- Method of accounting for geometric and material non-linearities.

- Method of loading (point loads, distributed loads, proportional and non-proportional loading, time-history loading).

- Solution procedure (Newton-Raphson, modified Newton-Raphson, KT01).

- System failure definition (limiting acceptable system deformations, acceptable extent of failed members/joints, member/joint failure v/s leg failure, foundation system failure).

Before embarking on a full three-dimensional non-linear analysis, it is wise to carry out a two-dimensional non-linear analysis to enable approximations of the likely levels of reserve and residual strength to be defined. A discussion regarding this form of approach is contained in Reference 3.5. The findings from this two-dimensional analysis can thereafter be used to judge the merits of carrying out a full three-dimensional pushover analysis. In any case, the two- and three-dimensional analyses allow investigation of full frame behaviour effects and, in the case of damaged components, they allow the consequences of the damage to be rationally appraised in a system rather than on a component basis.

The results from a non-linear analysis can also be used as input to a structural reliability assessment. Probability density functions can be formulated for component, system and foundation resistance (damaged or undamaged), as well as for loading (environmental, topsides, earthquake, etc) and, using reliability assessment methodologies, relative reliability and risk envelopes can be established.

At the present time, there are insufficient data to quantify absolute levels of risk. Reliability analysis provides a powerful tool for assessing the relative risk involved when considering more than one course of action. However, it should be recognised that application of this technique requires a non-linear ultimate strength analysis of the whole structure. Without the data obtained from such an analysis, it is not possible to effectively evaluate relative risks of system failure.

II 3.4 ASSESSMENT OF LOCAL EFFECTS

Local element assessment is an integral part of the modelling process in that it provides data on the local response and capacity of the isolated, damaged or understrength, element for input to the global structure assessment. The results of an assessment of this form can be used in one of two ways:-

i. Taking the results from a global analysis, use the generated loads to determine the integrity of the components of interest. Local investigations can take the form of simple codified assessments, application of existing test data, fracture mechanics, finite element analysis (linear-elastic for SCF determination but non-linear for ultimate load assessments) and generation of test data from a specific test programme.

ii. Use local assessments to establish stiffness characteristics of damaged elements for input to the global analysis. On completion of the global analysis the generated loadings can be used for utilisation checks on the basis of the capacities estimated from the local assessment. Investigations of this nature would typically involve finite element analysis or a specific test programme.

The remainder of this section gives guidance on the application of the individual techniques to the assessment procedure.

II 3.4.1 <u>Fracture Mechanics</u>

Fracture mechanics is a method of characterising the fracture behaviour of structural components in terms of parameters which can be used by an engineer. Generally, fracture analyses are performed either to:-

- determine acceptance levels for defects in welded joints

- predict the time to final rupture of a cracked structure or a structure containing defects.

The former relates primarily to new construction, and the latter concerns existing structures. These types of analyses can be sub-divided into two general categories, namely linear-elastic and elastic-plastic methods.

Linear-elastic fracture mechanics (LEFM) is based upon an analytical procedure that relates the stress field magnitude and distribution in the vicinity of the crack tip to:-

- the nominal stress applied to the component
- the size, shape and orientation of the crack and material properties.

LEFM is particularly useful for determining crack growth characteristics and, providing plane-strain conditions prevail, the onset of fracture or unstable crack growth.

For many applications concerned with relatively thick steels, LEFM is often invalidated due to the formation of large plastic zones at the crack tip. In these cases, elastic-plastic fracture mechanics is used. It is worth noting that, at high strain rates, LEFM still gives a reasonably accurate analysis.

Elastic-plastic fracture mechanics uses one or more of the following three techniques to determine resistance to rupture:-

- crack tip opening displacement (CTOD)[3.16]
- R-curve method[3.17]
- J-integral method[3.18].

The fracture mechanics techniques described above require, as input, the results from the global analysis (either fatigue or static loads), to determine the crack growth rate and, subsequently, resistance to fracture. Fracture mechanics can also be used to assess the value of remedial grinding.

II 3.4.2 Finite Element Analysis

Linear-elastic finite element (FE) analysis is used to determine SCFs or stiffness characteristics of elements; non-linear FE analysis is used to determine peak load capacities of joints or members and post-peak stiffness characteristics. In many cases, the two analyses would be combined because, at low load levels, the response of a carbon-steel component is linear-elastic.

Finite element analysis results are sensitive to the selection of element type, mesh density and boundary conditions. A calibration exercise should initially be undertaken to establish an FE strategy. This is best achieved through a numerical reproduction of a test result for a similar component. It is important that the modelled boundary end restraints are sufficiently removed from the component or area of interest in order to ensure validity of the analysis result.

Finite element analysis techniques are well documented in the literature. One important aspect of FE analysis relates to the treatment of the results. Consideration should be given to how the results of a finite element analysis are used in relation to component strengths in design. Tubular joint parametric equations for static strength, for example, are based on assessments of test results. Implicit within the scatter of experimental test results are variations on capacity arising from such sources as:-

- diametrical and ovality tolerances
- thickness tolerances
- residual stresses
- weld variability
- material properties.

Finite element analysis is carried out on 'perfect' specimens and usually with zero residual stresses. Hence, the results may be appreciably different for 'real' specimens. Either a sensitivity study should be carried out to quantify some or all of the above factors or consideration should be given to applying a partial safety factor to the results in addition to the commonly accepted safety factors.

II 3.4.3 Tests

A large database on screened test results for offshore structural components is now available (see Part IV of this document). It is therefore prudent to interrogate and review the database before embarking on a test programme to avoid generation of data which are expensive and unnecessary. It is important that any database is carefully screened to ensure that it is entirely applicable to, and representative of, offshore structures. Some simple guidelines to follow in this respect are noted below.

- Are the tests on specimens of sufficiently large size so that manufacture and welding is representative of offshore methods?
- Was fabrication carried using normal offshore fabrication specifications?
- Are true yield stresses and member geometries measured?
- Did the instrumentation accurately capture the response?
- Were the loads applied in the correct manner and at the correct rate?

- Is there a large scatter of results, suggesting inconsistencies in test procedures and methodology?

- Did the specimen fail as expected?

- Does the reported failure load represent the ultimate load, or a serviceability limit state or some other value dependent on test rig or equipment limitations?

II 3.5 CLOSURE

The primary objective of assessment engineering is to obtain as best an estimate of the structural strength of the platform components and system as possible, irrespective of the nature of damage, if any. It is important to deploy the full extent of present day offshore structural engineering technology, to allow rational decisions to be reached regarding the need to strengthen or repair. This Chapter has catalogued the present range of technological approaches deployed within the offshore industry in this respect. Once a decision has been made in favour of strengthening and/or repair, the next stage in the process relates to an appraisal of all available SMR techniques and the selection of the most appropriate scheme from technical, cost and safety standpoints. This is covered in the remainder of this manual.

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II 4 LEGISLATION AND REQUIREMENTS OF REGULATORY BODIES/CERTIFYING AUTHORITIES

II 4.1 INTRODUCTION

This chapter addresses the general requirements of relevant authorities with respect to strengthening, modification and repair (SMR). Any specific requirement relating to SMR techniques is addressed in Volume III as appropriate. The emphasis is towards UK, Norwegian and American Legislation.

II 4.2 OFFSHORE INSTALLATIONS COVERED BY UK LEGISLATION

The regulatory responsibilities for offshore installations in UK waters previously held by the Department of Energy were taken by the Health and Safety Commission and Executive, specifically the Health and Safety Executive (HSE) Offshore Safety Division, on 1st April 1991.

At that time, the Statutory Instrument "Offshore Installations (Construction and Survey) Regulations 1974 (SI 1974/289)" was adopted by HSE and these Regulations require that each offshore installation covered by the Regulations has a valid Certificate of Fitness which is issued and maintained by a recognised Certifying Authority. Any repairs required on offshore installations are to be notified to the appropriate Certifying Authority and carried out under its supervision and approval in order that the Certificate of Fitness be maintained. A valid Certificate of Fitness is recognised by the Regulations as evidence that the installation complies with all the requirements.

At a more detailed level, and in support of the above Regulations, HSE's guidance on repairs, strengthening and modification of offshore structures is contained in Section 60 of Offshore Installations: Guidance on design, construction and certification, Fourth Edition 1990. This section deals with structural repairs and modifications including procedures, general considerations, repairs by welding and repairs other than by welding.

Whilst the above legislative structure and supporting guidance is still applicable at the time of issue of this document (late 1995), HSE are shortly to introduce new legislation in the form of goal-setting objectives, see "Draft Offshore Installations and Wells (Design and Construction, etc.) Regulations 199-", Consultative Document CD89 July 1995 issued by HSE. These Design and Construction Regulations (DCR) are intended to replace the existing SI 1974/289 and will sit more easily with the new safety case regime. (The "Offshore Installations (Safety Case) Regulations - SI 1992/2885" require an offshore installation to have a safety case document; any subsequent repair or modification of the installation can only be made once HSE has approved an appropriate revision to the Safety Case for conducting the work.)

The most relevant clauses in the draft DCR as far as strengthening, modifications and repairs are concerned is given in Part II (Integrity of Installations) regulations 6 and 8 which state:

> *"6. The duty holder shall ensure that work of fabrication, construction, commissioning, modification, maintenance and repair of an installation, and activity in preparation for the positioning of an installation, is carried out in such a way that, so far as is reasonably practicable, its integrity will be ensured.*
>
> *8. (1) The duty holder shall ensure that suitable arrangements are in place for maintaining the integrity of the installation, including suitable arrangements for:*
> *(a) periodic assessment of its integrity; and*
> *(b) the carrying out of remedial work in the event of damage or deterioration which may prejudice its integrity."*

The duty holder in the case of fixed installations is defined as the operator and that for a mobile installation as the owner.

II 4.3 OFFSHORE INSTALLATIONS COVERED BY NORWEGIAN LEGISLATION

The Norwegian Petroleum Directorate (NPD) is the governing body responsible for offshore installations in the Norwegian sector of the North Sea.

No specific rules are available for repair work but the Regulations state that consent for any repairs must be obtained from NPD. However, it is implied that repairs should be based on the requirements for new construction.

The Regulations call for materials and their use to be in accordance with Norwegian Standards but also accept that other recognised national standards will be acceptable when equivalent.

The main Norwegian Petroleum Directorate regulations are:

- Regulations for structural design of load bearing structures intended for exploitation of petroleum resources (Publication YA 004).

This document contains the following relevant guidelines:

- Guidelines on the selection of steels and fabrication of steel structures for the petroleum activities (Publication YA 045).
- Guidelines on the design and analysis of steel structures in the petroleum activities (Publication YA 050).
- Guidelines for the determination of loads and load effects (Publication YA 037).

II 4.4 OFFSHORE INSTALLATIONS COVERED BY UNITED STATES LEGISLATION

The main regulatory body in the USA is the Minerals Management Service (MMS) of the US Government (Department of Interior).

No specific rules apply for the repair of offshore structures but individual repairs must be approved by the MMS and must be in accordance with the following codes and standards.

- American Institute of Steel Construction (AISC)

  Specification for the design, fabrication and erection of structural steel for buildings.

- American Petroleum Institute API RP2A

  Recommended practice for planning designing and constructing fixed offshore platforms.

- The Mineral Management Service OCS Regulations.

  Title 30 CFR Part 250.

II 5 SELECTION OF SMR TECHNIQUES

II 5.1 INTRODUCTION

Following an engineering assessment, and having arrived at the conclusion that some form of strengthening, modification or repair (SMR) is required, it is necessary to choose how best this may be done. In some cases the solution may be fairly obvious, but it is prudent to consider alternatives to ensure that the chosen solution is the best from technical, operational and economic viewpoints. The purpose of this Section II 5 is therefore two-fold:-

- to categorise approaches to SMR
- to identify candidate SMR schemes and techniques for given scenarios.

The broad range of possible SMR techniques is indicated in Figure 5.1.1 in which each technique has been put in one of several main categories. The individual techniques are applied in what may be termed SMR schemes. For example, a new bracing member may be installed using two stressed grouted clamps. In some successful and cost-effective SMR schemes, use will be made of more than one SMR technique. It is therefore necessary for the engineer not only to know the range of techniques available but also to appreciate their advantages, disadvantages and limitations. Every SMR scenario presents its own unique set of problems and therefore the engineer should consider all the factors relevant to the problem before making a final selection. It should be appreciated that the guidance cited below, with respect to appropriate SMR techniques for given common scenarios, can only be indicative; site specific criteria may make some of the suggested techniques unsuitable.

It is emphasised now that it is absolutely vital to know the root cause of the problem to ensure a successful repair. For example, repair welding of a crack is unlikely to be effective in the case of a fatigue problem, but may be entirely suitable if the crack was caused by installation damage. The use of the selection tables below will rule out some, even most, of the SMR techniques for a given scenario. A more detailed appraisal of the candidate techniques should then be undertaken by referring to Sections II 6 to II 13 as appropriate. In some cases, particularly with respect to clamp technology, it may be necessary to conduct initial sizing to compare alternative schemes. In these cases reference to Part III is required before a final selection can be made.

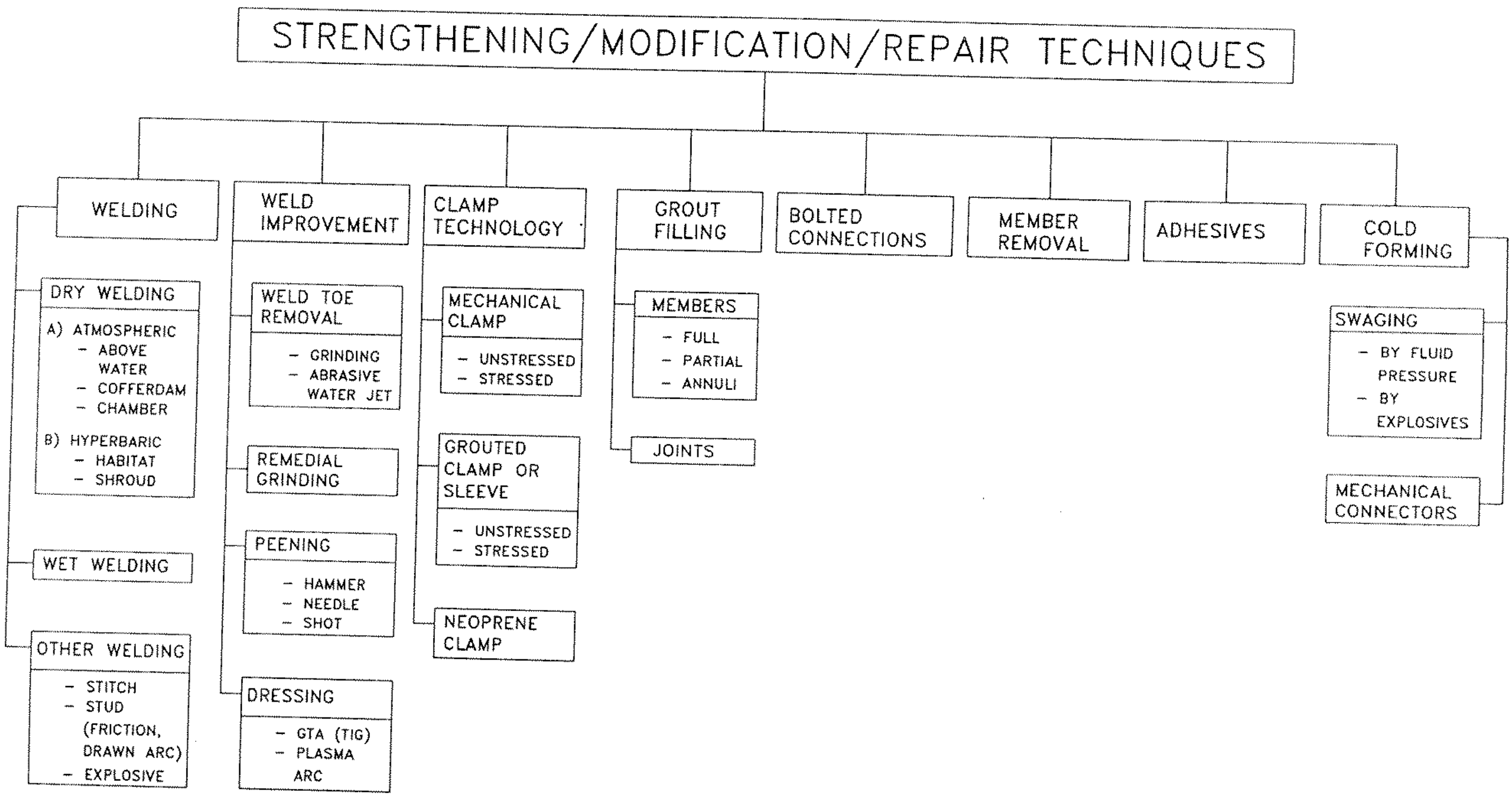

Figure 5.1.1: Overview of techniques used in strengthening, modification and repair

II 5.2 APPROACHES TO SMR

The seriousness, and hence an indication of the extent of SMR required, will be determined during the assessment phase of the study work. The first action in the consideration of SMR schemes is to identify whether a 'local' SMR will suffice, or whether a more elaborate 'global' solution is required. Clearly, a local SMR is likely to offer the cheaper solution, provided it meets the technical requirements.

There are essentially four basic approaches to SMR:-

- remove damage
- reduce loadings
- localised strengthening/repair
- global strengthening/repair by provision of new members.

These approaches can, and often are, used in combination.

II 5.2.1 Remove Damage

This approach, of course, is only applicable where the problem relates to damage, howsoever caused. Damage removal can be achieved in one of two ways:-

i. Removal (and possible replacement) of member

The damage is clearly removed if the affected member is cut out. However, unless it is a member no longer required for the in-place condition, perhaps as proven during the assessment phase or because its function has expired, it will need replacing. In that case only that part of the member containing the damage has to be cut out and replaced. The primary issue then becomes the method of attachment of the replacement member. Viable SMR techniques are welding, clamps and bolted connections. Temporary bracing may be required until the replacement member or part member is installed. Restraining relative movements may have to be addressed for welding and some types of clamping techniques.

ii. Crack removal

The removal of cracks can be achieved by properly executed remedial grinding. The resulting groove can be left as is or filled in with weld metal. In the case of cracks caused solely by fatigue loads (ie. not in combination with a fabrication defect), other SMR techniques will be required in addition to the grinding, unless the remaining planned life of the installation is sufficiently short.

II 5.2.2 Reduce Loadings

This approach is closely related to the first in that it involves removing parts of the structure. However, it is applicable to both damaged and undamaged structures. Two examples can be given:-

- An appropriate SMR can sometimes be effected with respect to a fatigue crack in a secondary member which connects with a primary member. Removal of the secondary member, leaving a stub with the crack still in place, will eliminate the fatigue loading on the crack and thus arrest its potential growth into the primary member.

- Anticipated problems in undamaged conductor guide frames may be circumvented by removing conductor plating and thereby reducing the 'sail' area. The reduced panting loads may extend sufficiently the fatigue lives of the joints to negate the need for future, more extensive, repairs.

Removing marine growth will also reduce loadings.

II 5.2.3 Localised Strengthening/Repair

In this approach, the member or joint is strengthened directly (leaving any damage in place) without altering load paths within the structure. Additional load may be attracted to the member or joint, however, either by virtue of its increased stiffness following the SMR or due to increased drag forces acting on, say, clamps. Three broad categories of techniques may be recognised as being applicable to localised strengthening/repair (S/R):-

i. Internal S/R

Grout filling is the main suitable technique although a bolted connection has been used inside a member on one occasion. Internal grout filling can be used:

- to act compositely with the steel to increase member stiffness and overall buckling capacity, or

- to act as a packer to restrain local shell distortions (ie. to decrease SCFs, to increase collapse strength of joints, to increase local buckling capacity (possibly at a dent) and to prevent radial collapse of a member against external loads arising from stressed clamps).

ii. External S/R

External S/R can be achieved by clamping technology or by employing welding to install doubler plates or sleeves. These techniques can be used for either members or joints.

S/R schemes which deploy external clamp or sleeve concepts may be designed to carry part of the load present in a component. In other words, the existing component is assumed to share the load with the S/R scheme. This may be applicable, for example, at a tubular joint which is shown to be understrength for a particular predicted design event, but there has been no damage to date. (If damage has occurred, it is usual to assume conservatively that the existing component has no residual strength and design the S/R scheme to carry the full applied load.) In load sharing schemes, it is necessary to determine the amount of load carried by each component. For member S/R, it is usually sufficient to apportion the load according to the relative stiffnesses of the components. For tubular joint S/R, the situation is more complicated due to the local flexibility of the joint and the clamp or sleeve. A finite element analysis may be carried out in this case to determine the state of stress, particularly in the joint. Where load-sharing schemes are employed, careful consideration should be given as to the confidence which may be placed on the estimate of the load apportioning and the consequences of not actually realising the estimate in practice.

iii. Fatigue life improvement

The fatigue life of welded components can be improved by modification techniques such as toe grinding. However, this technique cannot be applied to existing cracked joints unless the damage is first removed.

II 5.2.4 Global strengthening/repair

Global or total S/R implies that new load paths are created. Load is diverted away from the damaged or understrength component by:-

- providing a S/R scheme which is sufficiently stiff to attract a suitable proportion of the load which would otherwise have been applied to the defective part of the structure

- jacking load into the S/R components during the installation of the scheme so that they carry both the full or partial dead load and a proportion, depending on relative stiffness, of the live load.

There are, potentially, numerous schemes that could be proposed, eg. in terms of overall structural solution or where to locate new members. Again, each SMR scenario is different and it is not possible to be dogmatic about how to select the optimal scheme. Initially and most importantly, however, experience and engineering judgement will be brought to bear to reduce the number of candidate schemes to a few. The candidate schemes can then be appraised for:-

- technical sufficiency (this may require platform analyses incorporating the schemes)

- costs
- installation effort (including a consideration of water depth, cranage capacity, availability and experience of contractors, etc)
- operator preferences.

Careful consideration must be given to tolerances (length and angles) for problem-free installation. The use of grouted connections and clamps, with their forgiving annuli, and sliding or telescopic joints which can be fixed on installation by welding, grouting or stressing, are useful devices in this respect.

II 5.3 COMPARISONS OF TECHNIQUES

This section can be used as an aid to technique selection. Table 5.3.1 draws together salient information on each technique from the subsequent chapters in this Part II. Reference should be made to these chapters for further detailed information and possible limitations for application to the scenario under study.

Table 5.3.2 indicates the potential applicability of SMR techniques for selected defect scenarios. Again, reference should be made to subsequent chapters and a decision reached based on pertinent criteria such as:-

- technical performance
- reliability
- costs
- depth limitations
- offshore support requirements
- existing applications
- extent of background knowledge
- timescales for design/fabrication/installation
- tolerance acceptability
- post-installation inspection requirements
- potential problem areas
- remaining life of installation
- environmental and other legislative requirements
- operator preferences.

Case histories, presented in Appendix A, illustrate how some SMR scenarios have been tackled in the past. They illustrate the application of several techniques.

Technique	Used offshore	Data available for		Equipment needs	Offshore installation timescales	Onshore fabrication costs	Load penalties		Relative post installation inspection requirements	Design guidance available
		Static strength	Fatigue strength				Weight	Wave load		
Dry welding	yes	yes	yes	heavy	very slow	high for habitat	none	none	low	yes
Wet welding	yes	yes	yes	moderate	quick	none	none	none	low	yes
Toe grinding	yes	N/A	yes	low	moderate	none	none	none	moderate	yes
Remedial grinding	yes	yes	yes	low	moderate	none	none	none	moderate	yes
Hammer peening	yes	N/A	yes (for plates)	low	quick	none	none	none	moderate	no
Stressed mechanical clamps	yes	yes	yes	moderate	moderate	high	moderate	high	high	yes
Unstressed grouted connections	yes	yes	yes	moderate	moderate	low	low	low	low	yes
Unstressed grouted clamps without shear keys	yes	yes	yes	moderate	moderate	moderate	moderate	moderate	moderate	yes
Unstressed grouted clamps with shear keys	yes	yes	yes	heavy	slow	moderate	moderate	moderate	moderate	yes
Stressed grouted clamps	yes	yes	yes	moderate	slow	high	moderate	high	high	yes
Elastomer-lined clamps	yes	yes	yes	moderate	moderate	high	moderate	high	high	yes
Pressurised connections	no	yes	no	light	slow	moderate	low	low	low	no
Grout filling	yes	yes	no	light	quick	low	high	none	low	no
Bolting	yes	yes	yes	light	moderate	low	low	low	moderate	yes
Member removal	yes	N/A	N/A	moderate	quick	none	none	none	none	N/A
Adhesives	yes	yes	yes	light	quick	low	low	low	low	no
Swaging	yes	yes	yes	moderate	quick	moderate	low	none	low	yes

Note: N/A = not applicable

Table 5.3.1: Comparison of SMR Techniques

Technique	Defect								
	Fatigue crack	Non-fatigue crack	Dent	Corrosion	Inadequate static strength		Inadequate fatigue strength		Understrength topsides plating
					member	joint	high loads	fabr. fault	
Dry welding	yes(1)	yes	yes(3)	yes(3)	yes(1)	yes(1)	no	yes	yes
Wet welding	no(2)	yes	yes(3)	yes(3)	yes(1)	yes(1)	no	yes	no
Toe grinding	no	no	no	no	no	no	yes	no	yes
Remedial grinding	yes	yes(1)	no	no	no	no	no	no	no
Hammer peening	no	no	no	no	no	no	yes	no	yes
Stressed mechanical clamp	yes	yes	no	yes	yes	no	yes	yes	no
Unstressed grouted connection	yes	yes	yes	yes	yes	yes	yes	yes	no
Unstressed grouted clamp	yes	yes	yes	yes	yes	yes	yes	yes	no
Stressed grouted clamp	yes	yes	yes	yes	yes	yes	yes	yes	no
Stressed elastomer-lined clamp	no	yes	no	yes	yes(4)	no	no	no	no
Pressurised connections	yes	yes	yes	yes	yes	yes	yes	yes	no
Grout-filling	no	no	yes	no	yes	yes(4)	yes(4)	no	no
Bolting	no	yes	no	no	no	no	no	no	yes
Member removal	yes(5)	yes(5)	yes(5)	yes(5)	no	no	yes(5)	yes(5)	no
Adhesives	yes(6)	yes(6)	yes(3 or 6)	yes(3 or 6)	yes(3 or 6)	yes(6)	yes(3 or 6)	yes(3 or 6)	yes(3)
Swaging	yes(7)	yes(7)	no	yes(7)	no	no	no	no	no

Notes:
(1) Usually in conjunction with additional strengthening measures
(2) Except to apply weld beads in unstressed grouted connection/clamp repairs
(3) To apply patch plates
(4) Applicability depends on type and sense of loading
(5) If member is redundant
(6) Used as epoxy grout in clamps
(7) If damage can be by-passed

Table 5.3.2: SMR Techniques directly applying to various scenarios

II 6 BACKGROUND TO AND DESCRIPTION OF WELDING TECHNIQUES

II 6.1 GENERAL

Welding is often regarded as the best strengthening, modification and repair (SMR) technique and no doubt would be used even more often if it were not for certain operational difficulties in its execution.

There are several SMR welding techniques and a number of welding processes that can be considered:

- Dry welding at one atmosphere using cofferdam or pressure-resisting chambers. All normal welding processes can be used but GTAW, SMAW and, to a lesser extent, FCAW are the main methods used in practice.
- Hyperbaric welding using habitats. Main processes used are GTAW and SMAW although FCAW and GMAW are sometimes employed.
- Wet welding. Practically, only the SMAW process is used.
- Friction welding.

The welding process acronyms are explained in Section II 6.4.

II 6.2 DRY WELDING

II 6.2.1 <u>Description</u>

II 6.2.1.1 One-atmosphere welding

Because a large body of welding technology exists relating to normal atmospheric pressure, a logical approach to underwater welding repair is to duplicate surface welding conditions by providing a one-atmosphere environment at the repair site. Two methods are available which can achieve this:

i. Cofferdam

This essentially is a water-tight structure which surrounds the repair location and is open to the atmosphere. The structure can be open topped, as illustrated in Figure 6.2.1, or have a closed top with an access shaft to the surface.

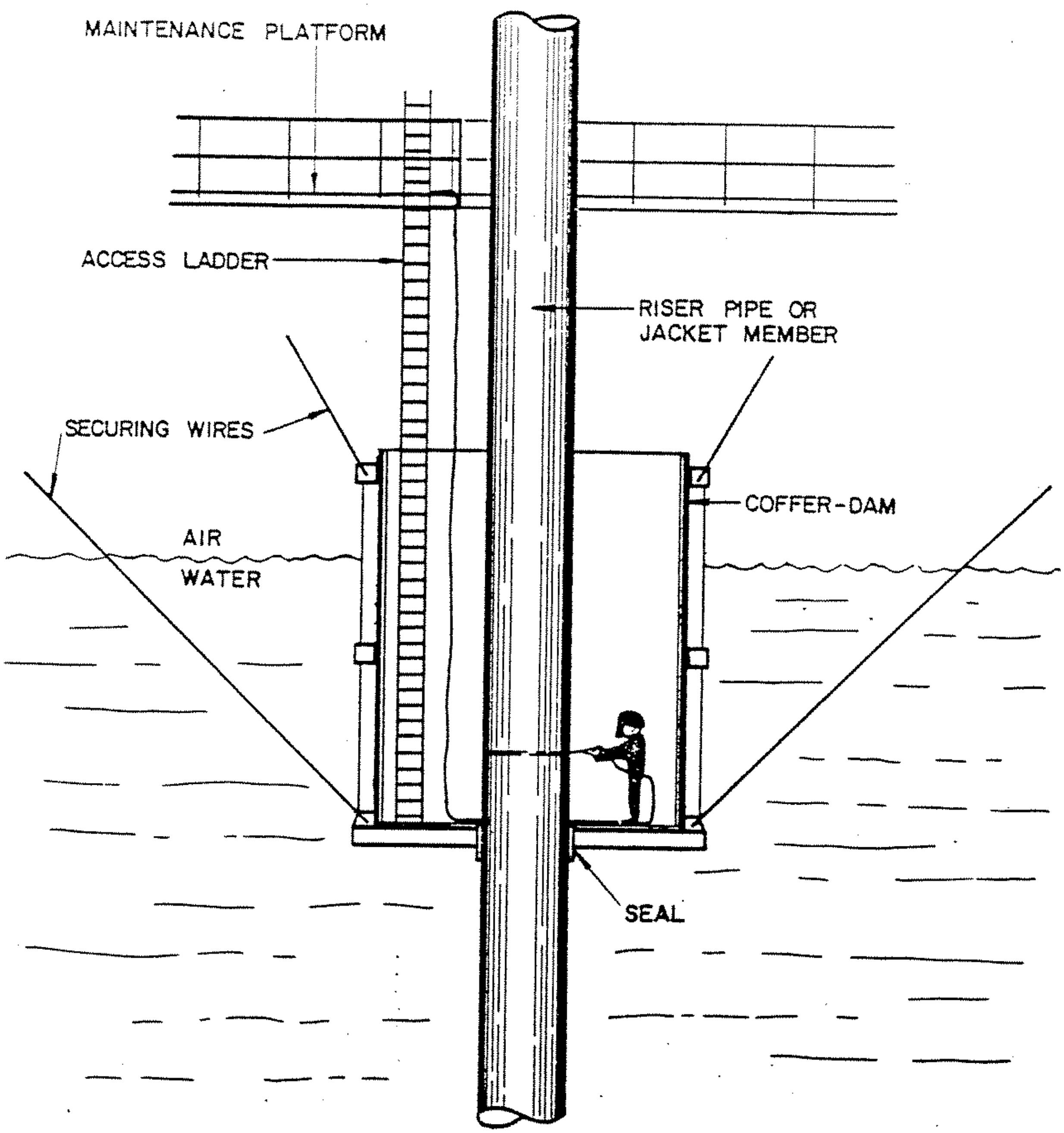

Figure 6.2.1: **Cofferdam repair**

ii. Pressure-resistant chamber

The worksite is surrounded by a chamber constructed as a pressure vessel, capable of withstanding the water pressure at the depth of the repair location. Once the chamber is in place and sealed to the structure, it is dewatered and the internal pressure can then be reduced to one-atmosphere. The repair crew can transfer to the welding chamber in a one-atmosphere environment, within a diving bell, to carry out the repair.

Because conditions within the cofferdam or welding chamber duplicate those on the surface, any normal welding process could be used. In practice, GTAW and SMAW predominate, with minor usage of FCAW.

II 6.2.1.2 Hyperbaric welding

Hyperbaric welding is the most widely used repair technique for primary structures and pipelines. The repair site is again enclosed within a working habitat which is dewatered by filling the habitat with gas. Since the gas and water will be at equal pressure at a point close to the bottom of the chamber, the maximum differential pressure will be at the top of the chamber, and obviously depends on the height of the chamber. This differential pressure (normally a few tenths of a bar (10's of kPa)) is easily resisted by lightweight habitats and simple flexible seals, making deployment and sealing of the workchamber operationally feasible.

A variety of habitats have been used, dependent on such factors as the extent of welding required, the complexity of the repair site geometry, depth of repair, welding process and ancillary equipment, and environmental conditions. Generally, designs of dry hyperbaric habitat fall into one of the following four groups:

i. Lightweight steel habitats

These are of stiffened plate construction and are fabricated in two or more sections to allow their placement around jacket members. They may have an open grate floor with an access hole, see Figure 6.2.2, or be fitted with a closed floor and access shaft. The latter is used in shallow depths where the shaft acts as a surge tube, thereby reducing the volume and pressure changes in the habitat which otherwise could affect diver physiology.

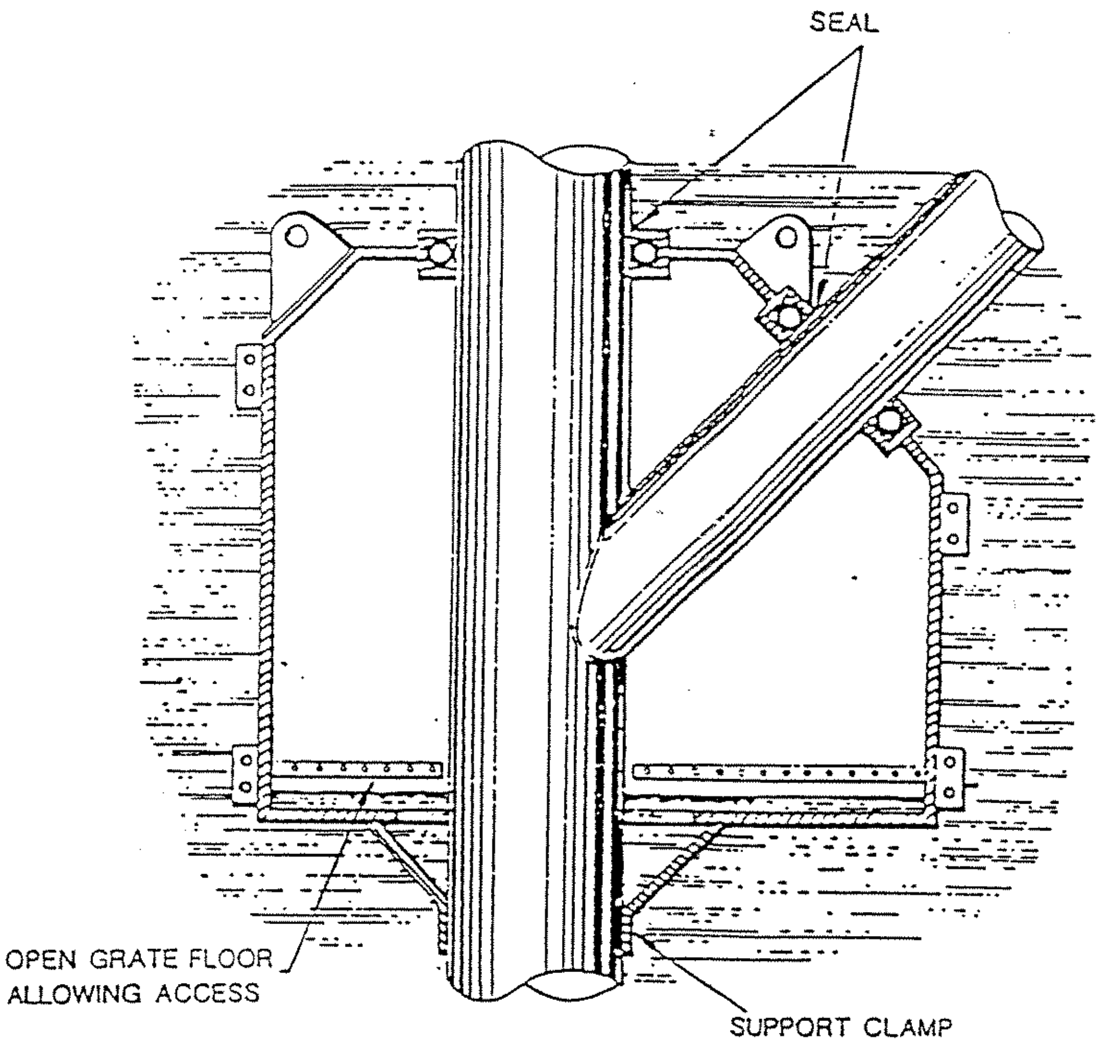

Figure 6.2.2: Hyperbaric welding habitat

ii. Inflatable flexible habitats

Because differential pressures are low, flexible habitats of sufficient strength are practicable and have been used, see Figure 6.2.3. The skin of the habitat takes up a shape dictated by the skin membrane stresses and the depth-dependent differential pressure, and is the same shape that would be obtained onshore by turning the habitat upside down and filling it with water.

iii. Mini habitats

As the name suggests, these are small constructions with just enough room for the arms and sometimes the head of the welder/diver, see Figure 6.2.4.

iv. Portable dry spot habitats

These, in essence, only protect the welding head and a small area around the weld. The clear plastic box, fitted with sponge or flexible rubber seals, moves with the head. These devices have not undergone as much development as either large habitat welding or wet welding.

The problem with hyperbaric techniques is that the environmental pressure at which the weld is carried out is essentially that of the worksite. These elevated pressures affect the gas/slag/metal reactions for all welding processes, and the high density gas enhances the rate of heat loss from the weld. Hyperbaric welding research is mainly concerned with ensuring that for any specific environmental pressure and composition, welding parameters can be specified which will ensure the production of welded joints with properties acceptable to the certification authorities responsible for the structure on which the weld is being made. These problems are discussed in more detail in Section II 6.4 and also in Part IV. Because the welding process has to be specially optimised for hyperbaric conditions, the number of techniques used has been limited. The great majority of welding is carried out using GTAW and SMAW, with small amounts of FCAW and GMAW.

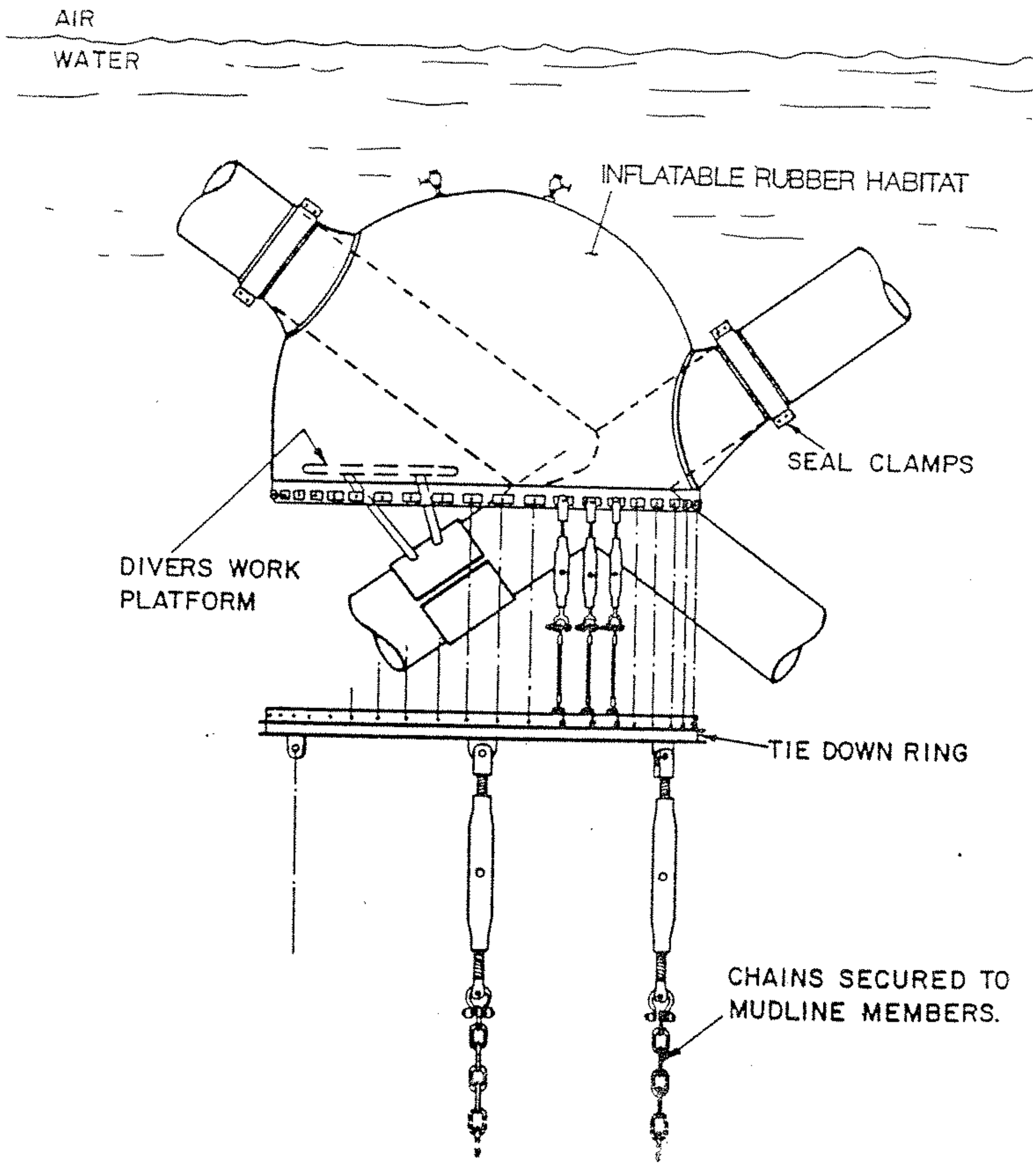

Figure 6.2.3: Inflatable habitat

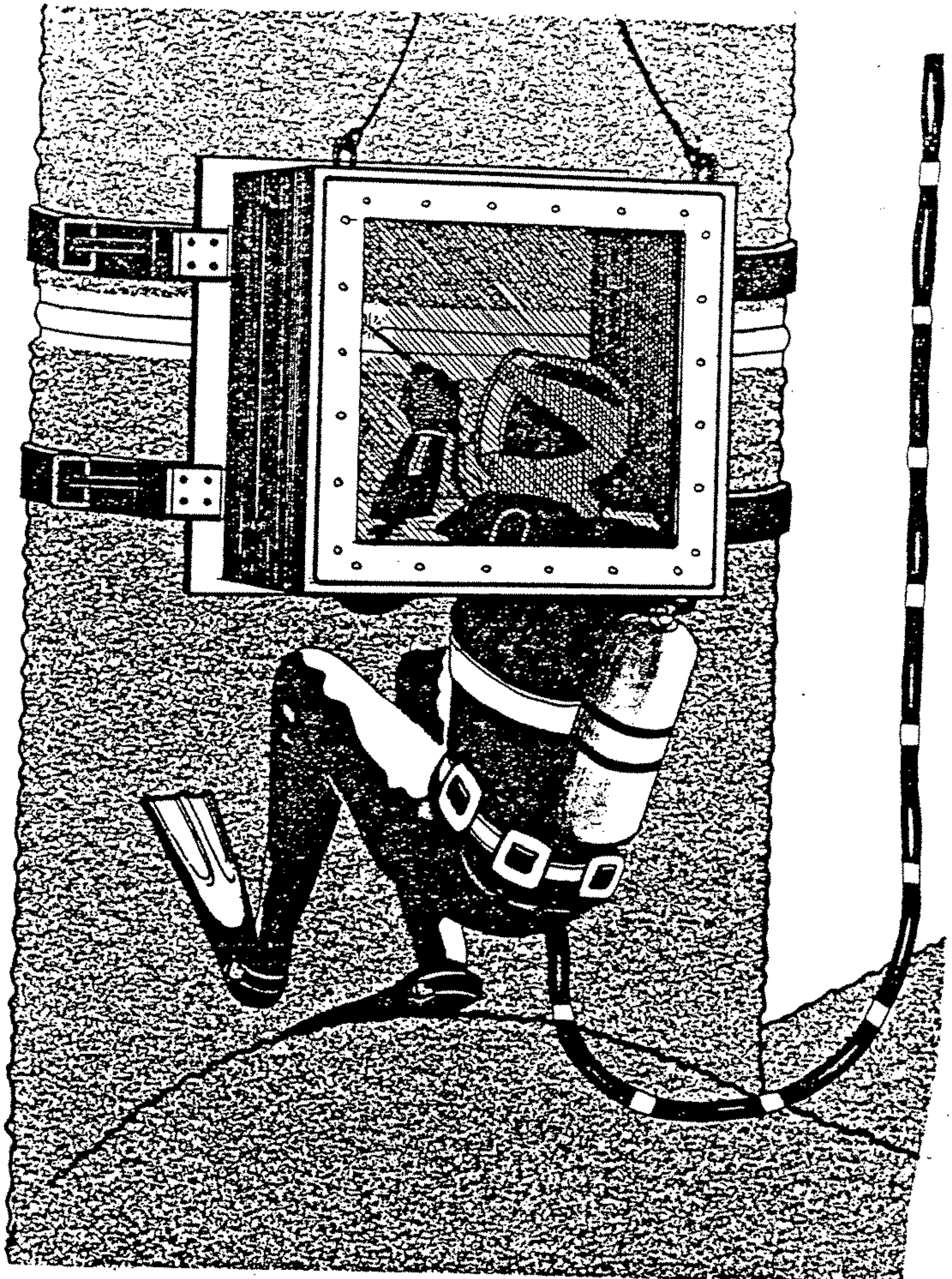

Figure 6.2.4: **Mini habitat**

II 6.2.2 Controlling Parameters for Design

Cofferdams, especially the open top variety, are uneconomic for depths greater than about 15m (50 feet) due to the substantial amount of steelwork required to resist the differential pressure. Sealing the cofferdam can also be a problem for these depths. Even for smaller depths, cofferdams are heavy items and this, together with the large environmental forces they attract, has limited their usage to members having appropriate strength and rigidity, such as leg members. Nevertheless, cofferdams should be considered for all splash zone repairs because of the significant advantages that are gained: protection from environment (shallow depth hyperbaric chambers suffer from wave depth effects) non-diver welders and inspectors have access, normal one-atmosphere welding processes are all applicable, and simplification of life support services.

Pressure-resistant chambers have been little used, despite having the great advantage that the technique can utilise the consumables and welding procedures developed for the original construction of the structure under repair. Their major drawback centres around the problem of sealing the working chamber to a structure with a joint capable of ensuring the pressure integrity of the chamber. Operational systems were developed some years ago for use in the Amazon basin, but these were used for pipeline tie-ins where the end of the pipeline could be fitted with a special coupling for the pressure joint, and where the work chambers were repositioned on the platform structure. This technique will undoubtedly be considered very carefully as a potential system for use in very deep water in a few years' time, but the engineering problems are formidable, and will become worse as water depth increases.

A significant design constraint for hyperbaric welding concerns the habitat. It may not be feasible to use a habitat around a complex joint due to the complexity of fitting and sealing the habitat around each of several brace members. It is also difficult to install a habitat in shallow water due to wave and current forces.

Habitats having flexible chambers can accommodate the significant departures from design geometry frequently encountered on offshore structures much more easily than rigid chambers. However, such chambers cannot be utilised as mountings for the ancillary equipment required for the repair procedure, and a support structure must be constructed within the chamber to provide secure standing room for the divers working within. This, together with the problem of reacting the buoyancy forces generated by the flexible chamber, complicate the installation procedure. The Magnus repair carried out in 1991 by Comex utilised a compromise solution in the form of an ingenious combination of flexible and rigid chamber to overcome access problems at the repair site.

The effect of depth on hyperbaric welding processes also requires most careful consideration. This topic is more fully addressed in Parts III and IV of this document.

II 6.2.3 Equipment and Offshore Support

A typical dry welding operation will require the following items of equipment:

- Purpose-built habitat or cofferdam
- Saturation diving support (habitat only)
- Environmental control equipment (habitat only)
- Pre- and post-weld heating equipment
- Welding equipment (often GTAW and one other)
- Weld inspection equipment
- Equipment to remove marine growth and grit blast to Sa 2½
- Temporary holding clamps to take weight of additional members and maintain root gaps
- Cranage.

Most underwater welding equipment is transported offshore in a standard transport container. It is common practice, upon reaching the worksite, to remove the service umbilicals and welding equipment, and utilise the container as the welding control station. Often, it will also be used as the diver communication centre. Space is therefore required, as close to the repair site as reasonably possible, for a container of this type, adequate supplies of gas for dewatering the chamber and supplying shielding gas if required, and to lay out service umbilicals. Adequate cranage is required to deploy welding chambers if required, and procedures developed to ensure that access to the repair site, both for cranes and diving personnel, is optimised.

For all forms of arc welding, a power supply of appropriate output characteristics is required. Normally, these operate from a three phase 380 or 440 V AC electrical supply, although units powered by other prime movers are also available. In practice, several or more kilowatts of electrical power will be required for welding operations, which will create a rapidly changing load for the power system. It is necessary to ensure that this can be supplied without disruption to other systems on the offshore platform or vessel from which the welding operation is being undertaken. More information on power supplies is given in Part III.

II 6.2.4 Inspection/Maintenance

Monitoring during welding is as important as post-weld inspection. It is essential that all organisations concerned with an underwater welding operation agree the parameters which will be monitored, and the techniques used to record them. All certification authorities have codes of practice relating to process monitoring, as do many offshore operators. Care should be taken to ensure that the process monitoring and quality assurance procedures selected conform to the requirements of all relevant organisations. Further detail is given in Part III.

At various stages during a welding repair procedure, it will be necessary to carry out some form of inspection to ensure that the weld meets the required standards. The techniques used are similar to those employed for surface based inspection of welds. Visual, magnetic particle, eddy current, ACPD, ultrasonic and radiographic techniques are all available for underwater use, and suitably trained diver-inspectors are available. As with process monitoring, care must be taken to ensure that the procedures selected conform with the standards of the certification and operating authorities.

Subsequent inspection would normally be of the same type and frequency as applied to other, similar, parts of the installation.

II 6.2.5 Timescales

The time expended on cofferdam or habitat design, fabrication and deployment can take up a considerable portion of the schedule, depending on the complexity of the enclosure. The time for weld procedure trials, and for welder/diver training and qualification must also be included in the schedule, although often this can be concurrent with habitat constructions.

Most of the time required for an underwater welding repair is not taken up by welding operations, but by the necessary preparatory work, and this must be planned with considerable care. Assembling and sealing the welding chamber can take a time comparable to the welding operation, especially around the more complex node geometries. A compromise must be struck between deploying the chamber in as few pieces as possible, so minimising the amount of underwater assembly work, and the increase in risk for divers working with large chamber components, especially where such work is to be carried out in tidal or splash zone conditions. It should not be assumed that the actual geometry of the structure is precisely as designed, and it is normal practice to undertake a survey of the location before building the welding chamber. It will be necessary to clean portions of the structure to ensure that an effective seal can be achieved, allowing the chamber to be dewatered. The welding chamber must be made sufficiently large to enable the welders to have effective access to the weld site.

II 6.2.6 **Previous Offshore Applications**

Repairs by both cofferdam and hyperbaric habitat welding techniques have proven track records. Hyperbaric welding has been used as an underwater SMR techniques since about 1970 and is the normal method for effecting subsea repairs in the North Sea. References 6.1 to 6.9 give details of some case histories.

II 6.3 WET WELDING

II 6.3.1 Description

Underwater welding, when the arc is operated in direct contact with the water, is termed wet welding. It is the oldest underwater welding technique, and has as its major advantage the lack of a requirement for a welding habitat or chamber. This advantage is offset against the poorer weld metal properties, the low deposition rate possible with wet welding, and the high levels of skill required by the welders. Although widely used in America, it has found little favour in Europe.

Virtually all wet welding is carried out using SMAW, although some research has been carried out into wet FCAW, and this is currently being evaluated for operational use.

The advantage of avoiding the assembly of a welding chamber around the repair site, is particularly relevant for platform repairs, where the geometry of the steelwork adjacent to the repair site may be complex, requiring any chamber to be assembled from several sections, a task which may be comparable in duration with the welding time. However, these advantages are compromised by limitations in the properties of the weld metal which is produced by the technique, a problem which is discussed in more detail in Part III.

II 6.3.2 Controlling Parameters for Design

It is generally accepted that the weld quality of welds formed by wet welding is not as good as those formed by dry welding techniques. Some of this is due to operational difficulties such as lack of visibility because of the gases liberated, but most of the difficulties are metallurgical in nature:

- dissociated water vapour leading to hydrogen entrapment and the possibility of hydrogen induced cold cracking (HICC)
- hard, crack-susceptible HAZ's caused by the rapid quenching effect of the water.

The fatigue strength of wet welds is lower than dry welds and thus wet welding is not recommended for locations subject to significant fatigue loading.

The above comments do not apply to friction welding. This can be used on members of any thickness and the fatigue life predicted using standard approaches.

II 6.3.3 Equipment and Offshore Support

A typical wet welding operation will require the followings items of equipment:

- diving team support
- welding equipment (power supply, gas supply, consumable handling units, welding torches and umbilicals)
- equipment to clean work area
- temporary holding clamps to take weight of additional members and maintain root gaps
- cranage.

The arrangements for wet welding are generally similar to those given in Section II 6.2.3.

II 6.3.4 Inspection/Maintenance

The inspection/maintenance methods given in II 6.2.4 are equally applicable.

II 6.3.5 Timescales

Wet welding is faster on the whole than dry welding, as fabrication and installation of a habitat is not required.

There are little factual data in the literature. Hughes et al[6.10] reports that over 12,000lbs (5440kg) of welding electrodes was used in 10,000 man hours of underwater work concerning wet welding of patch plates to correct corrosion faults in the Gulf of Mexico. Green[6.11] reports that it took 35 days for a wet welding team to effect repairs to two platforms taking 55 hours of arc time to lay 63 feet of ⅜" (9.5mm) fillet weld. The repairs involved 7 sites at various depths from 12 to 120 feet (3.7 to 36.5m).

II 6.3.6 Previous Offshore Applications

Wet welding is a popular method of repair in the Gulf of Mexico where a combination of shallow water, low fatigue environment, and operator philosophy has led to its usage for a number of years.

Although it has been used in the North Sea for the repair and attachment of secondary items, it has not been used for major structural repairs with one notable exception[6.12].

References 6.10 to 6.12 give case histories.

II 6.4 UNDERWATER WELDING PROCESSES

II 6.4.1 Hyperbaric Welding Processes

II 6.4.1.1 Shielded Metal Arc Welding (SMAW)

Also known as Manual Metal Arc (MMA) welding, hyperbaric Shielded Metal Arc Welding (SMAW) is carried out using flux covered welding electrodes, electrode holders and welding techniques very similar to those used for SMA welding on the surface. Because of the simplicity of the technique and equipment, and the availability of a relatively large number of diver/welders trained in SMAW, it is the most widely used operational repair technique. Due to excessive electrode burnoff, the process has heat input limitations, and careful procedure control is necessary to avoid problems relating to hydrogen-induced cold cracking (HICC).

II 6.4.1.2 Gas Tungsten Arc Welding (GTAW)

Also known as Tungsten Inert Gas (TIG) welding, Gas Tungsten Arc (GTA) Welding is the most controllable technique available to hyperbaric welding engineers. Similar techniques are used for both hyperbaric and surface welding.

Because of its simplicity, it has been widely studied by process physicists, and is better understood than other hyperbaric welding techniques. An arc is struck between a non-consumable tungsten electrode and the workpiece. Filler material can be added, in rod form by the welder. Automated variants on the process are also used underwater.

The high level of control available to a skilled diver / welder has led to its use for critical welding situations such as the root and hot pass welds, and for capping passes and temper beads where the shape and hardness of the material at the toe of a weld must be controlled, normally for fatigue resistance (TIG dressing).

II 6.4.1.3 Gas Metal Arc (Flux Cored Arc) Welding (GMAW/FCAW)

Also known as Metal Inert Gas (MIG) or Metal Active Gas (MAG) Welding, this technique, GMAW, is a consumable electrode process. A hand torch is used, and a continuous wire electrode, between 0.6 and 2 millimetres in diameter, is fed through it. This wire, and the workpiece, are connected to the output poles of the welding power supply, so that when the wire touches the workpiece, an arc is struck which melts the wire and the workpiece to form a molten weld pool. To protect this, a shielding gas is fed through the torch concentric with the wire in a similar manner to that employed for the GTA process.

GMAW was proposed for underwater use in the 1970's, but the welding equipment available at the time could not respond to the unusual demands of the hyperbaric environment, and the process lacked stability and adequate fusion characteristics. When these limitations were recognised, the use of tubular consumables (FCAW) was suggested as a way of overcoming the deficiencies. FCAW is a variant using tubular consumables covered with flux. Unfortunately, the relatively high cost of these consumables, and the relative complexity of GMAW welding equipment, with its associated consumable feeding systems, made the process unattractive to the offshore industry compared with hyperbaric SMAW, and little use has been made of the technique.

II 6.4.2 **Wet Welding Processes**

Virtually all wet welding is currently carried out using the SMAW process. Using waterproofed electrode holders and welding consumables, it is possible to maintain an arc underwater, although the welder's view of the weld pool can be disrupted by gases evolved from the breakdown of the electrode flux material.

During wet welding, because of the proximity of cold seawater to the weld pool, high cooling rates are experienced by the weldment. In addition, dissociation of water within the welding arc ensures the presence of hydrogen in the weld pool. Both of these phenomena adversely affect the final weld. The high cooling rates generated in the HAZ and weld metal, for the types of steel used in offshore construction, generate brittle metallurgical structures of low toughness.

FCAW has been proposed as an alternative to SMAW, and offers higher heat inputs and greater productivity, at the cost of more complex equipment. It is currently being evaluated for operational use, and information concerning the technique is limited.

II 6.4.3 Other Processes

II 6.4.3.1 Friction stud welding

Underwater friction stud welding is not a flexible joining system, but it is reasonably well-developed.

In essence, the stud is pressed against the object to which it is to be welded with a controlled level of force, and then rotated at a defined speed for a set time. The friction between the stud and the workpiece generates heat which raises the temperature of the material close to melting point adjacent to the interface. Once sufficient heat is generated, the rotation is stopped, and the stud pressed harder against the workpiece, and held until the material has cooled sufficiently to weld together. The joint area is protected from the cooling effects of the surrounding water by a polymeric shroud mounted over the stud prior to welding.

This system has been tested at pressures equivalent to 600 metres of water, and seems unaffected by depth. Current development programmes are seeking to make it deployable by current generation ROVs, and some operations have recently been undertaken utilising ROV deployment.

Although limited to circular or near circular studs of up to 30mm diameter, the system has found a wide range of applications in the fixing of protective anodes and their connections, and the mounting of shear pins.

Recently, a development of the friction weld process has been tried for the repair of cracks. A tapered hole is formed at the crack tip, and then a tapered pin is inserted and friction welded in place. A series of contiguous pins are thus formed, giving rise to the name of the technique - stitch welding. However, the process is still largely experimental and considerable development work would be required to bring it to full operational status.

II 6.4.3.2 Explosive welding

Explosive welding is a forge welding technique, in which the two components to be joined are positioned lapping over each other, with a slight angle between them. The explosive is coated onto the back of one plate - the 'flyer' and is ignited at one end. As the explosive burns, the flyer is driven forcefully against the 'anvil' plate, and if the surfaces are sufficiently clean, a solid phase bond is formed. The technique is reliable and forms joints rapidly, although it is confined to lap type geometries.

It has not been adopted offshore, largely owing to concerns over safety.

II 6.4.3.3 Plasma hyperbaric welding

Plasma welding is a development of GTAW in that the plasma arc is constricted by means of a copper or carbon constriction a few millimetres in front of the electrode tip. The process has enhanced arc stability compared to GTAW, and has greater resistance to external influences such as magnetic fields. However, considerable development work is required to bring it to operational status.

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II 7 BACKGROUND TO AND DESCRIPTION OF WELD IMPROVEMENT TECHNIQUES

II 7.1 GENERAL

Weld improvement techniques are solely concerned with the enhancement of fatigue life, and are not applicable to increasing static strength. Indeed, for some weld improvement techniques, particularly remedial grinding, the degradation of static strength has to be specifically considered.

Weld improvement techniques improve fatigue life by eliminating one or more deleterious aspects, which occurs at weldments, through one of the following mechanisms:-

- removal of inherent welding imperfections and other defects thus greatly extending the fatigue crack initiation period,
- local improvement of weld profile which reduces stress concentration factors and thereby the stress range acting at the weld,
- by introducing compressive residual stresses in the surface layer, replacing the tensile residual welding stresses,
- changing the orientation and shape of welding imperfections and other defects.

The weld improvement techniques that may be considered are:

- toe grinding
- remedial grinding
- hammer, shot and needle peening
- TIG and plasma arc dressing.

Weld improvement techniques are not always associated with weld failures, and in most cases the objective of a weld improvement operation is to reduce the potential for weld failure, ie. increase the endurance of the welded connection.

Before embarking on a weld improvement programme, it should be realised that bad welds remain, even improved, poor welds. Also weld improvement techniques may be ineffective where, for example, inaccessible root defects remain which then allow crack initiation and premature failure.

The overall benefits of local improvement can be limited by the fatigue conditions elsewhere. With potential improvements as large as 15 on life, root defects and inter weld bead cracking may become the limiting condition.

II 7.2 TOE GRINDING

II 7.2.1 <u>Description</u>

Toe grinding is the purposeful removal of weld and parent metal from the welded joint. The operation is normally undertaken with a grinding tool though milling is also used. Two major techniques have evolved:

- Disc grinding, using an angled tool technique to cut a groove in the weld toe. (Note: the use of a disc grinder is not recommended in this document.)

- Rotary burr grinding. A special portable machine tool is used to cut a neat groove in the weld toe. It is slower, more expensive and more specialised than disc grinding.

The aim of the operation is to excavate a regular groove, a circular curve, into the toe of the weld, thereby removing toe weld defects and providing a smooth transition from the weld profile to the parent plate. **The groove must remove some of the parent metal** in order to be effective, and yet a physical limit must be placed on the cut dimensions, x and y in Figure 7.2.1 (see Section II 7.2.2).

It is standard practice to specify the radius of the groove, R, to be used in the grinding operation. The norm is $R = 10\,mm \pm 0.5mm$, which is consistent with the manual arc welding of conventionally sized tubular joints. Smaller tool sizes may be justified with plate thicknesses less than 20mm, or where other welding techniques have been employed.

The toe is ground on both the chord and the brace side of the weld, as shown in Figure 7.2.1.

If inter-run failure is a possibility, full grinding will be necessary. Full burr grinding involves grinding the whole of the visible weld profile, using a rotary burring tool. The benefit of full as opposed to toe burr grinding is not clearly established.

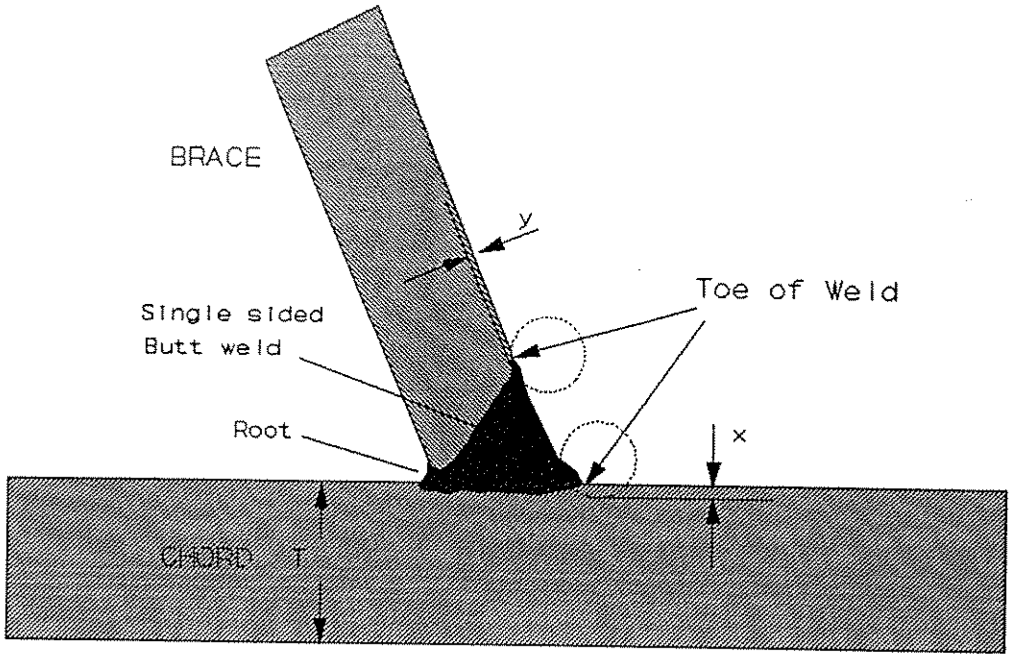


Figure 7.2.1: A typical application of toe grinding

II 7.2.2 Controlling Parameters

The depth of the cut relative to the thickness of the base metal is the key controlling parameter. The depth of the cut should be sufficient to remove some of the plate material.

The depth of cut into the local plate, x is given by:

$$0.5\text{mm} \leq x \leq 2\text{mm} \qquad \text{.... 7.2.2}$$

A limit of 5% of the plate thickness is normally placed on the depth of the cut. The depth of cut will usually be less on the brace than on the chord side, as shown on Figure 7.2.2.

Localized relaxation of thickness requirements can be given where a toe is to be ground in order to remove a surface defect, see Section II 7.2.4 below.

In certain cases, the benefits of toe grinding may not be realised in increased fatigue performance. Two major problems must be identified:

- Defects will remain in the root of a single-sided butt weld.

- If the joint is in a corrosive environment, the smooth toe ground area may quickly become an erosion site and corrosive cracks will take the place of the (removed) weld toe imperfections.

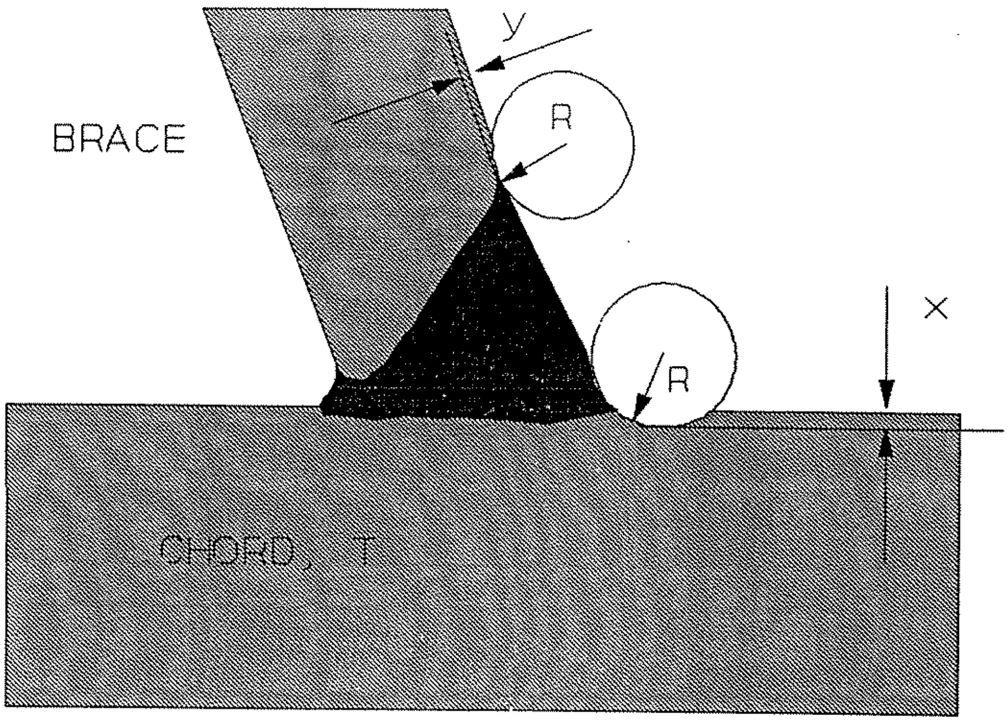


Figure 7.2.2: Weld toe grinding detail

II 7.2.3 Equipment and Offshore Support

Toe grinding (or milling) is undertaken by specialist diving and subsea contractors. It is normal for the contractor to supply men, light equipment and small tools.

Grinding of weld toes is undertaken using one of two tools:

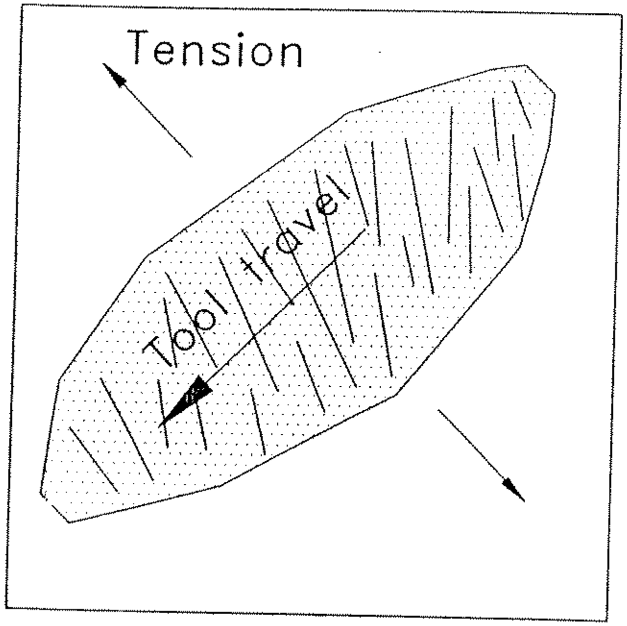
- Tungsten tipped rotary (or "Burr") grinder
- Disc grinder (gritstone in epoxy matrix type)

Both types of tools can be employed for toe grinding in air (ie. at the fabrication stage or for elements of the structure that are out of the water or inside a dry welding habitat). For wet repairs underwater, specialised grinding tools are required, which are readily available. Standard units are powered by hydraulic pressure; other power sources are rarely used for reasons relating to operator safety. The rotary head (or burr) grinder should be operated at a speed in the range 18,000 to 48,000 rpm (see for example section 2.1.6 in Tubby[7.1]) in order to achieve the desired effects. Surface trials should be undertaken to check the finish produced by an individual power tool/cutting head arrangement. The speed of operation of a revolving power tool will generally be lower in water than in air for a particular power input.

Burr grinding is the preferred method for weld improvement. The use of a disc grinder results in scratch marks which run along the length of the weld seam. A tungsten burr grinder produces much smaller scratches which are aligned across the weld, see Figure 7.2.3. Scratches produced by a rotary burr are less likely to lead to fatigue cracks because the principal stress in the weld is not aligned across the defect produced by the cutting operation. If a disc grinder is used, however, not only are the scratch marks deeper but they are also aligned across the direction of principal stress and therefore more likely to become crack initiation sites. Tubby[7.1] emphasises the problem of residual defects and suggests that weld polishing with emery bands may help bring a uniformity of finish to ground welds. Although not a problem for work carried out in the dry, the availability of underwater polishing equipment is not known.

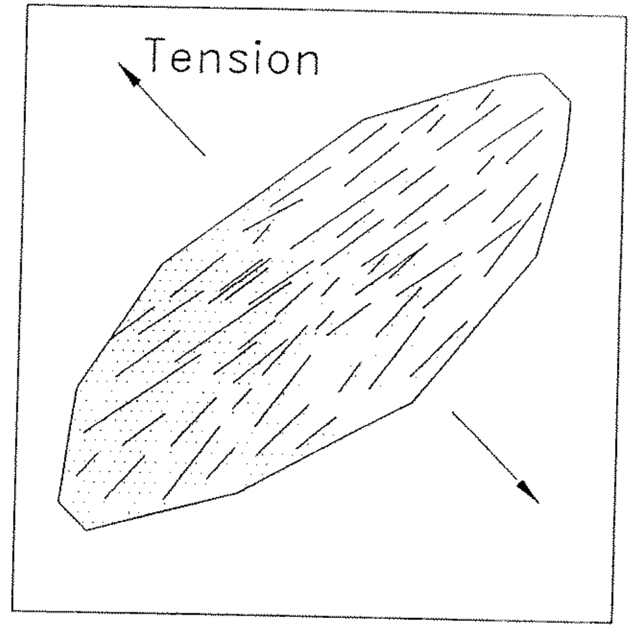
The process of abrasive water jetting has been described by King[7.2] as a possible means of achieving toe grinding. The technique is not recommended, for reasons relating to safety and tool control.

Grinding a joint in air demands little preparatory work, save to ensure that the work is adequately supported. For underwater repairs the position is completely changed. If a diver is to provide a reaction to a high powered grinding tool then a demountable cradle or strap must be provided. Marine growth must be removed from the target area to be repaired.



Burr rotation produces grooves across the direction of tool travel.

Burr Ground Sample



Disc grinding produces more scratches and deeper scratches. The scratches are aligned with the tool travel.

Disc Ground Sample

Figure 7.2.3: Scratches produced by grinding operations

II 7.2.4 Inspection and Maintenance

When toe grinding is performed using a disc grinder, the groove radius is not directly specified but a standard weld inspection tool is employed to check the finished weld.

If a weld is to be improved by grinding then detailed inspection is advised on completion of the toe grinding operation. It is important that NDE of the finished weld should target surface defects in the toe grooves and root defects in single sided weld preparations. Dye penetration and MPI techniques or eddy current methods should form the basis of testing for surface cracks for operations undertaken in air.

Inspection of the toe groove can turn up defects which require remedial action. Surface flaws can be re-ground up to the plate limit, or even deeper, see Section II 7.3. Root defects or other non-surface indications will need to be gouged and repaired in accordance with the weld repair procedure. The repaired weld will then be ground and inspected.

II 7.2.5 Environmental Considerations

Underwater toe grinding, if required, is likely to be confined to areas of a jacket structure where fatigue damage occurs. In broad terms, the conductor guides, conductor guide framing nodes, caisson and riser supports between elevations LAT -15.0m and LAT +12.00m are often the affected areas.

Grinding work will generally be in the domain of air divers. Environmental conditions will probably restrict the operations to periods of low wave height (2m significant sea state). In certain coastal sea areas, shallow diving is restricted by sea currents and may only be possible in a time slot one hour each side of slack water.

There may be justification for restricting the toe grinding to only certain areas of the node under consideration. The regions of the joint which have the highest pseudo-elastic stress in the fatigue climate should be selected if timing places a limit on the areas to be ground.

Grinding the weld provides a surface which will rust if the grinding is performed in air. Booth[7.3, 7.4] advises that the ground weld be treated with grease so that it is unable to rust under atmospheric conditions before it is immersed in seawater and hence protected by the jacket CP system.

II 7.2.6 Timescales

Toe grinding is time consuming work. Woodley[7.5] quotes typical rates for grinding as follows:

Technique	Rate
Disc grinding	0.5 man hr /m
Toe burr grinding	1.0 man hr /m
Full burr grinding	3.0 man hr /m

In a typical offshore node, the length of weld to be ground can be estimated as:

$$L = \frac{2\pi D_{brace}}{\sin\theta} \quad \text{.... 7.2.3}$$

where:

D_{brace} is the brace diameter
θ is the angle of inclination of the brace to the node barrel.

The time taken to perform operations underwater may be significantly longer than those recorded for work in the dry. Account should be taken of the time required for reaching and identifying the work place, setting up the supports, lowering equipment, taking photographs and the exchange of divers. A knowledge of diving tables and associated practices may be required to estimate the duration of a particular piece of work. It is often found that the actual work (ie. grinding the joint) is a minor element of the total time required for the job to be completed. However, the time to grind the work may be estimated from tool cutting times, and an estimate can be made based on crack length and depth of excavation.

If restricted diving conditions apply (see Section II 7.2.5) then the toe grinding program should be designed to allow a rational number of welds to be ground within the diving period.

II 7.2.7 Background Research

The application of disc grinding techniques has been considered in detail by Maddox[7.6].

Woodley[7.5] reports unsubstantiated stress range increases for the improvement of Class F details which may not be reliable.

Booth[7.7] confirms that the advantages of weld toe grinding should only be admissible for underwater welds which have adequate cathodic protection.

Mullen[7.8] outlines the base case for using toe grinding with test data derived for samples tested in air.

II 7.2.8 Previous Applications

Toe grinding has been accepted as a technique for dealing with "problem joints" in offshore platforms since 1980[7.9, 7.10].

The operation has been described on weld detail drawing sheets used for the fabrication of offshore jacket structures since that time.

Toe grinding would not normally be chosen as the sole repair technique for welded joints. The process is normally used in tandem with the remedial grinding described in Section II 7.3 below. However, in a structure where one joint has cracked, and another similar joint has not suffered visible damage (although subjected to the same conditions as the failed joint), the repair of the defective joint will often be accompanied by the toe grinding of the similar but undamaged joint.

II 7.3 REMEDIAL GRINDING

II 7.3.1 Description

Remedial grinding is probably the most common type of underwater repair work as it is standard practice to undertake remedial grinding of any joint found to be cracked.

Remedial grinding involves the excavation and removal of a crack sited at or near a welded joint by cutting a smooth shaped trench in the cracked metal, see Figure 7.3.1.

The bulk of the removed material would normally be removed with a heavy grinding disc and the profile of the trench would be improved by machining with a burring tool. In thin plate sections ($t < 15$ mm) only the burr grinding tool would be employed.

On completion of the first pass of grinding, the finished profile is checked. The ground area is extended until all traces of cracks have been removed. A dilemma occurs in the case of deep cracks which are not normally ground out to more than 90% of the plate thickness.

The remedial grinding is advanced until all the observed crack has been chased out. It is good practice to extend the grinding a distance t (t= plate thickness) past the end of the crack so that sub-surface defects are removed.

It is possible that remedial grinding by itself forms the permanent repair. However, the ground repair may subsequently form the weld preparation for a weld performed in a dry habitat, or it may be encapsulated by a bolted clamp.

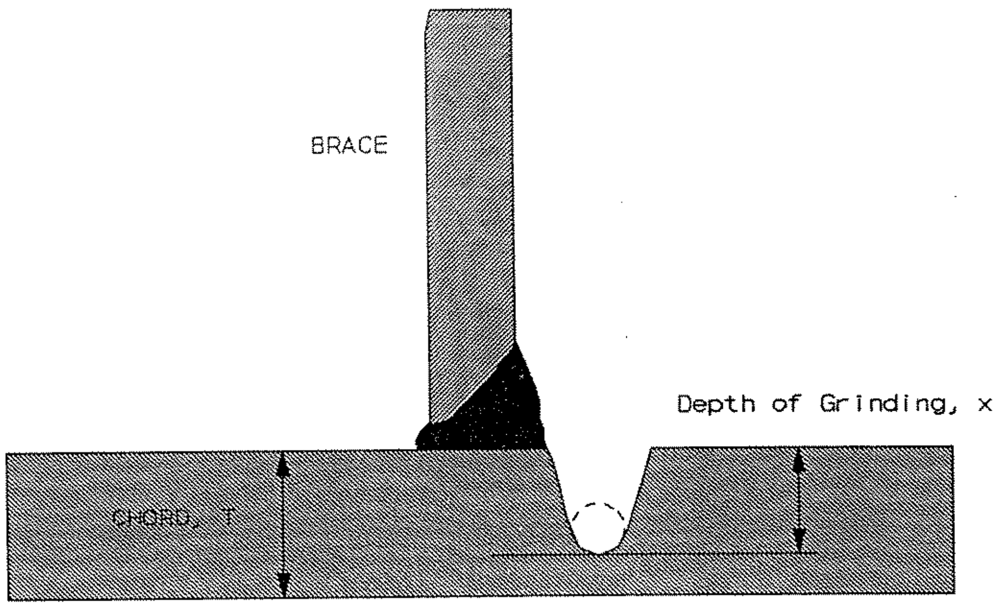


Figure 7.3.1: A deep remedial grind repair to a cracked joint

II 7.3.2 Controlling Parameters

The excavated groove should go deep enough to remove the cracked material (a process known as 'chasing'), without creating a free flooding situation in the joint. On the brace side, partial element severance needs to be fully considered, along with the need to provide alternative support for any tensile loads.

The depth of the remedial grinding should not normally exceed 90% of the plate thickness. It is normal to place a limit whereby the depth of grinding does not exceed 66% of the plate thickness.

In the case of the crack occurring on the chord side of a tubular joint, the depth of excavation may be increased in order to chase out the crack, with the excavation normally stopped off when a nominal metal thickness (one third of chord wall thickness) remains. The limiting groove depth is influenced by the attempt to keep the tubular element internally dry. If the crack has already gone through thickness, and the element is flooded, there is little point in stopping off the depth of the groove until the crack has been chased out.

The circumstances surrounding a cracked weld on the brace side of the joint is somewhat different. The brace stub thickness is usually thinner than that of the chord, and may often represent a highly stressed cross section. Care should be taken when grinding a brace stub which is loaded in tension. A check calculation should be performed so that a simple limit can be set on the depth of excavation to be made at the joint. The supervising engineer should have access to the in place and fatigue analysis computer analysis, along with the appropriate frame, node and detail drawings of the structure.

II 7.3.3 Equipment and Offshore Support

The equipment and support required for remedial and toe grinding are identical, see Section II 7.2.3.

II 7.3.4 Inspection

Remedial grinding must be allied with a surface crack inspection scheme. Either dye penetrant, eddy current or MPI methods are used. The crack grinding proceeds until no further defects can be observed in the parent material. Dye penetration tests are particularly useful as a skilled operator can chase out the visible dyeline under good lighting conditions. Such criteria may not apply to many underwater locations.

Electro-potential measurement techniques (ACPD and ACFM) are particularly accurate methods for determining crack location and size and are recommended for underwater work.

Inclined cracks can pose a problem as the underlying defect may extend well past the originally discovered surface cracks. Ultra-sonic or ACPD probing may be advised for mapping the possible extent of the cracked region before any grinding is attempted.

II 7.3.5 Environmental Considerations

See Section II 7.2.5.

II 7.3.6 Timescales

Remedial grinding is more time consuming than toe grinding (see Section II 7.2.6) due to the greater excavation depths involved. The defect needs to be chased out, smooth ground, inspected and then the whole process repeated if any remaining cracks are found.

II 7.3.7 Background Research

The quality of research undertaken into the repair of underwater joints by remedial grinding reflects the importance of this type of repair.

The publication OTH 89 307, Fatigue Performance of repaired tubular joints, by PJ Tubby of the Welding Institute[7.1] records the details of a joint industry project undertaken by the Welding Institute on behalf of a number of sponsor companies. The project, completed in 1987, concerned a series of fatigue tests on welded tubular T joints in steel in which fatigue cracks were repaired by a number of alternative methods. A number of principal findings were made, including the following for ground repaired joints:

> *"Removal of part-wall flaws by grinding is an effective repair method giving endurances after repair equal to or up to four times greater than the mean for unrepaired joints."*

Veritec report a project of similar proportions in their work of 1987. The project titled "Grind Repairs of Welded Structures" encompassed two phases, flat plate and tubular joints, and reported results for a series of defined tasks.

A stress analysis method is outlined and verified. The static strength of grind repaired joints was investigated. Physical testing was undertaken on samples of realistic sizes. For example,in one T joint static test, the chord element was 508 x 16 and the inclined brace stub was 245 x 10.

The VERITEC report confirms that the improved profile resulting from grind repair has a higher strength and reliability that a similar, cracked joint.

II 7.3.8 Previous Applications

Williams and Callan[7.11] give a detailed account of the grind repair of a very large defect on a tubular brace (caused by a dropped object) forming part of a steel offshore structure. The approach taken, which was to minimise the local SCF effect whilst removing a large section of the damaged element, is totally consistent with the design philosophy used for the grind repair of smaller weld defects. A novel approach was to model the defect in acrylic in order to establish the SCF associated with the repair. The acrylic model technique has been used by other researchers to determine SCF's for typical welded joint details.

Remedial grinding is an operation which is commonly undertaken in the course of underwater examination and repair. There have been numerous examples of this type of repair, which are treated as routine, with the result that few individual cases are recorded in the literature.

II 7.4 SHOT, NEEDLE AND HAMMER PEENING

II 7.4.1 Description

Peening is a cold working process in which the surface layer of the component (or weld) is plastically deformed either by high velocity shot (shot peening) or by a tool (needle or hammer peening).

Under each impact of shot or tool, a plastic zone is created; the material outside this zone being elastically deformed in compression. After the shot has recoiled, or the tool has passed by, the elastic stresses in the adjacent material will result in permanent compressive stresses within the plastically deformed zone. Gradually, as the treatment progresses, the whole surface layer will contain compressive residual stresses. Deeper in the material tensile stresses are induced which compensate for the compressive surface stresses.

It is the introduction of the compressive surface stresses which allows improved fatigue lives to be realised. The majority of all fatigue cracks initiate at weld toes where welding defects exist and where, normally, residual stresses are tensile. In a peened part, however, the residual stresses are compressive and the service stresses will superimpose as indicated in Figure 7.4.1. The net result is that the initiation of fatigue cracks will be delayed or even prevented.

During peening, work hardening occurs in the plastically deformed zone. The work hardening increases the yield strength and this also contributes to an increased fatigue strength.

In the case of hammer peening, and to a lesser extent needle peening, the weld toe profile may be improved, thus reducing the severity of the stress concentration leading again to an apparent increased fatigue life. Since the hammer tends to jump and miss small regions, a single pass along the weld toe is not usually sufficient to ensure that no area remains in the as-welded condition after peening. Consequently 2 to 4 passes are normally specified.

II 7.4.2 Controlling Parameters

As in all weld improvement techniques, the benefit of extended fatigue life may only be realised if premature failure does not occur at defects at the root or within the weld. Any defects likely to cause crack initiation must be eliminated.

Peening techniques have to be subject to careful control during their implementation. The degree of fatigue life improvement obtained is dependent on the care taken when applying the treatment and the work should only be performed by properly trained and qualified personnel.

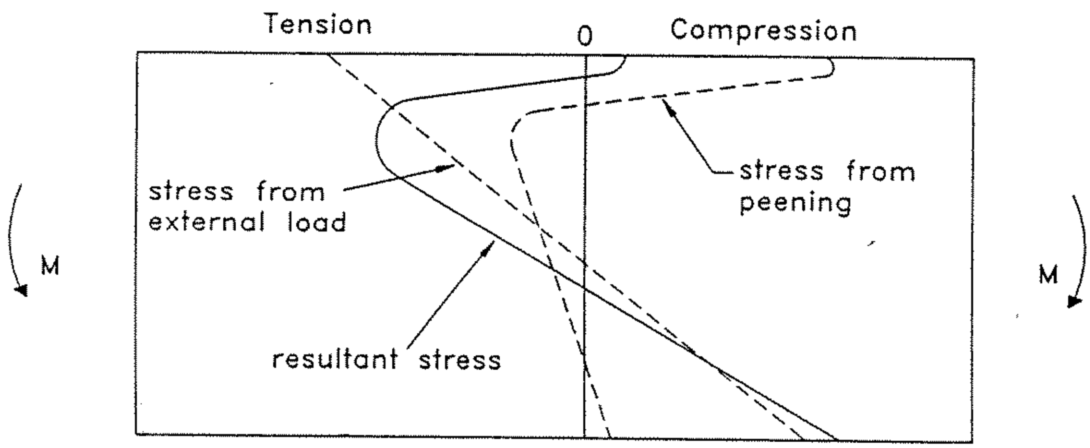


Figure 7.4.1: Applied, residual and resultant stresses

Peening relies to a large extent on the introduction of compressive residual stresses in the surface layer. The magnitude of these residual stresses depends on the intensity of the treatment and process controls such as the tool tip radius, hammer angle and velocity in the case of hammer peening.

Shot peening is unsuitable for application underwater (except possibly in a hyperbaric chamber) as the water slows the shot down and renders the technique ineffective.

There are few public domain data for hammer and needle peened tubular joints. The degree of fatigue life enhancement therefore has to be largely estimated from data obtained on plate cruciform specimens or from specially commissioned tests.

II 7.4.3 Equipment and Onshore Support

Peening is a technique which has been applied in air but not, to any great extent, underwater. As such the equipment used to date may be unsuitable for underwater use without further development.

During shot peening the surface of the steel is bombarded with small balls having a diameter from 0.2 to 2.0mm. The balls are made from steel, stainless steel, glass or ceramics, and are accelerated by a spinning wheel or by compressed air.

In hammer peening, a steel round-nosed tool bit is impacted onto the weld using pneumatic, electric or hydraulic power. The bit is round-nosed, typically of diameter 3 to 13mm. The tool forms a depression or groove, the depth of which is a function of material type, power input, and the shape of the bit. As well as introducing compressive residual stresses, hammer peening will also deform defects, such as undercuts and surface slag intrusions, to the benefit of improved fatigue life.

Needle peening has some operational similarity to hammer peening in that similar tooling is used. However, rather than just a single tool bit, the impacting is achieved with a bundle of rods (needles), each rod being about 3mm in diameter. Since the impact energy is spread amongst the rods, no groove is formed.

Recently, an ultrasonic tool has been developed in the Ukraine and is faster and more easily controllable than conventional pneumatic tools. Currently, work is underway to further develop it for underwater use. Its power output is such that it is similar in effect to needle peening.

II 7.4.4 Inspection and Maintenance

In all peening techniques, the magnitude of the compressive surface stresses, the depth of the surface layer and the uniformity of the operation should be reproducible. The inspection methods used to ensure that this is achieved differ between shot and hammer peening as discussed below. There does not appear to be a standard method for needle peening.

In the case of shot peening, an ALMEN strip is used[7.12]. The ALMEN strip is a standard metal strip attached to the workpiece and which is treated similarly (eg. rate of coverage) as for the weld toe. On removal, the strip will curl due to the residual stresses imparted by the treatment, and the amount of curling may be compared to the standard scale defining the ALMEN intensity. The ALMEN intensity is, to a first approximation, proportional to the cold working depth. To ensure uniformity a second control is generally used and this relates to coverage. Preliminary trials are carried out to establish the time for shot peening to completely remove a fluorescent dye applied before peening. This time is normally doubled, to obtain a coverage of 200%, in the actual operation to ensure uniformity[7.13].

The effectiveness of hammer peening depends on the number of passes, as mentioned in Section II 7.4.1, and the duration of the operation. If the operation is carried out too rapidly, the depth of the deformed area may be insufficient to surround all defects with the required level of residual compressive stress. The easiest parameter to inspect, to ensure that the operation has been carried out to a satisfactory standard, is the depth of the groove. This will depends on the strength of the steel and trials may be called for. As a guide, 4-pass hammer peening forms grooves of approximately 0.6mm and 0.45mm in steels having yield strength of around 275N/mm^2 and 350N/mm^2 respectively. Further passes should not appreciably alter the depth.

For both shot and hammer peening, the treated area may be inspected for any remaining defects using, for instance, MPI. Note that unless toe grinding has been carried out first, hammer peened welds may still give crack-like indications.

II 7.4.5 Timescales

Data on timescales are restricted to in-air applications performed in testing laboratories. Knight[7.14] quotes a time of 0.25 hours per metre length of weld for 4-pass hammer peening. This rate compares very favourably with the quoted rates for grinding which are almost identical to those tabulated in Section II 7.2.6. However, operator skill and experience, together with the differences entailed in underwater working, could alter the above rate considerably.

As in the case of grinding, the actual operation of peening may only form a minor element of the total time spent underwater.

II 7.4.6 Background Research

The improvement in fatigue life by shot peening has been explored by reference to data mainly generated from flat plate specimens[7.13 to 7.18] although some tubular joint data exist[7.19]. The tubular joint data confirmed the great improvement in life observed in the flat plate specimens. Furthermore, the improvements observed in air would appear to follow through to joints with adequate cathodic protection.

Most data are concerned with constant amplitude tests but the few variable amplitude tests have shown that shakedown (ie. partial loss of the compressive residual stresses caused by overstressing) should not be a significant factor.

The research conducted on hammer peening has, so far, all been directed at flat plate specimens[7.3, 7.7, 7.14, 7.20 and 7.21]. The data indicate that the effect of hammer peening may be dramatic, with over a ten-fold increase in fatigue life being possible.

For both shot and hammer peening, most benefit is obtained for the high cycle range of the load spectrum.

II 7.4.7 Previous Offshore Applications

Although operators are actively contemplating using peening, no offshore use has been reported. However, peening has been used onshore to good effect in such applications as bridges, earth-moving equipment and other structures subject to dynamic excitation.

II 7.5 OTHER IMPROVEMENT TECHNIQUES

There are a number of other weld improvement techniques reviewed in the literature[7.13, 7.22]. These include:

- Remelting techniques (dressing)

  Here the weld toe is remelted using autogenous GTA (TIG) welding[7.23, 7.24] or plasma arc welding. These processes remove the defects and generally improve the toe shape.

- Use of special electrodes

  These improve the wetting characteristics of the molten weld metal giving improved final bead shapes on solidification. They may give positional welding problems[7.22].

- Prestraining

  The effect or large tensile preloads is to reduce residual tensile welding stresses. Although apparently effective[7.21], it is not feasible for assembled structures.

In strengthening and repair scenarios, the above techniques will find either no or very little application. A possible exception may be in dry welding, in which remelting techniques are applied to the repair weld (see Section II 6.4.1.2).

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II 8 BACKGROUND TO AND DESCRIPTION OF CLAMPING TECHNOLOGY

II 8.1 GENERAL

This chapter presents the background to and the description of a number of strengthening and repair techniques which deploy clamping technology. The techniques covered are as follows:

- stressed mechanical clamps
- unstressed grouted clamps/sleeve connections
- stressed grouted clamps
- stressed elastomer-lined clamps

Several aspects will influence the selection of the most appropriate repair type and geometry. Factors that will determine this process include hoop stress checks on the existing member, any limitations on clamp geometry, magnitude of forces in existing member(s), availability of accurate survey measurements and the required design life of the repair.

Although inter-related to some extent with respect to load transfer, each technique exhibits advantages, disadvantages and limitations. The following sub-sections address each technique separately, under the following main headings:

- description
- controlling parameters for design
- equipment and offshore support
- inspection/maintenance
- timescales
- background research
- previous offshore experience

II 8.2 STRESSED MECHANICAL (FRICTION) CLAMPS

II 8.2.1 <u>Description</u>

A stressed mechanical clamp, otherwise known as a steel-to-steel friction clamp, comprises two or more segments of closely fitting stiffened saddle plates, stressed directly onto a tubular section by means of long studbolts. The strength of a mechanical connection is obtained from the steel to steel friction which is developed by means of external bolt loads which lead to compressive forces normal to the tubular/clamp saddle interface. Therefore the strength is dependent on the magnitude of the normal force and the effective coefficient of friction between the two steel contact surfaces. The clamp saddles are stiffened to ensure that studbolt loads can be carried without distress to the saddle itself or the tubular member. Figure 8.2.1 illustrates the various components of a mechanical clamp.

Stressed mechanical clamps rely on close tolerance steel-to-steel contact between the tubular member and the clamp saddles and therefore offer minimal translational or angular tolerances. Local yielding of a tubular member to make it conform to the saddle shape is not normally detrimental; nevertheless, stressed mechanical clamps require extremely accurate offshore surveys of the contact zone. Furthermore, very tight tolerances are required for the fabrication of the clamp. For these reasons, stressed mechanical clamps are not recommended for the strengthening and repair of tubular joints. However, they can be used for connecting new members, or for the strengthening and repair of intact and damaged members. Examples of such uses are shown in Figure 8.2.2.

II 8.2.2 <u>Controlling Parameters for Design</u>

The strength of a mechanical clamp is derived from friction. Therefore the strength is a function of :

- surface contact force which in turn is a function of bolt load
- surface condition of the steel surfaces at the interface
- bolt geometry and stiffness
- existing tubular member geometry and stiffness
- relative stiffness of clamp and tubular member
- connection length and number of effective friction surfaces

The major advantage of a mechanical clamp is that large forces can be transferred through friction over a short clamp length. This is particularly useful when space is limited. However, an important factor which governs the overall clamp length is the ability of the existing tubular member to resist the hoop buckling forces which are created as a result of studbolt tensioning. In particular, there is a small length of chord between the saddles which is completely unsupported and therefore prone to buckling. Member grouting (Section II 9.1) may be useful in this context.

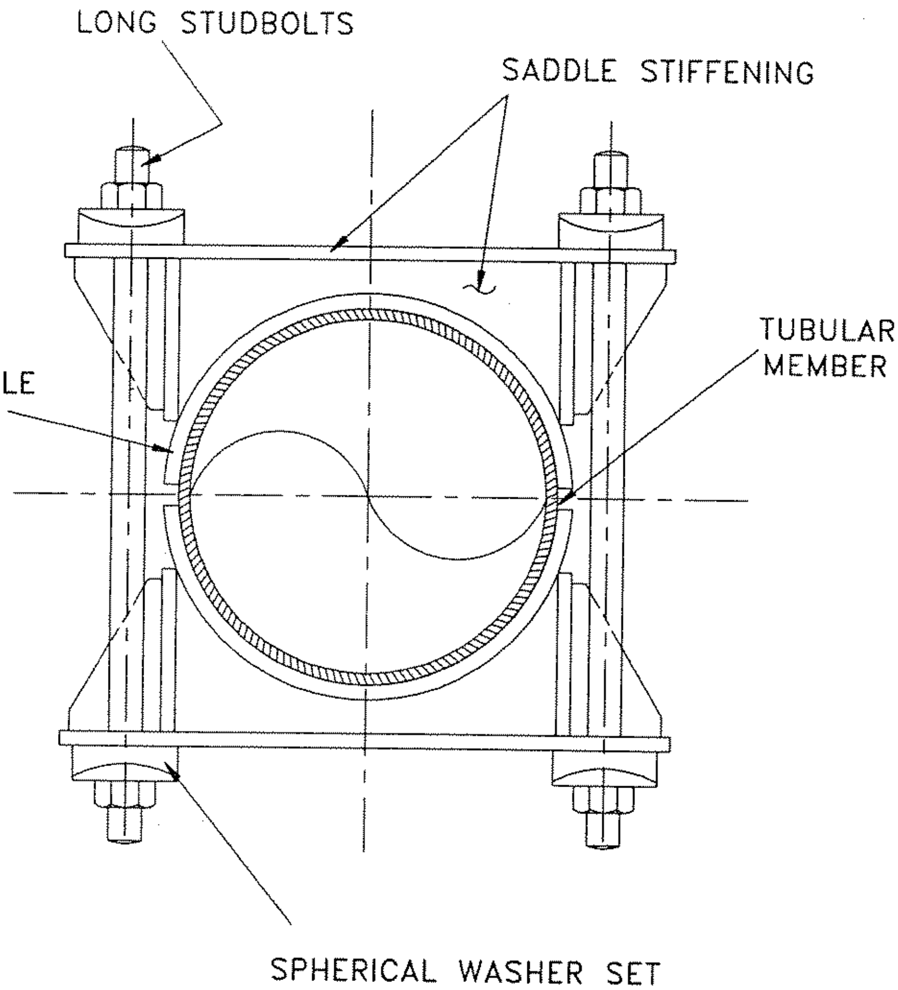
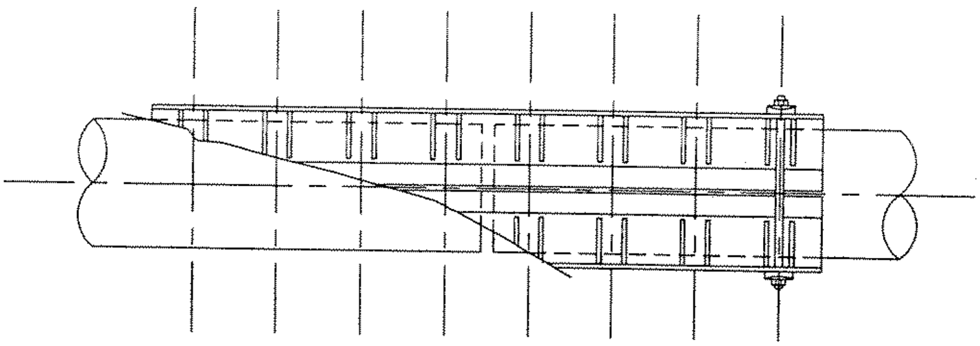
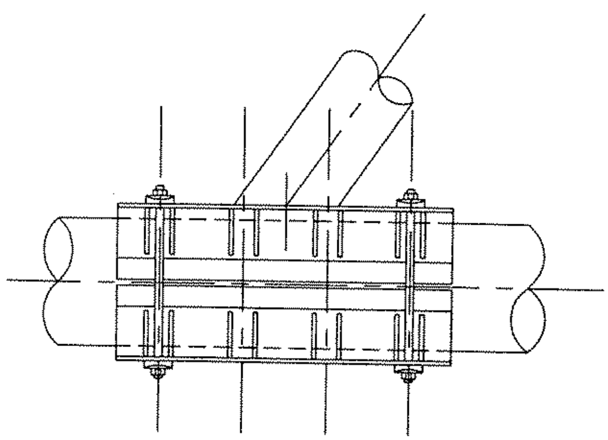


Figure 8.2.1: Typical stressed mechanical clamp



a) Connection of two members.



b) Attachment of a new brace.

Figure 8.2.2: Some applications of stressed mechanical clamps

Water depth (hydrostatic pressure) could be a governing factor in the design of steelwork where large areas of flat plate create watertight compartments. Hence, hydrostatic pressure may govern plate design unless the pressure is relieved through provision of holes in the otherwise enclosed chambers. Where flooding of chambers is allowed, consideration has to be given to internal corrosion protection.

Fatigue calculations for bolts and steelwork follow conventional procedures.

II 8.2.3 Equipment and Offshore Support

For diver-installation of a stressed mechanical clamp, the following equipment and personnel are required offshore:

- crane for lift purposes
- rigging for installation purposes
- underwater cutting and grinding equipment, if obstructions have to be removed
- underwater cleaning equipment
- studbolt tensioning equipment
- diving spread and divers
- monitoring equipment (eg. video/cameras)

II 8.2.4 Inspection/Maintenance

Stressed mechanical clamps should be inspected periodically to ensure continued satisfactory performance. In the absence of platform-specific requirements, it is usual to recommend an annual general visual inspection of all clamp steelwork and studbolts. A more detailed inspection should be performed within the first year following installation, and thereafter at regular intervals in accordance with the inspection philosophy for the platform. These inspections should check for:

- tension and corrosion of studbolts,
- presence of CP continuity straps (if used),
- usage of sacrificial anodes,
- crevice corrosion at the clamp/tubular member interface.

Dependent on predicted fatigue lives, MPI on critical welds should be performed.

II 8.2.5 Timescales

Installation timescales for stressed mechanical clamps vary depending on the complexity of the clamp (for instance, number of clamp segments), space limitations and water depth of the repair site. Two examples selected from literature, indicate installation times of two and six days. When offshore surveys, preparatory work like member cleaning and rigging are taken into account, the total installation timescales may approach one month.

II 8.2.6 Background Research

Between 1981 and 1984, significant research[8.1,8.2] was conducted into stressed mechanical clamps. The research can be summarised as follows:

(i) Eighty-four elastic and ultimate load tests encompassing

- various clamp length to tubular member diameter ratios
- various tubular member radial stiffness ratios
- various stiffness values for long studbolts
- studbolt load measurements

(ii) Three large scale elastic and fatigue tests for axial and OPB loads.

The data permits an expression for the slip strength of such clamps to be formulated. Most data relate to shot blasted surfaces, though members having mill scale and coal tar epoxy coatings have also been tested.

II 8.2.7 Previous Offshore Applications

Over the past twenty years there have been numerous applications of stressed mechanical clamps worldwide. Their use has been made to repair/strengthen jacket components damaged due to boat impact, fabrication flaws, fatigue cracking and corrosion.

II 8.3 UNSTRESSED GROUTED CLAMPS/SLEEVE CONNECTIONS

II 8.3.1 Description

An unstressed grouted clamp or sleeve connection comprises sleeves which are placed around a tubular member or joint with the annular space so created filled with grout. The sleeves may be split, as in an unstressed grouted clamp, or continuous as in a pile/sleeve connection. For split sleeves, short bolts are generally specified and these bolts are tightened prior to injection of grout into the annular space between the clamp and the existing tubular member. Figures 8.3.1 and 8.3.2 show typical details of an unstressed grouted clamp and sleeve connections.

The bond and interlock between the grout/steel interface provides the only means of load transfer between the tubular member and the clamp. Although bond and interlock may be sufficient in certain conditions, it is often necessary to substantially increase the length of the clamp to generate sufficient load transfer capacity. The provision of shear keys (usually in the form of weld beads) can increase the clamp capacity, but the need for underwater welding may render this option prohibitively expensive.

Unstressed grouted clamps and connections offer a versatile means for strengthening or repair of tubular joints and members since they require less accurate offshore surveys than methods described earlier. Both angular and translational tolerances can be readily accommodated.

As shown in Figure 8.3.3, unstressed grouted clamps may be used as follows:

- to strengthen or repair an existing tubular joint subjected to static and/or fatigue loads,

- to facilitate the attachment of a new member to the structure.

Some applications of sleeve connections are shown in Figure 8.3.4, and can be described as follows:

- to facilitate the attachment of a new member to the structure by providing length and fit-up adjustment. This can be accomplished by either a retractable sleeve which is slid over from one segment to the other segment of the new member, see Figure 8.3.4(a), or by a telescopic sleeve whereby the member is installed into location as a single piece, see Figure 8.3.4(b). Both member and sleeve are new fabrications, and the efficiency of the connection can be greatly enhanced by the provision of shear keys.

- to strengthen members by the use of a steel 'bandage' in order to enhance stability against local or overall buckling, see Figure 8.3.4(c).

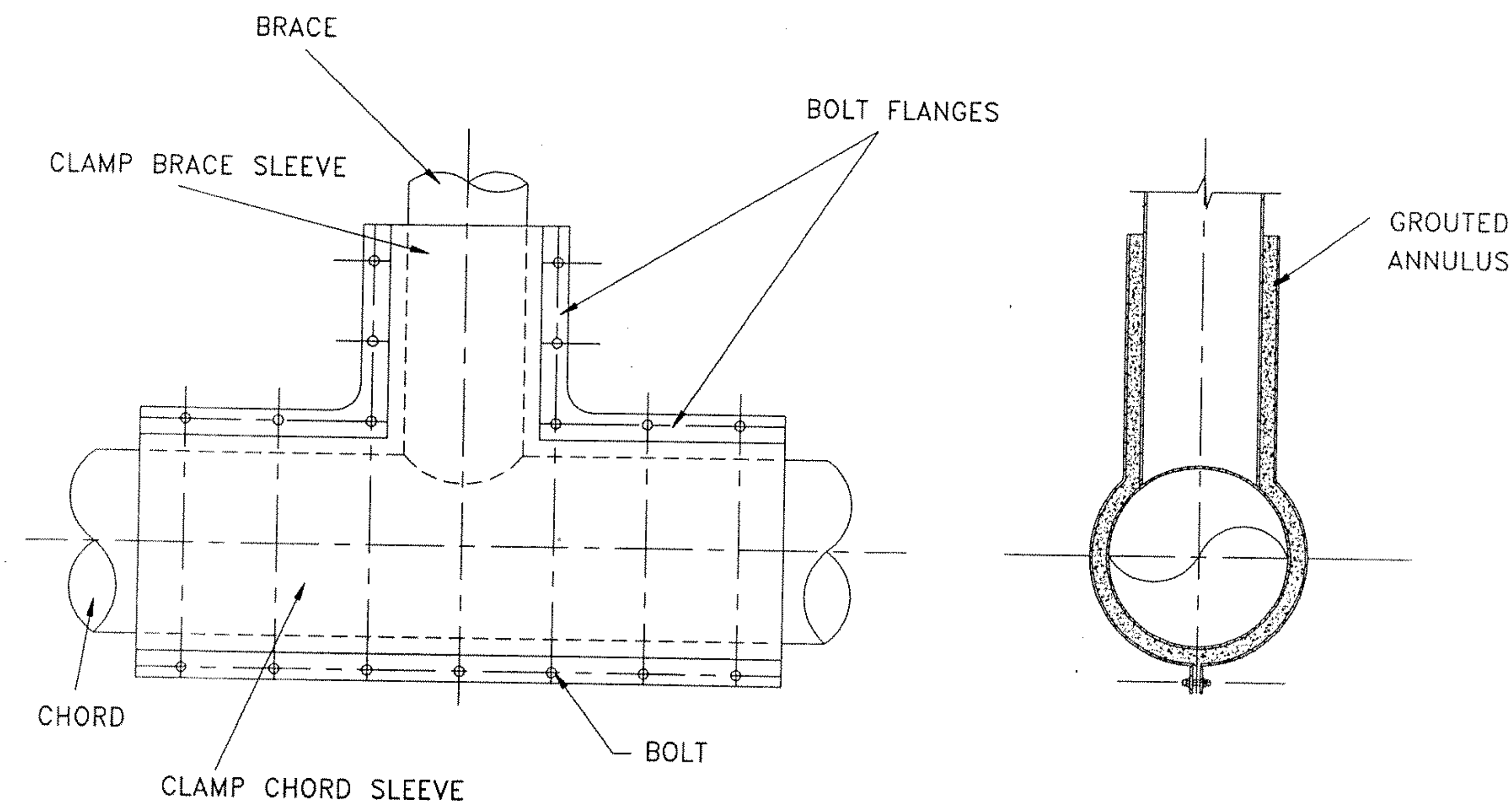
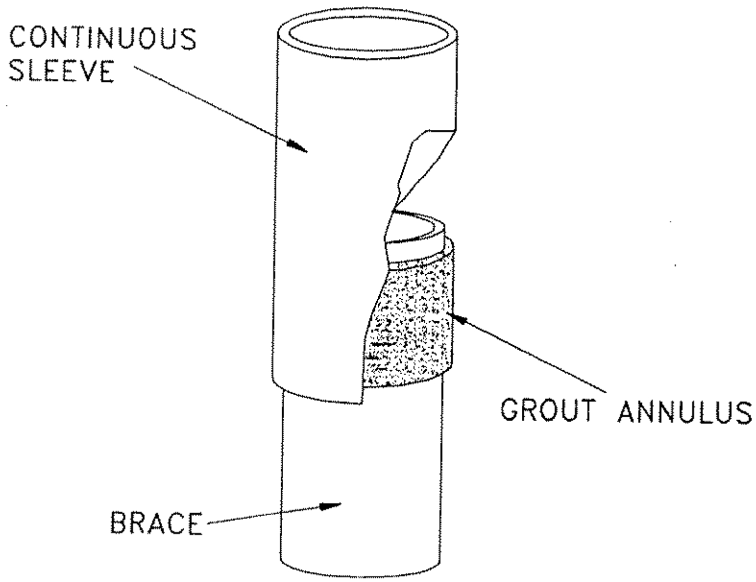
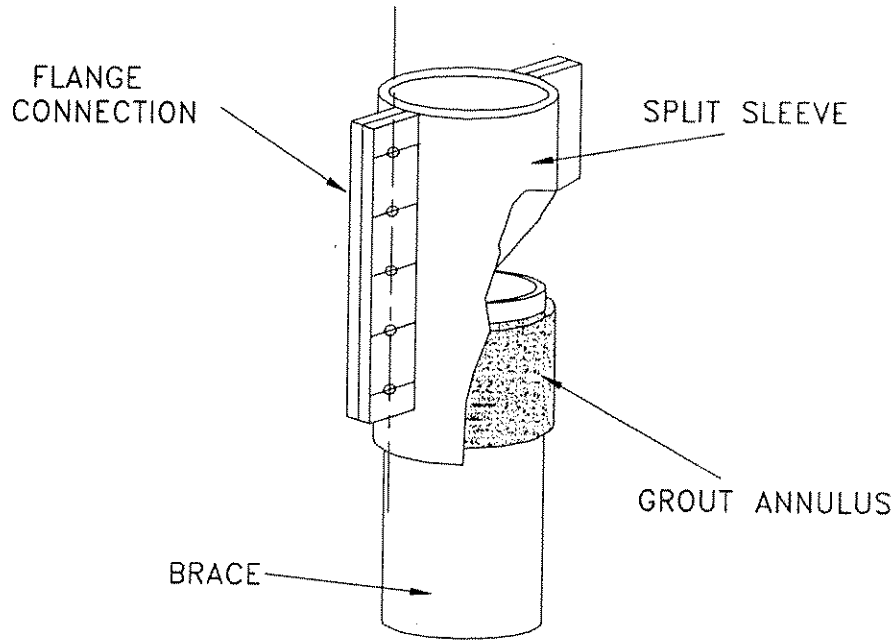


Figure 8.3.1: Typical unstressed grouted clamp



a) Continuous sleeve grouted connection.



b) Split sleeve grouted connection.

Figure 8.3.2: Examples of typical unstressed grouted sleeve connections

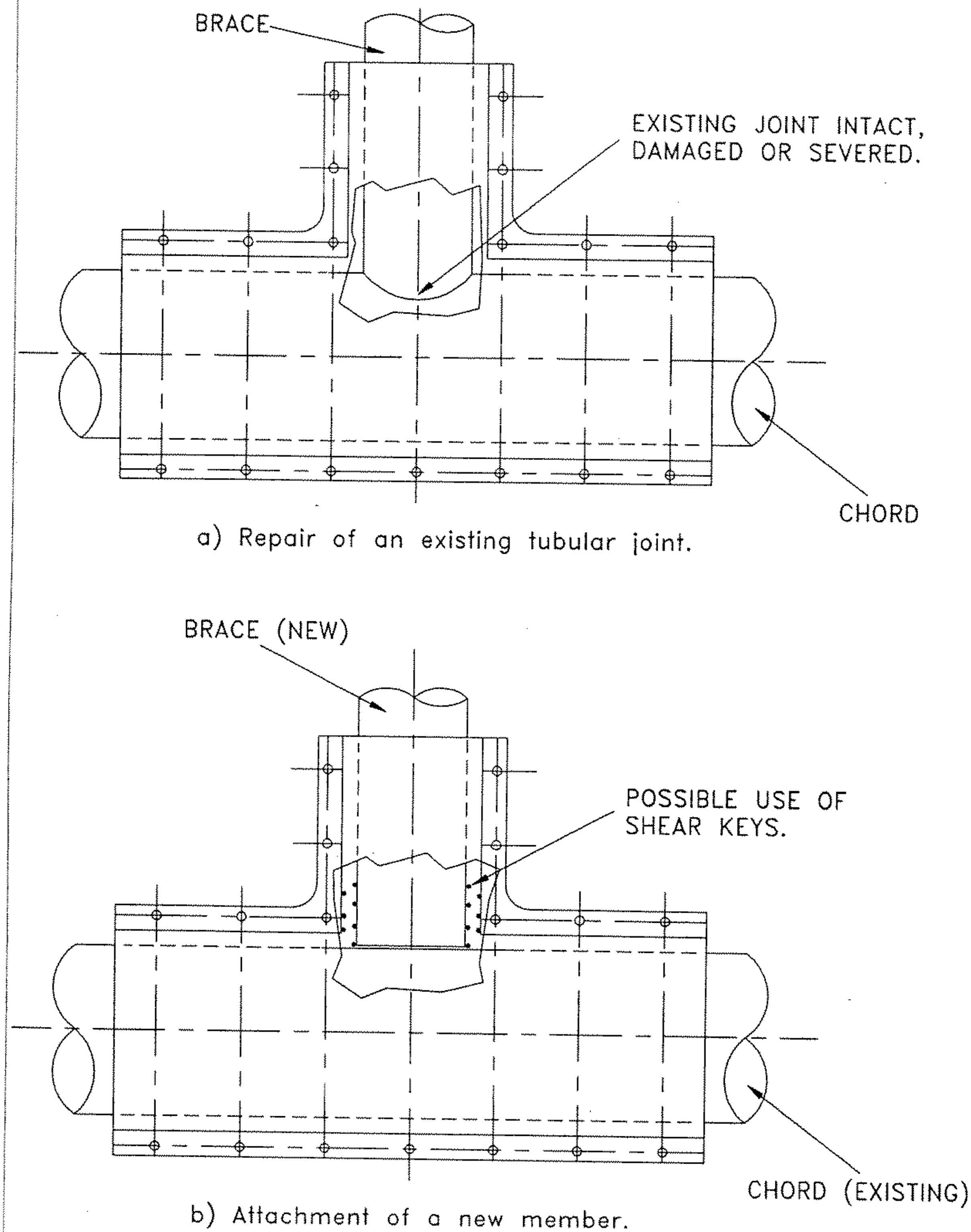


Figure 8.3.3: Some applications of unstressed grouted clamps

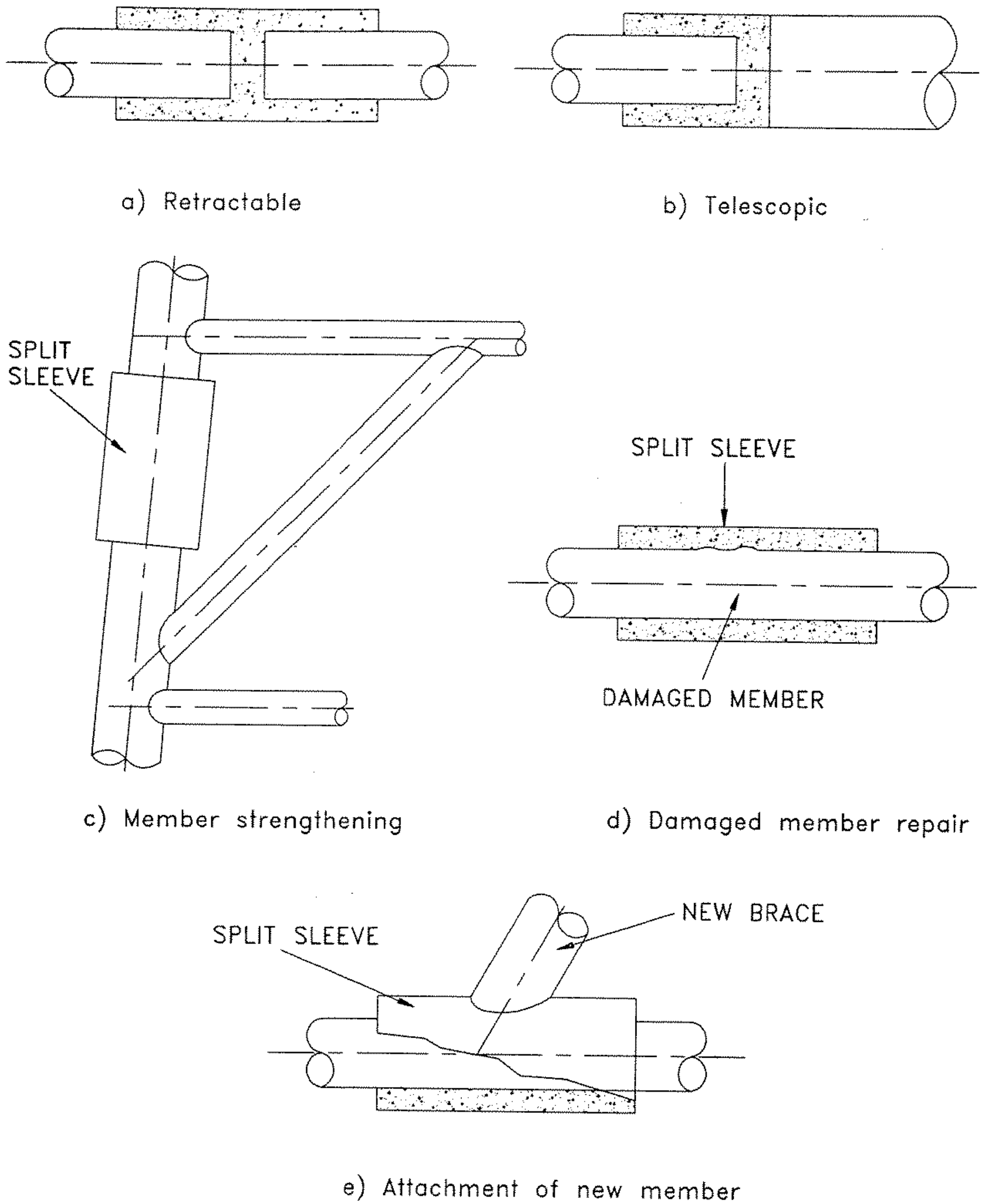


Figure 8.3.4: **Some applications of unstressed grouted sleeve connections**

- to strengthen or repair members which have sustained dents, punctures, corrosion or other damage, see Figure 8.3.4(d).

- to facilitate the attachment of a new member to the structure by creating a new joint, see Figure 8.3.4(e).

The first noted application above will normally be formed using a continuous sleeve. The remaining applications will be formed using split sleeves, to enable installation around existing tubulars.

II 8.3.2 Controlling Parameters for Design

The strength of an unstressed grouted clamp/sleeve connection is obtained from a combination of chemical bond, friction and mechanical interlock. Mechanical interlock may arise from micro and macro geometric imperfections, or from the addition of shear keys, eg. weld beads.

The following factors affect the strength of unstressed grouted clamps/sleeve connections:

- grout compressive strength and elastic modulus
- tubular member radial stiffness
- surface condition
- shear key geometry
- length for load transfer
- early age movements during the grout curing cycle
- grout pressure

Loading regime and the availability of space are dominant in deciding the suitability of unstressed grouted clamps/sleeve connections. Without the use of shear keys, the required connection lengths may be unacceptably large. Reduction in connection lengths may be achieved by the use of weld beads, friction welded studs, or other forms of shear key. However, the costs associated with the provision of such shear keys may be prohibitively high.

II 8.3.3 Equipment and Offshore Support

For diver-installation of an unstressed grouted clamp/sleeve connection, the following equipment and personnel are required offshore:

- crane for lift purposes

- rigging for installation purposes

- underwater cutting and grinding equipment, if obstructions have to be removed

- underwater cleaning equipment

- torque/tension studbolt equipment, except for continuous sleeve connection

- grouting spread

- diving spread and divers

II 8.3.4 Inspection/Maintenance

Once the connection steelwork has been installed and the bolts tightened in the case of clamps and split sleeve connections, grouting operations are carried out in which grout quality is assured through rational sampling and density measurements. Post-installation inspection is usually confined to a general annual visual inspection to identify that:

- no slippage of the connection is evident

- all studbolts are in place

- grout seals are intact

- CP potential levels are retained and that continuity straps (if any) are intact

- sacrificial anodes (if any) are sufficient.

Bolt tension checks are sometimes called for and these can be achieved by re-stressing or through checks of load indicating devices (if installed).

II 8.3.5 Timescales

Installation timescales for unstressed grouted clamps/sleeve connections vary depending on the complexity of the clamp and the number of pieces in which the clamp is installed. Furthermore, the total number of studbolts which have to be tensioned will influence the timescales.

As an example, two X joints strengthened using unstressed grouted split sleeve connections have been reported[8.3] with the following dive time analysis:

Activity	Dive Time (%)
Survey	11
Cleaning	11
Shear connector welding:	
Habitat installation & removal	30
Welding	18
Sleeve installation and bolting	20
Grouting	5
Inspection	5
TOTAL	100

It is important to note that curing time should be allowed in the programme to ensure that the unstressed grouted clamp/sleeve connection is not subjected to undue loading before the grout has attained sufficient strength. In certain instances, temporary clamps may be necessary.

II 8.3.6 Background Research

Numerous tests have been conducted over the past three decades. The original research thrust in this field was on continuous sleeve connections (pile/sleeve) and this has led to a large present-day database. Several codified guidance formulations have been developed in the last decade and these are widely used by the offshore industry. The validity and application limits of these formulations continue to be debated and a number of joint-industry funded projects are ongoing.

Some of the data were generated specifically for repair geometries; pile/sleeve connection geometries being somewhat different. This has allowed the extrapolation of data generated for pile/sleeve connections for application to geometries more akin to repair scenarios. An examination of the data shows that all the controlling parameters have been covered but that relatively few results exist for split sleeve plain pipes. However, given the relatively uncommon usage of this connection in predominantly axial load situations, that lack of data is not a serious handicap. More work is required to examine the effects of early age movements.

II 8.3.7 Previous Offshore Applications

Several applications of unstressed grouted clamps and sleeve connections are evident. Pile/sleeve connections for numerous jackets make use of this technique. For repair/strengthening, unstressed grouted clamps/sleeve connections have often been used to overcome fatigue cracks and damaged members due to boat impact.

II 8.4 STRESSED GROUTED CLAMPS

II 8.4.1 Description

A stressed grouted clamp is formed when two or more segments of oversized, strengthened saddle plates are stressed by means of long studbolts onto a tubular member after grout has been injected and allowed to cure in the annular space between the clamp and the tubular member, see Figure 8.4.1. This form of clamp is a hybrid between a stressed mechanical clamp and an unstressed grouted clamp. The strength of a stressed grouted clamp is obtained from a combination of 'plain-pipe' bond and grout/steel friction developed as a result of compressive force normal to the grout/tubular member interface.

Stressed grouted clamps offer the benefits of stressed mechanical clamps of high strength-to-length ratio, and the benefits of unstressed grouted clamps of the ability to absorb significant tolerances. This form of clamp is therefore very popular. Stressed grouted clamps can be used for similar applications to those defined for stressed mechanical clamps and unstressed grouted clamps (Sections II 8.2.1 and II 8.3.1).

II 8.4.2 Controlling Parameters for Design

The strength of a stressed grouted clamp is derived from a combination of chemical bond, mechanical interlock and friction at the grout/steel interface. The strength of such a clamp is therefore a function of the following:

- surface contact force due to studbolt load
- tubular member surface condition
- number of active friction faces
- studbolt geometry and stiffness
- existing member geometry and stiffness
- total clamp length

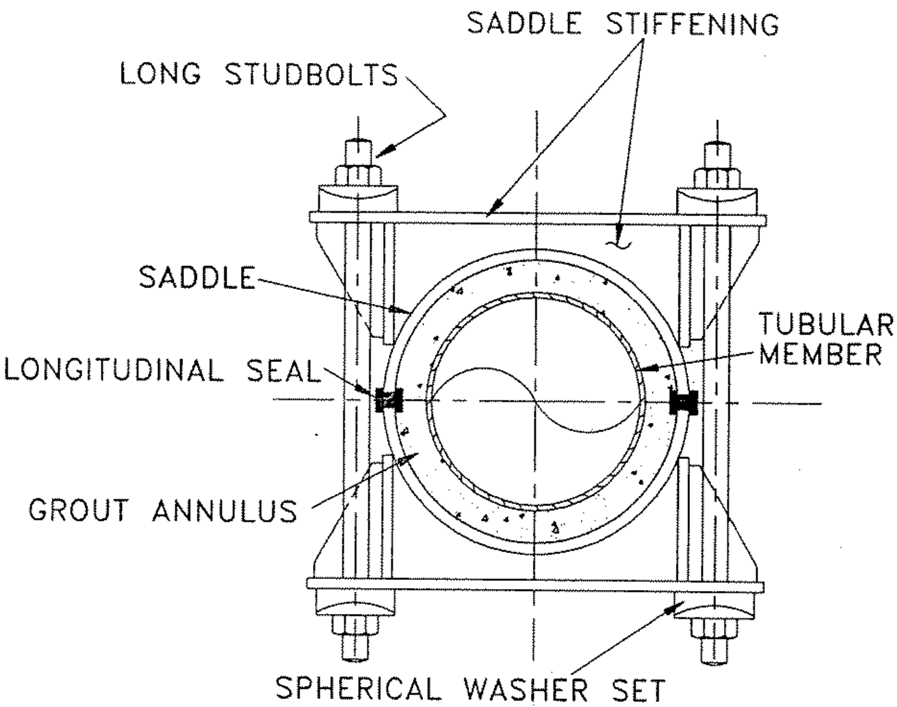


Figure 8.4.1: Typical stressed grouted clamp

An important factor which governs the clamp length relates to the ability of the existing tubular member to resist hoop stresses which are created as a result of studbolt loading. Member grouting may be useful in this context.

Water depth (hydrostatic pressure) could be a governing factor in the design of steelwork where large areas of flat plate create watertight compartments. Hence, hydrostatic pressure may govern plate design unless the pressure is relieved through provision of holes in the otherwise enclosed chambers.

Fatigue calculations for bolts and steelwork follow conventional procedures.

II 8.4.3 <u>Equipment and Offshore Support</u>

For diver-installation of a stressed grouted clamp, the following equipment and personnel are required:

- crane for lift purposes
- rigging for installation purposes
- underwater cutting and grinding equipment, if obstructions have to be removed
- underwater cleaning equipment
- torque/tension studbolt equipment
- grouting spread
- diving spread and divers
- monitoring equipment

II 8.4.4 <u>Inspection/Maintenance</u>

Grout quality is assured by taking samples of the grout immediately prior to pumping. Further, grout density is monitored from the outlet points on the clamp.

It is normal to carry out periodic inspections of the clamp to ensure that it is performing in a satisfactory manner. In the absence of a specific platform inspection programme, it is recommended that a general visual inspection of all clamp steelwork and studbolts be carried out each year. A more detailed inspection is recommended within the first year following installation and thereafter at regular intervals in accordance with the inspection philosophy for the platform. These inspections should focus on studbolt corrosion and loads, seals, CP potentials and wastage of sacrificial anodes. MPI of critical welds should be performed on the basis of predicted fatigue lives.

II 8.4.5 Timescales

The operations involved in the installation of a stressed grouted clamp can be summarised as follows:

- removal of obstructions (if any)
- cleaning and rigging at repair site
- installation of clamp, seal activation and installation of studbolts
- Grout pumping and cure time
- studbolt tensioning
- demobilisation

As an example, eight stressed grouted clamps were fully installed on a platform in the Gulf of Mexico in a total of 18 days. About one-third of this time was used in removal of obstructions whilst cleaning, clamp installation, grouting and studbolt tensioning took about equal time. It is important to recognise, however, that timescales are very dependent on the complexity of the clamp, access at the repair site, water depth at the repair site and environmental conditions that may severely limit dive time and greatly influence weather downtime.

II 8.4.6 Background Research

The research effort underlying the design of stressed grouted clamps has been through numerous tests as follows:

(i) over 75 elastic and ultimate load tests on large scale connections, encompassing:

- various clamp length to tubular member diameter ratios
- various tubular member radial stiffness ratios
- various stiffness values for long studbolts
- various short term and long term grout strengths
- studbolt load monitoring

(ii) 3 large scale elastic and fatigue tests on T clamps under axial or OPB cyclic loads.

In terms of ultimate slip tests, there are surprisingly few data given the popularity of this form of clamp. Not only is the tested number of such clamps small, but also rather limited geometric ranges have been studied. Further work is required in this important area (further discussion may be found in Part IV, Section IV 4.4.8).

II 8.4.7 Previous Offshore Applications

Stressed grouted clamps can be viewed as representing the strength advantages of stressed mechanical clamps with the tolerance advantages of unstressed grouted clamps; it is therefore not surprising to note that stressed grouted clamps are the most popular form of clamp concept deployed in industry.

II 8.5 STRESSED ELASTOMER-LINED CLAMPS

II 8.5.1 Description

Stressed elastomer-lined clamps are very similar to stressed mechanical clamps, except that an elastomer lining is bonded to the inside faces of the clamp saddle plates, see Figure 8.5.1. In general, the liner is made up of solid polychloroprene (neoprene) sheet. The strength of an elastomer-lined clamp is derived from external bolt loads which lead to compressive forces normal to the interface of the elastomer-lined saddle and the tubular member. The strength is therefore dependent on the magnitude of the normal force and the effective coefficient of friction between the liner/steel interface. The use of an elastomer offers a degree of translational and angular tolerance, thus removing the need for very accurate offshore surveys as required for stressed mechanical clamps.

Elastomer-lined clamps have not been used for primary structural repairs, because of concerns that the flexibility of the liner may reduce the efficiency of the repair system. The use of this type of repair scheme has been limited therefore to secondary components where stiffness is not critical to its effectiveness. Typical examples of the use of an elastomer-lined clamp are to seal holed caissons and for stub connections to appurtenances.

II 8.5.2 Controlling Parameters for Design

The controlling parameters for design are the same as those for stressed mechanical clamps (Section II 8.2.2). In addition, the liner material properties, such as hardness and thickness, will affect the overall performance in terms of friction capacity and stiffness. Elastomers, even when confined, are significantly less stiff than steel, and therefore, the load distribution for a repair

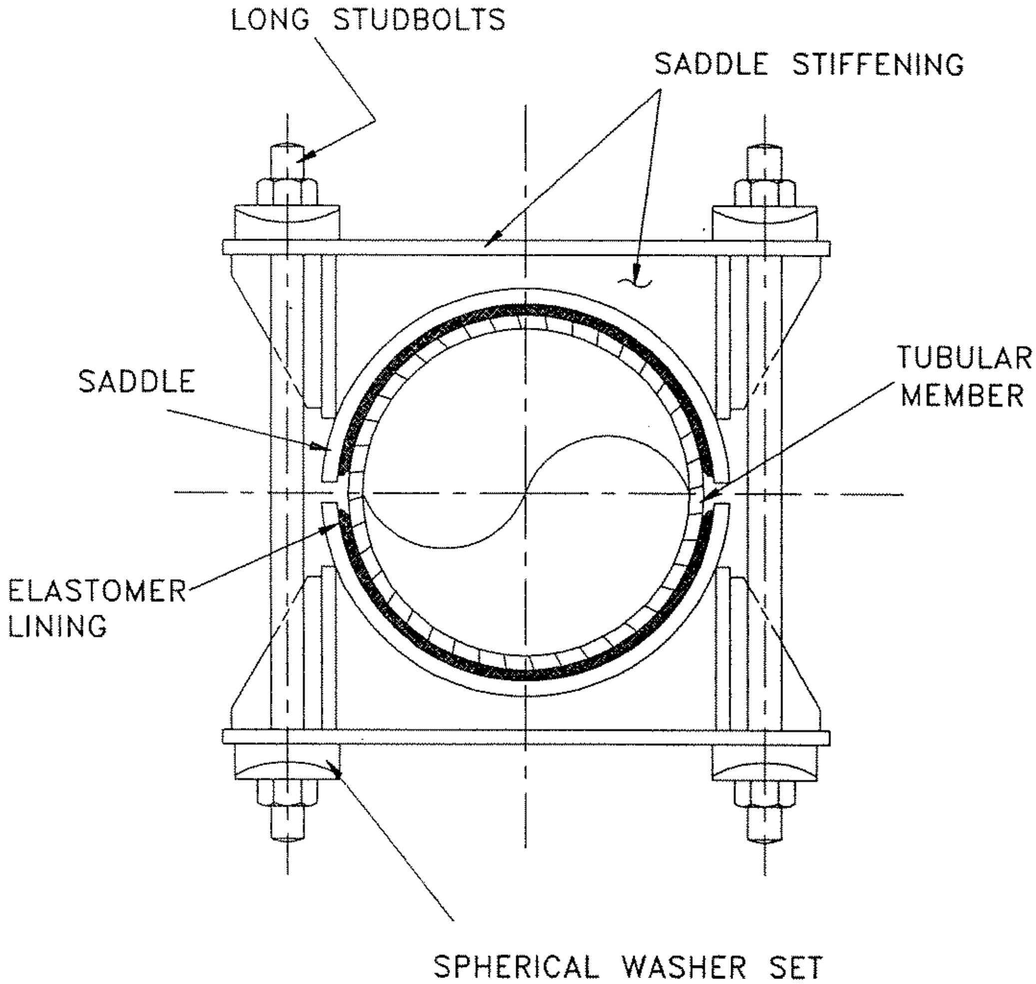


Figure 8.5.1: Typical stressed elastomer-lined clamp

scheme of this form requires careful evaluation.

The relatively low stiffness of the liner also impacts on the studbolt design giving rise to significant studbolt load fluctuations. Fatigue of studbolts therefore requires careful consideration. Studbolt losses occur as a result of elastomer relaxation, and therefore allowances should be made for this during design.

II 8.5.3 Equipment and Offshore Support

For diver-installation of a stressed elastomer-lined clamp, the following equipment and personnel are required offshore:

- crane for lift purposes
- rigging for installation purposes
- underwater cutting and grinding equipment, if obstructions have to be removed
- underwater cleaning equipment
- studbolt tensioning equipment
- diving spread and divers
- monitoring equipment

II 8.5.4 Inspection/Maintenance

Stressed elastomer-lined clamps should be inspected periodically to ensure continued satisfactory performance. In the absence of platform-specific requirements, it is usual to recommend an annual general visual inspection of all clamp steelwork and studbolts. At the end of the first year of service, and at regular intervals in accordance with the inspection philosophy of the platform, it is also recommended that studbolt tension loads be checked and, if necessary, re-stressed. An inspection for corrosion should be performed every second year. This inspection should check for:

- corrosion of studbolts,
- usage of sacrificial anodes.

Dependent on predicted fatigue lives, MPI on critical welds should be performed.

II 8.5.5 Timescales

Installation timescales for stressed elastomer-lined clamps vary depending on the complexity of the clamp (for instance, number of clamp segments), space limitations and water depth of the repair site. In general the timescales are similar to those for stressed mechanical clamps.

II 8.5.6 Background Research

The coefficient of friction between elastomer lining and steel has been examined by manufacturers of the elastomer. General purpose tests have been conducted on flat steel plate samples. A substantial literature search has indicated that no research has been conducted specific to clamps. The designs to-date have therefore utilised very conservative values for the coefficient of friction between elastomer lining and steel.

II 8.5.7 Previous Offshore Applications

Stressed elastomer-lined clamps have not been used commonly to strengthen/repair primary components of steel jacket structures. They have, however, been used in the past to strengthen/repair caissons and stub connections for appurtenances.

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8.2 Fern D T et al. 'Bolted Repair of Tubular Joints'. Paper 23 of Second Integrity of Offshore Structures Conference, Glasgow, 1981.

8.3 Tebbett I E and Robertson D A. 'Novel Underwater Strengthening System for Tubular Joints'. Paper OTC 4110. Offshore Technology Conference, May 1981.

II 9 BACKGROUND AND DESCRIPTION OF GROUT FILLING

II 9.1 GROUT FILLING OF MEMBERS

II 9.1.1 Description

Grout filling a tubular member increases both its cross-sectional (squash) strength and its overall strength (stability). Therefore, it provides a technique by which the member strength is increased without increasing the member diameter. The additional mass of the member, the additional earthquake load due to increased inertia and the extra stiffness of the member and tubular joints along its length must all be considered as direct implications of grout filling.

Design methods for concrete filled tubulars have been available for some time but only recently has work been carried out on grout filled tubulars, see Parts III and IV. The process provides a relatively easy way of strengthening members but certain precautions need to be taken; these are addressed in this section and Parts III and IV.

A member may need strengthening for one or more of the following reasons:

- Increased topside load
- Increased wave loading or potential ship impact loading
- Excessive corrosion reducing the wall thickness
- Code upgrading
- Member damage.

Grout filling of the steel tubular members may provide the solution in whole or in part to the above problems.

II 9.1.2 Controlling Parameters

This technique has proven benefits for compression loaded members with or without bending. Tests indicate that for slender or thick walled tubes there is little or no benefit to be had from this technique. This is because the grout is most efficient in a) increasing the squash load of the tube rather than increasing the section's radius of gyration, and b) preventing local wall buckling.

It is important that grouting procedures are developed which completely fill the tubular, as small voids close to the tubular joints at each end of the member will render the solution unworkable. This is because the only way that load can be transferred to the grout is by direct bearing on the grout column; sufficient load cannot be relied upon to be transferred in bond between the tube inner wall and grout (tests show that a progressive mode of failure acts). Void formation is also a potential problem at internally ring stiffened joints or at joints with expanded can diameters.

This technique applies to loads which act on the brace after the grout has cured. Any dead, or other loads, in the brace at the time of grouting will remain in the steel skin.

Large volumes of grout may be used in this process which can generate excessive heat whilst setting. The mix design should pay attention to this and ensure that the heat generated is not excessive as grout degradation will result.

II 9.1.3 Inspection

It is important to ensure:

- that grouting procedures are developed and tested which ensure complete grouting
- that the grout mix is tested to ensure that significant grout bleed does not occur
- that offshore inspection techniques are capable of ensuring that complete filling has occurred (eg. ultrasonics or probes).

No long-term maintenance is required.

II 9.1.4 Equipment and Offshore Support

The following equipment/personnel are required for offshore grout filling:

- Rigging
- Cutting/drilling equipment
- Divers to place grout inlets/outlets and assist
- Grouting operations (or ROV intervention if developed)
- Grouting spread
- Inspection/monitoring equipment.

II 9.1.5 Environmental Considerations

There are no special requirements relating to the environment with this technique.

There are no published data to assess the effect of this technique on the fatigue life of weldments along the length of a tube, or on the fatigue life of tubular joints at the end of a brace which has been filled.

II 9.1.6 Timescales

Based on an assessment of typical offshore timescales, a grouting operation should be achievable with 2-3 days offshore work.

II 9.1.7 Background Research

A significant amount of research in this field has been conducted, and a review of this research appears in Part IV. Much of the data relates to small scale test specimens.

II 9.1.8 Previous Applications

Grout filling has been adopted in a number of cases, and these are noted in the Appendix.

II 9.1.9 Summary

Tests results indicate that grout-filled tubulars are sensitive to the quality of the grouting operation. Offshore procedures and onshore trials should therefore reflect this sensitivity.

The technique is of demonstrable benefit to tubes with low L/D ratios and high D/T ratios and for members subject to bending and/or compression; there are no test data on tension loaded, grout filled, members, although benefit in this respect would be expected to be restricted.

II 9.2 GROUT FILLING OF JOINTS

II 9.2.1 Description

Grouted joints have the chord member fully filled with a cementitious grout material. Double-skin joints are those in which the chord member contains a pile and a grouted annulus. To provide additional strength, the pile can also be filled with grout.

A grouted joint is shown in Figure 9.2.1 and a double-skin joint is illustrated in Figure 9.2.2.

Filling tubular chord members with a cementitious material results in the efficient use of materials in applications most suited to their mechanical properties. The cementitious material is contained and therefore greater strength and ductility is achieved. The steel tubular is the containment medium

$\alpha = 2L/D$

$\beta = d/D$

$\gamma = D/2T$

$\tau = t/T$

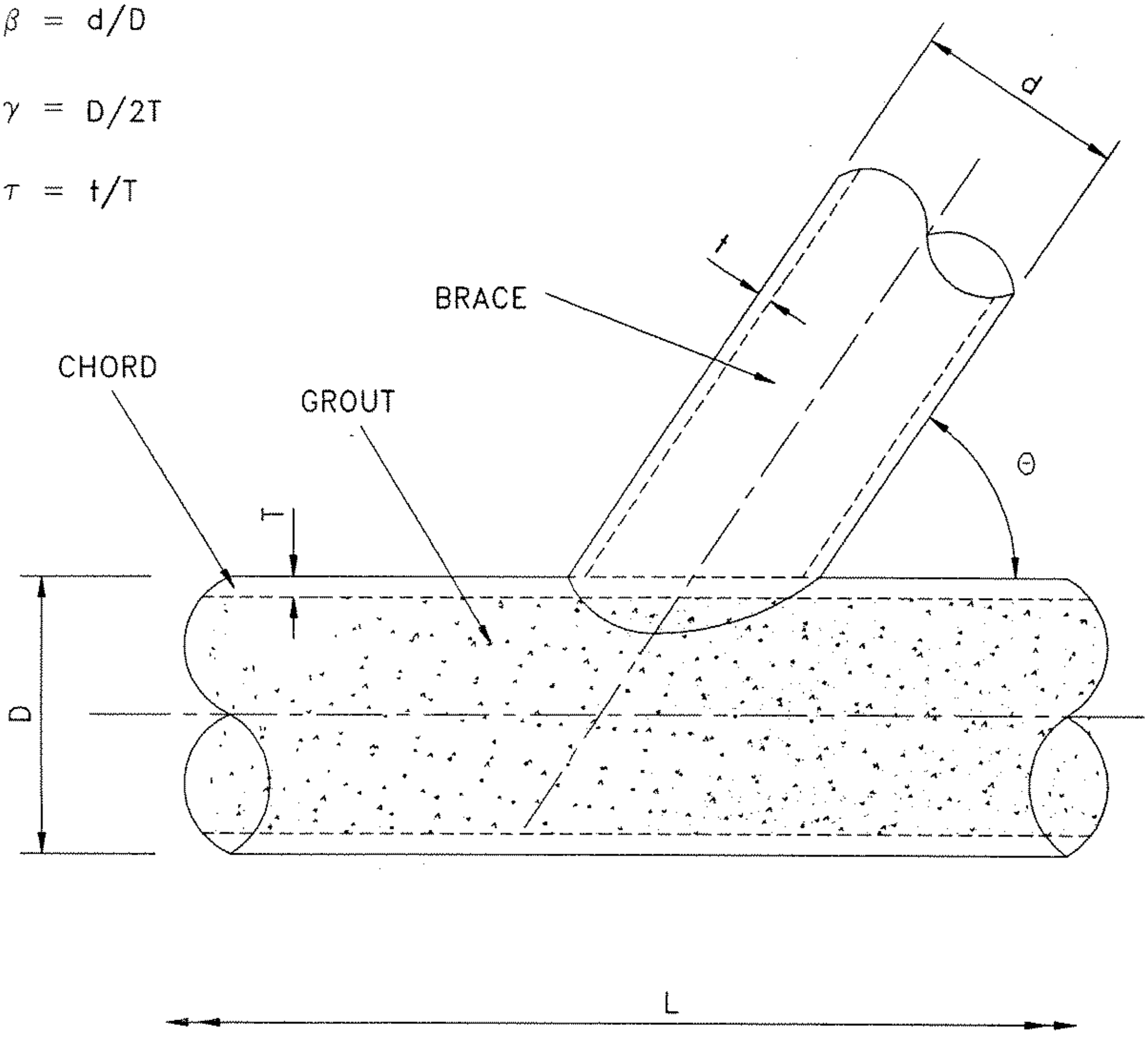

Figure 9.2.1: Details of a grouted joint

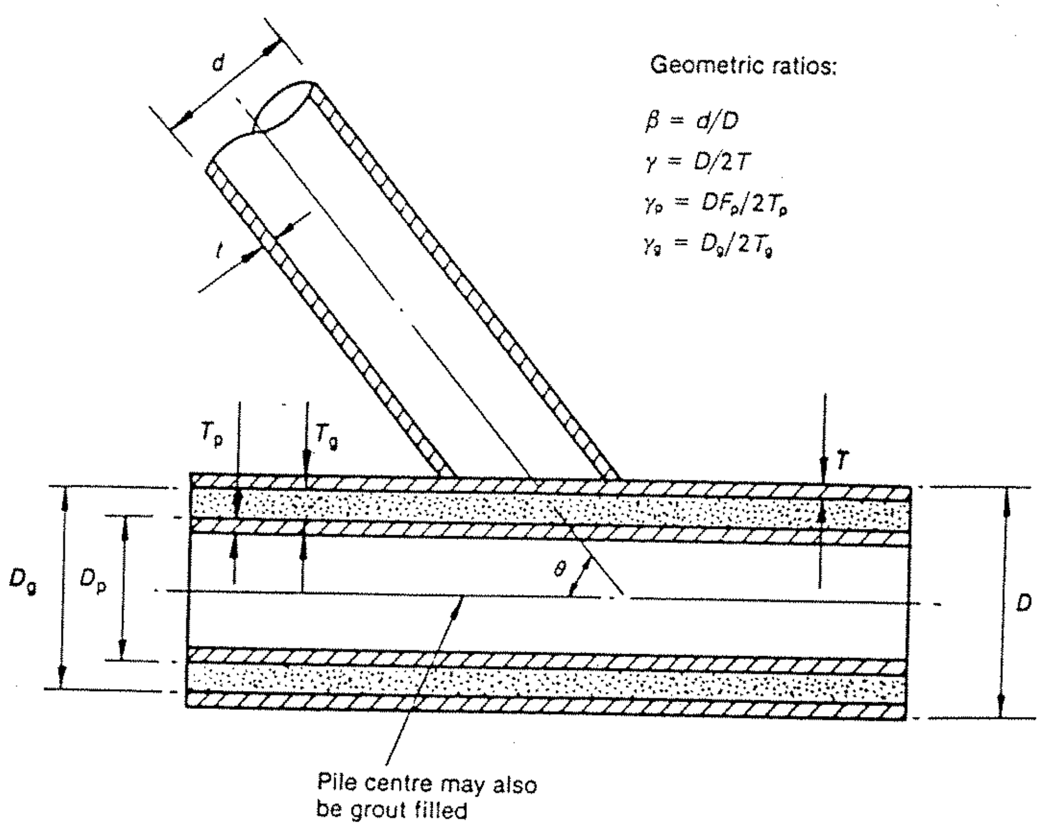

Figure 9.2.2: **Details of a double-skin joint**

and is therefore predominantly subjected to hoop loads, and the cementitious filling minimises any tendency for buckling of the steel shell. Grouting technology is well proven and offshore grouting works can be executed with confidence.

It is relatively easy to fill a chord member over its full length where the member is other than a jacket leg, as this avoids the necessity of cutting windows in the member to insert seals to localise the grout plug. The grout can be placed through 1 inch diameter inlets and outlets which may be drilled and tapped into the tubular wall. Jacket legs need only be filled up to the level which is required in view of the quantity of grout required.

II 9.2.2 Controlling Parameters

A number of technical benefits can be demonstrated through grout-filling of tubular joint chords, viz:

- The presence of the grout increases the radial stiffness of the chord member. The grout restricts local chord wall deformations, which leads to a reduction of deformation-induced bending stresses and associated SCFs.

- Any reduction in SCF implies an enhancement in fatigue life.

- The chord member bending stiffness is increased, resulting in a reduction of stresses at crown locations which are driven by the α ratio. The increased chord bending stiffness also implies that the capacity of large β ratio, grouted T/Y joints, subjected to axial loads, may not be limited by chord failure in the beam-bending sense.

- The grout severely restricts ovalisation of the chord cross-section, which indicates an increase in the capacity of grouted joints when compared with the ungrouted cases.

There is a paucity of data in grouted joints, and therefore specialist input is required to ensure safe compliance of the procedure described in Part III.

In many instances, the strength of a grouted joint may be limited by the strength of the incoming brace member. This places an absolute limit on the increase in strength available by this technique. Joints with low γ ratios and high β ratios may have restricted benefit from composite action.

Consideration should be given to the global response of a structure before grouting the entire length of a leg or other member. Grouting will stiffen the member, and may lead to additional load being attracted to the member which may overstress unstrengthened joints in its vicinity. Local grouting does not significantly change the stiffness of a member.

II 9.2.3 Inspection

Large volumes of grout may be used and, therefore, it is important that in the grout mix design to ensure that excessive heat is not generated by the exothermic cement hydration process. It is also necessary to ensure complete grouting, and to avoid grout bleed.

Following injection and setting of the grout, no further inspection is considered necessary as the grout is confined in a sealed environment which isolates it from attach.

II 9.2.4 Equipment and Offshore Support

To install a localised chord grout plug, the following equipment/personnel are required offshore:

- Rigging
- Cutting/drilling equipment
- Divers to install seals and assist with grouting operations
- Grouting spread
- Monitoring equipment.

II 9.2.5 Environmental Considerations

There are no special requirements relating to the environment with this technique.

II 9.2.6 Timescales

No published cases are available in the literature for this technique. However, based on experience of other operations and given good conditions, a grouting operation should be achievable in 3-4 days. This allows for installing grout bag seals, allowing seals to set and cure, and then grouting plug.

II 9.2.7 Summary

This is a relatively quick technique, and does not require large amounts of equipment or specialised techniques offshore. This technique is arguably the most cost-effective and technically-efficient technique available and, although specialist input to design is required, it is expected that this technique will find increasing application in the future.

II 10 BACKGROUND AND DESCRIPTION OF BOLTING

II 10.1 DESCRIPTION

In the following sections, the term 'bolt' will refer to a threaded fastener, either with an integral head of some manufactured geometry, or without (as in the case of a studbolt). The threaded fastener may screw into a threaded hole or into a nut.

A bolted joint is assumed to consist of three main elements:

- bolt, complete with nuts and washers
- fixing plate or structure, normally drilled with a clearance hole to accept the bolt
- parent plate or structure.

Bolts are, of course, used extensively offshore on topsides structural applications, but their subsea application is more restricted and is mainly for pipelines, as connections, flanges, and collars. Bolts are often incorporated into other repair methods, such as clamps, and may be used to introduce compressive stresses around defects[10.1].

The advantages of bolted repairs include:

- speed of application
- no delay in obtaining full strength
- ease of fabrication
- ready availability of components and bolts
- proven technology
- design using existing codes
- flexibility of use, including removal.

Bolted repairs are thus eminently suitable as temporary measures, to allow time for a more permanent job. They may also be attractive if access is difficult for other techniques such as hyperbaric or one-atmosphere dry welding. In principle, they are capable of being largely or completely deployed by ROV or other diverless methods, thereby minimising diver risk.

II 10.2 CONTROLLING PARAMETERS

The properties of a bolted joint depend on numerous factors, many of them interacting:

- role of the bolt (acting essentially as a shear pin or as the stressing component to induce frictional resistance)
- relative stiffnesses of bolt and components to be joined
- location and numbers of bolts
- nature of plate surfaces (planarity, friction coefficient etc)
- thread design and lubrication of thread
- preload
- properties of materials (galling, relaxation, corrosion resistance etc).

The design of the joint itself will be similar to normal onshore practice, with due allowance for the fatigue loading of offshore installations. Most of the differences from conventional design arise from the practical effects of the marine environment, to be discussed below.

Effective designs using bolts offshore must consider:

- fatigue
    - innate susceptibility of the bolting materials
    - effect of preload
    - effect of environment (corrosion fatigue?)
- effect of the design on bolt behaviour
    - bolt stress distributions (eg locations and numbers of bolts)
    - effect of the elasticity of other components (eg elastomers)
- bolt relaxation with time
- corrosion and hydrogen embrittlement (HE).

Three main factors affecting the response of the bolts to the above variables are: materials, preload, and thread manufacture.

- materials
    - static rupture behaviour

- creep/relaxation behaviour
- corrosion resistance
- if for subsea use, susceptibility to CP levels

- bolt preload

  - affects static joint behaviour (separation/loading)
  - affects fatigue behaviour

The value of the preload is itself affected by:

- method of application of preload

- friction (lubrication and galling)

- thread design and manufacture

  - cut or rolled threads
  - heat treatment
  - variability of bolts

Some of these factors will be detailed further below, but as it is not possible to give a comprehensive account of all these factors in this document, the reader is referred to standard texts such as Bickford [10.2]. In the following sections, attention focuses on factors differing substantially from conventional practice.

II 10.3 DESIGN CONSIDERATIONS

Offshore design codes do not cover bolting practice, but such repairs to topsides structures may be designed using onshore codes. However, these codes are not completely satisfactory for splash zone and underwater uses, and much more emphasis must be given to corrosion.

Most bolts used offshore conform to the usual onshore standards, and are usually made from low-alloy steels. Although these perform satisfactorily for many applications, in certain circumstances corrosion considerations may force selection of other materials.

Corrosion takes many forms, but for bolts used underwater on a cathodically protected structure, the most serious aspect is likely to be hydrogen embrittlement (HE). HE is sometimes also discussed under the terms 'environmentally-assisted cracking' (EAC), or 'stress corrosion cracking' (SCC) in the literature. For steels, susceptibility to HE increases with strength, and for some of the high-strength steels used in bolts, otherwise acceptable levels of cathodic protection (say - 1100mV Ag/AgCl) can cause failure.

The critical level of cathodic protection for a bolt depends on the alloy, its manufacture, including heat treatment, and the loading environment, including CP level, applied stress, and preload. It is not possible to give general guidance, other than to recommend that specialist advice should be sought.

It should be noted that cathodic (over)-protection is not the only way of introducing hydrogen into the material. Electroplated coatings, such as cadmium plating, supposedly used to protect the bolt, can in fact embrittle it because of hydrogen evolution during plating, unless the plating has been done properly or steps taken to remove absorbed hydrogen by a low-temperature heat treatment.

A number of materials possess increased resistance to HE, and are suitable for bolt manufacture. There are only limited sources of independent data available[10.3, 10.4, 10.5, 10.6], and since many of these alloys are proprietary, it is recommended that the alloy manufacturers and specialist bolt suppliers and makers be consulted prior to specification. Suitable materials may include some of the stainless steels, especially the higher-alloyed duplex grades and precipitation-hardening grades. Austenitic stainless steels are not recommended for marine bolts because of the risk of chloride stress corrosion cracking. Among the non-ferrous alloys, attention has been paid to many systems, particularly the nickel- and copper-based compositions such as the various Inconels, Incolloys, and Marinel. Some titanium grades have also been studied.

Correct heat treatment is essential for these sophisticated alloys at the appropriate stage during manufacture of the bolts. A number of bolt failures have occurred from errors in these steps, because of the formation of deleterious phases or otherwise unsuitable microstructures.

Corrosion may also take the form of general attack and waste the exposed parts of the bolt, or the first few threads (this is especially serious as these take most of the stress). Crevice corrosion can be a problem, and galvanic effects caused by using the wrong combination of materials should be avoided - it is better to make the bolt noble with respect to the structure, though slight differences should not be unduly serious. It is wise to consult the manufacturer for advice on corrosion effects, especially for some of the more exotic alloys.

Protective coatings such as PTFE not only avoid the hydrogen problem, but should also improve the resistance to galling and corrosion and minimise thread friction[10.7].

Fatigue failures may be minimised by attention to a number of factors, of which preload is probably the most important. Note that the influence of preload is not simple, and the precise effect depends on the load excursions seen by the bolt[10.2]. For bolts seeing only little load excursion, an increased preload has a moderately detrimental effect on fatigue life by increasing the mean load on the bolt. However, for high levels of bolt load excursion, increased preload markedly improves fatigue life by raising the critical load required for joint separation above the maximum external load to be seen by the joint, and by reducing the prying forces at the joint. There is also an intermediate loading regime where the effect of preload is indeterminate.

II 10.4 PRACTICAL CONSIDERATIONS

Since bolts are made by a large number of manufacturers and may be obtained from many stockholders and suppliers, it can be difficult to establish the ultimate source of some bolts. The fact that manufacturing standards may be identified on the bolt heading may not be significant, as it has been known for inferior and sub-standard bolts to carry such markings, and the only sure method of obtaining good quality bolts is to deal with reputable suppliers with good QA. In any event, it is highly advisable for the end-user to have good QC/QA systems, and the testing of batches of bolts is sound practice. In these ways, failures can then be minimised or, if they occur, traced to source. Should bolts be made to order then QC is even more important, especially if unusual alloys are being used.

Quality of manufacture is extremely important: sharp changes of section should be avoided, especially at the junction between the shank and the head of the bolt, and the thread profile should be smoothly finished. Incorrect heat treatment may result in micro-cracking of bolts at thread roots, or even the formation of embrittling phases in some alloys, with deleterious effects on fatigue and fracture performance. Poor quality electroplating can cause bolt failures.

There is relatively little difference in the performance of rolled threads and cut threads of high quality. In general, rolled threads are preferred for fatigue resistance, but may not be feasible for small production runs of odd sizes or for some alloys, especially if heat treatments are involved. Cut threads should be chased after cutting to improve surface finish.

The measurement of preload can be a problem[10.2]. Initial preload may be estimated by several methods. If both ends of the bolts are accessible, then a direct measurement of stretch can be made, and hence the preload estimated. Should this not be feasible, then indirect methods will have to be used, of varying efficacy and reliability. The use of torque to gauge preload is fraught with inaccuracy. Turn-of-nut techniques seem to offer some improvement, but the best results seem to be obtained by the use of bolt tensioners, which grip the bolt and stretch it before the nut is run down. There are several proprietary bolt tensioners available and the manufacturers should be consulted before installation of the bolts. Note that tensioning equipment may not be suitable for short bolts as the reduction in preload from transfer losses is proportional to the reciprocal of bolt length. Equipment using ultrasonic probes to measure bolt stretch has also been developed and offers the opportunity to assess bolting in blind holes.

Lubrication is crucial to prevent galling and to assist the development of preload in bolts tensioned by torque or turn-of-nut methods. It may be noted that for consistent results to be obtained, not only the type of lubricant to be used, but also the method of application and the quantity used, should be specified. As noted above, some protective coatings also improve lubrication.

It is important that the surfaces to be bolted are prepared properly so that little or no bending stresses are present in the bolt. Measures must be taken to minimise misalignment - if plane surfaces cannot be attained, even by spot-facing, then spherical washers may be used. This is particularly important for short bolts, and mounting tensioner jacks on spherical washers is recommended in that case.

There must, of course, be adequate access to the work site for the equipment and divers.

II 10.5 EQUIPMENT AND OFFSHORE SUPPORT

The following equipment and support may be required:

- divers and associated support
- survey equipment if required
- cleaning equipment
- hole-drilling equipment and preparation of surfaces
- bolt tensioners, and, if needed, hydraulic power
- gear to install any temporary support structure
- monitoring equipment to gauge preload or bolt stretch
- cranage.

II 10.6 TIMESCALES

The actual accomplishment of repairs and strengthening by bolting is quick, and the procurement and fabrication stages usually present no undue difficulty. The planning and design stages are likely to be the most time-consuming parts of the exercise.

II 10.7 INSPECTION/MONITORING DURING SERVICE

There are two problems: to determine the actual preload attained by the bolt on installation and during service; and secondly, to assess the integrity of the bolt, including its state of corrosion around the threads.

Although in principle, there are a variety of methods for fulfilling the various demands of inspection of bolts in service, the practical reality is quite difficult.

Ways of assessing the preload attained during bolting have been discussed above, and the methods for checking preload after a period of service are generally similar. In effect these require the tension to be re-applied to the bolt. In some cases, the simple check of applying a torque to ascertain that the bolt is still under load, or has not fractured, is all that is required, but it should be noted that corrosion may fix broken bolts quite effectively. It is recommended that the complete exercise of bolt-tightening be used.

The assessment of bolt integrity is even less easy. Although attempts have been made to develop ultrasonic methods, these have really not progressed beyond the laboratory, and in practice, the only solution is to remove the bolt for examination.

II 10.8 PREVIOUS OFFSHORE APPLICATIONS

Previous operational experience is limited. Bolted repairs were used on the Heather platform pile sleeve repair[10.8]. The technique has been used for a repair in Alaska, where tidal conditions meant only a short working window was available each day[10.9]. Extensive use was made of bolts in the alterations to the Ekofisk installation [10.10].

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II 11 BACKGROUND AND DESCRIPTION OF MEMBER REMOVAL

II 11.1 GENERAL

The purposeful removal of a structural element is a valid structural repair technique in its own right[11.1]. Element removal may also be a temporary measure, as in the first phase of a repair scheme[11.2], or so as to prevent escalation of damage especially if there is danger of a partially severed element falling into the water.

The analysis of the repair scheme is essentially concerned with checking the structural integrity with the chosen element removed, and with verifying the change in fatigue life of the remnant structure. Some engineering effort will be required to detail offshore operations, and to check the safety of any underwater work.

For strengthening work, it is unlikely that the removal of a structural element could increase the load carrying capacity of a framework. However, it is possible to shed loads by removing elements and this opens up the possibility of increasing the safety margin of the structure by changing the loading regime.

The removal of redundant appurtenances (or other non-load carrying elements) is particularly attractive if the superfluous members are found in the wave action zone. Several structural elements both above and below the waterline (eg. boat landings, fenders, pile guides, redundant caissons, etc.) may be removed as part of a load-shedding exercise.

In some instances the installation of an underwater repair demands the removal of structural and non-structural elements in order to provide access to the repair site. On completion of the repair, the larger elements may be reinstated. In the case of bolted clamp repairs, the design of a repair clamp often means that minor connections into the repaired node have to be severed and the cut elements are then permanently removed. In such cases the removal of the element does not constitute a repair by itself, but is a requirement of the installation procedure for the clamp.

Removing the conductor guides frame has been proposed by a number of authors[11.1, 11.3, 11.4]. It may be of value as a repair technique for older structures (designed before about 1981) because the understanding of how conductors behave under internally applied string loads has changed as a result of work undertaken in the early 1980's and reported by Imm and Stahl[11.5]. The earlier platform designs did not separate out the effects of internally and externally applied loads in conductors, producing a heavier design than would be now required. Consequently there will be an inherent capacity for the earlier type of conductors to span greater distances. This inherent strength may mean that there can be circumstances in which the removal of a damaged conductor bracing frame could be justified, as was found to be the case by Lang et al[11.3] in approximately 25% of the *Gulf of Mexico platforms* that he examined.

The severance of structural elements underwater is a relatively commonplace activity and in some cases no attempt is made to retrieve the cut element to the surface. For example, when a jacket structure is installed over a seabed template, then part of the docking structure usually needs to be severed in order to prevent damage to the lower plan bracing of the jacket under wave and current action. A similar difficulty arises when a conductor guide is located close to the mudline and deflects more under storm wave loads than the conductors. In such cases the conductor guides at the lowest level have been cut in order to isolate them from the global structural movements thus alleviating damage associated with differential movement.

In an ageing platform, there may be economies to be had by revisiting the structure and removing structural elements in order to reduce hydrodynamic loads and cathodic protection demand. Submerged pile guides are often removed at such a stage because they are fabricated from plate steel and consequently place a greater strain on the structural resources than a tubular element of equivalent weight. Other items which have been removed from platforms include the following:

- Caissons
- Conductor guide frames
- Launch rails
- Expended conductors and risers.
- Drilling Derrick
- Miscellaneous installation aids (bumpers, padeyes, guides).
- Pile sleeve grout lines.

The cutting techniques which may be used to remove structural elements underwater are generally similar to those used in air. The UEG publication UR18[11.6] has a section which details several tools and suppliers of underwater cutting equipment. A summary of the general limitations to the use of underwater cutting techniques is given in Table 11.1.1 below.

Simple guillotines and rotating discs can be used for thicknesses of steel up to 25mm. For greater thicknesses, Oxy-acetylene and oxy-arc methods are used. However, the acetylene fuelled method is depth limited. The oxy-arc technique, where the heat is supplied by as electric arc, may be employed for a greater range of water depths. However caution is advised when electric currents are controlled by free-swimming divers, because of the amplified effect of electric shocks when sustained underwater. Thermic lances may be used to cut steel and also grout filled steel elements underwater. In particularly awkward situations it may be necessary to use explosives, preferably shaped charges. The use of demolition charges is probably unsuitable for most SMR applications. In several offshore oil production zones there are strict controls governing the use of explosives with legislation covering prevention of theft, safety of personnel, other craft in the water and environmental impact.

Method Type	Cutting Technique	Steel Thickness Range (mm)	Water Depth Limit	Comment
Mechanical	Cutter	2-60		Used for weld preparation
	Wire saw			Closing of crack due to platform movement can be troublesome
	Abrasive water jet	2-230		Safety hazard
Thermal	Oxy-acetylene	10-40	6m	Decomposes under pressure
	Oxy-hydrogen	10-40	1500m	
	Oxy-arc	10-40		Electric shock hazard
	Thermic and ultra-thermic lance			Used to cut grout-filled members
	Plasma arc			
	Pyronol			Custom made 'firework' operating on thermic reaction
Explosives	Primer cord	2-6		May be wrapped around thin tubular sections and used as a cutter without main charge
	Shaped charges	20-120	> 7	Tailor-made charges in a soft metal casing with 'V' notch
Eletro-chemical	Spark corrosion			
	Assisted grinding			

Table 11.1.1: Summary of methods of making underwater cuts

II 11.2 ANALYSIS OF REMOVAL REPAIR

When an element is to be removed from an offshore structure, a systematic analysis of the effects of the element removal should be made. Section II.3.2 indicates the approach to be taken in analysing repair schemes and gives advice on dealing with element removal. The following notes give explicit advice for element removal schemes.

- An in-situ stiffness analysis of the modified structure should be undertaken. The analysis should take account of the change in the loading regime due to the removal of the member as well as the change in the stiffness and connectivity of the structural arrangement. Both calm and storm loadings should be computed. Element code-checks should be re-computed where the loadings have changed or where the effective brace lengths have been modified as a result of the element removal.

- The fatigue analysis of the modified structure should be assessed if significant changes are made to the loading or support conditions of all or part of the structure. In many cases a qualitative analysis will suffice. However, if primary structural elements are to be removed then a re-computation of the fatigue life of the remaining structure may be warranted.

- The element prestresses immediately prior to cutting should be computed and the spring-back of the cut element should be assessed. Account may need to be made of any tie-back arrangements necessary to achieve the safe removal of the element.

The above list is provided only as an indicative guide. There will be significant variation between projects.

When individual instances of repair are investigated it may be important to concentrate the analysis on a specific aspect of the removal. For example, Lang et al[11.3] reports that where the conductor guides were removed from structures it was necessary to re-evaluate the structural stability of the conductors themselves.

II 11.3 PRACTICAL CONSIDERATIONS

The offshore operations associated with an element removal scheme may involve multi-discipline engineering work both in planning and executing the works. Typical problems associated with making underwater cuts are detailed by Stevenson and Sleveland[11.1] who chose to rough-cut the (removed) brace using oxy-arc cutting whilst the final weld preparation for the replacement was made using a hydraulic cutter running on a guide ring.

The brace needs to be fully rigged before any cuts are made so that it does not drop through the water on completion of the cut. A lifting appliance and a lay-down area need to be provided. Account should be taken of any possible hydrodynamic or shock loads arising during element severance and removal. If a sub-assembly is to be removed, then temporary bracing may be required to stabilise the lifted item.

Cutting structural steelwork underwater can be a hazardous operation and needs to be planned with care. Special arrangements may be required in respect of environmental impact if underwater cutting is to be performed using explosives.

When tubular elements are severed care should be exercised to prevent the tubular filling with potentially explosive fumes. In many cases the preliminary work includes the drilling of relief holes in the tubular brace.

If a brace is to be cut then the possibility of the partially severed element rotating and injuring any persons in the water should be considered. Schemes should also examine the possible disruption of pressurised lines (risers, umbilicals, conductors) due to the inadvertent movement of a cut bracing section.

The cutting of a brace from a structure leaves a remnant stub, often with a rough cut edge. It is conventional to grind back the stub. The re-working of the stub may take one of three likely forms:

- Grinding the cut back so that a short, smooth stub profile remains.
- Using a combination of cutting and grinding so as to produce a flush finish to the node. This finish is unusual and may be called for in cases where the remaining node barrel may be susceptible to further fatigue damage.
- Machining the cut brace stub so as to produce a welding profile on the stub which then forms the connection for a "pup" piece to be installed at a later date.

There are very few circumstances in which it would be admissible to leave a rough cut stub on an existing structure.

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II 12 BACKGROUND AND DESCRIPTION OF ADHESIVES AND EPOXY GROUTS

II 12.1 DESCRIPTION

There are two main structural uses of resins offshore: as adhesives and as grout. Although epoxy resins form the matrix of many composite materials, such as glass-fibre reinforced plastics (GRP), which can be used for patching, such applications are not essentially structural repairs or strengthening, and so will not be considered further.

In other words, for structural steelwork, resins are employed not so much as a strengthening or repair technique in their own right, but rather as part of a glued joint, or in a role analogous to that of cementitious grout in grouted repairs. However, the employment of resins is not confined just to steelwork, and their potential is only now being explored[12.1]. Their use in repairing concrete structures is quite well established[12.2], and they provide a means of immobilising structural components, such as a thread-locking compound for bolts.

Whilst there are a range of structural resins, including acrylic, cyanoacrylic, and urethane products, the epoxy resins are the most commonly used and are available as one-component and two-component resins. They may have fillers added to increase the thixotropy or to improve the strength of the final joint. Various catalysts or inhibitors might be added to modify the curing behaviour.

By comparison with their long-standing successful use in the aerospace industry, the use of resins offshore is not well-established, and there is a considerable degree of justified suspicion by engineers concerning their reliability, especially as adhesives. Their successful use depends on a variety of factors, many of which are poorly understood and further dependent on ill-defined variables such as the cleanliness of the surface. Even under controlled conditions, adhesives can give unpredictable results. It is this uncertainty of the behaviour of the joints made, rather than any limitations of adhesives themselves, that have restricted their use.

Adhesives potentially offer a number of advantages:

- stresses are distributed more evenly over the entire joint
- dissimilar materials may be joined
- properties are largely independent of depth
- jointing by adhesives avoids heating of adjacent structure

- may be used for patch plates and other applications on topsides structures, without acting as a possible explosion- or fire-hazard
- may be applied in geometries to which access is relatively restricted
- normally perform well in fatigue if the joint is well-designed.

Epoxy grouts also allow bond strengths an order greater than those of cementitious grouts to be obtained.

In principle, the technique should be suitable for deployment by ROV, which could make it attractive for deepwater repairs.

Complicated geometries that may preclude the use of mechanical clamps or welding habitats, may be accessible by an epoxy resin injection technique.

However, for topsides applications, consideration may have to be given to their heat- and fire-resistance.

II 12.2 CONTROLLING PARAMETERS

The following parameters may affect the selection of resins:

- preparation requirements for the substrates
- design arrangements for injecting the resin or otherwise making the joint, including allowances for volume changes during curing
- curing period
- short pot life once mixed
- heat released in large pours by the exothermic reaction of curing
- arrangements for supporting the joint against relative movement during curing
- QA/QC needed to ensure the desired bond has been attained
- availability of information on long-term stability of the adhesive, including degradation and creep
- inspectability of joint during service
- ability to remove the joint should it prove defective or inadequate.

II 12.3 DESIGN CONSIDERATIONS

In view of the specialist technical nature of resins and because of the lack of applicable standards and codes, it is recommended that detailed liaison should be set up between designers, adhesive manufacturers, and installation contractors at an early stage in design.

The properties developed by a glued joint depend on a number of factors:

- properties of the adhesive, which will be influenced by the age and condition of the adhesive, its preparation, and the curing time and conditions

- degree of surface bonding to the substrate, itself dependent on the nature of the substrate and its surface, including roughness and cleanliness

- thickness of the adhesive layer

- design of the joint itself.

In general, only modest bond strengths are achievable by gluing, but these can be compensated for by good design, which can now be done using CAD and other design tools[12.3]. The strength of lap joints generally increase with the area of overlap, but decreases with the thickness of the glue layer. The mechanical properties of the adhesive may differ substantially from those of steel: the modulus of elasticity is typically one or two orders of magnitude reduced.

The resin itself exhibits visco-elastic and visco-plastic behaviour[12.4], which will be extensively modified by the design of joints.

Volume changes during the setting of epoxy resins can be quite considerable, and complicated, changing from expansion caused by heat evolution during the early stages of curing, to subsequent shrinkage. In general, all adhesives shrink as they set, whether from loss of solvent, cooling from a molten state, or from polymerisation during curing. Such volume changes should be allowed for in the design of the joint in order to avoid either over-loads of seals, for example, or de-cohesion of bonds. It may be necessary to allow for adequate cooling of large volumes of adhesive during setting.

Owing to the shrinkage, the adhesive in a completed joint is in a state of tensile residual stress, which is exacerbated by thicker glue layers, thereby reducing the strength of the joint. When the stress concentration at the ends of a lap joint are also included, the average shear strength of the joint can be reduced to only 30% of the shear strength of the bulk adhesive.

The performance of joints depends very much on the nature of the substrate, and it is advisable to test samples in the laboratory and to hold a full-scale trial onshore before the employment of an adhesive offshore. Performance may be heavily influenced by the joint configuration, so laboratory data should be used with caution.

Only limited data are available on the long-term behaviour and fatigue resistance under offshore conditions of joints made using adhesives[12.5,12.6,12.7]. Although trials have shown that the stability of some forms of adhesive underwater can be very good over a number of years[12.8], attention should nevertheless be given to the possible long-term effects of the underwater environment on the degradation of resins. In particular, water may penetrate the joint at the interface thereby weakening the bond.

Resins can 'creep' under load, though the mechanism is not the same as for metals, since it involves internal changes in the structure of the resin at the level of the molecular chains.

II 12.4 PRACTICAL CONSIDERATIONS

Surfaces must be free of loose debris, rust, and scale, and should be cleaned to Sa 2½. It is also necessary for the surface to be chemically clean if a good bond is to be achieved. In underwater applications, it is possible to use a hydrophobic agent to coat the steel before the adhesive proper is applied. The hydrophobic agent is then incorporated into the adhesive[12.8].

The equipment to be used to handle and inject the adhesive must obviously be capable of functioning reliably at the depths and temperatures under consideration. Injection is the most effective way of incorporating adhesives into closed spaces, but a variety of methods may be used for applying adhesives to joints before assembly. Note that the design of the injection equipment will affect the useable life of the mixed resin, and it may be necessary to compromise on life to ensure thorough mixing. It is possible to mix the components of epoxy resins at the injection head. Manufacturers should be consulted for the best method for a particular adhesive.

Curing can be a protracted process, requiring longer times at colder temperatures. In general, the slower the setting time, the stronger the eventual bond, but it should be noted that some adhesives will not set effectively below certain temperatures. The lower limit seems to be around 3°C, but the manufacturer of the resin should be consulted. The rates of setting can be controlled not only by altering the temperature (for example, by the use of heating pads), but also by the use of catalysts.

Obviously, during curing, the joint is weak and must be supported temporarily, yet the setting time should not be so rapid as to restrict the useful life of the mixed resin in the injection equipment. Any such support structure must not overload the main installation structure, and if not readily removable after the repair, may be designed to be incorporated into the final repair.

It may be necessary to add continuity straps to a glued joint, if part of the steelwork is electrically insulated by the adhesive, or electrically-conductive fillers may be considered.

Thought should be given to the flushing arrangements available should something appear amiss during injection.

II 12.5 EQUIPMENT AND OFFSHORE SUPPORT

The following equipment and support may be required:

- divers and associated support
- cleaning equipment
- resin transport (from the surface) and mixing equipment
- gear to install any temporary support structure
- resin injection equipment, including pumps
- monitoring equipment.

II 12.6 TIMESCALES

In view of the lack of experience with the method, and its likely dependence on the precise requirements of the repair, it is not possible to give estimates of the time required. However, the preparation for the repair is likely to form the overwhelming bulk of the effort and time, with the injection itself being relatively straightforward. Curing time might be protracted, especially at low temperatures, unless arrangements are made for the external application of heat.

II 12.7 INSPECTION/MONITORING DURING SERVICE

The non-destructive inspection of glued joints is difficult, although some ultrasonic methods have been developed for in-air use. It is not known whether this equipment has been marinised.

In view of the unreliabilities of the non-destructive examination of bonded joints, destructive testing of replicate samples prepared from the same batch of adhesive used in the joint is normally used to assess the success of joints, but it is recommended that the joint should be designed for a service life not requiring the reassurance of inspection.

Small core samples can be taken from a large volume of epoxy grouting injected into a repair, but it is probably best to try to assess the progress of the repair by indirect methods, such as monitoring displacements of the repair and adjoining structure.

II 12.8 PREVIOUS OFFSHORE APPLICATIONS

Resin, in the form of a grout, has been used on a novel repair in West Africa, but otherwise no major use of these materials seems to have been reported.

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II 13 BACKGROUND AND DESCRIPTION OF COLD FORMING TECHNIQUES

II 13.1 GENERAL

This section describes certain techniques that have been developed for joining tubular members but as of yet have not been applied in offshore structural repairs.

The techniques rely, to various degrees, on cold forming the tubulars. Two broad technique categories may be recognised:

- Mechanical connectors - using grab, twist and/or gripping devices to achieve the mechanical locking of two component parts.
- Swaging - forming a localised plastic region in a metal tube which creates an interference lock joint with another concentric tube.

Each may offer advantages in particular repair/strengthening situations as discussed below.

II 13.2 MECHANICAL CONNECTORS

II 13.2.1 <u>Description</u>

A variety of mechanical connectors have been developed for connecting and repairing pipelines. They are generally proprietary products and recourse to the manufacturer will be necessary to determine finally the suitability of any particular connector in an application.

The great majority of pipeline connectors are activated by torquing bolts or studbolts. The simplest connectors comprise two halves with the axis of the pipe laying in the plane of the split. Bolt tightening in these types of connectors induces a simple clamping action. Some types of connectors are multi-component devices incorporating a variety of metal-to-metal and/or elastomer seals. Loads are often transferred from the pipe to the connector by serrated, segmented, metal rings. These may rely on a wedging action to be produced between sliding angled faces within the connector when bolts are tightened.

Some of the attributes of mechanical connectors that may lead to their consideration in SMR applications are:

- no welding is required
- the connection can be made quickly

- full strength is obtained immediately on installation

- permanent or temporary SMR can be effected (some connectors can be reused)

- such connectors may be amenable to installation by ROV.

It may be possible to replace a structural member by using two connectors and a spoolpiece. Some connectors may have the facility to accommodate misalignment and even certain errors made in measurement of the replacement spool section.

II 13.2.2 Controlling Parameters for Design

As the range of tubular geometries for structural elements differ to that for pipelines, some mechanical connectors may be unsuitable due to:

- overall size (member diameter and length of connector)

- small tubular thickness in members may lead to crushing on connector activation

- the presence of girth or longitudinal weld reinforcement in members may preclude connector installation unless it is ground flush.

Pipeline connectors are necessarily designed to resist the pressure and axial tension found in pipelines. For structural SMR application, the loads will be somewhat different with axial and bending loads predominating, pressure being absent. The different arrangements used in mechanical connectors gives rise to a range of axial and bending load capacities; recourse to manufacturers' data would be necessary to determine safe capacities. Not all connectors will have been tested under fatigue situations appropriate to structural SMR applications.

II 13.2.3 Equipment and Offshore Support

The following equipment and personnel would be required for a diver-installed connector:

- crane

- rigging for installation purposes

- underwater cutting and grinding equipment, if obstructions have to be removed

- underwater cleaning equipment

- studbolt tensioning/torquing equipment

- diving spread and divers.

II 13.2.4 Inspection

The integrity of the connection is dependent on following the make-up procedure for the connector. It may be possible to get a positive indication of the connector's effectiveness by internally pressurising the member and testing for leakage. Naturally, if the test were to be used, the strength of the member and its lengthening would need to be considered.

At regular intervals, in accordance with the platform's inspection philosophy, the condition of the connector would be checked with respect to corrosion and studbolt tension.

II 13.2.5 Timescales

Requiring essentially mechanical operations, the installation of such connectors can be expected to be completed within one or two days.

II 13.2.6 Background Research

Many connectors have undergone substantial development programmes. Recourse to the manufacturer is required.

II 13.2.7 Previous Offshore Application

No experience of structural SMR using these connectors has yet been gained though individual manufacturers may have relevant information relating to pipeline repairs.

II 13.3 SWAGED CONNECTIONS

II 13.3.1 Description

Swaged connections were originally developed, in the context of the offshore industry, for well operations before being applied to pile-sleeve connections. In principle, the pile is expanded into a groove, or grooves, machined into the sleeve, thereby locking the two concentric tubulars together.

The established method for carrying out the pile expansion is by use of a special tool which is inserted down inside the pile to where the grooves on the sleeve are situated. Two seals are then activated and pressurised seawater between the seals is used to force the pile to expand plastically into the grooves of the sleeve. Use of explosive charges have been proposed, but a means of dewatering the annulus between the pile and sleeve is necessary for this to work[13.1]. This limitation, plus obvious concerns about the use of explosives, has prevented the explosive expansion technique being used offshore.

Details of a proprietary pile-sleeve swaged connection have been described by Clarke et al[13.2]. This and other commercial systems may be covered by patent protection.

The particular advantages of such systems which may make them a favourable SMR technique in certain applications are:

- the connection can be made quickly
- remote operation, often several hundred feet from power source
- full strength is obtained immediately once the inner member has been expanded
- such connections may be amenable to ROV operation
- a significant amount of developmental work has already been carried out and there is an established track record of usage.

II 13.3.2 Controlling Parameters for Design

There has to be sufficient access and length of inner member to allow the tool to be inserted. After making a connection, it might not be possible to retrieve the tool.

The capacity of the connection is a function of tubular geometry and groove details (number, depth and width). The tool manufacturer can give guidance on these aspects.

Tests[13.2] indicate that the load/extension behaviour of connections under axial tension exhibit a soft response. This response needs to be considered during the analysis stage in determining the loads in the connected member and in other nearby members.

II 13.3.3 Equipment and Offshore Support

The following equipment would be required:

- crane

- rigging for installation of member

- underwater cutting and grinding equipment, if obstructions have to be removed

- diving spread and divers if obstructions need removing, or if a stub needs end preparing.

In some situations, the end of a replacement member may be above the water granting easy access for tool insertion from the surface.

II 13.3.4 Inspection

The expansion process can be effectively monitored by observing the volume/pressure curve of the pressurising seawater as it is pumped. However, the proper formation of swaged connections is better confirmed by examining the profile of the expanded tube; this can be obtained by instrumentation affixed to the tool and operated as the tool is withdrawn[13.2].

II 13.3.5 Timescales

A typical operation involving the connection of four piles can be completed in 6 to 8 hours, with joint formation itself taking about one hour per pile[13.2]. For SMR applications, although joint formation times can be expected to be similar, rather more time and effort may be required in inserting the tool, particularly in horizontal members where the assistance of gravity can not be called on.

II 13.3.6 Background Research

A number of static and fatigue tests have been conducted on members up to 72 inches in diameter[13.2].

II 13.3.7 Previous Offshore Applications

Swaged connections have not yet been applied to SMR. However many subsea pile-sleeve connections have been made using the system.

REFERENCES

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APPENDIX A

PREVIOUS APPLICATIONS AND CASE HISTORIES

A PREVIOUS APPLICATIONS AND CASE HISTORIES

A.1 PREVIOUS APPLICATIONS

Many of the techniques described in Part II have seen extensive application worldwide. A cross section of applications up to the year 1987 appears in Table A.1. These applications reflect known cases, and projects which have considered strengthening/repair schemes but those which have not led to specific applications have been omitted. For more recent repair applications, reference may be made to a recent study by MTD[A.1] Table A.1, although exhaustive, cannot, and is not intended to, cover all application cases worldwide. Many strengthening/repair projects have been executed on a strictly confidential basis, and no knowledge or details thereof are available. However, it becomes clear from examination of Table A.1 that:

- A wealth of service information on strengthening/repair systems is now available.
- There has been an increasing need to strengthen/repair steel offshore platforms, particularly as the structures get older.
- Mechanical and/or grout systems are now routinely specified, and are acceptable to both operators and Certifying Authorities.
- Each application is, in essence, unique and reflects the problem case at hand.

In the following sub-sections, five specific case histories are presented, viz:

i. Strengthening to alleviate a static strength problem.
ii. Repair of tubular joints suffering from fatigue damage.
iii. Strengthening techniques following a supply vessel collision.
iv. Repair using wet welding techniques.
v. Repair using dry welding techniques.

YEAR	LOCATION	DAMAGE		DEPTH OF		STRENGTHENING/REPAIR DETAILS
		ORIGIN	NATURE	INSTALLATION (m)	STRENGTHENING/REPAIR (m)	
1973	North Sea	Collision	Dent	70	Splash zone	New brace member placed using cofferdam welding
1975	North Sea	Fatigue	Cracks	40	10	New braces attached to support conductor frame, using mechanical connections
1976	North Sea	(a)	(a)	25	0-25	New braces attached using over 20 mechanical clamps and grouted sleeves
1976	North Sea	Fatigue	Cracks	27	Splash zone	New members added for conductor support using welding, mechanical clamps and grouted connections
1976	North Sea	Installation	Dent	78	6	Mechanical connection placed around damaged member
1976	North Sea	Collision	Hole, Dent, Crack	87	Splash zone	Habitat welding used for repairing damaged node and replacing brace members
1977	North Sea	Dropped Object	Dent	70	11, 23	Habitat welding of damaged member
1977	North Sea	Dropped Object	Joint Cracks	31	17	Cracks welded up using hyperbaric welding
1978	North Sea	Installation	Hole	143	100	Bolted plates used to patch hole

Notes:
(a) Strengthening, not repair, no damage occurred
(b) All depths below MSL unless otherwise stated
(c) N/K = not known

Table A.1: Some previous applications of strengthening/repair systems (Continued...)

YEAR	LOCATION	DAMAGE		DEPTH OF		STRENGTHENING/REPAIR DETAILS
		ORIGIN	NATURE	INSTALLATION (m)	STRENGTHENING/REPAIR (m)	
1978	North Sea	Collision	Dents, Cracks	24	Splash zone	New brace, welded in air
1978	North Sea	Fatigue	Cracks	23	8, 15, 23	Various mechanical connections and clamps for joint by-pass system
1979	North Sea	Collision	Dents, Cracks	40	Splash zone	Members replaced using welding techniques, in air
1979	North Sea	Fatigue	Cracks	162	Splash zone	Welded plates and bolted plates to riser support brackets
1979	North Sea	Welding Fault	Cracks	32	Splash zone	New braces attached using mechanical connections
1979	North Sea	Collision	Dent	38	Splash zone	Welded sleeve over damaged diagonal
1980	North Sea	Installation	Dent	67	45	Mechanical connection placed around damaged member
1980	North Sea	Installation	Severed Joints	40	5	Weld made good using cofferdam welding
1980-81	North Sea	Fatigue	Cracks	145/110	various	Numerous grouted sleeves, mechanical clamps and stressed grouted clamps placed local to tubular joints. Grout filling to enhance strength of joints.

Notes:
(a) Strengthening, not repair, no damage occurred
(b) All depths below MSL unless otherwise stated
(c) N/K = not known

Table A.1: Some previous applications of strengthening/repair systems (...Continued...)

YEAR	LOCATION	DAMAGE		DEPTH OF		STRENGTHENING/REPAIR DETAILS
		ORIGIN	NATURE	INSTALLATION (m)	STRENGTHENING/REPAIR (m)	
1980-82	North Sea	Collision	Dent, Cracks	19	Splash zone	Welded sleeve over two damaged diagonals
1981	North Sea	Installation	Hole, Dent, Crack	140	45	Habitat welding, mechanical connection and grout filling of damaged member
1981	North Sea	Collision	Dent	70	Splash zone	Mechanical connections used to place new members
1981	North Sea	Collision	Dents, Cracks	40	Splash zone	Riser clamp replaced with mechanical connection
1981	North Sea	Collision	Hole, Dent, Crack	34	Splash zone	Mechanical connection and welding in air used to place new brace
1981	North Sea	Fatigue	Crack	70	6	Conductor guide tubular joints repaired by habitat weld
1981	North Sea	Collision	Dent, Crack	70	Splash zone	Mechanical connection placed around damaged area
1981	North Sea	Dropped Object	Dent, Crack	70	35, 70	Habitat welding, mechanical connection and grout filling of damaged members
1981	North Sea	Fatigue	Cracks	162	12	Mechanical clamp around cracked T-joint

Notes:
(a) Strengthening, not repair, no damage occurred
(b) All depths below MSL unless otherwise stated
(c) N/K = not known

Table A.1: Some previous applications of strengthening/repair systems (...Continued...)

YEAR	LOCATION	DAMAGE		DEPTH OF		STRENGTHENING/REPAIR DETAILS
		ORIGIN	NATURE	INSTALLATION (m)	STRENGTHENING/REPAIR (m)	
1981	North Sea	Collision	Dent, Cracks	25	Splash zone	New brace, placed using cofferdam welding
1981	North Sea	Fatigue	Cracks	140	11	More than 5 stressed grouted clamps connecting a new conductor guide frame
1981	North Sea	Fatigue	Cracks	140	40	3 stressed grouted clamps local to tubular joints
1981	North Sea	Collision	Dents, Bowing	100	Splash zone	New braces, attached by means of grouted connection, grouted sleeve, mechanical clamp and stressed grouted clamp
1981	North Sea	(a)	(a)	25	16	Grouted clamps local to X-joints; weld beads placed on existing members using habitat welding
1981	North Sea	Collision	Severed Joints Dent Bowing	37	+5	New brace member, welded into place in air
1981	North Sea	Collision	Dent, Cracks	31	Splash zone	Rewelded cracks, in air

Notes:
(a) Strengthening, not repair, no damage occurred
(b) All depths below MSL unless otherwise stated
(c) N/K = not known

Table A.1: Some previous applications of strengthening/repair systems (...Continued...)

YEAR	LOCATION	DAMAGE		DEPTH OF		STRENGTHENING/REPAIR DETAILS
		ORIGIN	NATURE	INSTALLATION (m)	STRENGTHENING/REPAIR (m)	
1981-82	North Sea	Fatigue	Cracks	156	various	More than 6 complex stressed grouted clamps for caisson attachments
1982	Offshore New Zealand	Fatigue/ Static	Cracks	N/K	various	More than 10 stressed grouted clamps and mechanical clamps
1982	North Sea	Fatigue	Cracks	140	various	Stressed grouted clamps
1982	North Sea	Static	(a)	25	various	Global external strengthening using stressed grouted clamps
1982	North Sea	Static	(a)	25	various	Strong-back scheme for X-joints using stressed grouted clamps
1982	Offshore Australia	Fatigue	Cracks	N/K	various	21 stressed grouted clamps and mechanical clamps for caisson/riser supports
1982-84	North Sea	Fatigue	Cracks	106	various	New braces attached using 5 mechanical clamps and grouted connections
1982-84	North Sea	Fatigue	Cracks	N/K	various	Patch plates placed around tubular joints, using habitat welding

Notes:
(a) Strengthening, not repair, no damage occurred
(b) All depths below MSL unless otherwise stated
(c) N/K = not known

Table A.1: Some previous applications of strengthening/repair systems (...Continued...)

YEAR	LOCATION	DAMAGE		DEPTH OF		STRENGTHENING/REPAIR DETAILS
		ORIGIN	NATURE	INSTALLATION (m)	STRENGTHENING/REPAIR (m)	
1983	North Sea	Fatigue	Cracks	142	Above water	Welded repairs to main deck support girder
1983	North Sea	(a)	(a)	82	50	Stresses grouted clamps for YT nodes, and as attachments for new brace
1983	North Sea	Fatigue	Cracks	102	9	12 stressed grouted clamps for conductor guide
1984	North Sea	(a)	(a)	145	0-10	Stressed grouted connection placed around leg
1984	North Sea	Fatigue	Cracks	110	N/K	Wet welding of new attachment plates
1984	North Sea	Fatigue	Cracks	151	Splash zone	16 grouted sleeves to restore capacity of conductors
1984	North Sea	(a)	(a)	42	Global and local at various depths	Global external brace system, using stressed grouted clamps as attachment points. Local stressed grouted clamps at K-joints
1984	North Sea	(a)	(a)	36	43	Grout filled chords to increase static capacity of joints
1984	North Sea	Fatigue	Cracks	37	7	Various stressed grouted clamps for conductor guide frame

Notes:
(a) Strengthening, not repair, no damage occurred
(b) All depths below MSL unless otherwise stated
(c) N/K = not known

Table A.1: Some previous applications of strengthening/repair systems (...Continued...)

YEAR	LOCATION	DAMAGE		DEPTH OF		STRENGTHENING/REPAIR DETAILS
		ORIGIN	NATURE	INSTALLATION (m)	STRENGTHENING/REPAIR (m)	
1985	North Sea	Fatigue	Cracks	162	100	By-pass bracings, using 5 stressed grouted clamps
1985	North Sea	(a)	(a)	27	various	New braces, attached using 8 stressed grouted clamps
1986	North Sea	Fatigue	Cracks	40	7	New braces installed using stressed grouted clamps, grouted connections and neoprene-lined mechanical connections
1987	North Sea	Subsidence	-	72	Splash zone	Over 50 stressed grouted connections, mechanical connections, grouted connections for additional braces and jacking guides
1987	Offshore Ireland	Fatigue	Cracks	90	various	Over 20 stressed grouted clamps and connections for additional braces to conductor frames and local joint strengthening
1987	North Sea	Fatigue	Cracks	162	N/K	Stressed grouted clamp placed around leg joint
1987	Offshore Nigeria	Blast	Holes, Dents, Cracks	33	7, 13, 21	Doubler plates and sleeves placed around damaged members wet welded in place

Notes:
(a) Strengthening, not repair, no damage occurred
(b) All depths below MSL unless otherwise stated
(c) N/K = not known

Table A.1: Some previous applications of strengthening/repair systems (...Continued...)

YEAR	LOCATION	DAMAGE		DEPTH OF		STRENGTHENING/REPAIR DETAILS
		ORIGIN	NATURE	INSTALLATION (m)	STRENGTHENING/REPAIR (m)	
1987	Gulf of Mexico	Installation	Collapsed brace	232	174-207	Brace replaced using habitat welding
1987	Gulf of Mexico	N/K	Cracks	70	10	- 2 braces installed, one end wet welded, other clamped. - 8 patch plates wet welded. 5 gusset plates wet welded. - 6 tubular joints repaired using wet welding
1987	Gulf of Mexico	N/K	Holes	14	0-14	Patch plates placed over holes, wet welded
1987	Gulf of Mexico	N/K	Cracks	9	Splash zone	Wet welded repair reattaching brace to leg with a new fabricated joint connection
1987	Gulf of Mexico	N/K	N/K	16	13	4 gusset plates wet welded to X brace
1987	Gulf of Mexico	N/K	N/K	N/K	2, 10	Brace replaced, using wet welding
1987	Gulf of Mexico	N/K	N/K	11	0-11	11 Braces replaced using wet welding and pinned hinges. Cracks at 10 joints repaired using wet welding

Notes:
(a) Strengthening, not repair, no damage occurred
(b) All depths below MSL unless otherwise stated
(c) N/K = not known

Table A.1: Some previous applications of strengthening/repair systems (...Continued...)

YEAR	LOCATION	DAMAGE		DEPTH OF		STRENGTHENING/REPAIR DETAILS
		ORIGIN	NATURE	INSTALLATION (m)	STRENGTHENING/REPAIR (m)	
1987	Gulf of Mexico	Fatigue	Cracks	N/K	8	2 braces installed using habitat welding
N/K	North Sea	Dropped Object	Hole	40	+6	Patch plates, welded in air
N/K	Offshore Ireland	Collision	Crack	Kinsale Head	Splash zone	Upper weld gouged out and rewelded, lower weld left intact
N/K	North Sea	Installation	Crack	23	23	Mud line tubular joint repaired by habitat weld

Notes:
(a) Strengthening, not repair, no damage occurred
(b) All depths below MSL unless otherwise stated
(c) N/K = not known

Table A.1: Some previous applications of strengthening/repair systems (...Continued)

A.2 CASE HISTORY I - INCREASING STATIC STRENGTH CAPACITY

General

This case history describes the use of a split-sleeve grouted clamp as a means of strengthening two X-joints on a Southern North Sea gas production platform[A.2].

Platform:	Conoco's Viking AR
Water Depth:	25m (81 ft)
Depth of Repair:	-15m (-50 ft)
Date of Repair:	1981

Details of Damage

No visible damage. Potential punching shear 'overstress' problem; checks indicated that the joints exceed API RP2A allowable stress levels.

Repair Options Considered

1. Welded 'strongback' across joints using either wet welding or hyperbaric welding. Rejected on grounds that the steel properties of jacket not known.

2. Mechanical clamp. Rejected due to lack of fit problems, and operator's own experience.

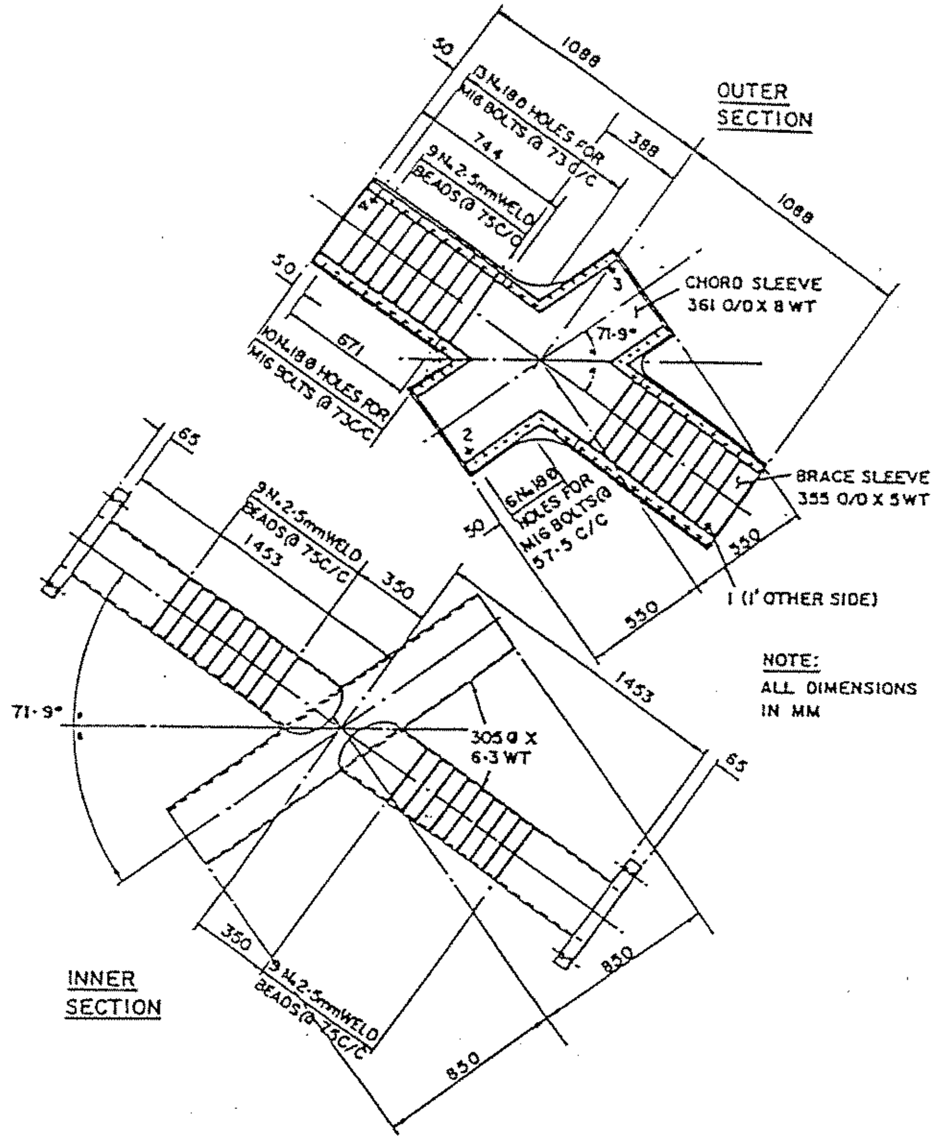
3. Grouted clamp designed to API RP2A. Rejected because of unacceptably long length of connection required.

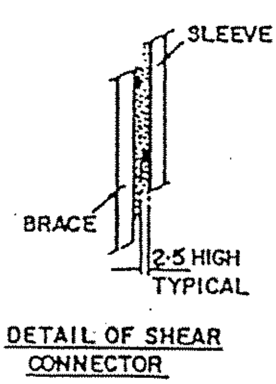
4. Split-sleeve, weld-beaded grouted clamp. Accepted primarily due to much reduced length of clamp vis a vis option 3, ability to absorb tolerances and avoidance of high quality structural weld underwater.

Execution of Strengthening

As part of the design, a structural test programme was executed. Details of the scheme are shown in Figure A.1. Design relied on procedures laid out in Part III.

A detailed joint survey was undertaken, followed by underwater cleaning to bright metal finish. Anodes removed for access. Shear keys placed within inflatable rubber welding habitats. SMA process used. Sleeve was installed in one piece using a hinge unit, with boits tightened using a combination of pneumatic and hydraulic torquing tools.





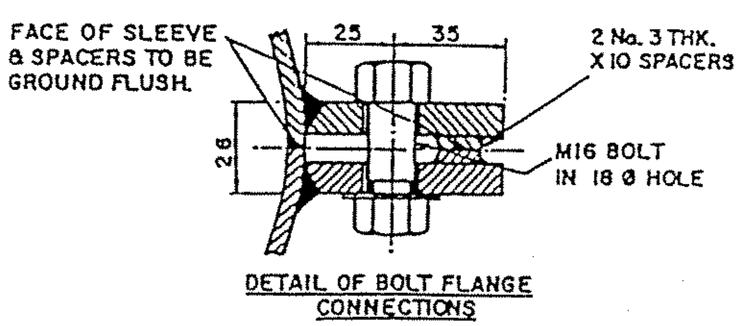


Figure A.1: Details of Grouted Clamp

Comments

GMAW welding systems proved unreliable. Problems encountered with habitat erection; diving contractor underestimated scope. Lump sum diving contract beneficial if obtainable. Underestimate of grout material, even with a factor of four on volume. Overran budget by approximately 6%. 60 days offshore work, including 35% downtime. Analysis of diving time indicates that underwater welding/habitat installation accounted for 45% of the total; significant benefits to be gained if underwater dry welding requirements can be removed.

A.3 CASE HISTORY II - REPAIRING FATIGUE DAMAGE

General

This case history describes the use of stressed grouted clamps as a means of repairing and strengthening tubular joints which have suffered fatigue damage.

Platform:	Northern North Sea
Water Depth:	102m (332 ft)
Depth of Repair:	-9m (-29 ft)
Date of Repair:	1983

Details of Damage

Cracks, located at the weld toes of several tubular joints between the first subsea conductor guide frame and the primary structure of the jacket.

Repair Options Considered

The choice was between underwater welding (hyperbaric or atmospheric) and stressed grouted clamps. Technically, both solutions acceptable. Economically, stressed grouted clamps preferred as the estimated cost and timescale was less than half that for welding. In addition, the joint reinforcement provided by the clamp option eliminated the need for repair of the cracks which were left as found.

Execution of Strengthening

Eight clamps strengthened YT configuration joints and four clamps strengthened T joints. Because of space limitations, the T joint clamps were designed on a load sharing basis, with part of the load retained by the original structure.

Comments

Underwater work carried out from air diving spread installed on spider deck. Average weight of clamp reported as 6.5 tons. Clamps installed on two halves, using platform crane and winch devices. Duration of offshore work was three months. Extremely successful operation. Significant savings over other considered options reported.

A.4 CASE HISTORY III - SUPPLY VESSEL COLLISION

General

This case history describes the use of different clamps as a means of attaching new brace members to reduce the effective length of a damaged diagonal member following a supply vessel collision[A.3].

Platform:	Northern North Sea
Water Depth:	104m (338 ft)
Depth of Repair:	Splash zone
Date of Repair:	1981

Details of Damage

Buckling and local dent at impact location of a diagonal member. The ends of the member were undamaged.

Repair Options Considered

The replacement of the damaged member by a new member, welded at the ends, was considered and rejected as, firstly, there would have been a time between the removal of the damaged member and its replacement when the structure was without any bracing in the affected bay and, secondly, the operator preferred to avoid offshore welding in the splash zone area. Preferred option was to reduce the buckling tendency of the damaged member by placing two new braces at appropriate locations. The braces would be attached by means of connectors.

Execution of Strengthening

A split-sleeve grouted connection was placed over the buckled and dented section. This was connected by new diagonal brace members, one to a leg and another to a horizontal member. The connection to the leg was by a stressed grouted connection; the attachment to the horizontal member was by a mechanical connection. Both these connections were above water. For tolerance, the longer of the two new braces was provided with a grouted connection.

Comments

Offshore work was carried out from scaffolding fixed to the jacket. The accident occurred in March, fabrication took place between July-September and the offshore installation was completed by November. The vessel involved was said to have been captained by a relatively inexperienced man. Delays caused to offshore installation by fabricator falling behind schedule. Delays aggravated by offshore work having to be completed outside the weather window and the operator incurred large standby costs. Operator considers he did not adequately assess capabilities of fabricators before placing contracts.

A.5 CASE HISTORY IV - WET WELDING

General

This case history describes the reattachment of a brace to a leg using a newly fabricated joint connection.

Platform:	Gulf of Mexico
Water Depth:	28 feet (8.4m)
Depth of Repair:	- 5 feet (-1.5m)
Date of Repair:	1987

Details of Damage

Diagonal brace member detached from leg. Cracking in leg extending over 40% of weld circumference.

Repair Options Considered

No details of options considered are given. However it is unlikely that a clamp would have been suitable given the proximity of a horizontal member and the extent of damage to the leg. Environmental conditions are not severe and, therefore, wet welding gives satisfactory results.

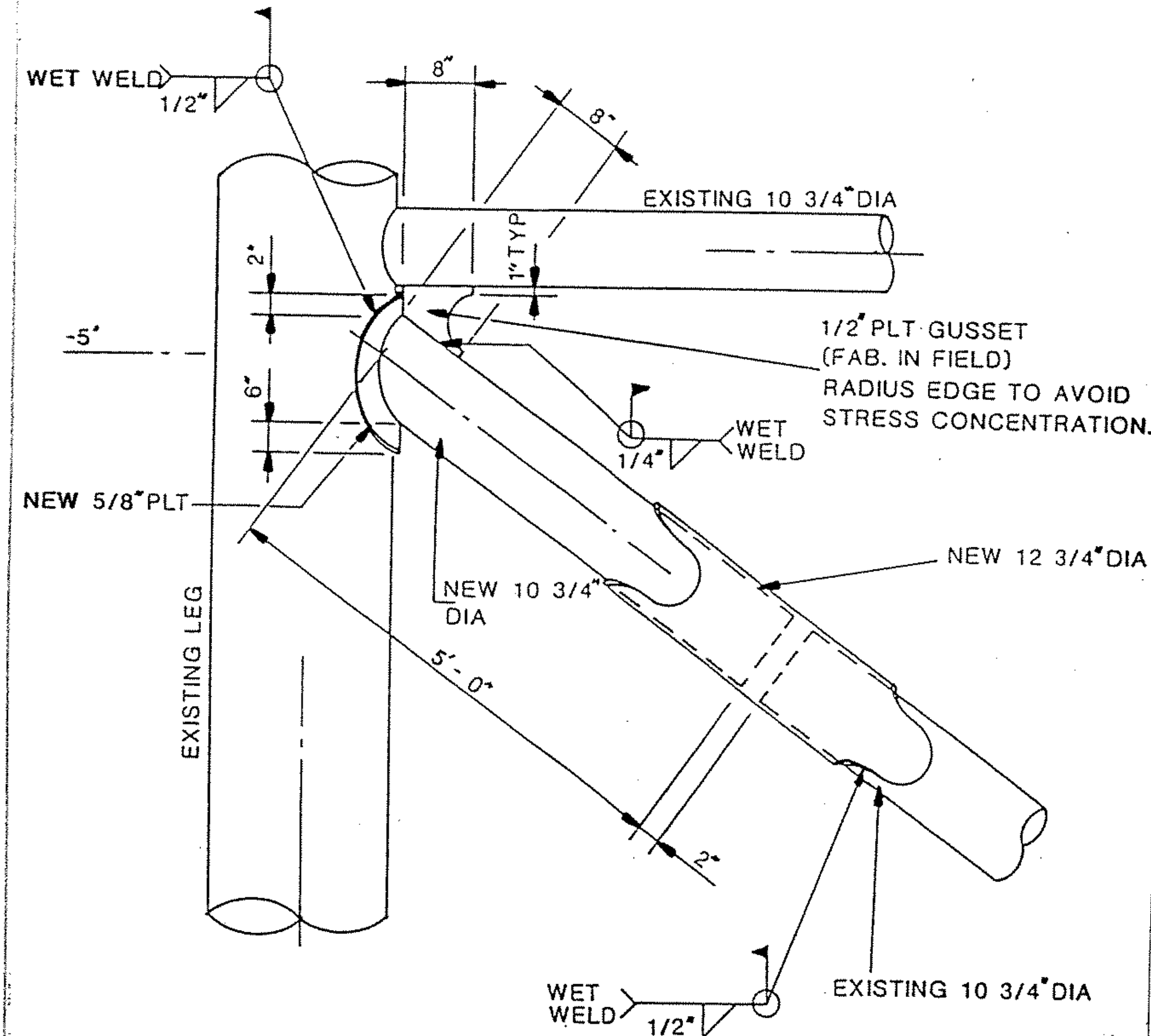


Figure A.2: Detail of wet welding repair
(Note: all dimensions in feet and inches)

Execution of Repair

See Figure A.2. Section of diagonal brace adjacent to leg removed. Crack arrestor holes drilled to arrest smaller cracks, more extensively damaged areas of leg removed. Leg ground to accept doubler plate with brace piece and slip sleeve attached. Doubler plate and slip sleeve welded using 1/2 in (12.7mm) underwater wet fillet weld. Gusset plate wet welded between new portion of brace and existing horizontal. SMAW welding process used, in accordance with AWS D3.6. Type B.1 anode was repositioned on the diagonal brace.

Comments

This repair was part of a programme of repair carried out on 16 platforms. Therefore, mobilisation/demob costs were shared between a number of platforms.

Five working days were spent at this platform on this particular repair. Total diving time was 67 hours, excluding anode replacement.

A.6 CASE HISTORY V - DRY WELDING

General

This case history describes the replacement of a diagonal brace in a vertical frame using hyperbaric welding. Two full penetration tubular joint welds and one castellated fillet weld were required.

Platform:	Gulf of Mexico
Water Depth:	-232m (761 feet)
Depth of Repair:	-174m (570 feet) and -207m (680 feet)
Date of Repair:	1987

Details of Damage

A 54" brace member was completely squashed flat. The cause was believed to be an initial dent which occurred during installation which, under the hydrostatic pressure, squashed leading to rapid progressive squashing of the entire member.

Repair Options Considered

No details of alternatives or clamping schemes are given. It was decided to provide a smaller diameter, thicker walled replacement member which would leave more working room within the habitats and have greater resistance to hydrostatic forces. The replacement member was 42" (1.07m) diameter with 1" (25mm) wall thickness.

Execution or Repair

See Figure A.3. Damaged brace removal required 16 offshore working days. The new brace was installed as follows:

- Main length of new brace installed and held in place with temporary holding clamps.
- Lower habitat installed.
- Profiled lower end of new brace full penetration welded to existing joint can at - 680 feet (-207m).
- Upper slip joint sleeve installed.
- Upper habitat installed.
- Profiled upper end of slip joint sleeve full penetration welded to 8 feet diameter leg.
- Final alignment of new brace and slip joint sleeve adjusted using jacking cylinders (see Figure A.3).
- Perform castellated fillet weld connection of slip joint sleeve to new brace within upper habitat.
- Full penetration welding performed using TIG root runs and MMA fillet runs, a total of 18 runs were required on each weld.

Comments

Habitats were open bottomed therefore no special construction or sealing requirements were needed.

Extensive weld procedure trials were undertaken to prove the procedures and welders.

Offshore timescales are reported as:-

Brace removal	16 working days
Brace replacement	92 working days

Total project duration from notification of problem to completion was 10.5 months. Weather downtime was minimal (3%).

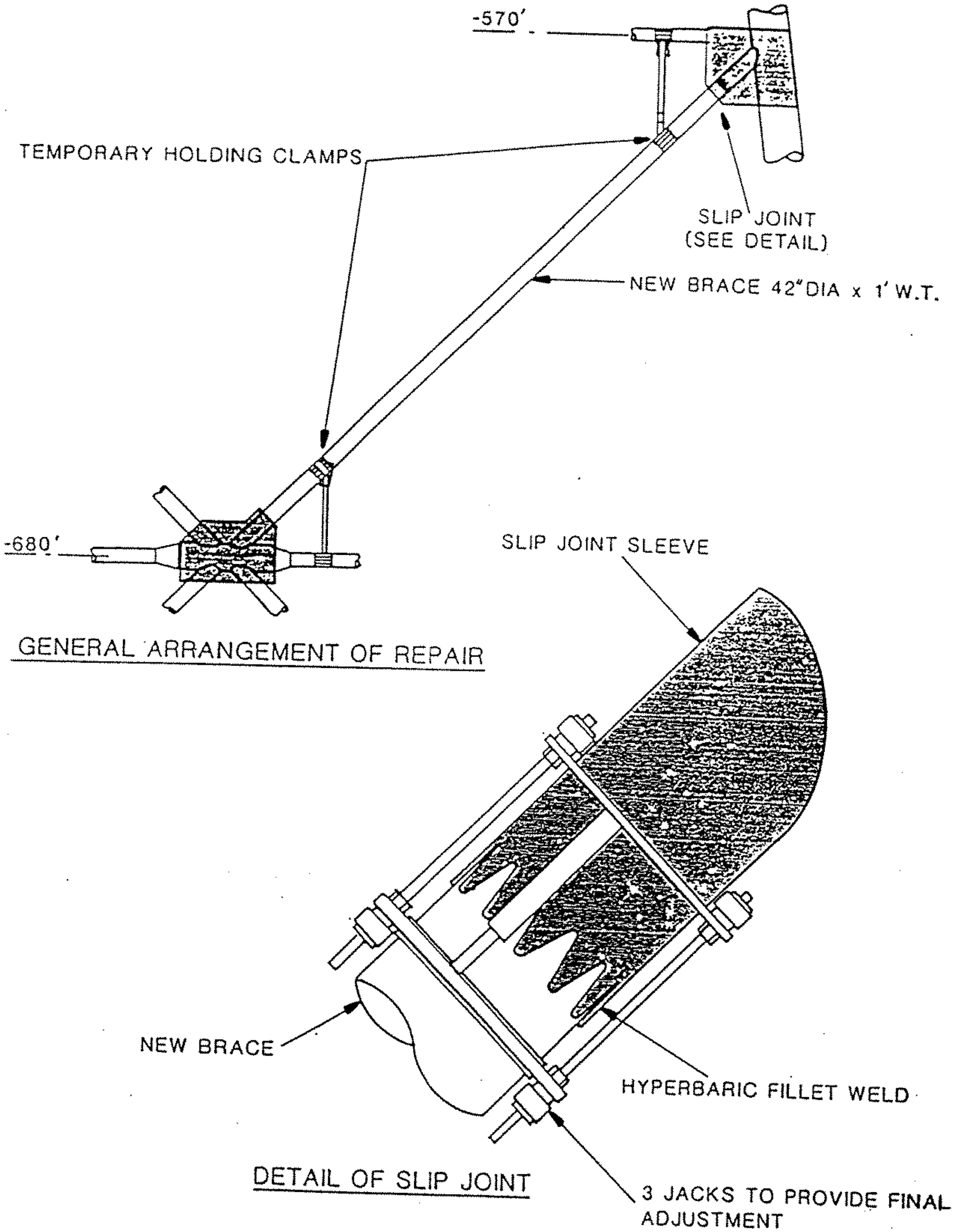


Figure A.3: General arrangement and detail of dry weld repair

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A.1 Marine Technology Directorate Limited. 'Review of Repairs to Offshore Structures and Pipelines'. MTD Publication 94/102, 1994.

A.2 Tebbett IE and Robertson DA. 'Novel Underwater Strengthening System for Tubular Joints'. Paper OTC 4110, Offshore Technology Conference, Texas, May 1981.

A.3 Underwater Engineering Group. 'Repairs to North Sea Offshore Structures - A Review'. Report UR21, 1983.

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STRENGTHENING, MODIFICATION AND REPAIR OF OFFSHORE INSTALLATIONS

PART III - DESIGN RECOMMENDATIONS

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2	Correction to Equation 4.2.11 (page III-4.20), enhancement to Section III 4.2.7 (pages III-4.30 to III-4.33b). Note only these pages plus cover sheet and this revision sheet are re-issued as Rev 2. February 1997.

STRENGTHENING, MODIFICATION AND REPAIR

OF OFFSHORE INSTALLATIONS

PART III - DESIGN RECOMMENDATIONS

STRENGTHENING, MODIFICATION AND REPAIR OF OFFSHORE INSTALLATIONS

PART III - DESIGN RECOMMENDATIONS

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STRENGTHENING, MODIFICATION AND REPAIR OF OFFSHORE INSTALLATIONS

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NOMENCLATURE

a_c	concrete contribution factor
A	total surface area of clamp (m^2); area of slip surface being mobilised (mm^2)
A_b	cross section area of one stud (mm^2)
A_{brace}	cross sectional area of brace (mm^2)
A_c	area of concrete at undented section (mm^2)
A_{clamp}	cross sectional area of clamp (mm^2)
A_g	area of grout at the dented cross-section (mm^2); area of grout (mm^2)
A_s	area of steel (mm^2)
A_{tr}, A_{tr}^*	transformed area at dented and undented cross-section (mm^2)
A_{total}	total cross sectional area of member and clamp (mm^2)
C	current density (mA/m^2)
C_c	correction factor for shear connector extending only partially around circumference
C_L	length correction factor
C_s	surface condition factor for bond component
C_s'	surface condition factor for frictional component
D	damage (fatigue); outer diameter
D'	mid thickness diameter
d	depth of dent, diameter of brace
d_l	bow in member
E_b	Young's modulus of studbolt material (N/mm^2)
E_g	Young's modulus of grout (N/mm^2)
E, E_s	Young's modulus of steel (N/mm^2)
e	external eccentricity of load
e_g	distance between the centroid of grout at the dented cross-section to the centroid at undented section
e_s	distance between the centroid of steel at the dented cross-section to the centroid at undented section
e_t	total eccentricity
e_{tr}	distance between the centroid of dented and undented transformed cross-section
f_{cc}	enhanced characteristic strength of triaxially contained concrete (N/mm^2)
f_{cu}	grout cube strength (N/mm^2)
f_e	Euler buckling stress (N/mm^2)
$\underline{F}$	axial force in member
F	axial force to be transferred to clamp
F_{max}	maximum axial force in member (kN)
F_{min}	minimum axial force in member (kN)
F_n	total studbolt load (kN)
f'_y	reduced nominal yield strength of steel (N/mm^2)
F_y, f_y	yield stress of steel (N/mm^2)
f_{ys}	yield strength of stud (N/mm^2)
g	gap between braces for K/YT joints
h	shear connector height (mm)
h_s	stud shear key height (mm)

I	moment of inertia
I_E	length of column for which the Euler load equals the squash load
I_g	moment of inertia of grout at dented cross section (mm^4)
I_s	moment of inertia of steel at dented cross-section (mm^4)
I_{tr}	transformed moment of inertia of dented cross-section (mm^4)
I_{tclamp}	torsional moment of inertia for clamp (mm^4)
I_{ttotal}	torsional moment of inertia for clamp and member (mm^4)
$I_{xxclamp}$	second moment of area for clamp about X-X axis (mm^4)
$I_{yyclamp}$	second moment of area for clamp about Y-Y axis (mm^4)
$I_{xxtotal}$	second moment of area for clamp and member about X-X axis (mm^4)
$I_{yytotal}$	second moment of area for clamp and member about Y-Y axis (mm^4)
IPB	in-plane bending
k	non-dimensionalised parameter, factor used in calculation of γ_e
K_a	approximate intersection length factor
K'_a	exact intersection length factor
K	effective length factor
K, K'	stiffness factors
K'	stress concentration factor relevant to the flange radius detail
K_b	axial stiffness of studbolt pair; studbolt stiffness parameter
K_c	radial stiffness of grout annulus/jacket member/grout fill combination
K_g	local groove SCF
K_o	codified formulae reduction factor
L	length of connection; member length; length of chord node barrel
L_b	bolt length (for split grouted connection) (mm); stressed length of bolt
L_c	circumferential length of shear connector (mm)
m	modular ratio of steel to grout; non-dimensionalised parameter
M	moment load
$\bar{M}$	maximum theoretical moment in member (kNm)
$\bar{M}_x$	torsion to be transferred to clamp (kNm)
Mx	torsion in member (kNm)
M_{ip}, M_{op}	in-plane and out-of-plane bending respectively (kNm)
$\overline{M_{ip}}, \overline{M_{op}}$	in-plane and out-of-plane bending transferred to clamp respectively (kNm)
M_k	characteristic moment capacity of joint
M_{max}	the greater of the in-plane or out-of-plane bending (kNm)
M_p	plastic moment capacity of intact steel section (kNm)
M_{pd}	plastic moment capacity of damaged steel section (kNm)
M_u	plastic moment capacity of grouted tubular (kNm); moment capacity of joint
n	number of studbolts
N	number of cycles (fatigue); total number of studbolts; sample size
N_{max}	number of cycles in 100 years
OPB	out-of-plane bending
p	member external pressure (N/mm^2)
p_o	annulus external pressure (N/mm^2)
P	beam-column strength determined by design procedure (kN), axial load
P_c	characteristic slip resistance (kN)
P_E	Euler buckling load

P_k	characteristic axial capacity of joint
P_{suc}	characteristic shear load of shear key stud (kN)
P_u	squash load (kN), axial capacity of joint
P_{col}	column strength (kN)
q	external radial pressure
Q_β	geometric modifier
r_{tr}	transformed radius of gyration of dented section (mm)
R	radius of curvature; property section ratio, stress ratio for fatigue loading; existing member outer radius
R_o	grout annulus external radius
R_1	existing member inner radius
s	shear connector spacing (mm)
S	stress range (fatigue)
SCF	stress concentration factor
SCF_E	Efthymiou predicted stress concentration factor
Sr_{max}	maximum stress range (N/mm^2)
S_b	bolt spacing (for split grouted connection)
S_h	circumferential spacing of stud shear key (mm)
S_i	longitudinal spacing of stud shear key (mm)
t	plate thickness, thickness of tubular member (mm), brace wall thickness
T	thickness of tubular member, chord wall thickness
T_{gs}	effective steel wall thickness for grout core
T_e	effective chord wall thickness for grouted joints
U	anode utilisation factor
V_{ip},V_{op}	in-plane and out-of-plane shear respectively (kN)
w	width of shear connector (mm)
w_{mip},w_{mop}	contact load/unit length due to in-plane and out-of-plane bending respectively (N/mm)
w_{vip},w_{vop}	contact load/unit length due to in-plane and out-of-plane shear respectively (N/mm)
w_{TOT}	resultant contact load/unit length due to in-plane and/or out-of-plane bending (N/mm)
W	weight of sacrificial anode required (kg)
w'	resultant contact load/unit length due to in-plane and/or out-of-plane shear
x	depth of cut or groove; distance from clamp centre line to centroid of half clamp about X-X axis
y	distance from clamp centre line to centroid of half clamp about Y-Y axis
Y	design life of repair (Years)
Z	capacity of alloy (Ah/kg)
z_{tr}	transformed section modulus with respect to the dented side (mm^3)
α	2L/D for chord (chord length ratio); distribution factor; angle shown in Figure 5.2.2; constant (= 1.9281)
β	d/D (joint diameter ratio)
Δ	sum of eccentricity and initial bow
δ	height of surface irregularities ($\delta/D_p = 1.25 \times 10^{-4}$ for rolled steel surface)
θ	brace joint intersect angle
γ	D/2T (chord thickness ratio)
γ_e	effective γ ratio for grouted joints

ζ	g/D (gap ratio for K/YT joints)
σ_a	axial stress (N/mm^2)
σ_A	acting slip stress due to axial load (N/mm^2)
σ_b	curve fitting constant; bond strength (N/mm^2); bending stress
σ_{ba}	allowable bond strength (to API RP2A) (N/mm^2)
σ_{bc}	characteristic bond strength (N/mm^2)
σ_B	acting slip stress due to bending load (N/mm^2)
σ_c	characteristic slip strength (N/mm^2)
σ_{cu}	grout compressive strength (N/mm^2)
σ_f	frictional strength (N/mm^2)
σ_h	hoop stress (N/mm^2)
σ_L	longitudinal stress (N/mm^2)
σ_s	slip strength (N/mm^2)
σ_{SFBnom}	maximum nominal stress in sleeve flanges (N/mm^2)
σ_T	acting slip stress due to torsion load (N/mm^2)
σ_u	ultimate axial stress (N/mm^2)
σ_y	yield stress of tubular (N/mm^2)
σ_{ys}	yield stress of stub shear key (N/mm^2)
τ	shear stress (N/mm^2), $\frac{t}{T}$ (thickness ratio for joint); interface shear stress induced by in-plane and out-of-plane bending (N/mm^2)
τ_a	axial interface slip stress (N/mm^2)
τ_b	bending interface slip stress (N/mm^2)
τ_{ip}, τ_{op}	shear stress due to in-plane and out-of-plane bending respectively (N/mm^2)
τ_s	allowable interface slip strength (N/mm^2)
τ_{max}	maximum shear stress (N/mm^2)
Γ	partial safety factor
$\Gamma(.)$	gamma function
Γ_f	safety factor for frictional strength
Γ_b	safety factor for bond
Γ_s	factor of safety on stud yield; safety factor for bending failure of stud shear key
Γ_t	safety factor for sleeve local buckling
Γ_T	factor of safety for failure
η_1, η_2	curve fitting constants
μ	frictional coefficient; curve fitting constant
μ_{allow}	allowable coefficient of friction
μ_c	characteristic frictional coefficient
μ_l	local frictional coefficient
μ_g	global frictional coefficient
μ_{mean}	mean frictional coefficient
μ_t	local frictional coefficient
μ_{ult}	ultimate coefficient of friction
Φ	diameter of studbolt (mm); diameter of stud shear key (mm)
λ	reduced slenderness parameter
β	ratio of smaller to larger end moments
υ	Poissons ratio

υ_g	Poisson's ratio for grout
υ_s	Poisson's ratio for steel
π	3.142

III 1 INTRODUCTION

This Part III presents detailed recommended practices for the design of various strengthening modification and repair (SMR) techniques. It can be used for concept design work as an aid to the selection process in deciding upon appropriate SMR techniques (see Part II) or for detailed design work having made a selection previously. A certain level of engineering design competence is assumed.

The recommended practices involving design formulations are based on the background data and the assessment of those data presented in Part IV. The purpose of the background data and assessments is to expose the basis of the recommendations to the engineer and, where necessary, to relevant authorities.

Certain information should be available to the designer to enable a safe solution to be engineered. A most important aspect is the structural geometry. Great caution must be exercised if as-built drawings or, even worse, approved for construction drawings only are available. These drawings prove, in many instances, to be inaccurate with braces wrongly positioned, even missing, dimensions erroneously marked and lack of information on the presence of doubler plates and anodes, etc. It is important to obtain accurate survey results, particularly in the case where some form of damage has been sustained. Any obstructions, eg. anodes, conductor guide plating etc, that may require removing should be noted and form part of the preparatory works to the SMR.

Other required information concerns loads, both static and fatigue, taking into account if necessary the additional load attracted by the repair itself, ie. such as due to increased stiffness and/or increased drag coefficients.

The properties of the steel in the vicinity of the SMR is also useful information. As a minimum this should be of the grade of steel, but often advantage can be taken of the actual yield properties (but see Section II 3.3.2.1 in Part II). More detailed metallurgical and chemical knowledge of the steel is required where welding is to be carried out. Again, in the context of welding, documentation relating to previous repairs is essential.

III 2 WELDING TECHNOLOGY

III 2.1 INTRODUCTION

The design of welded joints will be affected by the choice of welding process used. The properties of welds made by hyperbaric techniques, at least to 300 or 400 metres, can approach those produced by in-air welding, allowing similar design practices to be used, with one or two variants specific to certain techniques. Against this, hyperbaric techniques are expensive to implement, and it may be preferred to devote effort to designing a weld capable of safely using the reduced properties produced by the cheaper wet welding approach.

Therefore, this chapter gives guidance on those aspects of underwater welding likely to affect the selection of welding techniques and the planning of the repair. Only limited discussion is given on welding processes, as these either differ little from conventional surface-based practice or else, by contrast, require specialist advice in too great a detail to permit full discussion here. In view of the width and complexity of this subject, it is strongly recommended that the advice of a competent welding engineer experienced in underwater welding should be sought during the planning stages of underwater weld repairs.

It is not the intention of this chapter to replace established codes relating to welding, the most widely quoted of which is the specification for underwater welding produced by the American Welding Society, AWS D3.6-89[2.1]. This code is mainly concerned with wet welding and defines four types of weld. It also covers various processes and variables such as water depth. Further background can be found in Delaune[2.2].

Since offshore structures often differ from their design geometry by tens, and sometimes hundreds, of millimetres, it is normal practice at an early stage to carry out a precise survey of the geometry of the area to be repaired. This is especially pertinent if a member has to be replaced, so the new member can be machined to size, unless a telescopic joint is used to avoid fit-up problems.

Because most structural members are under load, care must be taken to avoid injury to divers or damage to equipment when cutting members for removal. Prior to cutting, a structural analysis (see Part II, Section II 3) must be carried out to ensure that the stability of the structure is not affected by the procedure. The member must be cut before any habitat is installed so that any movement caused by the release of locked-in stresses does no damage to the habitat. Removal of members may require additional temporary bracing to stabilise the structure, while the effects of the loading imposed on the structure by the equipment used in the procedure must also be evaluated.

III 2.2 DESIGN OF REPAIR JOINTS

Since welds made by hyperbaric and one-atmosphere welding are comparable with onshore welds, a repair made using these methods should not be unduly limited by the properties attained, and so in principle might be designed mainly with standard in-air experience.

However, this is not the case for wet welding. Even using the measures outlined in Section III 2.8.2, the weld properties are significantly degraded, sometimes substantially. Whilst such joints can be adequately strong (perhaps 80% as strong as dry welds), they lack ductility and toughness and so it is necessary to adapt the design to ensure structural reliability.

Some design expedients can be used to reduce the stresses in underwater welds:

- increase the length of welds
- increase the cross-section of welds
- change the orientation of the welds relative to the direction of stress (this depends on the main damaging components of the loading regime)
- move underwater welds away from hot-spots and other regions of stress concentration (by use of components fabricated above water).

Figure 2.2.1 illustrates typical examples of methods of attaching tubulars.

Joints may be designed with additional stiffening, such as saddle plates on tubular members, or plates between members, in order to resist relative movement of members which would generate strains in the weld metal. Specially shaped joining pieces can be orientated to minimise the amount of overhead welding required in a joint, as shown in Figure 2.2.2.

In common with all subsea work, it makes sense to design the repair so as much of the fabrication as practicable is done on the surface, especially any complicated aspects, leaving only simple tasks for subsea.

These techniques will add to the amount of work involved in a wet welding repair procedure, and, in specific cases, it will be necessary to balance this against the effort required to deploy a welding habitat to carry out a hyperbaric repair procedure.

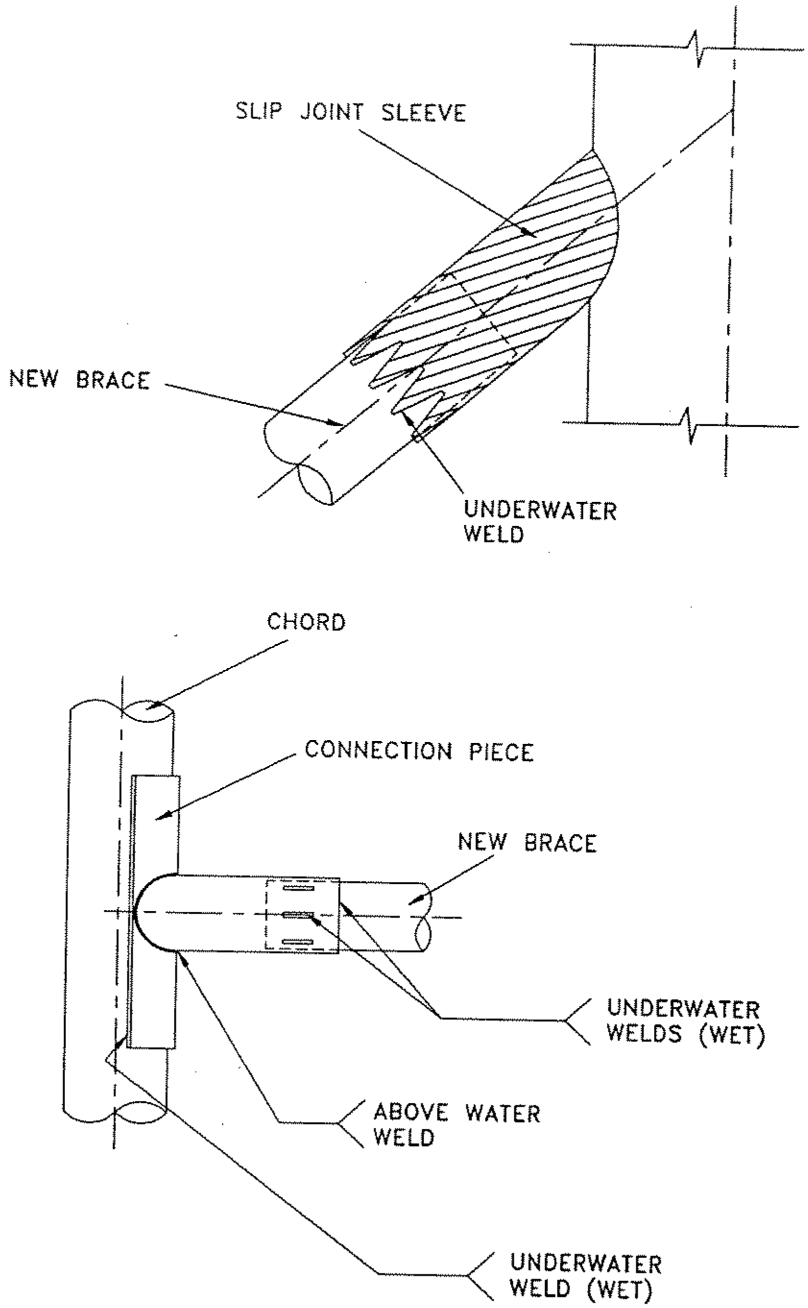

Figure 2.2.1: **Design examples illustrating methods to reduce stresses in underwater weldments**

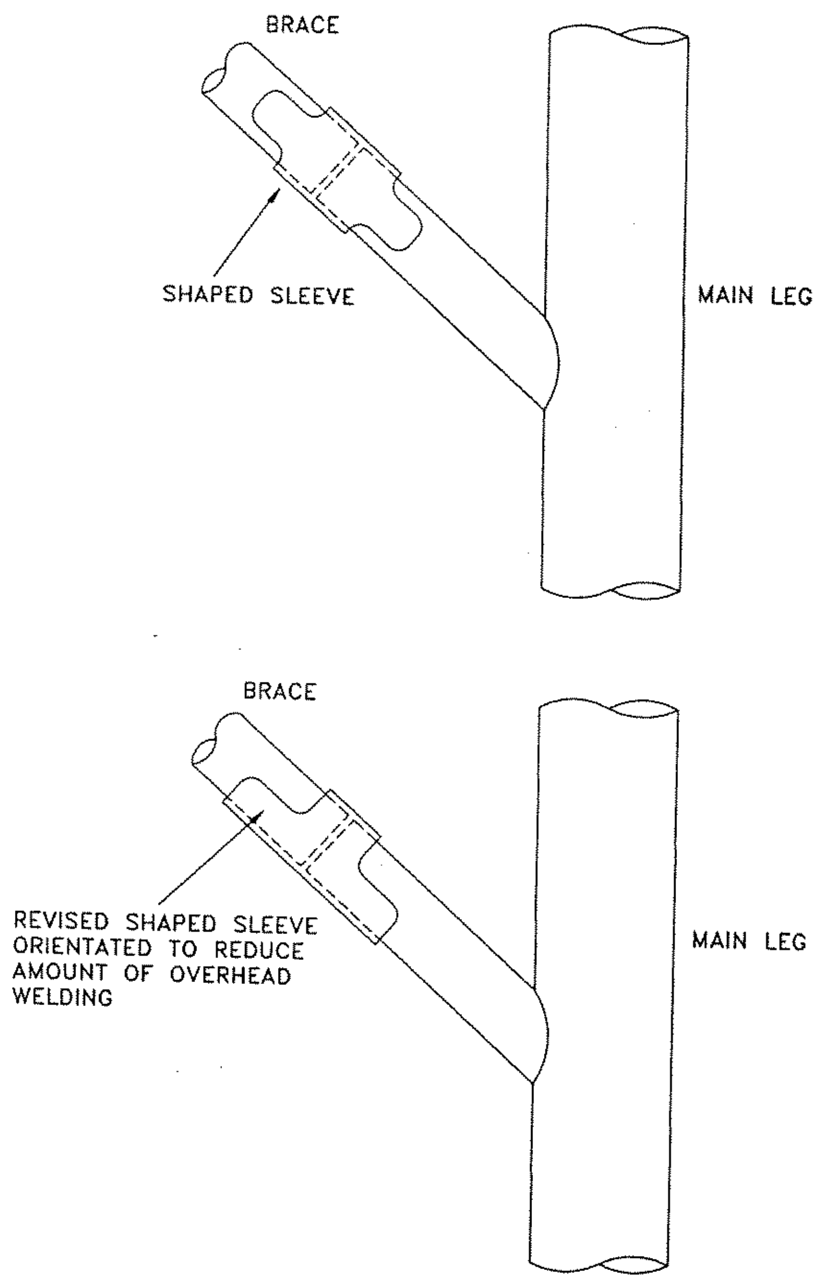

Figure 2.2.2: Design example illustrating attention to positional welding

III 2.3 JOINT PREPARATION AND COMPLETION

The preparation of joints is similar to that in surface-based welding, although manpower requirements may be a problem. Where sophisticated dry transfer welding habitats are used, as many as four crew members might be in the habitat at once, but this is the exception rather than the rule, and normally only one or two are present. Equipment such as weld preparation tools, joint alignment jigs, and inspection gear must be capable of being operated effectively by the limited manpower available.

If a welding chamber is to be used, the jacket structure must be cleaned of marine growth, corrosion and any loose paint, not only at the repair site but also to allow the effective sealing of the chamber to the structure. This is, of course, much easier for hyperbaric chambers than for one-atmosphere systems.

Repairs in platform jackets are frequently necessitated by fatigue cracks, which are usually ground or milled out by pneumatic or hydraulically-powered tools. Once an appropriate inspection, usually employing electromagnetic non-destructive examination (NDE) techniques, has confirmed that the crack has been completely removed, the resultant groove can be welded.

Note that the static strength of a member or joint under repair may well be appreciably diminished, both whilst a groove is unrepaired, owing to a reduced cross-section and changes in stress concentration, and during welding because of the low hot strength of the welded area.

If a crack has gone through the wall thickness, or the structure has been completely cut through for some reason, the root run of the weld repair is often done with GTAW, because of the greater control available, after which the joint is normally filled by SMAW or occasionally FCAW. (Note, these acronyms and the welding process are defined in Part II 6.4 of this document.)

It is essential to ensure that the tube ends to be joined are maintained in a fixed relative position during the repair. This is usually achieved with the aid of an alignment system in the form of exterior bracing, sometimes combined with the welding chamber, which holds the ends at the required position and attitude.

Lack of roundness can be a problem where two tubular members are to be girth welded, and use is made of similar techniques to those employed in surface-based welding, such as an alignment clamp - a circular frame mounted around the tube, and equipped with a series of radial screw jacks capable of forcing it into the required shape. It is not uncommon for this part of the procedure to take as long as the filling of the rest of the weld preparation. Once acceptable alignment has been achieved, the root weld can be laid, and the weld completed. Because of the irregular shape of the intersection at joints, manual welding is exclusively used for the complete orbital welds needed in this type of repair, although robotic systems have been proposed for future development. Stub/brace welding can be achieved with existing robotic systems.

Once the weld has been made, an appropriate time, usually more than 24 hours, should be allowed for any delayed cracking to occur before a final inspection is carried out, which should normally be to at least the same standards as would pertain for inspection of this area underwater. Some form of coating or anodic protection is often applied before the repair is judged to be completed.

III 2.4 QUALIFICATION, PROCESS MONITORING, AND SAFETY

The qualification procedures for one-atmosphere welding should pose no significant problems because of the technique's close similarity to surface-based welding. However, wet welding will demand greater effort and hence expense to develop procedures, with hyperbaric welding being even more demanding.

The weld repair procedures themselves tend to be highly specific, sometimes even relating to a single batch of welding electrodes or type of structural steel. It is rarely possible to transfer a procedure developed for one operator to a second organisation, even if two repairs are nominally very similar. Although the details vary from company to company, a welder's procedure qualification normally lasts three to six months, and is normally carried out at the greatest depth that will be encountered during the operation. If the range of depths in the envisaged repair is very great, additional trials at a shallower depth may be required.

Whatever procedure is finally selected, it is clearly important to monitor the welding current to ensure compliance with the specification. A wide range of instrumentation - analogue and digital meters, data loggers and the like - is available for monitoring.

The final specification for an instrumentation system must satisfy the information required by the welding team to ensure that the welding procedure is being adhered to, and the information required by any inspection authority. Obviously, it is also important that the systems should be reliable and robust in the offshore environment[2.3], and the instrumentation must be capable of periodically being calibrated.

In particular, system specification is influenced by any requirement for hard copy output from the instrumentation. If manual recording of readings taken by inspectors is acceptable, as it is in many parts of the world, a simple set of analogue or digital meters will suffice. However, if hard copy from data loggers, or systems such as the 'PAMS' or 'Monarc' briefcase monitors are required, the instrumentation becomes correspondingly more complex and costly, and more difficult to protect against the environment.

Underwater welding is not unduly hazardous, but it is appropriate to recognise that some precautions will be required additional to those needed for more usual offshore operations, or for conventional welding. It is not possible here to give a comprehensive catalogue of possible risks, and the need to consult experienced and qualified professionals is reiterated.

The obvious danger of using high electrical power in close proximity to seawater is not the only hazard. As noted below in more detail, some hyperbaric atmospheres increase flammability and can produce narcotic effects if inhaled. Consideration should also be given to the problem of fume from hyperbaric welding, which can not only handicap visibility within the chamber, but which might pose a health hazard if breathed in. Nevertheless, effective safety procedures have been developed by the industry and the safety record of underwater welding is generally good.

III 2.5 POWER SUPPLIES

Power for hyperbaric welding is normally provided from three-phase 380 or 440 VAC installed supply, although it can be provided by electric motor-generators and internal combustion engines, for example. Care should be taken to ensure that the supply is capable of meeting the extra demand caused by welding and associated equipment, which will partly depend on the technology of the power supply unit.

Older designs of power supply units use conventional transformers and rectifiers, with power regulation by thyristors (SCR's - silicon controlled rectifiers). Although proven and reliable, these systems are heavy and bulky, electrically inefficient, and have slow dynamic responses. The newer inverter systems are typically half the size of similarly rated conventional units, and offer much more flexible outputs. Whilst such control systems are commercially available for SMAW and GTAW, at present they must be specially built for GMAW. Inverter systems are currently up to 50% more expensive than comparable conventional units and tend to be less robust. Consideration may have to be given to air filtration and environmental protection if these units are to perform reliably.

With the exception of certain advanced welding techniques, discussed below, the power unit is usually sited at the surface, and thus appropriate allowances must be made for the resistance of the cables connecting the power supply on the surface with the underwater worksite. It is important to use cables for both the current supply and return paths, rather than allowing current to flow through the structure, if consistent operation of the power supply is to be ensured, and to avoid possible severe corrosion problems for the structure.

The additional resistance caused by the cables will affect the voltage required from the power supply. British Standard BS638 is relevant, but the figures given should be used for guidance only, as temperature changes resulting from the cooling of the cables by the seawater and their heating by the passage of current will change their resistance. It is recommended that the 'worst case' estimate of the output voltage required from the power supply (the highest envisaged arc voltage plus the cable voltage drops at maximum current) should not exceed 90% of the rated output voltage of the power supply. If necessary, the power supply must be uprated, or the voltage requirement should be reduced. This may be done by several means. It may be possible to operate at a lower current, or the resistance of the cables may be reduced either by shortening them or by increasing their cross section, including using multiple cables.

In order to attain an automatic compensation for cable resistance when using FCAW and GMAW, the voltage may be measured local to the arc (desirable anyway for accurate process monitoring) and fed back to the power source control system.

For more advanced welding processes such as high frequency pulsed GTAW or controlled transfer pulse GMAW or FCAW, it is necessary to allow for the additional inductance and capacitance of the welding cables, as these can affect performance. For these techniques, in contrast to the other hyperbaric methods, it is normal to position the welding power supply close to the repair worksite, usually attached to the welding habitat. The power unit is contained within a pressure vessel at one atmosphere pressure, and it is necessary to ensure adequate heat-sinking into the seawater for high power devices, either by direct conduction through the pressure vessel wall or by using gas as a heat transfer medium. Suitable underwater enclosures have been developed.

III 2.6 HYPERBARIC CHAMBER ENVIRONMENTS

The atmosphere used in a hyperbaric welding chamber can have a number of consequences, ranging from medical effects on divers to metallurgical implications for the welds, and these should be considered during the planning of weld repairs, and may even affect the choice of welding technique.

The three gases in widespread use for dewatering chambers are: air; and gas mixtures rich in argon or helium.

The use of air, though cheap, carries the need to consider the risk of fire, as the flammability of materials is greatly enhanced in hyperbaric air. The risks to the welder-diver are reduced if small, dry spot type systems are employed, in which the welder is not within the welding chamber, but large chamber operations using air must be considered hazardous at any depth. In addition, the weld pool must be very well protected by the welding process to avoid problems with nitrogen and oxygen.

Argon is approximately as dense as air, and its use is often cheaper than helium. Argon has a lower thermal conductivity than helium, thus producing a desirable slower cooling of the weld. However, it is narcotic to divers, and in the North Sea argon levels in the welding chamber are monitored when it is used as a shielding gas for GTAW operations.

Helium is about one-tenth the density of argon, and is generally more expensive, although the large amounts used in diving operations can make it more available in some situations. It is frequently used as a chamber gas in the North Sea, sometimes with oxygen additions, as for safety reasons, many inspection authorities prefer a chamber gas composition which is at least nominally breathable by the diver. Its high thermal conductivity can cause fast cooling rates for the weld, raising the hardness levels.

The use of pure helium as a shielding gas for GTAW, at pressures greater than splash zone depths, gives high rates of tungsten erosion in the torch[2.4]. This problem is overcome by employing argon/helium mixtures, but, because of the difference in density of these gases, care must be taken to ensure that the gas shielding is adequate when welding in the overhead position. This may affect the planning of some weld repairs, if proper gas cover cannot be attained. It is rarely feasible to mix gases on site, necessitating the use of bottled supplies, and attention should be paid to the recommendations of the gas suppliers.

Most hyperbaric welding processes produce high levels of weld fume, normally as finely divided oxides of iron and flux material[2.5]. These limit vision within the welding chamber, and can affect the reliability of equipment. Common practice is to remove this particulate material by means of recirculating fans and filter units operating at the ambient pressure of the chamber, and allowance should be made for these features in the design of the chamber. The operational procedures should allow for periodic filter changes.

III 2.7 METALLURGICAL CONSIDERATIONS

The metallurgy of hyperbaric welding essentially differs little from that of surface welding[2.6]. The main differences are the effects of enhanced cooling rates, as discussed below, and the compositional changes observed with processes utilising fluxes[2.7]. The precautions taken with regard to hydrogen-induced cold cracking (HICC) differ only in degree from those normally employed on the surface.

All structural steels currently used offshore are sensitive to some degree to the cooling rates experienced after welding. In general, the faster the cooling of the weld metal and associated heat affected zone (HAZ), the less favourable their metallurgical structure and properties. The time taken to cool between 800°C and 500°C is significant in determining the main microstructures of the weldment, whilst the time the weld metal remains above 100°C affects the hydrogen content of the weld region.

Unfortunately, the hyperbaric environment has an intrinsically higher thermal conductivity than air at one atmosphere, increasing cooling rates. The use of helium or argon exacerbates the problem, with the former having the higher conductivity. Cooling times at pressures equivalent to 200 to 300 metres are typically one-third of those on the surface, resulting in higher hardness material, particularly in the HAZ[2.8].

Only limited scope exists for using higher heat inputs during welding to counteract this effect: SMAW offers some flexibility in this respect. Careful control of bead size and placement in multi-pass welds can improve the hardness of the weld metal and HAZ, minimise hydrogen, and help reduce residual stresses by acting as a form of post-weld heat treatment (PWHT). The use of temper beads in the capping runs of welds is widespread[2.9].

Both pre- and post-weld heat treatment are desirable for minimising the risk of cold cracking and may reduce residual stresses, but although they can be applied in hyperbaric chambers, the conditions tend to make such practice difficult and ineffective, and it might be wise to design welds to avoid the need for these measures, or at least not to rely on their efficacy.

An important parameter commonly ignored in hyperbaric welding is the proper control of shielding gas flow. The fact that often the welding habitat is filled with inert gas does not reduce the importance of good shielding. Habitat gas is frequently humid, and may be contaminated by divers' breathing gas, which can cause welding problems. In addition, incorrect gas flow can create arc instability problems similar to those of arc blow, and easily confused with them. Unfortunately, suitable reliable gas flow monitoring systems are not readily available, so the weld procedure and design should recognise this lack of information.

There are few metallurgical problems associated with one-atmosphere chamber welding, other than perhaps an increased difficulty of keeping weld preparations clean and dry.

Wet welding can cause difficulties, chiefly the rapid quenching of the hot weldment and the high hydrogen uptake. Obviously, there is no possibility of effective pre- or post-weld heat treatment other than by using a chamber, and so the only feasible solution would appear to be the selection of weld metal capable of tolerating high levels of hydrogen. Other than this the high risk of HICC associated with wet welding largely has to be tolerated, possibly requiring a high level of repeated repair of welds.

III 2.8 WELDING PROCESSES

III 2.8.1 Dry Welding

The following sections will discuss only hyperbaric dry welding techniques, since, as has been pointed out several times already, underwater one-atmosphere dry welding is extremely similar to conventional practice.

III 2.8.1.1 Hyperbaric SMA Welding

Hyperbaric SMAW generally closely resembles surface-based SMAW, but certain differences do exist which can create problems.

SMAW in positions other than downhand is difficult owing to problems of controlling the weld pool caused by the increased burn-off rate of the electrodes resulting from the environmental pressure. The only practical solution has been to restrict the size of the weld pool, hence decreasing the heat input which, in the hyperbaric environment, leads to greater cooling rates with attendant problems. This may affect the design of weldments.

The balance of gas/slag/metal reactions within the arc is also disturbed by the environmental pressure, causing compositional changes in the weld metal, chiefly increasing carbon content and decreasing silicon and manganese[2.10], and increasing hydrogen absorption from the arc atmosphere. The effect of these changes must be evaluated during procedural qualification in order to ensure that the metallurgical and mechanical characteristics of the weld metal are acceptable to the certification authorities. In general, these factors produce welds which are more hardenable and sensitive to cooling rates than similar welds carried out on the surface.

Because of the danger of hydrogen cracking, precautions must be taken to minimise hydrogen pick-up, most of which comes from moisture absorbed within the flux coating of the electrode. It is thus necessary to ensure that good housekeeping measures are practised for the careful pre-drying of electrodes, and their storage in sealed, desiccated, or heated containers while being transported to the habitat. It is also vital to restrict the time an electrode is exposed to the habitat atmosphere before use. Identification of welding packages, usually by colour coding, is sometimes used, with the identification denoting only those packages allowed to be opened and used during specific portions of the welding.

Despite these drawbacks, SMAW is the most widely used hyperbaric welding technique, principally because of the simplicity of the equipment and the large number of divers familiar with the technique. Provided appropriate procedures are developed, the process can be used very effectively.

III 2.8.1.2 Hyperbaric GTA Welding

Of all the welding processes, GTAW provides the welder with the maximum amount of control over the weld bead shape and fusion level. For this reason, it is frequently used in critical areas, such as root welds and hot passes. Although perceived as capable of only low deposition rates, the process can in fact achieve deposition rates comparable with SMAW at depths of 150 to 200 metres, because of the higher voltages generated under hyperbaric conditions. Part IV contains additional information on the relationship between voltage and depth.

The increase in GTAW arc voltages with environmental pressure must be added to the voltage correction for the resistance of the cables connecting the power supply to the welding habitat, as discussed above. These factors must be specified before practical voltage operating limits can be established in a welding procedure.

There appears to be a maximum depth limitation to GTAW of 500m because the arc is destabilised as environmental pressure is increased.

Hyperbaric manual GTAW torches are normally identical with surface-based equipment, the sole modification being the removal of the operator's control switch on the torch, the power supply being controlled by the surface support crew.

GTAW is also used for automated welding, although development has been concentrated on equipment for pipeline joint geometries. However, such equipment can be used for making stub to brace welds.

III 2.8.1.3 Hyperbaric GMA/FCA Welding

The hyperbaric implementation of GMAW requires the installation, at the repair site, of a wire feed unit, which is normally protected from the surrounding water by a container pressurised a few tenths of a bar (some 10^4Pa) higher than ambient pressure. This additional equipment represents a complication and cost penalty for the process compared with SMAW.

Much of the equipment and process technology of FCAW is identical to GMAW, as are some of the problems such as those of complexity. Therefore, FCAW is not generally considered to be competitive on cost grounds with SMAW, despite the fact that specialised consumables have been developed capable of all positional operation at depths of 200 metres, with excellent weldability and weld metal properties. A small diameter (1mm) tubular consumable from Oerlikon has been shown to produce excellent welds at a reasonable cost at depths down to 400 metres, and has been used for a limited number of applications.

III 2.8.2 Wet Welding

Wet welding is widely used in the USA, and the most useful document providing design guidance for wet welding is the current AWS Standard D3.6[2.1]. In Europe, with lower temperatures, different steels, and larger structures, as well as possibly a difference in operating philosophy, wet welding has always been viewed with considerable suspicion.

A wet weld was first carried out in the North Sea on a major structural member in 1991[2.11], and there has been an increased level of interest in the technique recently. However, its use in the North Sea has not yet generally developed beyond the attachment of non-critical items such as anodes.

Until recently, wet welding was carried out exclusively using SMAW. However, wet FCAW equipment has been developed in Germany and Russia, and is being evaluated in the USA. Such equipment offers improved heat inputs and arc times, contributing to enhanced weld deposition rates, though at the expense of more complex welding equipment[2.12].

Wet welding suffers from a number of problems caused by the direct contact of the arc with the water. Although operating a welding arc surrounded by seawater presents obvious safety problems, these can be largely overcome by close adherence to operating procedures[2.13].

Of greater practical significance is the fact that the microstructure resulting from the very high cooling rates caused by the seawater quenching is very susceptible to the hydrogen generated in the arc, resulting in a high probability of hydrogen-induced cold cracking (HICC). Furthermore, the stability of the arc is reduced, requiring the use of short arc lengths[2.14]. These problems may well be exacerbated by the difficulty of welding in some positions underwater, especially overhead.

Various approaches are used to overcome these problems. Austenitic welding electrodes deposit material in which hydrogen is more soluble than ferritic weld metal, reducing the diffusion of hydrogen to the sensitive HAZ. High heat inputs combat the fast cooling rates, and welders are highly trained to use short arc lengths and carefully regulated electrode weave techniques to minimise generation of hydrogen and to avoid shape defects.

III 2.8.3 <u>Other Techniques</u>

A variety of other welding techniques and methods for use underwater have been investigated, ranging from explosive welding to friction welding. Most of these have either have not been developed past the experimental stage, but it is worth highlighting two methods, both in the early stages of adoption offshore, but which have considerable potential: friction welding, and hyperbaric plasma welding.

Friction stud welding is confined to circular geometries of stud at present, but has nevertheless been used for a variety of repair and construction tasks, principally the attachment of connectors for cathodic protection systems and shear pins for grouting operations[2.15, 2.16]. It is capable of deployment from a remotely operated underwater vehicle (ROV), the first underwater welding process with this ability.

Recently, some interest has been shown in developing a modification of the friction stud process, the so-called friction stitch welding[2.17]. Several different approaches have been mooted, and it is likely that the technique will occupy a niche in the repair market. However, it is not yet really commercially accepted and widely applied.

Hyperbaric plasma welding is not commercially available offshore at the time of writing. It possesses considerable potential and as it is undergoing much active research and development, it may assume an important role in the future.

Plasma welding is the only hyperbaric welding process for which conventional equipment cannot be directly used. The geometry of the welding torch has a significant influence on the welding performance, and optimisation of the torch geometry for specific environmental pressures is as yet incomplete. However, many current GTAW systems could be readily adapted to plasma operation should the need arise, as is likely to happen as operational depths increase.

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III 3 WELD IMPROVEMENT TECHNIQUES

III 3.1 INTRODUCTION

In general, weld improvement techniques only benefit fatigue performance. In certain cases an improvement may be sought in the static strength or stability of a structural arrangement. With the possible exception of the use of weld reinforcement details, it is unlikely that a weld improvement technique can result in a reliable increase in static strength.

The following table gives an appreciation of various techniques which are presently available:

IMPROVEMENT	USAGE	NOTES
Toe grinding	Repair in air & underwater	Proven technique. Check CP requirements
Remedial grinding	Repair in air & underwater	Proven technique. Check CP requirements
TIG dressing	Good practice	May be mandatory for some weld procedures, especially process pipework in topsides
Plasma arc dressing	Good practice	as above
Post weld heat treatment	Good practice	PWHT is not generally used for weld improvement but it may form part of a larger repair scheme. PWHT confers improved fatigue resistance to thicker sections.
Hammer/shot/needle peening	Good practice	Although the technique shows promise, insufficient data is available for general and non-specific usage.
Weld reinforcement	Good practice	Unlikely to gain acceptance as a sole repair technique. Can have negative effects.
Step alignment	Good practice	Removing steps, esp. in butt welds so as to decrease the local SCF effect. (No benefit for root cracking.)

Table 3.1.1: Status of techniques

In this Section, attention will be focused on the two most widely used weld improvement techniques, namely weld toe grinding and remedial grinding. These two procedures are afforded special attention because the overwhelming evidence of test data shows that these two procedures can be used to improve fatigue damage to offshore structures.

Other weld improvement techniques, such as hammer peening (which does in fact have the promise of being even more beneficial), are addressed in Part IV but not here as there is insufficient published information to derive firm recommendations.

III 3.2 TOE GRINDING

Toe grinding is the deliberate removal of weld and base metal from the point at which the weld merges into the plate. The process is widely acknowledged as one of the principal techniques for the improvement of the fatigue life of welded nodes.

The aim of toe grinding is to remove imperfections from the region where many cracks form in order to take advantage of the enhanced fatigue life of the improved joint. Physical defects found at the toe of a weld include weld slag, porosity and undercut. In some instances the method can be used explicitly for repairs, where a surface crack has been discovered. In such an instance, the crack can be removed where the defect is less than 2mm deep and 5% of the steel thickness. In most cases the crack will not be detected at such a stage.

III 3.2.1 Life Improvement

Toe grinding leads to the use of a modified S-N curve. The effect of altered local SCF is not computed.

Toe grinding is successful because it removes defects which can initiate cracking and because it improves the local weld profile. The enhancement achieved by toe grinding of welds has been established[3.1] as:

$$\text{Improved fatigue life} = 2.2 \times \text{Basic fatigue life} \qquad \ldots.\ 3.2.1$$

where the basic fatigue life is the life calculated for the original as welded condition.

The above formulation has been verified as a result of the work reported in Part IV, Section IV 3.2. It should only be used for burr ground welds or where a milling tool is used. It cannot be applied when disc grinding is used.

The improvement in fatigue life due to weld toe grinding should be confirmed by giving specific consideration to the root of the weld. In multi-pass welds, it is often convenient to grind the whole weld profile in order to ensure that the minor weld toes are also machined. Such an operation is termed full weld grinding. Use the same design curve for full weld as for toe grinding.

It is common for many surface indications to be discovered during routine inspections. Determining just when a surface indication has become a crack is a problem faced by many inspectors. One solution is to give dubious regions a 'flash' grind on the spot, thus removing the inspection problem and undertaking a *de facto* toe grind repair at one and the same time. This particular practice perhaps leads to the position where only the larger categories of crack are reported.

III 3.2.2 Practical Issues

The depth of the cut relative to the thickness of the base metal is the key controlling parameter. The depth of the cut should be sufficient to remove some of the parent material.

The depth of cut into the local plate, x, is given by:

$$0.5\text{mm below any undercut} \leq x \leq 2\text{mm} \qquad \ldots.\ 3.2.2$$

A limit of 5% of the plate thickness is normally placed on the depth of the cut. The depth of cut will usually be less on the brace than on the chord size, as shown in Figure 3.2.1.

Localized relaxation of thickness requirements can be given where a toe is to be ground in order to remove a surface defect.

The use of a disc grinder results in scratch marks which run along the length of the weld seam. A tungsten burr grinder produces much smaller scratches which are aligned across the weld. Scratches produced by a rotary burr are less likely to lead to fatigue cracks because the principal stress in the weld is not aligned across the defect produced by the cutting operation. If a disc grinder is used, however, not only are the scratch marks deeper but they are also aligned across the direction of principal stress and are more likely to become crack initiation sites.

The requirement to fully grind the toes of a particular weld may not be justified. Where a complete fatigue analysis has been performed, data is available to pinpoint the location of fatigue damage.

In most cases, it is likely that the potential for fatigue damage will be concentrated in the saddle area of the joint. A study was made of the SCF's on selected, fatigue sensitive nodes on a typical North Sea steel platform. Stress concentration factors were computed for 180 individual tubular brace connections on the steel jacket which had a basic fatigue life below 100 years. The Efthymiou SCF equations, presented in the Appendix, were then used to refine the fatigue analysis by calculating more accurately the SCFs at typical joints. The calculations only considered T and Y type connections.

The results of the study are summarised in Figure 3.2.2. The graph shows that brace axial (AX) and brace out-of-plane bending (OPB) at the saddle point are most likely to produce high pseudo elastic stresses. The pseudo elastic stress range of a joint will depend upon two factors - loading regime and SCF effects.

Interpreting the results of the SCF distribution suggests that, if detailed fatigue analysis is available, then this can be used to select the portion of the weld that requires toe grinding.

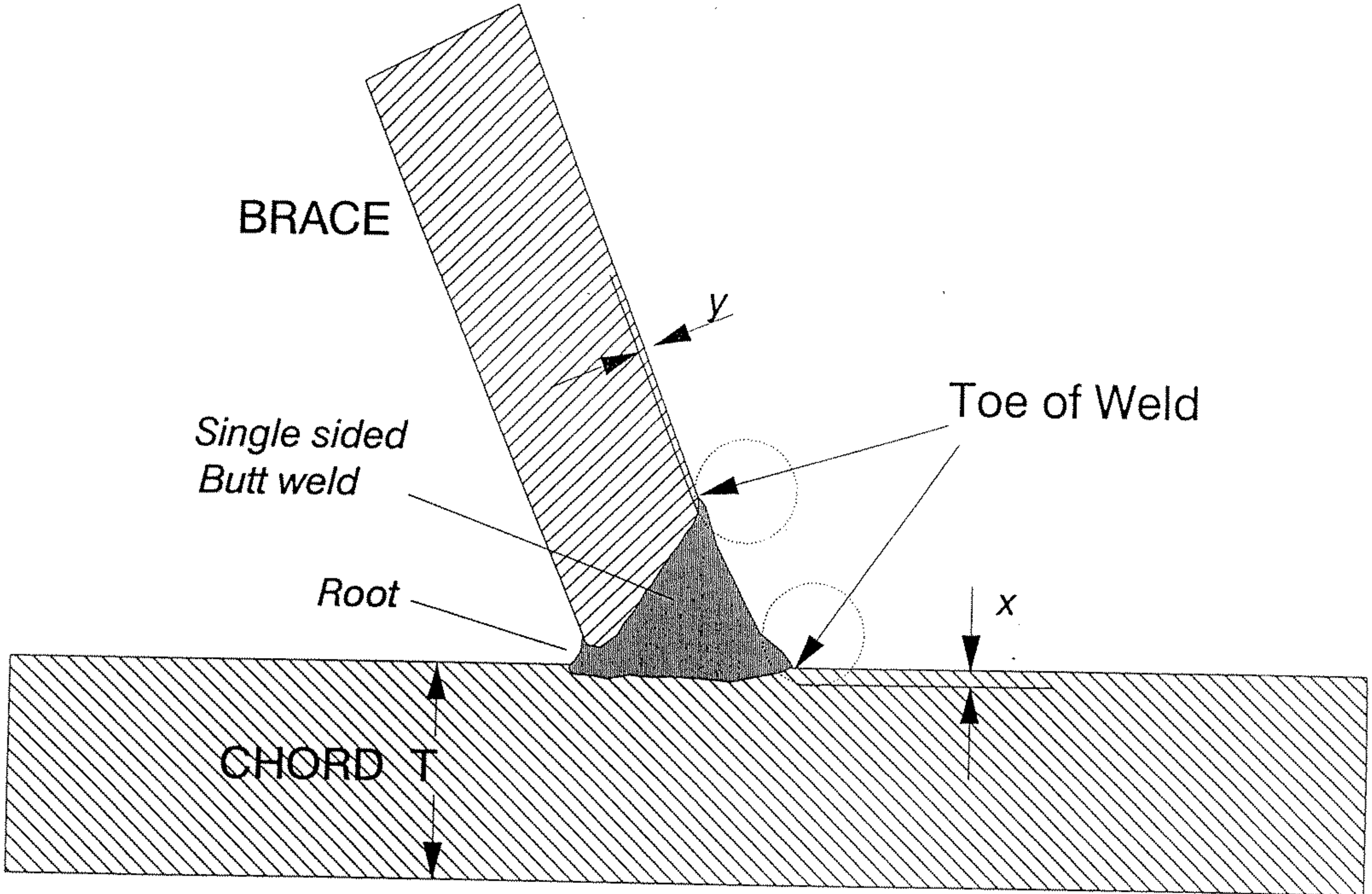


Figure 3.2.1: **Location and depth of cuts in toe grinding**

If there is a need to economise on toe weld grinding when a defect has been found then it would make sense to concentrate on areas which are :

- On the same side of the weld as the defect (brace/chord)
- In the quadrant covering the saddle region
- In the quadrant centred on the discovered defect.
- If an extra section of weld is to be ground then the weld which is on the opposite side of the brace to the defect should be chosen.

If a toe grind is to be made intermittent, then the run-out should be made on the face of the weld and not into the parent material.

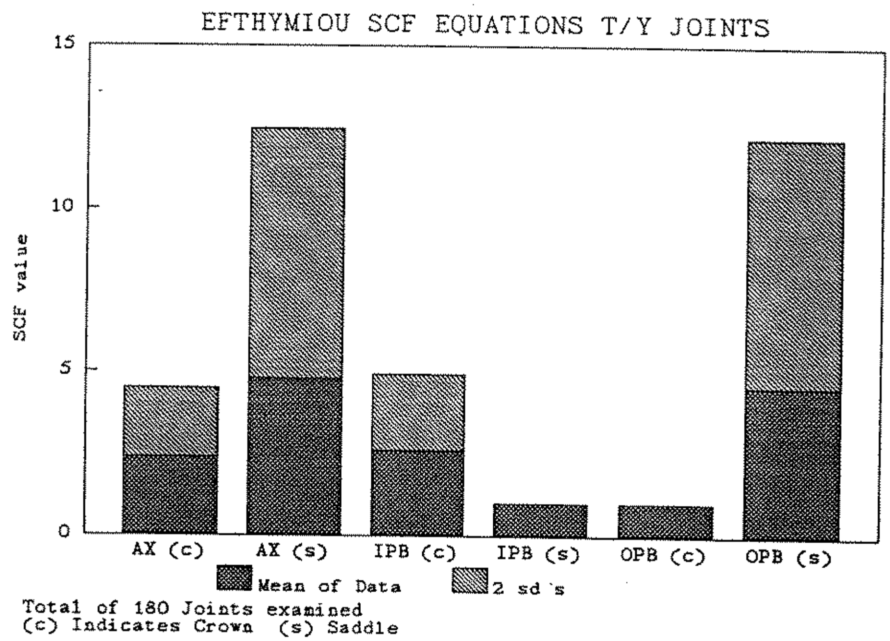


Figure 3.2.2: **Results of SCF value study in fatigue sensitive jacket nodes**

III 3.2.3 Inspection/Acceptance criteria

The grinding must cut into the parent metal for a distance of at least 0.5mm. Such a cut produces a trench which is 6.2mm wide on a flat piece of steel when using a cutting tool of diameter 20mm.

Standard practice dictates that the weld be profiled and then checked using MPI or dye penetration as appropriate. If the crack persists it is chased away until either the defect is removed or a deeper penetration is required. It is normally admissible to allow a cut of up to 5% of the plate thickness depth to be made to chase out a crack on a localised basis. If a deeper cut is required then the crack repair becomes a remedial grinding repair as detailed in Section III 3.3.

It is important that NDE of the finished weld should target surface defects in the toe grooves and root defects in single sided weld preparations. For many subsea joints, access will only be afforded to one side of the weld. Both MPI and ultrasonics will therefore be needed for practical underwater repair work. The MPI inspection is required to check out the surface condition of the weld/parent material whilst the UT is to confirm that the root of the weld is not also cracked.

Electropotential measurement methods are also recommended for tracing cracks in steel joints underwater.

III 3.3 REMEDIAL GRINDING OF CRACKS

Remedial grinding is one of the most common types of underwater repair work as it is standard practice to undertake remedial grinding of any joint found to be cracked.

The reason for the prevalence of remedial grinding is undoubtably associated with the comparatively short duration of planned underwater inspection programs for jacket structures.

Because cracked and subsequently ground joints have to be left 'pending' at the end of the inspection program they can form the only repair of that particular joint. However, the ground repair may subsequently form the weld preparation for a weld performed in a dry habitat, or it may be encapsulated by a bolted clamp.

Although the ground repair makes the joint weaker because steel is removed from a highly loaded part of a joint, it is much more resistant to fatigue effects than a cracked joint. This is due to the fact that any crack has an associated *initiation period*, and the simple act of grinding out an existing crack allows the clock to be wound back. Various test programmes as detailed in Part IV have sought to quantify the advantage of remedial grinding and a series of empirical equations have been developed for estimating the revised SCF of ground out weld cracks. A calculation of the remaining fatigue life of the joint can then be made. In some cases, the ground out weld can be left for the remaining life of the installation. (This approach is not the one recommended in Section III 3.3.1 below.)

Ground repaired cracks should not be left on structures which are subjected to underwater corrosion. This type of repair only works in air or underwater in regions covered by the CP system. Remedial grinding involves the excavation and removal of a crack sited at or near a welded joint by cutting a smooth shaped trench in the cracked metal, see Figure 3.3.1.

The bulk of the removed material would normally be removed with a heavy grinding disc or milling tool and the profile of the trench would be improved by machining with a burring tool. In thin plate sections (T < 15 mm) only the burr grinding tool would be employed.

On completion of the first pass of grinding, the finished profile is checked. The ground area is extended until all traces of cracks have been removed. It is good practice to extend the grinding a distance T (T= plate thickness) past the end of the crack so that sub surface defects are removed. Extending the groove around the weld is not recommended, but the engineer should advise the toe grinding of other portions of the subject weld if there is concern about fatigue in other sections of the weld.

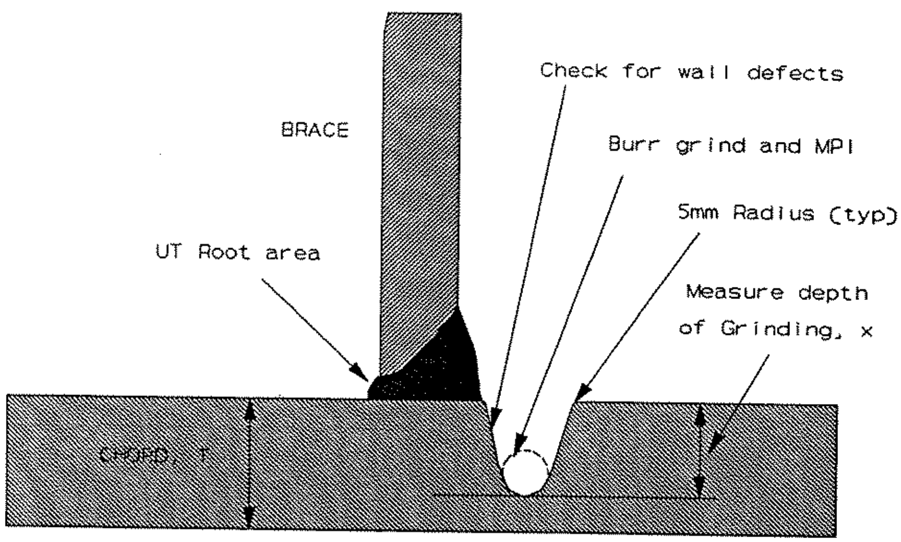


Figure 3.3.1: Remedial ground weld detail showing inspection points

III 3.3.1 Life Improvement

As a result of the studies conducted in Part IV, Section IV 3.3, the improvement in fatigue life due to remedial grinding is found to be the same as that afforded by toe grinding. Although this is not unexpected for very shallow remedial grinding (in that it becomes, in effect, a toe grinding operation), this is rather surprising when considering deep cracks. Measured stresses in the excavated groove are found to be a function of groove depth yet the fatigue life appears independent of it. The fatigue life may therefore be estimated by the same relationship which applies to toe ground joints, ie:

$$\text{Fatigue life} = 2.2 \times \text{Basic fatigue life} \qquad \text{.... 3.3.1}$$

where the basic fatigue life is the life calculated for the original as welded condition.

In addition to assessing the fatigue strength, a check should be made on the ability of the joint to resist static loads as given in the next subsection.

III 3.3.2 Static Strength Considerations

Static strength calculations should set a limit on the maximum amount of material that can be removed in remedial grinding as removing material from the node/brace interface weakens the joint.

Static strength calculations should be based upon storm wave loading with a return period which is relevant to the remaining life of the structure considered. In many instances there will be no justification for using an extreme storm wave loading which is less than that used for the original design. However, there may be a strong case for using a reliability based technique to compute the combined effect of wind, wave and current loads.

The repair will have taken place on the brace side or on the chord side of the connection. Brace side repairs need to be checked against one set of criteria, and chord side another. If both types of repair have been made then both checks need to be made.

III 3.3.2.1 Brace side grinding

If the remedial grinding has occurred in the brace connection, then the static strength check should concern itself with the evaluation of the element code-check. The reduced form of the equations should be used.

The minimum cross section formed by the groove should be sketched, and the elastic section properties computed for the reduced area. If the groove has been cut so that it lies at an angle to the true cross section, then the section properties should be computed for the oblique section and then transformed to the cross section.

The section properties are based upon a tubular of constant diameter and thickness. Where the section has been ground, a defect of a certain depth and length will have been introduced. This part of the tube will be weakened. The axial area and section modulus of the weakened section will need to be re-computed. Account will need to be taken of the change in the position of the neutral axis caused by the loss of axial area.

Note that punching shear checks are not necessary for reduced brace sections as the punching shear stress checks are determined by brace side forces and not brace side stresses.

III 3.3.2.2 Chord side remedial grinding

The chord barrel element should be checked for strength (ie. perform unity checks using reduced section properties as for Section IV 3.3.2.1 above). In most cases, the chord barrel will have been purposefully thickened for fatigue/punching shear strength and often this particular check can be made by inspection.

The capacity of the joint may be severely reduced, and may lead to a position whereby the joint is unable to support the long term storm loadings. In such a case, the remedial grind operation may only be justified as a short term expedient, with a more permanent repair developed at a later stage.

Several possible calculations options are outlined for tubular joints - the choice of a particular method depending on the severity of the grinding and the stress condition of the joint.

Option A: Basic

The assumption in this method is that the capacity of the joint is based on a chord wall thickness equal to the thinnest remaining thickness following grinding. The capacity checks should be satisfied for axial, in-plane and out-of-plane bending separately. In addition, the requirements for combined axial and bending should be met.

Option B: Simple

This is similar to Option A except that the axial capacity is estimated by down-grading the original (as-welded) capacity by the ratio of the reduced punching shear area to the original punching shear area. Again, individual and combined load actions should be checked.

Option C: Advanced

Compared to Option B, the moment capacities are now refined. It is suggested that the original capacities are down-graded by the ratios of the elastic moduli of the reduced to the original punching shear area. The basis of the calculation should involve integration of the section and take into account the position and extent of the groove.

FE modelling or other advanced techniques can be used to determine the static strength of the repaired joint. Care should be taken to ensure that the area of the groove is modelled in sufficient detail as to enable realistic results to be obtained from the analysis. Failure criteria should be established before the analysis is run.

III 3.3.3 Practical Issues

Remedial grinding is likely to be confined to areas of a jacket structure where fatigue damage occurs. In broad terms, the conductor guides, conductor guide framing nodes, caisson and riser supports between elevations LAT -15.0m and LAT +12.00m are often the affected areas.

Grinding work will generally be within the range of air divers, although deeper work would need to be carried out from a diving bell. Environmental conditions will probably restrict the operations to periods of low wave height (2m significant sea state). In certain coastal sea areas, shallow diving is restricted by sea currents and may only be possible for short periods around slack water.

Remedial grinding of cracked welds is performed using a combination of two tools:

- Tungsten tipped rotary (or "Burr") grinder

- Disc grinder (gritstone in epoxy matrix type)

When the work is in air, standard power tools are employed for remedial grinding, with the bulk of the excavation being made by the disc grinder, and the finish completed with the rotary burr tool. Depending on defect size, location and operator preference, all the work may be accomplished using the rotary tool.

Grinding work on the topsides of existing structures will be the subject of normal 'permit to work' practices. It will be necessary to select tools which come within the specification for topsides' working. The details are platform, and sometimes company, specific.

For repairs underwater, similar techniques are employed, but specialised grinding tools are required.

Underwater grinding equipment is readily available. Standard units are powered by hydraulic pressure. Other power sources are rarely used for reasons relating to operator safety. The rotary head (or burr) grinder should be operated at a speed in the range 30,000 to 45,000 rpm (see, for example, Section 2.1.6 in Tubby[3.2]) in order to achieve the desired effects. Surface trials should be undertaken to check the finish produced by an individual power tool / cutting head arrangement. The speed of operation of a revolving power tool will generally be lower in water than in air for a particular power input.

The rotary tool head size should be 20mm diameter. A smaller sized head may be used with thinner ($t < 20$) plate dimensions. The larger tools may be operated at lower (rpm) speeds. Trials or operator experience may assist the selection of the optimum tool for a particular operation.

Grinding a joint in air demands little preparatory work, save to ensure that the work is adequately supported. For underwater repairs the position is completely changed. If a diver is to provide a reaction to a high powered grinding tool then a demountable cradle or strap must be provided.

Marine growth must be removed from the target area to be repaired.

Underwater cathodic protection is important in stopping corrosion acceleration which could otherwise adversely affect the fatigue life of a cracked detail. If a platform uses an impressed current system then there is the possibility that corrosion protection will not be obtained at all times. Corrosion fatigue damage can mean that theoretical fatigue improvements associated with remedial grinding are not realised in practice.

Remedial grinding is time consuming work. The defect needs to be chased out, smooth ground, inspected and then the whole process repeated if any remaining cracks are found. The time taken to perform operations underwater may be significantly longer than those recorded for work in the dry. Account should bare taken of the time required for reaching and identifying the work place, setting up the supports, lowering equipment, taking photographs and the exchange of divers. A working knowledge of diving tables and associated practices may be required to estimate the duration of a particular piece of work. It is often found that the actual work (ie grinding the joint) is a minor element of the total time required for the job to be completed. However, the time to grind the work may be estimated from tool cutting times, and an estimate can be made based on expected crack length and depth of excavation.

III 3.3.4 Inspection/Acceptance Criteria

Remedial grinding must be allied with a surface crack inspection scheme. For work above water, either dye penetration or MPI methods are used. The crack grinding proceeds until no further defects can be observed in the parent material. Dye penetration tests are particularly useful as a skilled operator can chase out the visible dyeline under good lighting conditions. However, underwater repairs will generally be carried out using MPI techniques for crack inspection.

Inclined cracks can pose a problem as the underlying defect may extend well past the originally discovered surface cracks. Ultrasonic probing may be advised for mapping the possible extent of the cracked region before any grinding is attempted. ACPD and ACFM methods may be used underwater.

On completion of the repair, the groove must be carefully ground out to form a smooth root profile as shown in Figure 3.3.2. The groove should be formed with a burr grinding tool with a diameter of 20mm. Other grinding tools may be required to smooth the sides and transitions of a deeper repair groove.

The repair can only be accepted if the root, wall and transition sections are found to be free from defects. Video assisted inspection is normally required. Still photography should be used to produce record photographs.

A cast sample may be taken of the repair, using, for example, silicon.

Ultrasonic testing is advised on 100% of the root area of any single sided closure weld which is the subject of remedial repair on the external root only. This is because the improvement of the toe area of the weld will do nothing for the any cracks formed in the root of the joint.

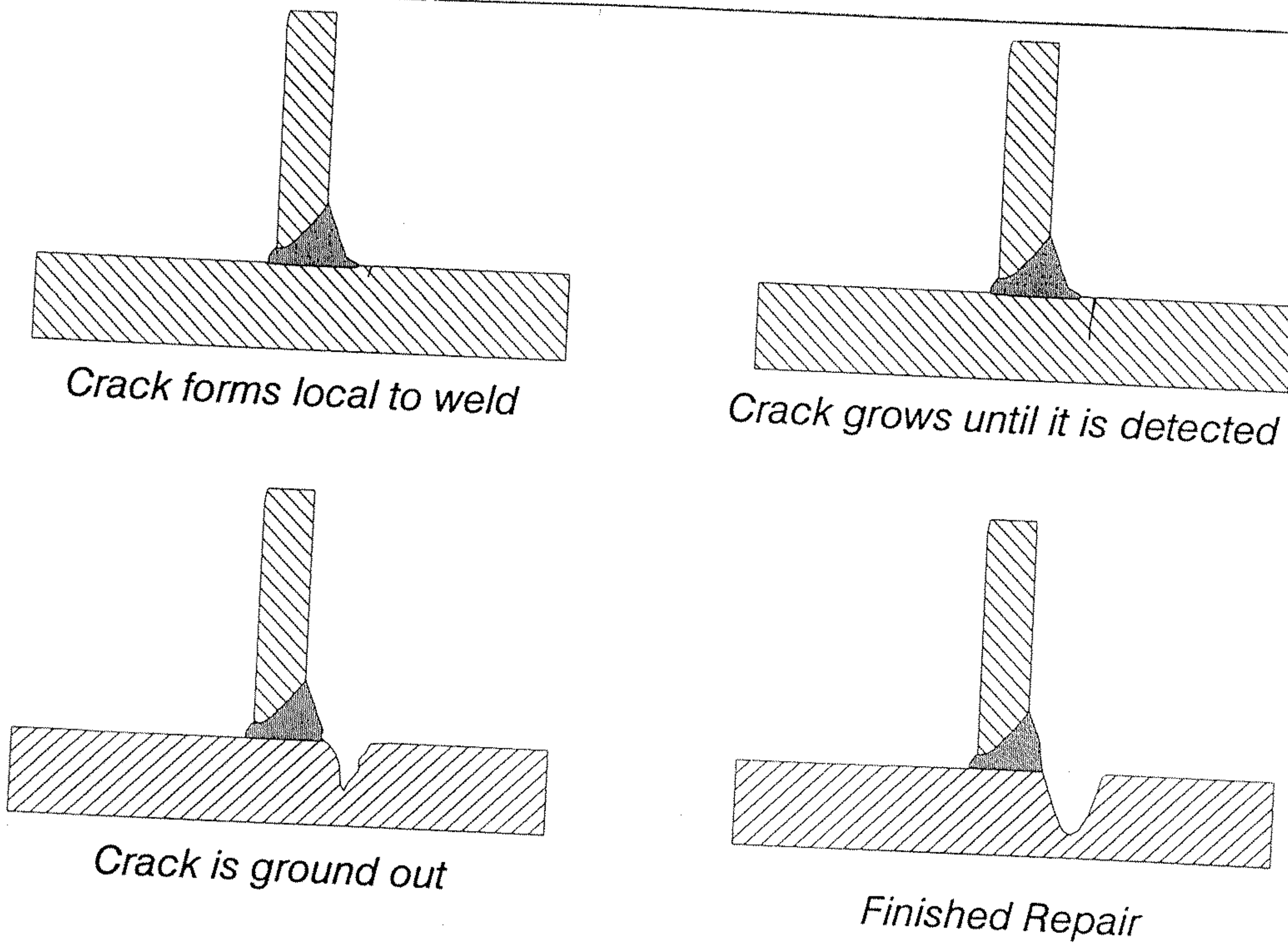


Figure 3.3.2: Remedial grinding - Completion of Repair

REFERENCES

3.1 Health and Safety Executive. "Offshore installations: Guidance on design and construction", HMSO, London.

3.2 PJ Tubby "Fatigue performance of repaired tubular joints" by The Welding Institute for the UK Department of Energy. OTH 89 307. HMSO, London 1989.

III 4 CLAMP TECHNOLOGY

III 4.1 INTRODUCTION

III 4.1.1 General

This section describes and gives guidance on the design of clamps for the repair and/or strengthening of steel jacket structures. It is important to note that although design guidance is given in some detail, the success in utilising clamping techniques is largely dependent on the design contractor's experience. The following techniques are considered:

- stressed mechanical clamps (steel friction clamps)
- unstressed grouted clamps/sleeve connections
- stressed grouted clamps
- stressed elastomer-lined clamps

Each of the above is an established technique, often used for static or fatigue strength improvements of members and joints. Clamps are also used in repair schemes that require the introduction of additional bracing members to the structure. Considerable research has been conducted over the past two decades, and significant data has been generated for most clamp types. This data have allowed the formulations for design to be developed. Clamp components such as saddles, side plates, stiffeners, flange plates, end capping plates, long studbolts, nuts and spherical washers are common to each clamping technique. Sleeve connections are made up of saddles (continuous or split) and use short bolts. Figure 4.1.1 shows the various components for clamps and split sleeve connections.

The design procedures that are given in the following sections are, in general, common to all clamp types and sleeve connections. Where there are differences in the procedures, these are clearly highlighted. The design procedures do not cater for clamps having 'discontinuous' flange plates, see Figure 4.1.2, or 'fin' plates. There are no test data available to permit design formulations to be verified for these clamps. Furthermore, fin plates offer very little extra rigidity against in-plane and out-of-plane bending which generally preclude them from design solutions.

General factors that affect design are the magnitude of loads, geometry and stiffness of the clamp/tubular/studbolt, studbolt load, coefficient of friction, shear key geometry, grout strength etc. However, the design of clamping techniques is not only influenced by the ability of the technique to carry load, but also by installation requirements and access. Therefore at all stages of design, the engineer must be aware of fabrication and installation tasks. Often these dictate the design.

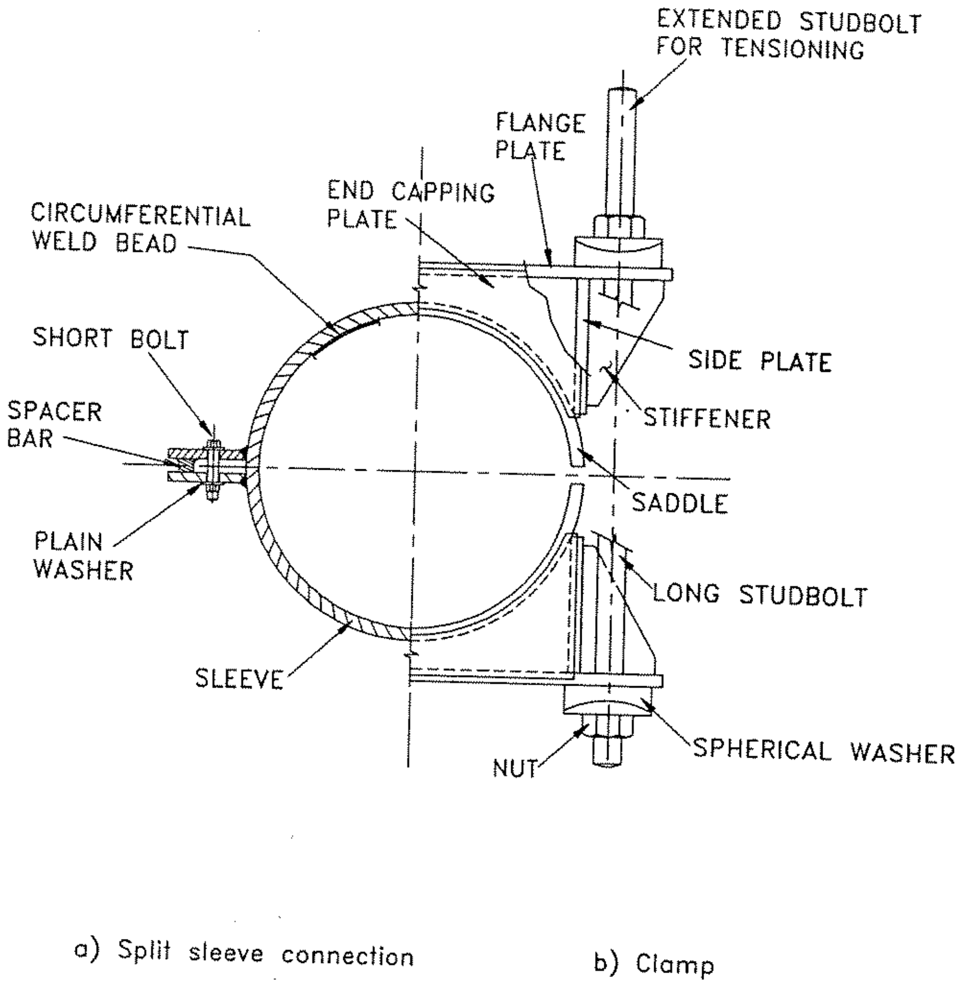


Figure 4.1.1: **Definition of steelwork for clamps and split sleeve connections**

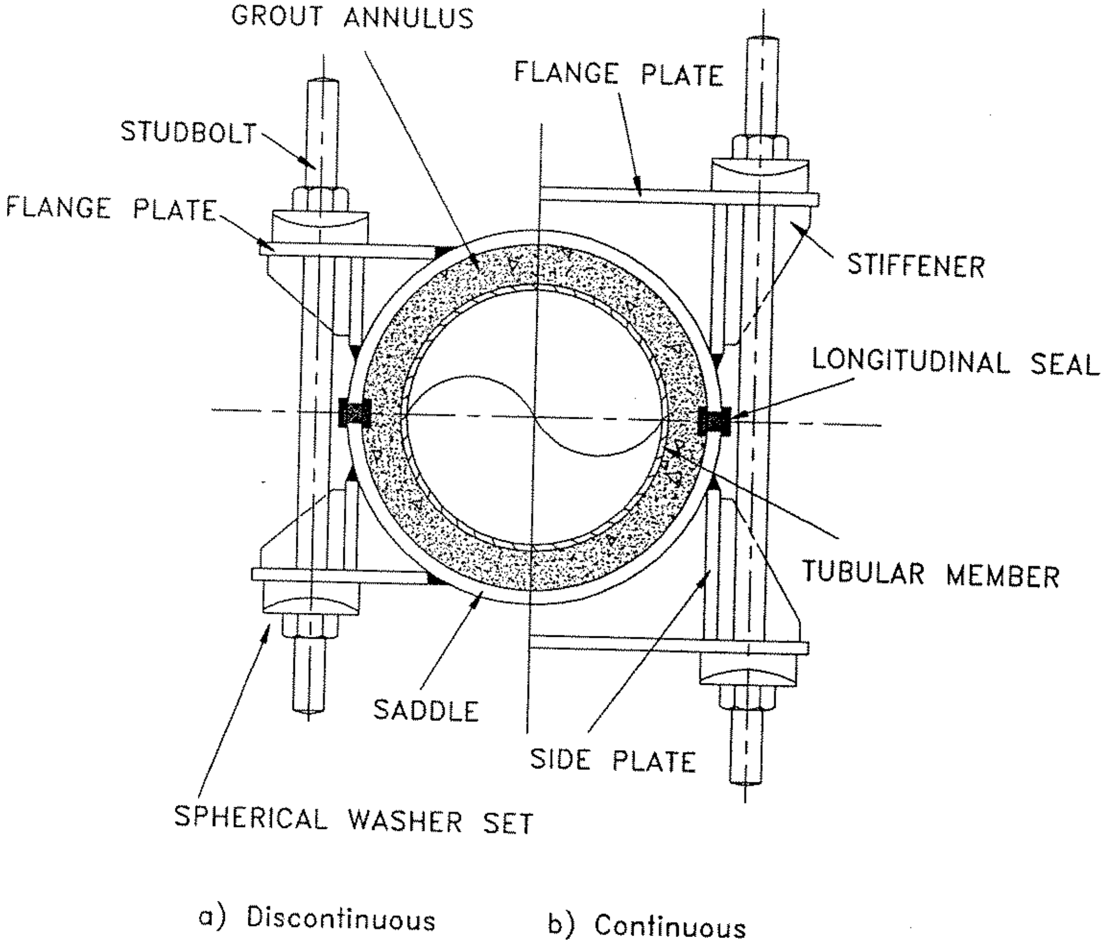


Figure 4.1.2: **Discontinuous and continuous flange plate clamp designs**

It is common practice to design for no load sharing between the clamp and the parent steelwork ie. it is assumed that the member is severed and that **all** the load must be transferred to the clamp and 'bridged' over the damage. It is recommended that preference be given to a 'no load sharing' design because all clamping techniques completely cover the parent steelwork, preventing the possibility of any future inspection/monitoring of the damage. For this reason it is not recommended to load share for a fatigue strength problem. However, in the case of strengthening or repair for static strength reasons, the engineer may wish to investigate load sharing.

The design recommendations and equations given in the following sections are based on 'continuous' flange plates. No test data are available for 'discontinuous' flange plates and thus the design recommendations and equations cannot be verified for such flange plate arrangements. Nevertheless, discontinuous flange plates have been used, sometimes in conjunction with circular strongbacks and/or thickened plates.

It is particularly important to conduct an underwater survey of the repair site. The survey needs to be especially accurate for cases where stressed mechanical clamps are to be deployed. The survey must encompass several linear and angular measurements of the repair site. Diameters across a number of clock positions, circumference measurements, deviations from the horizontal or vertical are typically required. Angular measurements for in-plane and out-of-plane braces must be recorded, although it should be noted that a small error in the angular measurement can result in a large linear error at a distance away from the joint, as shown in Figure 4.1.3. Therefore for long clamps, or where design tolerances are tight, linear measurements are recommended and these can be verified by angle measurements. It is particularly important to accurately locate the positions of section changes, welds, doubler plates, anodes or any other obstructions at or near the repair site. For stressed mechanical clamps additional measurements of ovality and out-of-roundness must be taken at a number of locations.

It is also recommended that onshore fit-up trials and diver training trials be performed. These are very useful in instances where the repair site is complex and/or access to the site is difficult.

III 4.1.2 Stressed Mechanical Clamps (Steel Friction Clamps)

A stressed mechanical clamp (also known as a friction clamp) comprises two or more segments of closely fitting stiffened saddle plates, stressed directly onto a tubular section by means of long studbolts. The strength of such a repair scheme is obtained from the steel to steel friction which is developed by means of external bolt loads. The strength is therefore dependent on the magnitude of the normal force and the effective coefficient of friction between the two steel contact surfaces. Surface condition of the tubular section is therefore an important factor, and cleaning to bare metal is normally a requirement. Figure 4.1.4 illustrates the various components of a mechanical clamp.

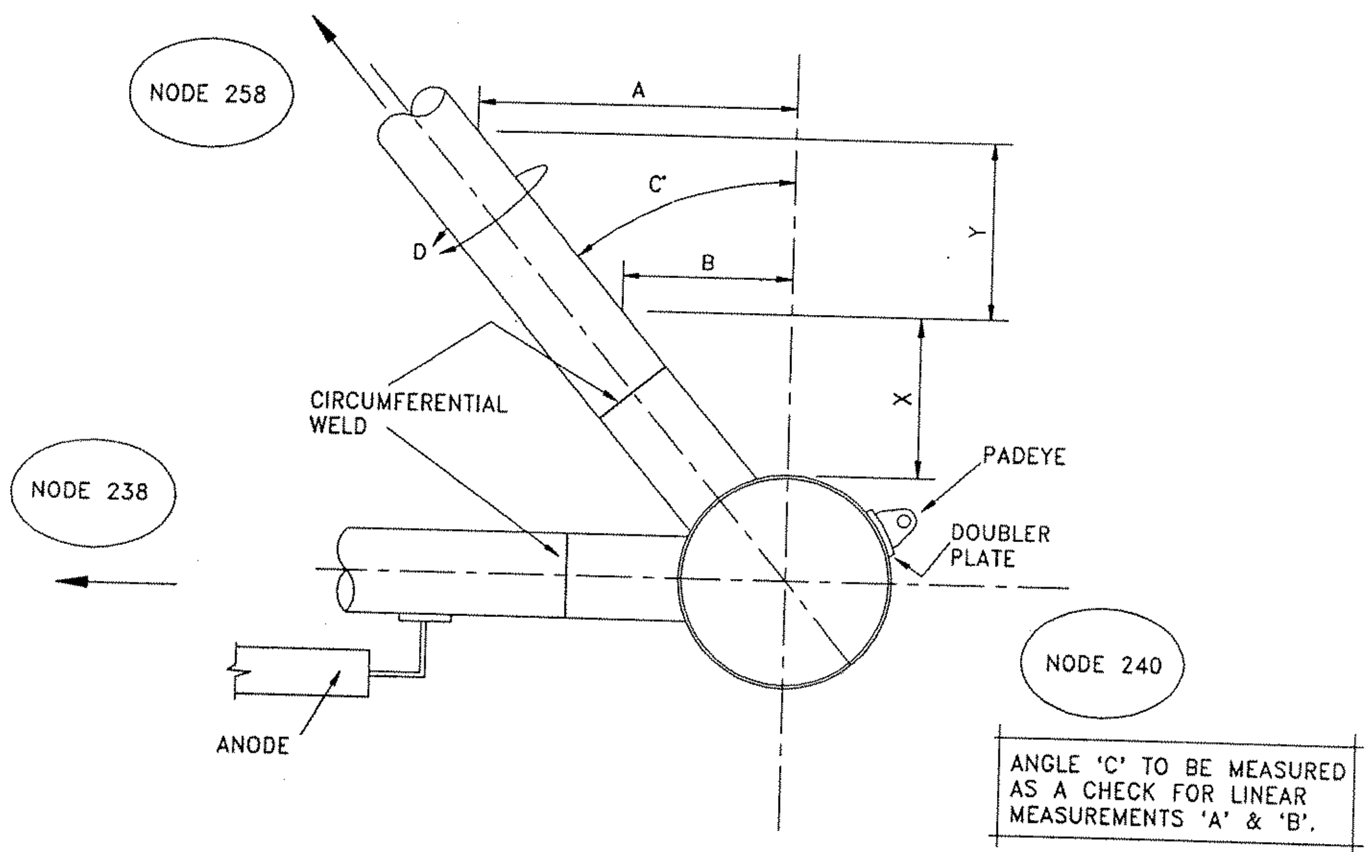


Figure 4.1.3: **Linear/angular measurements for an offshore survey (also shown are some potential obstructions)**

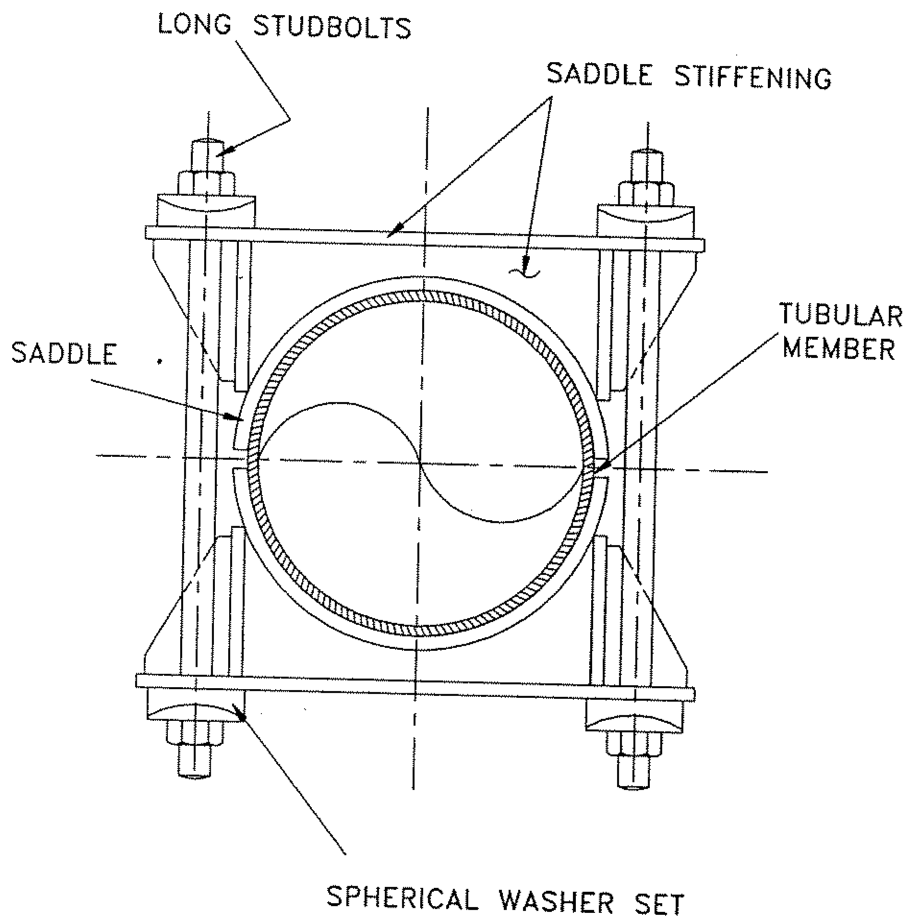


Figure 4.1.4: Typical stressed mechanical clamp

The major advantage of a stressed mechanical clamp is that large forces can be transferred through friction over a short clamp length, provided the existing tubular member is able to resist hoop buckling forces induced by bolt loading. Stressed mechanical clamps rely on close tolerance steel-to-steel contact between the tubular member and the clamp saddles and therefore require extremely accurate offshore surveys of the contact zone. Further, tight tolerances for the fabrication of the clamp are a necessity. Unless these concerns are resolved, stressed mechanical clamps are not generally suitable for the strengthening and repair of tubular joints. However, they can be used for connecting new members, or for the strengthening and repair of intact and damaged members.

III 4.1.3 Unstressed Grouted Clamps/Sleeve Connections

An unstressed grouted clamp or sleeve connection comprises sleeves which are placed around a tubular section with the annular space so created filled with grout. The sleeves may be split, as in an unstressed grouted clamp, or continuous as in a pile/sleeve connection. For split sleeves, short bolts are generally specified and these bolts are tightened prior to injection of grout. Figures 4.1.5 and 4.1.6 show typical details of an unstressed grouted clamp and sleeve connections.

Load transfer between the tubular section and the clamp or connection is by means of a combination of chemical bond, friction and mechanical interlock between the grout/steel interface. Often it is necessary to use a substantial clamp or connection length to generate sufficient load transfer capacity. The provision of shear keys or weld beads can increase clamp capacity, but the need for underwater welding greatly increases the cost of this option. Unstressed grouted clamps and connections do, however, offer a versatile means for strengthening or repair of tubular joints and members since they do not require stringent offshore surveys associated with stressed mechanical clamps. A major advantage of unstressed grouted clamps or sleeve connections is that both angular and translational tolerances can be readily accommodated within the grout annulus.

Unstressed grouted clamps may be used to strengthen or repair an existing tubular joint subjected to static and/or fatigue loads, or to facilitate the attachment of a new member to the structure. Sleeve connections can be used to facilitate the attachment of a new member to the structure by providing length and fit-up adjustment by either a retractable sleeve which is slid over from one segment to the other segment of the new member, or by a telescopic sleeve whereby the member is installed into location as a single piece. Alternatively, a new member may be attached to the structure by creating a new joint. Sleeve connections may also be used to strengthen or repair members by the use of a steel 'bandage' in order to enhance stability against local or overall buckling.

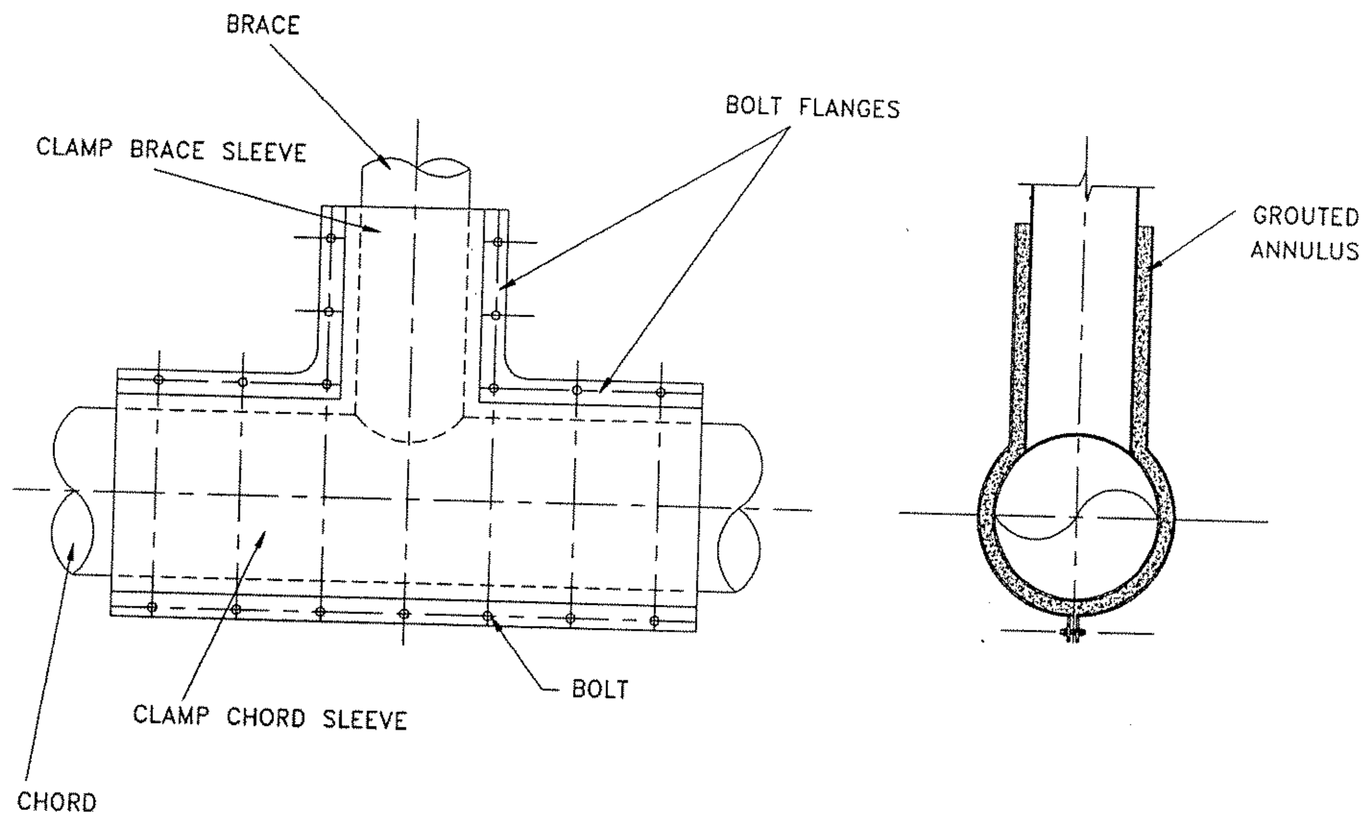
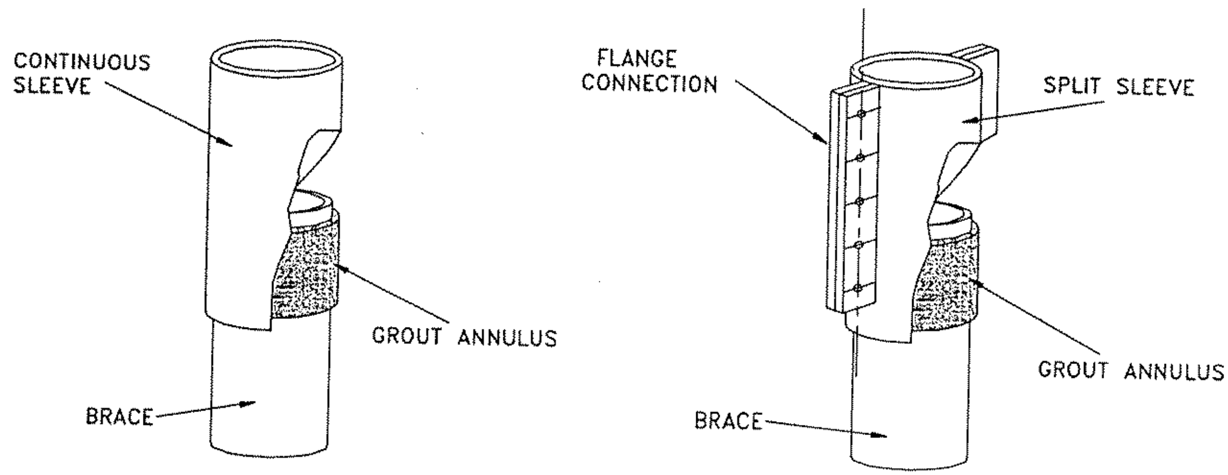


Figure 4.1.5: Typical unstressed grouted clamp



a) Continuous sleeve grouted connection.

b) Split sleeve grouted connection.

Figure 4.1.6: **Examples of typical unstressed grouted sleeve connections**

The strength of unstressed grouted clamps/sleeve connections is dependent on grout strength and elastic modulus, tubular surface condition and radial stiffness, shear key geometry and the total load transfer length.

III 4.1.4 Stressed Grouted Clamps

A stressed grouted clamp is formed when two or more segments of strengthened saddle plates are stressed by means of long studbolts onto a tubular section after grout has been injected and allowed to cure in the annular space between the clamp and the tubular, see Figure 4.1.7. This form of clamp is a hybrid between a stressed mechanical clamp and an unstressed grouted clamp. Strength is achieved from a combination of bond and grout/steel friction.

Stressed grouted clamps offer the benefits of high strength-to-length ratio and the ability to absorb significant tolerances. This form of clamp is therefore very popular and is commonly used for strengthening and repair of offshore structures.

The design of such a clamp is dependent on the surface contact force due to studbolt load, tubular section surface condition, number of active friction faces, clamp/studbolt/existing member geometry and stiffness and the total clamp length.

III 4.1.5 Stressed Elastomer-Lined Clamps

Stressed elastomer-lined clamps are similar to stressed mechanical clamps, except that an elastomer lining is bonded to the inside faces of the clamp saddle plates, see Figure 4.1.8. The liner is generally made up of solid polychloroprene (neoprene) sheeting. The strength of an elastomer-lined clamp is dependent on the magnitude of external bolt loads and the effective coefficient of friction between the liner/steel interface. The use of an elastomer offers a certain degree of translational and angular tolerance.

Elastomers, even when confined, are significantly less stiff than steel. The use of this type of repair technique is limited therefore to where stiffness is not critical to its effectiveness.

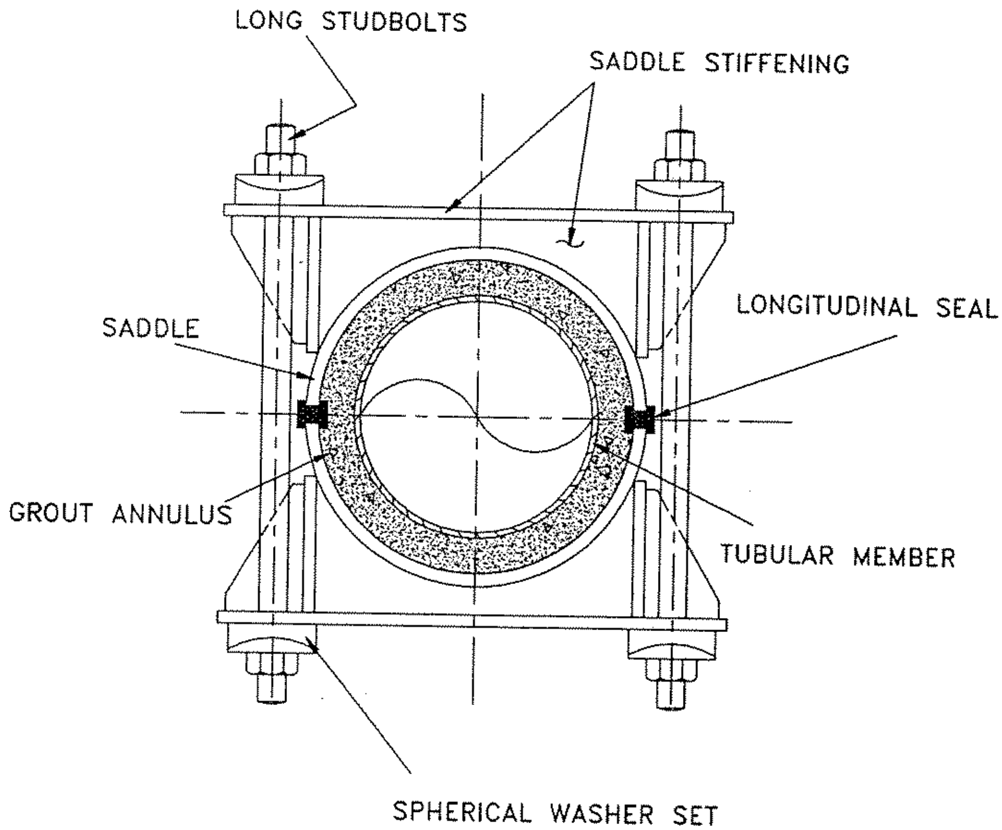


Figure 4.1.7: Typical stressed grouted clamp

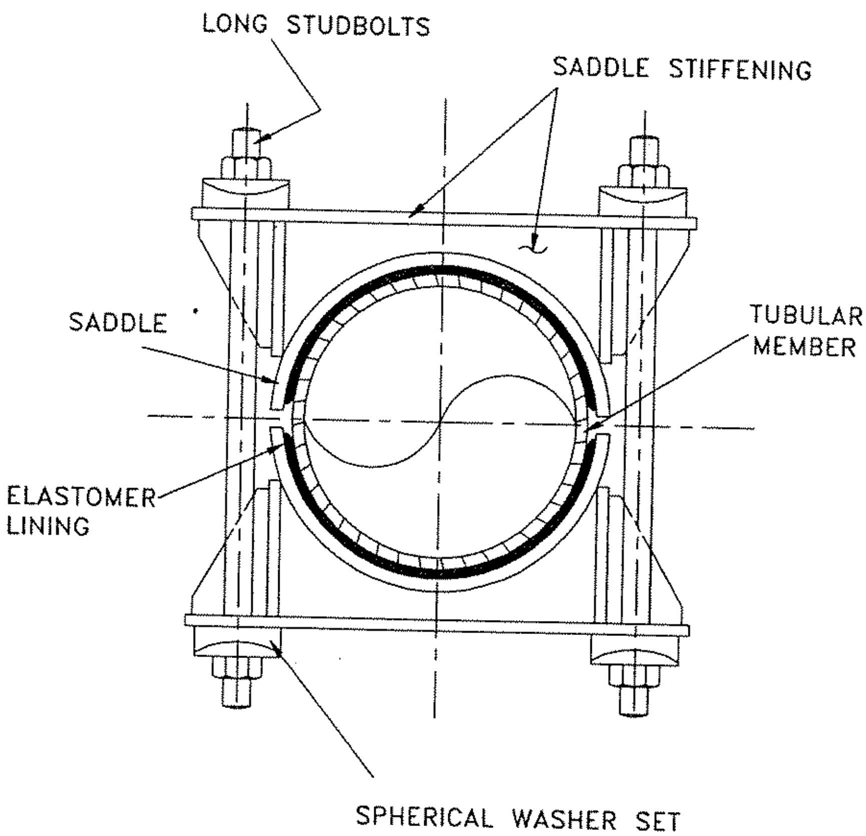


Figure 4.1.8: **Typical stressed elastomer-lined clamp**

III 4.2 DESIGN OF CLAMPS

III 4.2.1 <u>General</u>

This section presents procedures and guidance for the design of clamps. Detailed guidance is presented for clamp-specific requirements, but simple structural design of standard components is not addressed in detail. It is therefore assumed that the engineer is competent in the design of such standard components.

In Section III 4.2.2, the design steps are presented in tabular form. This table clearly shows that much of the design process is common to all clamp types; where variations occur, they are indicated. Subsequent sections give design guidance in a logical sequence which may be used for detailed engineering. For concept design, a number of the steps may be omitted.

The following guidance reflects best current practice based on substantial background data and assessments which are given in Part IV of this document. It should be noted, however, that some detailed aspects of design require optimisation, and in such cases, the guidance adopts a conservative approach. A number of design equations have been developed on the basis of background data, and the engineer is therefore further referred to Part IV of this document, should he wish to examine the development of such formulations.

It is important to state that the engineer must be constantly aware of the buildability and installability of the clamp. Compared to the costs of offshore activities, design and fabrication costs are comparatively small and therefore, the engineer should attempt to design the structure such that installation is simplified. Further, the engineer should not over-specify fabrication requirements, eg. by calling for stringent dimensional tolerances and the use of full penetration welds, unless absolutely necessary.

Information from the Client can often assist the design. For example, when checks are performed for hoop crushing, the Client may hold certification that indicates that the yield strength of the tubular being clamped is considerably in excess of the minimum required. This information can assist the engineer in some cases. As-built data, underwater video records and offshore survey information may also be obtained through the Client.

III 4.2.2 Design Procedure

Table 4.2.1 shows a logical sequence for the design of all clamp types. Examination of this table indicates that the design procedure is similar for most clamp types. The interaction of the various steps in the design process is illustrated in Figure 4.2.1. Both slip and prying criteria have to be satisfied. Note, it is not necessary to follow the sequence in order; sometimes it is useful to consider member crushing first so that provisional clamp lengths can be established quickly.

See Section	Item	Clamp type[1]				
		smc	ugc	ugs	sgc	selc
III.4.2.3	Problem appreciation	✓	✓	✓	✓	✓
III.4.2.4	Select provisional clamp geometry	✓	✓	✓	✓	✓
III.4.2.5	Determine acting slip stress	✓	✓	✓	✓	✓
III.4.2.6	Sélect studbolts and check slip strength	✓	✓	✓[2]	✓	✓
III.4.2.7	Check separation forces on clamp halves	✓	✓	X	✓	✓
III.4.2.8	Finalise studbolt loads	✓	✓	X	✓	✓
III.4.2.9	Check members for hoop crushing/yielding	✓	X	X	✓	✓
III.4.2.10	Check adequacy of grout	X	✓	✓	✓	X
III.4.2.11	Check adequacy of polychloroprene	X	X	X	X	✓
III.4.2.12	Design clamp steelwork	✓	✓	✓	✓	✓
III.4.2.13	Design grout seals	X	✓	✓	✓	X
III.4.2.14	Design installation aids	✓	✓	✓	✓	✓
III.4.2.15	Fatigue checks	✓[3]	✓[3]	✓[3]	✓[3]	✓[3]
III.4.2.16	Corrosion protection	✓	✓	✓	✓	✓

Notes:

1) Clamp types:
- smc - stressed mechanical clamp
- ugc - unstressed grouted clamp
- ugs - unstressed grouted sleeve connection
- sgc - stressed grouted clamp
- selc - stressed elastomer-lined clamp

2) Check slip strength only

3) If required

Table 4.2.1: Clamp design procedure

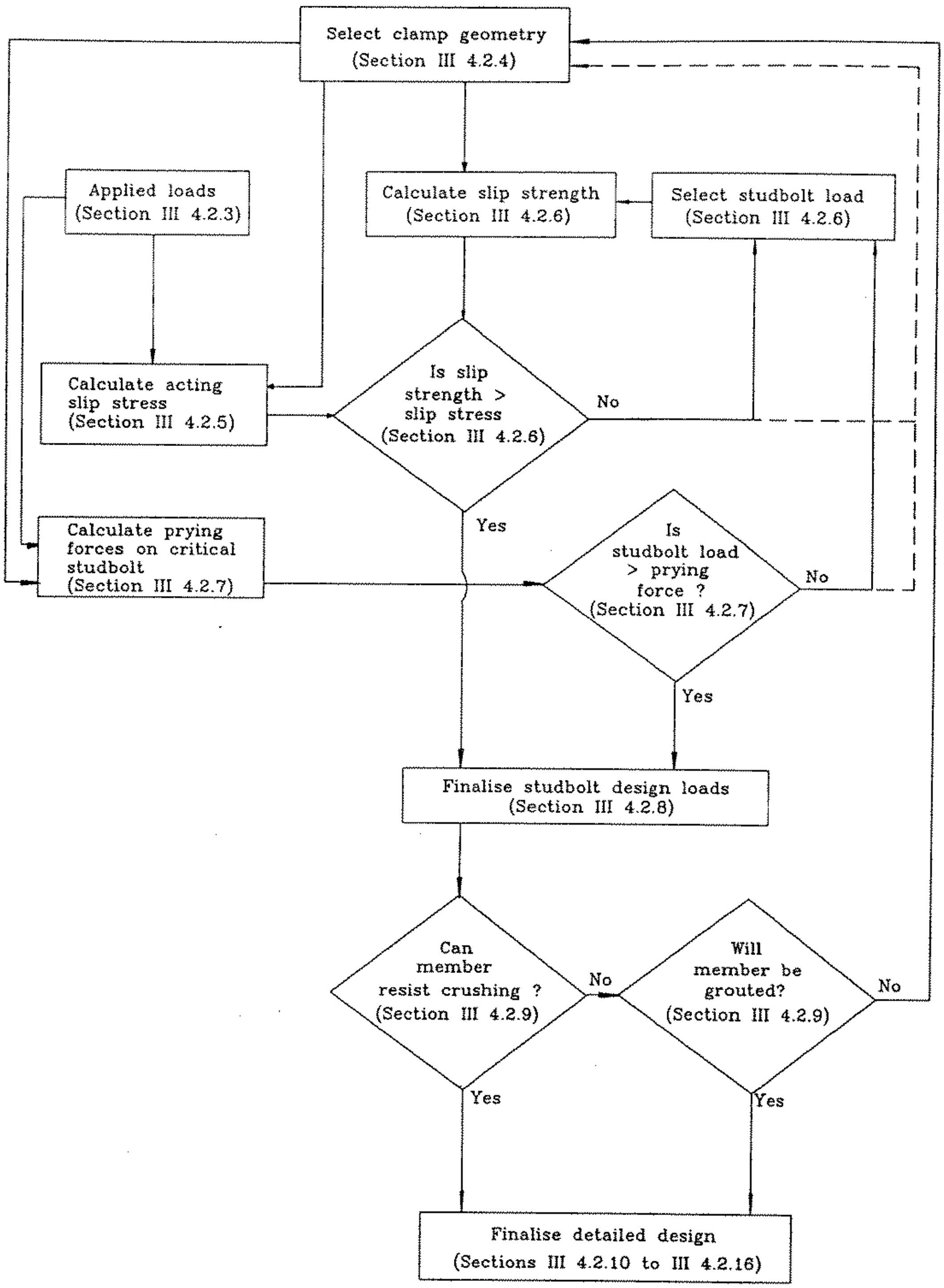


Figure 4.2.1: Interaction of clamp design processes

III 4.2.3 Problem Appreciation

Design of a clamp repair solution entails an understanding for the overall problem which includes structural loads, structure geometry and location of damage. Each of these, together with engineering judgement, will determine the final optimum solution.

Design of individual components will influence each other and the overall clamp geometry.

III 4.2.4 Selection of Provisional Clamp Geometry

Once an appreciation of the problem has been achieved, then the provisional clamp geometry may be selected to enable preliminary design to proceed. This will include clamp cross-section(s), clamp length(s), bolt spacing and, depending on clamp type, grout annulus size and weld bead size.

The geometry of the clamp may be dictated by limitations on space. When this is the case, careful consideration must be given to bolt spacing and subsequent stiffener spacing, to alleviate access difficulties during fabrication and clamp installation. Consideration should also be given in the selection of clamp geometry, so that it satisfies geometric limits of the design equations.

If a survey is to be carried out, then it is important to specify linear measurements with verification using angular measurements. Measurements should be triangulated and when possible, survey results should provide more than one way to calculate any one dimension. If accurate survey results are unavailable then this may determine the type of clamp repair that may be adopted.

III 4.2.5 Determination of Acting Slip Stress

III 4.2.5.1 General

An understanding of the critical loads is required since these will dictate the required slip resistance of the clamp. Loads which will contribute to the acting slip stress are namely axial load and torsion. Bending loads tend to enhance the slip capacity and therefore may be conservatively ignored in determining the acting slip stress.

The clamp will fall into one of two configurations, either a repair clamp to an existing structure or an addmember as presented in Figures 4.2.2 and 4.2.3, respectively. The philosophy adopted herein can be equally applied to one or the other. Firstly, the acting slip stress for a repair clamp will be considered.

III 4.2.5.2 Repair clamp

Figure 4.2.2 presents the applicable loads for one leg of a repair clamp. A repair clamp may be designed for partial load transfer when the existing structure is able to sustain load. If this is to be utilised in design, then only the dynamic load components need to be considered. The general form of the equation for determining acting slip stress is as follows:

$$\sigma_{sc} = \left[\sigma_A^{\,2} + (\alpha \ \sigma_T)^2\right]^{1/2} \qquad \text{.... 4.2.1}$$

where: σ_A and σ_T are the interface stresses due to axial load and torsion respectively,

α is a factor to be taken as unity, except for a weld bead connection. In this case α is to be taken as the ratio of the axial capacity of the weld bead connection to that of a similar connection without weld beads.

The acting slip stress components are derived from the following equations:

$$\sigma_A = \frac{\bar{F}}{\pi D L} \qquad \text{.... 4.2.2}$$

$$\sigma_T = \frac{2\bar{M}_x}{\pi D^2 L} \qquad \text{.... 4.2.3}$$

where:

$\bar{F}$ = axial force to be transferred to clamp (see below)
D = member external diameter
L = clamp length
M_x = torsion to be transferred to clamp (see below)

The value of the axial force transferred to the clamp, $\bar{F}$, will depend upon whether or not the clamp is designed for full load transfer (see Section III 4.2.15.1).

For full load transfer:

$$\bar{F} = F \qquad \text{.... 4.2.4}$$

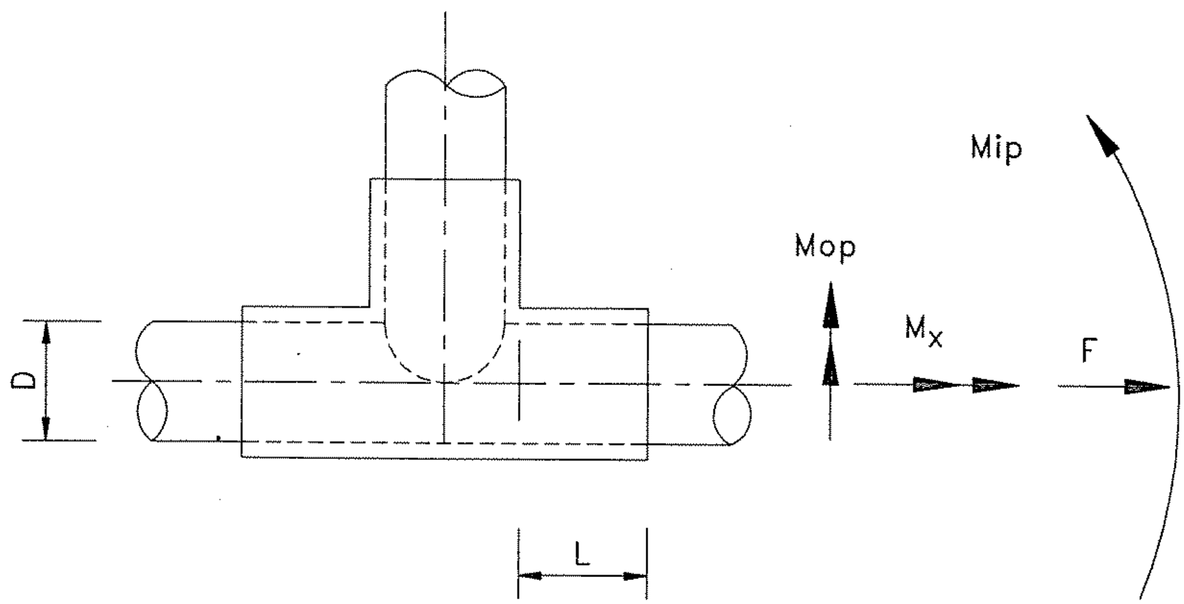


Figure 4.2.2: Loads to consider for design of repair clamp

For partial load transfer:

$$\bar{F} = F \left[\frac{A_{clamp}}{A_{total}} \right] \qquad \text{.... 4.2.5}$$

where:

F	=	axial load in member
A_{clamp}	=	x-sectional area of clamp
A_{total}	=	total x-sectional area of clamp and member

Torsion transferred to clamp, $\bar{M}_x$, is derived using the following equations:

For full load transfer:

$$\bar{M}_x = M_x \qquad \text{.... 4.2.6}$$

For partial load transfer:

$$\bar{M}_x = M_x \left[\frac{J_{clamp}}{J_{total}} \right] \qquad \text{.... 4.2.7}$$

where:

M_x	=	torsion in member
J_{clamp}	=	torsional stiffness of clamp
J_{total}	=	torsional stiffness of clamp (= J_{clamp}) plus torsional stiffness of enclosed part of member

The intention is to calculate an upper bound of the torsion to be transferred to the clamp and, for partial load transfer, this means having to find an upper bound estimate of the clamp torsional stiffness. Unless the torsion in the member outside the clamped region (ie. M_x) is significant, it may be appropriate to adopt conservatively Equation 4.2.6 to avoid calculating the clamp torsional stiffness. Note, the torsional stiffness of the clamp comprises the sum of the torsional stiffnesses of the enclosed "tubes" (formed by the saddle plate + flange plate + two side plates) in each clamp half about their individual centres of rotation, plus the torsional stiffness afforded by the opposing lateral displacements of the clamp halves as they undergo cantilever bending.

III 4.2.5.3 Addmember

Figure 4.2.3 presents the applicable loads for an addmember. An addmember should be designed for partial load transfer. The general form of the equation for determining acting slip stress is as follows:

$$\sigma_{sc} = \left[\sigma_A^2 + \sigma_T^2\right]^{1/2} \qquad \text{.... 4.2.8}$$

The acting slip stress components are derived from the following equations:

$$\sigma_A = \frac{\bar{F}}{\pi D L} \qquad \text{.... 4.2.9}$$

$$\sigma_T = \frac{4 M_{opb}}{\pi D^2 L} \qquad \text{.... 4.2.10}$$

where:

M_{opb} = out-of-plane bending in addmember.

The derivation of $\bar{F}$ for the chord in an addmember situation is as follows:

$$\bar{F} = 2 \left\{ F_{max} \left[1 + \frac{A_{clamp}}{2 A_{total}} \right] - F_{min} \left[1 - \frac{A_{clamp}}{2 A_{total}} \right] \right\} \qquad \text{.... 4.2.11}$$

where:

F_{max} = maximum axial load in member
F_{min} = minimum axial load in member
A_{clamp} = x-sectional area of clamp
A_{total} = total x-sectional area of clamp and member

Note, F_{max} and F_{min} are strictly the simultaneously occurring values giving the greatest value of F. They may, however, be taken as not being temporally connected.

III 4.2.6 **Select Studbolts and Check Slip Strengths**

III 4.2.6.1 General

The slip strength of a stressed clamp (mechanical, grouted or elastomer-lined) depends, inter alia, on studbolt load and, in the case of a stressed mechanical clamp also on studbolt stiffness. The following subsections give

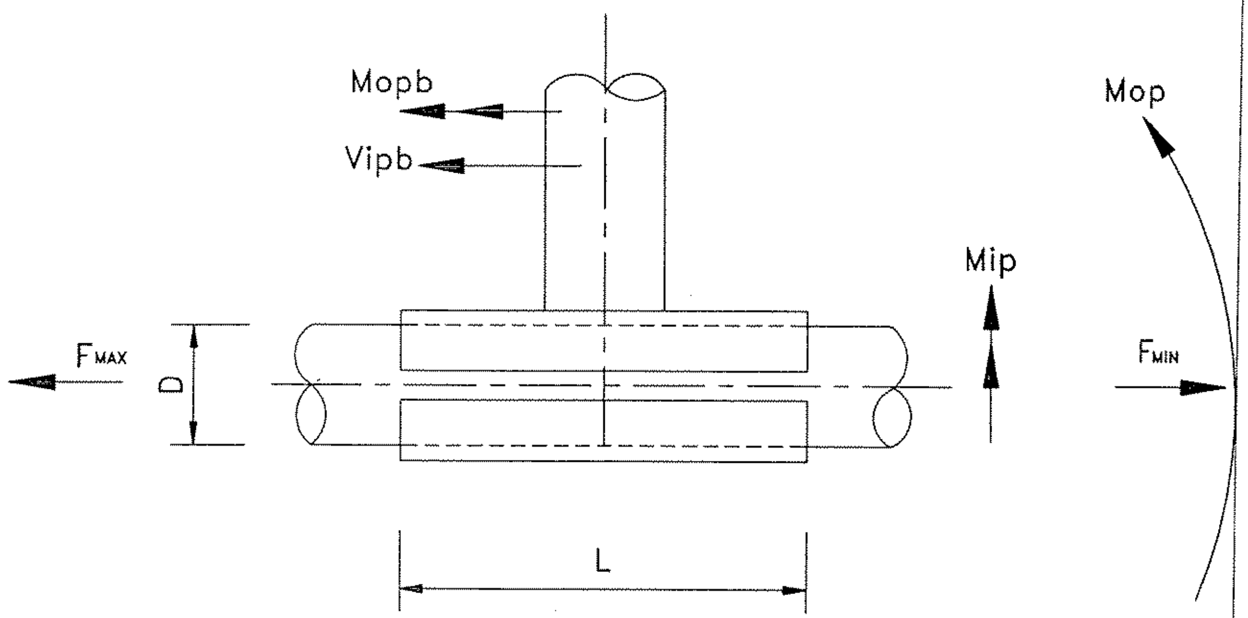


Figure 4.2.3: **Loads to consider for design of addmember clamp (note the chord moments are defined relative to the clamp split line)**

recommendations for establishing the slip strength of various clamp types and the calculated slip strength should be greater than the acting slip stress as derived in Section III 4.2.5. If this criteria is not met then selection of alternative studbolts or clamp geometry will be necessary.

In addition to providing sufficient slip strength, studbolt loads must prevent the two halves of the clamp separating under the action of applied loads. This criteria, which often governs, is addressed in Section III 4.2.7.

Consideration may be given to assessing the consequences should one or more bolts be lost after installation, perhaps as a result of corrosion or damage.

III 4.2.6.2 Stressed mechanical clamps

The following characteristic slip strength equation, as derived in Part IV - Background Data and Assessments, is recommended for stressed mechanical clamps:

$$\sigma_c = \frac{0.12\,C_s'}{\Gamma_f}\left[1 + 20\left(\frac{T}{D}\right)\right](1 + 66\,K_b)\left(\frac{F_n}{DL}\right) \qquad \ldots\ 4.2.12$$

The formula has been derived from experimental data of mechanical clamps within the following ranges:-

$$0.5 \leq \frac{L}{D} \leq 2.0 \text{ (though this parameter has negligible effect)}$$

$$20 \leq \frac{D}{T} \leq 50$$

$$0.002 \leq K_b \leq 0.01$$

In the above:

L = length of connection
D = diameter of tubular member
T = thickness of tubular member
K_b = bolt stiffness parameter

$$= \frac{n\ A_b\ E_b}{2\ L\ L_b\ E_s} \text{ in which}$$

n = number of studbolts in connection
A_b = net area of one studbolt
E_b = Young's modulus of studbolt material
L_b = stressed length of studbolt (nut face to face)

E_s = Young's modulus of chord steel

F_n = total studbolt load in connection

The surface condition factor, C_s', should be taken according to the following conditions:

Surface Condition	Surface Condition Factor C_s'
Shot blasted in air	1.00
Mill scale	0.85
Underwater grit blast	0.85
Coal tar epoxy	0.60

The factors of safety from the UK Health and Safety Executive Guidance Notes for the extreme and operating conditions, are:-

$$\Gamma_f = \begin{cases} 1.70 & \text{for extreme condition} \\ 2.25 & \text{for operating condition.} \end{cases}$$

See Section III 4.2.6.6 for further information on factors of safety.

III 4.2.6.3 Unstressed grouted clamps/sleeve connections

There are three equations that may be used to determine the characteristic slip strength of an unstressed grouted clamp/sleeve connection. These are from the UK Health and Safety Executive Guidance Notes, API RP2A and DNV Rules for classification of Fixed Offshore Installations. The application of these formulae are presented in Part IV - Background Data and Assessments.

The three applicable equations are presented in the following subsections.

III 4.2.6.3.1 HSE Guidance Notes[4.1]

The characteristic bond strength of a grouted connection, with or without mechanical shear connectors, incorporating a factor K_o as recommended in Part IV, is given by:-

$$\sigma_{bc} = K\, K_o\, C_L\, (9\, C_s + 1100\, h/s)\, \sqrt{\sigma_{cu}} \qquad \text{.... 4.2.13}$$

where:

$$K = \frac{1}{m}\left[\left[\left(\frac{D}{T}\right)_g\right]^{-1} + \left[\left(\frac{D}{T}\right)_p + \left(\frac{D}{T}\right)_s\right]^{-1}\right] \qquad \text{.... 4.2.14}$$

in which m is the modular ratio of steel to grout (to be taken as 18 in lieu of actual data); D is the outer diameter, T is the thickness; and g, p and s are subscripts denoting grout, pile and sleeve respectively.

and:

K_o = 1 for split or continuous plain pipe and weld bead continuous sleeve

K_o = 0.75 for weld bead split sleeve

h = minimum shear connector outstand

s = shear connector spacing

σ_{cu} = characteristic unconfined grout compressive strength in N/mm^2.

The coefficient for grouted length to pile diameter ratio, C_L, is tabulated as a function of L/D_p as follows:

L/D_p	C_L
2	1.0
4	0.9
8	0.8
$\geq$ 12	0.7

The surface condition factor, C_s, for shot blasted surfaces is specified as:-

h/s	C_s
$\geq$ 0.005	1.0
$<$ 0.005	0.6

The above equations may only be applied to connections which satisfy the geometrical ratio limits specified in Appendix A22.2.3 of the Guidance Notes. These limits are reproduced in Table 4.2.2, along with those of other codes discussed below.

The safety factor specified in the Guidance Notes (see Section III 4.2.6.6 for further information) is given as:-

<u>Grout Displacing Water</u>

Condition:	Safety Factor:
Extreme	4.5
Operating	6.0

Grout displacing drilling mud or other similar material

Condition:	Safety Factor:
Extreme	6.0
Operating	8.0

III 4.2.6.3.2 API RP2A[4.2]

The allowable bond strength for connections without shear keys should be taken as 0.138 N/mm^2 for loading conditions 1 and 2, and 0.184 N/mm^2 for loading conditions 3 and 4. Loading conditions 1 - 4 are defined in Clause 2.2.2 of API RP2A but briefly loading conditions 1 and 2 relate to maximum and minimum operational loads and loading conditions 3 and 4 correspond to extreme loads.

The allowable bond strength for connections with (or without) shear keys can be calculated from the following equations:-

- for loading conditions 1 and 2:-

$$\sigma_{ba} = K_o \left[0.138 + 0.5 \left(\frac{h}{s} \right) \sigma_{cu} \right] \qquad \text{.... 4.2.15}$$

- for loading conditions 3 and 4:-

$$\sigma_{ba} = K_o \left[0.184 + 0.67 \left(\frac{h}{s} \right) \sigma_{cu} \right] \qquad \text{.... 4.2.16}$$

where σ_{ba} is the allowable bond strength in N/mm^2

K_o = 1 for plain pipe and weld bead continuous sleeve

K_o = 1 for plain pipe split sleeve

K_o = 0.7 for weld bead split sleeve

h = minimum shear connector outstand

s = shear connector spacing

σ_{cu} = characteristic unconfined grout compressive strength in N/mm^2.

These equations should be used in conjunction with the geometrical and material limits given in API RP2A[4.2], reproduced in Table 4.2.2.

IV 4.2.6.3.3 DNV Rules for Classification of Fixed Offshore Installations[4.3]

The characteristic bond strength of a continuous grouted connection is specified as:-

$$\sigma_{bc} = \mu\, C_L\, K'\, E_s \left[2\left(\frac{\delta}{D_p}\right) + \frac{\sigma_{cu}^{0.15}}{16}\sqrt{\frac{2T_p}{D_p}\left(\frac{h}{s}\right)} \right] \qquad \ldots.\ 4.2.17$$

where K′ is a radial stiffness parameter defined as:-

$$K' = \left[\frac{D_p}{2T_p} + m\frac{2T_g}{D_p} + \frac{mD_s}{2(mT_s + T_g)} \right]^{-1} \qquad \ldots.\ 4.2.18$$

For a split grouted connection, the characteristic bond strength can be estimated using Equation 4.2.17, but the radial stiffness K′ is calculated from:-

$$K' = \left[\frac{D_p}{2T_p} + m\frac{2T_g}{D_p} + \frac{mD_g}{2(mT_s + T_g)} + \frac{2L_bS_b}{\pi A_b} \right]^{-1} \qquad \ldots.\ 4.2.19$$

The partial safety factors for ultimate limit state and progressive limit state are given as 3.0 and 2.6 respectively.

In the above, the notation is as in subsection III 4.2.6.3.1 but with the following additions:

μ = grout-to-steel coefficient of friction (μ = 0.7 unless otherwise documented)

δ = height of surface irregularities, local out-of-roundness, etc. (δ = 0.000125 D_p for rolled steel surfaces with normal fabrication tolerances)

S_b = studbolt spacing

The above equations are specified for use in the ranges defined in Table 4.2.2.

III 4.2.6.4 Stressed grouted clamps

The following characteristic design strength equation, as derived in Part IV - Background Data and Assessments, is recommended for a stressed grouted clamp:-

Code	$(D/T)_s$		$(D/T)_p$		$(D/T)_g$		L/D_p		h/D_p		D_p/s		h/s		w/h		σ_{cu} (N/mm^2)		Additional Limits
	L	U	L	U	L	U	L	U	L	U	L	U	L	U	L	U	L	U	
HSE[4.1]	50	140	24	40	10	45	2	-	0	0.006	0	8	0	0.04	1.5	3			
API[4.2]		80	0	40	7	45					2.5	8	0	0.10	1.5	3	17.25	110	σ_{cu} (h/s) ≤ 5.5
DNV[4.3]	18	140	10	60			2	-					0	0.04			4	90	$\sqrt{D_p T_p} \leq S$

L = Lower limit
U = Upper limit
w = Width of shear connector

Table 4.2.2: Applicable limits of HSE, API and DNV formulae

$$\sigma_{sc} = \left[\frac{0.95 \times C_s}{\Gamma_b} + \frac{0.35\, C_s'(F_n/DL)}{\Gamma_f}\right]\left[1-0.13\left(\frac{L}{D}\right)\right]\left[1+12\left(\frac{T}{D}\right)\right] \qquad \text{.... 4.2.20}$$

The formula has been developed from data within the following ranges:-

$0.9 \leq L/D \leq 2.2$
$17.7 \leq D/T \leq 50.0$
$21.2 \leq D_s/T_s \leq 36.0$
$7.4 \leq D_g/T_g \leq 18.2$
$50.0\ N/mm^2 \leq \sigma_{cu} \leq 78.0\ N/mm^2$
$1.3\ N/mm^2 \leq F_n/DL \leq 12.0\ N/mm^2$

The surface condition factors should be taken as follows:-

Steel surface condition	C_s	C_s'
Grit blasted	0.6	1.00

No test data are available for other surface conditions but reference may be made to Section III 4.2.6.2.

The following safety factors for bond and friction strengths, taken from the UK HSE Guidance Notes, are traditionally used with the above design equation.

$$\Gamma_b = \begin{cases} 4.5 & \text{for extreme loading conditions} \\ 6.0 & \text{for operational loading conditions} \end{cases}$$

$$\Gamma_f = \begin{cases} 1.7 & \text{for extreme loading conditions} \\ 2.25 & \text{for operational loading conditions} \end{cases}$$

Further information on safety factors is given in Section III 4.2.6.6.

III 4.2.6.5 Elastomer-lined clamps

The following equation, as derived in Part IV - Background Data and Assessments, is recommended for elastomer-lined clamps:

$$\sigma_c = \frac{\mu_{ult}}{\Gamma_f} \cdot \left[\frac{F_n}{DL}\right] \qquad \text{.... 4.2.21}$$

The coefficient of friction, μ_{ult}, may conservatively be taken as 0.2.

The following safety factors for friction strength, taken from the UK HSE Guidance Notes (see Section III 4.2.6.6), are recommended for use with the above design equation:

$$\Gamma_f = \begin{cases} 1.7 & \text{for extreme conditions} \\ 2.25 & \text{for operating conditions.} \end{cases}$$

The formula has been derived using a conservative coefficient of friction. No test data are available to determine the limits for application, but within the bounds of conscientious design, the formula should prove adequate.

III 4.2.6.6 Background to factors of safety

i) **Unstressed Grouted Connections**

The first codification of allowable bond stress was provided in early editions of API RP2A and was for plain pipe connections only. At the time, grout was used to fill the annulus between jacket legs and piles driven through the legs. The structural connection was achieved by welding the leg and pile together at the top of the leg, consequently grout was not the primary load path. When a value for allowable bond stress was introduced (eg. API RP2A 9th Edition 0.138 N/mm^2) a factor of safety of 6.0 was chosen. This was because, with the form of grouted connections in use at the time and the test data available, it would not change designs if the grout was made the only load carrying path between the leg and the pile. The same factor of safety was carried over when the first Edition of the DEn (now HSE) Guidance Notes was produced (1977 Ed). Note API now adopts a lower bound approach and the factor of safety is not quoted any longer.

Early testing work in the UK on fatigue of weld beaded grouted connections indicated that there appeared to be a fatigue threshold at approximately 0.2 $\times$ characteristic bond strength. Therefore, there was no pressure to change the factor of safety as the current design practice eliminated fatigue as a consideration.

Recently, work by DNV and others has looked at reducing the factor of safety in light of the greater knowledge and understanding of grouted connection behaviour, in conjunction with improved offshore grouting practices. However, it is understood that DNV has not considered the possibility of packer failure. With the recent practices using lower factors of safety it is now necessary to give specific consideration to design for fatigue.

ii) **Stressed Clamps (Grouted and Mechanical)**

The safety factors currently in use were developed during early ad-hoc repair projects and during the preparation of OTH 88 283[4.4] and DEn (now HSE) Guidance Notes, the safety factors being accepted by the CAs (DNV and Lloyd's) at the time. Design was based on limit state principles. Stressed connections do not exhibit as large a scatter on test results as grouted connections and are not sensitive to grout strength, therefore a typical steelwork design limit state factor of safety of 1.7 was applied to the frictional part of characteristic formulations. In light of the novelty of the technology at the time it was decided to apply the 1.7 factor to the extreme load condition rather than the normal condition, as was the case with steelwork design. Hence, the normal condition factor of safety $= 1.7 \times {}^4/_3 = 2.25$.

It is not the intention within this project to alter current factors of safety, as such a study is outside the scope of the project.

III 4.2.7 Check Separation Forces on Clamp Halves

The structural integrity of clamps is dependent on the frictional forces developed at the slip interface as a result of studbolt loads. It is therefore vital that the studbolt loads are proportioned such that positive contact pressure is maintained at all times during the design life of the clamp.

Bending moments and shear forces in the clamped tubular will tend to separate the clamp halves, and therefore, checks must be performed to ensure that the studbolt loads maintain a positive contact pressure. Two approaches to ensure that positive contact pressure is always maintained are given below, and these are termed the traditional approach and the refined approach respectively. The refined approach is based upon the assessment of test and numerical data generated under this project and is fully reported in Part V of this document. It removes the undue conservatism inherent in the traditional approach. However, it is suggested that a preliminary design check is first carried out using the traditional approach as it is relatively simple to apply. If the preload requirement so calculated is greater than that needed for preventing slip (see Section III 4.2.6), then recourse to the refined approach may prove of benefit.

For both the traditional and refined approaches, internal load distributions within the clamp are first assumed. These internal radial loads arise from the applied moments and shears. The distribution pattern arising from the moment resultant in the traditional approach is taken as triangular and this is assumed to be resited solely by the studbolts. The load in any given studbolt, and an end studbolt in particular, is found by integrating that part of the internal distributed load which can be attributed to the studbolt. The refined approach, on the other hand, recognises fundamental differences between IPB and OPB transfer mechanisms. In the case of IPB, advantage is taken of the bending rigidity of each clamp half to distribute the studbolt loads more evenly; a calibrated model using a rigid clamp is used as a basis. (For fatigue calculations, advantage is also taken of load sharing between studbolt loading and clamp preload relaxation.) In the case of OPB, the internal distributed radial loads give rise to internal torque loads are resisted by a couple from each studbolt pair (one studbolt loads whilst the other unloads) and circumferential shears within the annular material. Whereas grout is stiff enough to give rise to significant circumferential shears, neoprene lines are not. The loads in an end studbolt due to applied IPB and OPB (and shears) are added to establish the minimum preload requirement.

Traditional Approach

Satisfying the following inequality will ensure that no separation of the clamp halves will occur:

$$\frac{F_n}{N} > \Gamma \left[\frac{P}{2}\right] \quad \text{.... 4.2.22}$$

where: Γ = factor of safety ($\Gamma = 1.2$ is recommended based on current practice)
F_n = total studbolt load
N = total number of studbolts
P = bolt separation load per end pair of studbolts

The load P is represented by the shaded area in the lowest diagram in Figure 4.2.4, ie.:

$$P = (w_{END} + w_x)\frac{x}{2}$$

w_{END} and w_x are the internal loads per unit length at the clamp end and at a distance x from the end respectively (see Figure 4.2.4). They arise from applied bending moments (in-plane and out-of-plane) and shear forces (in-plane and out-of-plane) and are conservatively given by:

$$w_{END} = \frac{6}{\ell^2}\left(M_{ip}^{\ 2} + M_{op}^{\ 2}\right)^{1/2} + \frac{1}{\ell}\left(V_{ip}^{\ 2} + V_{op}^{\ 2}\right)^{1/2}$$

$$w_x = \frac{6}{\ell^2}\left[1 - \frac{2x}{\ell}\right]\left(M_{ip}^{\ 2} + M_{op}^{\ 2}\right)^{1/2} + \frac{1}{\ell}\left(V_{ip}^{\ 2} + V_{op}^{\ 2}\right)^{1/2}$$

Substituting the above in the expression for P, the following explicit simplification is obtained:

$$P = x\left[\left[1 - \frac{x}{\ell}\right]\frac{6}{\ell^2}\left(M_{ip}^{\ 2} + M_{op}^{\ 2}\right)^{1/2} + \frac{1}{\ell}\left(V_{ip}^{\ 2} + V_{op}^{\ 2}\right)^{1/2}\right] \quad \text{.... 4.2.23}$$

where M_{op}, M_{ip}, V_{op} and V_{ip} are the out-of-plane and in-plane moments and shears. The maximum values of the moments and shears occurring along the enclosed length (ℓ) of the tubular should strictly be selected, though normally the values at the nearest global analysis node will suffice. The dimensions x and ℓ are defined in Figure 4.2.4.

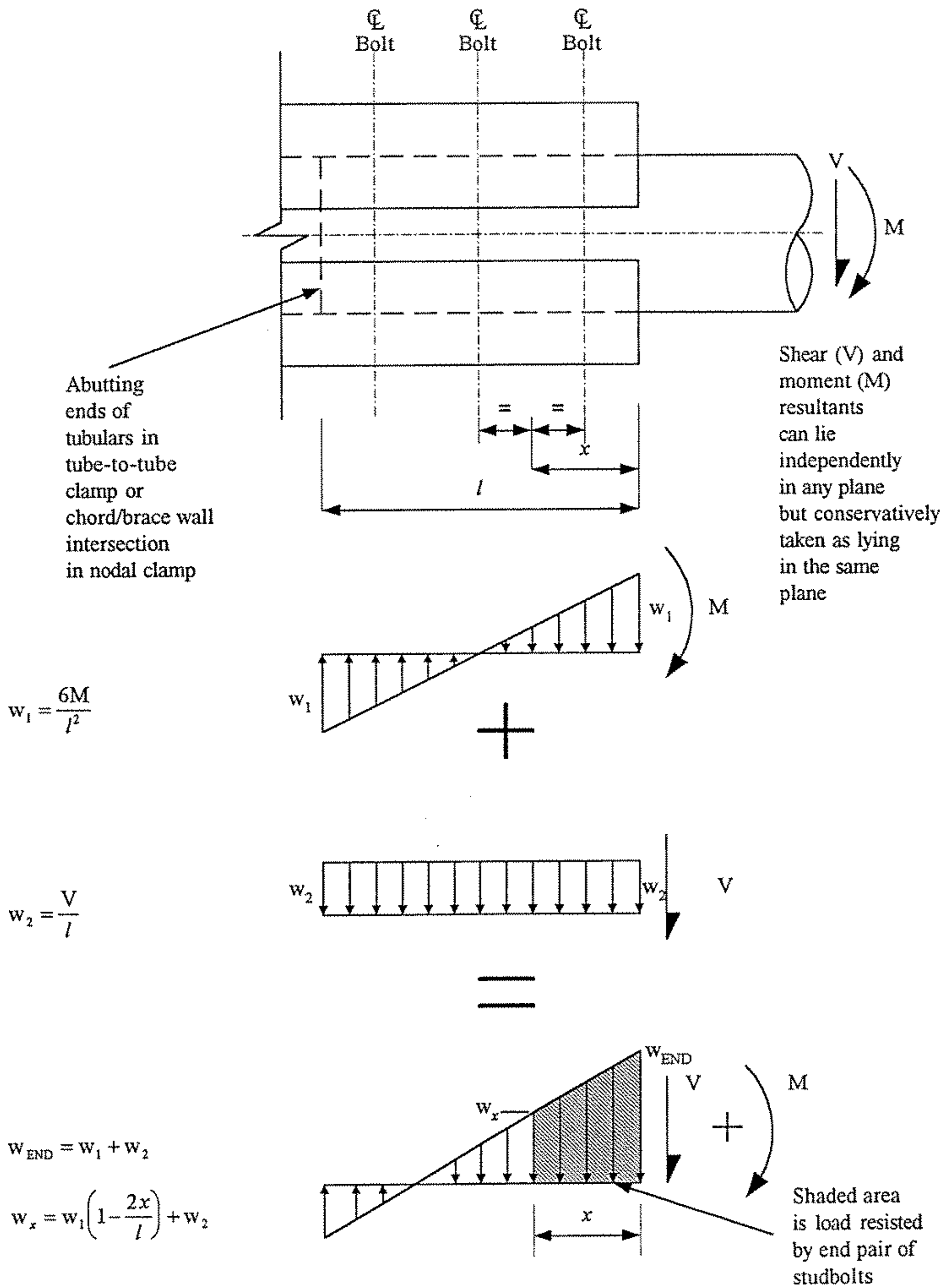


Figure 4.2.4: **Traditional approach for establishing studbolt loads due to applied moments and shears**

Refined Approach

The refined approach presented below is based on the assessment of the test and numerical data for tube-to-tube clamps subjected to pure bending. The generation of the data, and the explanation of clamp/tubular behaviour under such loads is fully detailed in Part V. The design philosophy developed for tube-to-tube clamps is extended to cover nodal clamps in Part V and below, but it is emphasised here that no confirmatory tests have been conducted on nodal clamps.

It has been established that different transfer functions, between applied moment and studbolt load, apply for in-plane and out-of-plane bending. The test data also permitted differentiation between the transfer functions applicable to preload requirements and to fatigue loading. This is reflected in the provisions below.

The following inequality should be satisfied to ensure that no separation of the clamp halves will occur:

$$\frac{F_n}{N} > \Gamma_i F_i P_{mi} + \Gamma_o F_o P_{mo} + \Gamma_v P_v \qquad \ldots .\ 4.2.24$$

where:

- F_n = total studbolt load
- N = total number of studbolts
- Γ_i = factor of safety for IPB (Γ_i = 2.0 is recommended)
- Γ_o = factor of safety for OPB (Γ_o = 1.0 is recommended)
- Γ_v = factor of safety for shear (Γ_v = 1.2 is recommended)
- F_i = reduction factor for IPB (no reduction for preload requirement, ie. F_i = 1.0)
- F_o = reduction factor for OPB (= 1.0 for neoprene-lined clamps; = 0.2 for stressed grouted clamps)
- P_{mi} = basic bolt load due to IPB:

$$= \frac{M_{ip} x/\ell}{2} \left[\frac{4-3x/\ell}{\ell_1} - \frac{2-3x/\ell}{\ell_2} \right] \quad \text{for tube-to-tube clamp}$$

$$= \frac{M_{ip} x/\ell}{2\ell_1^2} (\ell + \ell_1 - x) \quad \text{for nodal clamp}$$

- P_{mo} = basic bolt load due to OPB:

$$= \frac{M_{op}\ H\ x}{B\ \ell_1^2} (1 - x/\ell_1) \quad \text{for tube-to-tube and nodal clamps}$$

- P_v = basic bolt load due to shear:

$$= \frac{x}{2\ell_1} \left(V_{ip}^2 + V_{op}^2\right)^{1/2} \quad \text{for tube-to-tube and nodal clamps}$$

In the above the following definitions apply:

M_{ip} = in-plane bending moment (plane of bending is parallel to studbolt axes)

M_{op} = out-of-plane bending moment (plane of bending is orthogonal to studbolt axes)

V_{ip} = in-plane shear load

V_{op} = out-of-plane shear load

x = the distance from the end of the clamp to a point midway between the first and second pair of studbolts

H = overall height of clamp measured between outer surfaces of flange plates

B = transverse distance between studbolt rows

ℓ, ℓ_1 and ℓ_2 are defined for the different types of clamps in Figure 4.2.5.

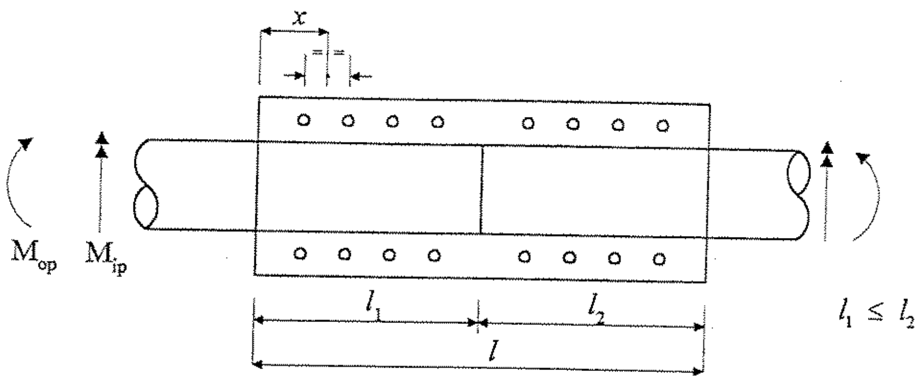
The above equations can also be used to assess the maximum studbolt load variation due to applied bending and shear fatigue loads (see also Section III 4.2.15). The maximum variation may be found by evaluating the right hand side of Equation 4.2.24 with the following modifications to the above:

$$\Gamma_i = \Gamma_v = 1.0$$

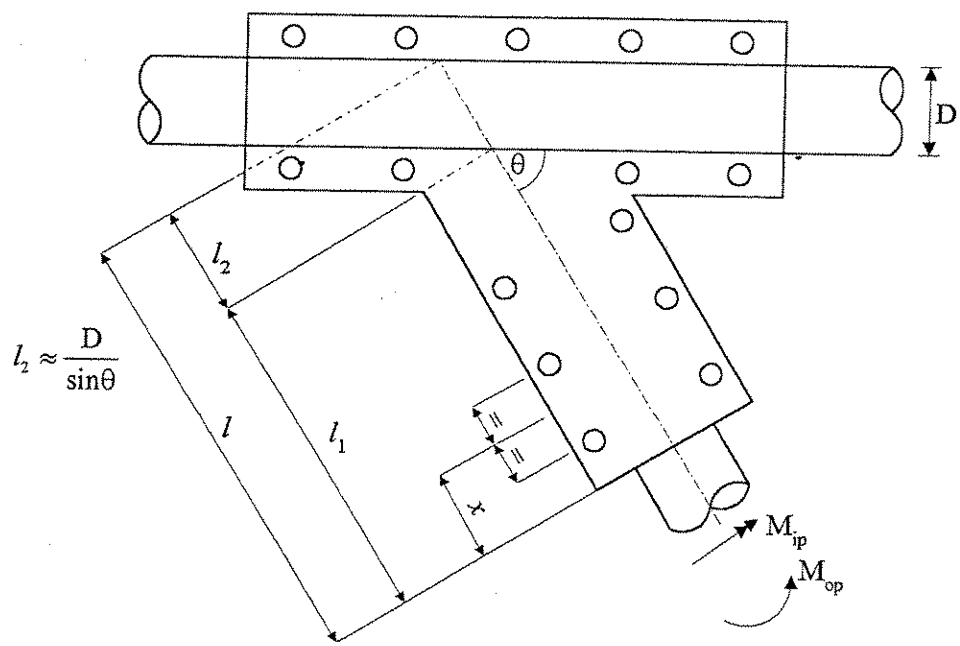
F_i = 0.2 for neoprene-lined clamps
= 0.05 for stressed grouted clamps.

III 4.2.8 Finalise Studbolt Loads

Once the checks for slip and separation have been carried out in Sections III 4.2.6 and III 4.2.7 respectively, then the larger required studbolt load is used for finalised studbolt sizing. Depending on the repair option adopted an allowance must be taken for transfer losses, grout shrinkage and elastomer creep.



a) Tube-to-tube clamp



b) Nodal clamp

Figure 4.2.5: **Definition of ℓ, ℓ_1 and ℓ_2 for various clamp types for the refined approach**

Jacking losses are usually of the order of 10%, but advice should be sought from the jack supplier. In consultation with a polychloroprene manufacturer, 10% is considered an appropriate allowance for long term creep for a thickness of 16mm. This value should also be confirmed from the suppliers of such linings. When a repair has utilised grout, a shrinkage allowance (commonly a value of 15% is taken) should be included in the finalised studbolt load. Grout ring effects as defined in Section III 4.2.9.3 and seal compression also reduce the effective load for friction.

Once all allowances for possible losses have been considered, the finalised studbolt load will be used in the check for hoop stress and design of clamp steelwork. The studbolt diameter can be chosen provided that two criteria are satisfied. Firstly, the long term stress in the studbolt should not be such that stress corrosion cracking becomes a problem (a limit of 0.6 F_y is commonly adopted in practice). Secondly, the short term stress must not exceed the elastic limit for bolting material (0.7 F_y is commonly adopted). Section III 7 presents the effective tensile areas that should be used for ISO metric screw threads and for Unified inch screw threads.

The above criteria may be summarised as follows. Defining F^{max} as the short term studbolt load and F^{min} as that portion of the studbolt load which directly gives the interface radial stress:

$$F^{max} \leq 0.7\ F_y$$

$$F^{min} = F^{max} - \text{(jacking losses)} - \text{(elastomer creep or grout shrinkage)} - \text{(allowance for grout ring effects and seal compression)}$$

$$\leq 0.6\ F_y$$

In the above, the bracketed terms are invoked depending on clamp type.

All studbolts in a repair must be tensioned uniformly. Where a repair involves clamping a joint, there may be a different set of studbolt loads for each clamped member. These loads may not vary significantly, allowing the possibility of using the same studbolt load in all bolts. This will mean increasing the lower load(s) to match the highest load. Clamp steelwork should therefore be designed for this higher load and hoop stress checks on the member should also be carried out at this higher load.

III 4.2.9 Check Members for Equivalent Stress

III 4.2.9.1 Introduction

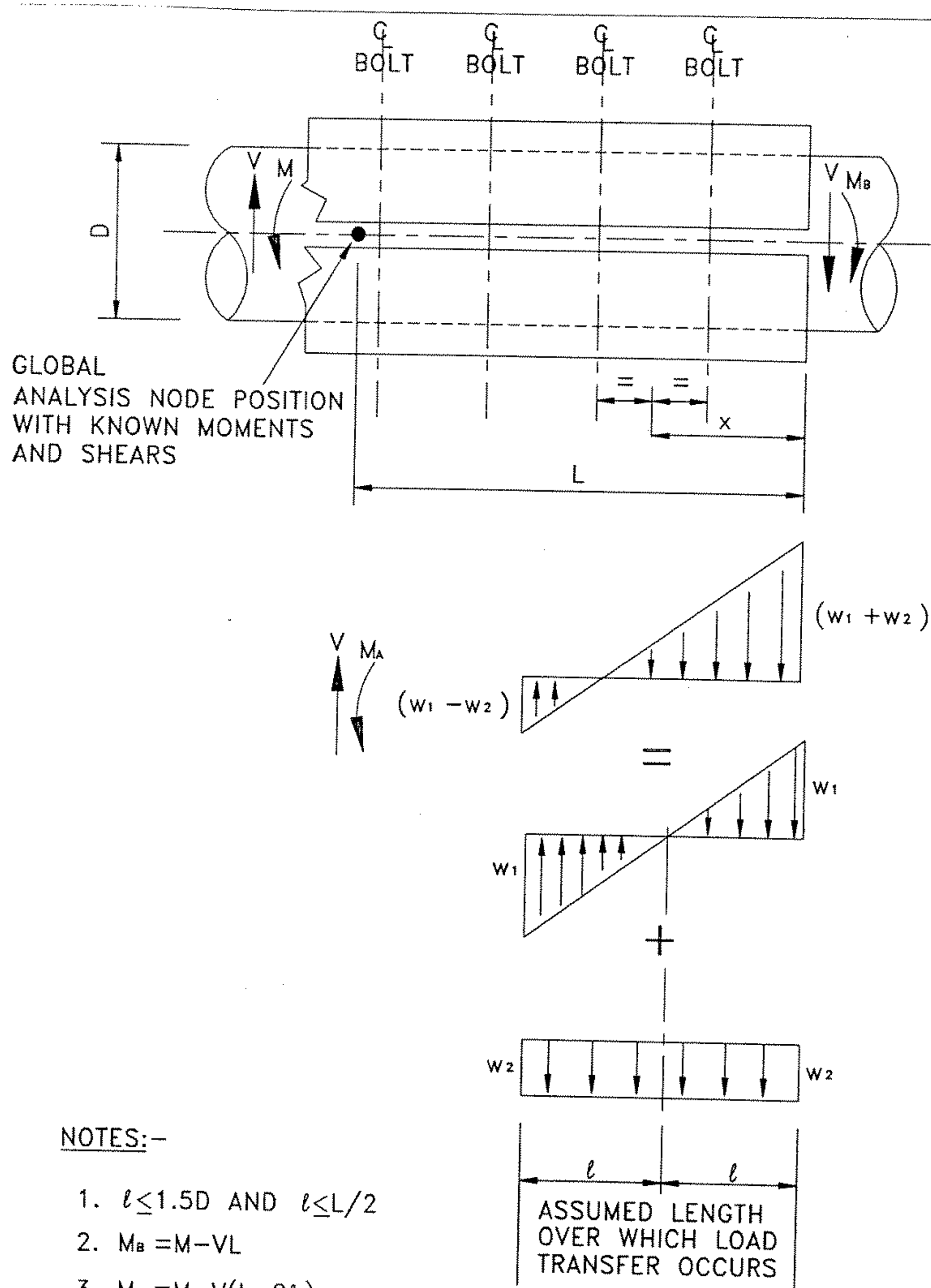


Figure 4.2.5: Moment and shear induced pressure distributions

Jacking losses are usually of the order of 10%, but advice should be sought from the jack supplier. In consultation with a polychloroprene manufacturer, 10% is considered an appropriate allowance for long term creep for a thickness of 16mm. This value should also be confirmed from the suppliers of such linings.

When a repair has utilised grout, a shrinkage allowance (commonly a value of 15% is taken) should be included in the finalised studbolt load. Grout ring effects as defined in Section III 4.2.9.3 and seal compression also reduce the effective load for friction.

Once all allowances for possible losses have been considered, the finalised studbolt load will be used in the check for hoop stress and design of clamp steelwork. The studbolt diameter can be chosen provided that two criteria are satisfied. Firstly, the long term stress in the studbolt should not be such that stress corrosion cracking becomes a problem (a limit of 0.6 F_y is commonly adopted in practice). Secondly, the short term stress must not exceed the elastic limit for bolting material (0.7 F_y is commonly adopted). Section III 7 presents the effective tensile areas that should be used for ISO metric screw threads and for Unified inch screw threads.

The above criteria may be summarised as follows. Defining F^{max} as the short term studbolt load and F^{min} as that portion of the studbolt load which directly gives the interface radial stress:

$$F^{max} \leq 0.7\,F_y$$

$$\begin{aligned} F^{min} &= F^{max} - \text{(jacking losses)} - \text{(elastomer creep or grout shrinkage)} - \text{(allowance for grout ring effects and seal compression)} \\ &\leq 0.6\,F_y \end{aligned}$$

In the above, the bracketed terms are invoked depending on clamp type.

All studbolts in a repair must be tensioned uniformly. Where a repair involves clamping a joint, there may be a different set of studbolt loads for each clamped member. These loads may not vary significantly, allowing the possibility of using the same studbolt load in all bolts. This will mean increasing the lower load(s) to match the highest load. Clamp steelwork should therefore be designed for this higher load and hoop stress checks on the member should also be carried out at this higher load.

III 4.2.9 Check Members for Equivalent Stress

III 4.2.9.1 Introduction

This check is only necessary for stressed clamp solutions where stressing of the studbolts induces additional hoop stress in the existing member. The studbolt load, as previously defined in Section III 4.2.8, is used to check equivalent stress. The allowable equivalent stress is given by 0.9 F_y (to AISC recommendations), with no ⅓ increase.

III 4.2.9.2 Equivalent stress calculation

Calculation of equivalent stress is carried out in accordance with the von Mises approach as defined by the following equations :

For longitudinal axial compression (σ_L positive):

$$(\sigma_h^2 + \sigma_L^2 - \sigma_h.\sigma_L + 3.\tau^2)^{1/2} \qquad \ldots.\ 4.2.25$$

For longitudinal axial tension (σ_L negative):

$$(\sigma_h^2 + \sigma_L^2 + \sigma_h.\sigma_L + 3.\tau^2)^{1/2} \qquad \ldots.\ 4.2.26$$

where: σ_h = hoop stress
σ_L = longitudinal stress (axial + bending)
τ = shear stress

The hoop stress is given by the following equation:

$$\sigma_h = p \cdot \frac{D}{2T} \qquad \ldots.\ 4.2.27$$

where: p = external pressure on member
D = member diameter
T = member thickness

For stressed grouted clamps, the member external pressure will differ from the pressure attributed to the studbolt force due to the grout annulus resisting a proportion of the load. An allowance may be taken for this compressive ring effect, as described in the following section, where the hoop stress is initially found to be too high.

III 4.2.9.3 Member pressure calculation

The pressure on the member is never greater than that directly calculated from the studbolt force and this can therefore be used as a conservative approximation. In the case of stressed grouted clamps, some of the studbolt load is resisted by the grout annulus acting as a compression ring, and this can be accounted for as detailed below.

The member pressure induced by studbolt loads will be different depending on whether the existing member is grout-filled or not. Here, the case of an empty member will be considered. The application of this approach can be applied to grout filled members with a simple extension to the methodology.

Figure 4.2.6 presents the pressure distribution for a non-grout filled member. The compatibility equation given by Roark[4.5] for this case is as follows:

$$\epsilon_{steel}(p, R) = \epsilon_{grout}(p_o, R) + \epsilon_{grout}(p, R) \qquad \text{.... 4.2.28}$$

where each component is defined as follows:

$$\epsilon_{steel}(p, R) = -\frac{p.R}{E_s}\left[\frac{(R^2+R_1^2)}{(R^2-R_1^2)} - \nu_s\right]$$

$$\epsilon_{grout}(p_o, R) = -\frac{p_o}{E_g}\frac{2.R_o^2.R}{R_o^2-R^2}$$

$$\epsilon_{grout}(p, R) = +\frac{p.R}{E_g}\left[\frac{(R_o^2+R^2)}{(R_o^2-R^2)} + \nu_g\right]$$

The grout properties may be taken as:

$$\nu_g = 0.2$$
$$E_g = E_s / 18$$

The compatibility equation can be solved in terms of a ratio between the applied external annulus pressure (p_o) and a reduced external member pressure (p). (Note, studbolt loads must be sufficiently increased to overcome the pressure differential in order to stress the existing member adequately, see Section III 4.2.8.)

III 4.2.9.4 Member buckling

For stressed mechanical or elastomer-lined clamps, there is a possibility of chord wall buckling at the clamp split line when the distance between clamp halves is large. Wide column analysis, assuming the sway mode, may be carried out to guard against this possibility.

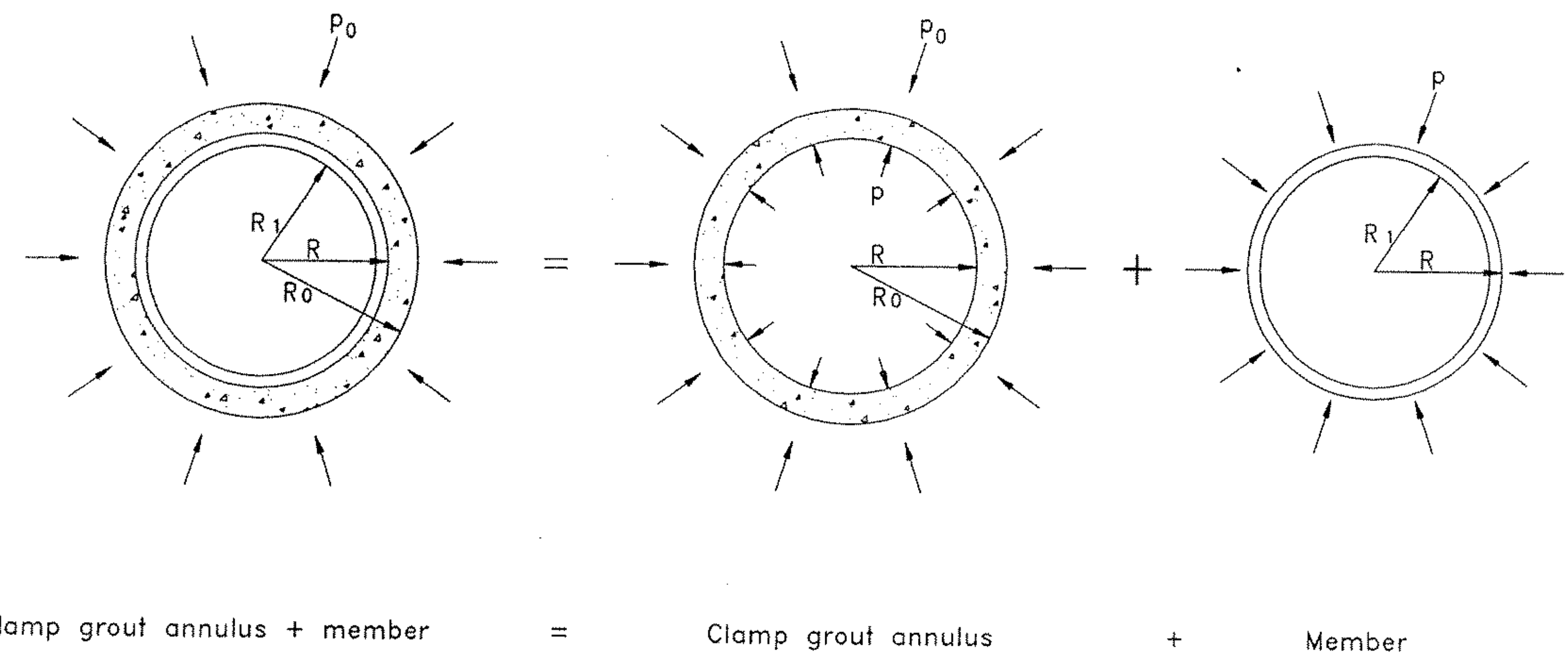


Figure 4.2.6: **Radial pressure distributions**

III 4.2.9.5 Member crushing

Where the studbolt load is such that member crushing is a problem, then member grout filling is a possible solution. Before deciding to adopt this option, two other solutions should be considered. These include increasing the clamp length or checking mill certificates for material local to the repair to ascertain whether steel yield strength is greater than that originally assumed.

III 4.2.10 **Check Adequacy of Grout**

This check, of course, is only applicable to unstressed grouted clamps/sleeves or stressed grouted clamps.

A minimum compressive strength of $40N/mm^2$ at 28 days is recommended to ensure durability of the repair. Compressive stress resulting from the studbolt tensioning is not normally a problem even when full strength has not been attained. However, a minimum compressive strength of $15N/mm^2$ should be achieved before the clamp is stressed, although this value might need to be greater depending on the maximum initial studbolt load.

Although grout crushing is not usually a problem, it should nevertheless be checked. The most critical case for grout crushing is at the location of the clamp extreme studbolts. At this position, in-plane and/or out-of-plane bending will generate highest compressive stresses in the grout. The compressive stress in the grout may be conservatively estimated from the summation of the grout stress due to the design studbolt loads (if applicable as is the case of stressed grouted clamps) and the grout stress due to the calculated studbolt load to prevent clamp half separation (from Section III 4.2.7 - whether or not this actually governed the design studbolt load).

III 4.2.11 **Elastomer Properties**

A polychloroprene (trade name "Neoprene") hardness of 65 IRHD (or 65 on shore A scale) should generally be specified. At this hardness durability and load transfer capability will maintain repair integrity. The polychloroprene should not absorb water to any appreciable extent. When the repair involves the use of a guide, it is usual to design for lateral loading only. In this instance the polychloroprene lining is usually ribbed and should be either PTFE or Xylan coated in order to reduce the possibility of attracting axial load.

The thickness requirement for clamps will depend on the type of repair adopted. If the clamp is a structural repair then the polychloroprene thickness will be relatively thin, but, must be chosen so as to absorb member irregularities. It is possible to notch the linear to take account of local obstructions (eg. doubler plates). The amount of load transfer depends on the relative stiffnesses of the member, clamp and polychloroprene. It is prudent to liaise with suppliers to check on the thicknesses available. (The standard thicknesses of ribbed neoprene, for use with guides, are 13mm and 25mm.)

For the temporary condition whilst the clamp is being deployed, the polychloroprene must be kept in position by bonding to the internal surface of the saddle. It is normal to ensure that the bond strength is greater than the slip stress. Once the clamp is in its service condition, the slip strength of the polychloroprene to member interface governs the ability of the clamp to transfer loads.

III 4.2.12 Design of Clamp Steelwork

III 4.2.12.1 General

The clamp steelwork is designed using standard engineering principles in accordance with current codes of practice. This section will cover design aspects peculiar to clamps which are not directly addressed in current codes of practice. The clamp geometry should be an important consideration whilst designing a repair scheme from the point of view of cost, fabrication and installation. The geometry may be influenced by limitations in repair size due to local restrictions of the damaged structure.

III 4.2.12.2 Design of flange plates

The flange plate provides three main functions, namely clamp rigidity, studbolt load transfer to stiffeners and reaction against shear caused by eccentricity of studbolt centre line to side plate. Figure 4.2.7 presents the general arrangement showing flange plate, stiffeners and side plate welded to the saddle. The flange plate must also resist hydrostatic load (unless the otherwise enclosed chambers are allowed to flood). If hydrostatic pressure becomes a problem resulting in excessive flange plate thickness then consideration may be given to providing holes to allow seawater to enter the cavity formed by the flange plate.

The governing design criteria for flange plates is usually bending resulting from studbolt load as opposed to hydrostatic load. Yield line analysis of the flange plate is the easiest method in determining strength. A factor of safety against plastic collapse of 1.8 is normally adopted. If a stiffener spacing is chosen such that there is little or no bending, then stiffener and side plate thickness will determine flange plate thickness if hydrostatic pressure does not.

III 4.2.12.3 Design of stiffeners and side plates

Stiffener spacing has the greatest effect on bending in the flange plate. The spacing should be selected such that the footprint of the spherical washers extends over the stiffeners and side plates. However, stiffener spacing must not be so close that welding becomes difficult. Generally, the stiffener spacing should exceed the stiffener outstand. If the stiffener outstand exceeds stiffener spacing, careful consideration should be given to the fabrication sequence.

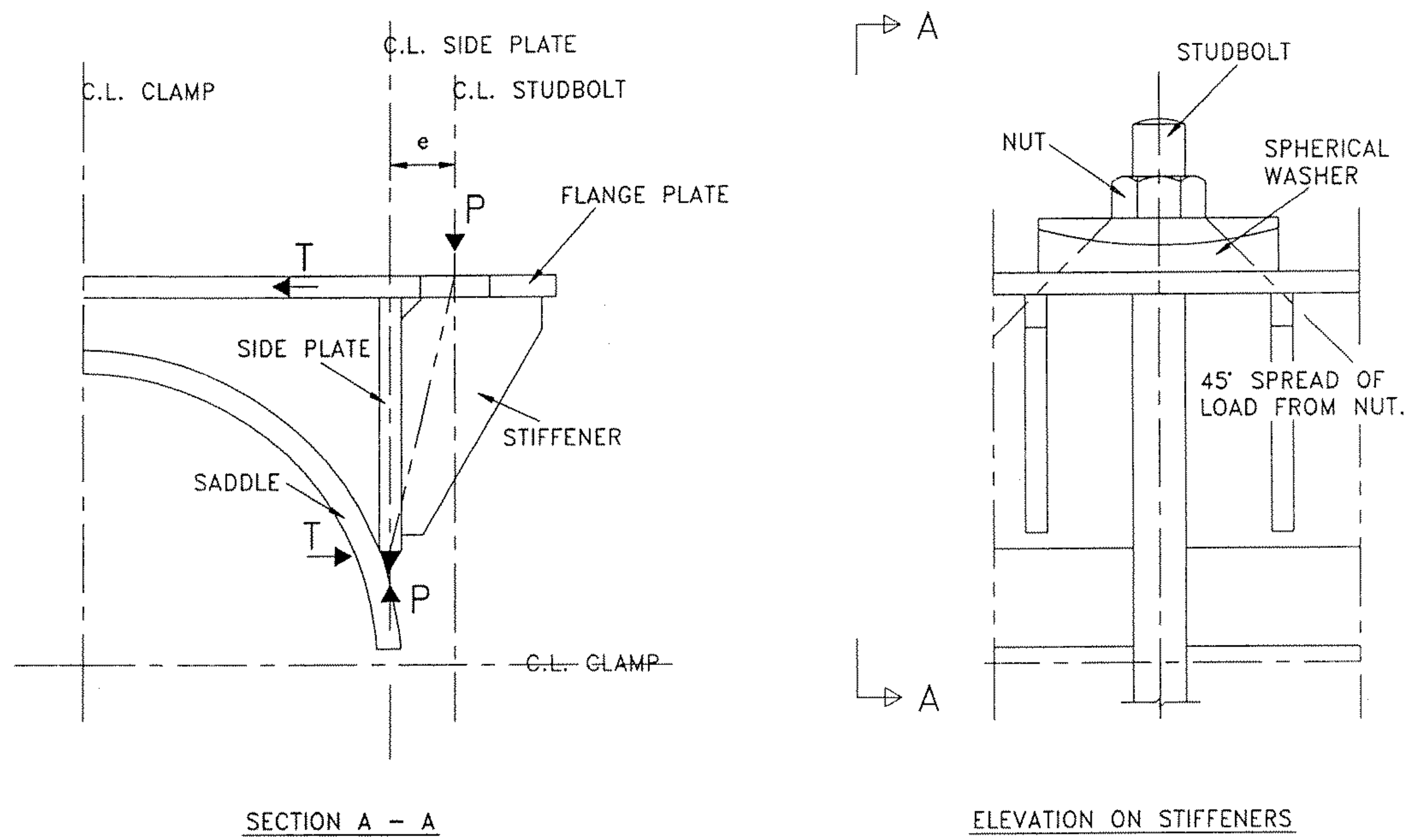


Figure 4.2.7: **Clamp steelwork details**

Studbolt loads derived in Section III 4.2.8 should be used in stiffener and side plate design. Design is carried out based on the stiffeners and a section of side plate acting as a section under an eccentric load. Bending caused by the eccentricity should be checked at suitable locations down the stiffener towards the side plate to saddle weld.

The studbolt load will distribute between the stiffeners and side plate. Each component should be designed once each reaction has been quantified. Stiffener to side plate weld must be designed for shear and the side plate should be designed for buckling between stiffeners.

The fillet weld attaching the side plate to the saddle must be designed for transverse shear (due to P and T in Figure 4.2.7) resulting from studbolt load, and also longitudinal shear resulting from the composite action of the clamp.

Consideration to installation aids should be considered at this stage, particularly when clamp hinges are required. This may be achieved by extending the stiffeners and staggering them on each clamp section so they form hinge plates.

III 4.2.13 Design of Grout Seals

Grout seals take the form of longitudinal and end seals. Longitudinal seals will provide a means of sealing between clamp saddles once the clamp is in position. End seals provide a means of sealing between the clamp saddle and member at each of the clamp ends.

End seals can be designed as either an external or internal seal as detailed in Figures 4.2.8 and 4.2.9. External seals require a greater amount of steel than internal seals and involve extra installation activities. The main advantage of external seals over internal seals is that all of the grout annulus can be utilised for tolerance. Internal seals can be installed in a lesser length of clamp which is particularly important where space is a limitation. Grout bags are a possible alternative but are cumbersome and require a greater length of clamp for installation.

Due to the manner in which end seals are activated, they need to be of a material which is soft enough to compress with the ability of relatively large deformations. Such a material is Sorbothane having a Shore hardness of 30 on the 00 scale. Consideration should also be given to properties of the material such as compressive modulus, tensile strength and compression set which influence the ability of the material to perform its task.

Longitudinal seals are typically formed in a channel section and are bonded to each side of the saddle split. Once the clamp is in position these longitudinal seals will meet and should compress to form a tight seal. The seals are usually formed using polychloroprene with a hardness of 35 IRHD. Consideration should also be given to properties of the material such as Young's modulus,

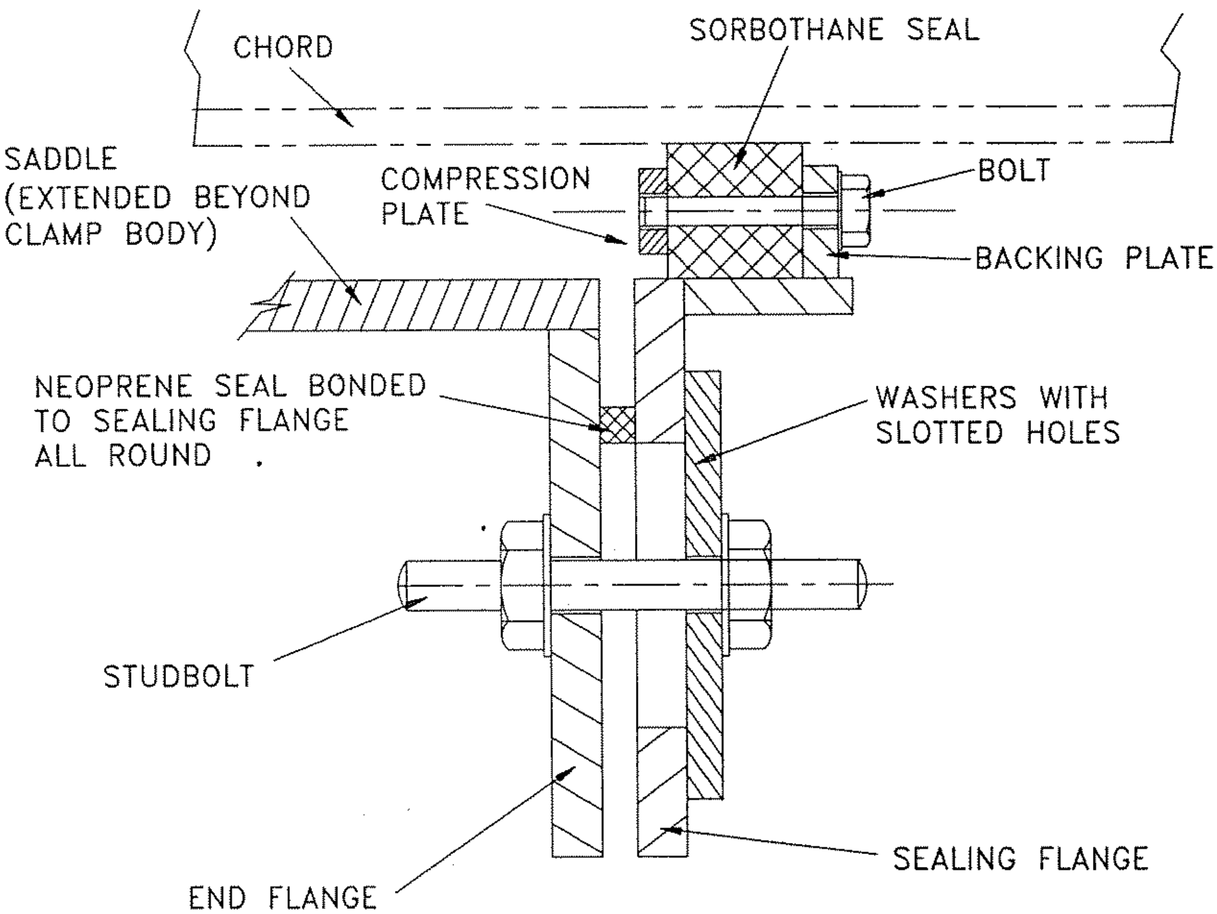


Figure 4.2.8: External seal example

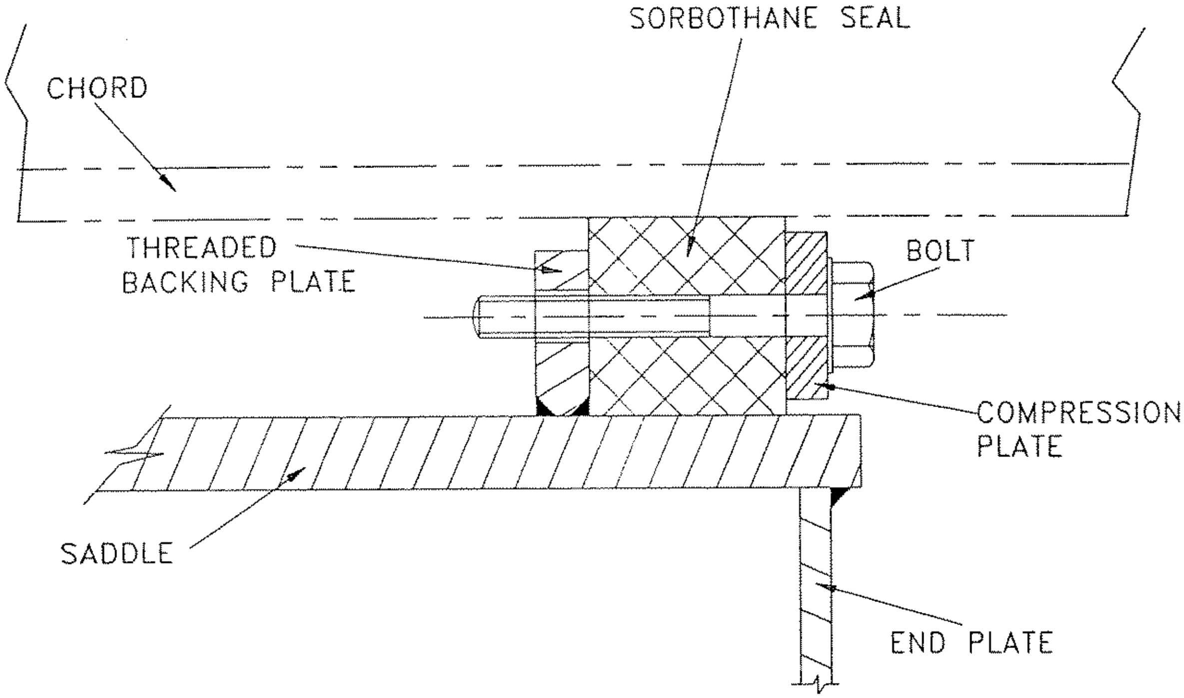


Figure 4.2.9: **Internal seal example**

bulk modulus, compression strain and water absorption which influence the ability of the material to perform its task.

All seal types should be designed to resist the differential pressure which will be applied during the grouting operation. This pressure will result from the head of grout and pumping pressure. These combined pressures may be partially mitigated by the external head of seawater. Losses in pressure due to friction in the grout line will be difficult to quantify. It is recommended that grout outlet/s remain open during the grouting operation.

Onshore trials are recommended to familiarise divers with clamp deployment and to test both the end and longitudinal seals. The pressure test should be carried out using a pressure that will simulate that expected during the offshore installation.

Care must be taken during transportation and deployment of the clamp to avoid possible damage to the seals. Procedures must allow for possible damage and have provisions for repair and/or replacement. These procedures should also have provision for potential problems during the grouting operation such as leakage.

III 4.2.14 Design of Installation Aids

There are a number of installation aids which are either necessary or optional to assist in clamp deployment as follows:

- padeyes
- centralisers
- stabbing guides
- hinges
- shackle holes
- grout inlets/outlets

Padeyes should be designed with an allowance for dynamic loading. Clamp weight should include all primary and secondary steel as well as installation aids and anodes where fitted. Liaison with the installation contractor is recommended to agree padeye locations so that clashes between slings and clamp steelwork, including any anodes, are avoided.

Centralisers perform an essential task in positioning the clamp before seal activation can commence. Once the grout has cured, but before studbolt tensioning commences, the centralisers must be released to prevent deformation of the existing member. Centralisers must be designed for the possible event of all or partial clamp weight together with dynamic loads being applied to the bearing tip during clamp installation. Positioning of centralisers on the clamp body must avoid possible clashes with items such as hydraulic jacks (though the centralisers could be removed before deploying the jacks).

There are types of repair where relative alignment of the clamp sections is difficult without the use of stabbing guides. The use of stabbing guides will also depend on the installation contractor's preference.

The majority of clamp installations will lend themselves to the use of hinges. Clamp deployment will generally determine the geometry. Hinges should be designed with consideration to the worst orientation possible of the clamp sections during installation. Slam loading from wave action, and therefore DAF allowances, should be considered in design. Spreader plates or props will be required to hold the clamp sections apart and should be designed accordingly.

The function of shackle holes is to aid installation by providing convenient sling attachment locations. Shackle holes are not a primary design feature, but consideration should be given to their position to maintain clamp integrity and aid installation.

The size and location of grout inlets/outlets may be determined by the requirements of the grouting contractor but, in any event, consideration should be given to their location in order to ensure complete grout filling of the annulus. It is recommended that extra inlets and outlets be provided for redundancy in case of blockage during the grouting operation.

III 4.2.15 Fatigue Checks

III 4.2.15.1 General

Fatigue checks should be considered for all clamp techniques. Fatigue checks may be required for the parent steelwork, clamp steelwork and the studbolts.

When the function of the clamp is to enhance the fatigue strength of a joint, then fatigue is a design condition itself. It is common practice to design for no load sharing between the clamp and the parent steelwork, ie. it is assumed that the member is severed and that **all** the load must be transferred to the clamp and 'bridge' over the damage. It is recommended that preference be given to a 'no load sharing' design because all clamping techniques completely cover the parent steelwork, preventing future inspection/monitoring of the damage. This is of particular importance for a joint where fatigue strength enhancement is required. However, in the case of strengthening or repair for static strength reasons, the engineer may wish to investigate load sharing.

III 4.2.15.2 Fatigue checks on existing steelwork

The introduction of a clamp onto a joint results in the reduction of hot spot stress due to the following reasons:

- the nominal loading of the member is resisted by composite action of the original member/clamp, leading to an apparent reduction in the SCF.

- the presence of the clamp (assuming that it is of the type which encloses the joint) restrains local chord wall deformations leading to direct reductions in the SCF.

III 4.2.15.3 Fatigue of clamp steelwork

Fatigue checks on saddle joints and platework may be required. As far as platework is concerned, traditional methods are applicable.

In a clamp around a nodal joint, the sleeves can be considered as intersecting tubulars (albeit split tubulars and with no chord plug). The assessment of data given in Part IV, Section IV 4.7.3.1, indicates that the application of tubular joint parametric equations, such as those in Appendix A, in conjunction with appropriate SN curves, is a conservative approach. As for the existing steelwork, the SCF is reduced due to composite action and the restraining effects on tubular wall deformation. It is therefore possible to allow for these factors in marginal cases, see Section IV 4.7.3.1 in Part IV for further details.

III 4.2.15.4 Fatigue of studbolts

The stress fluctuation in a studbolt may be conservatively calculated as follows. The flowchart given in Figure 4.2.10 illustrates the various steps involved. The clamp separation forces are related to the member fatigue load ranges by transfer functions. For member moments and shears, the transfer functions may be derived as given in Section III 4.2.7. The axial transfer function is derived by considering Poisson's contraction effects. Torsion does not cause a change in the studbolt load. The total separation force is found conservatively by adding all components. Since the clamp design ensures that there is no separation of the two clamp halves, the change in the studbolt load is related to the summed separation force as follows:

$$\Delta \text{ load per studbolt pair} = \frac{\text{summed separation force}}{1 + K_c/K_b}$$

where:

K_b = axial stiffness of studbolt pair

K_c = radial stiffness of grout annulus/jacket member/grout fill combination

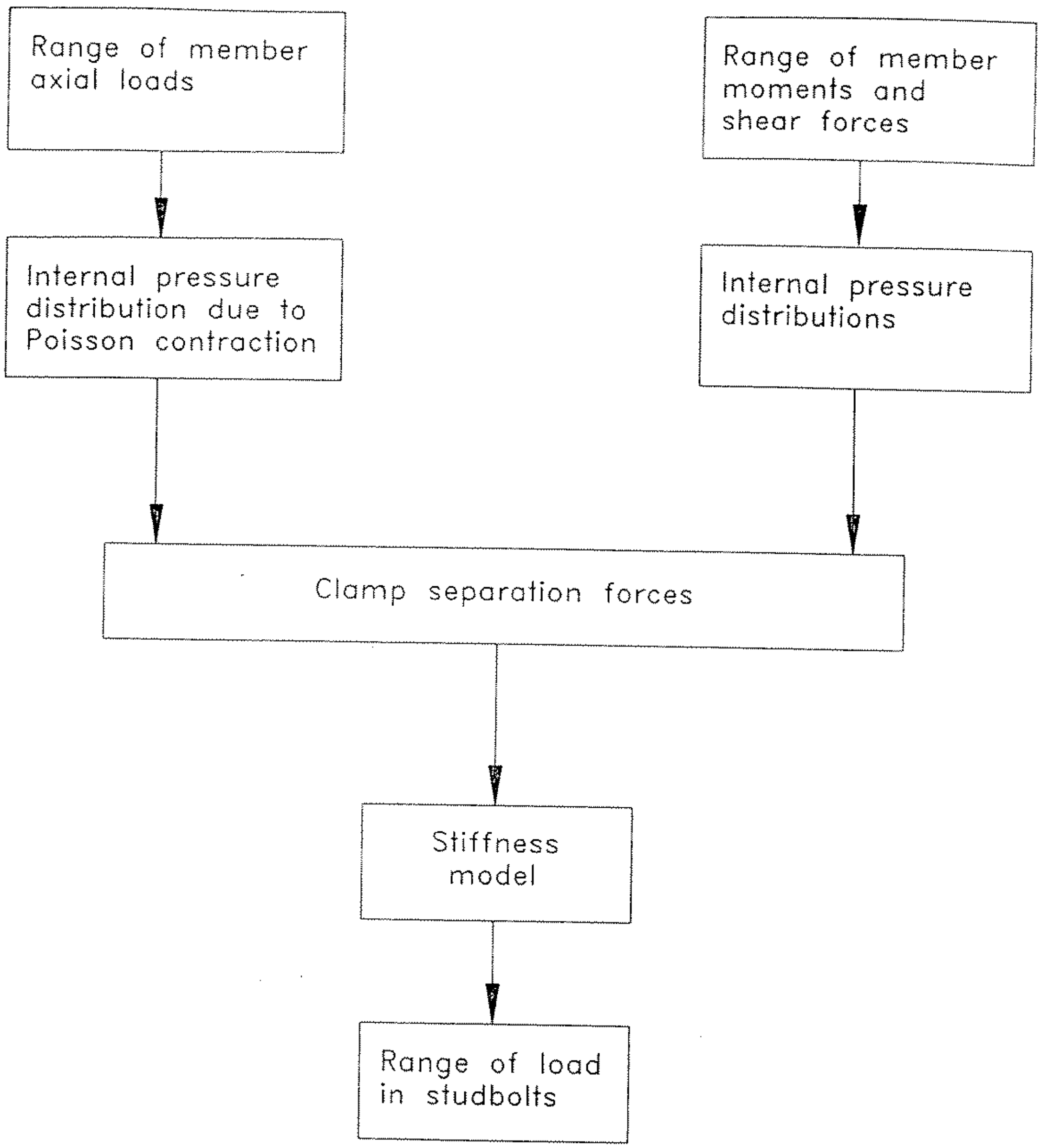


Figure 4.2.10: Flow chart for derivation of fatigue loads in studbolts

The maximum stress range in the studbolts is calculated from the greatest change of studbolt load.

Refer to Section III 7.2.3 for S-N behaviour of studbolts in grade B7/L7.

III 4.2.16 Corrosion Protection

Reinstatement of cathodic protection (CP) of the existing structure may be necessary where the installation of a clamp has led to the removal of an anode or anodes from the structure. Reinstatement can be achieved in one of two ways. The equivalent weight of removed anode(s) may be included in the CP for the clamp, provided there is enough space. A continuity strap between the clamp and structure will need to be provided. However, this approach may lead to excessive additional weight on the clamp. Another way of reinstating CP is to use bracelet anodes on the existing structure away from the repair location. This negates the requirement for continuity straps between the clamp and structure.

The clamp itself should be provided with CP. Occasionally the Operator may require the external surface of the clamp to be painted in addition to any CP. Some certifying authorities allow a reduction in current density, and hence the weight of anodes, when this is the case. The capacity of an anode is normally quoted by the anode manufacturer and their advice should be sought. Once the anode weight requirement has been calculated it is preferable to distribute the weight between clamp sections, according to their respective areas. If this is not possible then continuity straps must be provided for connectivity between clamp sections.

III 4.3 SPECIFICATIONS AND PROCEDURES

III 4.3.1 Introduction

For repair schemes that comprise clamps, it is necessary to prepare a number of specifications and procedures to ensure the use of appropriate materials, to specify fabrication tolerances for clamp steelwork and to ensure the successful installation and in-service performance of the clamp scheme.

Some specifications and/or procedures discussed within this section apply to all clamp types. Others are applicable to specific clamp types only. The applicability of each document is covered in each subsection below. Specifications and procedures that should normally be provided are as follows:

- steelwork specification
- fabrication addendum

- outline clamp installation procedure
- clamp seal (or elastomer) specification
- specification for the manufacture and testing of studbolts, nuts and washers
- grout mix and grouting procedure.

Each of the above documents should make reference, where appropriate, to the relevant Codes and Standards concerning each topic. These codes may cover material properties, tolerances, testing procedures, finishes and manufacture. The above-listed specifications and procedures are discussed in the following sub-sections.

III 4.3.2 Steelwork Specification

This document, which is applicable to all clamp types, should ensure that all steelwork meets the requirements of recognised codes of practice for offshore structural steel. The steel should be weldable and the specification should contain the requirements for properties such as carbon equivalent, chemical composition, minimum tensile strength, minimum yield strength, ductility, toughness, etc.

III 4.3.3 Fabrication Addendum

It is the purpose of this document to ensure that the achieved quality of fabrication is appropriate to the repair. The fabrication addendum is applicable to all clamp types and its extent depends on the base document (eg. EEMUA 158 or AWS D1.1). It should address the following five main areas:

- material preparation
- fabrication
- tolerances and distortion control
- inspection
- documentation.

These five areas should cover cutting and edge preparation, tolerances, fabrication sequence, welding, machining and finish of surfaces.

The documentation which the fabrication contractor is required to submit should also be covered, detailing such documents as quality assurance manual, welding procedures and qualifications, test certificates, survey reports, shop drawings and inspection records.

III 4.3.4 Outline Clamp Installation Procedure

This procedure should cover three main topics related to all clamp types. Outline clamp installation is the main section covering the sequence of events and tasks to be performed. This section must also specify the bolt tensioning procedure which should cover the following:

- calibration of hydraulic tensioning jacks
- grout strength requirements, prior to studbolt tensioning
- simultaneous studbolt tensioning/tensioning sequence
- load increments (often 3 No.) to full tension.

Other sections of this procedure should address pre-installation and post-installation activities. Pre-installation activities may include the assembly of an under-deck working platform, rigging, tugger line attachments on the jacket, cutting and removal of members and member cleaning.

Post-installation tasks may comprise the removal of manifolds used during hydraulic tensioning, placement of thread protection caps and visual checks to ensure satisfactory completion of the repair scheme. Pre- and post-installation activities will be specific to each repair site.

III 4.3.5 Clamp Seal (or Elastomer) Specification

This specification will apply to all clamp types except stressed mechanical clamps. In some instances, perimeter seals may also be specified for stressed mechanical clamps, in order to safeguard against crevice corrosion, although the effectiveness or indeed the necessity of these is unproven.

The purpose of this document is to stipulate elastomer properties in accordance with those assumed in design, to ensure satisfactory performance of the clamp during deployment and over the duration of the design life.

Unstressed and stressed grouted clamps differ from elastomer-lined clamps in that the elastomer function in grouted clamps is to act as a seal to prevent the seepage of grout during the grouting operation whereas in elastomer-lined clamps the lining is used as a load transfer medium between the clamp saddle and member. Hence seals need only have a short life; linings should have a long life.

A specification for bonding between the elastomer and steel is required. This specification will stipulate the bonding method, bond strength and tear resistance requirements and testing methods to prove the requirements. Allowable damage criteria must also be clearly stated, together with acceptable rectification procedures for damaged components.

III 4.3.6 Specification for the Manufacture and Testing of Studbolts, Nuts and Washers

This document is required for all clamp types and must address the material, manufacture and testing of studbolts, nuts and spherical washers. In addition, the specification should cover details of plating/coatings, quality assurance and transportation of components from the supplier to the fabrication yard.

Studbolts, nuts and spherical washers should be specified for material grade, heat treatment, chemical composition and material properties (tensile and yield strength, elongation, hardness, toughness), see Section III 7. Dimensional tolerances and threads must be specified for studbolts and nuts. Spherical washers provide a means of ensuring that the studbolts are not subjected to excessive bending loads. Several manufacturers provide a spherical washer assembly which can achieve this.

Corrosion protection of studbolts, nuts and washers is necessary to ensure the longevity of each component. Plating and coating are means by which this may be achieved. The specification must provide the required standards of plating and coating, whilst ensuring that the free running of nuts on the threads is not affected by such coatings.

Quality assurance of the mechanical properties of studbolts, nuts and washers is essential. Therefore the specification must cover material property tests, proof load tests and non-destructive tests. The requirements for certification and records should be specified.

III 4.3.7 Grout and Grouting Procedure

This document will only apply to stressed and unstressed grouted clamps. It must specify the following:

- grout mix and testing
- clamp grouting procedure.

Grout mix properties are important because the grout will act as a load transfer medium between the clamp and member. It is therefore important that the grout mix results in the required short and long term strength, limits the amount of shrinkage and is sufficiently fluid to completely fill the annulus between the saddles and the tubular member.

For either clamp type it is essential to specify procedures which ensure correct seal alignment and positioning. This is true for both longitudinal seals and end seals.

Prior to grouting a leak test must be performed to ensure correct functioning of seals and subsequent retention of grout. A procedure must be specified for the operation of clamp grouting to ensure adequate void filling with good consistency grout, grout sampling at appropriate intervals and the correct operation and sequence of opening and closing the inlet and outlet valves. It is essential that procedures adopted preclude any possibility of air entrapment. This procedure must also cover the contingency for a short or long stoppage. If the stoppage exceeds a pre-defined duration it will be necessary to execute annulus flushing procedures.

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4.5 Roark, R.J. and Young, W.C. 'Formulas for Stress and Strain'. McGraw-Hill International Book Company.

III 5 MEMBER GROUT FILLING

III 5.1 INTRODUCTION

This section recommends design methods to predict the ultimate axial load carrying capacity of undamaged and damaged structural tubular members which are partially or fully grouted. The configurations specifically examined are illustrated in Figure 5.1.1.

Grout filling offers benefits to undamaged members but offers greater benefits to members suffering limited damage following impacts from vessels or dropped objects. For the former, greater strength and stiffness is available without any potential increase in wave load forces although at some expense in weight and inertia (earthquake) loads. For the latter, growth in dent damage is inhibited enabling the full strength of the cross-section as damaged to be achieved. In the case of partial filling, the benefits of increased stiffness cannot necessarily be relied upon.

The practical difficulties of grout filling are not to be underestimated. As indicated in Figures 5.1.2 and 5.1.3, incomplete filling can easily occur leading to the loss of both stiffness and strength benefits. However, where lack of stiffness may not be a problem as, for example, with some leg members, only the strength benefits need be exploited by deliberately restricting the filling to the area of local damage.

In considering design methods, these fall into two main categories, codified and published. The codified methods relate only to undamaged members so there is some merit in considering these in isolation. However, because the data available by which to verify grouted undamaged tubulars is very limited (there is more relating to concrete-filled members but these are not considered appropriate to the present case), and because it is highly desirable that damaged tubulars with vanishingly small levels of damage can be treated as undamaged members, the approach adopted is to begin with codified methods. In doing so, properties relating to the damaged cross-section are introduced, and the misalignment and/or additional out-of-straightness (O-O-S) caused by the impact are taken into account.

In the light of the preceding, and because of differences in behaviour between fully and partially grouted tubulars, fully and partially grouted members will be treated separately below. Fully grouted members are treated in Section III 5.2. Factors affecting strength are reviewed first followed by the presentation of a suitable design approach. In Section III 5.3, partially grouted members are treated similarly.

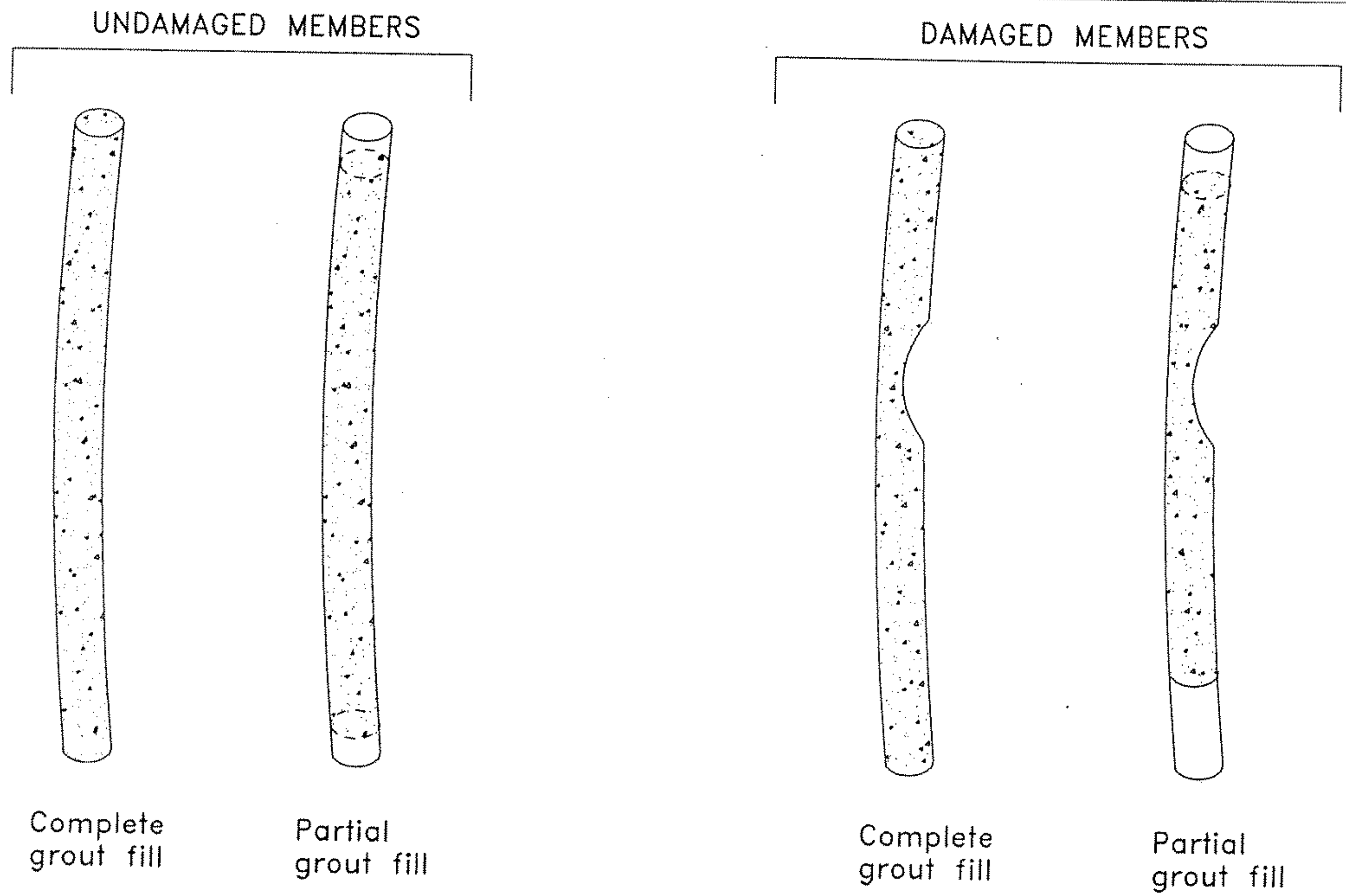


Figure 5.1.1: **Grout filled members considered in this section**

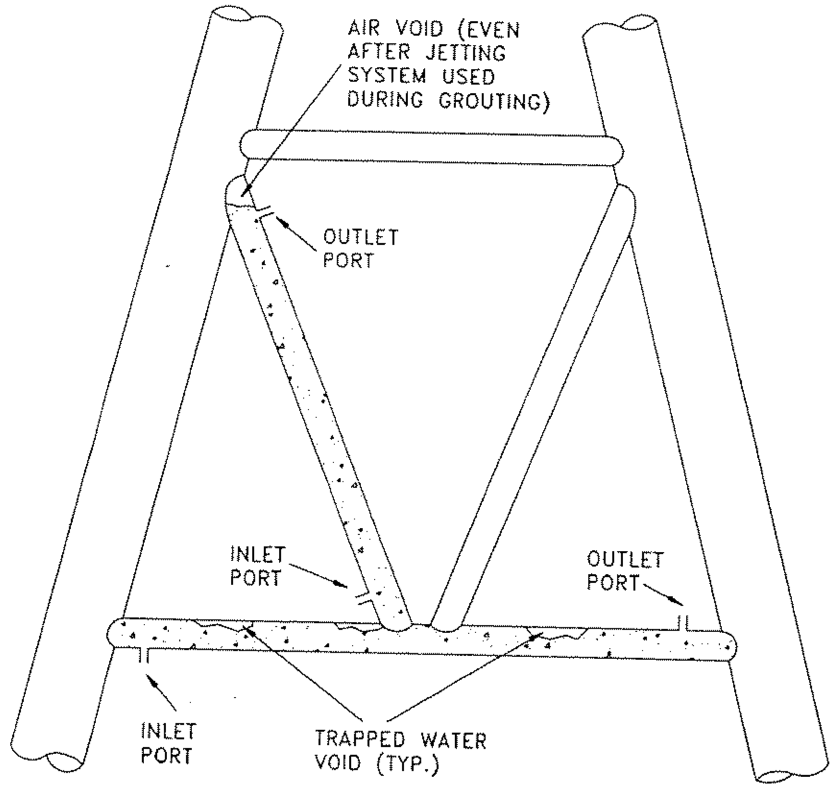


Figure 5.1.2: Typical incomplete grout-filled tubular members

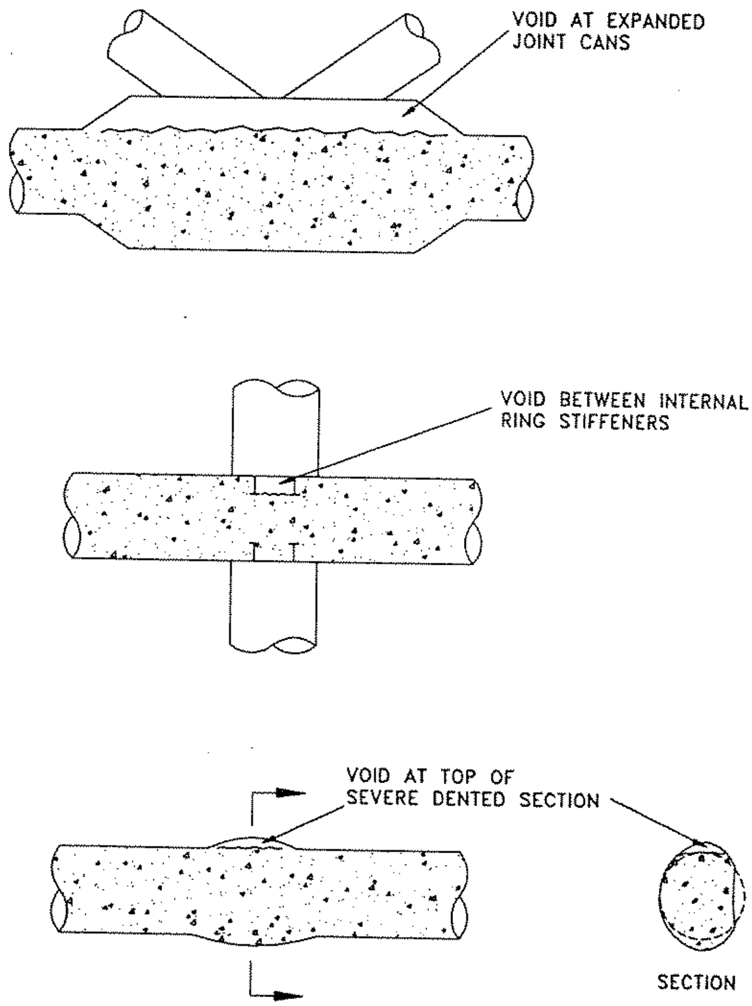


Figure 5.1.3: Incomplete filling at joints and damage locations

III 5.2 FULLY-GROUTED UNDAMAGED AND DAMAGED TUBULARS

III 5.2.1 Factors Affecting Strength

Fully grouted undamaged tubulars behave essentially as beam-columns and are therefore similarly affected by the same parameters which control beam-column response. These include:

- the 'squash' capacity P_u reflecting the ability of a short length of the member to sustain axial compression loading. This capacity is equal to the sum of the capacities of the individual components comprising the cross-section which in turn are usually equal to the product of the material strength and relevant cross-sectional area.

- the plastic moment capacity M_u which is the equivalent of the squash capacity for a section subjected to flexure. However, in determining this plastic moment capacity, account is taken of the inability of grout to sustain tension loading. Under squash conditions, steel is considered equally effective in tension and compression.

- the Euler buckling load P_E which is a measure of a member's resistance to overall buckling as a pin-ended column. This is a function of the flexural stiffness EI (second moment of area x elastic modulus) and the square of the length, thus $P_E = \pi^2 EI/L^2$. Members having low flexural stiffness or of long length have low Euler loads and are susceptible to buckling; these are described as 'slender'. Members of high flexural stiffness or of short length have high Euler loads and are susceptible to squashing rather than buckling; these are described as 'stocky'.

- the reduced slenderness parameter, λ, which is a measure of the slenderness or stockiness of a member. This parameter equals $\sqrt{P_u/P_E}$ and approaches zero for stocky members and infinity for very slender members although practical upper limits for this parameter are in the range of 2.0. Members with λ in the range 0.8 to 1.1 lie in the region where squash and buckling loads coincide. Because of this, such members are particularly sensitive to imperfections, both geometrical and material, and therefore to damage, both local and overall.

- material properties of strength and stiffness. For steel, yield stress f_y and elastic modulus E_s are the relevant basic variables. The method of determination can affect the values attributed to these variables. Yield stress is enhanced by the standard coupon test by between 5 and 15% due to strain rate effects. In practice, wave loading generates dynamic effects so the mill certificate value is probably most appropriate, as currently practised.

For grout, the compressive strength f_{cu} and elastic modulus E_g are the relevant variables. The values of these variables are dependent on the method of measurement as well as the curing procedure adopted. Measurements can be made on cubes or cylinders of different dimensions. The strength determined from a cylinder has been found to approximate 80% of the cube strength: the latter seems largely independent of size. Nevertheless, 75mm (3") cubes are the standard reference sizes. Because of major differences between a cube under test and grout in a member (strain rate, constraints), only some two-thirds of the grout strength is effective in prototype structures. Grout modulus has been found to be a function of grout strength reflecting a critical strain condition to describe grout failure. Different equations, however, have been derived to relate f_{cu} and E_g.

In deriving section properties, the difference in modulus between the steel and the grout is normally accounted for by reducing the stiffness contribution of the grout by the ratio of E_s/E_g, ie. the modular ratio m. Properties are then determined assuming the section, termed the transformed section, is composed entirely of steel.

- diameter to thickness ratio (D/T). For steel alone, squash is possible on members in compression when $D/T < 60$[5.1] (this limit is strictly also a function of f_y and E_s), and yield in bending on members with D/T ratios up to almost 120[5.1]. Beyond these values, reduced material strength applies because of the susceptibility of the section to local buckling. The corresponding limits when tubulars are grouted have not been established. From model tests, the largest D/T ratio examined was 76 without any obvious destabilising effects from local buckling. With grout inhibiting ovalisation and any inward buckling in undamaged members, it is expected that the D/T ratio below which yield can be expected to be achieved will lie in the range 100 to 120.

  When denting is present, grouting again inhibits ovalisation and inward buckling as well as preventing the growth of the dent. However, the dent itself can possibly precipitate forms of local buckling not normally encountered in undamaged tubulars. These have not been highlighted in model tests to date, so the full strength of the local damaged section can be realised for D/T ratios up to 80, the approximate limit for tests so far conducted.

- damage extent is in the form of dent depth d and overall bow d_1 which are usually considered in their normalised forms d/D and d_1/L. Increasing values of d/D imply greater relative cross-sectional damage generating an increasingly elongated section with a decreasing second moment of area. This increases the susceptibility to Euler buckling. With increasing d/D, the area available for grouting decreases so P_u also decreases with increasing relative dent damage.

Bow damage can be considered as an enhanced initial O-O-S. As this increases the eccentricity of any axial loading, beam-column strength will decrease as the relative bow damage increases.

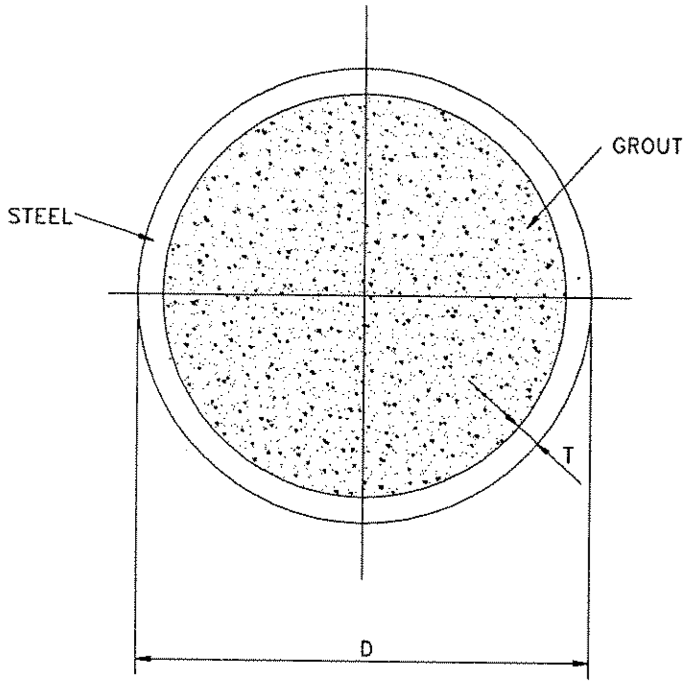
III 5.2.2 Design Procedure

The design procedure presented below is a distillation of several methodologies for dented tubulars as originally exploited by Parsanejad[5.2] augmented by the use of the full compressive strength available to stocky grout-filled members and the corresponding full plastic moment capacity. These replace the first yield alternatives adopted by Parsanejad. The enhancements were proposed by Loh in a major study of this topic[5.3] which exploited the benefits already recognised for concrete filled tubulars and reflected in relevant codes, namely, BS5400: Part 5[5.4] and AISC-LRFD[5.5]. These codes use column curves not widely known in offshore design. The well known RP2A column curve[5.1] has been adopted in their place. However, in order to achieve the best correlation with test data, and thereby probably the best representation of the physical response the compression-moment interaction approach reflected in BS5400 is used rather than that contained in RP2A.

The basic cross-sectional properties of an intact fully grouted tubular are as presented in Figure 5.2.1. For a dented section, Figure 5.2.2, these are modified as shown by some of the first group of equations listed in Step 2 of the following design procedure. From these the transformed section properties are determined from which the Euler load is calculated. Assuming the squash load is equal to the sum of its components, the reduced slenderness parameter can be found and thereby the column strength P_{col} based on the RP2A column curve K_1 (see Step 3). The plastic moment capacity of the dented steel section is then calculated and augmented by the grout flexural capacity to enable the full plastic moment strength M_u to be determined (Step 4).

For low levels of axial load, the beam-column flexural strength can be greater than M_u. The extent of this is determined by K_2. K_3 is then a measure of the degree of non-linearity over the remaining region of interaction, a positive value generating a convex shaped curve (parabolic), a negative one a concave form. These quantities are derived in Step 6. The eccentricity, e, in Step 6 is calculated as the ratio of applied moment to applied axial force, M/P. Depending on the total eccentricity of the axial load, Δ, the beam-column strength is then given by P (Step 7).

It is demonstrated in the Background Document that this methodology does not suffer from scale effects, and can therefore be used for the design or assessment of prototype structures. The available data covers dents up to 0.16 of the member diameter.



A_s	=	steel cross-sectional area	=	$\pi\,(D\text{-}T)T$
A_g	=	grout cross-sectional area	=	$\frac{\pi}{4}\,(D\text{-}2T)^2$
I_s	=	steel second moment of area	=	$\frac{\pi}{64}\left[D^4 - (D-2T)^4\right]$
I_g	=	grout second moment of area	=	$\frac{\pi}{64}\left[(D-2T)^4\right]$

Figure 5.2.1: Cross-sectional properties of a grout-filled undamaged member

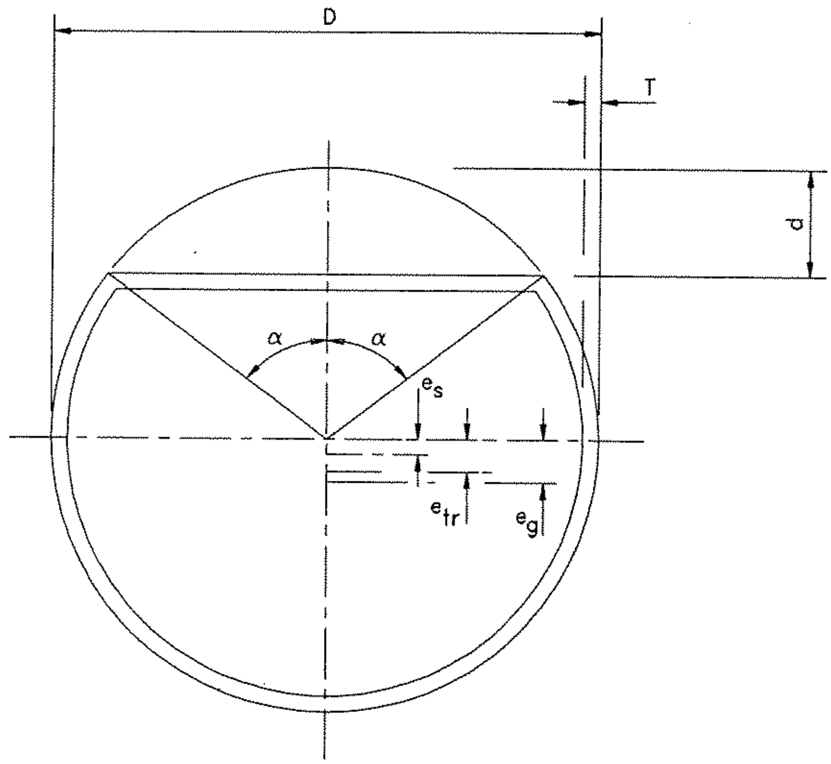


Figure 5.2.2: Cross-section at dent

DESIGN METHODOLOGY FOR FULLY OR PARTIALLY GROUTED, DAMAGED AND UNDAMAGED, TUBULAR MEMBERS

STEP 1 - Define data

D	=	outside diameter
T	=	steel thickness
L	=	effective buckling length of member
d	=	dent depth
d_o	=	overall bow
f_y	=	steel yield stress
f_{cu}	=	unconfined grout cube strength
E_s	=	Young's modulus for steel
E_g	=	modulus of grout
e	=	effective eccentricity of axial load (giving end moment)
β	=	ratio of smaller to larger end moment

STEP 2 - Calculate geometric and other derived quantities

$$m = E_s / E_g$$

$$\alpha = \cos^{-1}(1 - 2d/D)$$

$$D' = D - T$$

$$A_s = \pi D' T - D' T(\alpha - \sin\alpha)$$

$$A_g = (D' - T)^2(\pi - \alpha + \tfrac{1}{2}\sin 2\alpha)/4 \qquad \text{* (see note)}$$

$$A_{tr} = A_s + A_g / m$$

$$e_s = \tfrac{1}{2} D'^2 T \sin\alpha (1 - \cos\alpha) / A_s$$

$$e_g = (D' - T)^3 \sin^3\alpha / 12 A_g \qquad \text{* (see note)}$$

$$e_{tr} = (A_s e_s + A_g e_g / m) / A_{tr} \qquad \text{(not required if } I_{tr} \text{ is approximated)}$$

$$I_s = D'^3 T(\pi - \alpha - \tfrac{1}{2}\sin 2\alpha + 2\sin\alpha\cos^2\alpha)/8 - A_s e_s^2$$

$$I_g = (D' - T)^4(\pi - \alpha + \tfrac{1}{4}\sin 4\alpha)/64 - A_g e_g^2 \qquad \text{* (see note)}$$

$$I_{tr} = I_s + I_g/m + A_s(e_{tr} - e_s)^2 + A_g(e_{tr} - e_g)^2/m$$
$$\approx I_s + I_g/m \qquad \text{(recommended)}$$

Continued..

STEP 3 - Determine squash, Euler and column capacities
$P_u = A_s f_y + 0.67 A_g f_{cu}$ $P_E = \pi^2 E_s I_{tr} / L^2$ $\lambda = \sqrt{(P_u / P_E)}$ $K_1 = 1 - \lambda^2/4 \quad$ for $\lambda < \sqrt{2}$ $\quad\;\; = 1/\lambda^2 \quad$ for $\lambda \geq \sqrt{2}$ $P_{col} = K_1 P_u$
STEP 4 - Determine ultimate bending capacity
$M_p = ((D' + T)^3 - (D' - T)^3) f_y / 6$ $M_{pd} = M_p (1 - 0.5d/D' - 1.6(d/D')^2)$ $\rho = 0.6 f_{cu} / f_y$ * (see note) $n' = 5.5 (M_{pd} / M_p) (\rho D' / t)^{0.66}$ $M_u = M_{pd} (1 + 0.01 n')$
STEP 5 - Calculate constants for triaxial containment effects
$\phi = 0.02 (25 - L/D') \quad \geq 0$ $\delta = 0.25 (25 - L/D') \quad \geq 0$ $C_2 = (1 + \phi + \phi^2)^{-0.5}$ $C_1 = 4 \phi \delta C_2$
STEP 6 - Calculate quantities for use in interaction equation
$\gamma = 0.67 A_g (f_{cu} + C_1 f_y T / D') / P_u$ $K_{20} = 0.9 \gamma^2 + 0.2 \quad \leq 0.75$ $K_{30} = 0.04 - \gamma/15 \geq 0$ $K_2 = K_{20} (115 - 30 (2\beta - 1)(1.8 - \gamma) - 100\lambda) / (50(2.1 - \beta))$ but $0 \leq K_2 \leq K_{20}$ $K_3 = K_{30} + \lambda ((0.5\beta + 0.4)(\gamma^2 - 0.5) + 0.15) / (1 + \lambda^3)$ * (see note) $T_2 = 4K_3 / K_1$ $T_1 = 1 - K_2 / K_1 - T_2$ $\Delta = d_o + e$

Continued..

STEP 7 - Calculate axial capacity allowing for moments	
<u>Fully Grouted Member</u> P = $\frac{1}{2}(-M_u^2/P_{col}T_2\Delta^2 - T_1 M_u/T_2\Delta \pm ((-M_u^2/P_{col}T_2\Delta^2 - T_1 M_u/T_2\Delta)^2 + 4M_u^2/T_2\Delta^2)^{0.5})$ <u>Partially Grouted Member</u> P = $1/(1/P_{col} + T_1\Delta/M_u)$	
Note:	The terms above marked with a * should be set equal to zero when the member is partially grouted.

III 5.2.3 Design Considerations

From comparisons with screened data relating to fully grouted damaged and undamaged tubulars subjected to axial compression and combined compression and flexure, statistics concerning modelling accuracy have been determined. These are mean (or bias) of 1.079, standard deviation (sd) 0.142, and coefficient of variation (cov) 0.132, based on a sample size of 62. The best estimate of strength is found by multiplying the value of P derived by the bias, ie. 1.079. The characteristic strength (95% survivability at the 50% confidence level) is 1.079 - 1.653 x 0.142 = 0.844 times P. This value might be reduced should the member in question be considered critical, for example, a leg of a 4-leg jacket. Thus:

$$P_{mean} = 1.08P$$
$$P_{characteristic} = 0.844P$$

For use in a reliability evaluation, the mean and standard deviation would be used directly together with corresponding values for loading. A reliability index of 2.5 to 3.0 would produce a level of safety similar to that interpreted for current North Sea component designs. For critical members, 3.5 or even 4.0 might be more appropriate.

When evaluating an in-situ member following damage or for strengthening, the level of stressing already present cannot be ignored. A simple but acceptable approach to account for this would be to reduce the yield stress by an amount equal to the average axial stress in the member. This reduced value of yield stress would be used in place of f_y in the equations in the methodology presented in Section III 5.2.2.

$$f_y = \text{yield stress - average axial stress}$$

One of the limitations of the data examined to date is that all local damage has been in the form of denting. Other shapes of damage will occur in practice. These have been investigated in the case of tubulars where no grout infilling has been applied. In this the residual compressive strength of the damaged tubulars was determined by testing[5.6]. It was found that the residual strength was insensitive to the shape and location of the local damage. It also found that residual strength was insensitive to the shape of the bow deformation.

III 5.2.4 Practical Considerations

The practice of grouting offshore is well advanced in view of its widespread application in pile-sleeve connections. However, additional safeguards are necessary concerning filling and heat of hydration. Filling has to be 100% to ensure full composite action ensues and that no local buckling can occur at voids caused through access difficulties as illustrated in Figures 5.1.2 and 5.1.3.

Heat generated by hydration can be considerable especially when filling large diameter tubulars. The use of inert fillers to help reduce the effects of the heat such as thermal cracking can be considered. Onshore trials are recommended to help quantify the extent of this problem.

Onshore trials including correct conditions for curing are also recommended to provide a check on grout quality particularly should a high strength variety be required.

III 5.3 PARTIALLY-GROUTED UNDAMAGED AND DAMAGED TUBULARS

III 5.3.1 Factors Affecting Strength

It is apparent from relevant test results[5.7, 5.8] that for incomplete filling, full composite action cannot be guaranteed to occur. Nevertheless, the presence of the grout has been demonstrated to inhibit the onset of local buckling and the growth of local denting. Thus, compared with fully grouted members, partially grouted members have a stiffness based on the steel section only. Similarly, the strength is dictated by the steel although the local buckling which occurs in the absence of grout is inhibited, as it is for the fully-grouted members as discussed in Section III 5.2.1.

III 5.3.2 Design Procedure

The recommended approach is based directly on the procedure presented in Section III 5.2.2 for fully grouted members by assuming the grout does not contribute to either the axial compressive strength or the plastic moment capacity, and that the interaction between axial and flexure is linear. This is achieved by setting the terms marked with an asterisk in the methodology presented in Section III 5.2.2 to zero, as indicated therein. In this case, the calculation of the beam-column strength P is relatively simple.

III 5.3.3 Design Considerations

Based on a screened set of data, the statistics of the modelling accuracy are bias = 1.094, standard deviation = 0.088, and cov = 0.081, based on a sample of 28 models. The characteristic strength (95% survivability at the 50% confidence level) is 1.094 - 1.663 x 0.088 = 0.948 times P. Thus:

$$P_{mean} = 1.094P$$
$$P_{characteristic} = 0.948P$$

Compared with fully grouted members where grout strength and modulus contribute directly to a member's strength and stiffness, in this case there are no obvious criteria for helping to select appropriate grout properties. In this sense, quality control of the grout will be less onerous.

In implementing checks on partially filled tubulars, the capacity has to be checked within and outside the grouted region. Outside the region, the usual RP2A[5.1] requirements are invoked.

Other relevant design considerations are given in Section III 5.2.3.

III 5.3.4 Practical Considerations

Similar requirements exist here as for fully grouted members. Generally heat of hydration will cause fewer concerns. However, the introduction of, for example, grout bags may pose some additional problems particularly by prolonging underwater activities.

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III 6 GROUT-FILLED TUBULAR JOINTS

III 6.1 INTRODUCTION AND APPLICATION

The following procedures are concerned with the static and fatigue capacity of grouted and double-skin joints. The procedures have been prepared on the basis of findings from an exhaustive appraisal of this technology in Part IV of this document.

Grout-filling of chord members to enhance the static strength and fatigue endurance of tubular joints is recognised as perhaps the most cost-effective and technically efficient solution available for the strengthening, modification and repair of offshore installations.

The practice described in the following sub-sections has been calibrated, where possible, against test data in Part IV. However, there are significant gaps in the data from the standpoint of present-day potential applications, ie. the range of geometries, configurations, load cases and conditions for which chord grout-filling may be considered relevant and appropriate. Therefore, pending the generation of new data and information for this technology and further calibration and enhancement of the practice presented below, it is recommended that specialist advice is sought on a case-by-case basis, to judge and design the need for, and extent of, any further back-up experimental/numerical data required to support the practice defined below for the case under consideration.

The procedures herein apply to grout-filling of tubular joints, fabricated from steel plate, satisfying the requirements of API RP2A[6.1] or HSE Guidance Notes[6.2], or equivalent practices. The grout mix should meet the minimum requirements laid down in the HSE Guidance Notes for pile-sleeve connections, or equivalent specifications. The minimum grout compressive strength as measured by tests on 75mm cubes shall be 41.4 N/mm^2 at 28 days.

The procedures detailed below relate to T/Y, DT/X and non-overlapping K/YT joints. Joints of any other configuration shall be given special consideration. The geometric notation of a grouted joint is presented in Figure 6.1.1.

III 6.2 DETERMINATION OF DESIGN LOADS

The basic techniques of global analysis for simple joints are described in the HSE Guidance Notes, and are applicable to grouted/double-skin joints. Joint stiffness is generally increased, and the assumption of rigidity is reasonable for global considerations.

$\alpha = 2L/D$

$\beta = d/D$

$\gamma = D/2T$

$\tau = t/T$

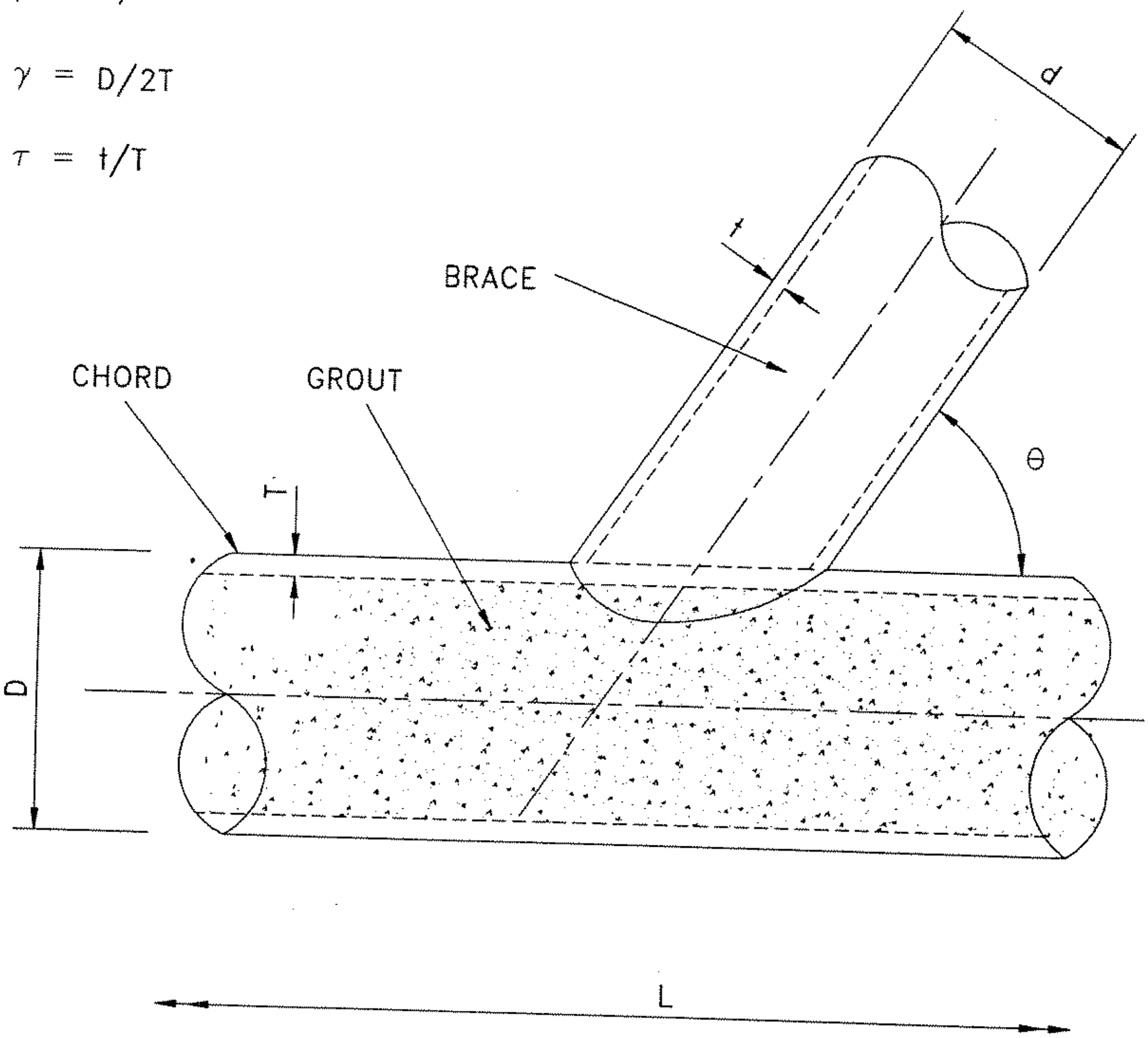

Figure 6.1.1: Notation for grouted joints

III 6.3 METHODS FOR DETERMINATION OF LOCAL JOINT BEHAVIOUR

In addition to provisions contained in the HSE Guidance Notes and Reference 6.1, the following guidelines should be considered:

- Numerical techniques

  In analysing grouted or double-skin joints, the problem arises of modelling contact at grout-steel interfaces. The simplest approach to the problem is to use a gapping technique to enable the interface to separate if tensile stresses in excess of the bond strength are detected. It is also possible to model friction, permitting slippage across the interface.

  For all types of nonlinear problems, the solution must be found by iteration. It is generally necessary to apply loads incrementally, performing iterations after each load step. Nonlinear capabilities are offered by a number of general purpose finite element codes, in addition to packages specifically written for nonlinear analysis. The general purpose codes are normally user-oriented, although the nonlinear capabilities may be very limited. A good nonlinear package will offer an extensive library of material models, geometric nonlinearity based on a large strain (eg. Eulerian or Lagrangian) formulation, with appropriate convergence schemes.

- Experimental techniques

  Local joint behaviour can be determined with certainty only by steel specimens. A substitute filler material for acrylic models has been used but correlation with steel specimens has not been possible.

III 6.4 STATIC STRENGTH DESIGN

III 6.4.1 <u>Design Acting Loads</u>

Design acting loads for each load component should be taken as:

P_d, M_{di}, M_{do} = calculated applied load (axial, in-plane and out-of-plane bending, respectively)

III 6.4.2 <u>Classification</u>

Each joint should be considered as a number of independent chord/brace intersections and the capacity of each intersection should be checked against the design requirements.

Each chord/brace intersection should be classified as Y, K, or X according to their configuration and load pattern for each load case. Examples of joint classification are given in API RP2A[6.1] and the HSE Guidance Notes[6.2].

III 6.4.3 Factors Affecting Strength

The following principal factors have been shown to affect the strength of a given grouted or double-skin tubular joint:

(i) Chord outside diameter

(ii) Brace outside diameter

(iii) Chord wall thickness

(iv) The included angle between chord and brace

(v) Gap between braces (for K joints only)

(vi) Chord material yield stress

(vii) Grout strength (minimum requirement of 41.4 N/mm^2 at 28 days)

(viii) Properties of pile for double-skin joints.

III 6.4.4 Grouted Joints

III 6.4.4.1 Capacity

The capacity of a grouted joint subjected to unidirectional loading may be derived as follows:

$$P = Q_u Q_f \frac{F_y T^2}{\sin \theta} \quad \text{.... 6.4.1}$$

$$M_i, M_o = Q_u Q_f \frac{F_y T^2 d}{\sin \theta} \quad \text{.... 6.4.2}$$

where P = strength for brace axial load

M_i = strength for brace in-plane moment load

M_o = strength for brace out-of-plane moment load

F_y = characteristic yield stress of the chord member at the joint (or 0.7 times the characteristic tensile strength if less). If characteristic values are not available specified minimum values may be substituted.

T	=	chord thickness
d	=	brace diameter.

P, M_i and M_o should not be allowed to exceed the brace capacity, and should not be taken to be less than the capacity for as-welded equivalent joints defined in the HSE Guidance Notes.

Q_f is a factor to allow for the presence of axial and moment loads in the chord. Q_f is defined as:

$$Q_f = 1.0 - 1.638\,\lambda\,\gamma\,U^2 \text{ for extreme conditions}$$

$$= 1.0 - 2.890\,\lambda\,\gamma\,U^2 \text{ for operating conditions}$$

where λ = 0.030 for brace axial load

= 0.045 for brace in-plane moment load

= 0.021 for brace out-of-plane moment load

and
$$U = \frac{\sqrt{(0.23P_dD)^2 + M_{di}^2 + M_{do}^2}}{0.72D^2\,T\,F_y}$$

All forces in the function U relate to the calculated applied loads in the chord. Note that U defines the chord utilisation factor. No composite action should be considered.

Q_f may be set to 1.0 if the following condition is satisfied:

$$\text{chord axial tension force} \geq \frac{1}{0.23D}\left(M_{di}^2 + M_{do}^2\right)^{0.5}$$

with all forces relating to the calculated applied loads in the chord.

Q_u is a strength factor which varies with the joint and load type. Q_u is defined in Table 6.4.1.

Load Direction	Joint Configuration		
	Y	X	K
Axial compression	*	*	*
Axial tension	$2.5\,\beta\,\gamma\,K_a$	$2.5\,\beta\,\gamma\,K_a$	$2.5\,\beta\,\gamma\,K_a$
In-plane bending	$1.5\,\beta\,\gamma$	$1.5\,\beta\,\gamma$	$1.5\,\beta\,\gamma$
Out-of-plane bending	$1.5\,\beta\,\gamma$	$1.5\,\beta\,\gamma\,/\,\sqrt{Q_\beta}$	$1.5\,\beta\,\gamma$

Notes:

1) * Limited by brace capacity. Q_u may be taken as a value which enables Equation 6.4.1 (without Q_f) to default to brace capacity.

2) $Q_\beta = 1.0$ for $\beta \leq 0.6$

$= 0.3/\beta(1 - 0.833\beta)$ for $\beta > 0.6$

3) $K_a = (1 + 1/\sin\theta)\,/\,2$

4) See Figure 6.1.1 for definition of β and γ.

Table 6.4.1: Q_u factor for grouted joints

III 6.4.4.2 Safety factors

The safety factors given below should be applied to the strengths P, M_i and M_o given above in the determination of permissible strengths P_c, M_{ci} and M_{co}.

Conditions	Safety Factor
Extreme	$1.28\,\Gamma$
Operating	$1.70\,\Gamma$

Γ is a partial factor to account for the level of confidence required in the solution. This factor will vary on a case-by-case basis, and reflects the adequacy of the above procedure to cover the case under consideration, and the availability of any back-up additional data. Specialist advice should be sought in this respect.

III 6.4.4.3 Unity check

For unidirectional or combined brace loads the following requirement should be satisfied:

$$\left|\frac{P_d}{P_c}\right| + \left(\frac{M_{di}}{M_{ci}}\right)^2 + \left|\frac{M_{do}}{M_{co}}\right| \leq 1.0 \qquad \text{....6.4.3}$$

In addition, the average shear stress at any transverse section in the chord, ignoring the presence of the grout, under the calculated applied loads should not exceed $0.53F_y$ for extreme conditions or $0.4F_y$ for operating conditions.

III 6.4.5 Double-Skin Joints

The procedures described above are applicable to joints with complete grout-filling of the chord member. For double-skin joints, (which comprise an inset pile within the chord and the annulus between the pile and chord filled with grout), the same procedures apply. In addition, double-skin joints may be prone to ovalisation failure. Therefore, an additional check against ovalisation failure is necessary. This may be conducted by following the procedures for as-welded joints defined in the HSE Guidance Notes, with chord wall thickness T replaced by an equivalent thickness T_e defined as follows:

$$T_e = (T^2 + T_p^2)^{½} \qquad \text{....6.4.4}$$

where T_p = wall thickness of inset pile.

III 6.4.6 Stiff Joints

Special consideration should be given to 'stiff' joints (low γ, high β), and the partial factor Γ in Section III 6.4.4.2 should be adjusted to eliminate the potential for unconservative predictions. Specialist advice should be sought in this respect.

III 6.5 SCFs AND FATIGUE

III 6.5.1 Basic SCF Formulations

It is recommended that the Efthymiou equations are adopted; these formulations are contained in the Appendix.

III 6.5.2 Calculation of T_e

It is recommended that an equivalent chord thickness is calculated to take account of the presence of grout in the chord. The recommended formulation for T_e is as follows:

$$T_e = (5D + 134T)/144 \qquad \text{.... 6.5.1}$$

There is sufficient evidence to suggest that T_e is not fully effective for IPB loaded joints (SCFs at crown positions). To account for this, it is recommended that the following approach is adopted:

- Calculated T_e using Equation 6.5.1
- Modify γ ratios in Efthymiou formulation in the following manner:

$$\gamma_e = \frac{D}{k\,T_e}$$

where γ_e = Equivalent γ for grouted and double-skin joints

D = Chord outside diameter

T_e = Equivalent chord thickness

k = 2.0 for axial and OPB load cases

= $2(T/T_e)^{0.9}$ for IPB loads.

III 6.5.3 Use of T_e for High β, Low γ Joints

Although adequately substantiated evidence for the effects of grouting in high β, low γ joints is not available, it is understood that these 'stiff' joints benefit little from chord grout-filling, for reasons noted in Part IV (Section IV 6.5.4.2). It is recommended that the benefits for joints with $\beta = 1.0$ and $\gamma \leq 12.0$ be given special consideration, and specialist advice should be sought.

III 6.5.4 Effects of α Ratio

The Efthymiou equations for chord and brace crown SCFs for axially loaded T/Y joints contain an α term to account for chord bending stresses. It is recommended that the α ratio is multiplied by a factor of 0.88 to account for reduced, bending-induced, stresses for grouted conditions. Alternatively, the factor can be determined from the following expression:

$$1 - (\gamma - 1)^4/(18\gamma^4 - 17(\gamma - 1)^4) \qquad \text{.... 6.5.2}$$

III 6.5.5 Safety Factors

The concept of Assessment SCF is introduced, defined as follows:

$$ASCF = \frac{SCF}{\Gamma} \qquad \text{.... 6.5.3}$$

where ASCF = Assessment SCF

SCF = SCF calculated on the basis of the above procedures

Γ = Partial factor selected to give the appropriate level of confidence.

Γ should be considered on a case-by-case basis, and selected to give a level of confidence in solution appropriate to the case in question. Specialist advice should be sought in this respect.

III 6.5.6 Fatigue S-N Curve

The fatigue curves for as-welded tubular joints given in the HSE Guidance Notes may be used. The input hot spot stress range may be calculated on the basis of nominal stress ranges amplified by an appropriate SCF calculated on the basis of the above procedures.

REFERENCES

6.1 American Petroleum Institute. 'Recommended practice for the planning, designing and constructing fixed offshore platforms'. API RP2A, Nineteenth Edition, August 1991.

6.2 Health and Safety Executive. 'Offshore Installations: Guidance on design and construction'. Fourth Edition, HMSO, London 1990.

III 7 BOLTING

III 7.1 GENERAL

Bolting, as a strengthening, modification or repair (SMR) technique in its own right, is only rarely used subsea and thus is mainly confined to topside applications. Only two published documents make reference to offshore repairs carried out by bolting. One describes the use of bolted patch plates to repair a damaged bottle section of rather complex geometry[7.1]; and the other the repair of a large severed bracing member under difficult environmental conditions which left only a 20 minute working period at the turn of each tide[7.2]. Both of these applications had unusual circumstances which led to the decision to use a bolted repair. However, subsea bolting may present an acceptable solution where welding is ruled out (eg. due to high carbon equivalent values or too short a working period) and clamping is impractical (due to geometrical considerations). For topsides applications, bolting is entirely suitable for:-

- forming connections between rolled sections
- adding stiffening plates
- conducting SMR in hot working areas.

Bolts and studbolts form an essential part of clamp technology and their long term integrity is of prime importance to the proper performance of the clamps.

III 7.2 DESIGN

Design considerations for bolted SMR schemes include:

- materials and specifications
- static strength
- fatigue strength

In addition to the above, practical aspects of installation and subsequent inspection and maintenance should be borne in mind.

III 7.2.1 <u>Material and Specifications</u>

The fastener material, manufacturing route and any protective coatings, must be considered in the design. Material selection depends upon several factors including the design strength required, corrosion resistance and accessibility for future inspection. The greatest care is required in selecting materials for the splash zone due to the particularly aggressive nature of this environment.

Low-alloy steels, stainless steels (including the austenitic-ferritic duplex types) and nickel alloys may be considered in order of increasing costs. The most popular materials for subsea work are L7 and B7 (1¼% chromium - molybdenum steels) and have proved themselves with a substantial track record. Macalloy bar used to be specified in clamps but has fallen out of favour following a number of hydrogen embrittlement failures. Some of the higher alloyed steels and the nickel alloys could be subject to attack by crevice corrosion.

Low-alloy steel studbolts should be protected by coatings, especially when subject to immersion. Metal plating should be carried out carefully to avoid hydrogen pick-up which increases the propensity to hydrogen embrittlement. Four coatings are commonly specified[7.3]:-

- Cadmium plating:

  This only serves as an installation aid.

- Galvanising:

  The zinc thickness is variable, being deepest at the roots of the thread and could reduce clearances to an unacceptable level.

- Chromium plating:

  The effectiveness of chromium plating is not justified by its expense.

- Coating with PTFE:

  Compared to other treatments, PTFE has extremely high resistance to attack from seawater. It has the further benefit of having a low coefficient of friction which allows nuts to be easily run down the studbolt - advantageous from a diver's point of view.

Studbolts used to be threaded only at each end. However, several studbolts failed where the first thread met the plain bar and now studbolts are threaded along their full length. Threading is achieved either by rolling or screw cutting. Rolled threads are to be preferred as they have much better fatigue resistance and distribute loads more evenly to the nut, because of the better surface finish and dimensional accuracy[7.3]. Note, however, that the work hardening introduced by rolling may be undesirable in some materials (eg. Monel alloy K-500) necessitating a subsequent heat treatment. This has to be done properly to avoid deleterious metallurgical phases, and sometimes cut threads may be preferred.

III 7.2.2 Static Strength

Threaded fasteners are essentially used in four different ways:-

- as elements to transfer shear forces between plies
- as elements to resist tensile forces (preloaded bolts)
- as elements which are highly tensioned such that the connected plies resist shear forces by the frictional resistance induced at the interfaces (ie. high strength friction grip (HSFG) bolted connections)
- as elements for applying stress such as used in clamps or jacking studbolts.

Much experience with the use of ordinary and HSFG bolts has been gained onshore and there are a wide variety of design codes which address these fasteners, eg. see References 7.4 and 7.5. However, there is a lack of clear guidelines with respect to studbolts. Present practice for clamp studbolts in B7 or L7 materials is to limit the nominal stress in the studbolts to 0.7 σ_y for short term loads or 0.6 σ_y for long term loads. These limits are imposed to avoid permanent set and bolt relaxation. The tensile stress area of a studbolt is dependent on nominal size and thread details. Slightly different formulae also apply on whether the thread form is of metric[7.6] or imperial[7.7] series. Table 7.2.1 gives the net stress areas for a selected range of sizes and thread forms. Note that there are larger sizes, and other series (ie. pitch size) which may be beneficial if higher stress areas are required for the same nominal size.

Iso Metric Coarse series			Unified Inch 8-UN series	
Nominal thread diameter (mm)	Pitch of the thread (mm)	Nominal stress area (mm^2)	Nominal thread diameter (inch)	Nominal stress area ($inch^2$)
8	1.25	36.6	1	0.606
10	1.5	58.0	1⅛	0.790
12	1.75	84.3	1¼	1.000
(14)	2	115	1⅜	1.233
16	2	157	1½	1.492
(18)	2.5	192	1⅝	1.78
20	2.5	245	1¾	2.08
(22)	2.5	303	1⅞	2.41
24	3	353	2	2.77
(27)	3	459	2¼	3.56
30	3.5	561	2½	4.44
(33)	3.5	694	2¾	5.43
36	4	817	3	6.51
(39)	4	976	3¼	7.69
42	4.5	1121	3½	8.96
(45)	4.5	1306	3¾	10.34
48	5	1473	4	11.81
(52)	5	1758		
56	5.5	2030		
(60)	5.5	2362		
64	6	2676		
(68)	6	3055		

Note: Sizes shown in brackets are non-preferred.

Table 7.2.1: Nominal stress area of studbolts

III 7.2.3 Fatigue Strength

There are few data on the fatigue strength of studbolts but, based on the assessment of data presented in Part IV, Section IV 4.7.4, the following S-N curves are recommended for B7 and L7 studbolts under axial (or bending) loads either in air or immersed if suitably protected:-

i) Metric series studbolts:-

$$\log N = \log 0.36 \times 10^{12} - 3 \log S \text{ (units N/mm}^2\text{)} \quad .... 7.2.1$$

ii) Unified inch (8-UN) series:-

$$\log N = \log 0.109 \times 10^{12} - 3 \log S \text{ (units ksi)} \quad .... 7.2.2$$

In the above, N is the number of load cycles and S is the stress range. If studbolts are to be used unprotected then the life N derived above should be halved.

III 7.3 PRACTICAL CONSIDERATIONS

There are two main methods for inducing tension in bolts and studbolts, namely torquing and jacking. Hydraulic wrenches are available to turn the nut for the former method but it has to be calibrated, and calibrated underwater if the bolts/studbolt are to be torqued subsea. Torquing may be selected where low tensions are required, few studbolts are involved, space is limited for hydraulic tensioning jacks or where the studbolts are so short that transfer losses using the jacking method would be unacceptable. The advantages in the jacking method are[7.3]:-

- all studbolts may be tensioned simultaneously
- less effort required from the diver
- bolt load is independent of thread lubrication and condition
- large tensions can be applied
- no torque induced stresses exist.

Long studbolts are preferred to short ones in fatigue situations as they have a smaller stress range due to their lower stiffness. They will have less preload loss in grouted clamps when the grout shrinks or creeps. Long studbolts can also absorb the effects of non parallelity of the seatings at either end of the studbolt, either due to initial imperfections or subsequent deformation. Spherical washers can take out initial non parallelity but it is uncertain whether they are effective for load induced deformations.

For short studbolts (say $L/D < 10$), consideration should be given to using hydraulic tensioners which sit on top of the spherical washer because the following advantages accrue:-

- The tensioning force is always axial to the studbolt, thus eliminating any bending stresses.

- The spherical washers are compressed against their seating faces during tensioning, thus eliminating some of the transfer losses.

Where underwater drilling of bolt holes is required, due attention should be given to removing all swarf between the faying surfaces of plies, particularly if the connection is to utilise friction grip principles.

III 7.4 INSPECTION AND MAINTENANCE

The main inspection requirements are to ensure that no studbolt is missing and that the studbolt tension is within the design range. There are load-indicating devices on the market (eg. the Rotabolt system) which require studbolt modifications before installation. These are worthy of consideration. Alternatively, the studbolt tension can be checked by a re-tensioning operation using hydraulic jacks. This task is made easier by installing grease-packed thread protector caps when the studbolts are first installed. Recently, acoustic methods have become available; these require the studbolts to be individually calibrated.

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7.1 Thompson JM and White AR. 'The Heather platform leg repair'. Paper OTC 3529, 11th Offshore Technology Conference, Houston, Texas, April 30 - May 3, 1979.

7.2 Hull TL. 'The replacement of a major submerged structural brace on MGS Platform A, Cook Inlet, Alaska'. Paper OTC 2165, 7th Offshore Technology Conference, Houston, Texas, May 5-8, 1975.

7.3 Scott GA. 'Studbolting experience in the underwater environment of the North Sea'. Paper OTC 5050, 17th Offshore Technology Conference, Houston, Texas, May 6-9, 1985.

7.4 American Institute of Steel Construction. 'Manual of Steel Construction'.

7.5 British Standards Institution. 'BS 5950: Structural use of steelwork in buildings. Part 1: Code of Practice for design in simple and continuous construction: Hot rolled sections'. BSI, 1990.

7.6 British Standards Institution. 'BS 3643: Specification for ISO metric screw threads. Part 1: Principles and basic data'. BSI, 1981.

7.7 The American Society of Mechanical Engineers. 'ASME B1.1: Unified inch screw threads (UN and UNR thread form)'. ASME, 1989.

APPENDIX A

SELECTION OF SCF EQUATIONS FOR AS-WELDED TUBULAR JOINTS

A.1 INTRODUCTION

During the course of preparation of this document, it has become necessary to select a set of SCF formulations for as-welded tubular joints. This Appendix describes the reasons leading to the selection of Efthymiou equations which have been adopted throughout this document. Nevertheless, other formulations, eg. those of Wordsworth, may be acceptable alternatives.

A.2 AVAILABLE FORMULATIONS

There are several sets of equations available for the calculation of SCFs. These have mostly been derived by theoretical (eg. finite element) analyses or by small scale acrylic model tests on joints of simple configurations eg. T, Y, DT, X and K. The following sets of SCF formulae are presently available:

- Kuang/Potvin et al[A.1] These formulae cover simple joints (T, Y, K and YT) for various types of load, although they do not cover DT/X joints or SCFs in K/YT joints subjected to out-of-plane moment loads. The formulae have been developed from finite element analyses.

- Wordsworth[A.2] These formulae cover all simple joint types and have been developed from small scale acrylic model tests.

- Gibstein[A.3] The formulae cover T joints only, and are therefore limited in scope in comparison with the extensive number of other joint types in service (eg. Y, DT, X, K and YT joints). The formulae have been developed from finite element analyses.

- UEG[A.4] The formulae recommended in the UEG design guide relate to the Wordsworth set of equations which have been suitably modified for high β and γ ratio joints. These modifications, plus extension of the validity ranges in some cases, were the result of reliability studies which indicated that SCFs were underestimated in these regions.

- Buitrago et al[A.5] This paper presents parametric equations in terms of influence coefficients for T/Y joints and K/YT joints. The formulae are a result of extensive finite element analyses.

- Efthymiou[A.6] This paper presents the most recent set of equations developed through finite element analyses. The formulae cover all simple joint types, overlapping K joints and specification of influence functions for extension to cover multiplanar joints.

- Marshall[A.7, A.8] Marshall adopted and modified the Kellogg[A.9] formula (based on an analogy of the behaviour of a circular cylinder subjected to uniform circumferential loads) to develop relationships for simple joints.

- Gibstein[A.10] The formulae cover $\beta=1.0$ K joints which can be non-overlapping, overlapping, or overlapped and stiffened. The validity ranges place a severe restriction on the use of these equations; they are only valid for a specific brace included angle ($\Theta=55°$), a specific gap factor for non-overlapping ($\zeta=0.20$) and overlapping ($\zeta=-0.26$) joints and a specific stiffener arrangement and size.

- API RP2A[A.11] Tabulated SCF equations are provided, based on the modified Kellogg formula[A.7, A.8]. A reduction factor of 0.625 for brace side SCFs is introduced, based on the formula suggested in Reference A.7. The equations cover T/Y, DT/X and K/YT joints for axial, IPB and OPB loads, for both the brace and chord side of the weld.

- Marshall Correction Factor[A.12] With respect to the Kuang/Potvin brace SCF formulations, Marshall recommends an expression to account for potential overestimation of brace SCFs.

A.3 ASSESSMENT AND RECOMMENDATIONS

A number of reliability-type assessments of the accuracy of the various formulations has been carried out to date by various investigators. Notably, the findings from the following assessments require consideration:-

- UR33[A.4]
- Van Delft et al[A.13]
- Papers OTC 5306 and OTC 5662[A.14, A.15]
- Sharp et al[A.16]

A number of observations can be made on examination of the above assessments:-

- The assessments have all been carried out with respect to available steel model test data, and relate essentially to simple joints.

- It is clear that conclusions drawn from a reliability-type assessment are significantly influenced by the investigators' perception of what constitutes a valid datapoint. The databases adopted by the six different assessments above can be considered to be diverse, although it is important to note that specific sets of data accessible by one investigator may not be available to another.

- Despite the above reservation, several common trends have been identified from the assessments, and are noted below.

- Kuang/Potvin et al The assessments indicate a spread of predictions (in respect of ratios of test result to prediction) for test results which fall within the validity ranges. A significant proportion of the available data fall outside the noted ranges of applicability. The Marshall correction factor significantly overcompensates for potential over-prediction of SCFs on the brace side of the weld.

- Gibstein (T joints only) The majority of the test results fall outside the validity range.

- Wordsworth Tightly banded (in the sense of ratios of result to prediction plotted against number of results). The majority of the results fall within the validity ranges of geometry, configuration and load-case.

- UEG Tightly banded, with a shift towards predicting higher (upper bound) SCFs when compared with Wordsworth. This is not unexpected as the UEG formulations in essence encompass the Wordsworth equations with additional Q_β and Q_γ terms to compensate for any potential under-prediction for large β and large γ ratio joints. As these two terms are applied for all joint types and load-cases, significantly conservative estimates of SCFs are possible.

- Buitrago The majority of the test results fall outside the validity ranges, with respect to the β, γ and α ratios. (This observation is indicative of the extremes of joint geometries in the database, as the Buitrago validity ranges were established on the basis of the most frequently occurring geometries in practice.)

- Efthymiou Tightly banded. The majority of test results fall within the validity ranges.

The assessments reported in the above five investigations reveal, collectively, that the formulations derived by Efthymiou and Wordsworth demonstrate the best relative correlation with test data, when compared with other approaches and within the confines of the data presently available.

Whilst both sets of equations are expected to provide similar SCF values, it can also be observed from the assessments that:-

- The Wordsworth formulae can under-predict SCFs in certain instances due to the manner in which the acrylic models were prepared, particularly for X joints with high β ratios where saddle cut-backs were not specified in the models. If this under-prediction is rectified, the Wordsworth formulae indicate a positive skew normal distribution when all ratios of test result to prediction are considered together. This implies that the SCF prediction err on the conservative side of actual measured values.

- The Efthymiou equations essentially predict mean SCF values.

During the course of interviews undertaken by MSL Engineering with the offshore industry as part of this present project, it became very clear that the Efthymiou equations represent the most popular and preferred set by the offshore industry. On this basis, and on the basis of the above discussions, the Efthymiou equations have been selected for obtaining estimates of SCFs for as-welded joints. The Efthymiou equations are reproduced at the end of this Appendix.

REFERENCES

A.1 Kuang JG, Potvin A et al. 'Stress concentrations in tubular joints'. Paper OTC 2205 of Offshore Technology Conference, Texas, 1975. (Modifications in SPE Journal, Pages 287-299, August 1977).

A.2 Wordsworth AC. 'Stress concentration factors at K and KT tubular joints'. Fatigue in offshore structural steels. Inst of Civil Engineers, Westminster, London, February 1981.

A.3 Gibstein MB. 'Parametric stress analysis of T joints'. Paper 26, European Offshore Steels Research Seminar, Cambridge, November 1978.

A.4 Underwater Engineering Group. 'Design of tubular joints for offshore structures'. UEG publication, UR33, 1985.

A.5 Buitrago J et al. 'Combined hot-spot stress procedure for tubular joints'. Paper OTC 4775 of Offshore Technology Conference, Texas, 1984.

A.6 Efthymiou M. 'Development of SCF formulae and generalised influence functions for use in fatigue analysis'. OTJ '88, Surrey, October 1988, plus updates.

A.7 Kinra RK and Marshall PW. 'Fatigue analysis of the Cognac platform'. SPE 8600, J Petroleum Technology, March 1980.

A.8 Marshall PW and Luyties WH. 'Allowable stresses for fatigue design'. Boss '82 Conference held at Massachusetts Institute of Technology, Cambridge, Mass, August 1982.

A.9 Kellogg MW. 'Design of piping systems'. Second Edition, Wiley, 1956.

A.10 Gibstein MB. 'Stress concentration in tubular K-joints with diameter ratio equal to one'. Paper TS10 of proceedings of the 3rd International ECSC Offshore Conference on Steel in Marine Structures (SIMS '87), Delft, Netherlands, 1987.

A.11 American Petroleum Institute. 'Recommended practice for planning, designing and constructing fixed offshore platforms'. API RP2A, Nineteenth Edition, August 1991.

A.12 Marshall PW. 'A review of stress concentration factors in tubular joints'. Report CE-32, Shell Oil Company, Houston, Texas.

A.13 Van Delft DRV et al. 'The results of the European fatigue tests on welded tubular joints compared with SCF formulas and design lines'. Paper TS24 of Proceedings of the 3rd International ECSC Offshore Conference on Steel in Marine Structures (SIMS 87), Delft, Netherlands, 1987.

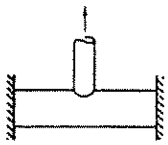
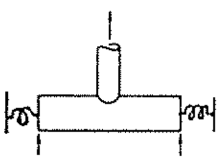
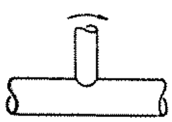
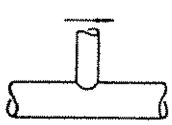
A.14 Lalani M et al. 'Improved fatigue life estimation of tubular joints'. Paper OTC 5306 of Offshore Technology Conference, Texas, 1986.

A.15 Tolloczko JA and Lalani M. 'The implication of new data on the fatigue life assessment of tubular joints'. Paper OTC 5662 of Offshore Technology Conference, Texas, 1988.

A. 16 Sharp JV et al. 'Implications of new data for the fatigue performance of tubular joints'. OTJ '88, Surrey, October 1988.

Efthymiou Equations, plus updates

Table I - Parametric equations for SCFs in T/Y-joints

Load type and fixity conditions	SCF equation	Eqn. No.	short chord correction
Axial load - chord ends fixed 	**chord saddle:** $\gamma\tau^{1.1}\,[1.11-3(\beta-0.52)^2]\,\sin^{1.6}\theta$	T1	F1
	chord crown: $\gamma^{0.2}\tau[2.65+5(\beta-0.65)^2] + \tau\beta(0.25\alpha-3)\sin\theta$	T2	None
	brace saddle: $1.3 + \gamma\tau^{0.52}\alpha^{0.1}[0.187-1.25\beta^{1.1}(\beta-0.96)]\,\sin^{(2.7-0.01\alpha)}\theta$	T3	F1
	brace crown: $3 + \gamma^{1.2}[0.12\exp(-4\beta) + 0.011\beta^2-0.045]+\beta\tau(0.1\alpha-1.2)$	T4	None
Axial load - General fixity conditions 	**chord saddle:** $[T1]+C_1(0.8\alpha-6)\tau\beta^2(1-\beta^2)^{0.5}\sin^2 2\theta$	T5	F2
	chord crown: $\gamma^{0.2}\tau[2.65+5(\beta-0.65)^2] + \tau\beta(C_2\cdot\alpha-3)\sin\theta$	T6	None
	brace saddle: eqn. T3		F2
	brace crown: $3 + \gamma^{1.2}[0.12\exp(-4\beta) + 0.011\beta^2-0.045] + \beta\tau(C_3\cdot\alpha-1.2)$	T7	None
In-plane bending 	**chord crown:** $1.45\,\beta\tau^{0.85}\gamma^{(1-0.68\beta)}\sin^{0.7}\theta$	T8	None
	brace crown: $1 + 0.65\,\beta\tau^{0.4}\gamma^{(1.09-0.77\beta)}\sin^{(0.06\gamma-1.16)}\theta$	T9	None
Out-of-plane bending 	**chord saddle:** $\gamma\tau\beta(1.7-1.05\beta^3)\,\sin^{1.6}\theta$	T10	F3
	brace saddle: $\tau^{-0.54}\gamma^{-0.05}(0.99 - 0.47\beta + 0.08\beta^4) * [T10]$	T11	F3

Short chord correction factors ($\alpha < 12$)	Chord-end fixity parameter
$F1 = 1 - (0.83\beta-0.56\beta^2-0.02)\gamma^{0.23}\exp[-0.21\,\gamma^{-1.16}\alpha^{2.5}]$	$C_1 = 2\,(C-0.5)$
$F2 = 1 - (1.43\beta-0.97\beta^2-0.03)\gamma^{0.04}\exp[-0.71\,\gamma^{-1.38}\alpha^{2.5}]$	$C_2 = C/2$
$F3 = 1 - 0.55\beta^{1.8}\gamma^{0.16}\exp[-0.49\,\gamma^{-0.89}\alpha^{1.8}]$	$C_3 = C/5$
Where $\exp[x] = e^x$	C = chord-end fixity parameter
	$0.5 \leq C \leq 1.0$, Typically $C = 0.7$

Table 2 - Equations for SCFs in X-joints

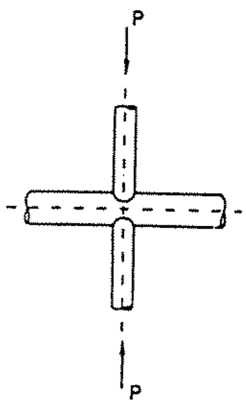
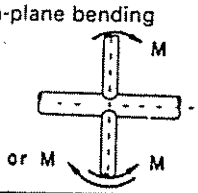
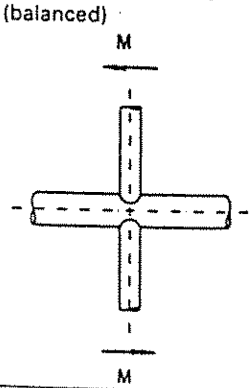
Load type	SCF equation	Eqn. No.
Axial load (balanced) 	**Chord saddle:** $3.87\ \gamma\tau\beta\ (1.10-\beta^{1.8})\ (\sin\theta)^{1.7}$	X1
	Chord crown: $\gamma^{0.2}\tau\ [2.65 + 5\ (\beta-0.65)^2] - 3\tau\beta \sin\theta$	X2
	Brace saddle: $1 + 1.9\gamma\tau^{0.5}\ \beta^{0.9}\ (1.09-\beta^{1.7})\ \sin^{2.5}\theta$	X3
	Brace crown: $3 + \gamma^{1.2}\ [0.12\ \exp(-4\beta) + 0.011\beta^2-0.045]$	X4
	In joints with short chords ($\alpha < 12$) the saddle SCFs can be reduced by the factor F1 (fixed chord ends) or F2 (pinned chord ends) where $F1 = 1 - (0.83\beta-0.56\beta^2-0.02)\gamma^{0.23}\exp[-0.21\ \gamma^{-1.16}\alpha^{2.5}]$ $F2 = 1 - (1.43\beta-0.97\beta^2-0.03)\gamma^{0.04}\exp[-0.71\ \gamma^{-1.38}\alpha^{2.5}]$	
In-plane bending 	**Chord crown:** eqn. T8 **Brace crown:** eqn. T9	
Out-of-plane bending (balanced) 	**Chord saddle:** $\gamma\tau\beta(1.56-1.34\beta^4)\ (\sin\theta)^{1.6}$	X5
	Brace saddle: $\tau^{-0.54}\gamma^{-0.05}(0.99-0.47\beta+0.08\beta^4)\cdot[X5]$	X6
	In joints with short chords ($\alpha < 12$) eqns. X5 and X6 can be reduced by the factor F3 where $F3 = 1 - 0.55\beta^{1.8}\gamma^{0.16}\exp[-0.49\ \gamma^{-0.89}\alpha^{1.8}]$	

Table 2 - Equations for SCFs in X-joints (cont'd)

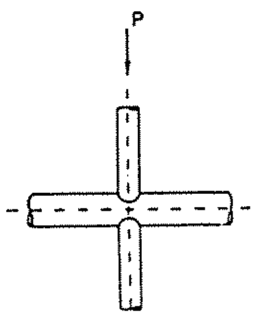
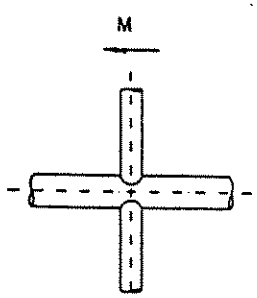
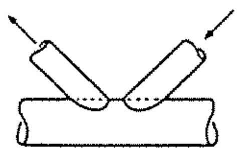
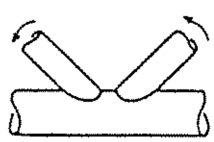
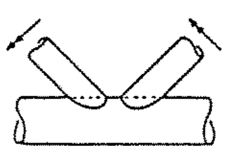
Load type	SCF equation	Eqn. No.
Axial load on one brace only 	**Chord saddle:** [T5] * $[1-0.26\beta^3]$ **Chord crown:** [T6] **Brace saddle:** [T3] * $[1-0.26\beta^3]$ **Brace crown:** [T7] In joints with short chords ($\alpha < 12$) the saddle SCFs can be reduced by the factor F1 (fixed chord ends) or F2 (pinned chord ends) where $F1 = 1 - (0.83\beta - 0.56\beta^2 - 0.02)\gamma^{0.23}\exp[-0.21\ \gamma^{-1.16}\alpha^{2.5}]$ $F2 = 1 - (1.43\beta - 0.97\beta^2 - 0.03)\gamma^{0.04}\exp[-0.71\ \gamma^{-1.38}\alpha^{2.5}]$	X7 X8
Out-of-plane bending on one brace only 	**Chord saddle:** [T10] **Brace saddle:** [T11] In joints with short chords ($\alpha < 12$) eqns. T10 and T11 can be reduced by the factor F3 where $F3 = 1 - 0.55\beta^{1.8}\gamma^{0.16}\exp[-0.49\ \gamma^{-0.89}\alpha^{1.8}]$	

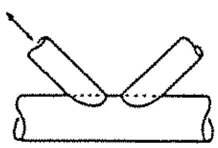
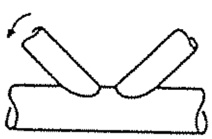
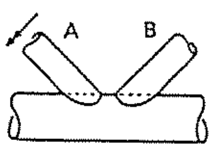
Table 3 - Equations for SCFs in gap/overlap K-joints

Load type	SCF equation	Eqn. No.	short chord correction
Balanced axial load 	**Chord SCF:** $\tau^{0.9}\gamma^{0.5}(0.67-\beta^2+1.16\beta)\sin\theta\left[\frac{\sin\theta_{max}}{\sin\theta_{min}}\right]^{0.30}\left[\frac{\beta_{max}}{\beta_{min}}\right]^{0.30}$ * $[1.64 + 0.29\,\beta^{-0.38}\,\mathrm{ATAN}\,(8\zeta)]$	K1	None
	Brace SCF: $1 + [K1]\,(1.97 - 1.57\,\beta^{0.25})\,\tau^{-0.14}\,\sin^{0.7}\theta\, +$ $C.\beta^{1.5}\gamma^{0.5}\tau^{-1.22}\sin^{1.8}(\theta_{max}+\theta_{min})$ * $[0.131 - 0.084\,\mathrm{ATAN}\,(14\zeta + 4.2\beta)]$	K2	None
	where C = 0 for gap joints C = 1 for the through brace C = 0.5 for the overlapping brace Note that τ, β, θ and the nominal stress relate to the brace under consideration. ATAN is arctangent evaluated in radians		
Unbalanced IPB 	**Chord crown SCF:** eqn. T8 (For overlaps exceeding 30% of contact length use 1.2 * T8) **Gap joint-brace crown SCF:** eqn. T9 **Overlap joint-brace crown SCF:** [T9] * $(0.9 + 0.4\,\beta)$	K3	
Unbalanced OPB 	**Chord saddle SCF adjacent to brace A:** $[T10]_A\,[1 - 0.08\,(\beta_B\gamma)^{0.5}\exp(-0.8x)]\, +$ $[T10]_B\,[1 - 0.08\,(\beta_A\gamma)^{0.5}\exp(-0.8x)]\,[2.05\beta_{max}^{0.5}\exp(-1.3x)]$ where $x = 1 + \frac{\zeta\sin\theta_A}{\beta_A}$	K4	F4
	Brace A saddle SCF: $\tau^{-0.54}\gamma^{-0.05}\,(0.99 - 0.47\beta + 0.08\,\beta^4)\cdot[K4]$	K5	F4

$F4 = 1 - 1.07\,\beta^{1.88}\exp\,[-0.16\,\gamma^{-1.06}\,\alpha^{2.4}]$

$[T10]_A$ is the chord SCF adjacent to brace A as estimated from eq. T10
Note that the designation of braces A and B is not geometry dependent. It is nominated by the user

Table 3 - Equations for SCFs in gap/overlap K-joints (cont'd)

Load type	SCF equation	Eqn. No.	short chord correction
Axial load on one brace only 	**chord saddle:** [T5]		F1
	Chord crown: [T6]		—
	Brace saddle: [T3]		F1
	Brace crown: [T7]		—
	Note that all geometric parameters and the resulting SCF's relate to the loaded brace		
IPB on one brace only 	**Chord crown:** eqn. T8		—
	Brace crown: eqn. T9		—
	Note that all geometric parameters and the resulting SCF's relate to the loaded brace		
OPB on one brace only 	**Chord saddle:**		
	$[T10]_A\ [1 - 0.08\ (\beta_B\gamma)^{0.5}\exp(-0.8x)]$	K6	F3
	where $x = 1 + \dfrac{\zeta \sin\theta_A}{\beta_A}$		
	Brace saddle:		
	$\tau^{-0.54}\gamma^{-0.05}(0.99 - 0.47\beta + 0.08\ \beta^4)\cdot[K6]$	K7	F3

Short chord correction factors

$F1 = 1 - (0.83\beta - 0.56\beta^2 - 0.02)\gamma^{0.23}\exp[-0.21\ \gamma^{-1.16}\alpha^{2.5}]$

$F2 = 1 - (1.43\beta - 0.97\beta^2 - 0.03)\gamma^{0.04}\exp[-0.71\ \gamma^{-1.38}\alpha^{2.5}]$

$F3 = 1 - 0.55\beta^{1.8}\gamma^{0.16}\exp[-0.49\ \gamma^{-0.89}\alpha^{1.8}]$

Table 4 — Equations for SCFs in KT-joints

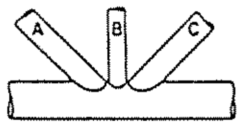
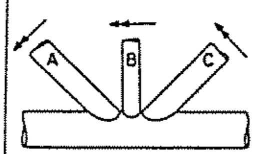
Load type	SCF equation	Eqn. No.
Balanced Axial load 	**Chord SCF**: Eqn. K1 **Brace SCF**: Eqn. K2 For the diagonal braces, A & C use $\zeta = \zeta_{AB} + \zeta_{BC} + \beta_B$ For the central brace, B use ζ = maximum of ζ_{AB}, ζ_{BC}	
In-plane bending	**Chord crown SCF**: Eqn. T8 **Brace Crown SCF**: Eqn. T9	
Unbalanced Out-of-plane bending 	**Chord saddle SCF adjacent to diagonal brace A:** $[T10]_A\,[1 - 0.08\,(\beta_B\gamma)^{0.5}\exp(-0.8x_{AB})] \cdot [1 - 0.08(\beta_C\gamma)^{0.5}\exp(-0.8x_{AC})]$ $+ \quad [T10]_B\,[1 - 0.08\,(\beta_A\gamma)^{0.5}\exp(-0.8x_{AB})] \cdot [2.05\beta_{max}^{0.5}\exp(-1.3x_{AB})]$ $+ \quad [T10]_C\,[1 - 0.08\,(\beta_A\gamma)^{0.5}\exp(-0.8x_{AC})] \cdot [2.05\beta_{max}^{0.5}\exp(-1.3x_{AC})]$ where $x_{AB} = 1 + \dfrac{\zeta_{AB}\sin\theta_A}{\beta_A}$ $x_{AC} = 1 + \dfrac{(\zeta_{AB} + \zeta_{BC} + \beta_B)\sin\theta_A}{\beta_A}$	KT1
	Chord saddle SCF adjacent to central brace B: $[T10]_B\,[1 - 0.08\,(\beta_A\gamma)^{0.5}\exp(-0.8x_{AB})]^{(\beta_A/\beta_B)^2} \cdot$ $[1 - 0.08\,(\beta_C\gamma)^{0.5}\exp(-0.8x_{BC})]^{(\beta_C/\beta_B)^2}$ $+ \quad [T10]_A\,[1 - 0.08\,(\beta_B\gamma)^{0.5}\exp(-0.8x_{AB})] \cdot [2.05\beta_{max}^{0.5}\exp(-1.3x_{AB})]$ $+ \quad [T10]_C\,[1 - 0.08\,(\beta_B\gamma)^{0.5}\exp(-0.8x_{BC})] \cdot [2.05\beta_{max}^{0.5}\exp(-1.3x_{BC})]$ where $x_{AB} = 1 + \dfrac{\zeta_{AB}\sin\theta_B}{\beta_B}$ $x_{BC} = 1 + \dfrac{\zeta_{BC}\sin\theta_B}{\beta_B}$	KT2
OPB brace SCFs	OPB brace SCFs are obtained directly from the adjacent chord SCFs using $\tau^{-0.54}\gamma^{-0.05}(0.99 - 0.47\beta + 0.08\beta^4) \cdot SCF_{chord}$ where SCF_{chord} = KT1 or KT2	

Table 4 — Equations for SCFs in KT-joints (cont'd)

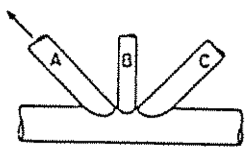
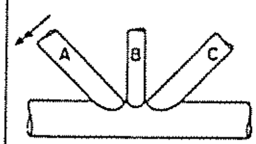
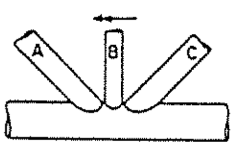
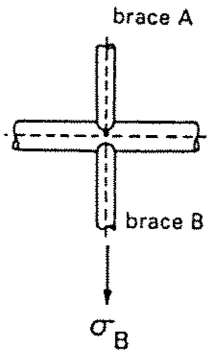
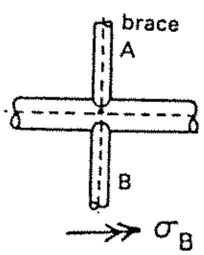
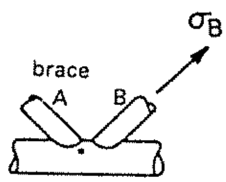
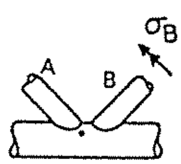
Load type	SCF equation	Eqn. No.
Axial load on one brace only  	**Chord saddle:** [T5] **Chord crown:** [T6] **Brace saddle:** [T3] **Brace crown:** [T7]	
Out-of-plane bending on one brace only   	**Chord SCF adjacent to diagonal brace A:** $[T10]_A\,[1-0.08\,(\beta_B\gamma)^{0.5}\exp(-0.8x_{AB})]\cdot[1-0.08(\beta_C\gamma)^{0.5}\exp(-0.8x_{AC})]$ where $x_{AB}=1+\dfrac{\zeta_{AB}\sin\theta_A}{\beta_A}$ $x_{AC}=1+\dfrac{(\zeta_{AB}+\zeta_{BC}+\beta_B)\sin\theta_A}{\beta_A}$ **Chord SCF adjacent to central brace B:** $[T10]_B\,[1-0.08\,(\beta_A\gamma)^{0.5}\exp(-0.8x_{AB})]^{(\beta_A/\beta_B)^2}\cdot$ $[1-0.08\,(\beta_C\gamma)^{0.5}\exp(-0.8x_{BC})]^{(\beta_C/\beta_B)^2}$ where $x_{AB}=1+\dfrac{\zeta_{AB}\sin\theta_B}{\beta_B}$ $x_{BC}=1+\dfrac{\zeta_{BC}\sin\theta_B}{\beta_B}$	KT 3 KT4
OPB brace SCFs	OPB brace SCFs are obtained directly from the adjacent chord SCFs using $\tau^{-0.54}\gamma^{-0.05}(0.99-0.47\beta+0.08\beta^4)\cdot SCF_{chord}$	

Table 5 - Influence functions for X-joints under AXIAL LOAD and OPB

Load type	Influence function for brace A	Eqn. No.
Axial load 	**Chord saddle:** $\sigma_B \frac{A_B \sin\theta_B}{A_A \sin\theta_A}\left[[X1]_A - [X7]_A\right]$	IX 1
	Chord Crown: $\sigma_B \frac{A_B \sin\theta_B}{A_A \sin\theta_A}\left[[X2]_A - [T6]_A\right]$	IX 2
	Brace saddle: $\sigma_B \frac{A_B \sin\theta_B}{A_A \sin\theta_A}\left[[X3]_A - [X8]_A\right]$	IX 3
	Brace Crown: $\sigma_B \frac{A_B \sin\theta_B}{A_A \sin\theta_A}\left[[X4]_A - [T7]_A\right]$ Where A_A = cross section area of brace A A_B = cross section area of brace B	IX 4
OPB 	**Chord saddle:** $\sigma_B \frac{Z_B \sin\theta_B}{Z_A \sin\theta_A}\left[[X5]_A - [T10]_A\right]$	IX 5
	Brace saddle: $\sigma_B \frac{Z_B \sin\theta_B}{Z_A \sin\theta_A}\left[[X6]_A - [T11]_A\right]$ where Z_A = section modulus of brace A Z_B = section modulus of brace B	IX 6

Note: In the above expressions the influence function is only the geometric part, i.e. without the nominal stress, σ_B.
The total expression gives a hot spot stress contribution.

Table 6 - Influence functions for K-joints under AXIAL LOAD and OPB

Load type	Influence function for brace A	Eqn. No.
Axial load – 	**Chord saddle:** $\sigma_B \dfrac{A_B \sin \theta_B}{A_A \sin \theta_A} \left[[T5]_A - [K1]_A \right]$	IK 1
	Chord Crown: $\sigma_B \dfrac{A_B \sin \theta_B}{A_A \sin \theta_A} \left[[T6]_A - [K1]_A \right]$	IK 2
	Brace saddle: $\sigma_B \dfrac{A_B \sin \theta_B}{A_A \sin \theta_A} \left[[T3]_A - [K2]_A \right]$	IK 3
	Brace Crown: $\sigma_B \dfrac{A_B \sin \theta_B}{A_A \sin \theta_A} \left[[T7]_A - [K2]_A \right]$ where A_A = cross section area of brace A	IK 4
OPB 	**Chord saddle:** $\sigma_B \left[[K4]_A - [K6]_A \right]$	IK 5
	Brace saddle: $\sigma_B \left[[K5]_A - [K7]_A \right]$	IK 6

Note: In the above expressions the influence function is only the geometric part, i.e. without the nominal stress, σ_B.
The total expression gives a hot spot stress contribution.

Table 7 - Influence functions for KT-joints under AXIAL LOAD

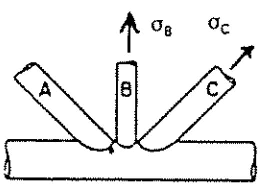
Load	Influence function for brace A	Eqn. No.
	Chord saddle: $\sigma_B \frac{A_B \sin\theta_B}{A_A \sin\theta_A}\left[[T5]_A - [K1]_{AB}\right] + \sigma_C \frac{A_C \sin\theta_C}{A_A \sin\theta_A}\left[[T5]_A - [K1]_{AC}\right]$	IKT 1
	Chord crown: $\sigma_B \frac{A_B \sin\theta_B}{A_A \sin\theta_A}\left[[T6]_A - [K1]_{AB}\right] + \sigma_C \frac{A_C \sin\theta_C}{A_A \sin\theta_A}\left[[T6]_A - [K1]_{AC}\right]$	IKT 2
	**Brace saddle:** $\sigma_B \frac{A_B \sin\theta_B}{A_A \sin\theta_A}\left[[T3]_A - [K2]_{AB}\right] + \sigma_C \frac{A_C \sin\theta_C}{A_A \sin\theta_A}\left[[T3]_A - [K2]_{AC}\right]$	IKT 3
	Brace crown: $\sigma_B \frac{A_B \sin\theta_B}{A_A \sin\theta_A}\left[[T7]_A - [K2]_{AB}\right] + \sigma_C \frac{A_C \sin\theta_C}{A_A \sin\theta_A}\left[[T7]_A - [K2]_{AC}\right]$	IKT 4
	where A_A – cross section area of brace A $[T5]_A$ – SCF equation T5 evaluated using the geometric parameters of brace A $[K1]_{AB}$ – SCF equation K1 evaluated using braces A and B with brace A acting as reference brace i.e. τ, β and θ refer to brace A	

Table 8 : Hot spot stresses in KT-joints under OPB

Hot spot stress expression	Eqn. No.
Chord saddle hot spot stress adjacent to diagonal brace A: $\sigma_A\ [T10]_A\ [1 - 0.08\ (\beta_B\gamma)^{0.5}\exp(-0.8x_{AB})] \cdot [1 - 0.08(\beta_C\gamma)^{0.5}\exp(-0.8x_{AC})]$ $+ \sigma_B \cdot [T10]_B\ [1 - 0.08\ (\beta_A\gamma)^{0.5}\exp(-0.8x_{AB})] \cdot [2.05\beta_{max}^{0.5}\exp(-1.3x_{AB})]$ $+ \sigma_C \cdot [T10]_C\ [1 - 0.08\ (\beta_A\gamma)^{0.5}\exp(-0.8x_{AC})] \cdot [2.05\beta_{max}^{0.5}\exp(-1.3x_{AC})]$ where $x_{AB} = 1 + \dfrac{\zeta_{AB}\sin\theta_A}{\beta_A}$ $x_{AC} = 1 + \dfrac{(\zeta_{AB} + \zeta_{BC} + \beta_B)\sin\theta_A}{\beta_A}$	HSS 1
Saddle hot spot stress in diagonal brace A: $\tau^{-0.54}\gamma^{-0.05}(0.99-0.47\beta+0.08\beta^4) \cdot HSS1$	HSS2
Chord saddle hot spot stress adjacent to central brace B: $\sigma_B\ [T10]_B\ [1 - 0.08\ (\beta_A\gamma)^{0.5}\exp(-0.8x_{AB})]^{(\beta_A/\beta_B)^2} \cdot$ $[1 - 0.08\ (\beta_C\gamma)^{0.5}\exp(-0.8x_{BC})]^{(\beta_C/\beta_B)^2}$ $+ \sigma_A \cdot [T10]_A\ [1 - 0.08\ (\beta_B\gamma)^{0.5}\exp(-0.8x_{AB})] \cdot [2.05\beta_{max}^{0.5}\exp(-1.3x_{AB})]$ $+ \sigma_C \cdot [T10]_C\ [1 - 0.08\ (\beta_B\gamma)^{0.5}\exp(-0.8x_{BC})] \cdot [2.05\beta_{max}^{0.5}\exp(-1.3x_{BC})]$ where $x_{AB} = 1 + \dfrac{\zeta_{AB}\sin\theta_B}{\beta_B}$ $x_{BC} = 1 + \dfrac{\zeta_{BC}\sin\theta_B}{\beta_B}$	HSS3
Saddle hot spot stress in central brace B: $\tau^{-0.54}\gamma^{-0.05}(0.99-0.47\beta+0.08\beta^4)$ HSS3	HSS4

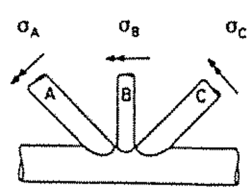


Table 9 : Influence functions for non-planar braces

<table>
<tr><th>Load type</th><th>Influence function</th><th>Eqn. No.</th></tr>
<tr><td>Axial load</td><td>

Chord saddle:

$$\frac{P_2}{A_i \sin \theta_i} \left[[X1]_i - [T5]_i \right]$$

Chord Crown:

$$\frac{P_1}{A_i} \left[\frac{C}{2} \alpha \beta_i \tau_i \right]$$

Brace saddle:

$$\frac{P_2}{A_i \sin \theta_i} \left[[X3]_i - [T3]_i \right]$$

Brace Crown:

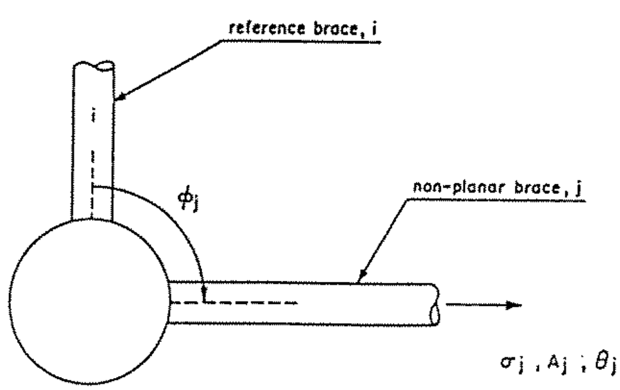
$$\frac{P_1}{A_i} \left[\frac{C}{5} \alpha \beta_i \tau_i \right]$$

where

$$P_1 = \sum_{j=1}^{n} \sigma_j A_j \cos \phi_j \cdot \sin \theta_j$$

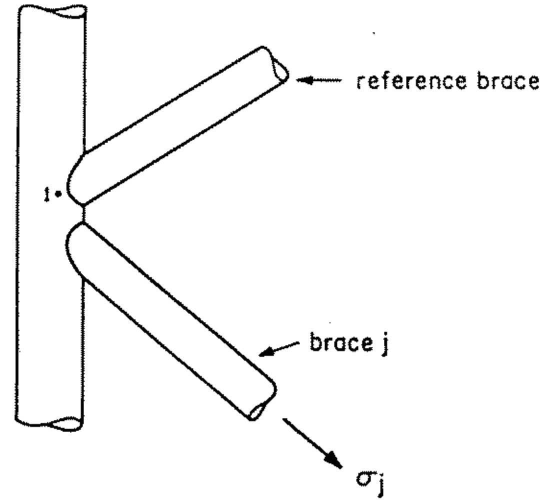
$$P_2 = \sum_{j=1}^{n} \sigma_j A_j \cos 2\phi_j \cdot \sin \theta_j$$

i = suffix denoting the brace under consideration
j = suffix denoting non-planar braces
n = number of non-planar braces
σ_j = nominal axial stress on brace j
A_j = cross section area of brace j
C = chord-end fixity parameter ($0.5 \leqslant C \leqslant 1.0$)



</td><td>IM 1
IM 2
IM 3
IM 4</td></tr>
</table>

(a)

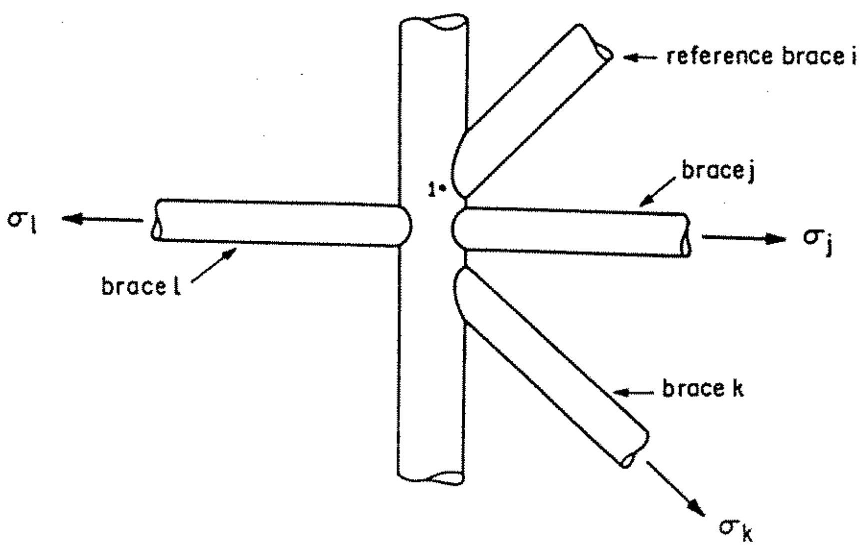


IFij = Influence function at chord saddle location of brace i due to axial load on brace j.

σ_i = hot spot stress at chord saddle of brace i due to axial load on brace j.

$\sigma_i = \sigma_j * IFij$ (geometry)

(b)

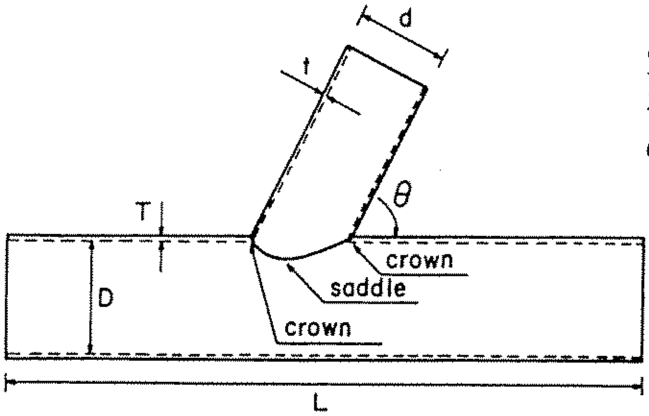


σ_i = hot spot stress at chord saddle of brace i due to axial loads on braces j, k and l

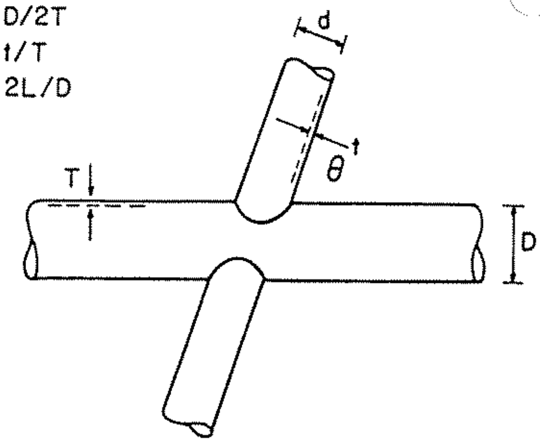
$\sigma_i = \sigma_j * IFij + \sigma_k * IFik + \sigma_l * IFil$

DRAW. No.: H 71706/1	DEFINITION OF INFLUENCE FUNCTION	FIGURE No. 1

$\beta = d/D$
$\gamma = D/2T$
$\tau = t/T$
$\alpha = 2L/D$

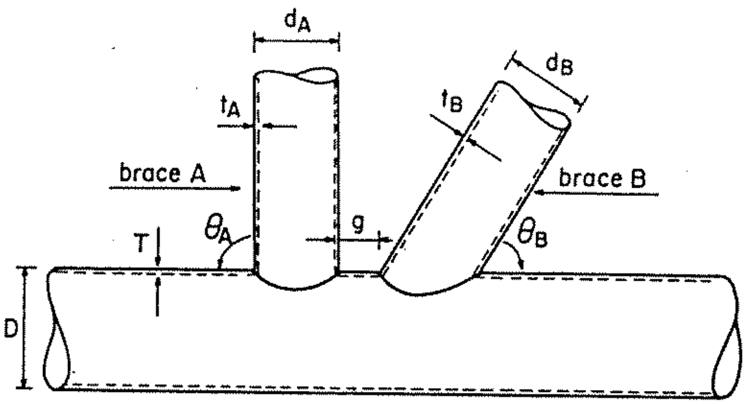


T OR Y - JOINT



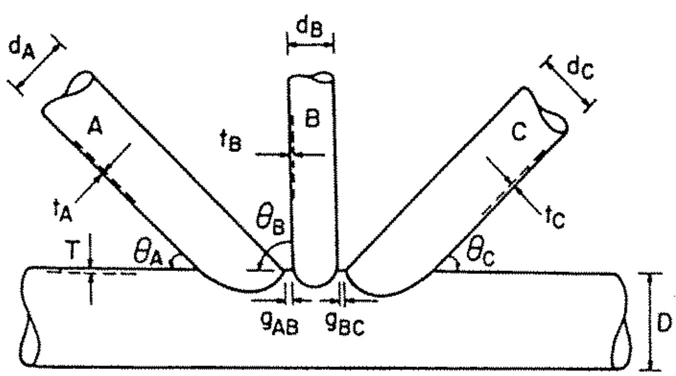
X - JOINT

$\beta_A = d_A/D, \tau_A = t_A/T, \gamma = D/2T$
$\beta_B = d_B/D, \tau_B = t_B/T, \zeta = g/D$



K- JOINT

$\beta_A = d_A/D$	$\tau_A = t_A/T$	$\zeta_{AB} = g_{AB}/D$
$\beta_B = d_B/D$	$\tau_B = t_B/T$	$\zeta_{BC} = g_{BC}/D$
$\beta_C = d_C/D$	$\tau_C = t_C/T$	$\gamma = D/2T$



KT- JOINT

ALL SHOULD READ $\leq$ not $\geq$

VALIDITY RANGE

$0.2 \geq \beta \geq 1.0$

$0.2 \geq \tau \geq 1.0$

$8 \geq \gamma \geq 32$

$4 \geq \alpha \geq 40$

$20° \geq \theta \geq 90°$

$\frac{-0.6\beta}{\sin\theta} \geq \zeta \geq 1.0$

DRAW. No.: H71706/2	DEFINITION OF GEOMETRICAL PARAMETERS	FIGURE No. 2

STRENGTHENING, MODIFICATION AND REPAIR OF OFFSHORE INSTALLATIONS

PART IV - BACKGROUND DATA AND ASSESSMENTS

STRENGTHENING, MODIFICATION AND REPAIR OF OFFSHORE INSTALLATIONS

PART IV - BACKGROUND DATA AND ASSESSMENTS

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STRENGTHENING, MODIFICATION AND REPAIR OF OFFSHORE INSTALLATIONS

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NOMENCLATURE

a_c	concrete contribution factor
A	total surface area of clamp (m^2); area of slip surface being mobilised (mm^2)
A_b	cross section area of one stud (mm^2)
A_{brace}	cross sectional area of brace (mm^2)
A_c	area of concrete at undented section (mm^2)
A_{clamp}	cross sectional area of clamp (mm^2)
A_g	area of grout at the dented cross-section (mm^2); area of grout (mm^2)
A_s	area of steel (mm^2)
A_{tr}, A_{tr}^*	transformed area at dented and undented cross-section (mm^2)
A_{total}	total cross sectional area of member and clamp (mm^2)
C	current density (mA/m^2)
C_c	correction factor for shear connector extending only partially around circumference
C_L	length correction factor
C_s	surface condition factor for bond component
C'_s	surface condition factor for frictional component
D	damage (fatigue); outer diameter
D'	mid thickness diameter
d	depth of dent, diameter of brace
d_l	bow in member
E_b	Young's modulus of studbolt material (N/mm^2)
E_g	Young's modulus of grout (N/mm^2)
E, E_s	Young's modulus of steel (N/mm^2)
e	external eccentricity of load
e_g	distance between the centroid of grout at the dented cross-section to the centroid at undented section
e_s	distance between the centroid of steel at the dented cross-section to the centroid at undented section
e_t	total eccentricity
e_{tr}	distance between the centroid of dented and undented transformed cross-section
f_{cc}	enhanced characteristic strength of triaxially contained concrete (N/mm^2)
f_{cu}	grout cube strength (N/mm^2)
f_e	Euler buckling stress (N/mm^2)
$\underline{F}$	axial force in member
F	axial force to be transferred to clamp
F_{max}	maximum axial force in member (kN)
F_{min}	minimum axial force in member (kN)
F_n	total studbolt load (kN)
f'_y	reduced nominal yield strength of steel (N/mm^2)
F_y, f_y	yield stress of steel (N/mm^2)
f_{ys}	yield strength of stud (N/mm^2)
g	gap between braces for K/YT joints
h	shear connector height (mm)
h_s	stud shear key height (mm)

I	moment of inertia
I_E	length of column for which the Euler load equals the squash load
I_g	moment of inertia of grout at dented cross section (mm^4)
I_s	moment of inertia of steel at dented cross-section (mm^4)
I_{tr}	transformed moment of inertia of dented cross-section (mm^4)
I_{tclamp}	torsional moment of inertia for clamp (mm^4)
I_{ttotal}	torsional moment of inertia for clamp and member (mm^4)
$I_{xxclamp}$	second moment of area for clamp about X-X axis (mm^4)
$I_{yyclamp}$	second moment of area for clamp about Y-Y axis (mm^4)
$I_{xxtotal}$	second moment of area for clamp and member about X-X axis (mm^4)
$I_{yytotal}$	second moment of area for clamp and member about Y-Y axis (mm^4)
IPB	in-plane bending
k	non-dimensionalised parameter, factor used in calculation of γ_e
K_a	approximate intersection length factor
K'_a	exact intersection length factor
K	effective length factor
K, K'	stiffness factors
K'	stress concentration factor relevant to the flange radius detail
K_b	axial stiffness of studbolt pair; studbolt stiffness parameter
K_c	radial stiffness of grout annulus/jacket member/grout fill combination
K_g	local groove SCF
K_o	codified formulae reduction factor
L	length of connection; member length; length of chord node barrel
L_b	bolt length (for split grouted connection) (mm); stressed length of bolt
L_c	circumferential length of shear connector (mm)
m	modular ratio of steel to grout; non-dimensionalised parameter
M	moment load
$\bar{M}$	maximum theoretical moment in member (kNm)
$\bar{M}_x$	torsion to be transferred to clamp (kNm)
Mx	torsion in member (kNm)
M_{ip}, M_{op}	in-plane and out-of-plane bending respectively (kNm)
$\overline{M_{ip}}$, $\overline{M_{op}}$	in-plane and out-of-plane bending transferred to clamp respectively (kNm)
M_k	characteristic moment capacity of joint
M_{max}	the greater of the in-plane or out-of-plane bending (kNm)
M_p	plastic moment capacity of intact steel section (kNm)
M_{pd}	plastic moment capacity of damaged steel section (kNm)
M_u	plastic moment capacity of grouted tubular (kNm); moment capacity of joint
n	number of studbolts
N	number of cycles (fatigue); total number of studbolts; sample size
N_{max}	number of cycles in 100 years
OPB	out-of-plane bending
p	member external pressure (N/mm^2)
p_o	annulus external pressure (N/mm^2)
P	beam-column strength determined by design procedure (kN), axial load
P_c	characteristic slip resistance (kN)
P_E	Euler buckling load

P_k — characteristic axial capacity of joint
P_{suc} — characteristic shear load of shear key stud (kN)
P_u — squash load (kN), axial capacity of joint
P_{col} — column strength (kN)
q — external radial pressure
Q_β — geometric modifier
r_{tr} — transformed radius of gyration of dented section (mm)
R — radius of curvature; property section ratio, stress ratio for fatigue loading; existing member outer radius
R_o — grout annulus external radius
R_1 — existing member inner radius
s — shear connector spacing (mm)
S — stress range (fatigue)
SCF — stress concentration factor
SCF_E — Efthymiou predicted stress concentration factor
Sr_{max} — maximum stress range (N/mm^2)
S_b — bolt spacing (for split grouted connection)
S_h — circumferential spacing of stud shear key (mm)
S_i — longitudinal spacing of stud shear key (mm)
t — plate thickness, thickness of tubular member (mm), brace wall thickness
T — thickness of tubular member, chord wall thickness
T_{gs} — effective steel wall thickness for grout core
T_e — effective chord wall thickness for grouted joints
U — anode utilisation factor
V_{ip}, V_{op} — in-plane and out-of-plane shear respectively (kN)
w — width of shear connector (mm)
w_{mip}, w_{mop} — contact load/unit length due to in-plane and out-of-plane bending respectively (N/mm)
w_{vip}, w_{vop} — contact load/unit length due to in-plane and out-of-plane shear respectively (N/mm)
w_{TOT} — resultant contact load/unit length due to in-plane and/or out-of-plane bending (N/mm)
W — weight of sacrificial anode required (kg)
w' — resultant contact load/unit length due to in-plane and/or out-of-plane shear
x — depth of cut or groove; distance from clamp centre line to centroid of half clamp about X-X axis
y — distance from clamp centre line to centroid of half clamp about Y-Y axis
Y — design life of repair (Years)
Z — capacity of alloy (Ah/kg)
z_{tr} — transformed section modulus with respect to the dented side (mm^3)
α — 2L/D for chord (chord length ratio); distribution factor; angle shown in Figure 5.3.2; constant (= 1.9281)
β — d/D (joint diameter ratio)
Δ — sum of eccentricity and initial bow
δ — height of surface irregularities ($\delta/D_p = 1.25 \times 10^{-4}$ for rolled steel surface)
θ — brace joint intersect angle
γ — D/2T (chord thickness ratio)
γ_e — effective γ ratio for grouted joints

ζ	g/D (gap ratio for K/YT joints)
σ_a	axial stress (N/mm^2)
σ_A	acting slip stress due to axial load (N/mm^2)
σ_b	curve fitting constant; bond strength (N/mm^2); bending stress
σ_{ba}	allowable bond strength (to API RP2A) (N/mm^2)
σ_{bc}	characteristic bond strength (N/mm^2)
σ_B	acting slip stress due to bending load (N/mm^2)
σ_c	characteristic slip strength (N/mm^2)
σ_{cu}	grout compressive strength (N/mm^2)
σ_f	frictional strength (N/mm^2)
σ_h	hoop stress (N/mm^2)
σ_L	longitudinal stress (N/mm^2)
σ_s	slip strength (N/mm^2)
σ_{SFBnom}	maximum nominal stress in sleeve flanges (N/mm^2)
σ_T	acting slip stress due to torsion load (N/mm^2)
σ_u	ultimate axial stress (N/mm^2)
σ_y	yield stress of tubular (N/mm^2)
σ_{ys}	yield stress of stub shear key (N/mm^2)
τ	shear stress (N/mm^2), $\frac{t}{T}$ (thickness ratio for joint); interface shear stress induced by in-plane and out-of-plane bending (N/mm^2)
τ_a	axial interface slip stress (N/mm^2)
τ_b	bending interface slip stress (N/mm^2)
τ_{ip}, τ_{op}	shear stress due to in-plane and out-of-plane bending respectively (N/mm^2)
τ_s	allowable interface slip strength (N/mm^2)
τ_{max}	maximum shear stress (N/mm^2)
Γ	partial safety factor
$\Gamma(.)$	gamma function
Γ_f	safety factor for frictional strength
Γ_b	safety factor for bond
Γ_s	factor of safety on stud yield; safety factor for bending failure of stud shear key
Γ_t	safety factor for sleeve local buckling
Γ_T	factor of safety for failure
η_1, η_2	curve fitting constants
μ	frictional coefficient; curve fitting constant
μ_{allow}	allowable coefficient of friction
μ_c	characteristic frictional coefficient
μ_l	local frictional coefficient
μ_g	global frictional coefficient
μ_{mean}	mean frictional coefficient
μ_t	local frictional coefficient
μ_{ult}	ultimate coefficient of friction
Φ	diameter of studbolt (mm); diameter of stud shear key (mm)
λ	reduced slenderness parameter
β	ratio of smaller to larger end moments
υ	Poissons ratio

υ_g	Poissons ratio for grout
υ_s	Poissons ratio for steel
π	3.142

IV 1 INTRODUCTION

IV 1.1 GENERAL

This Part IV contains all the background data and details of the assessment of those data leading to the recommendations given in Part III. The purpose of this is to allow the designer to assess the basis of the recommendations and to facilitate the development of revisions as and when new data become available. The justification of the recommendations is also necessary where Certifying Authority approval is required.

All strengthening, modification and repair (SMR) techniques are addressed in this part. Only those techniques which are sufficiently developed or have actually been used are covered in Part III. The one exception to this is member removal which is addressed in Part III but not here due to the lack of data of a research nature.

For various SMR techniques, data has been collated from public domain sources and from previously confidential data released to this project. Screening criteria have been applied, as given under the relevant sections of this part. Certain SMR schemes may require specific experimental investigations to be carried out and the availability of the verified databases permits the results to be interpreted with the benefit of a larger database, leading to reduced statistical penalties for using a single or small number of tests to support a design.

IV 1.2 STATISTICAL CALCULATIONS

A number of statistical studies have been conducted, as described in the relevant sections of this part, to derive mean equations and characteristic equations. Fatigue S-N design curves have been based on mean curves minus two standard deviations, as this is the commonly accepted practice for such curves. For a very large sample size, the fatigue design curves correspond to 97.5% survivability.

For all other statistical calculations, the characteristic value is defined at the 95% survivability level. When dealing with a finite set of results, the characteristic value should properly be calculated with a specified level of confidence. In this work a 50% confidence level has been taken, in common with other offshore codes, eg. Reference 1.1. This is to say that the calculated characteristic value has a 50% probability of being greater (and 50% probability of being lesser) than the true characteristic value that would be derived from an infinite population of results. Tables, given in Reference 1.2, were consulted to find the number of standard deviations (which depends on the number of results) that had to be subtracted from the best fit mean line. The best fit mean line (or surface) was derived by the least squares method based on percentage (not absolute) differences.

REFERENCES

1.1 Health and Safety Executive (formerly the UK Department of Energy). 'Offshore Installations: Guidance on design, construction and certification'. 4th Edition, HMSO, London 1990.

1.2 Baker, M.J. 'Variability of the strength of structural steel - A study in structural safety'. CIRIA Technical Note No. 44, April 1973.

IV 2 WELDING TECHNOLOGY

IV 2.1 INTRODUCTION

In Part III, certain aspects of underwater hyperbaric welding were noted as requiring additional detail which was not justified in that discussion. Since high environmental pressures are not normally considered in conventional welding, yet can produce marked effects, it is the purpose of the following section to comment on some of these aspects.

The following information has been drawn together and condensed from a survey of the metallurgical literature. Further details may be found in the references listed in Parts II and III, as well as in the Bibliography in Part VII.

IV 2.2 SMA WELDING

The main effects of pressure on hyperbaric SMAW seem to be on the degree of fusion or penetration of welds, the control of the weld pool, and on the chemistry of the weldment.

The degree of fusion with the parent plate decreases with depth under SMAW. Although the burn-off rate of the electrodes can be increased by as much as 50% with pressure, the arc voltages are not significantly higher, as they are controlled by the presence of easily ionised material within the arc atmosphere derived from the flux. Therefore, the total arc energy does not increase with depth, and because an increasing amount of it is being used to melt the electrode, the level of parent plate fusion is reduced.

Control of the weld pool under hyperbaric conditions is extremely difficult for positions other than downhand. The only feasible solution has been to limit the size of the weld pool, and hence the heat input, so that surface tension forces can control the pool. Unfortunately, low heat inputs increase the cooling rates of the weld metal and HAZ, with the metallurgical consequences described in Part III, Section III 2.7.

The chemistry of the weld undergoes complicated alterations with pressure, being basically controlled by the effect of pressure on the reactions in the arc. One commonly occurring reaction is the breakdown of carbonates in the flux to form carbon monoxide and dioxide, which form part of the gas shield for the weld pool. These gases are subject to equilibria both with each other and with their dissociation products of carbon and oxygen. These last two species can be taken into solution in the weld pool. With increased pressure, the volume of gas is reduced, leading to a shielding problem. There are two factors at work: the reactions are driven by the laws of chemical equilibria to lower the net volume of gas; and what gas is produced is compressed by the pressure.

The net effect is to produce weldments that have been poorly protected and which have higher levels of carbon and lower levels of deoxidants such as silicon and manganese. In general, the welds are more hardenable and sensitive to high cooling rates than welds carried out on the surface, and should be evaluated before acceptance.

The high environmental pressure and humidity levels enhance the rate at which moisture is absorbed into the flux coating of the electrode, thus leading to a danger of hydrogen cracking (HICC). Some precautions against this are noted in Part III, Section III 2.

IV 2.3 GTA WELDING

Increased pressure mainly affects the voltage, process efficiency, and arc stability of GTAW.

The most noticeable effect of elevated pressure is that arc voltages increase. At the currents normally used, arc voltage is only marginally affected by operating current.

The arc voltage in GTA has three elements - two 'fall' voltages and the 'column' voltage. The fall voltages are associated with the energy required to displace the electrons of the current from within the electrode to the arc, and from the arc into the weld pool. These do not seem to be significantly affected by environmental pressure or composition, and total about 9 volts. The column voltage is proportional to the arc length, and is measured in terms of the electric field strength (voltage drop per mm of arc length). At atmospheric pressure, the electric field strength for argon is about 0.8 V/mm, and for helium approximately 1.8 V/mm. The electric field strength is proportional to the square root of the absolute pressure relative to atmospheric pressure. For example, at a depth of 30 metres, where the gauge pressure is 3 atmospheres (3 bar, 3 x 10^2kPa), the absolute pressure will be four times atmospheric, and the electric field strength will be twice that at sea level.

The arc voltage can be expressed as:

$$V_{ARC} \approx 9 + E.\ell.\sqrt{(P/P_A)}$$

where: V_{ARC} is the arc voltage (V)
E is the electric field strength at one atmosphere (V/mm)
P is the absolute pressure (bar, kPa)
P_A is the atmospheric pressure (bar, kPa)
ℓ is the arc length (mm).

If a mixture of gases is used, the electric field strength is proportional to the composition, thus a 75% argon, 25% helium mixture by volume would have a field strength of:

$$E = 0.75 * 0.8 + 0.25 * 1.8 = 1.05 \text{ V/mm}$$

For a surface power supply, it would be necessary to add the additional voltage for the resistance of the cables, as discussed in Part III, Section III 2.5.

Because of their effects on GTA arc voltage, these factors must be specified before practical voltage operating limits can be established in a welding procedure.

The process efficiency of GTAW varies with pressure in a complicated manner. It initially declines from a typical figure of 90% at one atmosphere to 70% at 6 bar (6 x 10^2kPa), recovering to about 75% by 8 bar (8 x 10^2kPa), and remaining constant thereafter. This behaviour is not the same as that of SMAW, and consequently, at a depth of approximately 300 metres, GTAW can achieve deposition rates comparable with SMAW.

As in surface-based practice, deposition rates can be improved by the addition of helium to the usual argon-rich shielding gas, though excessive helium will cause electrode erosion.

However, as environmental pressure is increased, the stability of the GTA arc is reduced. Some of the problems stem from poor-quality tungsten electrodes, and care should be taken to use only high-quality electrodes. Arc instability is also produced by purely hyperbaric phenomena, which take the form of a random oscillation of the arc around its root on the tungsten electrode, the magnitude of the instability being linked to the mass flow of the shielding gas supply. The arc is also more susceptible to external magnetic fields with depth, and these two effects limit the practical depth for hyperbaric GTAW operations to around 500 metres, although this may vary in specific situations.

Conventional striking systems for hyperbaric environments use carbon blocks close to the weld preparation, onto which the tungsten electrode can be touched to start the arc without damage to the tungsten electrode. Modern inverter power systems can allow direct touch striking onto the weld itself, though their use is not widespread for hyperbaric applications and in this form has not really progressed beyond the laboratory. Surface techniques of high frequency initiation are considered to be neither practicable nor safe for the welder-diver.

IV 2.4 GMA WELDING

The main problems with hyperbaric GMAW are the complexity of the equipment, the need to cater for fluctuations in the feed of the consumable, and limitations to positional welding.

The components of GMAW require constant inspection and maintenance, and the vast majority of problems with GMAW can be ascribed to this part of the welding system. This militates against the use of GMAW hyperbarically, where simplicity and reliability are highly prized attributes.

The usual way of dealing with the problem of feed variations, by using a welding power supply of virtually constant potential output, causes random variations in arc voltage and current, which can cause problems with the shape and fusion characteristics of the weld, and these effects are enhanced by the high electric field strengths present in hyperbaric environments.

With the steels used offshore, when operating in the spray transition regime of GMAW, the current is above 200 A, and the weld pool is too large to be controlled in other than the downhand position. The other main metal transfer mechanism of 'short-arc' welding is not often suitable for offshore use owing to its low heat inputs.

Hyperbaric GMAW does have a number of significant attractions, principally its low hydrogen potential and high deposition rates. Attempts have been made to improve the control over the system, especially the development of Controlled Transfer Pulse (CTP) GMAW, but although pulsed current solid wire hyperbaric GMAW shows promise as a potential repair technique, it is not available commercially offshore, and more work is needed to establish welding procedures.

IV 3 WELD IMPROVEMENT TECHNIQUES

IV 3.1 GENERAL

Weld improvement techniques are applied to welds to improve their fatigue performance. No gain in the static strength can be obtained by these techniques (except, perhaps, the use of remedial grinding when brittle fracture of a cracked joint may be a possibility).

The weld improvement techniques that have been examined, at least in the laboratory, encompass:

- toe grinding and remedial grinding, using rotary burrs or grinding discs
- hammer, shot or needle peening
- dressing the weld toe using TIG or plasma arc
- improved weld profile shape
- hole drilling at end of cracks.

Note that remedial grinding and hole drilling are applicable to situations where a crack exists and thus they may not be recognised in the literature as weld improvement techniques. However, remedial grinding and toe grinding only differ in practice by the amount of material removed and it is therefore convenient to treat them both under this Section IV 3.

Fatigue cracking is normally associated with weld toes. It is at these locations that small discontinuities (eg. slag inclusions), inherent to welding, arise and high tensile residual stresses are present due to differential thermal contraction which occurs between the weld and parent metal during the weld cooling process. Fatigue life improvement is achieved by addressing one or both of these aspects (discontinuities and/or residual stresses) or by reducing the SCF at the weld.

IV 3.2 TOE GRINDING

IV 3.2.1 <u>Existing Guidance</u>

The primary design reference for toe grinding operations is the UK HSE (formerly DoE) Design Guidance for Offshore Installations[3.1]. The document gives explicit details of the use of the toe grinding weld improvement technique in Appendix A.21. The content of the document is clear and is reproduced verbatim hereafter:

"(iii) Weld Improvement

For welded joints involving potential fatigue cracking from the weld toe an improvement in strength by at least 30%, equivalent to a factor of 2.2 on life *can be obtained by controlled local machining or grinding of the weld toe. This is carried out either with a rotary burr or by disc grinding. The treatment should produce a smooth concave profile at the weld toe with the depth of the depression penetrating into the plate surface to at least 0.5mm below the bottom of any visible undercut and ensuring that no exposed defects remains. The maximum depth of local machining or grinding should not exceed 2mm or 5% of the plate thickness. In the case of a multi-pass weld more than one weld toe may need to be dressed. Where toe grinding is used to improve the fatigue life of fillet welded connections, care should be taken to ensure that the required throat size is maintained. It is recommended that no advantage for toe grinding should be taken at the initial design stage. (Note: the benefit of grinding may be claimed only for welded joints which are adequately protected from sea water corrosion). Any credit for other beneficial treatments should be justified by evidence submitted to the Certifying Authority.*

Overall weld profiling is obviously to be encouraged, but until further data is obtained no improvement in fatigue strength can be allowed unless accompanied by toe grinding.

It should also be noted that in the case of partial penetration welds, where failure may occur from the weld root, grinding of the weld toe cannot be relied upon to give an increase in strength. "

The notes are presently being redrafted but the substance of the 1990 version (given above) is not altered. However, attention is drawn to a number of points:

- Non-destructive examination must be used to approve the as-ground finished surface.

- Scope is given for using hammer or shot peening to enhance fatigue life. No explicit rules are given, but the door is left open for techniques to be used where allied with a validated test programme.

Fatigue design of new structural frameworks is undertaken using client approved "Design Briefs". Although the design briefs are confidential documents, there is a general uniformity in approach throughout the design industry because of the interchange of personnel associated with project work. It is understood that some of the fatigue design briefs which are in current use do advise the application of toe grinding as a standard approach for dealing with node connections which have a lower than target fatigue life. Full weld grinding is a well-established procedure, which may be arbitrarily chosen by the client team for use in a particular project.

IV 3.2.2 Review of Existing Work

Knight[3.2] reports tests on 12.5mm plate samples in air. The tests were for BS 4360 Grade 43A steel and higher strength steel.

Booth (also reported under UKOSRP I)[3.3] examined the fatigue behaviour of a number of types of weld improvement in air and in a simulated seawater environment. His work concentrated on plate elements but gives the basis for the development of design rules for plates subject to underwater immersion with and without CP protection. He finds that if a weld is toe ground and then placed underwater with no cathodic protection, then the fatigue life improvement resulting from the toe grinding is lost. This is probably because the ground weld rapidly becomes a site for corrosion and, as a result is subject to corrosion fatigue which totally negates the influence of weld toe grinding.

Booth's findings are consistent with the view that the operation of toe grinding produces two improvements:

- Production of a distinct crack initiation period which is longer than that associated with the as welded condition.

- Reduction of the effective stress concentration factor in the region of probable crack growth.

A number of proving experiments with direct validity for offshore design work have been carried out under the auspices of the United Kingdom Offshore Steel Research Project (UKOSRP)[3.4,3.5]. Attention has been paid to the behaviour of improved welds both in air and underwater. The work forms an important bridge between the work of Booth and that of Knight because it emphasises the need to neglect weld grinding improvements in freely corrosive environments but to allow the full benefit of air testing to be read across to the underwater (but cathodically protected) area.

Data presented by the three sources quoted above are of direct importance to the work described herein. Other sources[3.6,3.7] have undertaken similar test programmes and produce the same range of results - principally that weld improvement techniques do work.

IV 3.2.3 Database -

The experimental tests are described in the Databases IV 3.2.1 to IV 3.2.3. Details of individual test programmes are summarised below using the same terms as are used in the Databases.

Knight[3.2] reports on tests carried out by the (UK) Welding Institute to determine the efficacy of grinding and peening. Tests were carried out in air on BS 4360 Grade 43a steel samples, using as welded, toe ground and disc ground welds. The weld detail was a 12.5mm plate with non-load carrying plates attached both sides using a double sided 10mm fillet. Fatigue data was obtained, which is shown on Figure 3.2.1. Similar tests were conducted for high strength steel having a yield strength of 685 N/mm^2. The results are shown in Figure 3.2.2.

Booth tested joints in the **UKOSRP-I** project[3.4] in simulated seawater in order to determine the improvement associated with toe grinding and hammer peening. All his test samples were 38mm thick, and consisted of cruciform joints loaded in bending. Booth's work shows that the benefit of toe grinding can be used in seawater provided that an adequate CP system is in operation. Booth's data is presented in Figure 3.2.3 and 3.2.4.

UKOSRP-II[3.5] investigated 38mm thick cruciform load carrying joints and the tests were carried out in simulated seawater with varying degrees of CP protection. Samples were as welded, and toe ground, using a rotary burr. The tests were all on BS 4360 Grade 50D steel. See Figures 3.2.5 and 3.2.6.

Mullen[3.6] tested 25mm thick T samples of K prep butt welds. The plate material was ASTM A572 Grade 50 steel, similar to BS 4360 Grade 50D. Mullen's test data is not presented in detail and cannot be included in the database, but his work provides further confirmation of the effects of toe grinding, and in particular, the benefit of using rotary burrs as opposed to disc grinding.

Work undertaken in **Norway**[3.7 and 3.8] has been included in the database. The test was a four point bending test, in which the welded attachment is not directly loaded. The samples were of Grade 50D type material (Plate was described as Statoil Type 1 with $F_y = 368$ N/mm^2). Samples were toe ground and as welded. Other tests were performed. The plates were 30mm thick and a K prep butt weld was used throughout. Data is displayed on Figure 3.2.7.

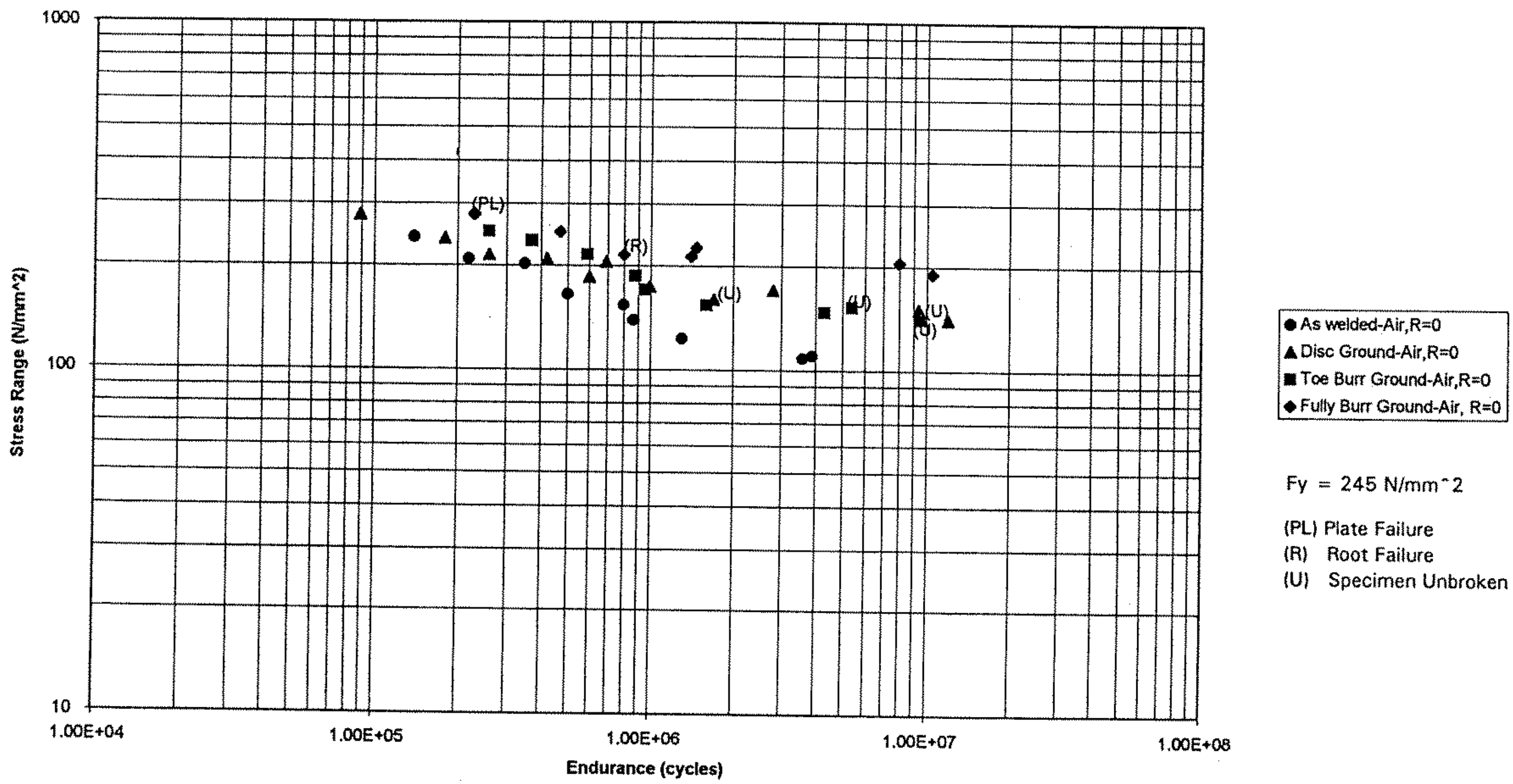

Figure 3.2.1: Data derived from Knight's mild steel fatigue tests

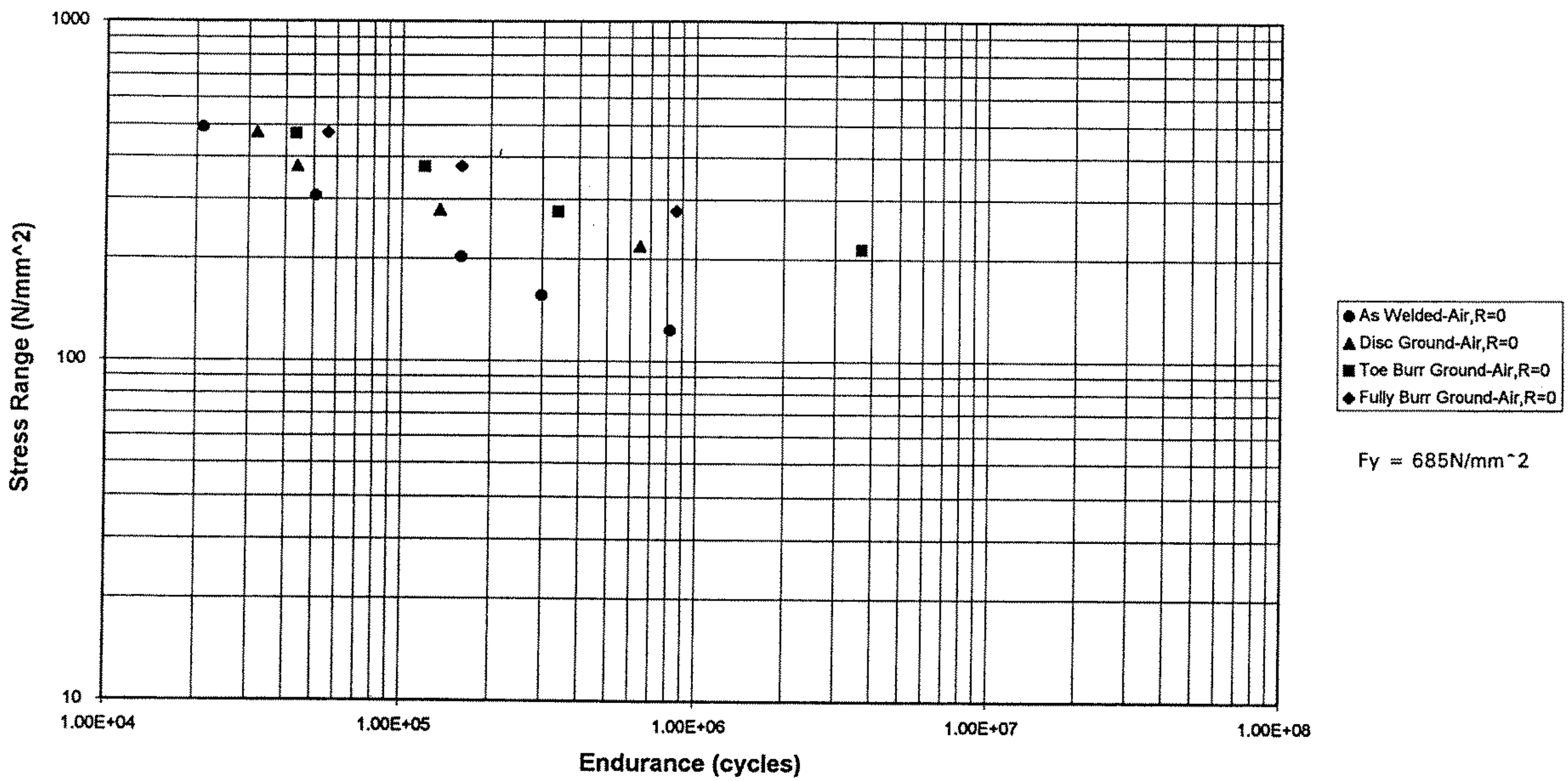

Figure 3.2.2: Data derived from Knight's high strength steel fatigue tests

S M:\C111\00R222_1\T3_2_1-7.XLS

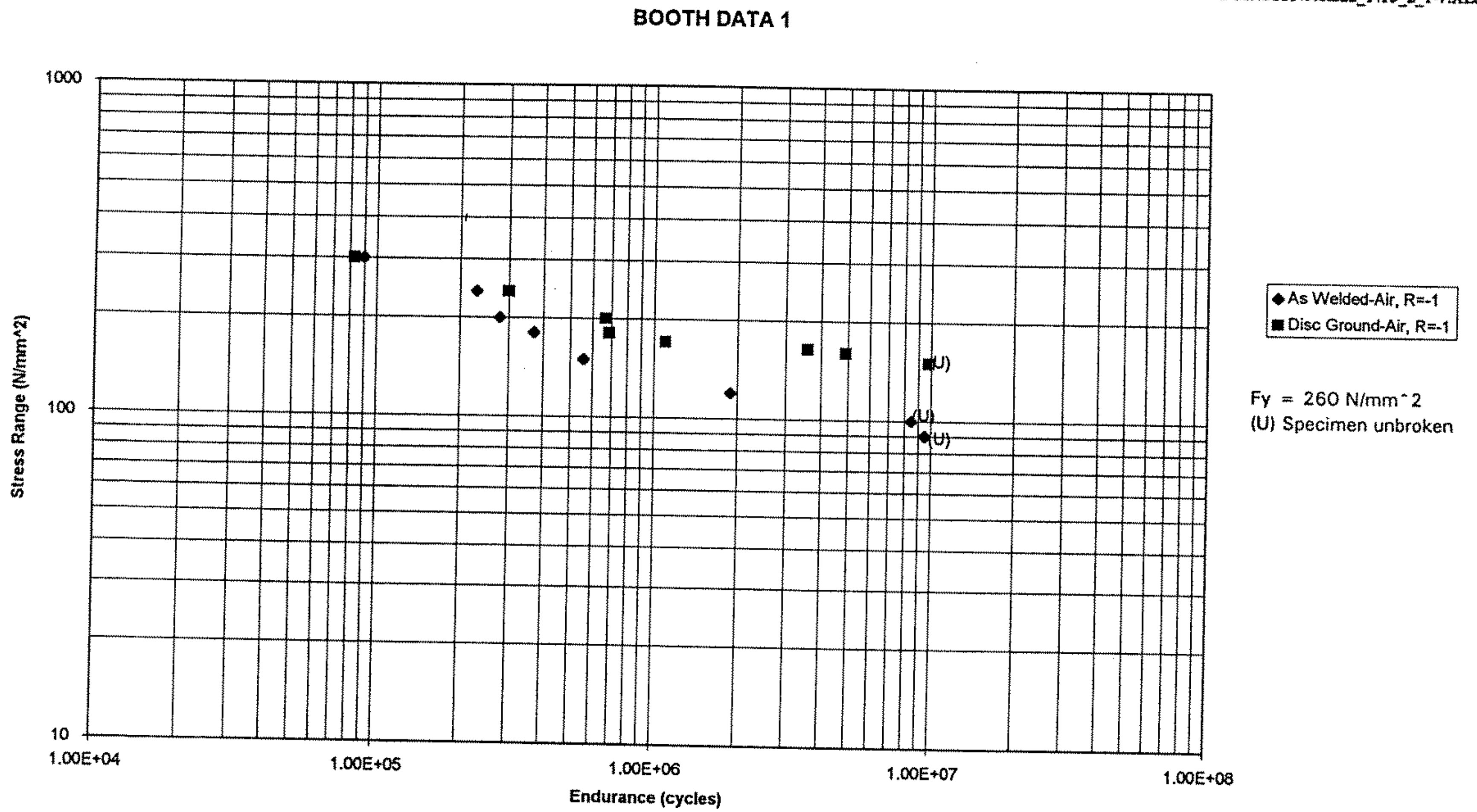

Figure 3.2.3: Data derived from Booth's tests (UKOSRP I), R = -1

MSL

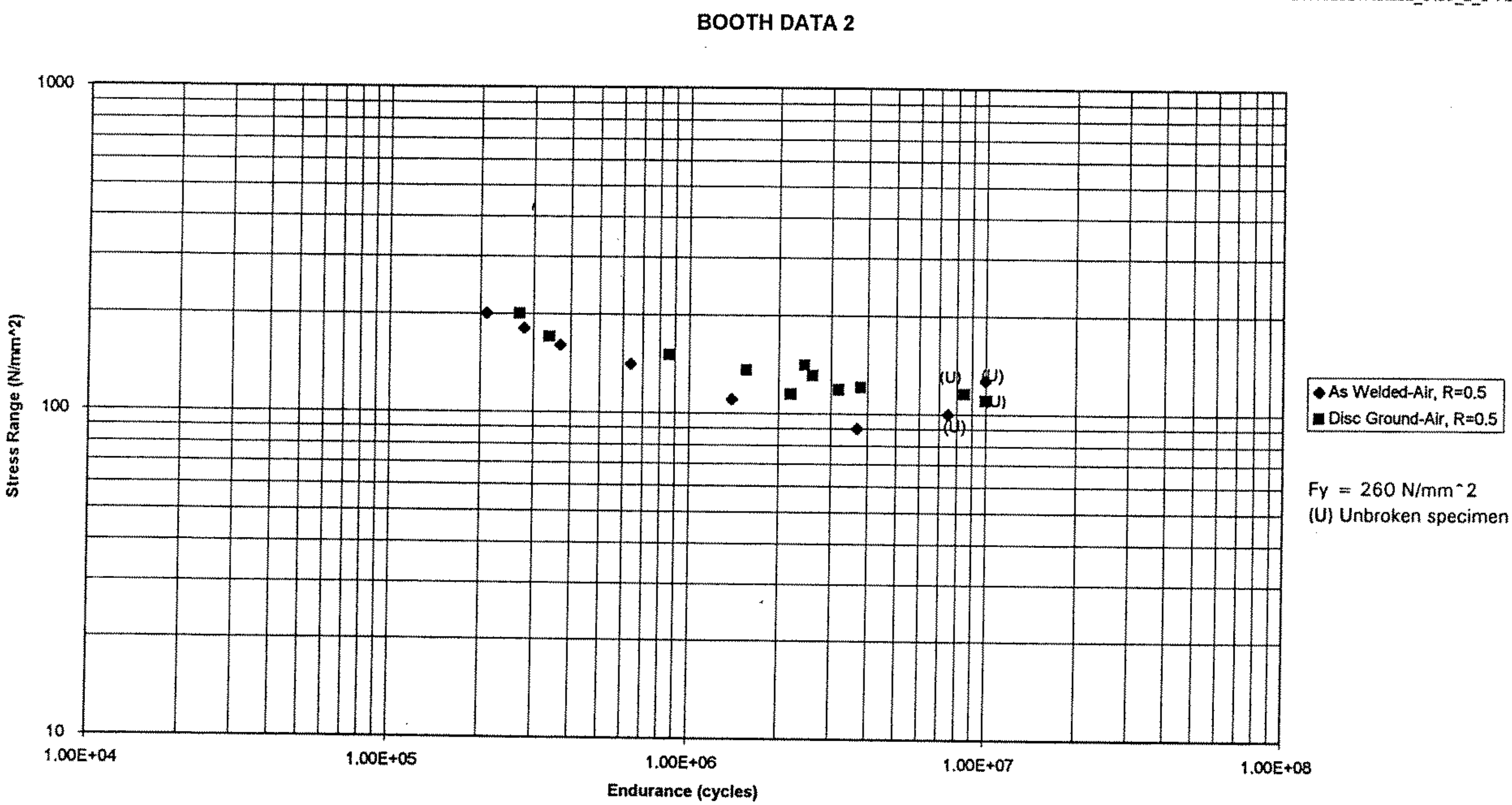

Figure 3.2.4: Data derived from Booth's tests (UKOSRP I), R = 0.5

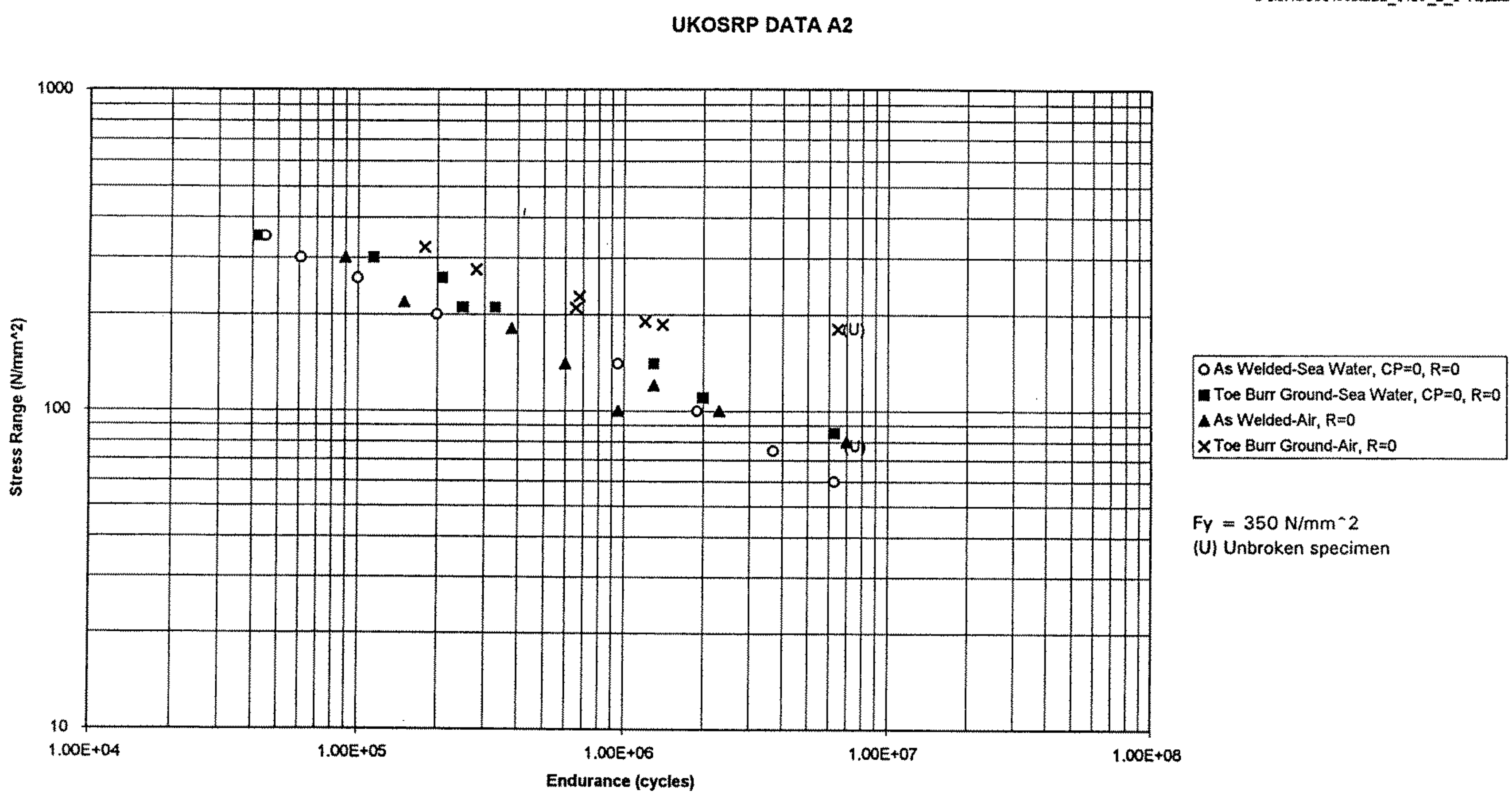

Figure 3.2.5: Data derived from UKOSRP II (no CP)

S M:\C111\00R222_1\T3_2_1-7.XLS

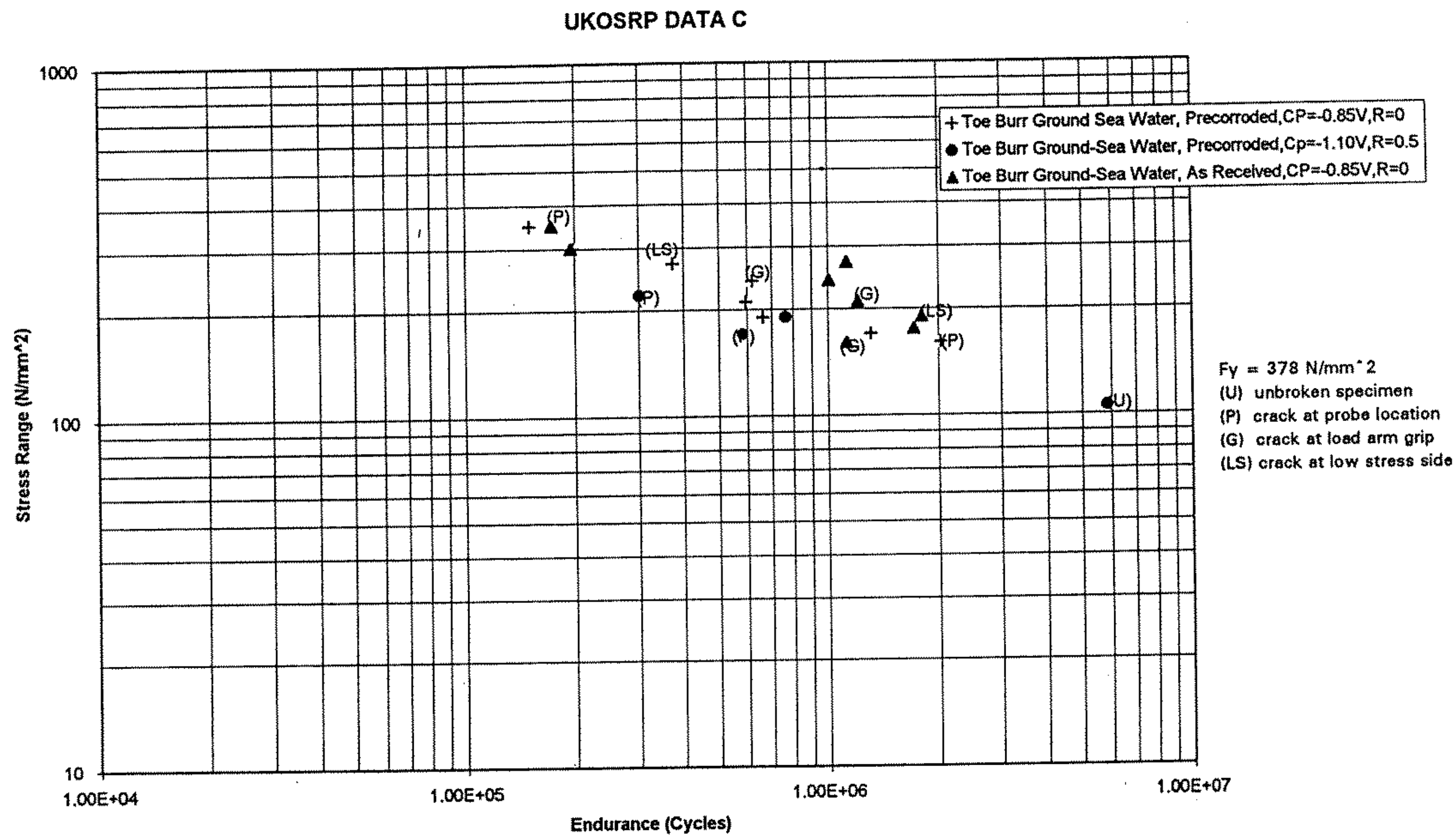

Figure 3.2.6: Data derived from UKOSRP II (with CP)

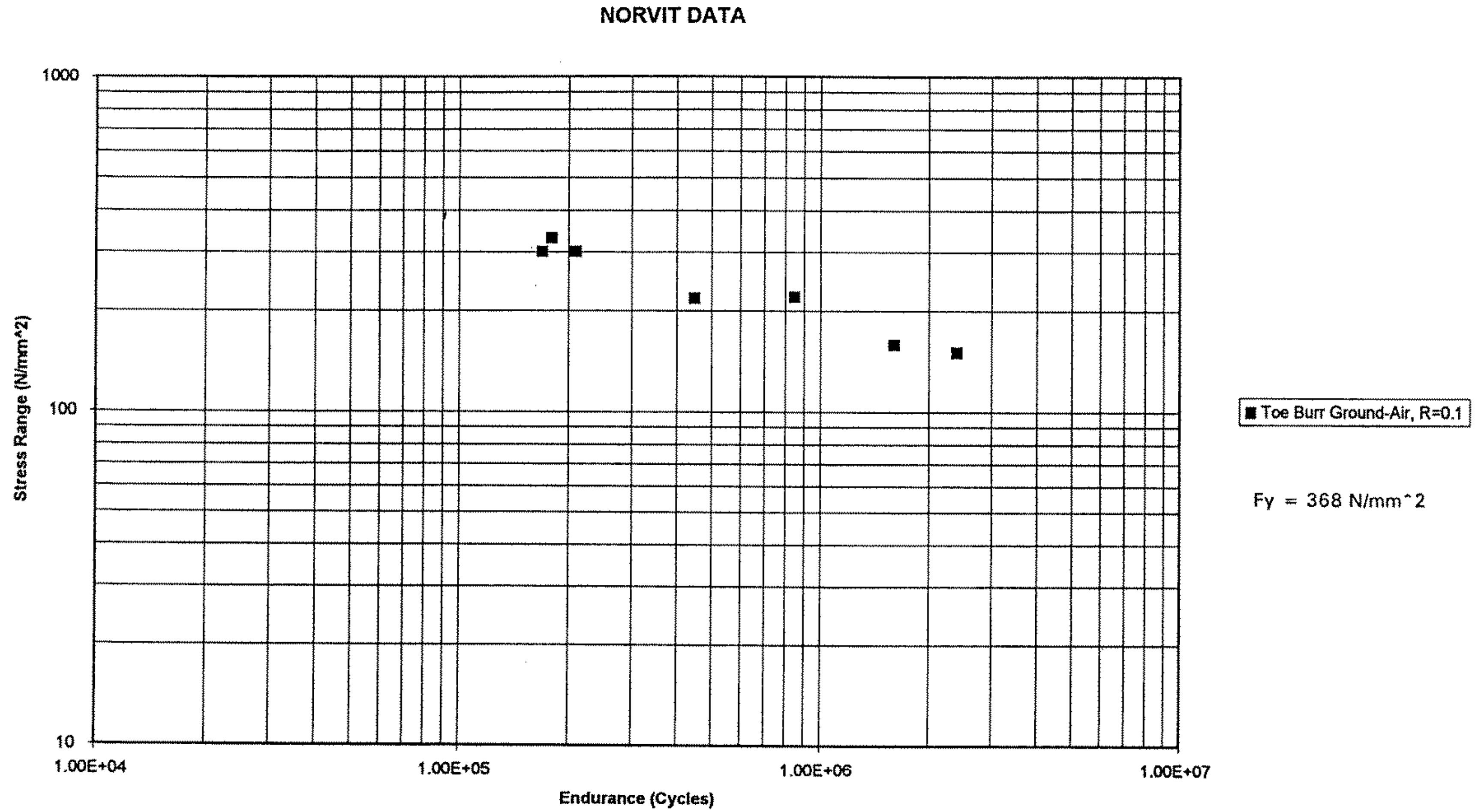

Figure 3.2.7: Data derived from Norwegian work

IV 3.2.4 Engineering Studies

This section is concerned with the analysis of the database describing the fatigue life improvement of toe ground welds. It shows how the experimental data was analysed and how the design guidance was derived.

After examining the available data, three data groups corresponding to Databases 3.2.1 to 3.2.3 were identified as follows:

- As welded samples. A total of 46 results were identified.

- Disc Ground samples. The toe has been ground using disc techniques. A total of 34 samples were identified.

- Burr ground samples. The grinding has been undertaken using a rotary grinding tool. A total of 65 samples were identified, of which 10 related to fully burr ground samples.

Prior to detailed analysis, the data was adjusted to normalise to a plate thickness of 38mm. The correction applied was according to the following equation:

$$S = S_m \left[\frac{t_m}{t_o} \right]^{0.25} \qquad \text{.... 3.2.1}$$

where

t_0 is the reference thickness (38 mm)
t_m is the actual plate thickness in mm
S_m is the applied stress range in N/mm^2
S is the adjusted stress range in N/mm^2.

The above data groups, after excluding runners and where premature failures occurred at, for example, specimen grip points, are illustrated in Figures 3.2.8 to 3.2.10 respectively. A number of S-N lines have been superimposed on these figures and these have also been adjusted to apply to a 38mm thickness where appropriate. The following comments and inferences can be made with respect to the figures:

- Except for the high yield steel, the as welded data in Figure 3.2.8 give a design line (mean minus two standard deviations) that concurs with the HSE F-curve (adjusted for 38mm thickness). This indicates that the as welded data, and by inference the ground data, from the source references are valid.

- The as welded high yield data do not conform to the same S-N endurance pattern as the other samples. Standard practice for fatigue analysis is to make no distinction between the fatigue performance of plate materials falling in the range of BS 4360 steel grades 40 to 55; the high yield steel is well outside this range.

- The disc ground data are compared with the HSE F-curve and the curve with a 2.2 enhancement of life following HSE recommendations in Figure 3.2.9. It can be observed that many data fall beneath the latter curve, indicating that the 2.2 enhancement of life cannot be taken for disc grinding.

- The same two curves are compared with the burr ground data in Figure 3.2.10. Here the 2.2 enhancement in life appears to be vindicated.

- Figure 3.2.10 shows that 2.2 times the F-curve is also applicable for the burr ground high yield data. Considering the low results for the as welded high yield data in Figure 3.2.8, it may be concluded that burr grinding is a most effective technique for high yield steels.

Before finally accepting the HSE recommendation of a 2.2 life enhancement, two other aspects need to be discussed:

- There is an apparent trend in Figure 3.2.10 of the data becoming ever closer to the enhanced curve at the high stress/low cycle end of the spectrum, ie. the data fall about a mean line of different slope to the F-curve. This observation is supported by the results of remedial ground joints discussed in Section IV 3.3 which can be expected to give similar results to the present data for very small groove depths. For typical offshore platforms where fatigue may become a problem, the great majority of damage is associated with the low stress/high cycle end of the spectrum. It may therefore be concluded that even if data should fall below the enhanced curve for very high stresses (outside the existing range of data), then this should prove of no consequence.

- All data within this Section IV 3.2 relate to plate samples and therefore its applicability to tubular joints needs to be considered. Again, a useful comparison can be made to the remedial ground joints in Section IV 3.3. This comparison, particularly of Figure 3.2.10 with the remedial ground data in Figure 3.3.8, indicates that the observations for plate specimens will hold for tubular joint specimens.

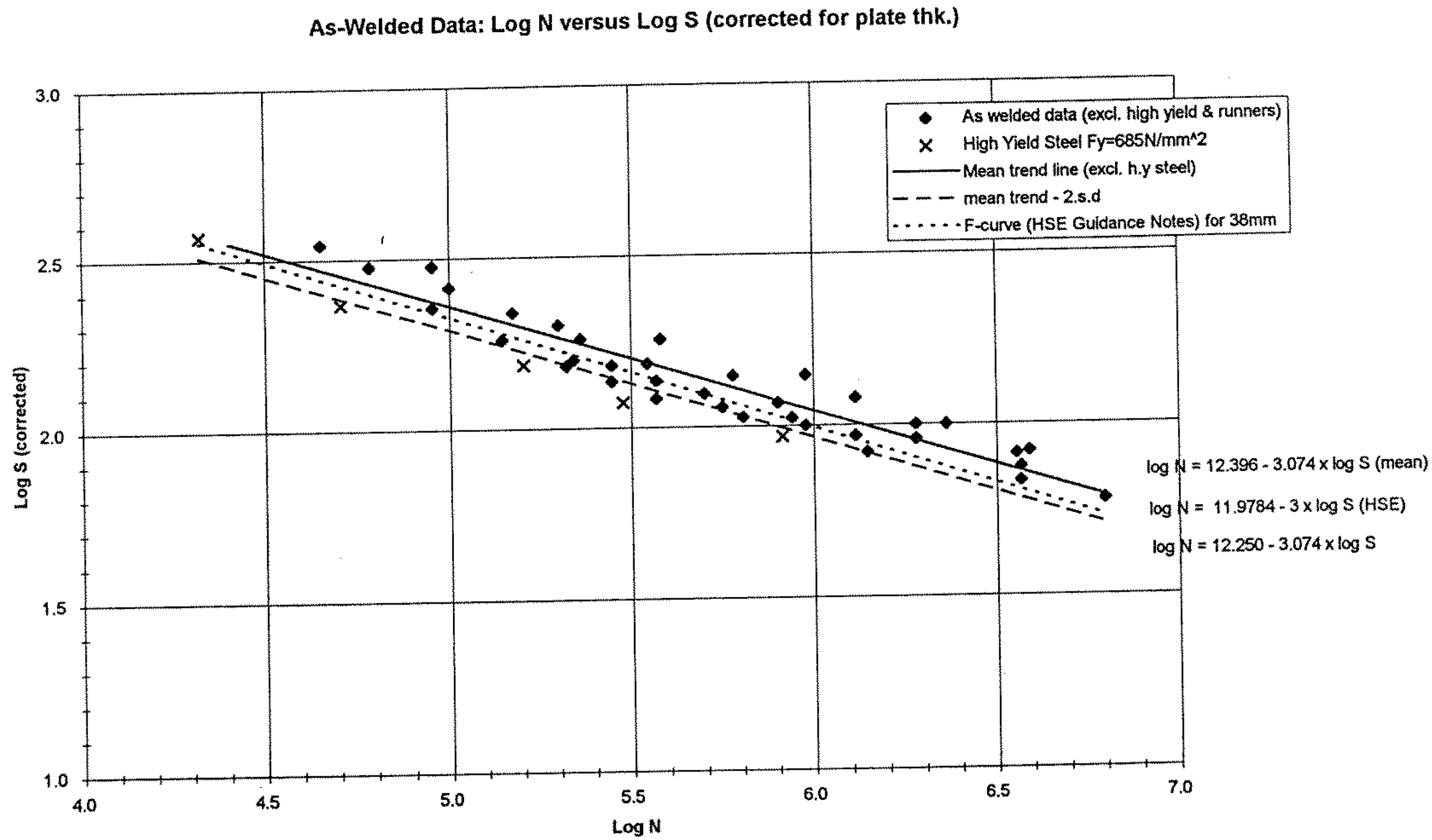

Figure 3.2.8: As welded samples

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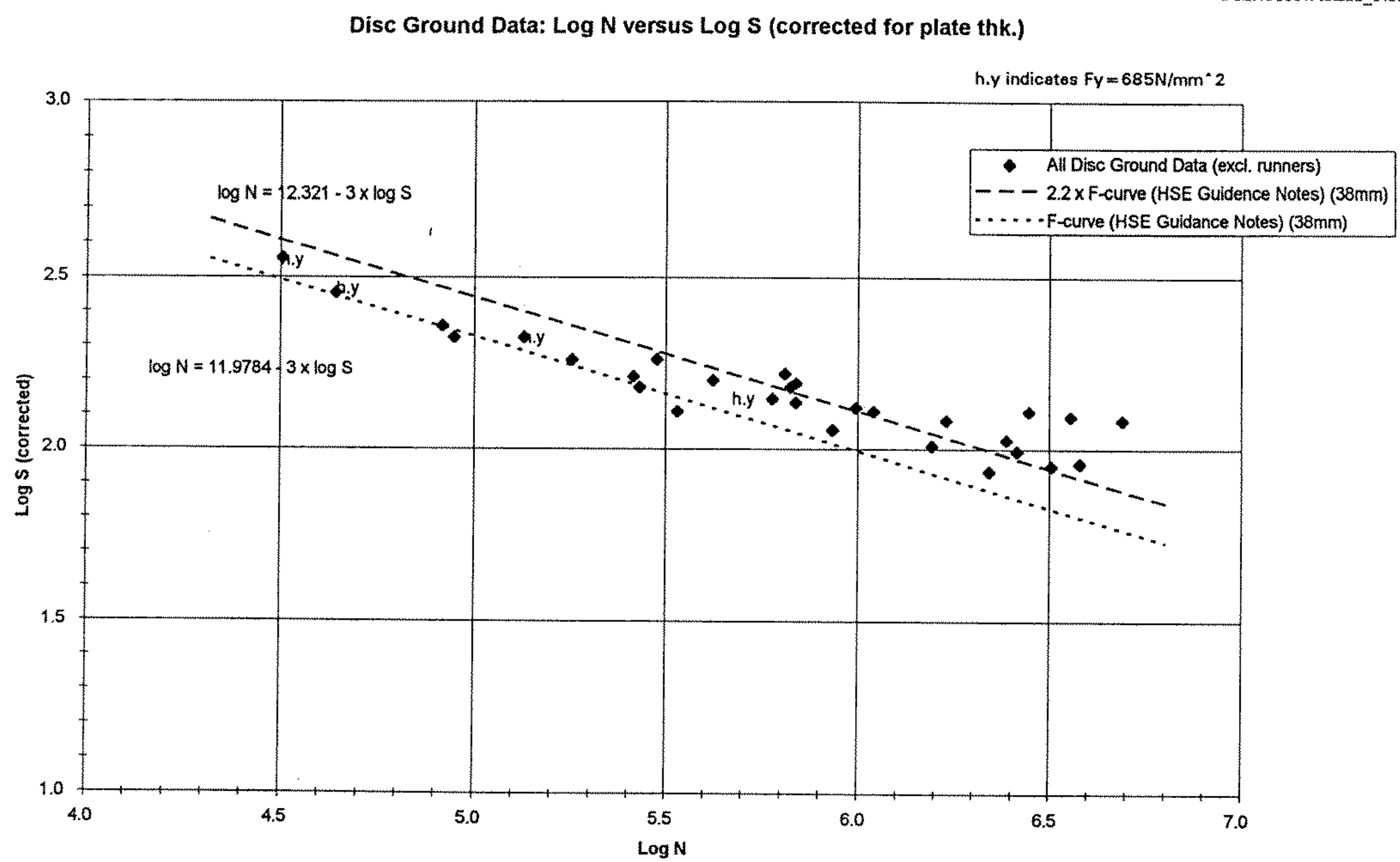

Figure 3.2.9: Disc ground samples

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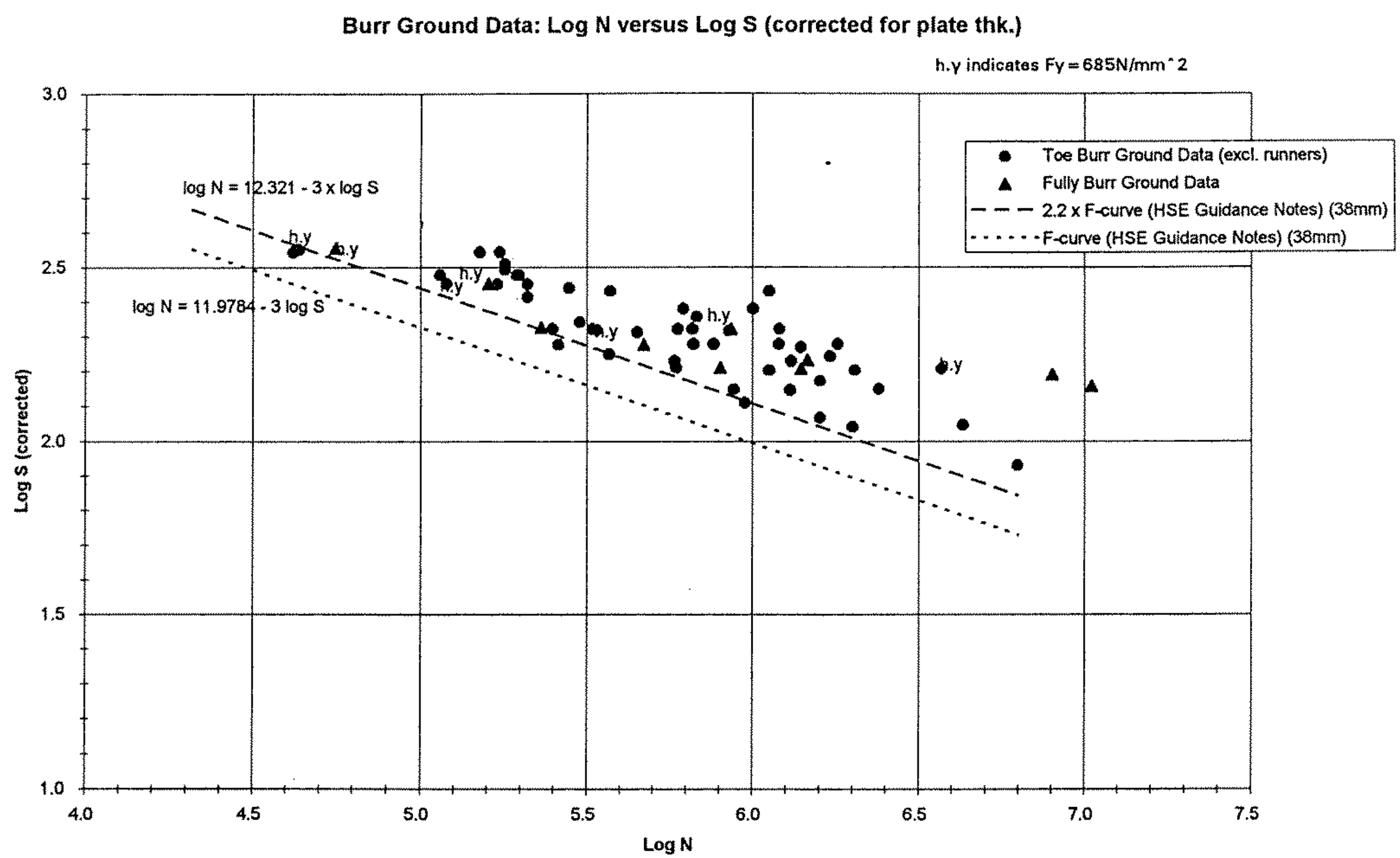

Figure 3.2.10: Burr ground samples

IV 3.2.5 Recommendations / Summary

The HSE guidance of realising an improvement on fatigue life by a factor of 2.2 following toe grinding is largely vindicated but not when grinding has been carried out by a disc. The guidance with respect to grinding to a depth of 0.5mm below any visible undercut should be observed.

Grinding of joints involving high strength steel may be particularly beneficial.

IV 3.3 REMEDIAL GRINDING OF CRACKS

Remedial grinding of joints differs from toe grinding in that the depth of the cut, critical to the definition of either process, is greater than 2mm or 5% of the thickness of the parent plate. As a consequence of deep grinding, the joint may be weakened and needs to be re-assessed for both fatigue and extreme event loadings. The maximum allowable depth of cut cannot be known *a priori* and depends on the loading and design of the joint. However, it is usual to set a limit on the groove depth of 2/3rd of the plate thickness.

IV 3.3.1 Existing Guidance

The UK Department of Energy published design data relating to the use of a local SCF factor, K_g to account for the influence of a local grind repair. The design equation is to be used for tubular joints.

The American Welding Society has also undertaken an examination of the effect of localised grooves for T butt and cross joints in plate samples.

For Tubular Joints where x/t > 0.10, the DoE equation is :

$$K_g = 1.92 + 2.7\frac{x}{t} \qquad \text{.... 3.3.1}$$

The AWS equation is to be used for flat plate applications and has no known validity range:

$$K_g = 1.64 + 5.63\frac{x}{t} \qquad \text{.... 3.3.2}$$

IV 3.3.2 Review of Existing Work

Remedial grinding has been examined by Veritec[3.9,3.10] in Norway using full scale joints. Similar work has been undertaken at the Welding Institute in the UK (TWI) under the aegis of a joint industry project [3.11]. Both the UK and the Norwegian projects were of a similar nature, and were carried out at much the same time (1987-89).

The overall scheme in both cases was to fabricate a full scale joints, produce a deep ground trench, and to test the resulting joint under fatigue loading conditions. In both cases, investigations were made into the local SCF or K_g effect.

The TWI tests looked only at axial load in the brace, whereas the Norwegian study considered both axial and brace out-of-plane moments.

IV 3.3.3 Database

An evaluation of technical test data relevant to the calculation of the effects of remedial grinding has been made. The detailed findings are reported in Section IV 3.3.4 below.

Screening criteria are as follows:

- Tests must be performed on low carbon steel, broadly conforming to BS 4360 grade 43, grade 50 or grade 55.
- Tests must have been performed since 1975.
- Tests must be on tubular steel joints, using weld details currently in use for jacket construction.
- Where data relates to testing some samples in water, then a description of the test fluid must be provided.

The data is presented in Database 3.3.1. All data relates to the grinding of 16mm thick tubulars.

IV 3.3.4 Engineering Studies

Engineering data presented in the two remedial repair studies performed by TWI[3.11] and Veritec[3.9] is detailed and specific to the testing undertaken and is of direct relevance to many design problems. The following general comments apply to the results of the two studies undertaken by TWI and Veritec.

- When a joint is repaired, the removal of the crack is in itself a major development which quickly modifies the resistance of the surrounding material to further cracking.
- On repair, the fatigue life of the cracked joint is basically returned to the as built status, and new cracks will require an initiation period.
- The ground repair should be excavated so as to remove all traces of the crack, and should be finished with a rotary burr grinding tool.

The data presented in Database 3.3.1 has been examined to establish the basis for design guidance. The local SCF factor has been investigated, and possible S-N curves have been evaluated.

Cutting out a deep crack leaves a groove behind. The groove will lead to an amplification of local stress levels. In assessing the fatigue performance of the repaired joint, account has to be taken of this application. Representing the pseudo-elastic stress as S, and the nominal brace acting stress as S_{nom} then:

$$S = S_{nom} K_t K_g \qquad \text{....3.3.3}$$

where

K_t is the SCF associated with the node geometry and
K_g is the SCF multiplier associated with the remedial repair groove

The stresses so calculated would be used with an associated S-N curve to establish the fatigue damage. From a designer's point of view, he would need guidance on K_t, K_g and the associated S-N curve. K_t, the SCF due to node geometry, can be established from existing parametric SCF equations. It would be necessary therefore to formulate guidance on K_g and the associated S-N curve. It should be recognised that these two aspects are interrelated and a different K_g formulation will be associated with a different S-N curve.

It is shown in the following subsections, however, that the approach is not entirely reliable. It is concluded that a very much simpler approach, similar to that for toe-ground joints, is applicable.

IV 3.3.4.1 Local groove SCF

Figure 3.3.1 shows the how the local groove SCF multiplier, K_g, varies with groove depth. The mean of the data is plotted and compared to the DoE design curve for tubulars. The DoE curve described by Equation 3.3.1 is confirmed as an upper bound solution to the test data. The mean curve is given by:

$$K_g = 1.80 + 1.27 \frac{x}{t} \qquad \text{. . . . 3.3.4}$$

IV 3.3.4.2 S-N data

The data points relating to the T-3 series of tests in Database 3.3.1 are for brace grinding. Detailed analyses of these data show no basic difference to the chord grinding data and hence both sets are treated together below.

Figure 3.3.2 shows the measured S-N data for both crack initiation and failure (not including runners). Naturally, the designer will normally have to estimate the stress S as the product of the node SCF (from parametric equations) and a

K_g factor. Here, the introduction of a parametrically defined SCF is avoided as this introduces inaccuracies between predicted SCFs and measured SCFs which is a problem common to all joints, and not just ground joints. Rather the predicted stress S is taken as the measured as welded value (ie. the hot spot stress) times a K_g factor. Two K_g formulations, corresponding to Equations 3.3.1 and 3.3.4, have been used, as shown in Figure 3.3.3. As expected, the DoE K_g provides an upper bound.

Taking the data based on the mean estimate of K_g first, the mean S-N curve and design S-N curve (mean minus 2 standard deviations) are plotted on Figure 3.3.4. The design curve equation may alternatively be given as:

$$\log N = 17.639 - 4.415 \log S \qquad .... 3.3.5$$

Superimposed on Figure 3.3.4 are four lines using the logarithm of the (as-welded) hot spot stress (log (HSS)) as the ordinate value. These are:

- the HSE T-curve for 16mm thickness
- the HSE T-curve with the life enhanced by a factor of 2.2; ie. the curve that is relevant to toe ground joints (see Section IV 3.2)
- the inferred design line for a zero depth groove (K_g = 1.8) and using the as-welded HSS for the stress
- as immediately above but with a groove depth of 70% of the wall thickness (K_g = 2.69).

A comparison of the last two mentioned lines with the first indicates the act of remedial grinding restores the fatigue strength of joint to at least to the original as-welded condition. Furthermore, the line for a zero depth groove (K_g = 1.8) may be considered to be closely equivalent to that which would be obtained for a toe ground joint. Its position, relative to the 2.2 x T-curve, only approximately matches the equivalent plot for toe ground plate data depicted in Figure 3.2.10 in the previous section.

Taking the data based on the DoE upper bound value of K_g, the corresponding data and curves are shown in Figure 3.3.5. It can be seen that the curve for a zero depth groove (K_g = 1.92) is even further removed from the 2.2 x T-curve. Further investigations reveal that the zero depth groove curve (and the design S-N curve) are dependent on the selected function for K_g.

Furthermore, sensitivity plots show that the effect of groove depth has not been captured by either (mean of DoE) K_g formulation, see Figures 3.3.6 and 3.3.7. Studies show that a constant K_g factor provides a better basis for assessing the fatigue strength of remedial ground joints. Different constant K_g factors give rise to different S-N curves (based on groove stress), but to the same inferred design line based on the as-welded hot spot stress.

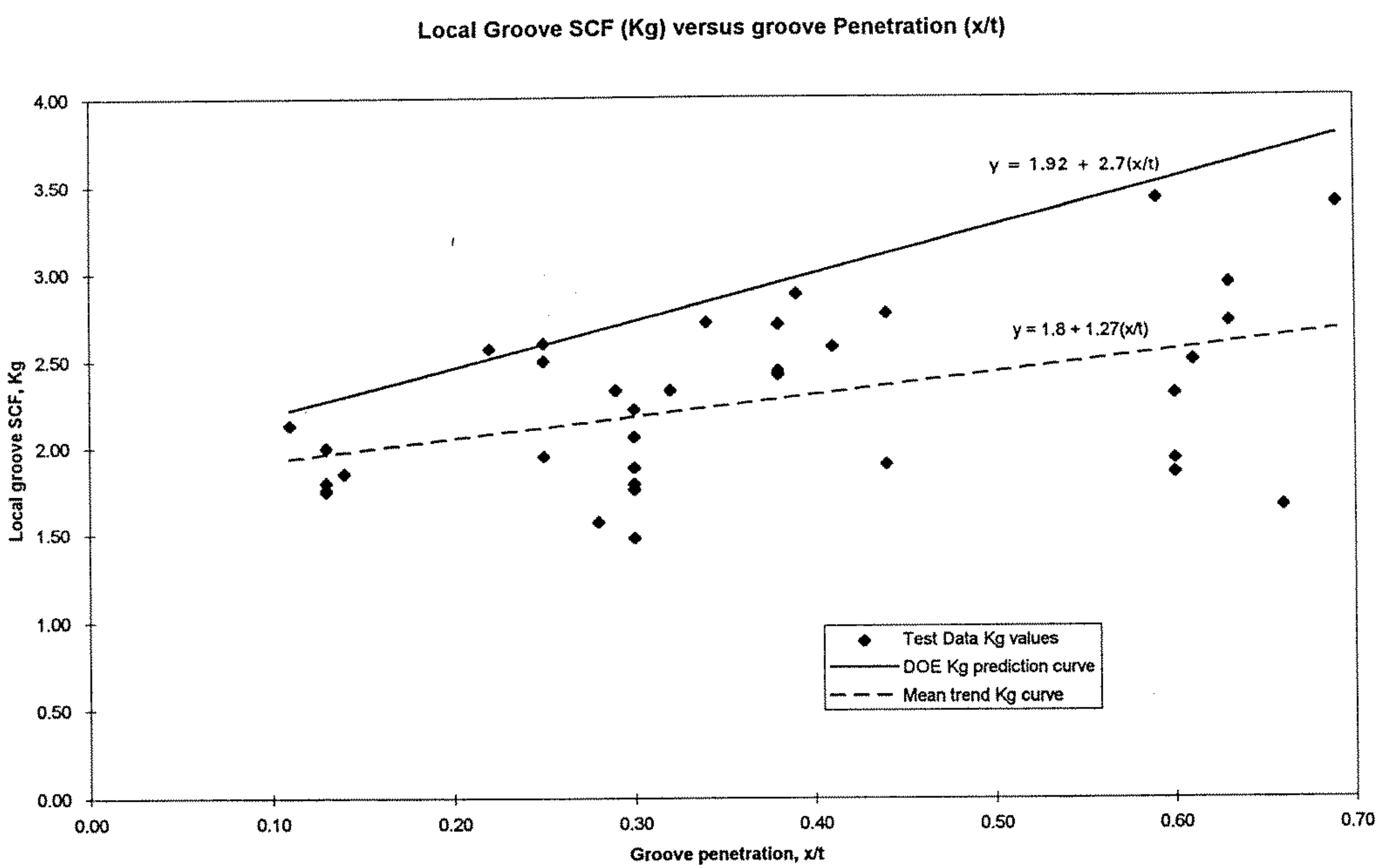

Figure 3.3.1: Analysis of SCF results for remedial grinding

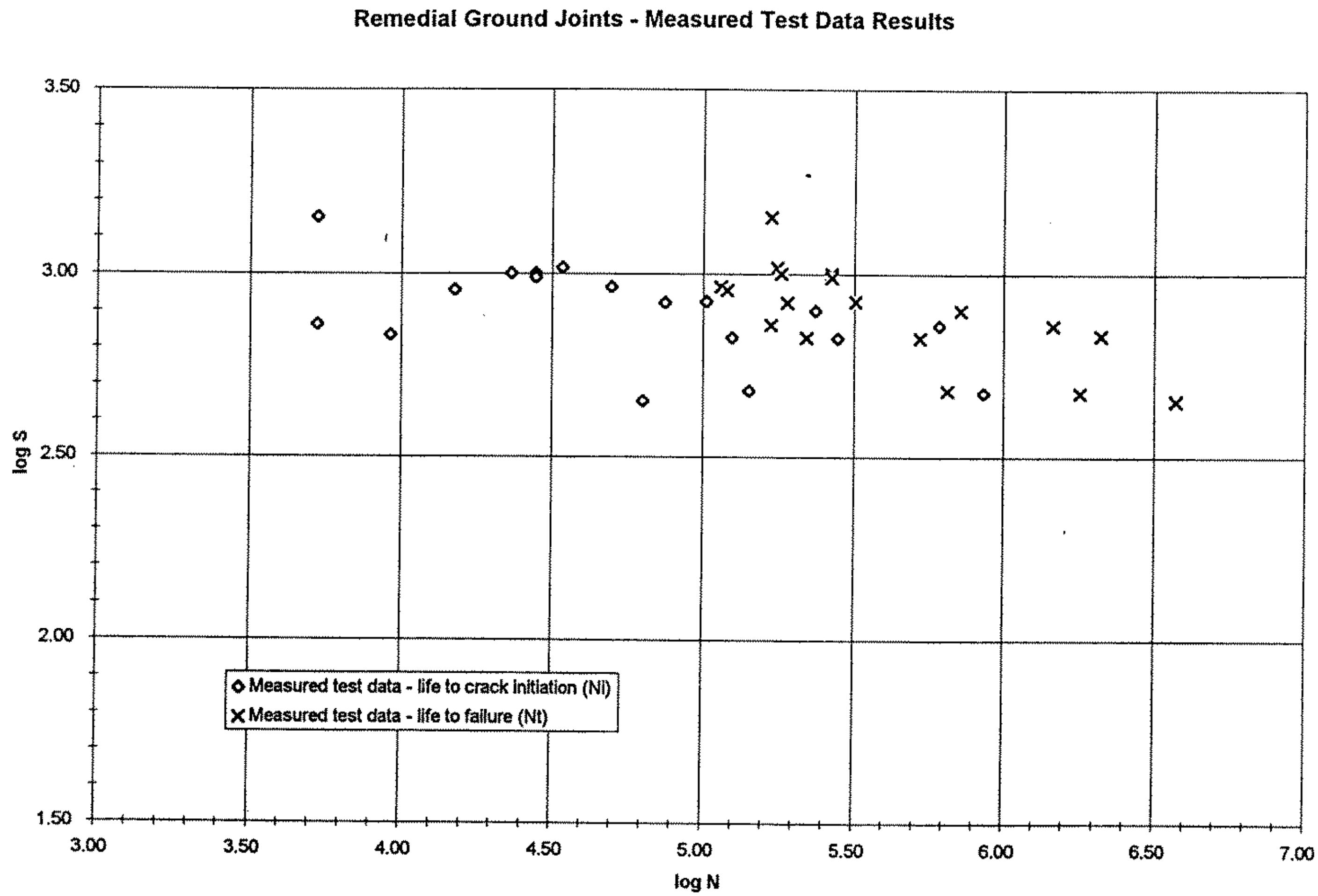

Figure 3.3.2: Measured crack initiation and failure S-N data

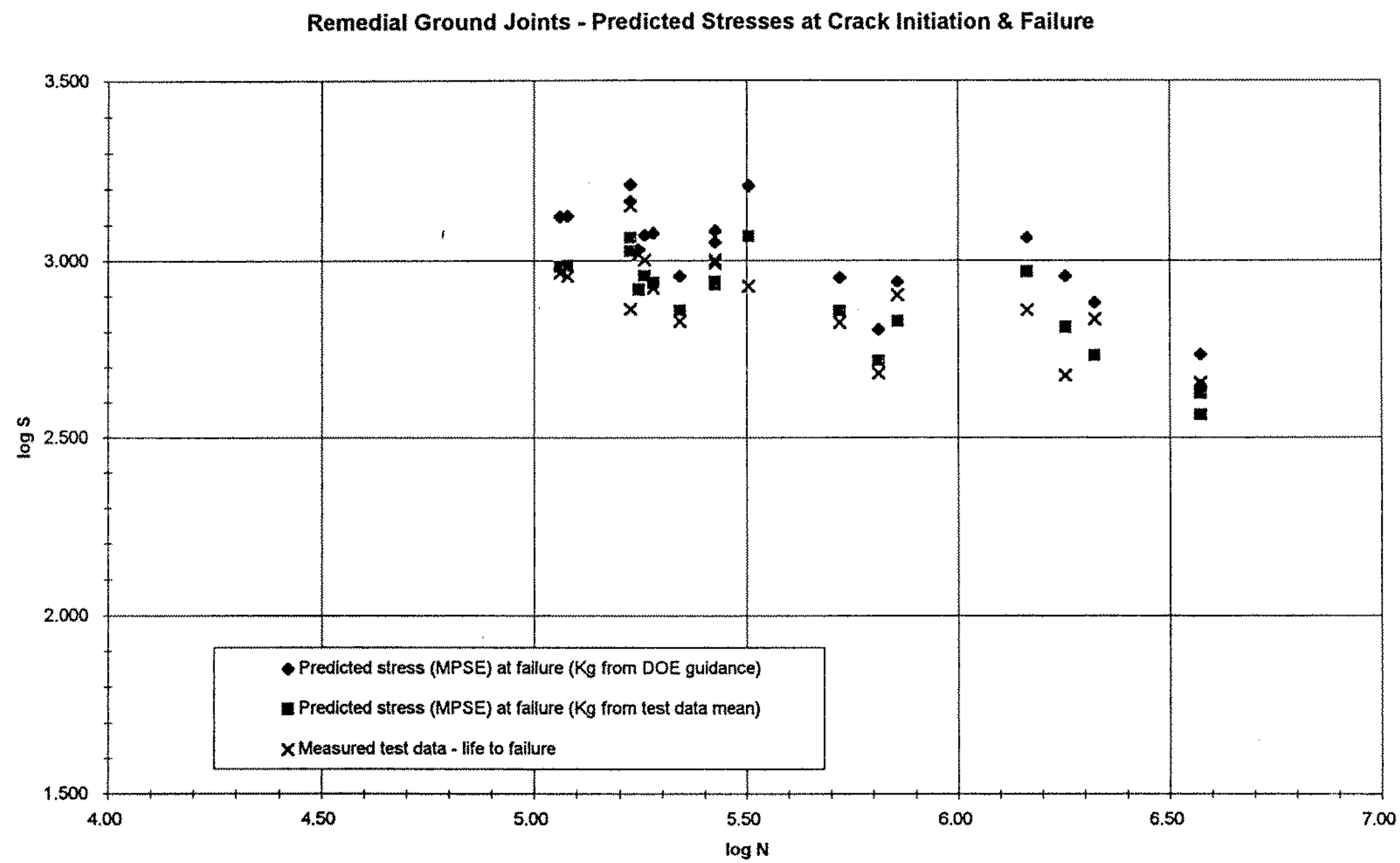

Figure 3.3.3: S-N failure data using predicted stresses

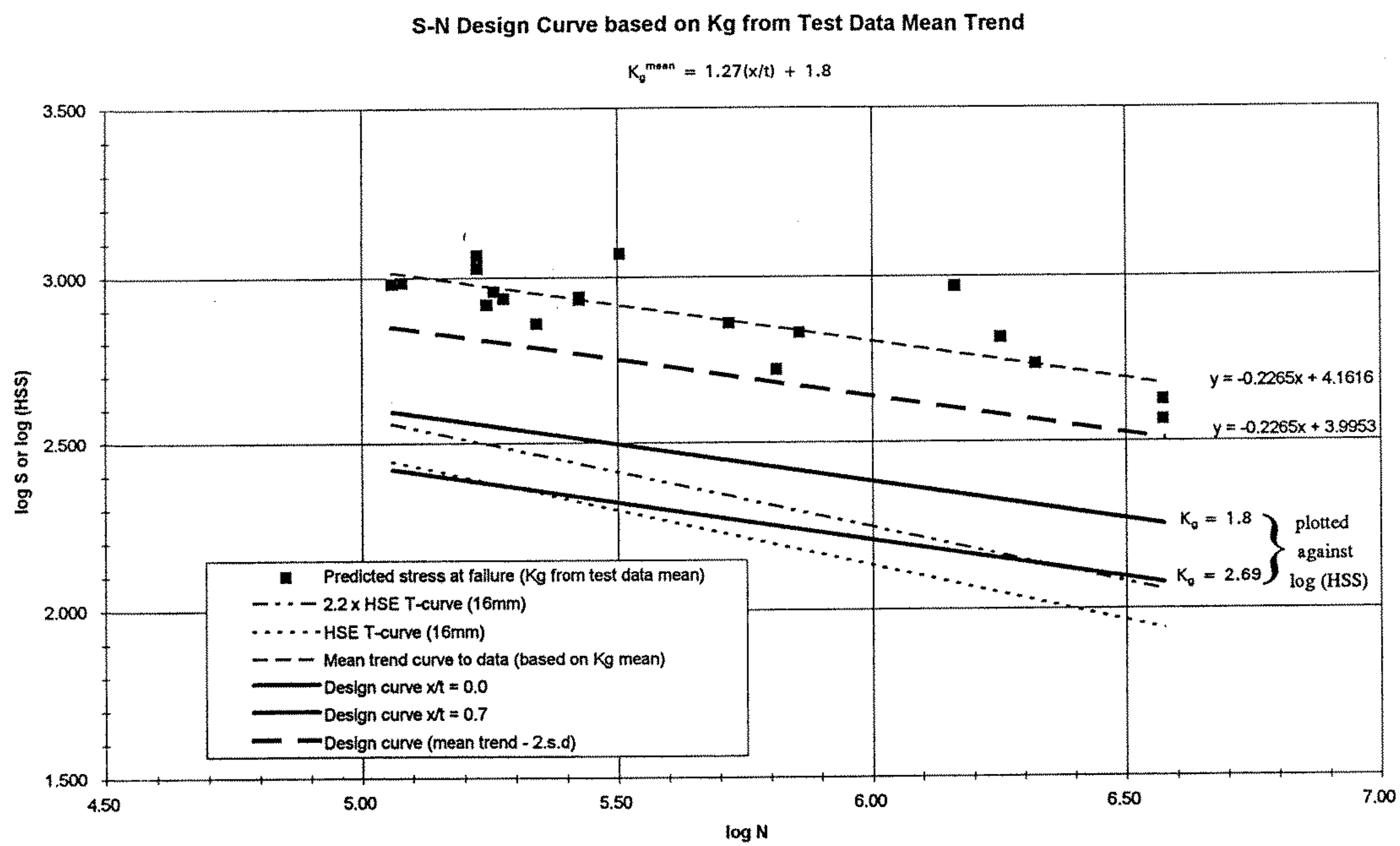

Figure 3.3.4: Mean and design S-N curves based on mean K_g

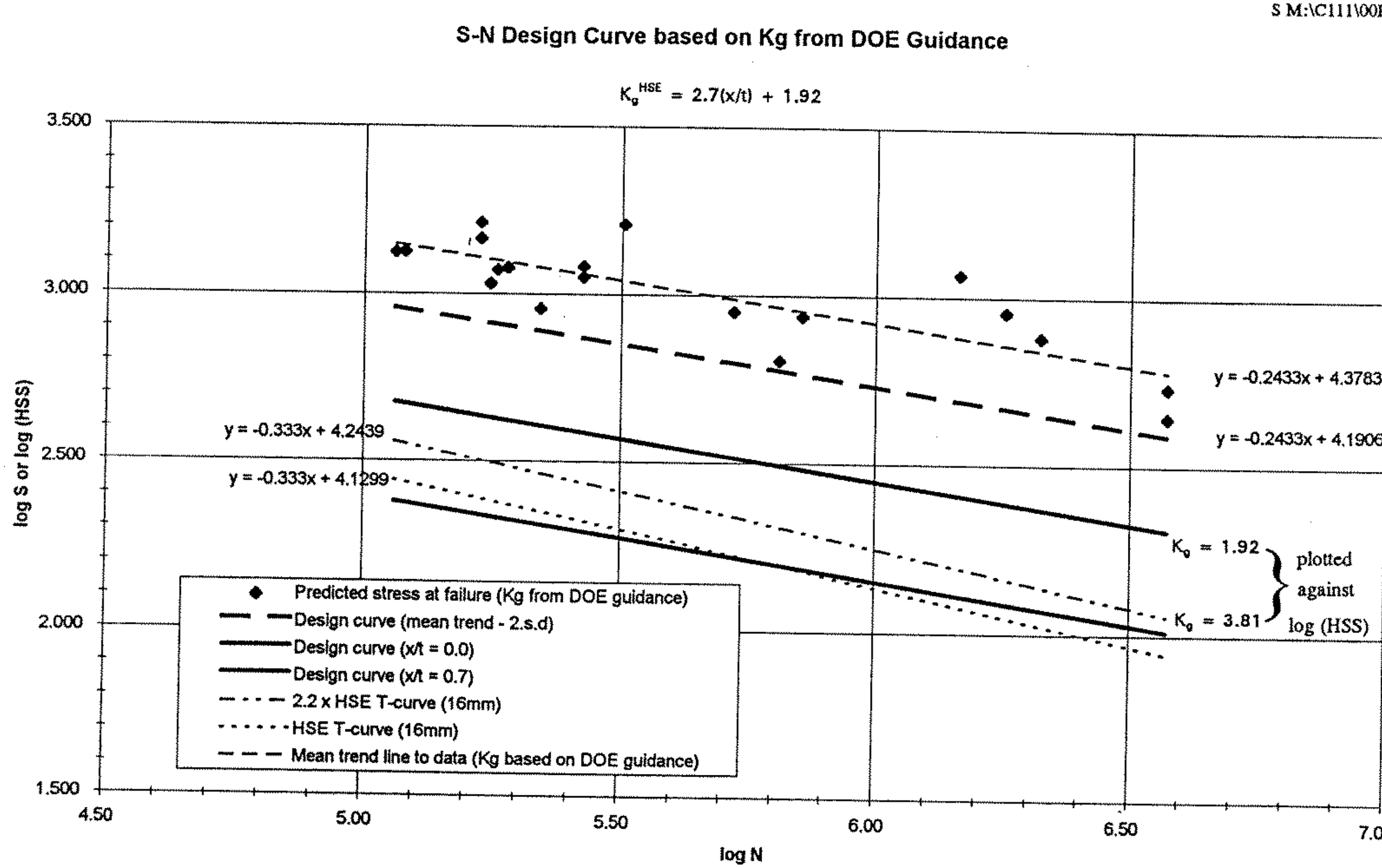

Figure 3.3.5: Mean and design S-N curves based on DoE K_g

S M:\C111\00R222_1\T3_3_4-7.XLS

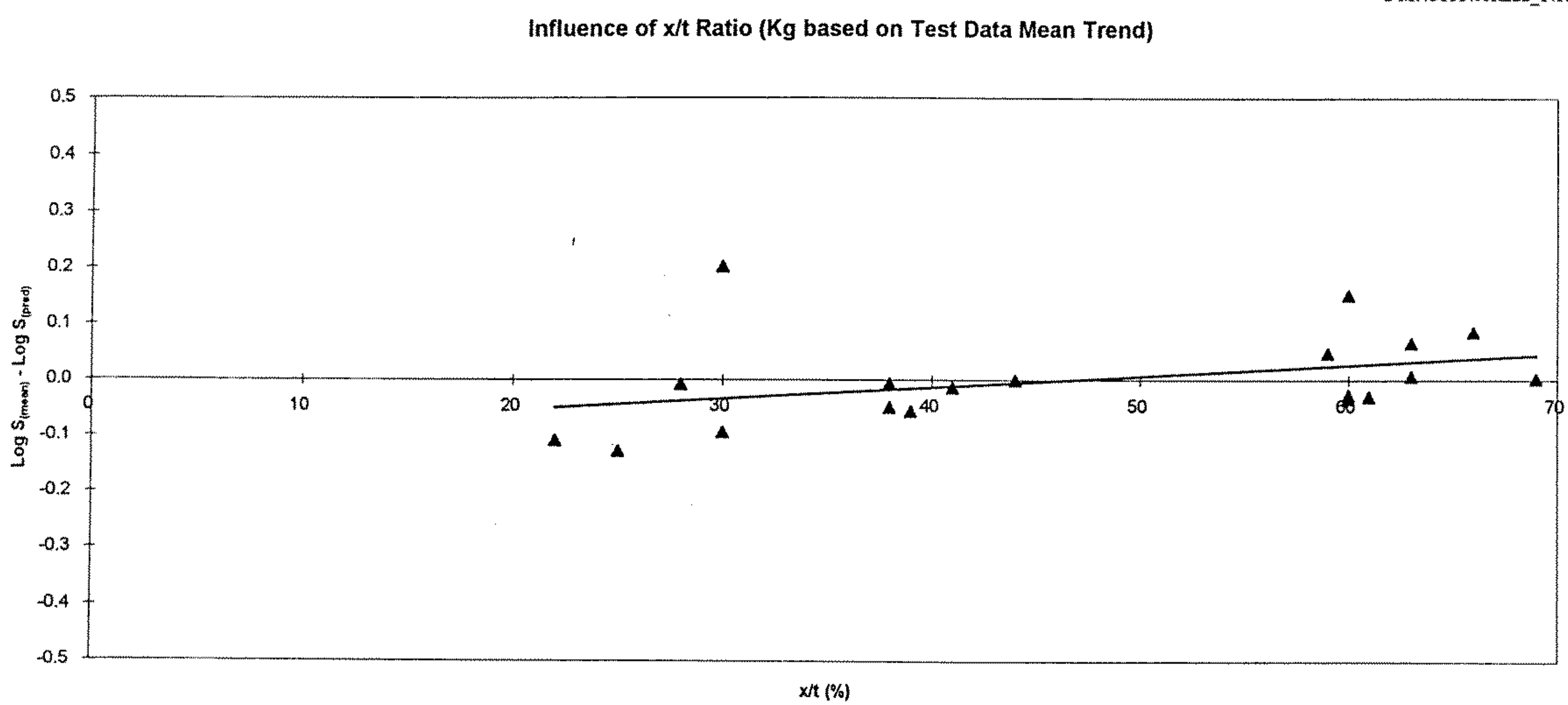

Figure 3.3.6: Sensitivity plot of groove depth, K_g based on mean of data

S M:\C111\00R222_1\T3_3_4-7.XLS

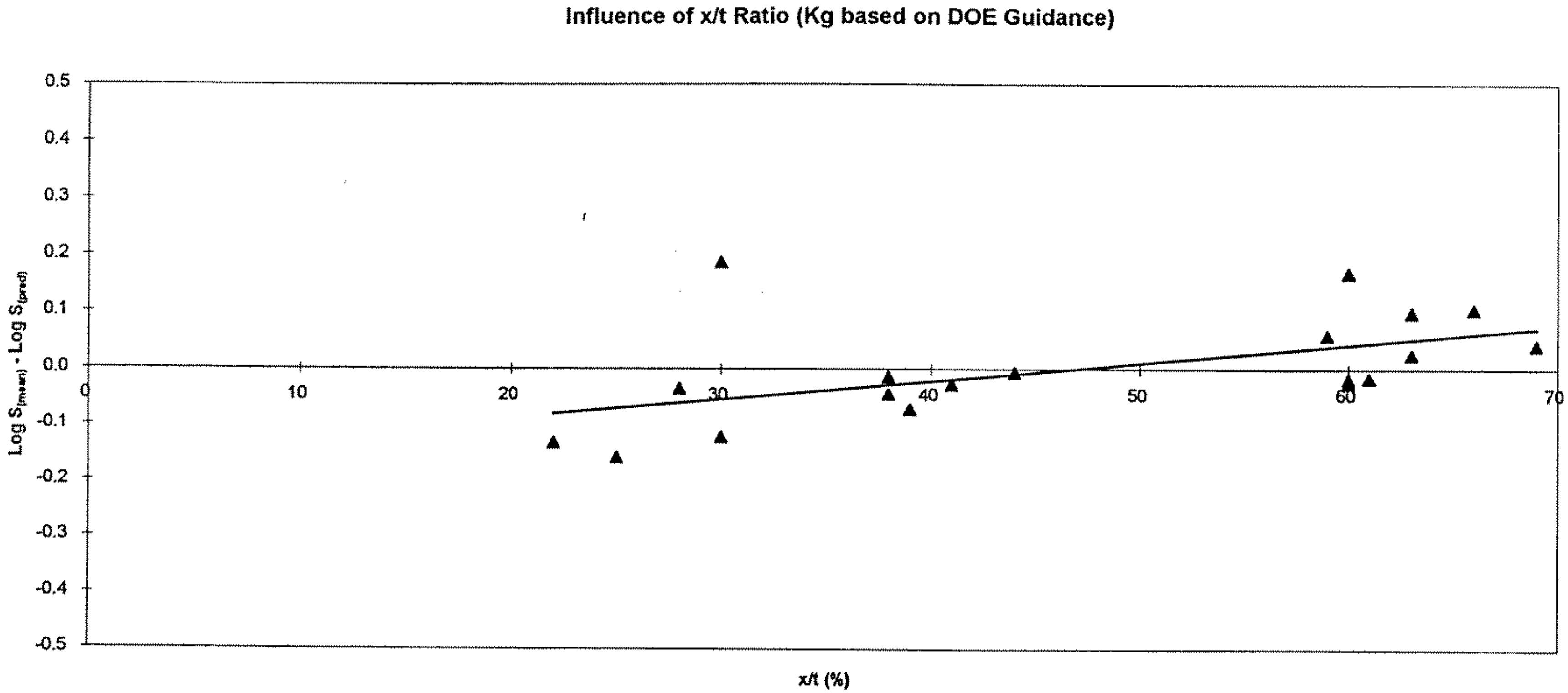

Figure 3.3.7: Sensitivity plot of groove depth, K_g based on upper bound of data

For simplicity and convenience, a K_g factor of unity may be selected such that the S-N curve is ostensibly based on the as-welded hot spot stress. The resulting mean and design S-N curves are shown in Figure 3.3.8. Note that as a result of using a constant value for K_g, in this case unity, only one design curve results and is applicable for all groove depths. It can be observed that the data with different groove depths (as indicated) are equally disposed about the mean line in Figure 3.3.8. This is emphasised in the groove depth sensitivity plot in Figure 3.3.9. The mean trend line is difficult to see on this plot as it lies on the abscissa. Although the overall mean point of all data would be expected to lie on the abscissa, the zero slope of the mean trend line is quite remarkable. To some extent the zero slope is fortuitous as removal of a single data point does lead to a finite value for the slope.

The S-N design curve in Figure 3.3.8, with the stress based on the as welded hot spot stress, is applicable to all groove depths and in particular to depths obtained by toe grinding. It is therefore instructive to compare this figure with the toe ground plot in Figure 3.2.10 in Section IV 3.2. The latter figure relates to plate data and thus relates to the F-curve and its enhancement by a factor of 2.2. Strong similarities exist between Figures 3.2.10 and 3.3.8 in that both sets of data:

- lie above the enhanced as welded S-N curves
- fall into a diverging band with increasing number of cycles
- form trend lines having different slopes (flatter) than the as welded design lines.

Taking the view that the similarities indicate the same basic behaviour of toe ground and remedial ground joints, it is therefore recommended that rather than the design S-N line of Figure 3.3.8 be taken, the enhanced T-curve (bg a factor of 2.2) should be adopted as the basis for assessing the fatigue strength of remedial ground joints. This may not be as unduly conservative as might appear in Figure 3.3.8 for the low stress/high cycle end of the spectrum, because the limited amount of remedial ground data (compared to the toe ground data of Figure 3.2.10) may not show the full spread.

Similar observations made with respect to toe ground joints at the high stress/low cycle end of the spectrum apply here.

IV 3.3.5 Recommendations / Summary

Remedial grinding should be seen as the first line of defence in the war against underwater fatigue. The operator should be aware that remedial grinding should be promptly undertaken on discovery of a crack.

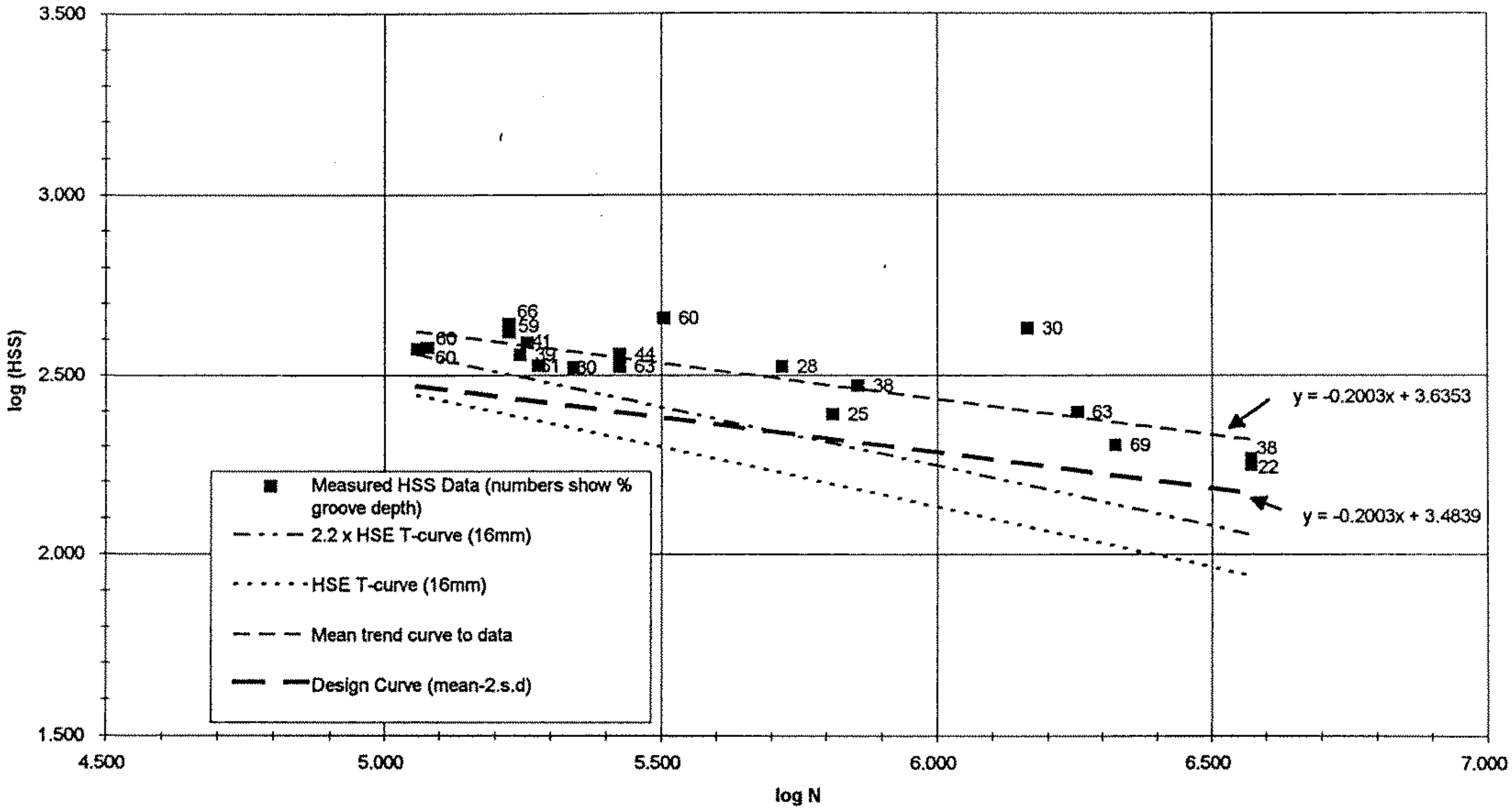

Figure 3.3.8: Mean and design S-N curves based on $K_g = 1$

S M:\C111\00R222_1\T3_3_2-9.XLS

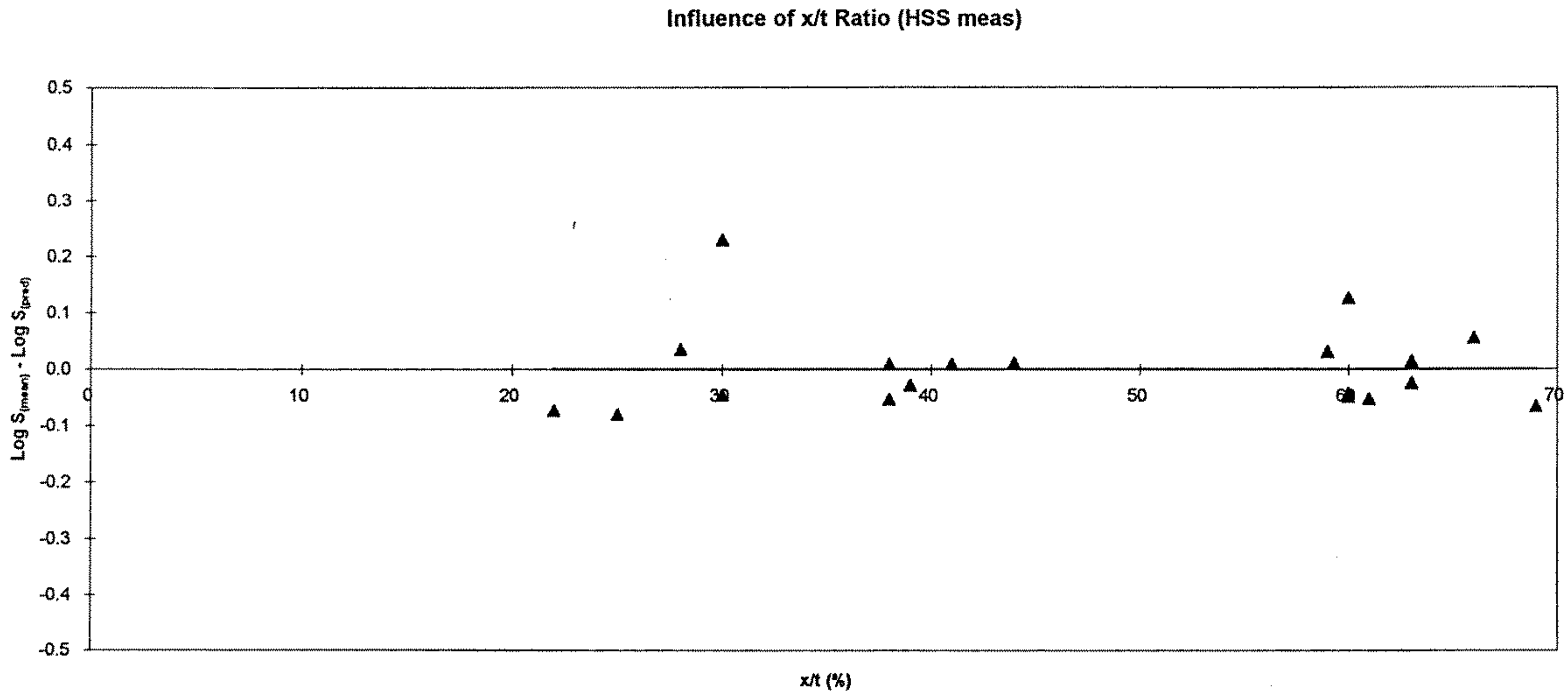

Figure 3.3.9: Sensitivity plot of groove depth, $K_g = 1$

It is recommended that the fatigue assessment of a remedial ground joint be exactly the same as that of a toe ground joint, ie. take the as welded hot spot stress (SCF derived from parametric equations) and use it is in conjunction with an enhanced HSE T-curve. The groove should be finished off, at least, with a burr grinder rather than a disc grinder.

In addition, the static strength of the ground joint should be assessed.

IV 3.4 OTHER TECHNIQUES

IV 3.4.1 General

Other weld improvement techniques which are within the range of use of current design techniques are discussed in this section of the report. The concern is not with 'high technology' approaches, but with the application of known techniques, perhaps specialised in another sphere of engineering technology, which may be beneficially used in the repair of offshore structures.

Some of the techniques discussed are already in use for offshore repair work, as is the case with remedial hole drilling. The intention of including such alternative repair options is to portray the test data for such options in a even light so as to enable potential users to make a reasonable judgement of their value.

IV 3.4.2 Peening

Peening is the purposeful hammering of the surface of a weld to remove the built in stress associated with welding. For the treatment to be effective, it should be applied when the metals joined have been allowed to cool down.

Peening is a technique which has been developed particularly to enhance the fabricated strength of various metal components (which may not have been welded), and is applied to a variety of products using a number of different processes, such as:

- Hammer peening - originally, to use a ball headed hammer to strike the metal with the intention of leaving a permanent concave mark on the metal face. Now a pneumatic tool is used for production work.

- Needle peening - an alternative tooling approach to hammer peening whereby a multiple set of rods are used to transfer the compressive blow.

- Shot peening - normally used for component peening - the item is placed in a chamber and bombarded with high velocity, small, hard spherical particles. Process similar, but on a larger scale to the discharge from a shot-gun.

For welded joints, only hammer and needle peening need be identified as practical options. Within the classification, it is implicit that the semi automatic processes are to be used.

Knight[3.2] and Booth[3.3] give full details of hammer and shot peening techniques which were applied to plate specimens. The test pieces were subjected to fatigue testing in a simulated underwater environment. It was not possible to obtain a failure in the peened welds, with the samples repeatedly failing at the load gripper locations. It was concluded that the hammer and shot peening were able to improve the weld to an extent that it had a higher fatigue endurance than the parent material.

The relevant data from these two sources are given in Database 3.4.1 and are plotted in Figure 3.4.1. The promise that the technique holds may be appreciated when this figure is compared to the toe ground plate data in Figure 3.2.10. A remarkably improved fatigue life is clearly demonstrated.

Booth cautions against over-reacting to the result. The behaviour of tubular steel joint welds will not be the same as that recorded in simple butt-welded plates. If a compressive stress is purposefully locked into a weld by peening, then it will possibly be the subject of shakedown - a process by which residual stresses can be released by flexing the steel element, usually to a near yield condition.

It is because of the difficulty with shakedown that many researchers are unable to make firm predictions about how welded tubular steel joints will behave in practice if they are subjected to peening before immersion in seawater.

IV 3.4.3 Dressing

Weld dressing is a post welding operation that seeks to improve explicit areas of the weld by using a welding process that is inherently more expensive than that which has been used to lay down the greater volume of the weld.

Although there are a number of different options which may be identified, the two major classes of this type of weld improvement technique are:

- TIG weld dressing
- plasma arc weld dressing

Bignonnet[3.12] discusses TIG dressing along with other weld improvement techniques and shows that the improvement in air is much better than that achieved underwater.

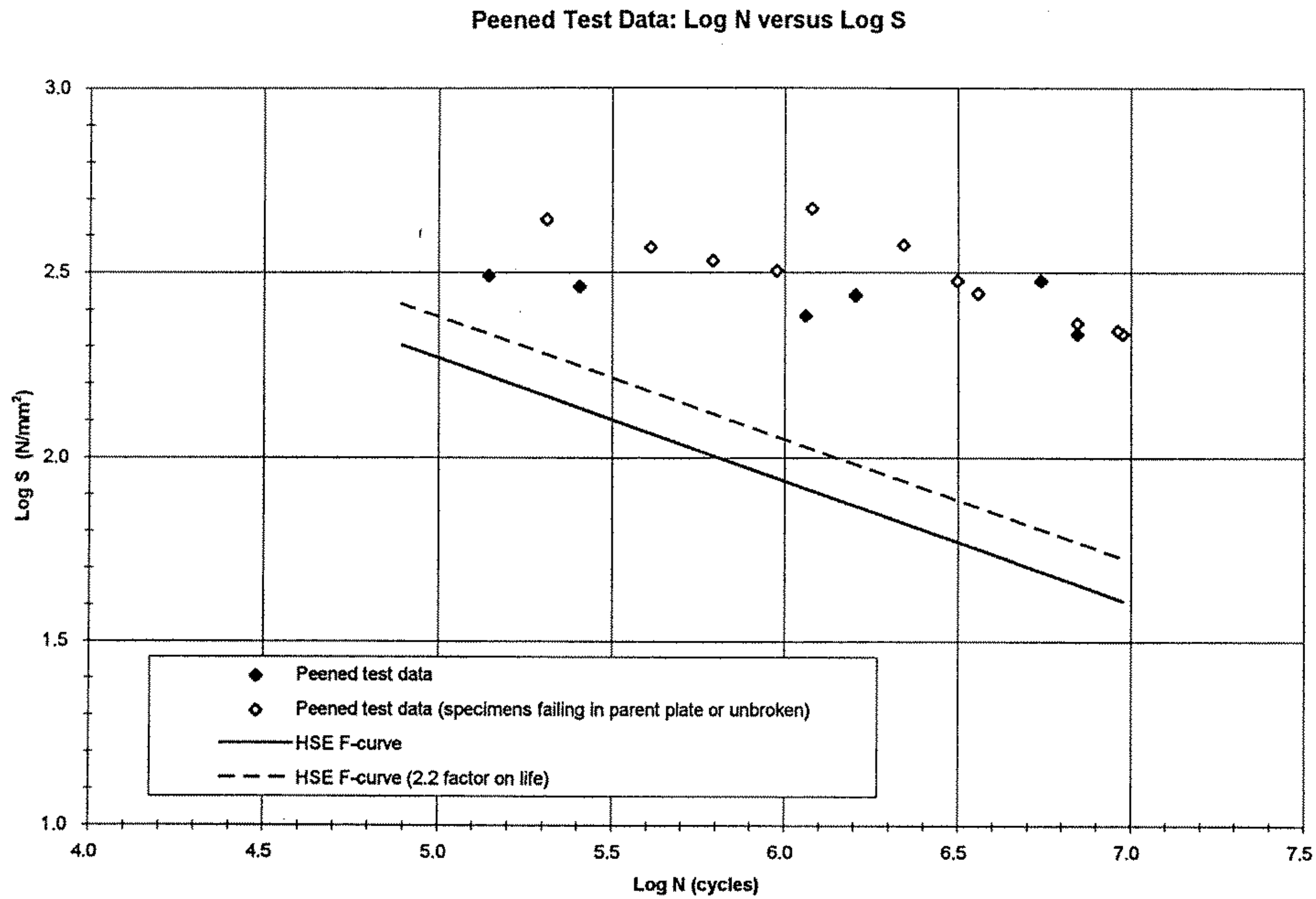

Figure 3.4.1: S-N data for hammer peened plate specimens

Haagensen[3.13] reports on work undertaken at SINTEF in Norway on the use of TIG dressing for weld improvement. The key activity was to re-melt the weld surface as laid down by another process and to leave an improved surface (both in terms of defects and profile) on the dressed weld. He reports a drop in measured local SCF factors which account for the improvement of fatigue lives in air. The case for use of the technique for submerged parts of the structure is not so clear with embrittlement and differential corrosion not fully explained.

IV 3.4.4 Hole Drilling

Hole drilling is a repair technique which has been used as a crack arrestor in a variety of materials, and is perhaps best known for repairs in plate glass. Although the drilling of arrestor holes is an understandable action it is an action which is unlikely to be of any advantage unless it is allied with the insertion and tightening of a HSFG bolt[3.14].

Tubby[3.11] relates experimental experiences with arrestor hole drilling and concludes that there is no particular advantage associated with their use in structural tubular joints.

IV 3.4.5 Improved Weld Profile

Little data is directly available in the literature for improved weld profile effects upon fatigue life.

Improved weld profile techniques have been used in the design of offshore tubular steel platforms for the last 15 years. On many projects, standard welding detail drawings show how the weld is to be profiled around the joint for both single and double sided weld preparations.

Mullen[3.6] reports improvements in fatigue life of joints which have been purposefully reinforced.

The effect of weld profile is also discussed by Bignonnet[3.12] and Tomkins et al[3.15]. Bignonnet describes how the 'fluidity' of welding metal can be beneficially used to produce a smooth profile in the finished weld. Tomkins comments on how welding process can influence the shape and appearance of the weld toe. He appraises the differences in weld profiles associated with different fabricators. However, Tomkins finds that :

> *".. those joints which feature unsatisfactory weld profiles, as determined by lack of overall concavity and lack of smooth weld toe blending , do not produce significantly different fatigue endurances for nominally similar joints with satisfactory weld profiles. ..."*

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3.14 BSI "The Use of High Strength Friction Grip Bolts" BS 4604.

3.15 Tomkins et al. "Background to New Fatigue Design Guidance for Steel Welded Joints in Offshore Structures". UK DoE, HMSO, London 1984.

Test Data Ref	Plate thk. mm	Cycles N	Stress N/mm2	Log N	Corrected Log S	CP	Weld Profile	R	Steel Type	Comment
	As-Welded Samples									
3.2	12.5	1.40E+05	240	5.146	2.259	Air	Welded	0	4360-43a	
3.2	12.5	2.20E+05	208	5.342	2.197	Air	Welded	0	4360-43a	
3.2	12.5	3.50E+05	202	5.544	2.185	Air	Welded	0	4360-43a	
3.2	12.5	5.00E+05	165	5.699	2.097	Air	Welded	0	4360-43a	
3.2	12.5	8.00E+05	154	5.903	2.067	Air	Welded	0	4360-43a	
3.2	12.5	8.70E+05	139	5.940	2.022	Air	Welded	0	4360-43a	
3.2	12.5	1.30E+06	123	6.114	1.969	Air	Welded	0	4360-43a	
3.2	12.5	3.90E+06	110	6.591	1.921	Air	Welded	0	4360-43a	
3.2	12.5	3.60E+06	108	6.556	1.913	Air	Welded	0	4360-43a	
3.2	12.5	2.10E+04	490	4.322	2.569	Air	Welded	0	Fy=685	
3.2	12.5	5.10E+04	307	4.708	2.366	Air	Welded	0	Fy=685	
3.2	12.5	1.60E+05	202	5.204	2.185	Air	Welded	0	Fy=685	
3.2	12.5	3.00E+05	156	5.477	2.072	Air	Welded	0	Fy=685	
3.2	12.5	8.20E+05	123	5.914	1.969	Air	Welded	0	Fy=685	
3.3	38	9.00E+04	300	4.954	2.477	Air	Welded	0	50 D	
3.3	38	1.50E+05	218	5.176	2.338	Air	Welded	0	50 D	
3.3	38	3.80E+05	180	5.580	2.255	Air	Welded	0	50 D	
3.3	38	6.00E+05	140	5.778	2.146	Air	Welded	0	50 D	
3.3	38	1.30E+06	120	6.114	2.079	Air	Welded	0	50 D	
3.3	38	9.50E+05	100	5.978	2.000	Air	Welded	0	50 D	
3.3	38	2.30E+06	100	6.362	2.000	Air	Welded	0	50 D	
3.3	38	7.00E+06	80	6.845	1.903	Air	Welded	0	50 D	U
3.3	38	4.50E+04	350	4.653	2.544	none	Welded	0	50 D	sea water
3.3	38	6.10E+04	300	4.785	2.477	none	Welded	0	50 D	sea water
3.3	38	1.00E+05	260	5.000	2.415	none	Welded	0	50 D	sea water
3.3	38	2.00E+05	200	5.301	2.301	none	Welded	0	50 D	sea water
3.3	38	9.50E+05	140	5.978	2.146	none	Welded	0	50 D	sea water
3.3	38	1.90E+06	100	6.279	2.000	none	Welded	0	50 D	sea water
3.3	38	3.70E+06	75	6.568	1.875	none	Welded	0	50 D	sea water
3.3	38	6.30E+06	60	6.799	1.778	none	Welded	0	50 D	sea water
3.3	12.5	9.00E+04	300	4.954	2.356	Air	Welded	0	4360-43a	
3.3	12.5	2.30E+05	240	5.362	2.259	Air	Welded	0	4360-43a	
3.3	12.5	2.80E+05	200	5.447	2.180	Air	Welded	0	4360-43a	
3.3	12.5	3.70E+05	180	5.568	2.135	Air	Welded	0	4360-43a	
3.3	12.5	5.60E+05	150	5.748	2.055	Air	Welded	0	4360-43a	
3.3	12.5	1.90E+06	120	6.279	1.958	Air	Welded	0	4360-43a	
3.3	12.5	8.50E+06	100	6.929	1.879	Air	Welded	0	4360-43a	U
3.3	12.5	9.50E+06	90	6.978	1.834	Air	Welded	0	4360-43a	U
3.3	12.5	2.10E+05	200	5.322	2.180	Air	Welded	0.5	4360-43a	
3.3	12.5	2.80E+05	180	5.447	2.135	Air	Welded	0.5	4360-43a	
3.3	12.5	3.70E+05	160	5.568	2.083	Air	Welded	0.5	4360-43a	
3.3	12.5	6.40E+05	140	5.806	2.025	Air	Welded	0.5	4360-43a	
3.3	12.5	1.00E+07	126	7.000	1.980	Air	Welded	0.5	4360-43a	U
3.3	12.5	1.40E+06	110	6.146	1.921	Air	Welded	0.5	4360-43a	
3.3	12.5	7.50E+06	100	6.875	1.879	Air	Welded	0.5	4360-43a	U
3.3	12.5	3.70E+06	90	6.568	1.834	Air	Welded	0.5	4360-43a	

NOTES:

(1) Log S values include a correction factor to account for variations in plate thickness

(2) Key to database comments:

U	Sample unbroken at the end of the fatigue test
P	Fatigue crack formed at the location of a test probe
G	Fatigue crack formed at the load arm gripper
LS	Fatigue crack formed on the low stress side of the weld
PL	Fatigue crack formed in the parent plate
R	Fatigue crack formed in the weld root

(3) Steel Types used in tests

4360 43a	BS 4360 Grade 43a low carbon weldable steel
50 D	BS 4360 Grade 50 D or equivalent as per EEMUA standard
Fy=685	A High tensile strength steel.
Fy= 368	Norwegian steel specification - Statoil type 1

Database 3.2.1: As welded plate samples

Test Data Ref	Plate thk. mm	Cycles N	Stress N/mm2	Log N	Corrected Log S	CP	Weld Profile	R	Steel Type	Comment
	Disc Ground Samples									
3.2	12.5	8.90E+04	278	4.949	2.323	Air	Disc	0	4360-43a	
3.2	12.5	1.80E+05	239	5.255	2.258	Air	Disc	0	4360-43a	
3.2	12.5	2.60E+05	214	5.415	2.210	Air	Disc	0	4360-43a	
3.2	12.5	4.20E+05	209	5.623	2.199	Air	Disc	0	4360-43a	
3.2	12.5	6.90E+05	205	5.839	2.191	Air	Disc	0	4360-43a	
3.2	12.5	6.00E+05	185	5.778	2.146	Air	Disc	0	4360-43a	
3.2	12.5	9.90E+05	174	5.996	2.120	Air	Disc	0	4360-43a	
3.2	12.5	2.80E+06	170	6.447	2.110	Air	Disc	0	4360-43a	
3.2	12.5	1.70E+06	160	6.230	2.083	Air	Disc	0	4360-43a	
3.2	12.5	9.40E+06	150	6.973	2.055	Air	Disc	0	4360-43a	U
3.2	12.5	1.20E+07	140	7.079	2.025	Air	Disc	0	4360-43a	U
3.2	12.5	3.20E+04	473	4.505	2.554	Air	Disc	0	Fy=685	
3.2	12.5	4.40E+04	375	4.643	2.453	Air	Disc	0	Fy=685	
3.2	12.5	1.35E+05	278	5.130	2.323	Air	Disc	0	Fy=685	
3.2	12.5	6.45E+05	218	5.810	2.218	Air	Disc	0	Fy=685	
3.3	12.5	8.30E+04	300	4.919	2.356	Air	Disc	0	4360-43a	
3.3	12.5	3.00E+05	240	5.477	2.259	Air	Disc	0	4360-43a	
3.3	12.5	6.70E+05	200	5.826	2.180	Air	Disc	0	4360-43a	
3.3	12.5	6.90E+05	180	5.839	2.135	Air	Disc	0	4360-43a	
3.3	12.5	1.10E+06	170	6.041	2.110	Air	Disc	0	4360-43a	
3.3	12.5	3.60E+06	164	6.556	2.094	Air	Disc	0	4360-43a	
3.3	12.5	4.90E+06	160	6.690	2.083	Air	Disc	0	4360-43a	
3.3	12.5	9.80E+06	150	6.991	2.055	Air	Disc	0	4360-43a	U
3.3	12.5	2.70E+05	200	5.431	2.180	Air	Disc	0.5	4360-43a	
3.3	12.5	3.40E+05	170	5.531	2.110	Air	Disc	0.5	4360-43a	
3.3	12.5	8.60E+05	150	5.934	2.055	Air	Disc	0.5	4360-43a	
3.3	12.5	2.44E+06	140	6.387	2.025	Air	Disc	0.5	4360-43a	
3.3	12.5	1.56E+06	135	6.193	2.010	Air	Disc	0.5	4360-43a	
3.3	12.5	2.60E+06	130	6.415	1.993	Air	Disc	0.5	4360-43a	
3.3	12.5	3.80E+06	120	6.580	1.958	Air	Disc	0.5	4360-43a	
3.3	12.5	3.20E+06	118	6.505	1.951	Air	Disc	0.5	4360-43a	
3.3	12.5	8.50E+06	115	6.929	1.940	Air	Disc	0.5	4360-43a	U
3.3	12.5	2.20E+06	114	6.342	1.936	Air	Disc	0.5	4360-43a	
3.3	12.5	1.00E+07	110	7.000	1.921	Air	Disc	0.5	4360-43a	U

NOTES:

(1) Log S values include a correction factor to account for variations in plate thickness

(2) Key to database comments: –

- U — Sample unbroken at the end of the fatigue test
- P — Fatigue crack formed at the location of a test probe
- G — Fatigue crack formed at the load arm gripper
- LS — Fatigue crack formed on the low stress side of the weld
- PL — Fatigue crack formed in the parent plate
- R — Fatigue crack formed in the weld root

(3) Steel Types used in tests

- 4360 43a — BS 4360 Grade 43a low carbon weldable steel
- 50 D — BS 4360 Grade 50 D or equivalent as per EEMUA standard
- Fy=685 — A High tensile strength steel.
- Fy= 368 — Norwegian steel specification - Statoil type 1

Database 3.2.2: Disc ground plate samples

Test Data Ref	Plate thk. mm	Cycles N	Stress N/mm2	Log N	Corrected Log S	CP	Weld Profile	R	Steel Type	Comment
	Burr Ground Samples									
3.2	12.5	2.60E+05	250	5.415	2.277	Air	Toe Burr	0	4360-43a	
3.2	12.5	3.70E+05	235	5.568	2.250	Air	Toe Burr	0	4360-43a	
3.2	12.5	5.90E+05	215	5.771	2.212	Air	Toe Burr	0	4360-43a	
3.2	12.5	8.80E+05	186	5.944	2.149	Air	Toe Burr	0	4360-43a	
3.2	12.5	9.50E+05	170	5.978	2.110	Air	Toe Burr	0	4360-43a	
3.2	12.5	1.60E+06	154	6.204	2.067	Air	Toe Burr	0	4360-43a	
3.2	12.5	4.30E+06	147	6.633	2.047	Air	Toe Burr	0	4360-43a	
3.2	12.5	5.40E+06	152	6.732	2.061	Air	Toe Burr	0	4360-43a	U
3.2	12.5	9.50E+06	140	6.978	2.025	Air	Toe Burr	0	4360-43a	U
3.7/3.8	30	1.80E+05	330	5.255	2.493	Air	Toe Burr	0.1	Fy=368	
3.7/3.8	30	1.70E+05	300	5.230	2.451	Air	Toe Burr	0.1	Fy=368	
3.7/3.8	30	2.10E+05	300	5.322	2.451	Air	Toe Burr	0.1	Fy=368	
3.7/3.8	30	4.50E+05	218	5.653	2.313	Air	Toe Burr	0.1	Fy=368	
3.7/3.8	30	8.50E+05	220	5.929	2.317	Air	Toe Burr	0.1	Fy=368	
3.7/3.8	30	1.60E+06	158	6.204	2.173	Air	Toe Burr	0.1	Fy=368	
3.7/3.8	30	2.40E+06	150	6.380	2.150	Air	Toe Burr	0.1	Fy=368	
3.2	12.5	4.35E+04	470	4.638	2.551	Air	Toe Burr	0	Fy=685	
3.2	12.5	1.20E+05	374	5.079	2.452	Air	Toe Burr	0	Fy=685	
3.2	12.5	3.40E+05	275	5.531	2.319	Air	Toe Burr	0	Fy=685	
3.2	12.5	3.70E+06	213	6.568	2.208	Air	Toe Burr	0	Fy=685	
3.2	12.5	5.60E+04	473	4.748	2.554	Air	Full Burr	0	Fy=685	
3.2	12.5	1.60E+05	375	5.204	2.453	Air	Full Burr	0	Fy=685	
3.2	12.5	8.60E+05	277	5.934	2.322	Air	Full Burr	0	Fy=685	
3.3	38	1.80E+05	323	5.255	2.509	Air	Toe Burr	0	50 D	
3.3	38	2.80E+05	276	5.447	2.441	Air	Toe Burr	0	50 D	
3.3	38	6.80E+05	228	5.833	2.358	Air	Toe Burr	0	50 D	
3.3	38	6.60E+05	210	5.820	2.322	Air	Toe Burr	0	50 D	
3.3	38	1.20E+06	190	6.079	2.279	Air	Toe Burr	0	50 D	
3.3	38	1.40E+06	186	6.146	2.270	Air	Toe Burr	0	50 D	
3.3	38	6.50E+06	180	6.813	2.255	Air	Toe Burr	0	50 D	U
3.3	38	4.20E+04	350	4.623	2.544	none	Toe Burr	0	50 D	sea water
3.3	38	1.15E+05	300	5.061	2.477	none	Toe Burr	0	50 D	sea water
3.3	38	2.10E+05	260	5.322	2.415	none	Toe Burr	0	50 D	sea water
3.3	38	2.50E+05	210	5.398	2.322	none	Toe Burr	0	50 D	sea water
3.3	38	3.30E+05	210	5.519	2.322	none	Toe Burr	0	50 D	sea water
3.3	38	1.30E+06	140	6.114	2.146	none	Toe Burr	0	50 D	sea water
3.3	38	2.00E+06	110	6.301	2.041	none	Toe Burr	0	50 D	sea water
3.3	38	6.30E+06	85	6.799	1.929	none	Toe Burr	0	50 D	sea water
3.5	38	1.51E+05	350	5.179	2.544	-0.85V	Toe Burr	0	50 D	
3.5	38	1.98E+05	300	5.297	2.477	-0.85V	Toe Burr	0	50 D	
3.5	38	3.73E+05	270	5.572	2.431	-0.85V	Toe Burr	0	50 D	LS pre-corroded
3.5	38	6.20E+05	240	5.792	2.380	-0.85V	Toe Burr	0	50 D	G pre-corroded
3.5	38	5.96E+05	210	5.775	2.322	-0.85V	Toe Burr	0	50 D	pre-corroded
3.5	38	6.64E+05	190	5.822	2.279	-0.85V	Toe Burr	0	50 D	pre-corroded
3.5	38	1.31E+06	170	6.117	2.230	-0.85V	Toe Burr	0	50 D	pre-corroded
3.5	38	2.04E+06	160	6.309	2.204	-0.85V	Toe Burr	0	50 D	P pre-corroded
3.5	38	3.02E+05	220	5.480	2.342	-1.10V	Toe Burr	0.5	50 D	P pre-corroded
3.5	38	7.66E+05	190	5.884	2.279	-1.10V	Toe Burr	0.5	50 D	pre-corroded
3.5	38	5.83E+05	170	5.766	2.230	-1.10V	Toe Burr	0.5	50 D	P pre-corroded
3.5	38	5.86E+06	105	6.768	2.021	-1.10V	Toe Burr	0.5	50 D	U pre-corroded
3.5	38	1.73E+05	350	5.238	2.544	-0.85V	Toe Burr	0	50 D	P as received
3.5	38	1.95E+05	300	5.290	2.477	-0.85V	Toe Burr	0	50 D	as received
3.5	38	1.12E+06	270	6.051	2.431	-0.85V	Toe Burr	0	50 D	as received
3.5	38	1.00E+06	240	6.002	2.380	-0.85V	Toe Burr	0	50 D	as received
3.5	38	1.21E+06	210	6.081	2.322	-0.85V	Toe Burr	0	50 D	G as received
3.5	38	1.81E+06	190	6.256	2.279	-0.85V	Toe Burr	0	50 D	LS as received
3.5	38	1.72E+06	175	6.235	2.243	-0.85V	Toe Burr	0	50 D	as received
3.5	38	1.12E+06	160	6.051	2.204	-0.85V	Toe Burr	0	50 D	G as received
3.2	12.5	1.05E+07	190	7.021	2.158	Air	Full Burr	0	4360-43a	
3.2	12.5	8.00E+06	205	6.903	2.191	Air	Full Burr	0	4360-43a	R
3.2	12.5	1.40E+06	213	6.146	2.208	Air	Full Burr	0	4360-43a	
3.2	12.5	8.00E+05	215	5.903	2.212	Air	Full Burr	0	4360-43a	
3.2	12.5	1.46E+06	226	6.164	2.233	Air	Full Burr	0	4360-43a	
3.2	12.5	4.70E+05	250	5.672	2.277	Air	Full Burr	0	4360-43a	
3.2	12.5	2.30E+05	280	5.362	2.326	Air	Full Burr	0	4360-43a	PL

NOTES:

(1) Log S values include a correction factor to account for variations in plate thickness

(2) Key to database comments:

U	Sample unbroken at the end of the fatigue test
P	Fatigue crack formed at the location of a test probe
G	Fatigue crack formed at the load arm gripper
LS	Fatigue crack formed on the low stress side of the weld
PL	Fatigue crack formed in the parent plate
R	Fatigue crack formed in the weld root

(3) Steel Types used in tests

4360 43a	BS 4360 Grade 43a low carbon weldable steel
50 D	BS 4360 Grade 50 D or equivalent as per EEMUA standard
Fy=685	A High tensile strength steel.
Fy= 368	Norwegian steel specification - Statoil type 1

Database 3.2.3: Burr ground samples

Ref	Specimen no	MED %	Joint Details & Test Type	Fy N/mm^2	Environment	CP	R	HSS N/mm^2	Kg	MPSE N/mm^2	Ni	Nt	Comments
	T-2-M1-30	30	T Joints,Axial	348.5	Air	0	-1	350	1.79	627	4.30E+06	4.30E+06	(UC)
	T-2-M1-30	30	Chord:508x16 mm		Air	0	-1	411	1.48	609	4.30E+06	4.30E+06	(UC)
	T-2-M2-30	30	Brace:244x10 mm		Air	0	-1	386	1.88	726	1.20E+06	1.46E+06	(U)
	T-2-M2-30	30			Air	0	-1	426	1.76	725	6.10E+05	1.46E+06	
	T-2-M1-60	60			Air	0	-1	382	1.93	737	3.20E+05	3.20E+05	(UC)
3.9	T-2-M1-60	60			Air	0	-1	456	1.85	844	1.03E+05	3.20E+05	
	T-3-M1-30	30	T Joints,Axial	395.5	Air	0	-	-	2.22	-	-	-	
	T-3-M1-30	30	Chord:508x45 mm		Air	0	-	-	2.06	-	-	-	
	T-3-M2-30	29	Brace:244x16 mm		Air	0	-1	284	2.33	654	6.60E+05	9.25E+05	(U)
	T-3-M2-30	32			Air	0	-1	284	2.33	661	1.08E+06	1.08E+06	(UC)
	T-3-M1-60	61			Air	0	-1	335	2.50	837	7.50E+04	1.90E+05	
	T-3-M1-60	60			Air	0	-	-	2.31	-	-	-	
	12A	11	T Joints ,OPB		Air	0	-1	363	2.13	773	1.19E+05	1.76E+05	(U)
	25A/1	13	Chord:457X16 mm	Chord:383 or	Air	0	-1	277	1.80	498	1.97E+05	-	
	25B/1	13		376	Air	0	-1	295	1.76	518	-	-	
	22B/1	13	Brace:229x12 mm	Brace:387	Air	0	-1	-	1.75	-	-	-	
	21A	13			Air	0	-1	226	2.00	452	1.11E+06	-	Cracks outside excavation
	18B	14			Air	0	-1	243	1.85	450	2.06E+05	6.49E+05	(U)
	10A	22			Air	0	-1	176	2.57	452	-	3.74E+06	Failed side B only
	25A/2	25			Air	0	-1	277	2.50	693	7.96E+04	-	
	25B/2	25			Air	0	-1	295	2.60	766	-	-	
	18A	25			Air	0	-1	246	1.95	480	1.42E+05	6.49E+05	
	22A/1	28			Air	0	-1	-	1.57	-	-	-	
3.11	14B	34			Air	0	-1	378	2.72	1029	2.32E+04	1.82E+05	(U)
	10B	38			Air	0	-1	184	2.44	449	6.38E+04	3.74E+06	
	25A/3	38			Air	0	-1	277	2.42	670	1.22E+05	7.20E+05	(U)
	25B/3	38			Air	0	-1	295	2.71	798	2.36E+05	7.20E+05	
	12B	39			Air	0	-1	360	2.88	1039	3.43E+04	1.76E+05	
	14A	41			Air	0	-1	389	2.58	1004	2.32E+04	1.82E+05	
	17A	44			Air	0	-1	210	1.90	570	1.63E+06	2.11E+06	(U)
	11B	44			Air	0	-1	362	2.77	1004	2.80E+04	2.66E+05	
	22A/2	59			Air	0	-1	416	3.42	1423	5.30E+03	1.68E+05	
	21B	63			Air	0	-1	249	2.72	474	8.61E+05	1.80E+06	
	11A	63			Air	0	-1	334	2.94	981	2.80E+04	2.66E+05	
	22B/2	66			Air	0	-1	439	1.66	729	5.30E+03	1.68E+05	
	17B	69			Air	0	-1	201	3.39	682	9.23E+03	2.11E+06	

MED = Maximum Excavation Depth, % of chord wall
Kg = MPSE/HSS HSS = As welded Hot Spot Stress MPSE = Maximum Principal Stress in Excavation
N_i = Number of cycles for first MPI detected crack
N_t = Number of cycles for through-thickness cracking of remaining thickness after grinding(deduced from ACPD measurements
which were subject to an error of 15% too early in terms of number of cycles)
(U) = Specimen Unbroken (UC) = Specimen Uncracked
Note:
T-3 Specimens were brace repairs

Database 3.3.1: Remedial ground joints

Test Data Ref	Plate thk. mm	Cycles N	Stress N/mm2	Log N	Log S	CP	Weld Profile	R	Steel Type	Comment
	Peened Samples									
3.2	12.5	1.40E+05	310	5.146	2.491	Air	Hammer (4 pass)	0	4360-43a	
	12.5	2.55E+05	290	5.407	2.462	Air	Hammer (4 pass)	0	4360-43a	
	12.5	1.15E+06	242	6.061	2.384	Air	Hammer (4 pass)	0	4360-43a	
	12.5	9.50E+06	215	6.978	2.332	Air	Hammer (4 pass)	0	4360-43a	U
	12.5	1.20E+06	470	6.079	2.672	Air	Hammer (4 pass)	0	High yield	PL
	12.5	2.20E+06	375	6.342	2.574	Air	Hammer (4 pass)	0	High yield	PL
	12.5	1.60E+06	275	6.204	2.439	Air	Hammer (4 pass)	0	High yield	
	12.5	7.00E+06	215	6.845	2.332	Air	Hammer (4 pass)	0	High yield	
3.2	12.5	2.05E+05	440	5.312	2.643	Air	Hammer (4 pass)	-1	4360-43a	PL
	12.5	4.10E+05	370	5.613	2.568	Air	Hammer (4 pass)	-1	4360-43a	PL
	12.5	6.20E+05	340	5.792	2.531	Air	Hammer (4 pass)	-1	4360-43a	PL
	12.5	9.50E+05	320	5.978	2.505	Air	Hammer (4 pass)	-1	4360-43a	R
	12.5	3.15E+06	300	6.498	2.477	Air	Hammer (4 pass)	-1	4360-43a	R
	12.5	5.50E+06	300	6.740	2.477	Air	Hammer (4 pass)	-1	4360-43a	
	12.5	3.60E+06	278	6.556	2.444	Air	Hammer (4 pass)	-1	4360-43a	U
	12.5	7.00E+06	230	6.845	2.362	Air	Hammer (4 pass)	-1	4360-43a	U
	12.5	9.20E+06	220	6.964	2.342	Air	Hammer (4 pass)	-1	4360-43a	U

NOTES:

(1) Key to database comments:

U	Sample unbroken at the end of the fatigue test
PL	Fatigue crack formed in the parent plate
R	Fatigue crack formed in the weld root

(2) Steel Types used in tests

4360 43a	BS 4360 Grade 43a low carbon weldable steel
High yield	Fy = 485 N/mm^2

Database 3.4.1: Hammer peened plate samples

IV 4 CLAMP TECHNOLOGY

IV 4.1 INTRODUCTION

IV 4.1.1 General

This Chapter IV 4 concerns the background to all types of clamps and connections. There are a number of basic forms and variants to these forms to consider, including:

- stressed mechanical clamps incorporating steel-to-steel interfaces between each clamp and member or joint
- unstressed grouted clamps incorporating steel-to-grout interfaces, which may be plain or include shear keys (of various types)
- stressed grouted clamps incorporating plain steel-to-grout interfaces
- stressed neoprene-lined clamps incorporating steel-to-elastomer interfaces.

The main subject matter within this background is the slip strength of a connection. Because each of the above forms has a different interface and/or stress condition, the load transfer and failure mechanisms can also be expected to differ. This in turn leads to different formulations of the slip strength equations.

For the purposes of rationalisation of the design procedures given in Volume III, the slip strength for all types of clamps is expressed in terms of interface shear stress. This approach also has another major advantage in that it avoids any possible ambiguity in the meaning assigned to the coefficient of friction. This ambiguity, unfortunately, presently exists and, given its importance to clamp design, is discussed in the next subsection.

In addition to static slip strength, this chapter discusses the effects of early age movement, fatigue loading, and studbolt considerations.

IV 4.1.2 Definitions

IV 4.1.2.1 Coefficient of friction

Friction, as a concept, is most normally introduced at school in terms of a block sliding across a table. The product of the reaction, R, on the block and the coefficient of friction gives the frictional resistance, F. It is possible to divide the frictional resistance and the reaction by the contact area to define the frictional shear stress, τ_F, as a function of the contact stress, p. This is illustrated in Figure 4.1.1(a). Note that there is no need to redefine the coefficient of friction; for a given reaction force or corresponding contact stress, the same frictional resistance will result.

Now consider the case of a single saddle plate (from one clamp half) bearing on a tubular member as in Figure 4.1.1(b). It is possible to define the coefficient of friction in a global sense, ie. as the ratio of F/R (here R can be interpreted as the total load arising from the studbolts and F as half of the clamp slip load). This definition is implicit in the equations given in Reference 4.1, for the slip strength of stressed mechanical clamps. Using subscript 'g' to denote global, the friction equation is thus:

$$F = \mu_g R \qquad \text{....4.1.1}$$

It is also possible to use a local definition for the coefficient of friction (μ_ℓ = friction shear stress/contact stress). In this case the radial pressure distribution has to be assumed and the corresponding shear frictional stress calculated. Assuming that the circumferential tension in the saddle plate in constant (ie. hoop friction is ignored) then the radial contact pressure is also constant and can be related to the studbolt force through equilibrium considerations. The interface contact area is fixed by the tubular diameter and length of saddle plate. As shown in Figure 4.1.1(c), the resulting friction equation is:

$$F = \mu_\ell \frac{\pi}{2} R \qquad \text{....4.1.2}$$

A comparison of equations 4.1.1 and 4.1.2 shows that for a given reaction R and frictional resistance F, the numerical value of the global coefficient μ_g is $\pi/2$ times greater than that of the local coefficient μ_ℓ.

It should be emphasised that the difference between the two definitions (local or global) should not lead to different slip resistances. However, it is clearly important to know which definition is being used so that the corresponding coefficient can be applied consistently. In this manual, unless stated otherwise, reference to coefficient of friction shall always imply the local definition for which contact stress and frictional shear stress shall apply.

In the above, a constant radial contact pressure was assumed. There are no experimental data to support or to deny this assumption. It is likely, in fact, that the distribution is not uniform due to ovality of both the clamp saddle and the enclosed member, and due to differential straining of the saddle during studbolt tensioning operations. However, and as implied above, any assumption will lead to the same calculated axial slip resistance given that the same assumption is used in both slip data assessment and slip prediction. In the absence of any data relating to the torsional slip resistance of clamps, this has to be necessarily inferred from axial data. Again, it can be shown that any consistently applied radial contact pressure distribution assumption does not affect the calculated slip resistance as long as the actual distribution applies to

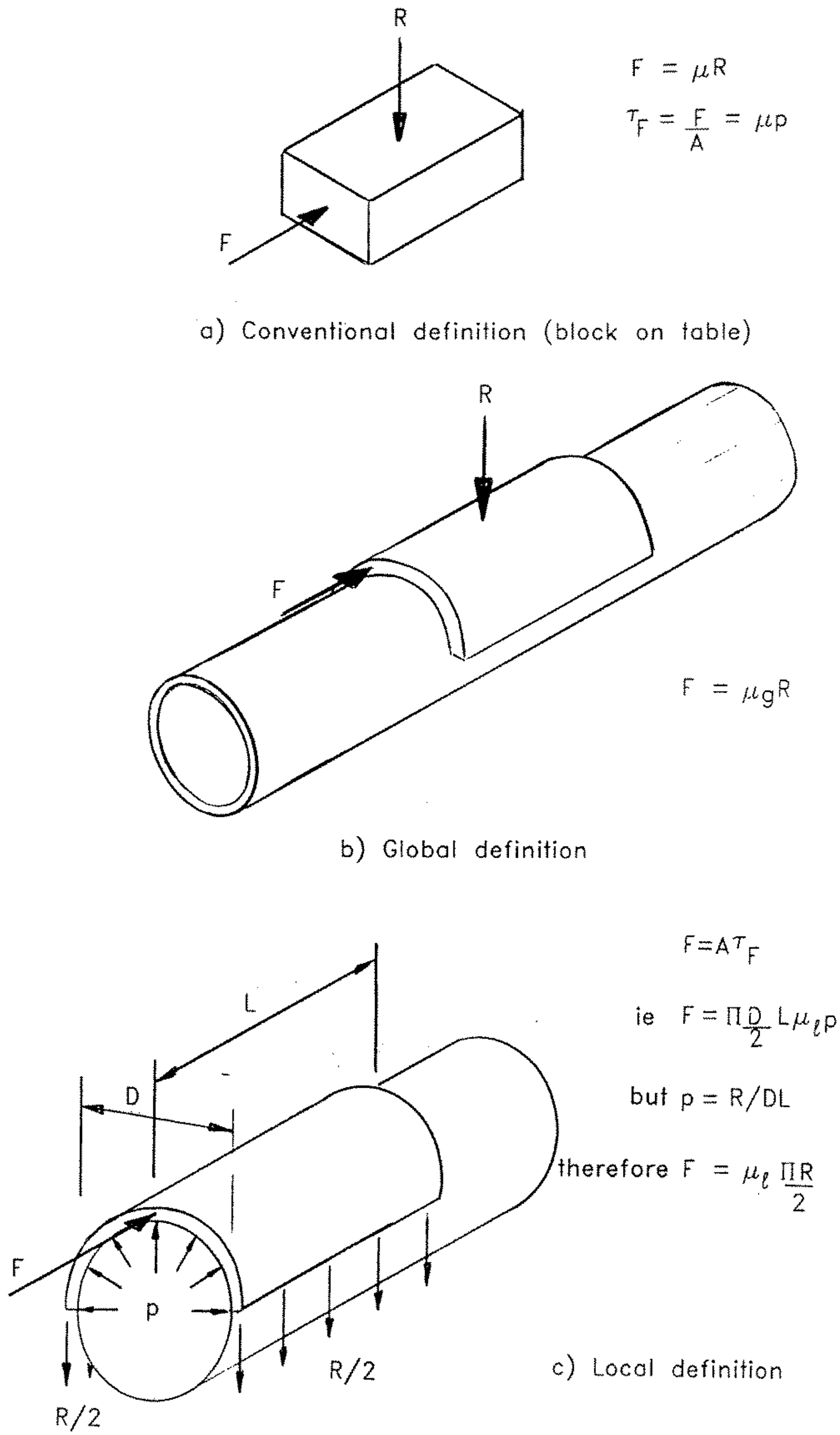


Figure 4.1.1: Definitions of coefficient of friction

both axially loaded and torsionally loaded clamps. This must be so for small values of loading and should apply sensibly up to the point of initial slipping near peak loads. It may not be true thereafter. Nevertheless, it is considered that torsional slip resistances should be reasonably accurately predicted from axial slip data.

IV 4.1.2.2 Test specimen type

Various types of specimen are discussed in this chapter, including static test specimens, fatigue test specimens, etc. These may be further subdivided into load type, ie. axial, bending or combined loading. With respect to axial loads, it is sometimes important to distinguish the sense of stresses in the sleeve and the member. Although four combinations are possible, only three have been tested as shown in Figure 4.1.2. In practice, only the arrangements shown in Figures 4.1.2(b) and (c) will occur. However, the arrangement shown in Figure 4.1.2(a) has sometimes been used as it will maximise radial Poisson effects and thus lead to a lower bound test result for the resistance of the connection.

It may be noted that for grouted clamps (stressed or unstressed), failure is normally associated with the inner grout-to-steel interface. Therefore, the Poisson contraction/dilation of the member assumes greater importance than that of the clamp. The arrangements shown in Figures 4.1.2(a) and (c) may consequently give similar results.

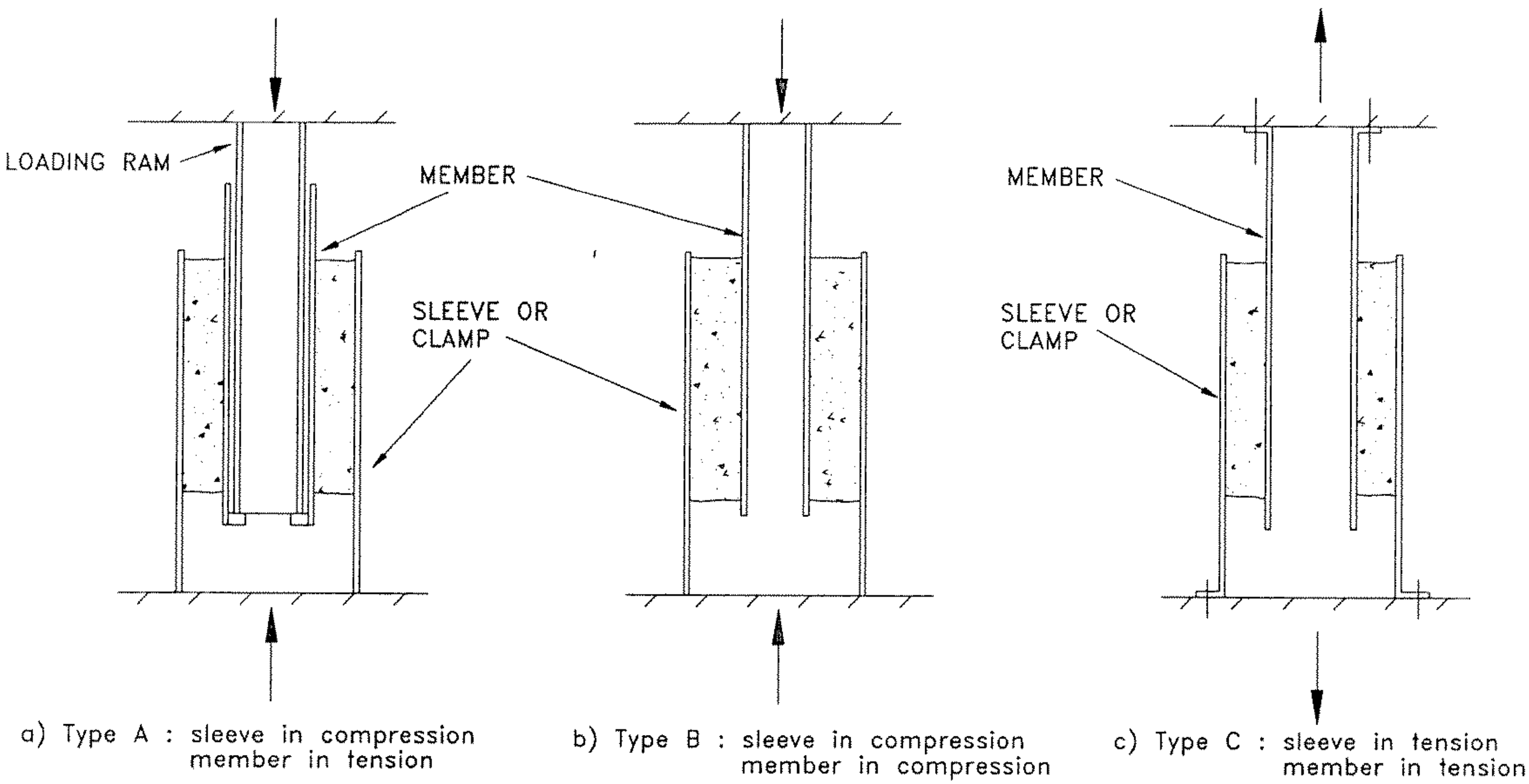


Figure 4.1.2: **Types of test arrangements**

IV 4.2 SLIP STRENGTH OF STRESSED MECHANICAL CLAMPS

This Section IV 4.2 examines the slip strength of stressed mechanical clamps. The means by which a mechanical clamp transfers loads is by friction resistance at the steel-to-steel interface and thus the frictional coefficient is the main factor investigated.

IV 4.2.1 Summary of Existing Guidance

IV 4.2.1.1 General

Codified design guidance on mechanical clamps is only given in the UK Health and Safety Executive Guidance Notes (4th edition). Design formulae for slip strength can be found in the literature.

IV 4.2.1.2 UK Department of Energy Guidance Notes[4.2]

The maximum value of the frictional coefficient, in the absence of experimental data, between steel-to-steel interfaces is required not to be higher than 0.25. This frictional coefficient is valid for grit blasted, water jetted and/or wire brushed steel surfaces. It is not stated which definition of the coefficient of friction (see IV 4.1.2) should be utilised. Safety factors of not lower than 1.7 and 2.25 are recommended for extreme and operating conditions respectively. Factors affecting tensioned bolts are briefly described.

IV 4.2.1.3 OTH 88 283[4.1]

This report has proposed an equation for the frictional coefficient. The proposed equation was developed using a least square analysis based on minimising the absolute differences. The characteristic global frictional coefficient is shown to be:-

$$\mu_c = 0.18C_s' \left[1 + 20 \left[\frac{T}{D}\right]\right] (1 + 66\,K_b) \qquad \text{....4.2.1}$$

where μ_c is the characteristic global frictional coefficient; C_s' is the surface condition factor; (T/D) is the thickness to diameter ratio of the member; K_b is the studbolt stiffness parameter. This studbolt stiffness parameter is defined as follows:-

$$K_b = \frac{n\,A_b\,E_b}{2LL_bE_s} \qquad \text{....4.2.2}$$

in which n is the number of studs in the connection; E_b is the Young's modulus of studbolt material; L is the length of the clamp; L_b is the stressed length of the studbolt (nut face to face); and E_s is the Young's modulus of the chord steel.

The coefficient of friction in Equation 4.2.1 is a global definition (see Section IV 4.1.2), ie. the axial capacity may be found as:-

$$P_c = 2\,\mu_c\,F_n \qquad \text{....4.2.3}$$

where F_n is the total studbolt force. The factor of 2 arises because each half of the clamp is active in providing the total slip resistance. Substituting Equation 4.2.1 into Equation 4.2.3:-

$$P_c = 0.36\,C_s' \left[1 + 20\left[\frac{T}{D}\right]\right] (1 + 66\,K_b)\,F_n \qquad \text{....4.2.4}$$

Equation 4.2.1 was derived from the reported experimental data. Altogether 10 mechanical clamps were tested. The effects of clamp stiffness and bolt stiffness parameters reflected in the equation are based on limited data. Recorded data of three tests from each specimen were used in the least square analysis, even though the surface condition of the steel-to-steel interface may be altered in each test. The surface condition factor C_s' was determined from the frictional coefficients of steel-to-steel interfaces of mechanical clamps and flat plates.

The characteristic strength is specified for a 95% probability of survival with 95% confidence level. The above equation is specified for use within the following limits:-

$$0.5 \le \frac{L}{D} \le 2.0$$

$$20 \le \frac{D}{T} \le 50$$

$$0.002 \le K_b \le 0.01$$

The safety factors to apply to the predicted friction coefficient are specified as 1.70 and 2.25 for extreme and operating conditions respectively.

IV 4.2.2 Critique of Existing Guidance

The main source of information on which mechanical clamps have been designed are in OTH 88 283. Despite the great advance made on its publication, it contains a number of shortcomings in the analysis of the accepted data:-

- The curve fitting exercise was based on minimising the squares of the absolute differences between the data points and the fitted curve. It is now generally accepted that this may introduce a bias into the fitted curve and that minimising the squares of the percentage differences is a better approach.

- The development of the friction equation was carried out in stages. That is to say each parameter, eg. T/D, was investigated in turn by only selecting those data which maintained all other parameters as constant values. Having then found the effect of each parameter individually, they were then combined and a further analysis conducted to establish the overall curve fitting factor (in effect the basic friction factor). This approach is liable to introduce errors as it ignores the effect on any one parameter, of those data points in which two (or more) parameters alter. (These points are, however, used in establishing the overall curve fitting factor.) A more correct approach is to consider all data points simultaneously in a multi-variate analysis which gives a true best-fit curve (or more precisely a best-fit surface in multi-dimensional space).

- The characteristic strength was calculated on the basis of 95% probability of survival at a 95% confidence level. This is unduly conservative and does not correspond to present day practices for limit state analysis which call for a 95% probability of survival at a 50% confidence level. This confidence level is adopted in the UK Health and Safety Executive Guidance Notes for the strength of many structural components, eg. tubular joints and pile/sleeve connections.

In view of the above, it is appropriate to reanalyse the data. In the development of the new formulations for slip strength, a different definition of the coefficient of friction to OTH 88 283 is used as explained in Section IV 4.1.2.

IV 4.2.3 Database

The database on mechanical clamp test results is presented in Database 4.2.1 at the end of this Chapter. Essentially, only those results previously reported in OTH 88 283 are available.

Ten mechanical clamp specimens were tested nine times each, with the studbolt force changed after every set of three tests. Inspection of the results show that whereas the individual results of any one particular set of three tests were fairly consistent, the measured frictional coefficient dropped between subsequent sets. Furthermore, it did not matter whether the studbolt force was increased or decreased between subsequent sets - the frictional coefficient always dropped. It is most likely that there was a progressive degradation of the steel surfaces, although it is not clear why this is only observed between sets of three results and not between each test. Noting that it is the 'first' slip strength which is of importance, and that the first three results of any one specimen are consistent, the data in Database 4.2.1 only include the first set of three results for each specimen.

The test results presented in Database 4.2.1 satisfy the following screening criteria:

i. clamped member diameter $\geq$ 150mm,
ii. measurement of maximum load taken at peak of load displacement curve,
iii. a consistency between tabulated results and text.

The compiled data are further filtered for the purpose of statistical analysis as described below. This is to avoid the introduction of inadvertent modelling bias into the derived friction coefficient equation.

IV 4.2.4 General Form of Frictional Coefficient Equation

The slip strength of a mechanical clamp is obtained through the friction developed at the steel-to-steel interface under a pressure load. This slip strength is influenced by several factors:-

- magnitude of the pressure load;
- surface condition of the steel-to-steel interface;
- length to diameter ratio of the clamp;
- stiffness of the existing tubular on which the clamp is connected;
- stud bolt stiffness.

A non-dimensional surface factor C_s' is introduced to take account of various steel surface conditions. The surface condition has been identified as a major factor affecting the frictional coefficient.

The length to diameter ratio has a minor influence on the frictional coefficient. This is expected because when slipping commences, the shear stresses induced by friction along the connection length are reasonably uniform.

The radial stiffness of a mechanical clamp/clamped tubular member assembly does affect the slip strength. The stiffened saddle plates in all tested mechanical clamps were relatively much stiffer than the existing members. Hence the member diameter to thickness ratio D/T is considered to be much more important than the clamp geometry. Available experimental results have indicated that frictional coefficients increase with T/D. This is because smaller Poisson strains occur in thicker walled tubes leading to smaller losses of pressure on the steel-to-steel interfaces. A linear correction factor $[1 + \eta_1 (T/D)]$ is postulated, where η_1 is a constant to be determined from a regression analysis. The studbolt axial stiffness is also relevant to the pressure on the steel-to-steel interface. To account for the diameter reduction of an existing tubular member on the studbolt force, a studbolt stiffness parameter was introduced in Reference 4.1. The experimental results are found to exhibit a linear relationship between frictional coefficient and studbolt stiffness. Therefore it is not unreasonable to have a linear correction factor $(1 + \eta_2 K_b)$ in the frictional coefficient expression, where η_2 is a regression constant.

From the above arguments, the slip strength equation was tentatively postulated to take the following form:-

$$\mu_{mean} = \mu_b \, C_s' \left[1+\eta_1 \left(\frac{T}{D}\right)\right] (1 + \eta_2 \, K_b) \left[1+C_L \left(\frac{L}{D}\right)\right] \quad ....4.2.5$$

where μ_b is the basic frictional coefficient; η_1, η_2 and C_L are the curve fitting constants. Here the frictional coefficient is defined as the ratio between the average shear stress and the average pressure on the steel-to-steel interface (see Section IV 4.1.2).

IV 4.2.5 Development of Mean and Characteristic Formulations

Both mean and characteristic coefficient equations are given. The characteristic friction coefficient is taken as that which will be exceeded by 95% of all test results. Note, however, that when dealing with a finite set of results, this value can only be calculated to a certain level of confidence. Here, a 50% confidence level has been adopted for calculating the characteristic strength. That is to say, the calculated characteristic strength has a 50% probability of being greater (and 50% probability of being lesser) than the characteristic strength which would be derived from an infinite population of results.

All the test results were plotted in Figure 4.2.1 and the scatter of the test data reflects the effects of various factors mentioned above. A multi-variate nonlinear regression analysis was carried out to determine the curve fitting constants in the proposed frictional coefficient formula. The test data used are those of mechanical clamps with shot blasted surfaces (for which $C_s' = 1.00$). The remaining data were chosen to calibrate the values of the surface condition factor calculated from the test data of flat plates. The fitted expression thus derived is:-

$$\mu_{mean} = 0.128 \, C_s' \left[1+0.19 \left(\frac{L}{D}\right)\right] \left[1+19.81 \left(\frac{T}{D}\right)\right] (1 + 67.95 \, K_b) \quad ....4.2.6$$

As shown in the above equation, and as expected, the length effect on the frictional coefficient is negligibly small for the practical L/D range and it is justifiable to ignore it. A slightly modified formula is therefore proposed and the fitted expression is:-

$$\mu_{mean} = 0.131 \, C_s' \left[1 + 19.76 \left(\frac{T}{D}\right)\right] (1 + 68.16 \, K_b) \quad ....4.2.7$$

The modelling bias factor is defined as the ratio between measured and predicted frictional coefficients. The mean and standard deviation of this bias factor can be used to judge the quality of the fitted curve. The mean bias and standard deviation for the above expression are found to be 1.00 and 0.056 respectively. It may be observed that the coefficients in at least the bracketted terms in Equation 4.2.7 are very similar to those in Equation 4.2.1. Indeed, putting 20 and 66 in Equation 4.2.7 and refitting, the leading constant becomes 0.132 and the COV is unchanged at 0.056. For a sample size of 21, the characteristic bias is given by 1.00 - 1.67 x 0.56 = 0.907. The leading constant in the characteristic strength equation would thus become 0.132 x 0.907 = 0.120. Taking into account that this value relates to the local frictional coefficient, the inferred global coefficient leading constant is 0.120 x $\pi/2$ = 0.188 which can be compared to 0.18 in Equation 4.2.1. It is thus concluded that the design guidance given in OTH 88283 has been largely validated and the recommended characteristic equation in terms of local friction strength is:

$$\mu_c = 0.12\, C_s' \left[1 + 20 \left(\frac{T}{D}\right)\right] (1 + 66\, K_b) \qquad \text{....4.2.8}$$

The characteristic slip force for an axially loaded clamp, P_c, is thus:-

$$P_c = A\, \mu_c\, p = \pi\, D\, L\, .\, \mu_c\, .\, F_n/DL$$

$$P_c = 0.38\, C_s' \left[1 + 20 \left(\frac{T}{D}\right)\right] (1 + 66\, K_b)\, F_n \qquad \text{....4.2.9}$$

where F_n is the total studbolt force. This may be directly compared to Equation 4.2.4.

As can be seen in Figures 4.2.2 and 4.2.3, the measured and predicted frictional coefficients show good correlation. The quality of the fitted curve can also be judged by inspecting any apparent variation of the modelling bias with the governing parameters. As expected, the modelling bias does not vary with the length to diameter ratio (see Figure 4.2.4). Figures 4.2.5 and 4.2.6 illustrate the variations of the modelling bias on the stiffness parameters of the existing tubular and studbolts. On the whole, the proposed formula is not biased towards any particular governing parameter.

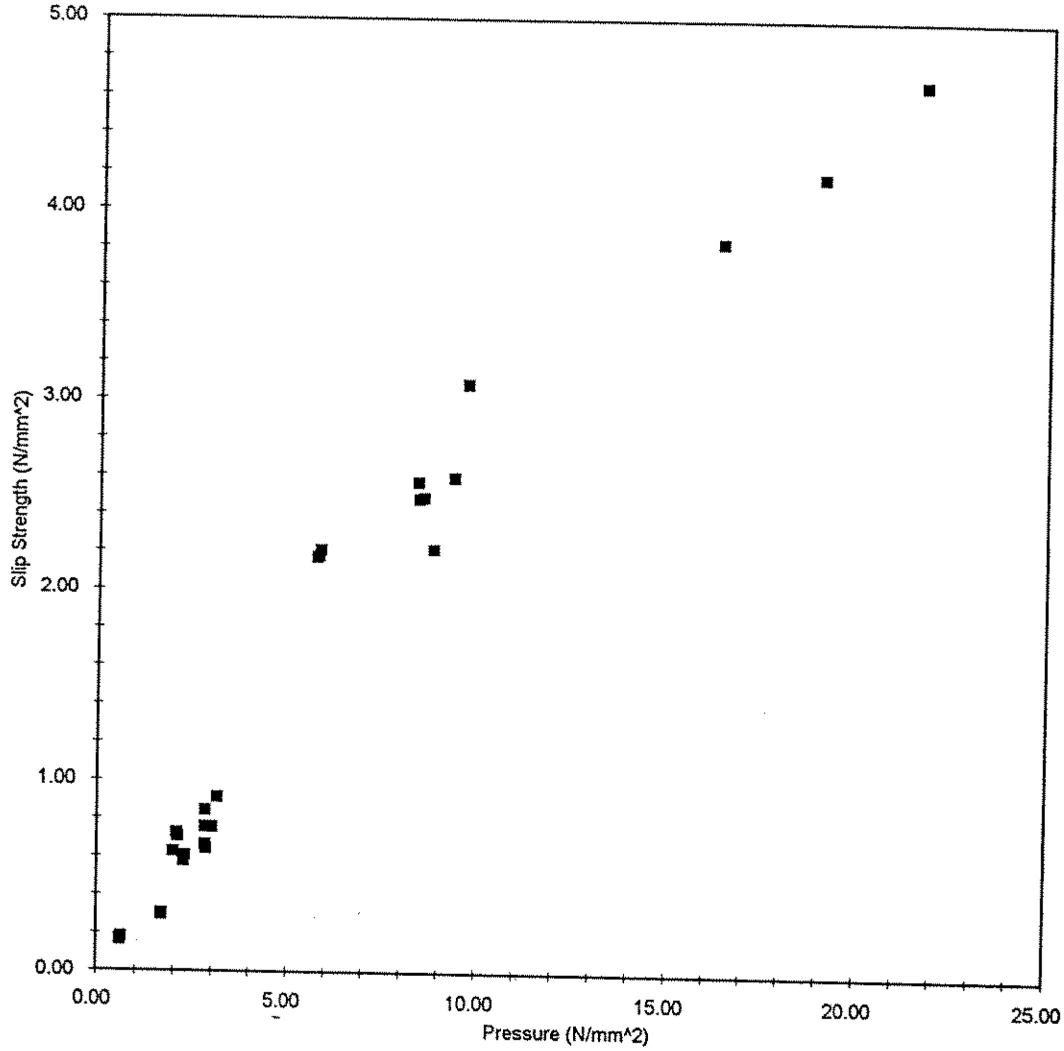


Figure 4.2.1: **Tested slip strength and interface pressure of mechanical clamps**

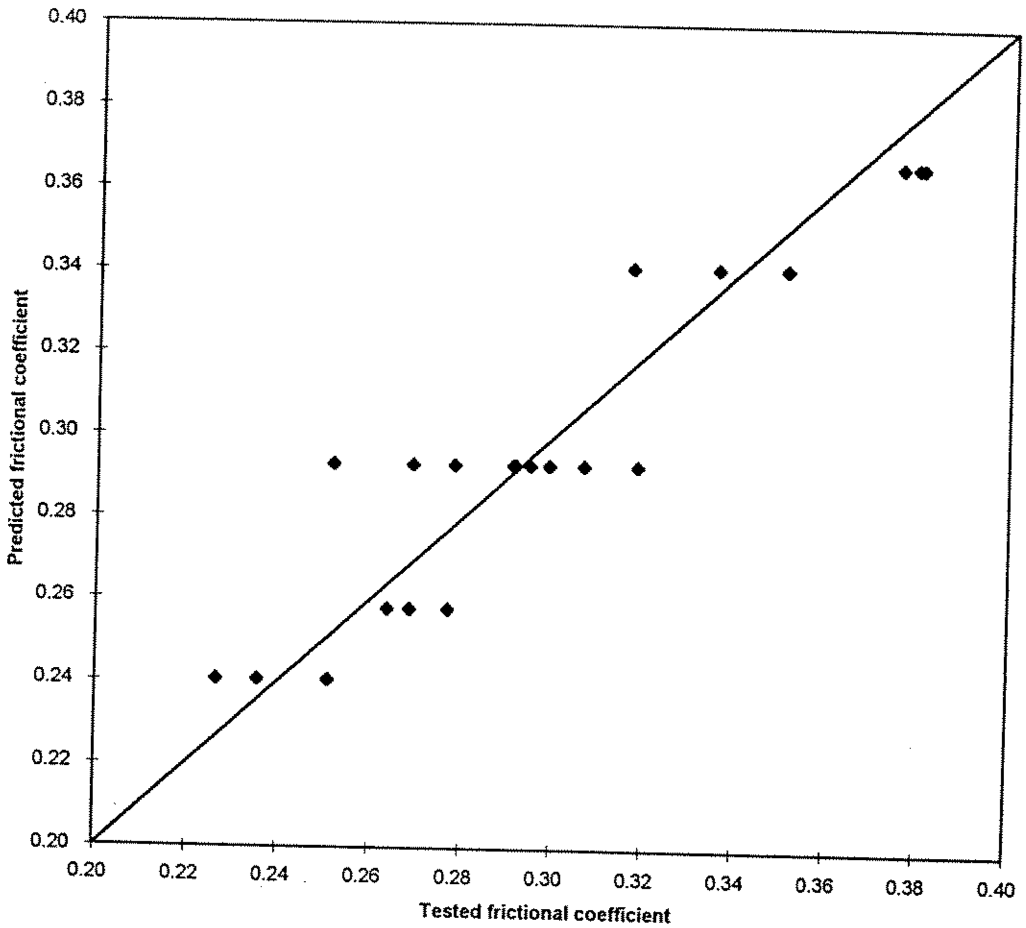


Figure 4.2.2: **Tested and predicted frictional coefficients of mechanical clamps**

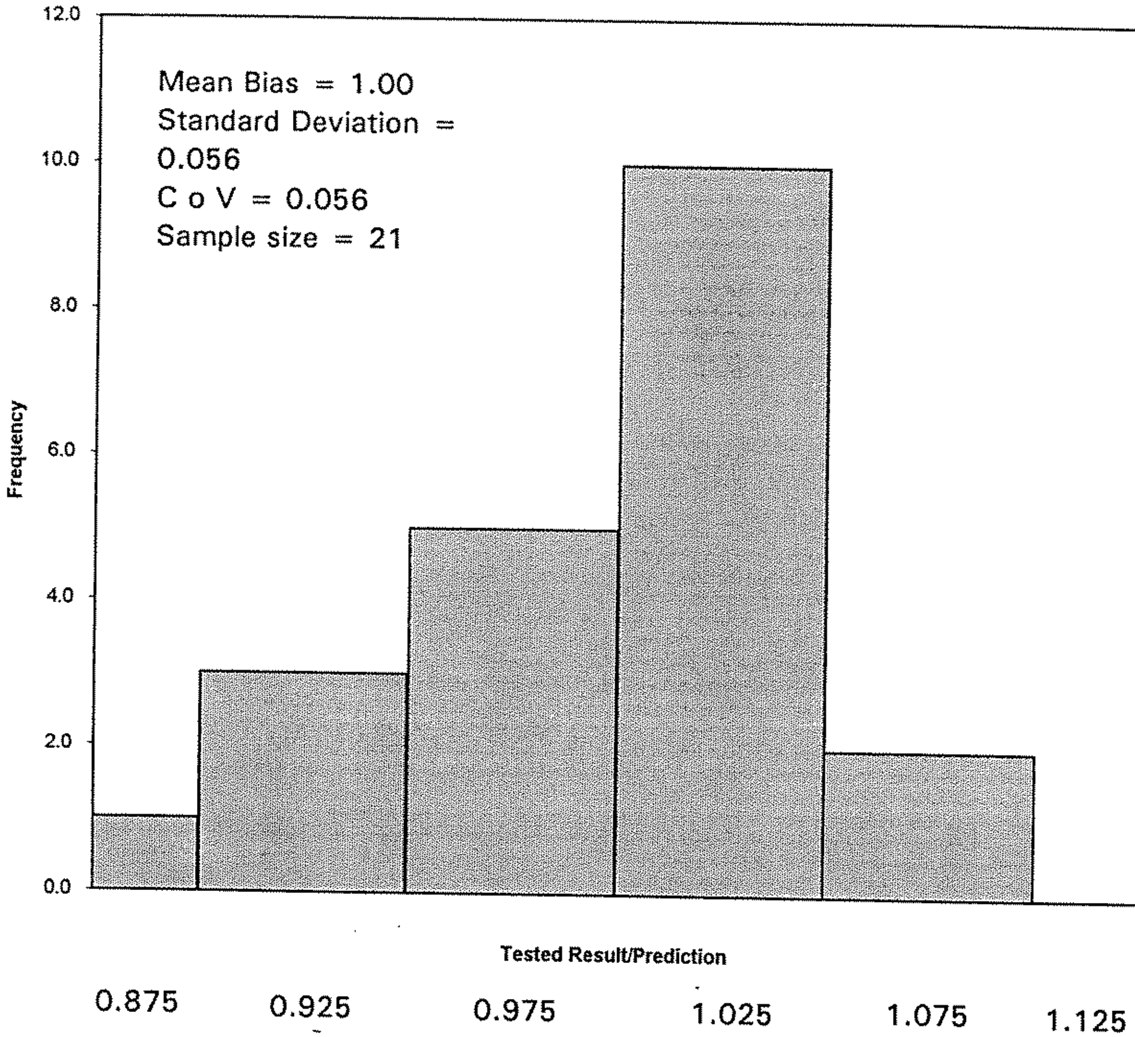


Figure 4.2.3: **Histogram of modelling of frictional coefficient of mechanical clamps**

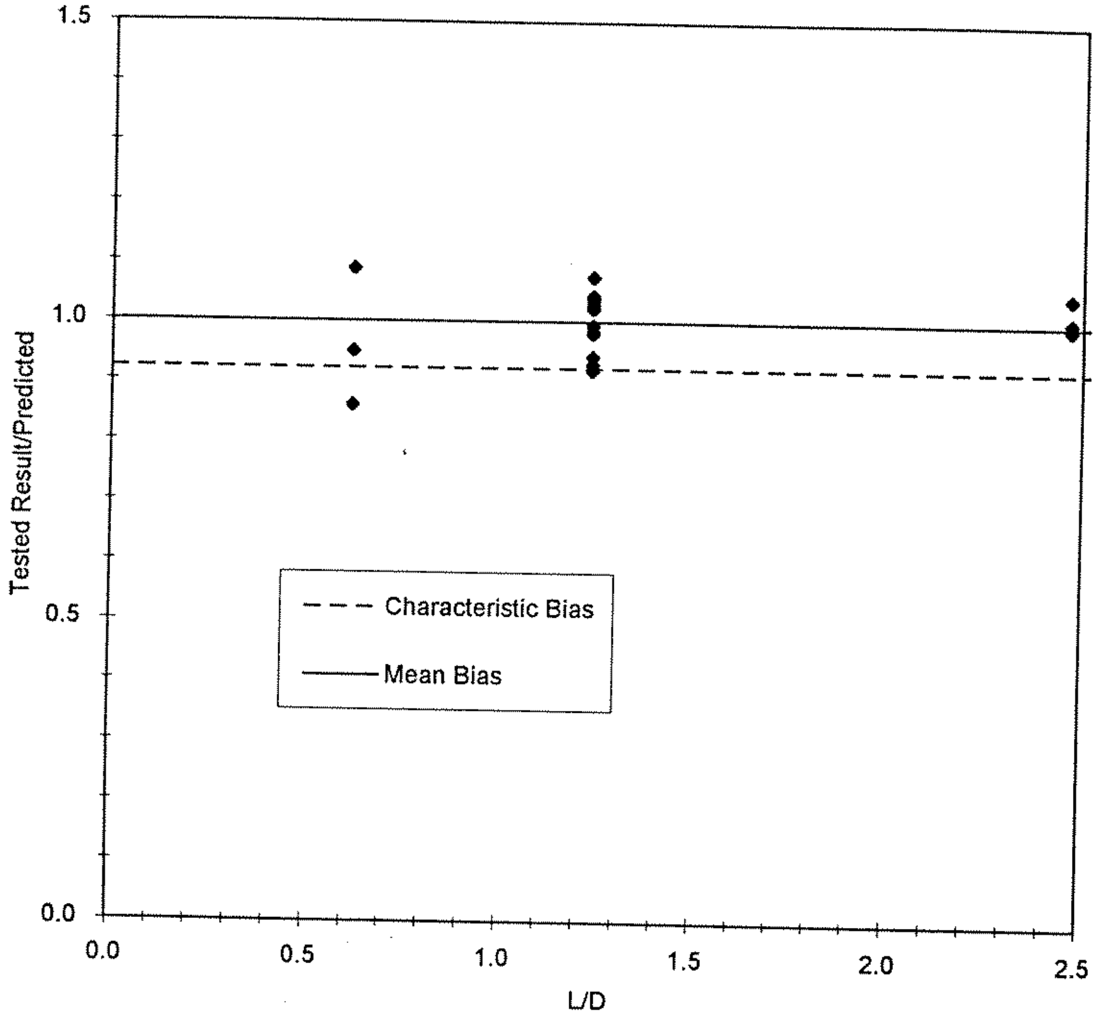


Figure 4.2.4: Effects of L/D ratio on modelling bias

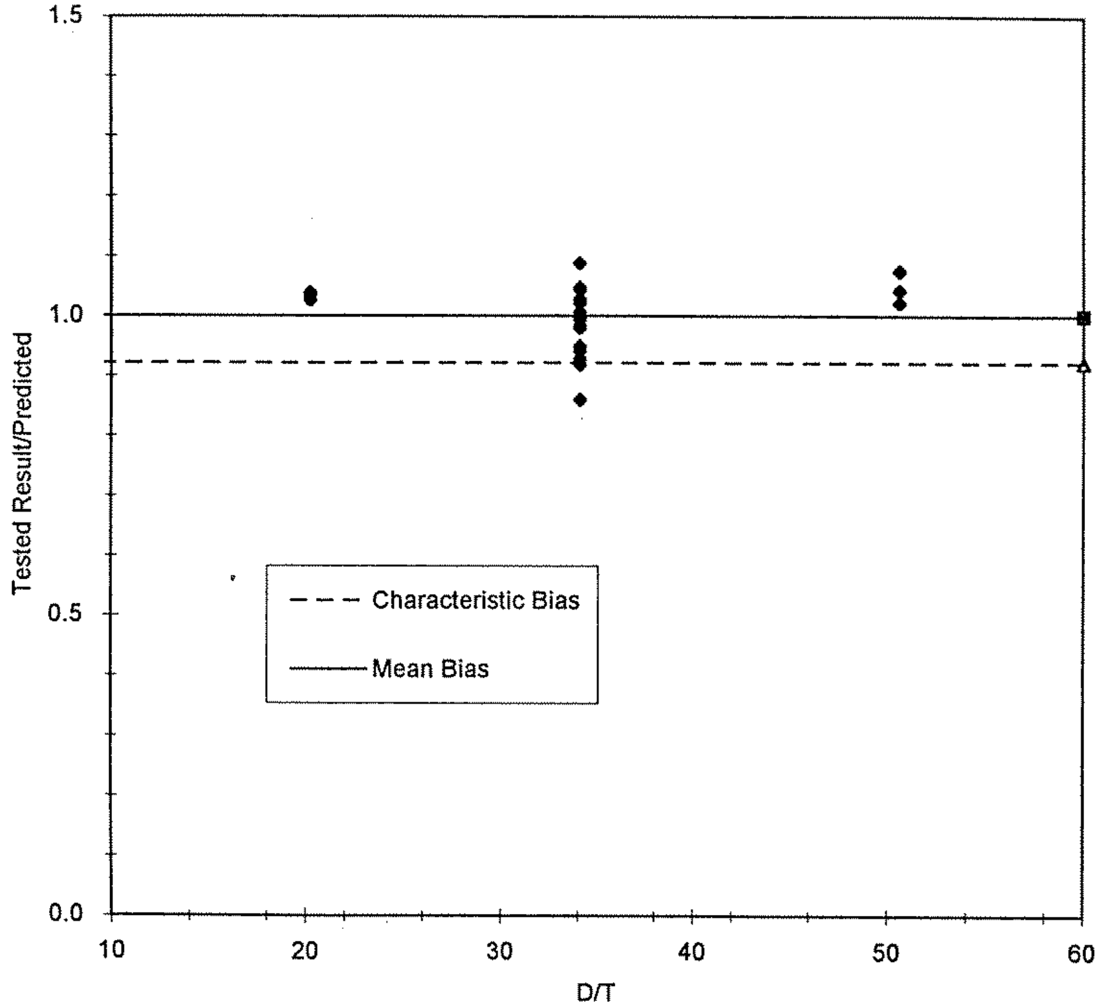


Figure 4.2.5: **Effects of D/t ratio on modelling bias**

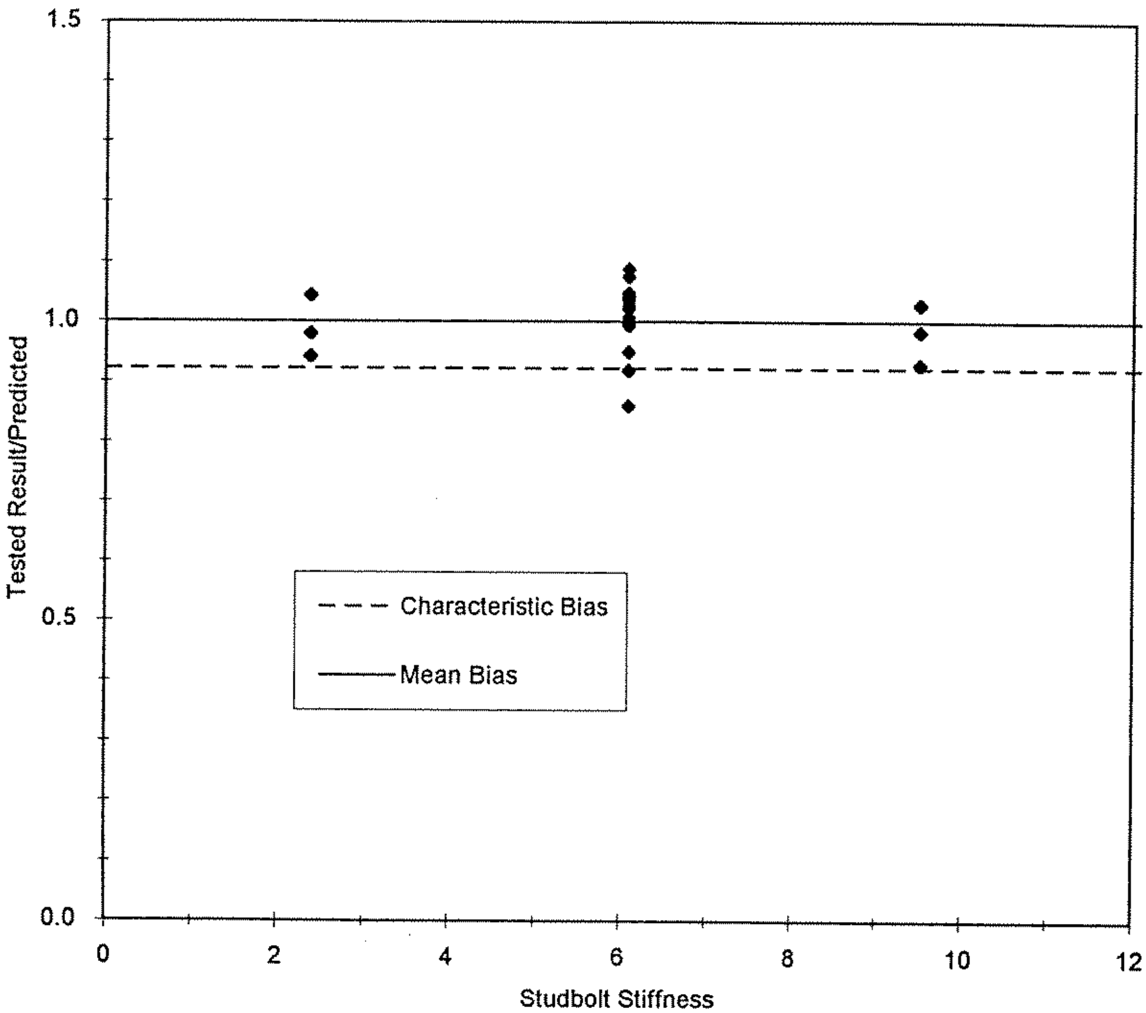


Figure 4.2.6: **Effects of studbolt stiffness on modelling bias**

IV 4.2.6 Surface Condition Effects

The mean frictional coefficient for mill scale surfaces is found to be 0.245, if test data of specimens 8 and 10 are averaged. The mean frictional coefficient for coal tar epoxy surfaces (specimen 9) is 0.180. Specimen 2, which has the same D/T and studbolt stiffness as specimens 8 to 10, has a mean frictional coefficient of 0.287 and is therefore selected to be representative of the mean for shot blasted surfaces. The non dimensional surface condition factor therefore takes the following values:-

Surface Condition	Surface Condition Factor C_s'
Shot Blasted	1.00
Mill Scale	0.85
Coal Tar Epoxy	0.63

If the frictional behaviour of a mechanical clamp/member assembly is assumed to be similar to that of two flat plates, the surface condition factor for a grit blasted surface subsequently subject to salt water or salt air exposure (up to 336 hours, see Database 4.2.2) can be calculated as follows:-

$$C_s' = \frac{0.329}{0.379} = 0.868$$

where 0.379 is the characteristic frictional coefficient of 10 blast cleaned flat plate specimens and 0.329 is that of 19 flat plate specimens with various salt exposed surface conditions (see Database 4.2.2).

IV 4.2.7 Recommendations

The following recommendations are based on the findings reported above. In some instanees, numerical factors have been rounded for design purposes and so that a greater accuracy than that which the data permits is not implied.

The following characteristic slip strength is recommended for a stressed mechanical clamp:-

$$\sigma_c = \frac{0.12\,C_s'}{\Gamma_f}\left[1 + 20\left[\frac{T}{D}\right]\right](1 + 66\,K_b)\left[\frac{F_n}{DL}\right] \qquad \ldots.4.2.10$$

The formula has been derived from experimental data of mechanical clamps within the following ranges:-

$$0.5 \leq \frac{L}{D} \leq 2.0 \text{ (though this parameter has negligible effect)}$$

$$20 \leq \frac{D}{T} \leq 50$$

$$0.002 \leq K_b \leq 0.01$$

The surface condition factor, in the absence of other data, should be taken according to one of the following values:-

Surface Condition	Surface Condition Factor C_s'
Shot blasted in air	1.00
Mill scale	0.85
Underwater grit blast	0.85
Coal tar epoxy	0.60

It is recommended to retain the safety factors of the UK Health and Safety Executive Guidance Notes for the extreme and operating conditions, namely:-

$$\Gamma_b = \begin{cases} 1.70 & \text{for extreme condition} \\ 2.25 & \text{for operating condition.} \end{cases}$$

IV 4.3 SLIP STRENGTH OF UNSTRESSED GROUTED CLAMPS/SLEEVE CONNECTIONS

IV 4.3.1 Introduction

The strength of a grouted connection is obtained from a combination of chemical bond, friction and mechanical interlock. Mechanical interlock may arise due to micro- and macro- geometric imperfections, or deliberate addition of shear keys, eg. weld beads. Pile/sleeve connections have been widely deployed in fixed jacket platforms, and these connections normally have continuous sleeves. 'Split sleeve' grouted clamps are primarily used for repair and strengthening.

The original research thrust in this field was on continuous pile/sleeve connections and has led to a large present-day database. Several codified guidance formulations for such connections have been developed in the last decade and are widely used by offshore industry around the world. The validity and applicable limits of these formulations are still being debated, and several on-going joint industry projects have been formed aiming to validate and improve the current strength formulae by experimental and analytical means. Within the context of the present project, these formulations are appraised for application to connections used in repair situations (using either continuous or split sleeves). (The UK HSE Guidance Notes specifically states that when designing a grouted connection for a repair scheme, reference is to be made to the section on pile/sleeve connections contained within the Guidance Notes.) A comparison of the data with the available codified guidance for pile/sleeve connections has indicated that the present formulations may be suitable with modifications for application to connections in repair situations.

IV 4.3.2 Databases

All relevant public domain and other accessible data have been compiled into databases according to connection type and loading condition. The main bulk of the test data are from OTH reports (References 4.1 and 4.3). The Databases can be found at the end of this Chapter.

Database	Type of unstressed grouted connection
4.3.1	Plain pipe continuous sleeve
4.3.2	Plain pipe continuous sleeve (expansive grout)
4.3.3	Weld beaded continuous sleeve
4.3.4	Plain pipe split sleeve (axial tension)
4.3.5	Plain pipe split sleeve (axial compression)
4.3.6	Plain pipe split sleeve (bending/tension)

Database	Type of unstressed grouted connection
4.3.7	Weld beaded split sleeve (axial tension)
4.3.8	Weld beaded split sleeve (axial compression)
4.3.9	Weld beaded split sleeve (bending/tension)
4.3.10	Partial weld bead split sleeve
4.3.11	Stud connector split sleeve
4.3.12	Stud/strap continuous sleeve

The screening criteria used in the preparation of the databases are as follows:

- inner member diameter $\geq$ 150mm
- grout strength at time of test $\geq$ 4 N/mm^2
- L/D ratio $\geq$ 0.9
- weld bead height and spacing (or equivalent for other types of connectors) to be the same for inner member (the brace) and the outer member (the sleeve).

IV 4.3.3 Summary of Existing Guidance

IV 4.3.3.1 General

Codified design guidance on pile/sleeve connections can be found in the UK Health and Safety Executive (formerly Department of Energy) Guidance Notes, API RP2A and DNV Rules for Classification of Fixed Offshore Installations. Several design equations for slip strength can be found in the literature.

IV 4.3.3.2 HSE Guidance Notes[4.2]

Section 60.5.4 of the HSE Guidance Notes, dealing with repairs, directs the reader to the rules for pile/sleeve connections (given in Section 22 of the Guidance Notes). These rules may be summarised as follows.

The characteristic bond strength of a grouted connection, with or without mechanical shear connectors, is given by:-

$$\sigma_{bc} = K\, C_L\, (9\, C_s + 1100\, h/s)\, \sqrt{\sigma_{cu}} \qquad \text{.... 4.3.1}$$

where

σ_{bc} is the characteristic bond strength in N/mm^2; σ_{cu} is the characteristic unconfined grout compressive strength in N/mm^2; C_L is the coefficient to account for the effects of grouted length to pile diameter ratio, C_s is the surface condition factor; h is the minimum shear connector outstand; and s is the nominal shear connector spacing. K is the stiffness factor defined as:-

$$K = \frac{1}{m}\left[\left(\frac{D}{T}\right)_g\right]^{-1} + \left[\left(\frac{D}{T}\right)_p + \left(\frac{D}{T}\right)_s\right]^{-1} \qquad \text{.... 4.3.2}$$

in which m is the modular ratio of steel to grout; D is the outside diameter; T is the wall thickness; and g, p and s are subscripts denoting grout, pile and sleeve respectively. The coefficient for grouted length to pile diameter ratio is tabulated as a function of L/D_p as follows:

L/D_p	C_L
2	1.0
4	0.9
8	0.8
≥ 12	0.7

The surface condition factor for shot blasted surfaces is specified as:-

h/s	C_s
≥ 0.005	1.0
< 0.005	0.6

The above equations may only be applied to connections which satisfy the geometrical ratio limits specified in Appendix A22.2.3 of the Guidance Notes. These limits are reproduced in Table 4.3.1, along with those of other codes discussed below.

The safety factor specified in the Guidance Notes is given as:-

<u>Grout Displacing Water</u>

Condition:	Safety Factor:
Extreme	4.5
Operating	6.0

<u>Grout displacing drilling mud or other similar material</u>

Condition:	Safety Factor:
Extreme	6.0
Operating	8.0

Code	$(D/T)_s$		$(D/T)_p$		$(D/T)_g$		L/D_p		h/D_p		D_p/s		h/s		w/h		σ_{cu} (N/mm^2)		Additional Limits
	L	U	L	U	L	U	L	U	L	U	L	U	L	U	L	U	L	U	
HSE[4.2]	50	140	24	40	10	45	2	-	0	0.006	0	8	0	0.04	1.5	3			
API[4.4]		80	0	40	7	45					0	8	0	0.10	1.5	3			
DNV[4.6]	18	140	10	60			2	-			2.5	8	0	0.10	1.5	3	17.25	110	σ_{cu} (h/s) ≤5.5
													0	0.04			4	90	$\sqrt{D_p T_p} \le s$

L = Lower limit
U = Upper limit

Table 4.3.1: Applicable limits of HSE, API and DNV formulae

IV 4.3.3.3 API RP2A[4.4]

The allowable bond strength for connections without shear keys should be taken as 0.138 N/mm^2 for loading conditions 1 and 2, and 0.184 N/mm^2 for loading conditions 3 and 4. Loading conditions 1 - 4 are defined in Clause 2.2.2 of API RP2A but briefly loading conditions 1 and 2 relate to maximum and minimum operational loads and loading conditions 3 and 4 correspond to extreme loads.

The allowable bond strength for connections with shear keys can be calculated from the following equations:-

- for loading conditions 1 and 2:-

$$\sigma_{ba} = 0.138 + 0.5 \left[\frac{h}{s}\right] \sigma_{cu} \qquad \text{.... 4.3.3a}$$

- for loading conditions 3 and 4:-

$$\sigma_{ba} = 0.184 + 0.67 \left[\frac{h}{s}\right] \sigma_{cu} \qquad \text{.... 4.3.3b}$$

where σ_{ba} is the allowable bond strength in N/mm^2.

These equations should be used in conjunction with the geometrical and material limits given in API RP2A[4.4], reproduced in Table 4.3.1.

The theoretical and experimental background to the above equation has been summarised in Reference 4.5.

IV 4.3.3.4 DNV Rules for Classification of Fixed Offshore Installations[4.6]

The characteristic bond strength of a continuous grouted connection is specified as:-

$$\sigma_{bc} = \mu\, C_L\, K'\, E_s \left[2\left[\frac{\delta}{D_p}\right] + \frac{\sigma_{cu}^{0.15}}{16} \sqrt{\frac{2T_p}{D_p}} \left[\frac{h}{s}\right] \right] \qquad \text{.... 4.3.4}$$

where K′ is a radial stiffness parameter defined as:-

$$K' = \left[\frac{D_p}{2T_p} + m\frac{2T_g}{D_p} + \frac{mD_s}{2(mT_s + T_g)} \right]^{-1} \qquad \text{.... 4.3.5}$$

C_L is defined as that in the HSE Guidance Notes; μ is the frictional coefficient between grout and steel and 0.7 is specified. For a split grouted connection, the characteristic bond strength can be estimated using Equation 4.3.4, but the radial stiffness K′ is calculated from:-

$$K' = \left[\frac{D_p}{2T_p} + m\frac{2T_g}{D_p} + \frac{m D_s}{2(mT_s + T_g)} + \frac{2L_b S_b}{\pi A_b} \right]^{-1} \qquad \ldots.\ 4.3.6$$

in which L_b is the bolt length; S_b is the bolt spacing; and A_b is the cross sectional area of one bolt. The resistance partial safety factors for ultimate limit state and progressive limit state are given as 3.0 and 2.6 respectively. The loading partial factors from the code should also be applied. The above equations are specified for use in the ranges defined in Table 4.3.1 above.

The background for this DNV rule has been summarised in Reference [4.7].

IV 4.3.4 Applicability of Existing Guidance to Strengthening/Repair Situations

IV 4.3.4.1 General

It has already been noted that unstressed grouted connections were initially developed for pile/sleeve application. In their application to the strengthening/ repair of tubular jacket members, three major differences should be borne in mind:

- Except for a telescopic joint application, the unstressed grouted clamp will be split to enable its installation around an existing member. The two halves of the clamp are joined by two rows of short bolts passing through flanges located along the split line. The effect of splitting the sleeve is to reduce its radial stiffness. It is generally accepted that this will impair the load carrying capacity compared to a continuous sleeve connection.

- Pile D/T ratios tend to be low (generally D/T $<$ 40) to allow for pile driving forces. The D/T ratios of jacket tubular members are typically in the range 30 to 60, and may be higher. The D/T ratio of the sleeve in a pile/sleeve connection also tends to be low where the jacket leg forms the sleeve. For jackets with pile clusters, the sleeve D/T can be very high. These (leg) sleeve D/T ratios encompass the requirements for strengthening/repair sleeve D/T ratios.

- Compared to pile/leg sleeve connections, the ratio of bending/axial loads is higher for clamps used in repairs.

It is therefore appropriate to concentrate on these aspects when appraising the applicability of the three codes, discussed in Section IV.4.3.3, in conjunction with available data. For example, the allowable code ranges for sleeve and tubular member (pile or brace) D/T ratios, and experimental data for axially loaded plain pipe specimens, are indicated in Figure 4.3.1. The corresponding data for weld bead specimens are illustrated in Figure 4.3.2. It is clear that there are considerable differences in the D/T ranges specified by the codes and that there are far fewer data available for split sleeves than continuous sleeves. The applicability of the codes have to be assessed on an individual basis because

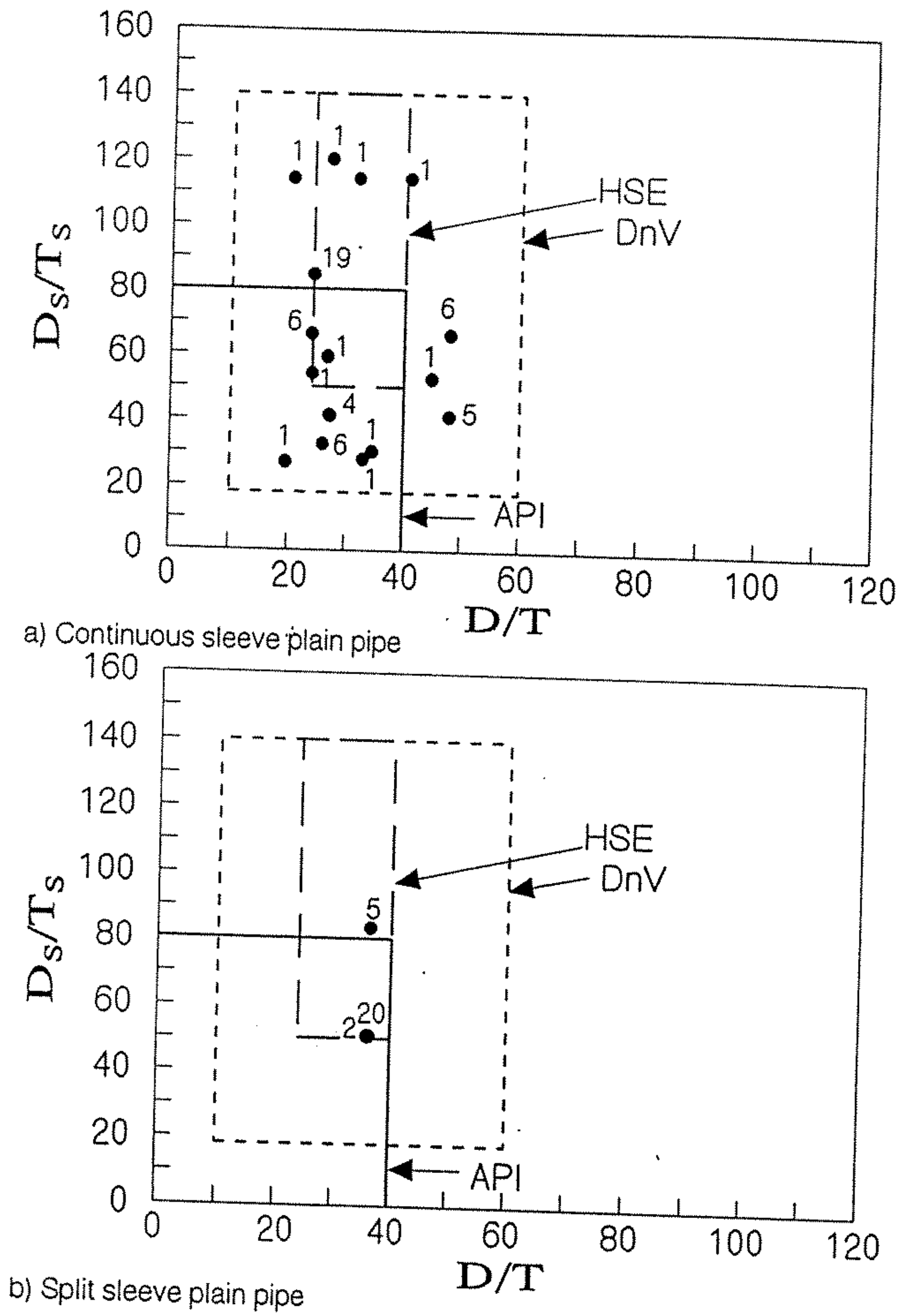
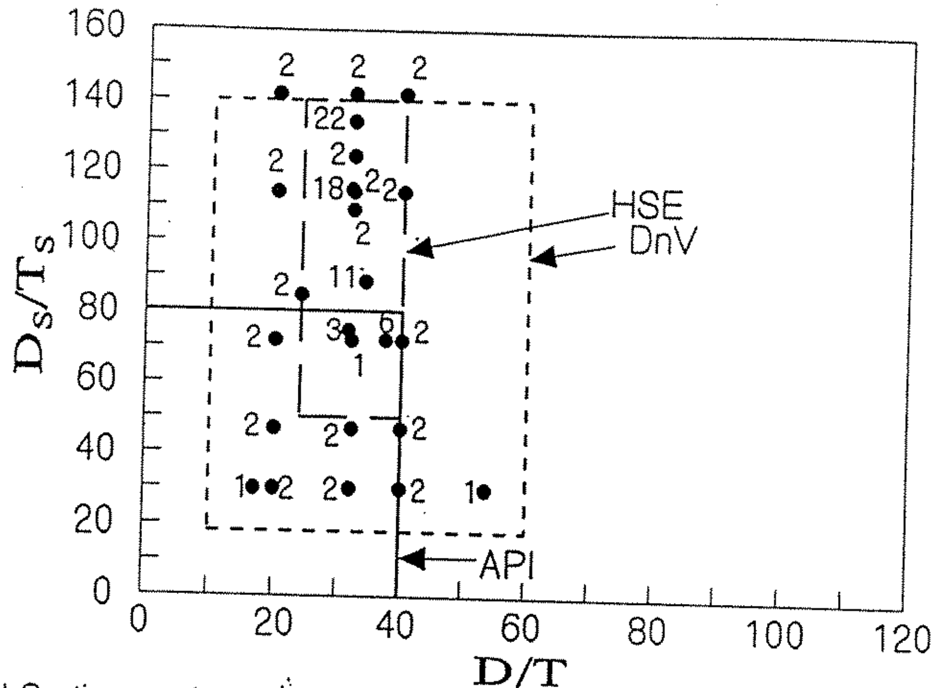
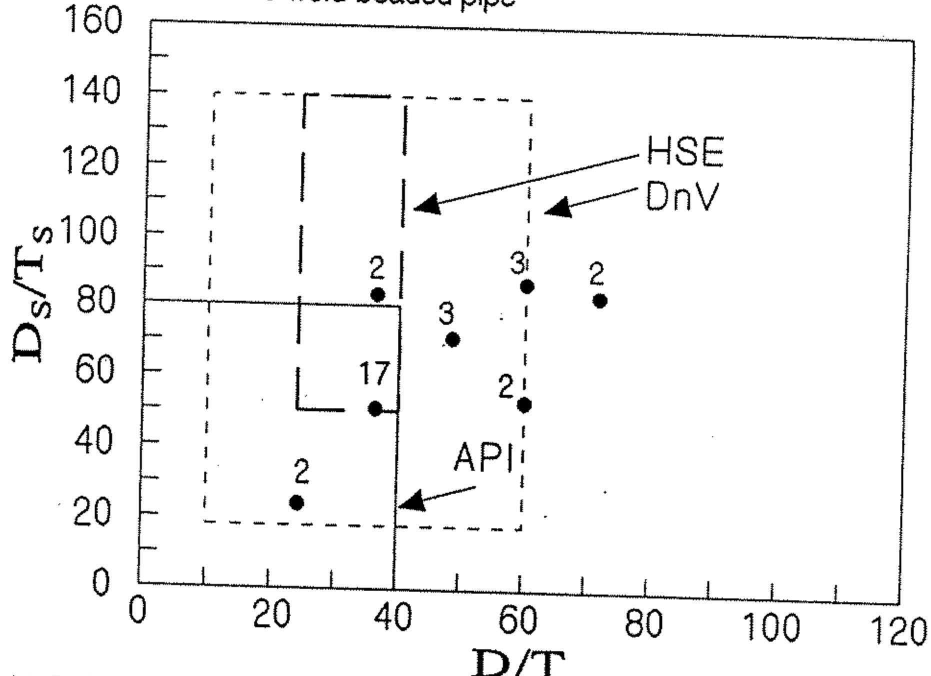


Figure 4.3.1: **D/T ratios for plain pipe sleeve and inner member: ranges of various codes and experimental data (N° of specimens indicated at each data point)**



a) Continuous sleeve weld-beaded pipe



b) Split sleeve weld-beaded pipe

Figure 4.3.2: **D/T ratios for weld-beaded pipe sleeve and inner member: ranges of various codes and experimental data (N^o of specimens indicated at each data point)**

of these differences and because the design formulations differ greatly in the effect of various parameters (eg. the grout cube strength, σ_{cu}). It is convenient, however, to recognise the broad division between plain pipe connections and weld-beaded connections.

IV 4.3.4.2 Formulations for Plain Pipe

Axial Loading

For each of the three codes (HSE, API and DNV) in turn, the results for continuous sleeve connections, split sleeve connections under tension and split sleeve connections under compression (Databases 4.3.1, 4.3.4 and 4.3.5 respectively) were filtered to retain only those satisfying the code range of applicability. In this context, tension and compression connections are defined by the state of stress in the member (see Figure 4.1.2 and Section IV 4.1.2.2). The ratios of test to predicted slip strengths were formed and the resulting frequency histograms are shown in Figure 4.3.3. The API ratios, averaging about 5, are much higher than the ratios of either HSE or DNV because API is an allowable stress code whereas HSE and DNV are limit state codes. The predicted slip strengths were calculated according to the codes, summarised in Section IV 4.3.3, but without invoking the explicit factors of safety given for HSE and DNV. It may be noticed that the DNV histogram captures many more data; this is due to the greater range of applicability of the code. The results of statistical analyses of the above strength ratios are given in Table 4.3.2 below. The characteristic ratios are calculated for 95% survivability at the 50% confidence level, see Section IV 1.2. When examining these results, it must be borne in mind that only the DNV rules make specific allowance for the existence of any split in the sleeve. The HSE and API rules strictly apply to continuous sleeves only.

A number of observations can be made with respect to the tabulated results:

i) for all three codes, the mean strength ratio of split sleeve connections in compression exceeds that of connections in tension. In the case of HSE and DNV, this exceedence is approaching a factor of 2.

ii) For all three codes, the mean strength ratio of split sleeve connections in compression exceeds the ratio for continuous sleeves.

iii) Only in the case of the DNV code, for which the effects of a split are modelled, does the mean strength ratio for split sleeve connections in tension exceed the continuous sleeve mean strength ratio.

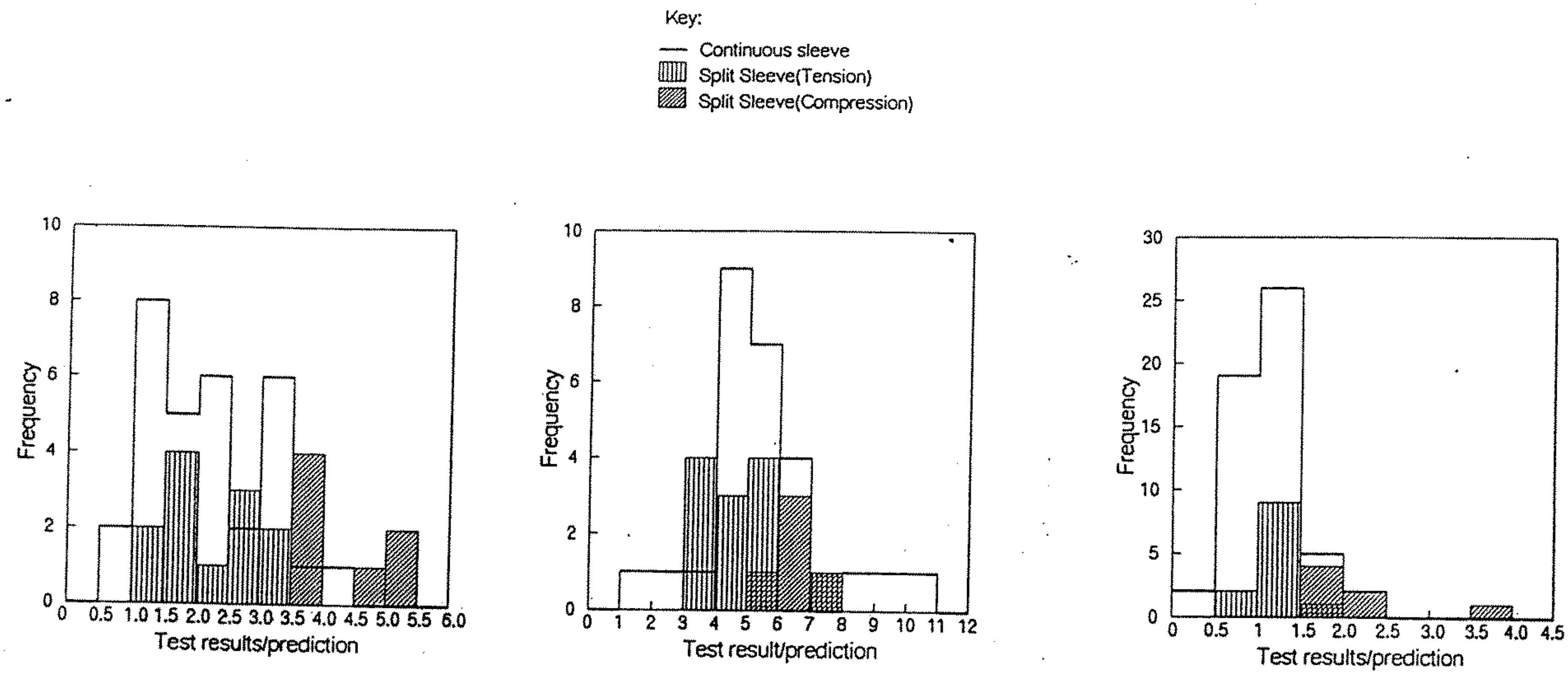
iv) For the characteristic ratios, corresponding observations to i) and ii) above can be made.

v) The characteristic ratio for split sleeve tension connections exceeds the characteristic ratio of continuous sleeve connections for all three codes (unlike the corresponding observation for mean ratios noted in iii) above in which this was only so for the DNV code).

Code	Type of Connection	No. of Data	Mean Ratio	STD of Ratio	COV	Characteristic Ratio
HSE	Continuous sleeve	23	2.210	1.023	0.463	0.504
	Split sleeve (tension)	12	2.149	0.702	0.327	0.962
	Split sleeve (compression)	7	4.213	0.793	0.188	2.840
API RP2A	Continuous sleeve	20	5.465	1.846	0.338	2.380
	Split sleeve (tension)	10	4.652	1.130	0.243	2.729
	Split sleeve (compression)	4	6.576	0.532	0.080	5.602
DNV	Continuous sleeve	54	1.155	0.392	0.253	0.507
	Split sleeve (tension)	12	1.275	0.278	0.218	0.805
	Split sleeve (compression)	7	2.187	0.635	0.290	1.087

( ) = state of stress in member

Table 4.3.2: **Statistical analyses of measured to predicted strength ratios for plain pipe connections (specimens satisfy code limits)**



a) HSE

b) API RP2A

c) DNV

Figure 4.3.3: Results for plain pipe connections satisfying limits of application for HSE, API and DNV

Before drawing conclusions from the above observations, it is appropriate to comment on the filtered databases which formed the basis of Figure 4.3.3 and Table 4.3.2. The extent of unfiltered data (with respect to code compliance) for plain pipe connections (continuous or split sleeves) is shown in Figure 4.3.1. The data is presented in terms of the tubular member and sleeve slendernesses. It should be noted that even if a test result lies within a particular code's slenderness boundary, it may not be included in the filtered database due to the possibility of another criteria not being satisfied. It may also be noted that both tension and compression data are included in the figure.

Nevertheless, despite the fact that relatively small numbers of data points remain after filtering, it would appear, on the evidence of the characteristic ratios set out in Table 4.3.2, that the existing guidance given for pile-sleeve connections can be safely applied to split sleeve plain pipe connections.

An examination of Figure 4.3.1(b) suggests that it is not possible to extend the range of applicability of present rules due to the general paucity of data. Indeed, it could even be mooted that more onerous limits than those given in Table 4.3.1 should be introduced for split sleeve connections. A more onerous limit for $(D/T)_s$ would be somewhat academic since sleeve D/T ratios used in repair situations tend to be similar to the D/T of the clamped jacket member, and thus limited to, say, 80 at the upper end of the range anyway. Neither is it considered necessary to impose a more onerous upper limit on member D/T ratio. This is because any effect of the split on tubular stiffness becomes proportionally less important as the tubular D/T ratio increases (this applies to both member and sleeve tubulars). Naturally, this assumes that bolt pretension is sufficient to keep the flanges at the split line in compression at all times, with no gap between the flanges being formed due to, say, applied out-of-plane bending moments.

Bending Loads

Results from plain pipe unstressed grouted connections subjected to pure bending or combinations of axial tension plus bending loads are summarised in Database 4.3.6. The ratio of σ_b/σ_a given in the table refers to the ratio of extreme fibre bending stress to axial stress in the brace member.

No failures were observed in the pure bending tests, though permanent set displacements did result (ie. extension on the tension side and compression on the compression side) due to yielding of the steel.

For those specimens subjected to combined axial tension and bending loads, the ratios of test to predicted axial strengths were formed for the three codes. The effect of bending loads was ignored in calculating the axial strength. The results are tabulated in Table 4.3.3. The results are also illustrated in Figure 4.3.4 in terms of the applied interface shear stresses. These have been calculated according to the equations given in Section III 4.2.5 of Part III. In the figure, additional results for pure tensile loading are included; these relate to split sleeve specimens of similar geometry (ie. the first 8 specimens of Database 4.3.4).

Specimen	Constant Parameters	σ_b/σ_a	Test/Predicted axial strength ratio		
			HSE	API RP2A	DNV
S1		0	1.868	5.109	1.395
S1A		0	1.587	7.065	1.830
S2		0	0.904	3.098	0.823
			Mean = 1.453	Mean = 5.091	Mean = 1.349
S13	K = 0.013	0.5	1.448	6.359	1.649
S14	L/D = 2.2	0.5	1.215	6.087	1.560
			Mean = 1.332	Mean = 6.223	Mean = 1.605
S11		1.0	1.352	5.870	1.523
S12		1.0	1.455	5.598	1.469
			Mean = 1.404	Mean = 5.734	Mean = 1.496
S9		2.0	1.618	6.359	1.666
S10		2.0	1.385	5.870	1.527
			Mean = 1.502	Mean = 6.115	Mean = 1.597
S15		0	1.884	4.076	1.419
S16	K = 0.010	0	1.760	4.348	1.490
			Mean = 1.822	Mean = 4.212	Mean = 1.455
S21	L/D = 2.2	1.0	2.058	5.978	2.012
S22		1.0	1.907	5.870	1.963
			Mean = 1.983	Mean = 5.924	Mean = 1.988
S23		0	1.667	5.000	1.474
S23A	K = 0.013	0	1.113	4.185	1.206
S24	L/D = 4.2	0	1.208	4.402	1.272
			Mean = 1.329	Mean = 4.529	Mean = 1.317
S29		1.0	1.323	5.000	1.440
S30		1.0	1.113	4.185	1.206
			Mean = 1.218	Mean = 4.593	Mean = 1.323

Table 4.3.3: Test to Predicted strength ratios of split sleeves in tension with or without bending (plain pipe specimens)

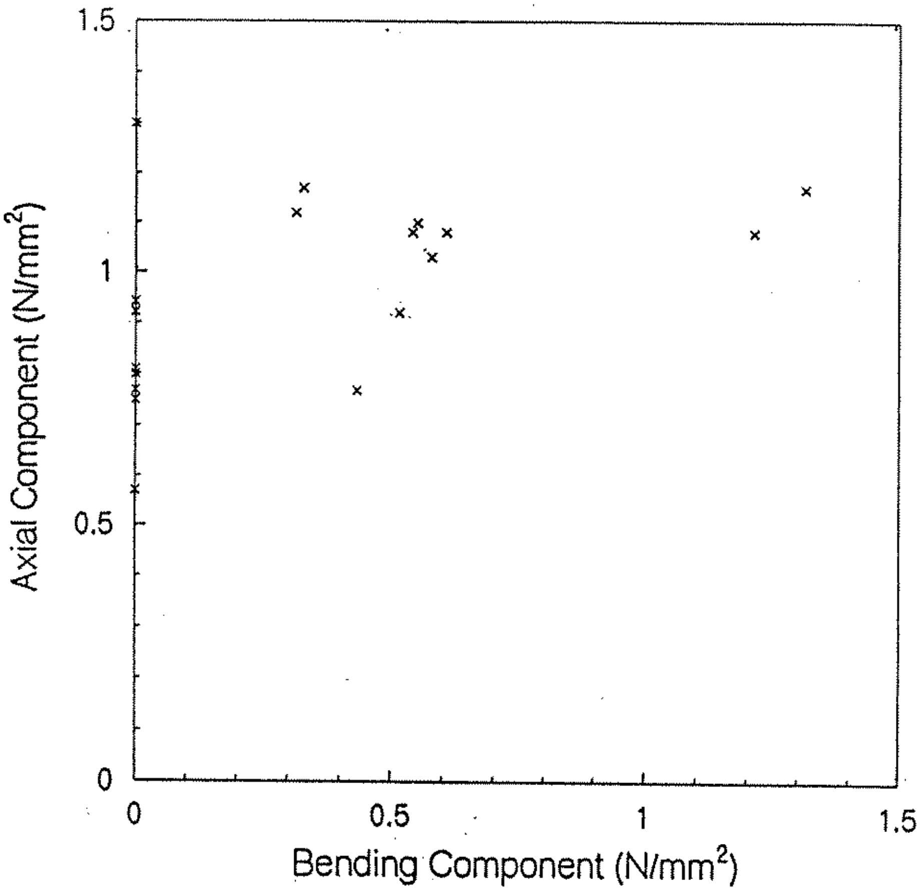


Figure 4.3.4: **Interaction between bending and axial components of interface shear stress (plain pipe connection)**

The results generally indicate that bending has no adverse effect on connection axial capacity. Indeed, there is sufficient evidence to postulate that the axial capacity is enhanced by bending. It may be conjectured that bending is transferred from the member to the sleeve by radial bearing forces so disposed as to give a couple equal to the applied moment. These bearing forces enhance the axial capacity by increasing the frictional component at the grout-to-steel interface.

For design purposes, it is conservative and therefore appropriate to ignore the effects of moments on axial slip strengths.

IV 4.3.4.3 Formulations for weld beaded pipe

Axial Loading

The data for weld-bead connections were analysed in a similar manner as carried out for plain-pipe connections.

For each of the three codes (HSE, API and DNV) in turn, the results for continuous sleeve connections, split sleeve connections under compression (Databases 4.3.3, 4.3.7 and 4.3.8 respectively) were filtered to retain only those satisfying the code range of applicability.

The ratios of test to predicted slip strength were formed and the resulting frequency histograms are shown in Figure 4.3.5. The results of statistical analyses of the ratios are given in Table 4.3.4.

An examination of Table 4.3.4 reveals that there is no significant difference in the mean strength ratio for split sleeves in tension and split sleeves in compression. This is in contrast to plain pipe results (Table 4.3.2) in which the compressive data were approaching twice the strength of tensile data. Indeed for ratios based on HSE and DNV, the weld bead compressive mean ratios are less than the tensile ratios.

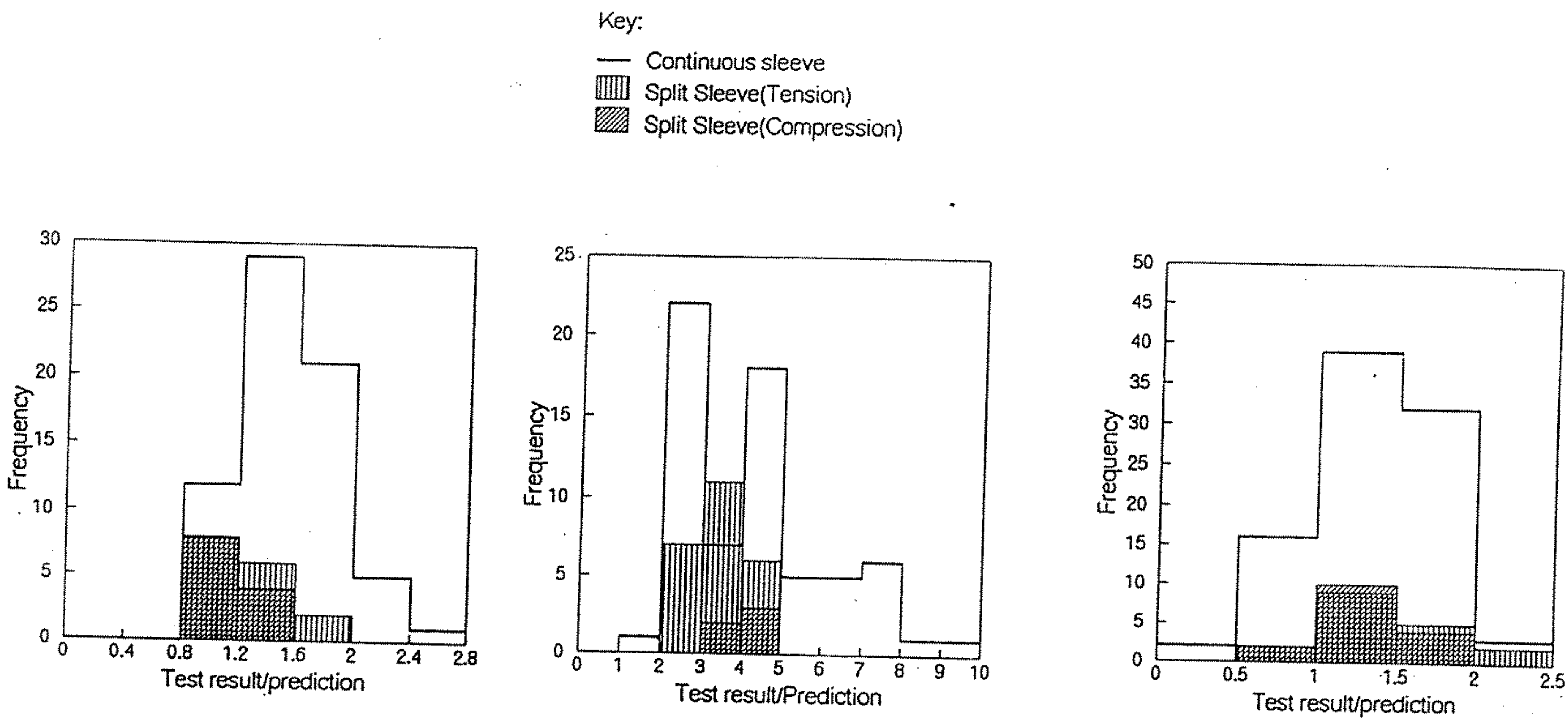
There is a further important difference between weld-bead connections and plain pipe connections. Whereas the introduction of a split into a plain pipe connection reduces its strength only marginally, there are large differences in strength between continuous and split sleeve weld-bead connections. (This is not immediately apparent for the DNV results in Table 4.3.4 as the DNV rules make specific allowance for any split in the strength formulations. It is not unexpected, therefore, that the DNV strength ratios for continuous and split connections should appear approximately equal.)

It may be observed in Table 4.3.4 that the standard deviations of the strength ratios for split sleeves are generally less than those relating to continuous sleeves. The derived characteristic strength ratios tend to mask the above observations made for the mean strength ratios. This is particularly so in the case of the API results where now the (characteristic) split sleeve strength ratios lie above the continuous sleeve strength ratio. However, it is considered that the characteristic results may be spurious due to the effects of there only being limited data and the data, such as it is, being concentrated at very few (D/T)s, D/T points, see Figure 4.3.2. It is most likely that the addition of a new variable (ie. the split) into an experimental programme will tend to increase the standard deviation, not reduce it. It is therefore conjectured that had the split sleeve database been larger, with a wider spread of variables (eg. D/T ratios) represented, the standard deviations would be larger, certainly not less, than the standard deviation of continuous sleeve results. To summarise, the characteristic ratios shown in Table 4.3.4 should not be relied upon.

Code	Type of Connection	No. of Data	Mean Ratio	STD of Ratio	COV	Characteristic Ratio
HSE	Continuous sleeve	51	1.503	0.345	0.229	0.932
	Split sleeve (tension)	6	1.331	0.268	0.201	0.862
	Split sleeve (compression)	14	1.127	0.149	0.132	0.876
API RP2A	Continuous sleeve	27	5.412	2.150	0.397	1.834
	Split sleeve (tension)	10	3.863	0.289	0.075	3.371
	Split sleeve (compression)	4	3.890	0.786	0.202	2.452
DNV	Continuous sleeve	69	1.421	0.294	0.207	0.935
	Split sleeve (tension)	16	1.408	0.355	0.252	0.812
	Split sleeve (compression)	14	1.357	0.178	0.131	1.057

( ) = state of stress in member

Table 4.3.4: Statistical analyses of measured to predicted strength ratios for weld bead connections (specimens satisfy code limits)



a) HSE

b) API RP2A

c) DNV

Figure 4.3.5: **Results for weld bead connections satisfying limits of application for HSE, API and DNV**

Taking, therefore, the mean strength ratios as a more robust basis for assessing the applicability of the existing guidance to split sleeves, the results of Table 4.3.4 may be normalised with respect to the relevant continuous sleeve result as shown in Table 4.3.5.

Code	Type of Connection	Normalised Ratio
HSE	Continuous sleeve	1.000
	Split sleeve (tension)	0.885
	Split sleeve (compression)	0.750
API RP2A	Continuous sleeve	1.000
	Split sleeve (tension)	0.71
	Split sleeve (compression)	0.72
DNV	Continuous sleeve	1.00
	Split sleeve (tension)	0.99
	Split sleeve (compression)	0.95

Table 4.3.5: Normalised strength ratio for weld-bead connections

Having accepted that split sleeve weld-bead connections:

- have similar strengths in tension and compression (in contrast to plain pipe connections)
- are significantly weaker than continuous weld-bead connections (again in contrast to plain pipe connection)

an explanation for this behaviour should be offered. Both observations have as their basis the fact that much larger hoop tensile stresses are induced in a weld-bead connection. This was recognised in Reference 4.1. The hoop stress in weld bead connections are mainly generated at loads around ultimate strength and are due to the dilation of the sleeve when relative sliding of the tubulars occurs past a grout wedge formed in front of the brace shear keys, see Figure 4.3.6.

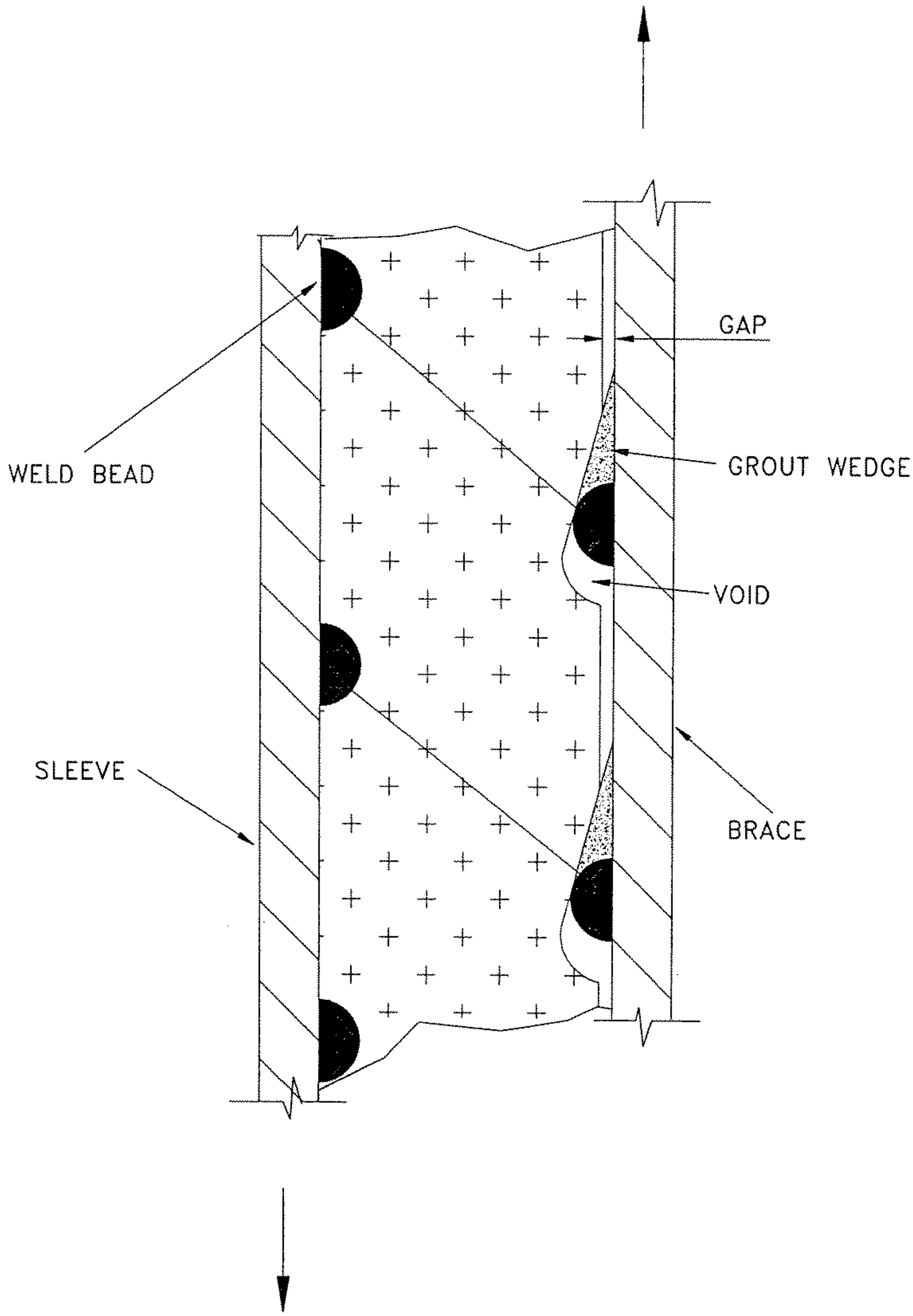


Figure 4.3.6: Failure mechanism of a weld-bead connection

Diametric measurements of split and continuous sleeve connections[4.1] have revealed the following:

- The average diametric displacements at working load, that is one sixth of ultimate, and at half ultimate load were similar for split and continuous sleeves, being approximately 1/2000th and 1/250th of the brace diameter at the two loads respectively.

- At ultimate load the average displacement for continuous sleeves was 1/40th of the brace diameter whereas that for split sleeves was greater at 1/17th.

It may be concluded from these measurements that the split does not affect the behaviour at working loads.

It is important that the induced hoop stresses do not prematurely open up the split. Otherwise the connection will lose part of its radial stiffness and therefore fail at a lower load than expected. If it is assumed, conservatively, that frictional effects can be ignored then the total outward radial force due to wedge action is κ P where κ is the aspect ratio of the wedge (triangle base/height) and P the axial load on the connection. This is equivalent to a radial pressure of $\kappa P/(\pi DL)$. The radial pressure induces a total (hoop) force of (radial pressure x DL), see Figure 4.1.1(c). This force is the required total precompression for both sides of the split taken together. Designating this total precompression as 2 P_s, it may be shown:

$$2 P_s = \kappa P . DL / (\pi DL)$$

$$\text{ie.} \quad P_s = \kappa P / (2\pi) \qquad \ldots.\ 4.3.7$$

The wedge aspect ratio has been observed as being around 5. Using $\kappa = 5$ in Equation 4.3.7 and a factor of safety of 1.2 leads to a required precompression force at each side of the split of 0.95 of the axial load on the connection. This compares favourably well with the value of 0.90 derived in Reference 4.1 (using a completely different model) and which has stood the test of time.

Note that the bolt loads will generally be required to be greater than the above precompression due to prying action more commonly associated with normal bolted connections, see Figure 4.3.7.

To summarise, it is recommended that reduction factors of 0.75 and 0.70 respectively are applied to the existing HSE and API rules for pile/sleeve connections (see Section IV 4.3.3) for application to split sleeve repair connections. DNV already specifically considers split sleeves and observing that the normalised ratios (in Table 4.3.5) are close to unity, no reduction factor is considered necessary. It is further recommended that the bolt/flange details at the split are designed to induce a precompression at each sleeve-to-sleeve junction of at least 0.95 times the axial load.

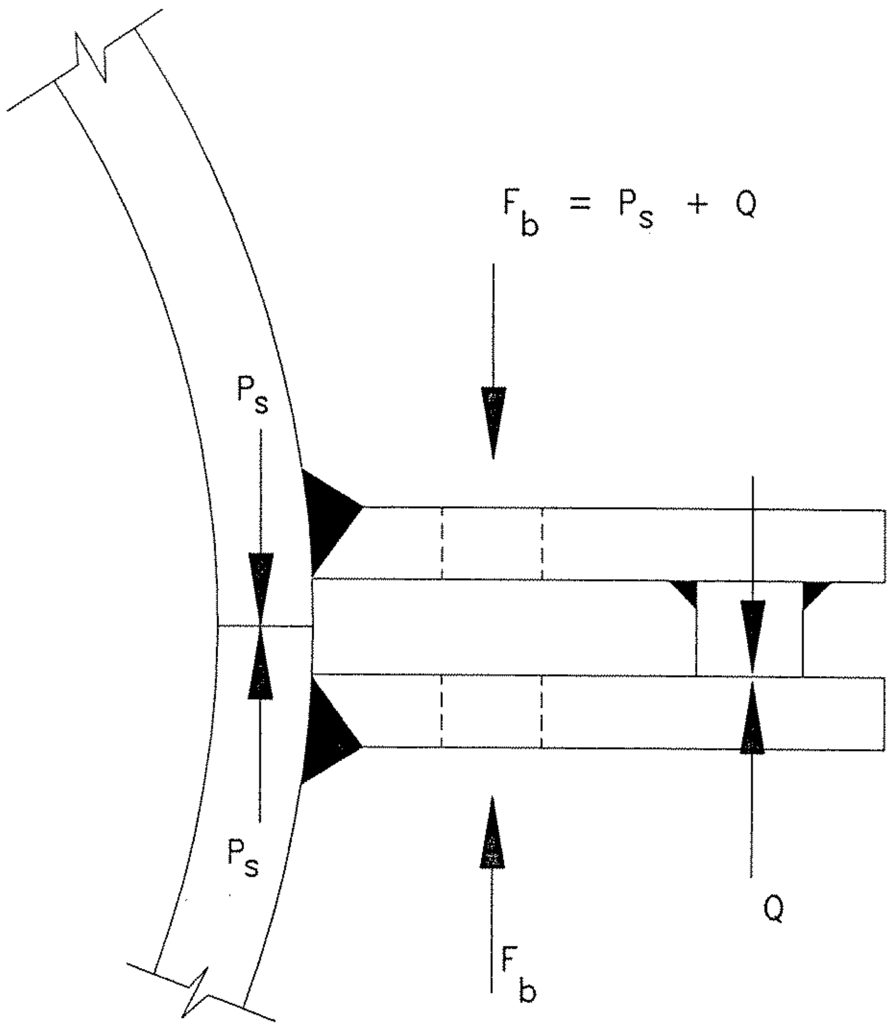


Figure 4.3.7: Typical flange detail

Bending loads

The results obtained from weld-bead connections subjected to combined bending and axial loading are given in Database 4.3.9. No data are available for weld-bead connections under pure bending.

The processed results are shown in Table 4.3.6 and in Figure 4.3.8 from which it may be inferred that the presence of bending loads does not impair the axial capacity of the weld-bead connections. This observation was also made with respect to plain pipe connections.

IV 4.3.5 Effect of Early Age Movements

The effects of early age movements, ie. those occurring during the grout curing period, are potentially the greatest for unstressed grouted connections than any other form of clamp connection. This is because for other forms of connection, all or a high proportion of the connection's capacity relies on friction; and the strength assigned to the frictional component is independent of early age movements.

Two scenarios should be considered, namely the placing of a clamp around an existing node and the use of a telescopic connection in a single member. The unrestrained movements that may be sustained between a member and clamp during the curing period relate to environmental loads incurred during the first 24 hours or so. The unrestrained movement may be estimated from the member stresses arising from such environmental loads. A sensible stress variation for the first 24 hours could be 10% of yield. This corresponds to a relative movement for steels having σ_y = 355N/mm^2 of 0.1 x 355 L/E$_s$ where L is the length over which relative straining can occur. For example, a clamp sited around a node can be considered as having a common origin with that node (relative movement is zero at origin) and thus L may be interpreted as the connection length. For L = 2000mm, the relative movement may be calculated as 0.34mm. However, in the case of a telescopic connection, L must be taken as the total length of the member concerned. A length of 15m is not untypical for which the relative movement is 2.5mm.

Specimen	Constant Parameters	σ_b/σ_a	Test/predicted axial strength ratio		
			HSE	API RP2A	DNV
T1	h/s = 0.0212 K = 0.013 L/D = 2.3	0.0	1.070	3.523	1.427
T2		0.0	1.219	4.130	1.509
			Mean = 1.145	Mean = 3.827	Mean = 1.468
T13		5.0	1.446	5.108	1.597
T14		5.0	1.162	4.225	1.171
			Mean = 1.304	Mean = 4.667	Men = 1.384
T11		1.0	1.064	3.512	1.409
T12		1.0	1.454	5.157	1.586
			Mean = 1.259	Mean = 4.335	Mean = 1.493
T9		2.0	1.194	4.075	1.452
T10		2.0	0.970	3.110	1.387
			Mean = 1.082	Mean = 3.592	Mean = 1.420
T15	h/s = 0.0212 K = 0.010 L/D = 2.3	0.0	1.727	4.567	1.911
T16		0.0	1.646	4.218	2.032
			Mean = 1.687	Mean = 4.393	Mean = 1.972
T21		1.0	1.827	4.857	1.979
T22		1.0	1.705	4.546	1.832
			Mean = 1.766	Mean = 4.702	Mean = 1.906
T23	h/s = 0.0212 K = 0.013 L/D = 4.2	0.0	1.135	3.536	1.363
T24		0.0	1.286	4.181	1.365
			Mean = 1.210	Mean = 3.859	Mean = 1.364
T29		1.0	1.132	4.253	1.191
T30		1.0	0.822	2.729	1.245
			Mean = 0.977	Mean = 3.491	Mean = 1.218
A1	h/s = 0.0333 K = 0.0095 L/D = 2.0	0.0	1.282	4.240	1.075
A2		0.0	1.288	4.222	1.180
A7.2		0.0	1.330	4.300	1.320
			Mean = 1.300	Mean = 4.254	Mean = 1.192
A8		0.416	1.591	5.248	1.379
A9		0.416	1.562	5.174	1.226
			Mean = 1.577	Mean = 5.211	Mean = 1.303

Table 4.3.6: Test to predicted strength ratios of split sleeves in tension with or without bending (weld-bead specimens)

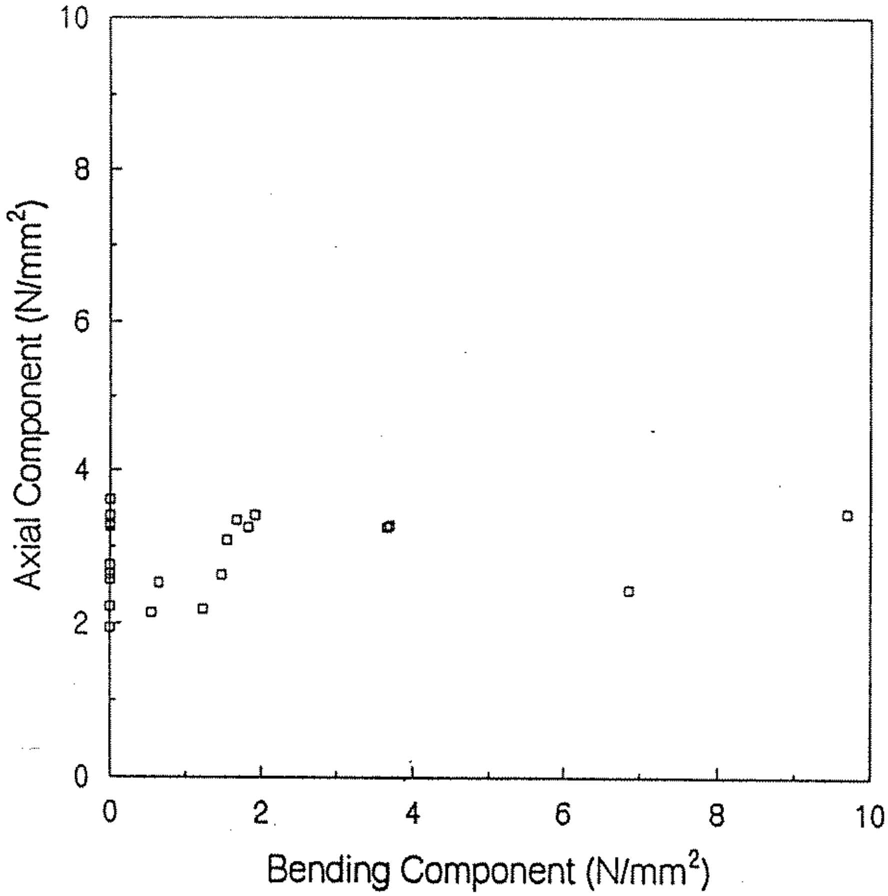


Figure 4.3.8: **Interaction between bending and axial components of interface shear stress (weld bead connections)**

The effect of early age movements has been studied experimentally, References 4.1 and 4.3. Initial input displacements of up to $\pm$ 3.0mm were used in these tests (a pile diameter of 324mm was used throughout). It was found that the early age movements did not affect the strength achieved by the connections whether they were plain pipe or weld-bead specimens. However, the stiffness of cycled specimens were up to 6 times smaller than uncycled specimens. At the end of the cycling period, slotting behaviour in which the weld bead appears to move in a slot was observed for those specimens cycled at displacements of over $\pm$1mm. The specimens cycled at $\pm$0.56mm did not exhibit any sign of damage.

Since the stiffness of a connection is important in that a loss of stiffness would lead to a loss of member load, it seems necessary to restrain the relative movement that would otherwise occur to below $\pm$ 0.6mm. This need not apply to those cases where explicit analyses are undertaken accounting for reduced connection stiffness. Nevertheless, the designer should also be made aware of slotting as being indicative of the early stage of fatigue failure, see Section IV 4.7. On the basis of the above calculations it appears unnecessary to restrain relative movements of clamps around nodes, even where this is feasible.

IV 4.3.6 Variant Forms of Shear Keys

IV 4.3.6.1 Partial circumference weld beads

Weld beads extending only partially around the circumference may be selected where it is not practical to weld all around a member.

The available data for such connections are presented in Database 4.3.10 where the only variables are the % of the circumference welded and, inevitably, the grout strength. This Database may be supplemented by specimens T1 and T2 from Database 4.3.7. Specimens T1 and T2 are geometrically similar although slightly longer, but are 100% welded around the circumference.

It has been conjectured in Reference 4.1 that the actual h/s factor is reduced in direct proportion to the amount of circumference welded. Thus, defining C_c as a factor to be multiplied by the actual h/s to give an equivalent h/s:

$$(h/s)\ \text{equivalent} = C_c\, h/s \qquad \text{.... 4.3.8}$$

where: $C_c = L_c/\pi D$

L_c = total length of weld bead at a circumference

D = diameter of brace member.

The equivalent h/s is to be used instead of the actual h/s in Equations 4.3.1, 4.3.3 and 4.3.4 in Section IV 4.3.3.

The justification for the simple design approach is illustrated in Figure 4.3.9 for the various design codes covered in Section IV 4.3.3. It shows sensitivity plots, with respect to the % of welded circumference, of the test result to code prediction ratio. For all three codes, the plots show a reasonably flat response.

It should be noted that three quarters of the specimens indicated a slip of up to a little over 0.5mm in the load deflection graphs well before ultimate load was reached. It was conjectured that this was due to failure of the bond in the unbeaded region of the tubular. This is plausible given that the stiffness response of a plain pipe connection is greater than that of a weld-beaded connection. This being so, the use of such partial weld connections cannot be generally recommended.

IV 4.3.6.2 <u>Discrete stud connections</u>

Discrete studs as shear keys are not normally selected where weld beads can be applied. However, studs may provide an acceptable solution where underwater welding of beads is not viable.

A limited test programme, reported in Reference 4.1 and summarised in Database 4.3.11, has identified the following modes of failure:

- bond slip failure of the grout/steel interface (as for weld bead specimens)
- local deformation failure of the tubular wall at stud locations
- combined bending/shear failure of the studs.

The bond slip failure stress may be found by using an equivalent h/s in the slip equations of Sections IV 4.3.3.2 to IV 4.3.3.4, as appropriate, where:

$$(h/s)\ \text{equivalent} = \frac{\phi\ h_s}{S_h\ S_\ell} \qquad \text{.... 4.3.9}$$

in which ϕ = stud diameter (ie. width)
h_s = stud height (ie. projection into annulus)
S_h = circumferential stud spacing
S_ℓ = longitudinal stud spacing

This equation is based on the projected area of the equivalent bead being equal to that of the studs.

There are too few data to derive firm design formulae for the other two modes of failure although bounded expressions are given in Reference 4.1.

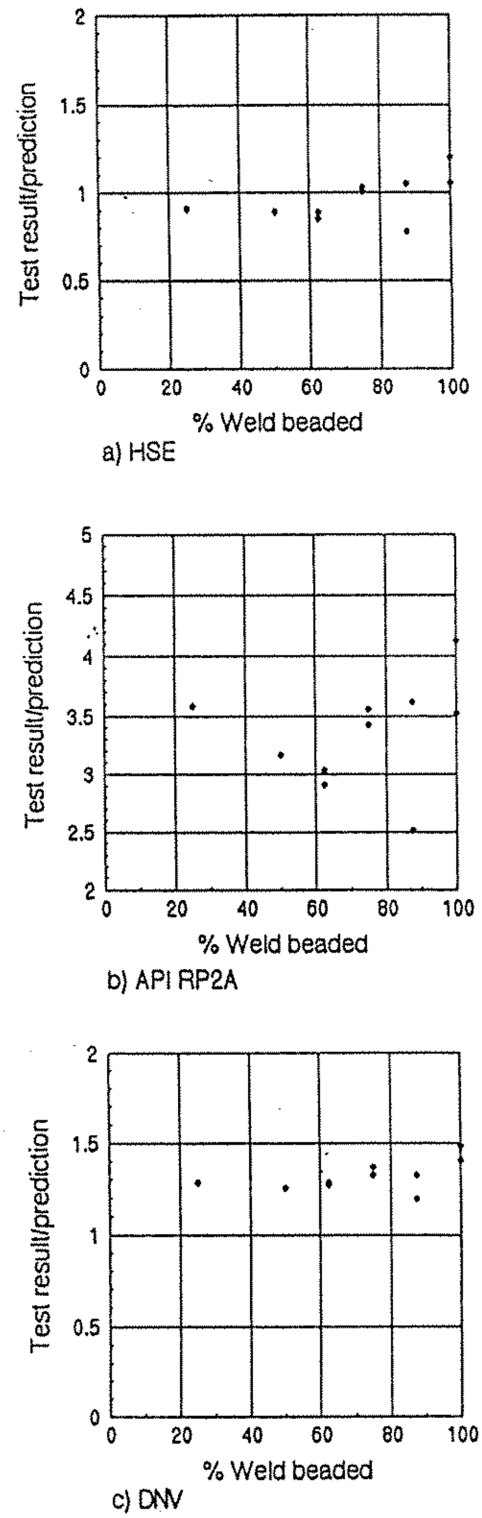


Figure 4.3.9: **Sensitivity plots for partial circumference weld beads**

To prevent local deformation of the tubular wall, which could precipitate more general buckling, the following equation should be observed:

$$\phi \quad < \quad \frac{2.3\ T}{\Gamma_T} \sqrt{\frac{f_y}{f_{ys}}} \qquad \ldots.\ 4.3.10$$

where T is the thickness of the tubular
f_{ys} is the yield strength of stud
Γ_T is the factor of safety for this failure mode with a recommended value of 1.4.

The derivation of Equation 4.3.10 is based on ensuring yield occurs first in the stud rather than in the tubular, yielding in the tubular being estimated from standard solutions for a couple applied to the centre of a circular plate.

The final mode of failure, involving the studs themselves, will limit the load transfer through restriction of the stud contribution but not of the plain pipe component which remains unaffected. By assuming an elliptical interaction curve for stud failure by combined bending/shear, and by assuming the load on any one stud is triangular with the load concentrated towards the stud/tubular junction, the following single stud capacity can be derived:

$$P_s \quad = \quad \left[\left(\frac{4\sqrt{3}}{f_{ys}\,\pi\,\phi^2} \right)^2 + \left(\frac{2h_s}{f_{ys}\,\phi^3} \right)^2 \right]^{-½} \qquad \ldots.\ 4.3.11$$

This stud capacity is to be associated with the area $s_h\ s_l$. The equation may be rewritten as:

$$P_s \quad = \quad \frac{\pi\,\phi^2 f_{ys}}{4\sqrt{3}}\ [1 + (0.91 h_s/\phi)^2]^{-½} \qquad \ldots.\ 4.3.12$$

Alternatively, if the characteristic stud shear strength is available, P_{Sc}, with the studs prepared under the same conditions as to be used in practice, it will be more economical to use:

$$P_s \quad = \quad P_{Sc}\ [1 + (0.91\ h_s/\phi)^2]^{-½} \qquad \ldots.\ 4.3.13$$

in lieu of Equation 4.3.12.

The allowable bond strength must therefore also be limited by:

$$\sigma_{bc} \quad = \quad P_s/S_h S_l \qquad \ldots.\ 4.3.14$$

The allowable connection capacity is found from the stud transfer capacity plus the plain pipe component, both suitably factored. For example, in the case where HSE formulation is being adopted, the following relationship holds:

$$\sigma_{bc} = \frac{P_s/(S_h S_l)}{\Gamma_s} + \frac{9KC_L C_S\sqrt{\sigma_{cu}}}{\Gamma_b} \qquad \text{.... 4.4.15}$$

where K, C_L, C_S, σ_{cu} and Γ_b are defined in Section IV 4.3.4.

Γ_s is a factor of safety on stud yield which may be taken as 1.70 for extreme conditions and 2.25 for operating conditions.

and all other terms are as set out above.

All three modes of failure should be considered in the design. However, it must be emphasised that only 4 test results are presently available and the above equations can only be regarded as tentative. They cannot be applied outside the following ranges of stud geometry without further verification:

Stud height/diameter ratio	$0.33 \leq h_s / \phi \leq 2.0$
Stud longitudinal spacing ratio	$0 \leq D / S_l \leq 3.0$
Stud hoop spacing ratio	$0 \leq D / S_h \leq 16$
Stud height ratio	$0 \leq h_s / D \leq 0.07$

The studs should be placed symmetrically around the brace member, and care should be taken to avoid positioning studs opposite the split in the sleeve.

It is recommended that presently, ad hoc tests on specific proposed arrangements are carried out. A greater research effort would be required to investigate the following areas of uncertainty:

- Effects of stud spacing on:

  i. brace wall bending (interaction of studs)
  ii. grout slip failure plane interactions

- Optimum h_s / ϕ ratio

- Fatigue performance

- Strength of stud/brace weld.

IV 4.3.6.3 Stud/strap connectors

A relatively recent development is the attachment of a strap collar or collar segments to a brace member by using studs, References 4.8 and 4.9. The new repair method (the 'grouted stud/strap connection') combines two proven techniques, namely grouted pile-sleeve connection technology and stud friction welding. By friction welding studs through a pre-drilled collar, a shear key with similar characteristics to a weld bead can be formed subsea on the jacket member. The sleeve would be provided with weld beads, as normal. The general scheme is illustrated in Figure 4.3.10. The three-studded collar segments are intended to be used with a semi-automatic stud welding machine, described in Reference 4.9.

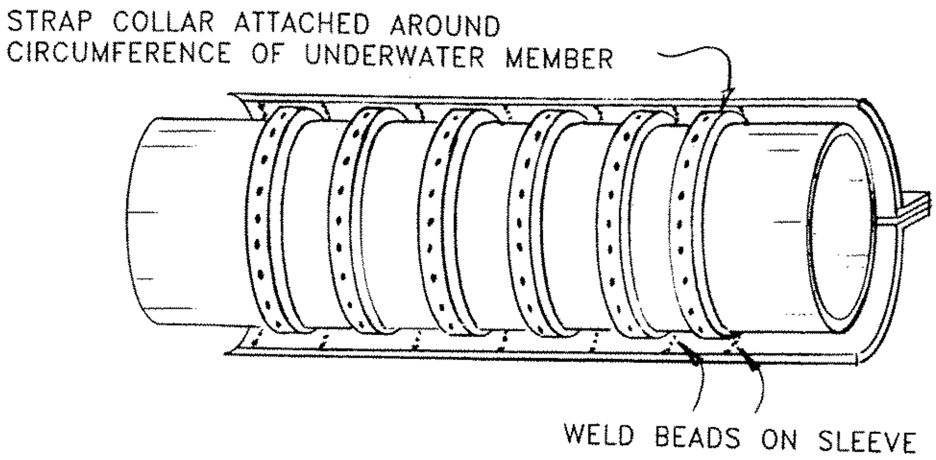
The repair technique may provide an acceptable solution where the underwater welding of weld beads is not viable. It is a patented system. It is claimed that the repair technique offers potential benefits in three areas compared to stressed grouted clamps, namely: weight, installation schedule and post installation inspection requirements.

The background research for the grouted stud/strap connection is reported in Reference 4.9. A programme of small and large scale tests has been conducted to demonstrate:

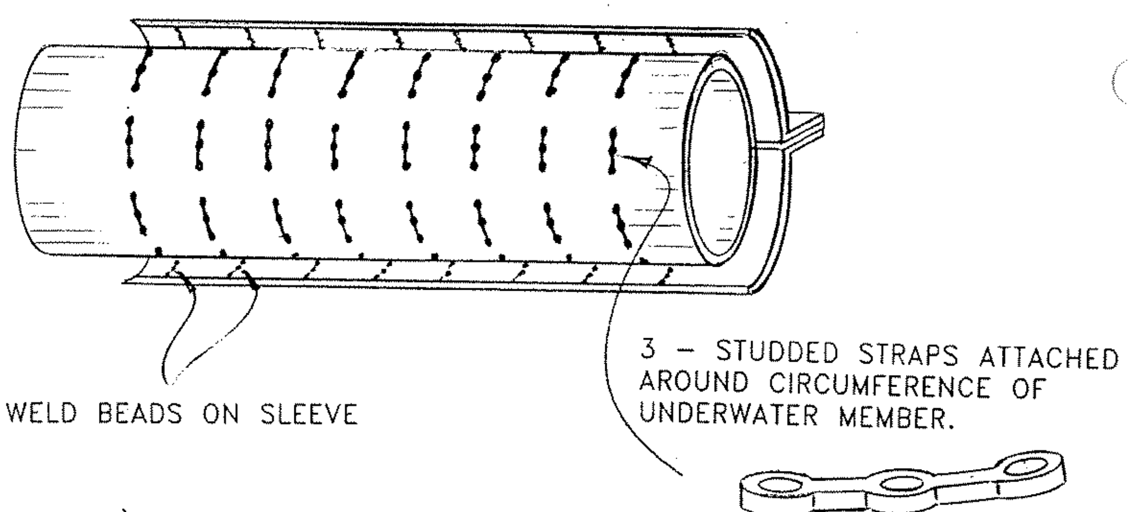
- static shear strength of stud/parent metal friction welds for specimens including strap
- fatigue endurance limit of through-strap friction welded studs
- ultimate capacity of grouted stud/strap connections in comparison to that of conventional weld beaded configurations.

In addition to the above, preliminary welding trials were carried out to optimise geometries of the strap and stud for welding. Some 32 single-stud and 23 three-stud stud/strap specimens were tested in the static trials which indicated that failure was associated with the stud shaft rather than the fusion zone and at a load which comfortably exceeded the tensile strength of the stud $/\sqrt{3}$. The 15 fatigue specimens indicated that a stress range threshold of 70N/mm^2 exists, and that the BS5400 Part 10 'W' class S-N curve may be adopted conservatively for design purposes.

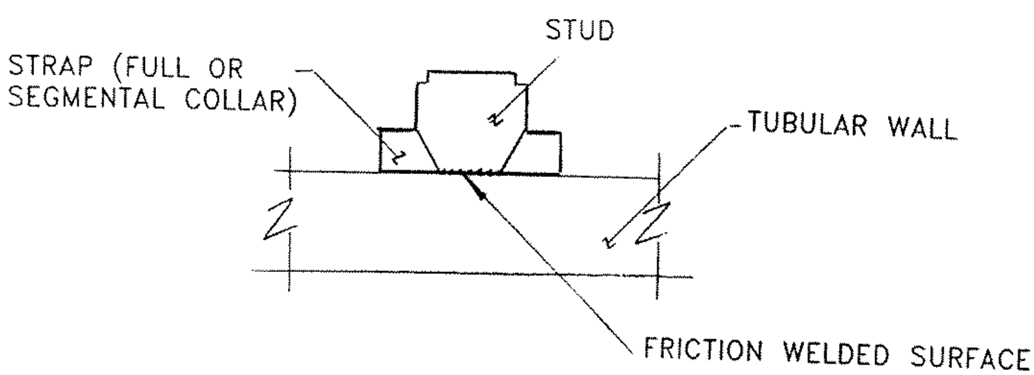
Two large scale grouted stud/strap tubular specimens were tested, with the objective to confirm the behaviourial similarity of stud/strap and weld beads as shear keys. Details of these tests are given in Database 4.3.12. (A third specimen, with only weld beads, was also tested under the programme, see specimen No. 3 in Database 4.3.3.) All three specimens generated similar load-displacement curves under axial loading. This indicates that provided local failure in the strap or at the stud/tubular interface is avoided by appropriate design, the design equations established for weld beaded connections may be adopted for stud/strap connections.



a) Split sleeve, full collar



b) Split sleeve, segmental collar



c) Details of stud / strap

Figure 4.3.10: The stud/strap connection

IV 4.3.6.4 Epoxy-aggregate coatings

A preliminary investigation into the axial strength of grouted pile/sleeve connections with steel surfaces coated with epoxy containing crushed flint is reported in Reference 4.10. For the conditions investigated, the results of the four tests indicate that:

- failure was associated with the grout, not the steel-epoxy or epoxy-aggregate interfaces, thereby demonstrating the effectiveness of the coating
- the coated connections exhibit more ductility than, and match or exceed the strength of, equivalent weld bead connections.

It was conjectured that a coated connection can utilise grout more effectively than an equivalent shear key connection because the grout's strength is mobilised more evenly along the length of the coated connection than is the case with discrete positions of weld beads.

An equivalent shear key outstand-to-spacing ratio (h/s) was calculated for the epoxy-aggregate surfaces and this averaged out as 0.0183 (corresponding to shear keys 10mm high at a spacing of 546mm) for the four specimens.

Although the technique looks promising for pile-sleeve connection technology, its use in repair/strengthening situations appears to be limited. The primary difficulty is in applying the coating underwater; no suitable equipment presently exists for mixing and applying the epoxy or for applying the aggregate (even if this in principle was attainable). Thus, the use of coated connections in underwater repair situations must be dismissed. For above water repair applications, there still may be difficulties in applying the coating. There is, also, the problem with the paucity of data. It is most likely that specific tests would be required if this technique ever found a potential application in a repair/strengthening situation.

IV 4.3.7 **Recommendations for Further Work**

The following items requiring further study with respect to unstressed grouted connections are based on observations made during the preparation of this Section IV 4.3. No attempt has been made to prioritise the following items or assess the extent of work implied by each. The items are:

- As mentioned in the introduction (Section IV 4.3.1), there are presently on-going joint industry initiatives reappraising unstressed grouted pile/sleeve connections. As such split sleeve connections lie outside the scope of these studies. It is recommended that as and when new pile/sleeve formulations are developed and published, their relevance to split sleeve (repair) connections is assessed.

- It has been shown during axial loading tests that the presence of weld beads causes diametrical expansion of the sleeve at failure and this causes high bolt loads in split sleeve connections. It is recommended that split sleeve bolt load distributions are studied more fully. Studies should examine bolt load distribution as a function of sleeve and member D/T ratios, weld bead height, grout strength and length of connection. The non-linear distribution longitudinally should also be considered.

- No tests have been carried out on weld bead connections under pure bending. It is recommended that such tests are conducted for split sleeves as a confirmatory and precautionary measure. The combined effect of clamp splitting forces and diametrical expansion will apply a significant load to the end bolts.

- Additional information is required on early age movements. It is recommended that initially a more thorough study is made on the likely movements to be incurred during the grout setting and early curing period. This study should include within its remit the accuracy of weather forecasting over the period as grouting operations are tied to a suitable weather window. If suitable weather windows can be identified, early age movements could be constrained to acceptable values. However, further tests are required to examine such aspects as effect of member diameter and weld bead height (for constant h/s).

- Further work on discrete stud connectors is required if this technique is to be used. This would most likely be on an ad hoc testing basis.

- Further developmental work is required for stud/strap connections, eg. on early age movement effects, if this technique is to be used.

IV 4.3.8 **<u>Summary</u>**

The axial slip strength of unstressed grouted connections, whether they be continuous or split sleeved and whether they be plain pipe or weld beaded, can be established by referring to existing formulations (HSE, API or DNV, see Section IV 4.3.3) and applying overall reduction factors. The recommended overall reduction factors are given in Table 4.3.7.

Code	Plain Pipe	Weld bead	
		Continuous sleeve	Split sleeve
HSE[4.2]	1.0	1.0	0.75
API[4.4]	1.0	1.0	0.70
DNV[4.6]	1.0	1.0	1.0

Table 4.3.7: Reduction factors to apply to code formulations

There is evidence that bending loads do not impair the axial capacity, at least for bending loads inducing bending stresses up to twice the axial stress. Plain pipe specimens did not fail under pure bending.

The flange bolts in split sleeve connections need to be pretensioned to induce precompression at the mating surfaces of the sleeve. For weld bead split sleeves, the bolt loads are recommended to be sufficient to induce a sleeve precompression (at each split) of 0.95 times the axial load on the connection. (A separate check is required to ensure the clamp does not open up under bending loads.)

It is recommended that, unless other data is available, early age axial movements should be restricted to 0.6mm. This has implications for telescopic joints but most likely not for clamps located around existing nodal joints.

Other forms of shear keys and partial circumference weld beads may be acceptable. Guidance is given in Section IV 4.3.6.

Topics for further study are itemised in Section IV 4.3.7.

IV 4.4 SLIP STRENGTH OF STRESSED GROUTED CLAMPS

IV 4.4.1 Introduction

The major means by which a stressed grouted clamp transfers load is by frictional resistance at the grout/steel interface, although bond at the interface also plays a part. There are three methods which have been used, at least in laboratory trials, to impose the normal forces required to develop frictional resistance. These methods, the first two of which are illustrated in Figure 4.4.1, are:

- Studbolts which are stressed after the grout annulus has cured sufficiently.
- A secondary annulus formed on the inner surface of the saddle which is pressurised with grout after the primary annulus has cured sufficiently.

- Use of expansive grouts.

However, only the first method has been used offshore and as such bolted stressed grouted clamps are investigated in more detail than the others.

IV 4.4.2 Summary of Existing Guidance

IV 4.4.2.1 General

Codified design guidance on stressed grouted clamps is only given in the UK Health and Safety (formerly the UK Department of Energy) Guidance Notes. Several design formulae for slip strength are given in the literature.

IV 4.4.2.2 UK Health and Safety Guidance Notes[4.2]

This document has highlighted several key areas related to the grout annulus, although no slip strength formula is specified. The maximum value of the frictional coefficient, in the absence of experimental data, between grout and steel interfaces is required to be not higher than 0.25. This frictional coefficient is for grit blasted, water jetted and/or wire brushed steel surfaces. Safety factors of not lower than 1.7 and 2.25 are recommended for extreme and operating conditions. Factors affecting tensioned bolts are described. No limits on the allowable bond strength are specified.

IV 4.4.2.3 OTH 88 283[4.1]

This report has proposed an equation for slip strength. The proposed equation was developed using least square analyses based on absolute differences. The characteristic strength was calculated as:-

$$P_c = (0.38 \times 10^{-3} A C_s + 0.22 F_n C_s') \left[1 + 33 \left(\frac{T}{D}\right)\right] \quad \text{.... 4.4.1}$$

where P_c is the characteristic slip strength per effective slip surface (kN); F_n is the total studbolt load (kN); C_s and C_s' are the surface condition factors for the bond component and for the frictional component respectively; A is the bond area of the slip surface being mobilised (mm²); T is the member thickness (mm) and D is the member diameter (mm).

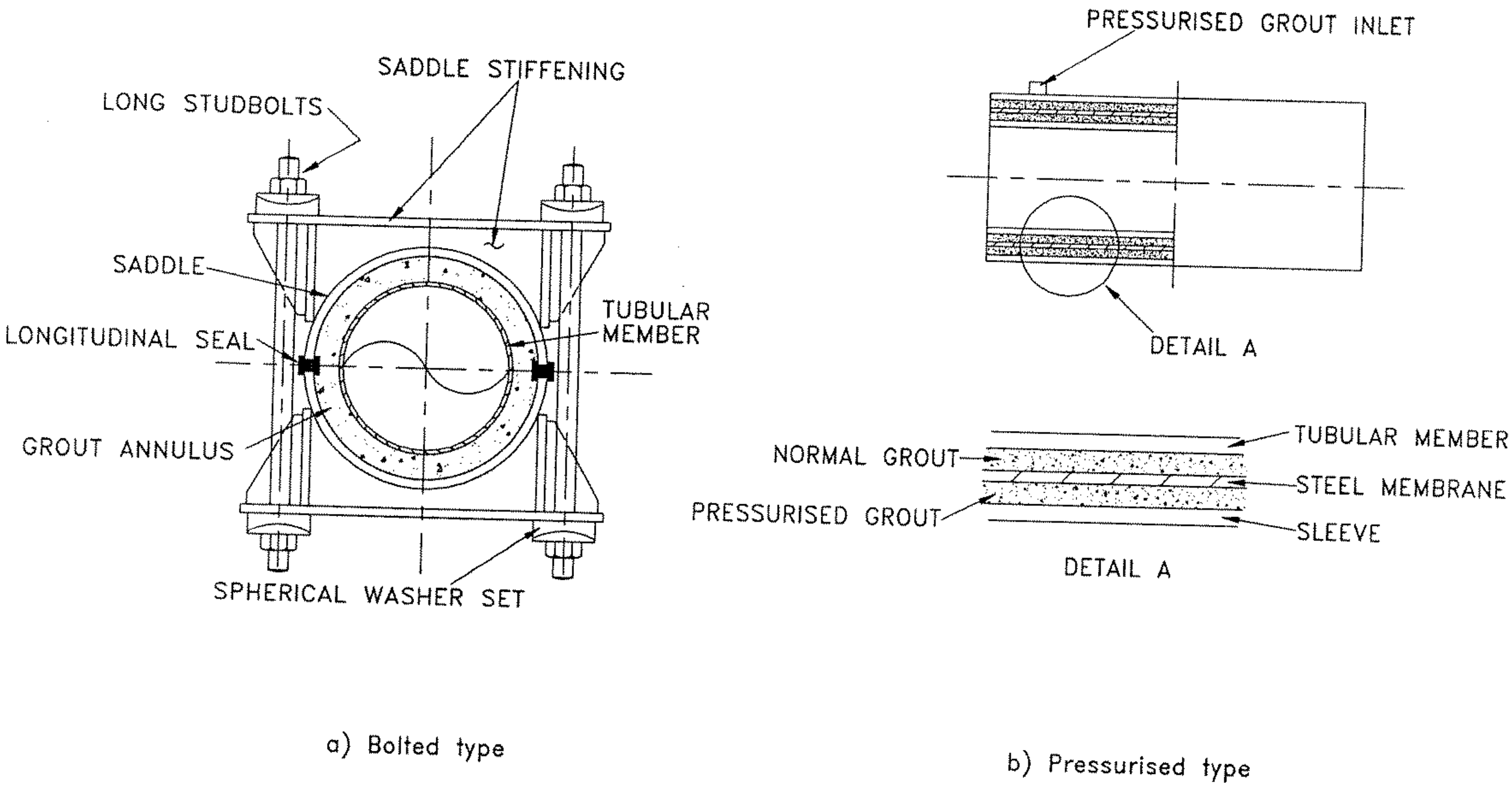


Figure 4.4.1: Bolted and pressurised types of stressed grouted clamps

The above equation was based on the reported experimental data. Altogether 10 test specimens of long bolted clamps were used. The effect of clamp stiffness reflected in the equation is based on limited data. It is worthy to note that data from three tests on a single specimen were used in the least square analysis even though any bond would be broken in the first test. The characteristic strength is defined for a 95% probability of survival with 95% confidence level. The above equation should only be used within the following limits:-

$$0.5 \leq L/D \leq 2.0$$
$$20 \leq D/T \leq 50$$
$$20\text{mm} \leq \Phi$$
$$\sigma_{cu} \geq 40 \text{ N/mm}^2 \text{ @ 28 days}$$

where Φ is the diameter of the long studbolts and σ_{cu} is the grout cube strength.

The safety factors on the bond strength are specified as 4.50 and 6.00 for extreme and operating conditions respectively. The corresponding factors on the frictional strength are 1.70 and 2.25.

IV 4.4.2.4 Elnashai, Carroll and Dowling[4.11]

The most recent reference published by the above group gives an expression for predicting the frictional strength for both pressurised (double skin annulus) and expansive grout clamps. The equation was calibrated by both experimental and finite element results though no calibration constants are included in the paper. The equation takes account of two factors affecting frictional resistance, namely length to diameter ratio and radial stiffness.

IV 4.4.2.5 Grundy and Foo[4.12]

The frictional resistance equation in this reference was developed for expansive grout clamps. Both experimental and numerical studies were conducted. The frictional strength is given as:-

$$\sigma_f = \mu \, p \, C_L \qquad \text{.... 4.4.2}$$

Where σ_f is the frictional strength (N/mm^2); μ is the frictional coefficient; p is the normal pressure (N/mm^2); and C_L is a length correction factor.

For surfaces with mill scale, protected with varnish, and for surfaces which have been shot blasted, the frictional coefficient was specified to be 0.8 and 1.1 respectively. The length factor C_L was calibrated by finite element and cylindrical shell bending analysis methods.

IV 4.4.2.6 Critique of existing guidance

The main source of information on which stressed grouted clamps have been designed is OTH 88 283, Reference 4.1. As for mechanical clamps, that document was a great advance but suffers from the same shortcomings in the analysis of stressed grouted clamps:-

- The curve fitting was based on minimising the absolute differences rather than the percentage differences.
- The development of the slip strength equation was carried out in stages rather than using a multi-variate fitting program.
- The 95% probability of survival was calculated at the 95% confidence level rather than the 50% confidence level.

In addition to the above, the following comments can be made:-

- In OTH 88 283, the analysis included three tests from each specimen despite the fact that the bond component of strength would be lost after the first test.
- Since the publication of OTH 88 283, new test data have become available, albeit not of the bolted type of stressed grouted clamp. Nevertheless, useful information can be obtained from such data with appropriate normalising techniques.
- No length factor effect is given in the formulations in OTH 88 283 despite its recognition in unstressed grouted clamps.

It is therefore appropriate to compile a new screened database and to carry out a regression analysis on the data using modern methodologies. This is the subject of Sections IV 4.4.3 to IV 4.4.6.

IV 4.4.3 **Database**

The database on stressed grouted clamp test results is presented in Database 4.4.1. Altogether 23 test specimens are recorded in this database, of which 11 specimens are for the bolted type, 10 for the pressurised grout type and 2 for the expansive grout type.

The bolted type of specimens were taken from OTH 88 283[4.1]. As mentioned above, each specimen was tested more than once and the first three tests for each specimen was used in the analysis presented in OTH 88 283. Here, only the first result for each specimen is considered on the basis that the bond component would be necessarily impaired after the first test. All bolted specimens in the database had shot blasted surfaces, except for specimen 11/1 which had a mill scale surface condition.

The following screening criteria were applied in the compilation of the database:-

- clamped member diameter $\geq$ 150mm
- grout cured at same temperature as grout in clamp
- grout cube strength $\geq$ 10N/mm^2
- maximum load defined as peak of load displacement curve
- consistency between tabulated results and text.

The data from the Imperial College group (on double annuli and pressurised grouted specimens) were not fully reported in the references, particularly with respect to the effective clamp lengths. Discussions were therefore held with the authors to ascertain this information.

The compiled data are further filtered for the purpose of statistical analysis as described below. This is to avoid inadvertent modelling bias being introduced in the derived slip strength equation.

IV 4.4.4 General Form of Slip Strength Equation

The slip strength of a stressed grouted clamp consists of two distinct components, i.e. bond strength at the grout/steel interface and friction developed due to a pressure normal to the interface. Under a sufficiently large pressure, the friction component becomes the main contributor to the slip strength. Apart from the pressure, the slip strength is influenced by several other factors:

- surface condition of the steel which interfaces with the grout;
- length to diameter ratio (L/D);
- radial stiffness of the clamp.

Nondimensional surface factors C_s and C_s' are introduced for bond and frictional components respectively to account for various steel surface conditions. For a given pressure load, the radial stiffness may affect the pressure value on the grout/member interface in two ways. Firstly the radial deformation of the clamp leads to different pressure levels on grout/steel interfaces. The pressure on the grout/steel interface is also influenced by the axial load in the existing member, as further radial deformation is induced by the Poisson effect. The radial stiffness is related to the bond strength in the strength equation of plain grouted connections. On the whole, the slip strength of stressed grouted clamps with existing members under tension increases with a larger thickness to diameter ratio. Since all the stressed grouted specimens have very stiff saddles, the radial stiffness is only represented by the member thickness to diameter ratio.

For long connections, both bond and friction components of the slip strength have been found to decrease exponentially with the length to diameter ratio[4.13]. A 10% reduction of slip strength was found numerically when the length to diameter ratio was changed from 1.0 to 2.0. This numerical finding is also supported by test results for bolted types of clamps. Hence the length to diameter ratio is retained in the slip strength equation. A linear reduction factor is incorporated in the slip strength equation on the basis that length to diameter ratios of practical connections lie within a fairly modest range, say 1.0 to 3.0.

From the above arguments, the slip strength equation takes the following form:-

$$\sigma_s = (C_s\,\sigma_b + C_s'\,\mu\,p)\left[1-\eta_1\left(\frac{L}{D}\right)\right]\left[1+\eta_2\left(\frac{T}{D}\right)\right] \qquad \ldots.\ 4.4.3$$

where μ is the frictional coefficient; p is the pressure per unit interface area, σ_b is the bond strength. η_1, η_2, μ and σ_b are the curve fitting constants to be specified. For the tubular members in the test specimens which have been shot blasted, the surface factors C_s and C_s' are taken as 0.6 and 1.0 respectively for this standard surface condition (in conformity with the values in OTH 88 283 and the UK HSE Guidance Notes).

IV 4.4.5 Development of Mean and Characteristic Formulations

Each test result is closely scrutinised to ensure that the slip strength equation is not influenced by some extreme case. All the data points in the database are shown in Figure 4.4.2. The higher slip strength values of the expansive grout clamps G2 and H2 are due to the fact that expansive grout has a higher frictional coefficient. Frictional coefficients of expansive grout for surfaces with mill scale protected with varnish and for shot blasted surfaces were quoted to be 0.8 and 1.1 respectively. These values represent roughly a 100% increase over the frictional coefficients of the grout material used in the bolted clamps and pressurised grout clamps. These two sample points were therefore excluded from the statistical analysis. There is only one test result for stressed grouted clamps with (L/D) equal to 0.5 (specimen 5/1). As this length to diameter ratio falls outside the normal design range, it was decided to remove this test result from the data analysis. Specimen 11/1 was also excluded due to the different surface condition employed and the suspicion that yielding of steelwork had occurred.

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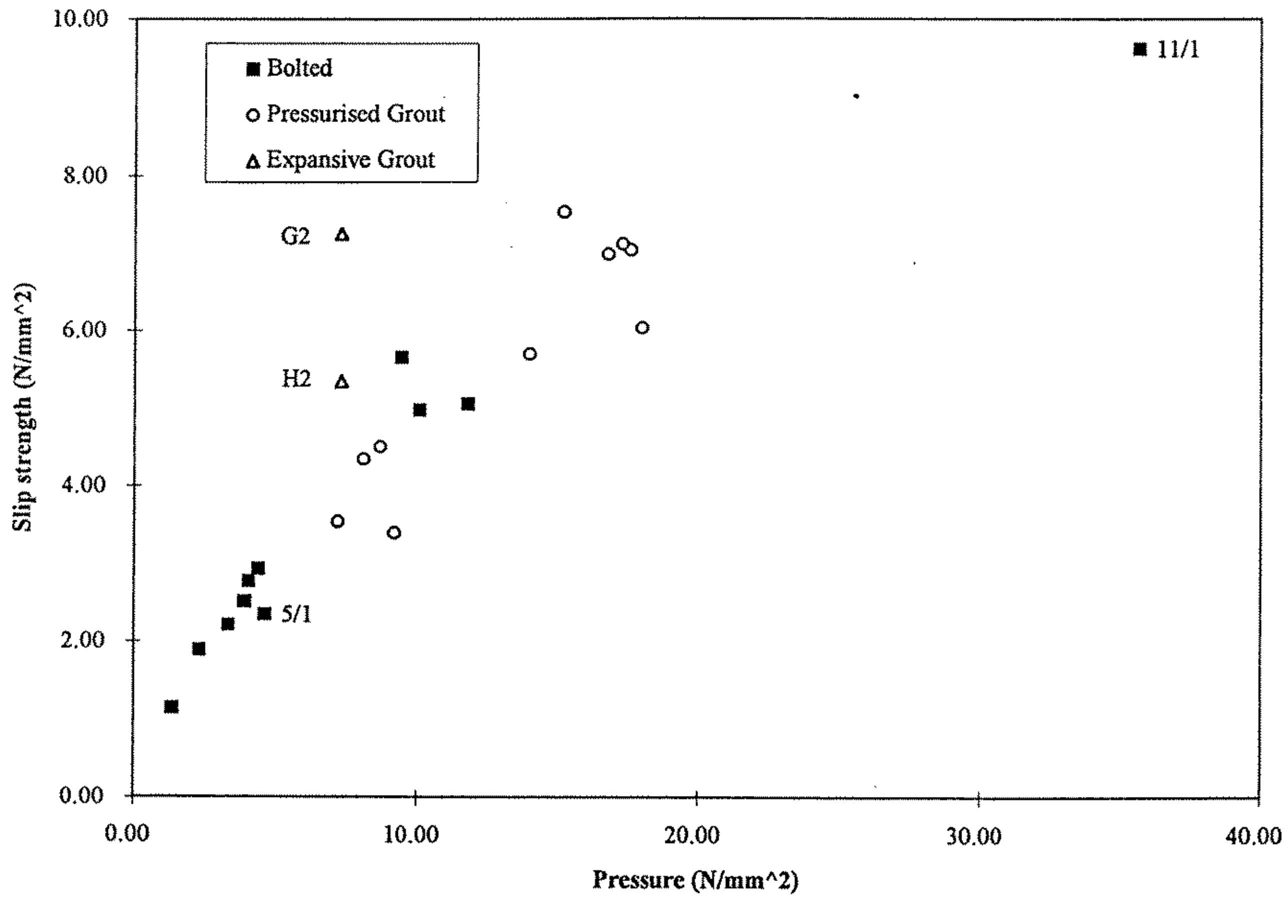


Figure 4.4.2: Tested slip strengths and pressures of stressed grouted clamps

The bond strength (for unstressed grouted connections) has been found to be related to grout cube strength. However the grout cube strengths of the stressed grouted clamps were not clearly documented. The bond strength in a stressed grouted clamp only accounts for a small percentage of the total slip strength. It is therefore reasonable not to retain the cube strength in the strength equation. However designers should be aware of the applicability of the strength equation. Database 4.3.1 concerns plain pipe, continuous sleeve, unstressed grouted connections. Excluding those specimens within this database having low grout strengths or very high sleeve D/T ratios, an average bond strength of 0.98N/mm^2 is indicated. Ideally, any proposed strength equation for stressed grouted connections should degenerate, at least approximately, to this value at zero bolt load.

Only two L/D ratios were tested for the filtered database of the bolted type specimens; one specimen (4/1) relates to L/D = 2.33 and all others to L/D = 1.10. Specimen 2/1 has identical geometry and a similar interface pressure to specimen 4/1. Comparing the frictional coefficients (= slip strength divided by the interface pressure) of these two specimens, it is found that the coefficient is reduced by a factor of 0.87 as L/D changes from 1.10 to 2.33. Assuming that the effect of L/D is linear, this corresponds to a factor of approximately 0.90 when L/D changes from unity to 2.0. Comparisons can also be made for the pressurised grout clamps. For these specimens, changing L/D from 0.9 to 1.9 results in two factors, ie. 0.68 (for specimens A2 and B2) and 0.71 (for specimens C2 and D2). These two factors are very similar and it may therefore be inferred that for a given clamp type, the effect of L/D is constant and that the two types of clamp differ with respect to the effect of L/D. However, with such few data, such inferences must be tentative and therefore a conservative approach is adopted here. It is recommended that the reduction from L/D = 1 to 2 for bolted clamp types be taken as 0.85 (compared to the inferred test result of 0.90 above). This leads to a length correction term of:

$$\left[1 - 0.13\left(\frac{L}{D}\right)\right]$$

It is now assumed that the final slip strength equation for bolted stressed grouted clamps is of the following form:-

$$\sigma_s = (C_s\,\sigma_b + C_s'\,\mu\,p)\left[1-0.13\left(\frac{L}{D}\right)\right]\left[1+\eta_2\left(\frac{T}{D}\right)\right] \quad \text{.... 4.4.4}$$

A nonlinear regression analysis, based on percentage differences, of only the bolted stressed grouted clamps with a constant L/D ratio of approximately unity gives:

$$\sigma_s = (1.055\,C_s + 0.392\,C_s'\,p)\left[1-0.13\left(\frac{L}{D}\right)\right]\left[1+11.959\left(\frac{T}{D}\right)\right] \quad \text{.... 4.4.5}$$

The ratio between tested slip strength and predicted slip strength is defined as the modelling bias. The mean and standard deviation of this modelling bias for all bolted stressed grout clamps are calculated to be 1.002 and 0.060 respectively. The mean strength equation gives lowest and highest modelling bias values of 0.92 and 1.08 respectively (see Figures 4.4.3 and 4.4.4).

Figures 4.4.5 to 4.4.7 were plotted to demonstrate the goodness of fit and the effects of some key parameters on the modelling bias. As the strength equation explicitly accounts for the effects of (L/D), (T/D) and pressure, the modelling bias shows no apparent dependency on these parameters.

For a sample size of 9, the characteristic strength equation is given as:-

$$\sigma_{sc} = (0.949\ C_s + 0.353\ C_s'\ p) \left[1-0.13\left(\frac{L}{D}\right)\right]\left[1+11.959\left(\frac{T}{D}\right)\right] \qquad \ldots.\ 4.4.6$$

For the tested clamps in Database 4.4.1, the grout sealing details were such that there was no continuity of grout between the two halves forming each clamp. Thus, the radial pressure, p, in Equation 4.4.6 can be deduced directly from the total bolt load, F_n, as:-

$$p = F_n / DL \qquad \ldots.\ 4.4.7$$

where D is a reduced diameter to account for the gap between the two halves of the grout ring.

For modern clamps, in which there is a continuity of grout at the split line, F_n should be interpreted as the effective total bolt load, taking proper account of grout ring effects on reducing the contact pressure at the inner grout/steel interface.

IV 4.4.6 <u>Surface Condition Effects</u>

The surface condition factors C_s and C_s' on the slip strength of stressed grouted clamps are likely to be similar to that of plain grouted clamps and mechanical clamps. However, there are insufficient data to confirm this conjecture.

IV 4.4.7 <u>Effect of Early Age Movements</u>

No test has been conducted on a stressed grouted connection which has been subjected to displacement cycling during the grout setting period.

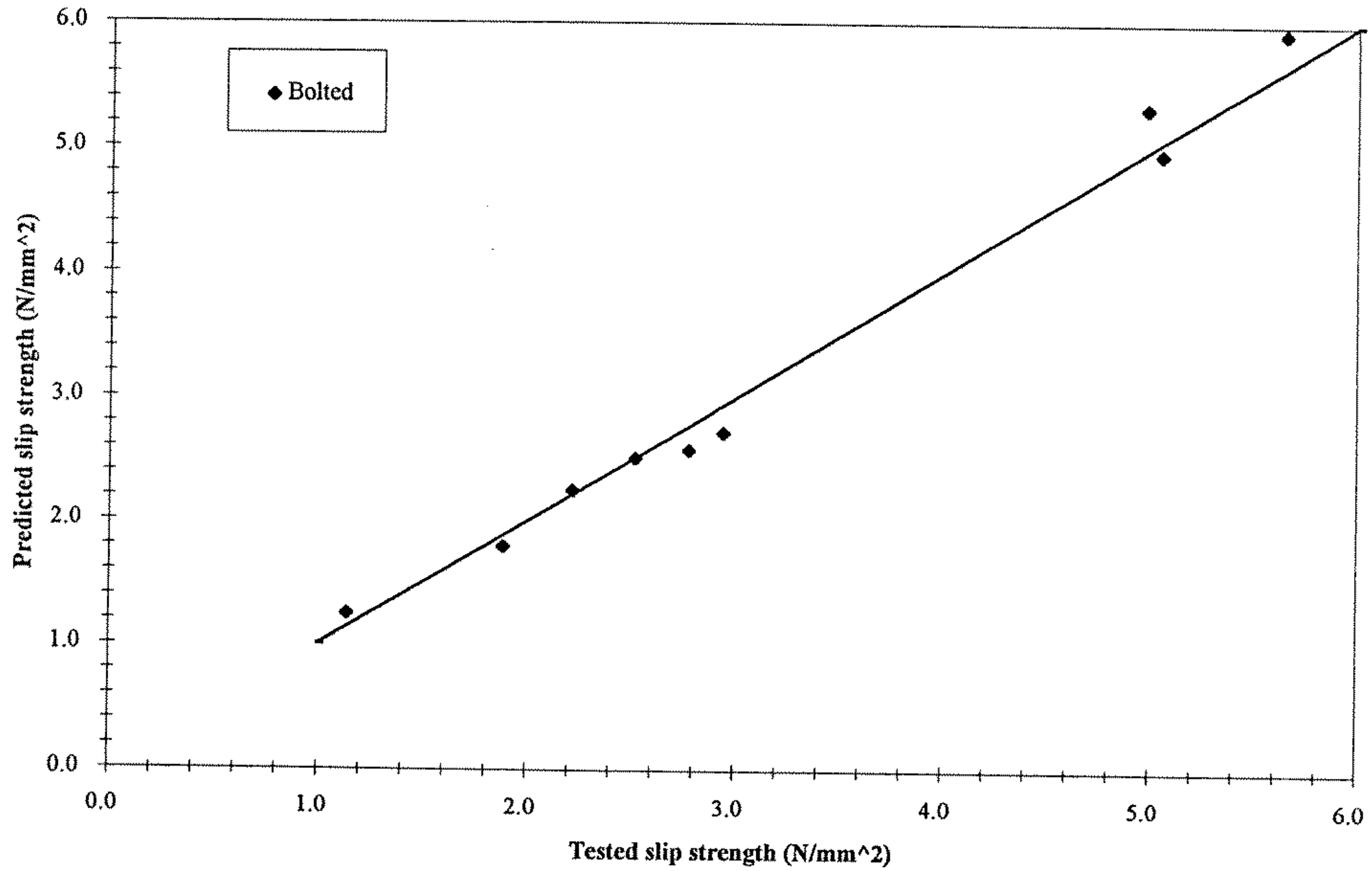


Figure 4.4.3: Tested and predicted slip strengths of stressed grouted clamps

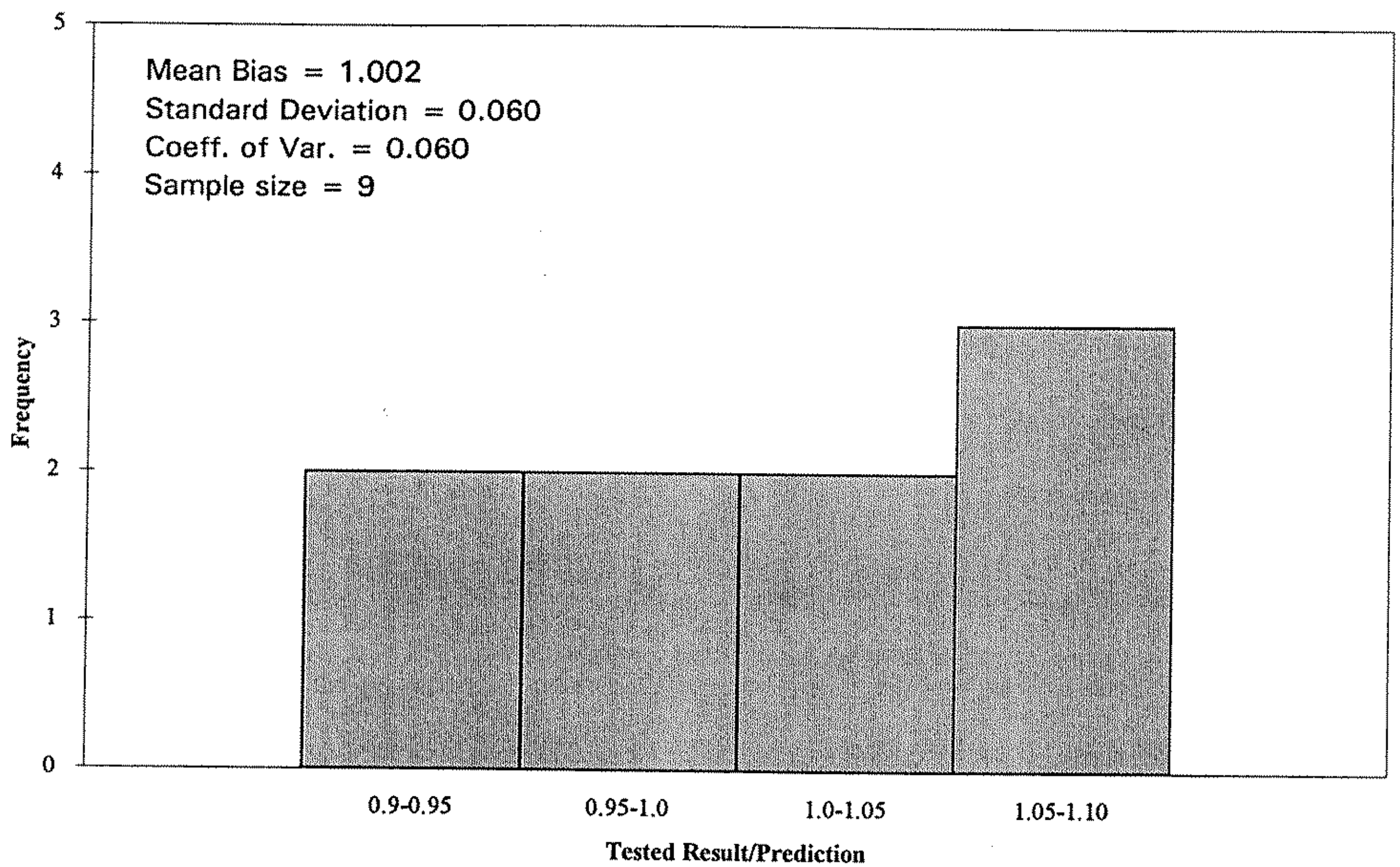


Figure 4.4.4: Histogram of modelling bias of slip strength of stressed grouted clamps

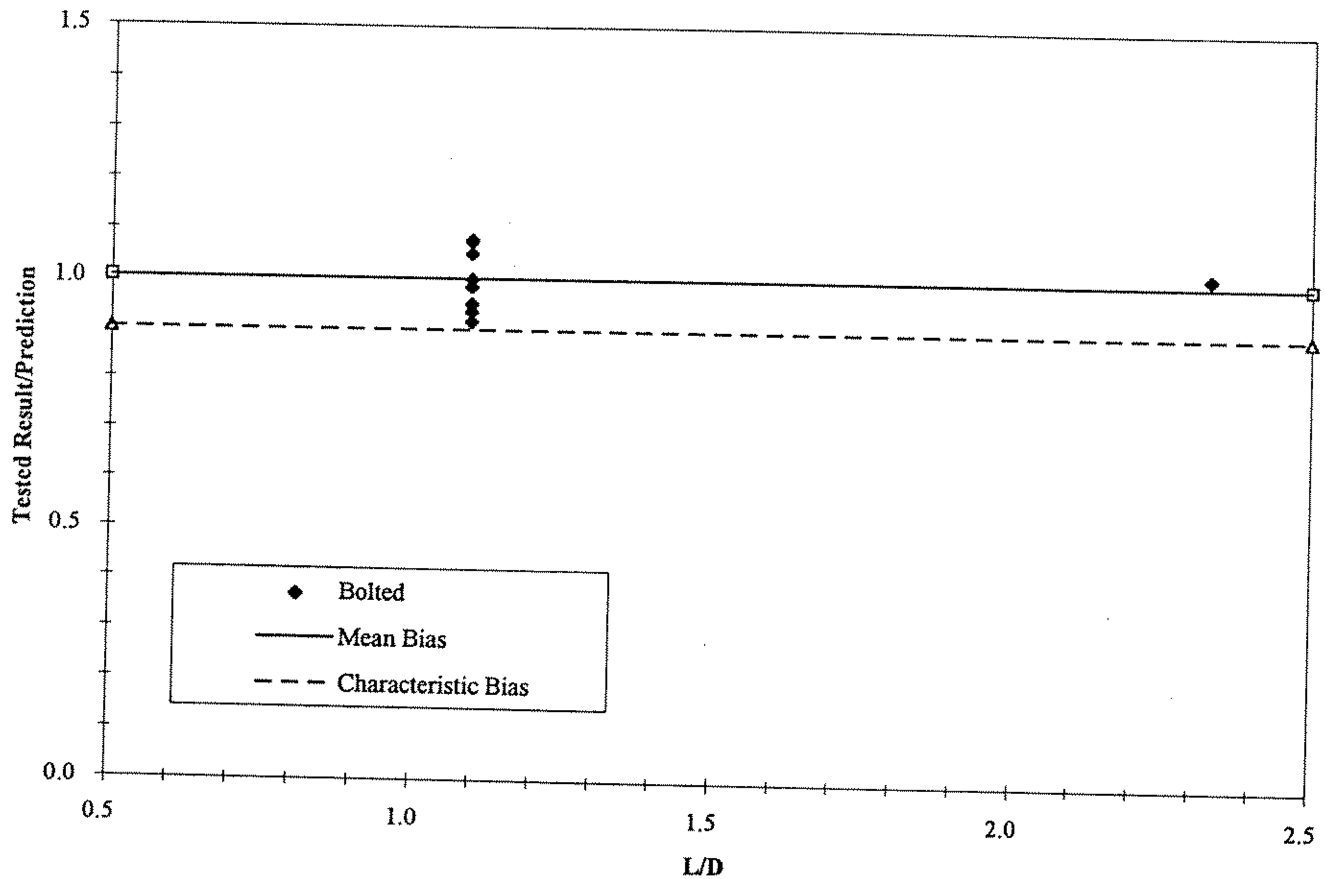


Figure 4.4.5: Effects of L/D ratio on modelling bias

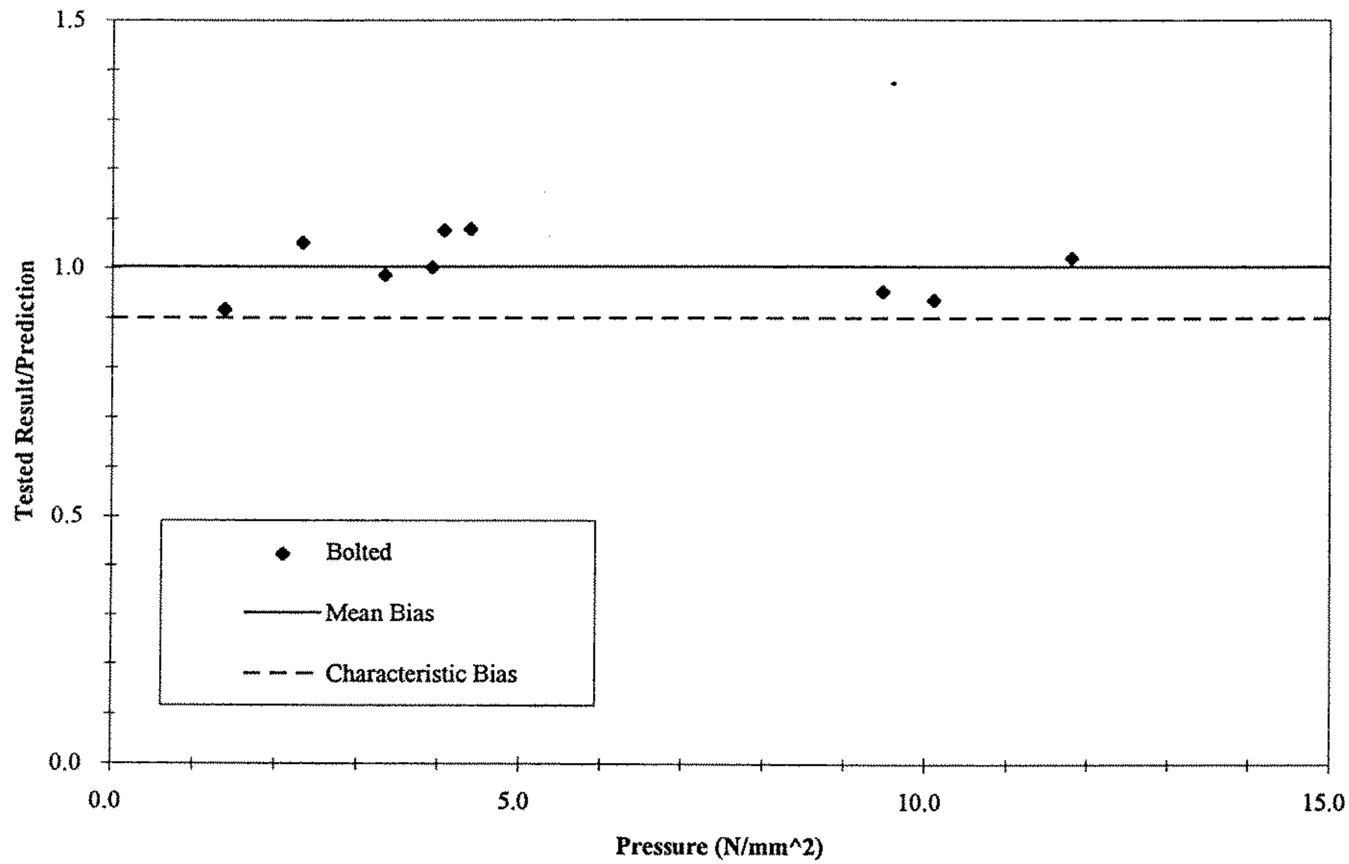


Figure 4.4.6: Effects of pressure on modelling bias

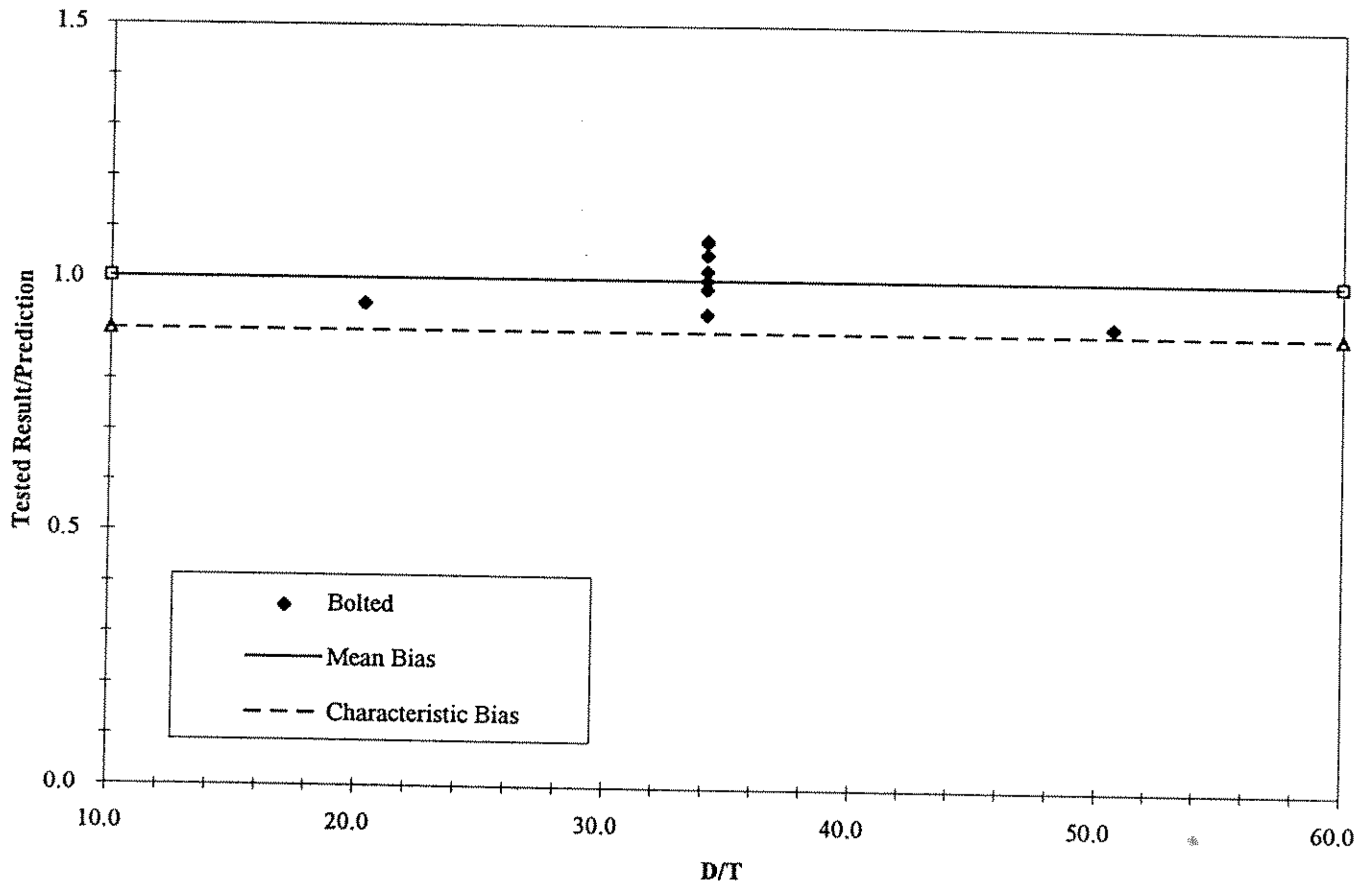


Figure 4.4.7: Effects of D/T ratio on modelling bias

When considering the possible effect of early age displacements on the subsequent behaviour of stressed grouted connections, the load transfer mechanism must be borne in mind. That is, the major component is due to friction for which early age displacements have no consequence (the studbolts being tensioned after the grout has sufficiently cured). Typically the frictional component is several times greater than the bond component.

It has been noted in Section IV 4.3.5 that early age displacement cycling does not affect the axial capacity of plain pipe unstressed grouted specimens. However, the cycling did affect the stiffness of the connections.

It may be inferred, therefore, that early age displacement cycling is not going to affect the subsequent strength of stressed grouted clamps, at least for displacement up to ± 3mm. It is also postulated that the stiffness is not significantly affected due to the fact that most of the clamp stiffness is derived from frictional effects. It is concluded that early age displacement cycling should not be a design consideration for stressed grouted clamps. Nevertheless, confirmatory tests are called for.

IV 4.4.8 Recommendations for Further Work

It is somewhat surprising, given the popularity of stressed grouted clamps as a repair technique, that the database for such clamps is so limited. An examination of Database 4.4.1 indicates that not only is the tested number of the bolted type of clamp small, but also rather limited geometric ranges have been studied. Further work must therefore be directed at testing to furnish more data for calibration purposes. The following items ideally should be covered in the test programme:-

- surface condition effects
- wider range of member and sleeve D/T ratios
- higher levels of prestress at the interface (ie. increased studbolt loads)
- distribution of studbolt loads at working and failure loads due to axial and/or bending moments (see Part V of this document)
- confirmatory tests for early age displacement cycling
- length effects
- size effects
- effect of internal grout filling on tubular stiffness
- effect of torsional loads.

IV 4.4.9 Summary

The following characteristic design strength equation is recommended for a stressed grouted clamp:-

$$\sigma_{sc} = \left[\frac{0.95C_s}{\Gamma_b} + \frac{0.35C_s'(F_n/DL)}{\Gamma_f}\right]\left[1-0.13\left(\frac{L}{D}\right)\right]\left[1+12\left(\frac{T}{D}\right)\right] \qquad \ldots.\ 4.4.8$$

The formula has been developed from data within the following ranges:-

$0.9 \leq L/D \leq 2.2$
$17.7 \leq D/T \leq 50.0$
$21.2 \leq D_s/T_s \leq 36.0$
$7.4 \leq D_g/T_g \leq 18.2$
$50.0\ N/mm^2 \leq \sigma_{cu} \leq 78.0\ N/mm^2$
$1.3\ N/mm^2 \leq F_n/DL \leq 12.0\ N/mm^2$

The following safety factors for bond and friction strengths, taken from the UK HSE Guidance Notes, are recommended for use with the above design equation.

$$\Gamma_b = \begin{cases} 4.5 & \text{for extreme loading conditions} \\ 6.0 & \text{for operational loading conditions} \end{cases}$$

$$\Gamma_f = \begin{cases} 1.7 & \text{for extreme loading conditions} \\ 2.25 & \text{for operational loading conditions} \end{cases}$$

The surface condition factors should be taken as follows:-

Steel surface condition	C_s	C_s'
Grit blasted	0.6	1.00

IV 4.5 SLIP STRENGTH OF STRESSED ELASTOMER-LINED CLAMPS

IV 4.5.1 Existing Guidance

Current practice in the design of stressed elastomer-lined clamps generally utilises a coefficient of friction of 0.2 for a rubber-steel interface. This has been a long standing figure unaltered due to the lack of test data relevant to clamp technology.

For polychloroprene liners, the UK Health and Safety Guidance Notes[4.2] limits the coefficient of friction to 0.2 under ultimate conditions. For other filler mediums the Guidance stipulates careful investigation to justify an ultimate value. Allowable design values of the friction coefficient are then obtained by applying the following safety factors to the ultimate value:

Under extreme conditions:

$$\mu_{allow} = \frac{\mu_{ult}}{1.7} \qquad \text{.... 4.5.1}$$

Under operating conditions:

$$\mu_{allow} = \frac{\mu_{ult}}{2.25} \qquad \text{.... 4.5.2}$$

An early publication by Webco Ltd.[4.14] confirms that a coefficient of friction of 0.2 is appropriate. A publication by the Malaysian Rubber Producers Research Association[4.15] states that the coefficient of friction against most dry surfaces is generally about unity, but, for design purposes it is usually assumed that slip due to a shear force will not occur if the ratio of maximum shear force to minimum compressive force is less than 0.2 for rubber to steel. It is noted that these values for coefficient of friction are based on a relatively hard compound material which inherently has a resistance to long-term creep.

The surface condition of the steel is important in providing the required load transfer through friction. The UK HSE Guidance Notes specify that existing steelwork should be gritblasted and/or water jetted and/or wire brushed to bare metal (if possible) and the contact surface of the clamp should be shotblasted and left unpainted.

IV 4.5.2 Recent Research

Recent unpublished research, based on dynamic tests, indicate values for coefficient of friction as high as 0.8 for "wet" plain linings. Also, current manufacturers literature states that tests carried out on both dry and wet surfaces yield a coefficient of friction of the order of 0.8 $\pm$25%

Further recent tests have indicated an average value of 0.5 for the coefficient of friction. Where the tests included a variation in applied pressure, the indication was that the coefficient of friction reduced with increased pressure. However, these higher pressures were considerably greater than those that would be used in a stressed clamp.

IV 4.5.3 Recommendation

It is evident from the lack of sufficient data particularly relevant to clamp technology, that a revised coefficient of friction value can not be justified. However, further research may yield a coefficient of friction as high as 0.5.

On the basis of current information it is recommended that designs be based on coefficient of friction values given in the UK HSE Guidance Notes.

The following equation should be used to determine the minimum net contact force required to resist slip:

$$\sigma_c = \frac{\mu_{ult}}{\Gamma_f} \cdot \left[\frac{F_n}{DL}\right] \qquad \text{.... 4.5.3}$$

where: Γ_f = 1.7 for extreme conditions
Γ_f = 2.25 for operating conditions

IV 4.6 EFFECTIVE STUDBOLT LOADS

The integrity of stressed grouted or stressed mechanical clamps depends, inter alia, on effective preload being maintained in the studbolts. The following factors can affect the studbolt loads.

IV 4.6.1 Transfer Losses

The preferred practice for tensioning studbolts is by hydraulic jacks manifolded to a common oil pressure feed. By this means all studbolts are simultaneously tensioned, thus avoiding sequential incremental tightening methods with the attendant risk of relaxation when adjacent studbolts are further tensioned. (Equally importantly, it is far quicker in use offshore and therefore offers economic advantages.)

In hydraulic tensioning methods the studbolt is put into tension via a 'reaction nut' at the back of the jack. A second nut is then rotated along the studbolt thread until it is brought to bear onto the clamp such that, when the jack pressure is released, the studbolt load is transferred to the second nut. During the transfer process, deformations occur in the threads of the studbolt and nut, and possibly in the bedding down of washers and the bending of flange plate. These deformations effectively constitute a shortening of the studbolt and consequently lead to a loss of the tension. This so called transfer loss depends on a number of inter-related factors which are difficult to quantify (eg. clearances, thread tolerance etc) and are thus not amenable to calculation. It may be noted that for a given studbolt load, the deformations would be expected to be similar whether the studbolt be long or short. The resulting loss of strain and hence load would therefore be commensurately greater in short studbolts. Nevertheless, it is current practice to adopt a blanket value of 10% to account for long studbolt transfer losses. Manufacturers are sometimes willing to provide estimates of the losses.

IV 4.6.2 Bedding Down Effect

The action of applied loads on the clamped members can change studbolt loads; not only instantaneously but also on a long term basis as the clamp 'settles down'. There is very little information available on this aspect but it is suggested that any long term changes have to be only a small proportion of transfer losses, and one adequately accounted for in the 10% blanket value assigned to transfer losses.

IV 4.6.3 Grout Shrinkage/Creep

Limited data are available on these effects on studbolt tension[4.1]. Measurements made on Macalloy studbolt tensions in stressed grouted connections destined for long term tests are summarised in Table 4.6.1.

Clamp No.	Studbolt No.	Studbolt diameter (mm)	Original studbolt load (kN)	Studbolt load after 12 months (kN)	% change in load
8/1	1	20	82.6	80.3	2.7
	2		92.2	88.9	3.7
	3		86.0	84.3	2.0
	4		88.3	84.9	3.8
					mean = 3.1
9/1	1	32	106.1	100.4	5.4
	2		115.8	107.8	6.9
	3		142.7	109.3	23.4
	4		80.2	95.5	-19.0
					mean = 7.2
10/1	1	40	134.6	119.0	11.5
	2		111.7	96.6	13.5
	3		93.8	85.2	9.2
	4		107.5	96.6	10.0
					mean = 11.1

Table 4.6.1: Studbolt Loads

The losses are not attributable to relaxation effects as the loads in the studbolts were too small a proportion of the ultimate capacities. It was also noted that the smaller diameter bars were in fact more heavily stressed for which relaxation effects should have therefore been more pronounced.

The long term losses were very likely caused by creep and/or shrinkage of the grout. If this is so then a large proportion (45%) of the losses would have occurred in the first 20 days. Note that all clamps in Table 4.6.1 had a grout annulus thickness of 20mm (see Database 4.4.1) and therefore, if creep/shrinkage is the explanation for the losses, the period between grouting and tensioning will play a part in determining the losses.

The current practice of allowing a 15% figure for such losses, and follow up checks of studbolt tensions, appears to be vindicated.

IV 4.6.4 Grout Ring Losses

It should be recognised that modern stressed grouted clamp designs allow a complete annulus of grout to be cast. Since the studbolts are necessarily tensioned only after the grout has attained a certain level of strength, not all of the studbolt load is effective at the inner grout/steel interface as a proportion is carried directly by the grout. The calculated required load at this interface (where clamps are disposed to fail) has to be increased to give the design studbolt load. The increase can be estimated using standard thick ring solutions as given, for instance, in Reference 4.16.

IV 4.6.5 Elastomer Creep

There are presently no published data of direct relevance to creep of elastomers within clamps. Consultation with manufacturers would suggest that a value of 10% for a long term allowance would be conservative. (The value of 15% used for grout includes an allowance for shrinkage effects.)

IV 4.7 FATIGUE

IV 4.7.1 Introduction

Practically all clamps and sleeve connections will be subjected to cyclic loading, although in many instances this will be of low magnitude compared to design static conditions. However, many clamps are used to address a specific known fatigue problem (eg. repair of a cracked node) or the loads are such that the fatigue performance of the repair has to be carefully assessed to ensure long term structural integrity. There are potentially four areas to be considered, namely fatigue of the:-

- original steelwork
- clamp steelwork
- studbolts
- grout and grout/steel interface.

It is necessary to establish the relevant stress in each component for assessing its fatigue performance. It is at this stage that the design philosophy with respect to load sharing between the clamp and original structure should be recalled, and the stresses assigned in accordance with that philosophy.

IV 4.7.2 Original Steelwork

Although not generally to be recommended, there may be a requirement to adopt a design philosophy which relies on load sharing between the original structure (whether this be damaged or undamaged at the repair/strengthening location) and the clamp.

For an uncracked node, the remaining fatigue can, in principle, be estimated from the nominal member loads and the SCFs pertaining to the node before and after clamp installation. Appropriate summation techniques (eg. Palmgren-Miner rule) would be employed to assess the damage contributions pre- and post-installation. The SCFs are generally reduced on clamp installation due to two effects:

- Part of the nominal loading of the member is taken up by the clamp leading to an apparent reduction in SCF at the node. The partitioning of the load depends on the type of load (eg. axial or bending) and thus the relevant geometric section property ratio (eg. ratio of clamped/unclamped areas or ratio of second moment of areas).

- The presence of the clamp restrains local deformations leading directly to a true reduction of SCFs.

The available databases on SCF reductions are given in Database 4.7.1 to 4.7.3 for stressed mechanical clamps, unstressed grouted clamps and stressed grouted clamps respectively. These data have been extracted from OTH 88 283[4.1] and a report on testing work carried out for Shell Exploration and Production[4.17]. The latter work related to a Y-joint strengthened by a clamp which consisted of two separate stressed grouted connections, one on the brace and the other on the chord, joined by 'cheek' plates. The actual intersection was not surrounded by grout and therefore only the first SCF reduction mechanism noted above was effective.

A detailed examination of the above data shows that clamping certainly does reduce SCFs, and that broadly the degree of reduction is dependent on the relevant section property ratio (eg. the area ratio in case of SCFs for axial loading). The data exhibits wide scatter as illustrated in Figures 4.7.1 and 4.7.2

which show all the SCF reduction ratios (ie. SCF with clamp/SCF without clamp) plotted against the relevant section property ratio. The two curves shown in the figures relate to the design rules proposed in Reference 4.1:

For axial loads:

$$\frac{SCF_R}{SCF_U} = 0.73 \frac{A_J}{A_C} \quad \text{but} \ngtr 1.0 \qquad \ldots. 4.7.1$$

For bending loads:

$$\frac{SCF_R}{SCF_U} = 2.17 \frac{I_J}{I_C} \quad \text{but} \ngtr 1.0 \qquad \ldots. 4.7.2$$

Where SCF_R = SCF for repaired joint
SCF_U = SCF predicted for unrepaired joint based on parametric equations
A_J = Joint brace section area
A_C = Composite brace section area
I_J = Joint brace second moment of area
I_C = Composite brace second moment of area

The composite brace section properties are defined as the sum of the appropriate joint brace section property and the clamp brace section property.

Note, Reference 4.1 developed the above expression with respect to predicted (UEG Design Guide) rather measured SCF values of the unrepaired joint, and for mechanical clamps and stressed grouted clamps which completely enclosed the nodes.

A study of Figure 4.7.1 and 4.7.2 in conjunction with Databases 4.7.1 to 4.7.3 confirms that the above expressions are not applicable to either unstressed grouted clamps or to stressed grouted clamps which do not enclose the node.

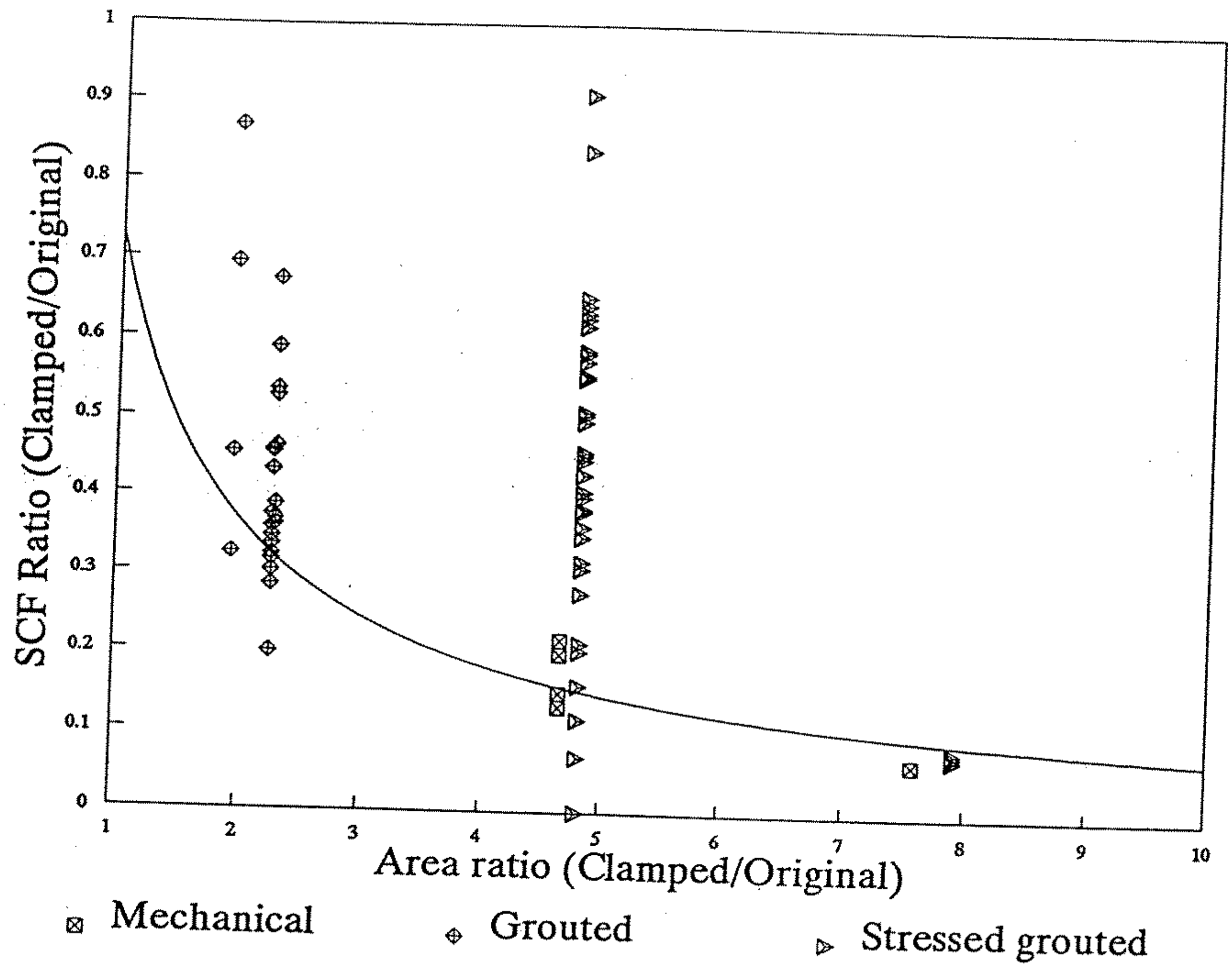


Figure 4.7.1: **Effect of area ratio on SCF reduction ratio (all axial loading data)**

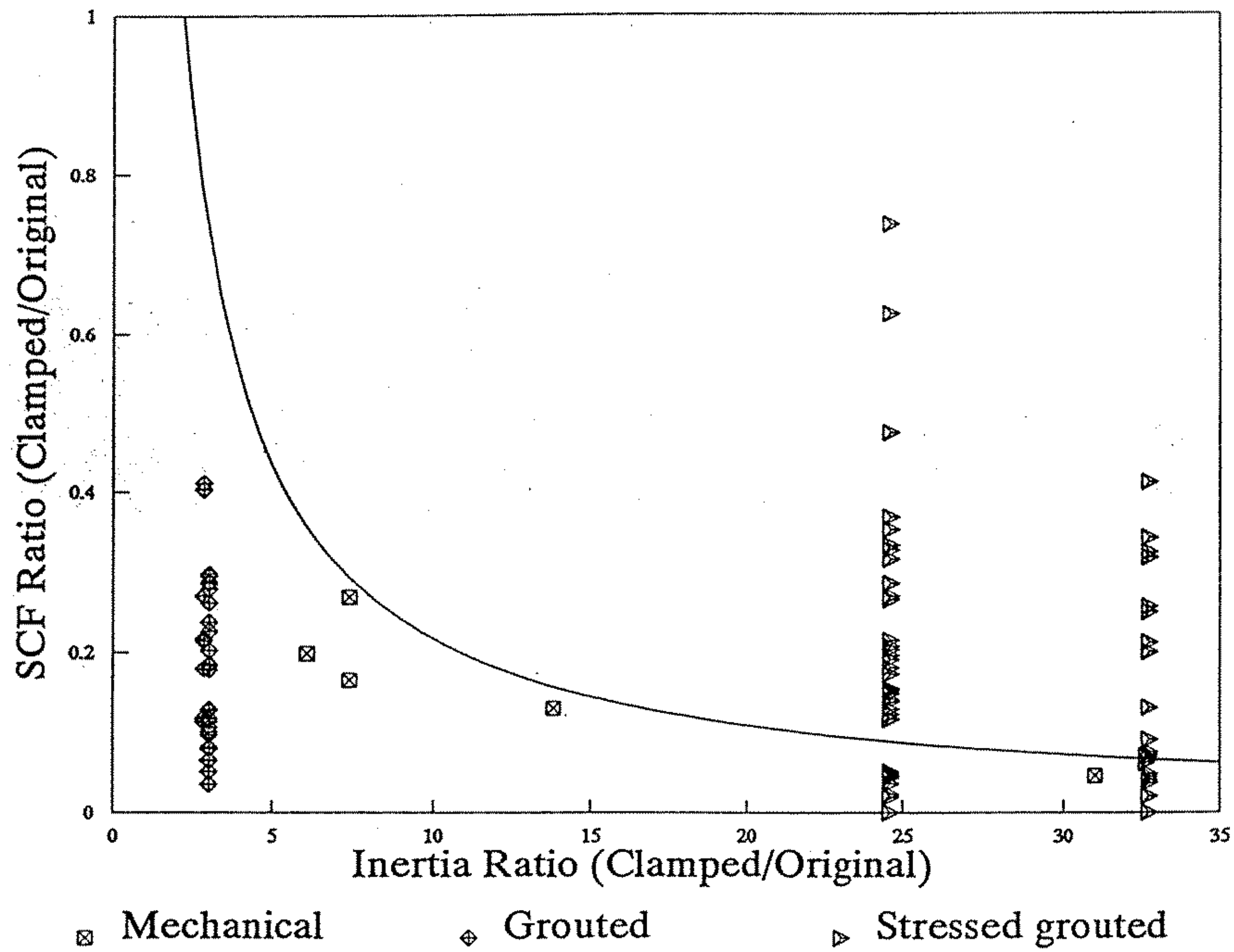


Figure 4.7.2: **Effect of inertia ratio on SCF reduction ratio (all bending data)**

IV 4.7.3 Clamp Steelwork

A number of fatigue tests have been carried out and are reported in Reference 4.1. A useful distinction can be drawn between the fatigue behaviour of the sleeves (ie. saddle plates) and that of the rest of the clamp steelwork.

IV 4.7.3.1 Sleeves

In a clamp around a nodal joint, the sleeves can be considered as intersecting tubulars (albeit split tubulars). As such, it may be conjectured that their fatigue behaviour is governed by similar considerations brought to bear on normal tubular joints, eg. use of a hot spot stress in conjunction with an appropriate S–N curve. Presently, there is no parametric formula for SCFs in sleeves, and therefore the design artifice of adjusting existing formulae for normal joints is investigated. The Efthymiou set of parametric equations, as set out in the Appendix to Volume III, are selected as the most appropriate set, these being considered as the most accurate for tubular joints.

The available database for SCFs in sleeves is given in Database 4.7.4; all data relate to unstressed grouted clamps. In Database 4.7.4, the predicted SCFs in the sleeves are determined from the Efthymiou parametric equations and making the following assumptions:-

- the sleeve is considered to be an as-welded tubular joint, ie. no account is taken of longitudinal splits or of the 'missing' chord plug at the intersection,
- the section properties of the bolt flanges are ignored,
- the presence of the original tubular joint inside the sleeve is ignored.

The measured to predicted SCF ratios are plotted in Figure 4.7.3 as a function of the relevant section property ratio. The relevant section property ratio for axial loading is the ratio of the total area (of brace + sleeve + sleeve flanges) to the brace area, that for moment loading is based on second moment of areas and fibre distances, thus:

$$\text{Area ratio} = \frac{A_b + A_s + A_f}{A_b} \qquad \text{.... 4.7.3a}$$

$$\text{Modulus ratio} = \left[\frac{I_b + I_s + I_f}{I_b}\right] \frac{y_b}{y_s} \qquad \text{.... 4.7.3b}$$

where: A, I and y refer to area, inertia and fibre distance respectively and subscripts b, s, f refer to brace, sleeve and flange respectively.

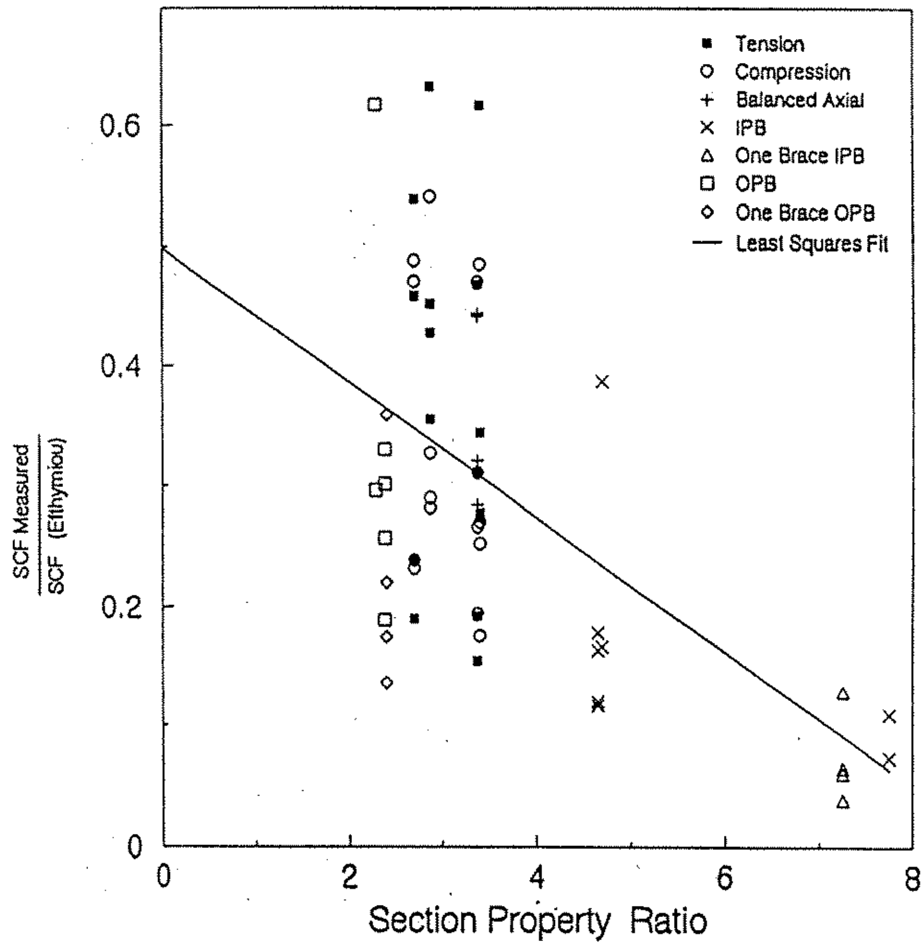


Figure 4.7.3: **Relationship between sleeve SCF ratio and section property ratio**

The results have been presented in this format because the measured SCFs relate to nominal stresses in the brace, whereas predicted SCFs relate to nominal stresses in the sleeve. By invoking the relevant section property ratio, it is possible to account for this inconsistency.

The best fit (least squares) line is shown in Figure 4.7.3. It has the following form of equation:

$$SCF_s/SCF_E = 0.5 - 0.0561 \text{ [Section Property Ratio]} \qquad \text{.... 4.7.4}$$

in which SCF_s is the predicted sleeve SCF (rather than measured) and SCF_E is the Efthymiou value. The frequency histogram is shown in Figure 4.7.4 and considerable scatter in the data is indicated. (Histograms of individual loading modes do not indicate that the scatter is due to combining different populations.) With a sample size of 58 and a standard deviation of 0.414, the characteristic (95% survivability at 50% confidence level) is found as 1.684 times the mean (note a high, conservative, prediction of the SCF is required). The recommended SCF prediction equation for design therefore becomes:

$$SCF_s = SCF_E \; (0.842 - 0.0945 \text{ [Section Property Ratio]}) \qquad \text{.... 4.7.5}$$

The SCF_s is multiplied by the brace nominal stress to give the hot spot stress to apply with the appropriate S-N relationship.

IV 4.7.3.2 Other clamp steelwork

One stressed mechanical T clamp and one stressed grouted T clamp suffered fatigue cracking during tests reported in Reference 4.1. Both failures were associated with the junction between the brace flange plate and the chord flange plate. In the test specimens, the brace flange plate was welded orthogonally to the chord side plate. (In more modern designs the brace/chord flange plate is in one plane and is cut from a single plate.) The tests verify the appropriateness of existing fatigue guidance for welded steelwork for application to clamp steelwork.

Guidance is given in Reference 4.1 for estimating the flange stress in unstressed grouted clamps. No further data has been made available since Reference 4.1 was published and a review of the recommendations indicates they are still valid. These are therefore reproduced here.

The maximum stress in the flange can be expressed as:

$$\text{flange stress} = K' \sigma_{SFB\ nom} \qquad \text{.... 4.7.6}$$

Where K' = a stress concentration factor relevant to the flange radius detail

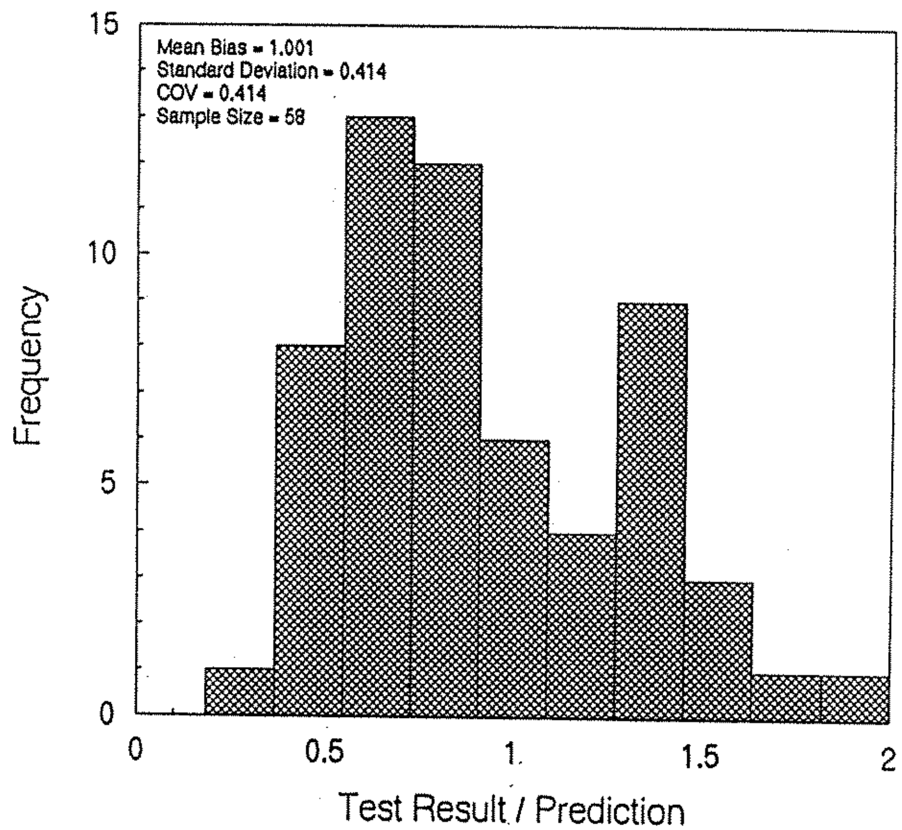


Figure 4.7.4: **Histogram of ratio of measured to predicted SCF values**

$\sigma_{SFB\ nom}$ = maximum nominal stress in sleeve flanges based on the full composite section (ie. joint and sleeve and flange) and using simple bending theory.

K′ can be determined from the expression:

$$K' = 1.3K^* \sin\theta \text{ (Flange Property Factor)} \qquad \ldots .\ 4.7.7$$

where 1.3 = a factor to account for flange bending due to chord wall flexibility, based upon the available test results compared with predictions

K^* = the flange detail SCF based on standard solutions presented by Hartman and Levin[4.18], which are reproduced in Figures 4.7.5 and 4.7.6.

$\sin\theta$ = sine of the angle of intersection between the brace and the chord

$$\text{Flange Property Factor} = \frac{\text{nominal flange cross sectional area}}{\text{minimum flange cross sectional area at bolt holes.}}$$

It is recommended that the flange stress so calculated is used with the Class C S-N curve, on the basis that corrosion pits may form on free surfaces on the flanges.

IV 4.7.4 Studbolts

Studbolts are a critical component of stressed clamps and the integrity of such clamps depends on the long term performance of the studbolts. In the few reported clamp fatigue tests in which studbolt tensions have been measured[4.1], the maximum studbolt load range was below the endurance limit of the studbolt material. The observation that low studbolt loads are induced by external loading has been confirmed by new test data generated within this project, see Part V. The new data also permit a sensible design approach for estimating the studbolt loads.

The most common materials presently specified for clamp studbolts are B7/L7 grades. Test data relating to the fatigue strength of large diameter studbolts of ASTM A193 Gr. B7 material are presented in Database 4.7.5. All data come from a single source[4.29]. The data relate to tension-tension axial fatigue loading on two sizes of studbolts, and cantilever bending fatigue on a single size of studbolt. All studbolts were formed with cut threads and therefore the data can be expected to be conservative for application to rolled threads. For those studbolts which were not fully threaded, an abrupt end to the thread was left, ie. no thread run-out was formed.

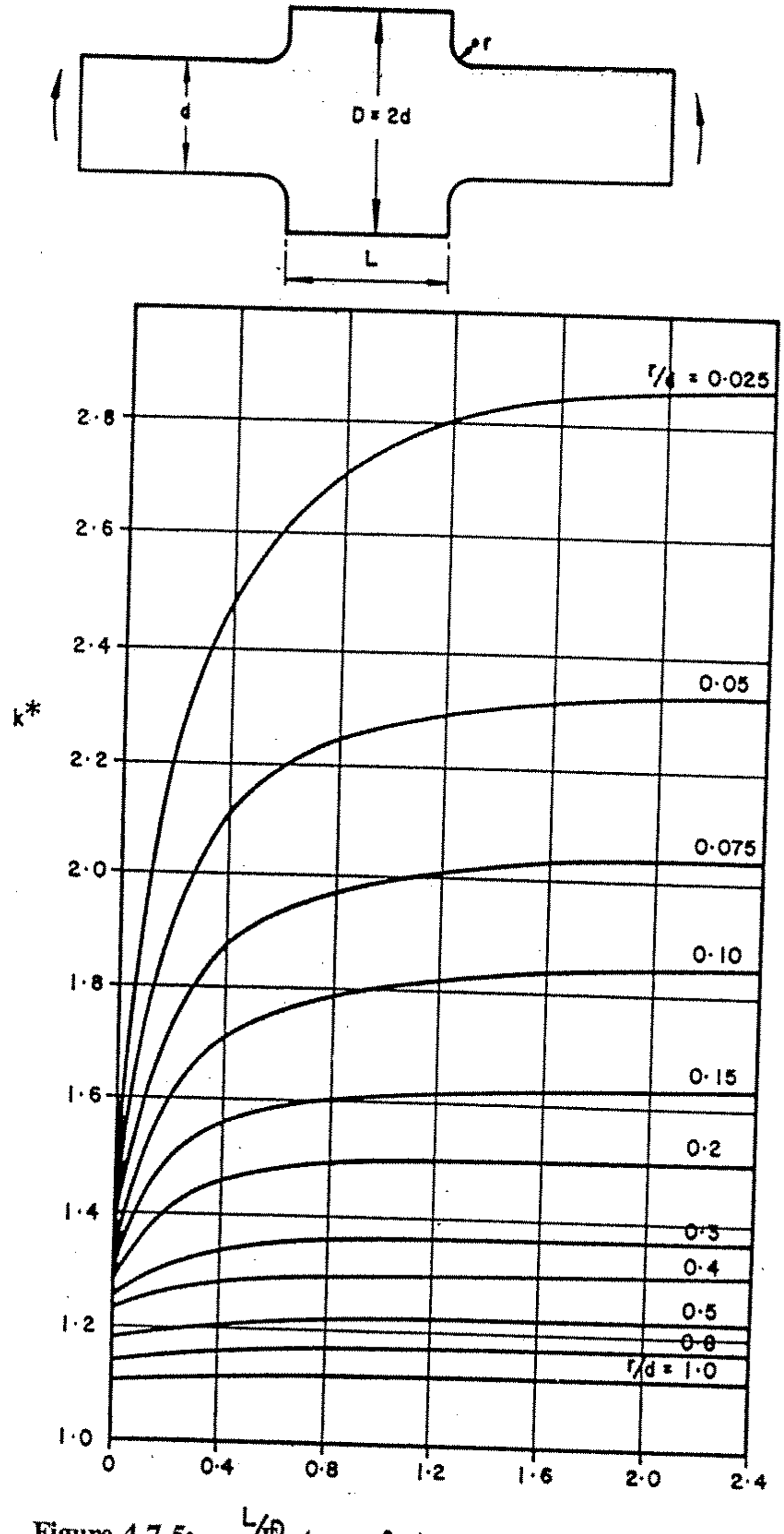


Figure 4.7.5: Factors of stress concentration K* plotted against L/D (D/d = 2)

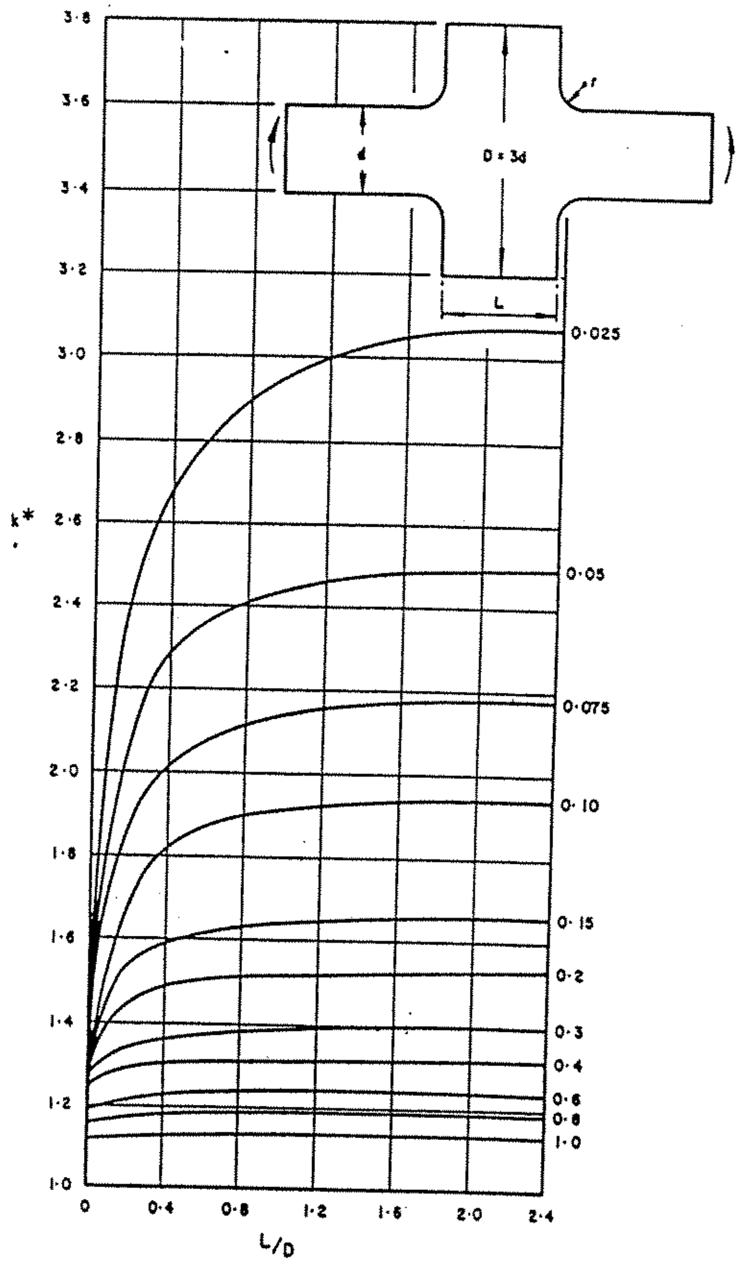


Figure 4.7.6: **Factors of stress concentration K* plotted against L/D (D/d = 3)**

All the data are plotted in Figure 4.7.7. Superimposed on the figure is a design line (two standard deviations below the mean line) for that subset of data considered most appropriate in the present context, ie. tension-tension single-nut specimens which failed. It may be observed that this subset is conservative with respect to bending failure and for specimens provided with two nuts. Calculations were performed for both ASME (imperial units) and ISO (metric units) definitions of net stress areas and the resulting studbolt stresses were found to be very similar, see Database 4.7.5. Thus the design curve may be applied to either metric or imperial studbolt/thread sizes (after suitable conversion of units).

Studbolts designed for unprotected exposure to a sea water environment should have their lives, as derived from the basic design curve, reduced by a factor of 2 in accordance with HSE Guidance Notes[4.2]. Datapoints on Figure 4.7.7 indicate a non-propagating stress range of very approximately 60 N/mm^2 occurring between 10^6 and 10^7 load cycles. The design curve has not been modified to allow for this effect as it is not applicable to unprotected studbolts exposed to sea water[4.2].

IV 4.7.5 <u>Grout and Grout/Steel Interface</u>

IV 4.7.5.1 <u>Plain Sleeve Connections</u>

Database 4.7.6 contains details of the available plain sleeve connection fatigue data. All data relate to continuous sleeves. Other data, Reference 4.19, were not included on the grounds that expansive grouts were used.

The results indicate that the fatigue resistance of these connections is generally high with specimens able to sustain a very high proportion of the predicted static capacities for large numbers of cycles. Notably, in one case (specimen 3A3), a nominal load ratio (ie. applied load to predicted static failure load) range of nearly unity was applied for 2 x 10^6 cycles with little permanent relative displacement of the pile and sleeve occurring. Generally such displacements are related to the load ratio applied to the specimen during cycling. For maximum applied load ratios of approximately 0.4, 2.0 and 0.5 for HSE, API and DNV codes respectively, the data indicate no relative slip would have occurred, see Figure 4.7.8.

Three specimens in Database 4.7.6 are combined axial/bending specimens. These data do not suggest that the presence of bending loads decreases the axial fatigue strength.

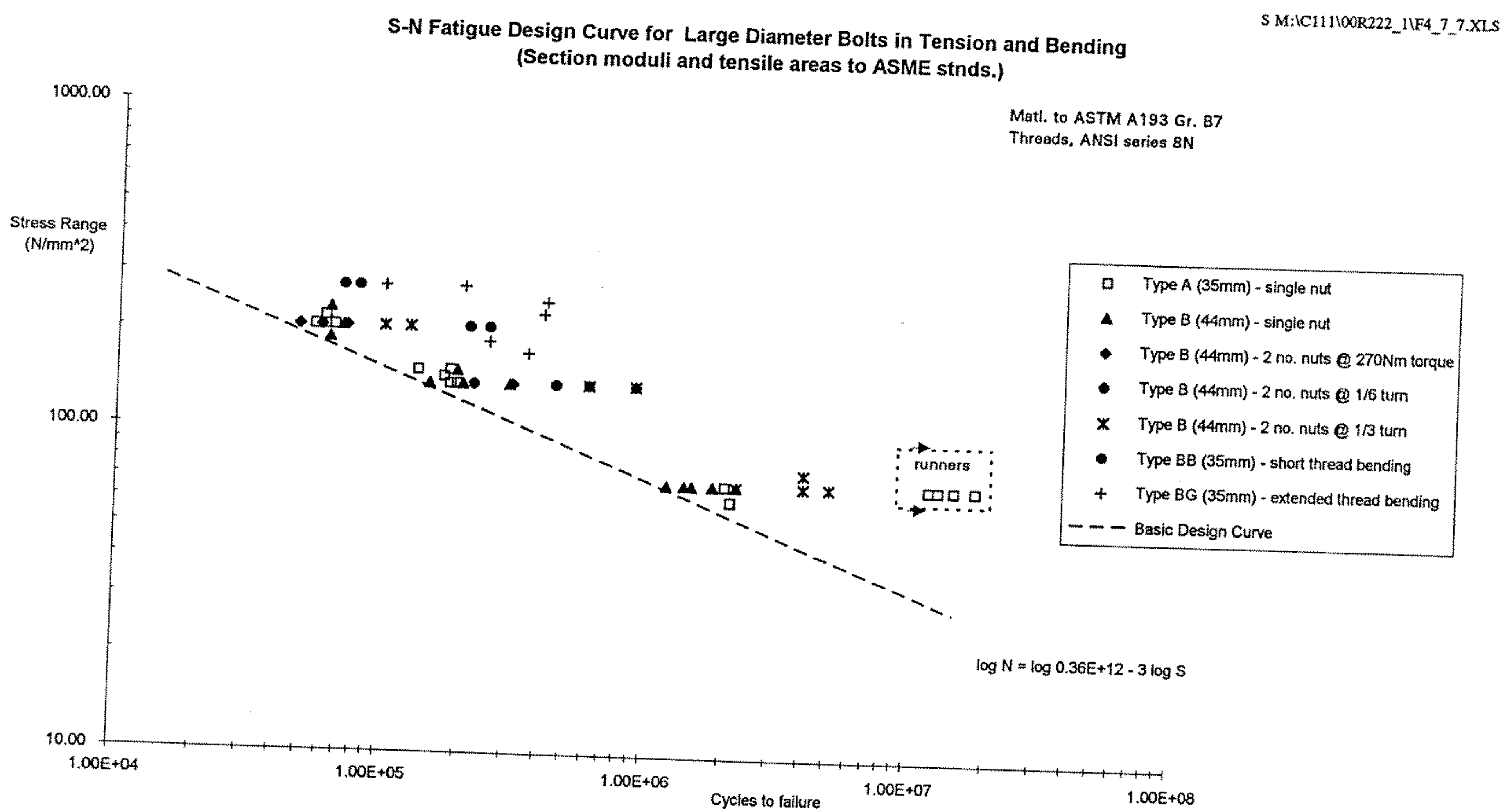
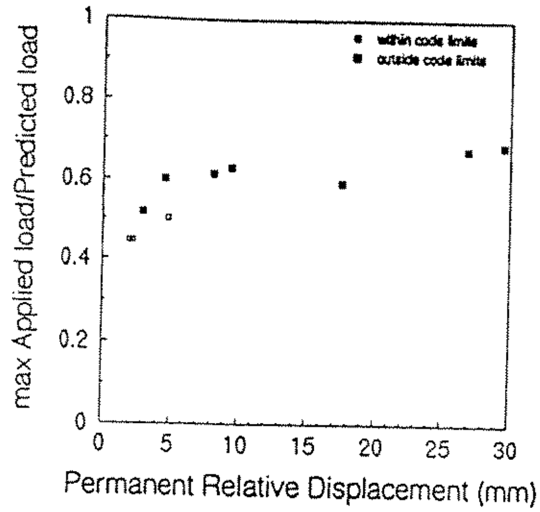
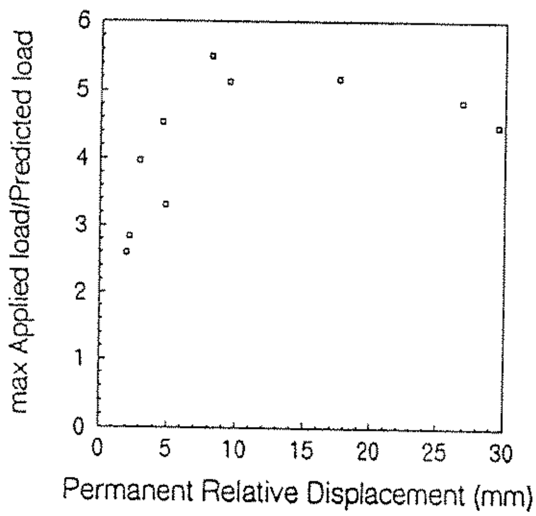


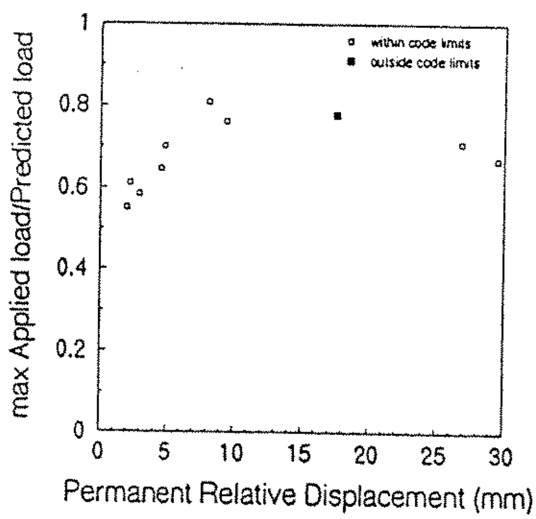
Figure 4.7.7: Fatigue behaviour of B7 studbolts



a) HSE



b) API



c) DnV

Figure 4.7.8: **Permanent relative displacement after 2 x 10^6 cycles as a function of maximum load ratio used in fatigue tests (plain sleeve data)**

With current factors of safety, the ratios of maximum permissible bond strength to mean ultimate strength for HSE, API and DNV are 0.17, 1.00 and 0.33 respectively. The above results would then suggest that the fatigue limit state is adequately covered by the static design clauses. However, it should be noted that the tests to date do not cover a wide range of tubular geometries. Additionally, the following aspects require further consideration:

- The length effect observed during static tests may be different for fatigue tests due to the possibility of progressive debonding during cycling.
- The tests to date have been carried out with continuous sleeves. There may be a lower endurance limit for split sleeves (as is indicated for weld bead connections discussed below).
- Interaction between early age cycling and subsequent fatigue behaviour has not been assessed.

IV 4.7.5.2 Weld Bead Connections

There are substantially more data relating to well bead connections than there are to plain pipe connections. Weld bead connection data exists for continuous sleeves, Database 4.7.7, and for split sleeves, Database 4.7.8.

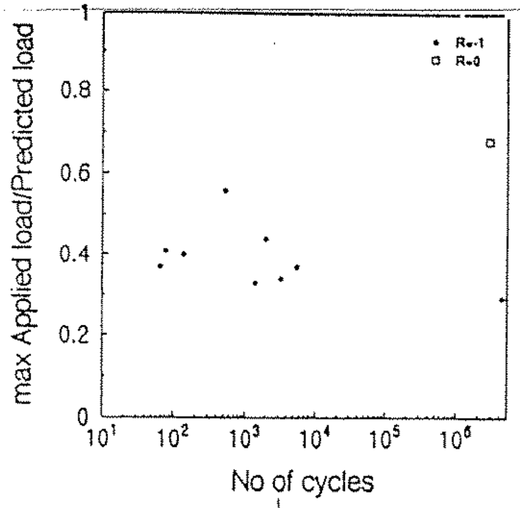
It may be observed that fatigue tests have been carried out for unidirectional loading and for fully reversed loading (R values ranging from +0.5 to -1). In the context of repair and strengthening, as opposed to pile/sleeve connection application, the fully reversed loading assumes greater importance.

Considering the continuous sleeve data first, Figure 4.7.9, the results indicate a lower bound given by the specimens subject to fully reversed loading ($R = -1$). The presence or absence of bending does not appear to affect the lower bound line. The tests results of specimens subjected to unidirectional load indicated that their fatigue damage susceptibility is reduced. This observation gives credence to the hypothesis that fatigue failure of weld bead specimens is due to the weld bead sliding in and impacting at the end of a slot itself created.

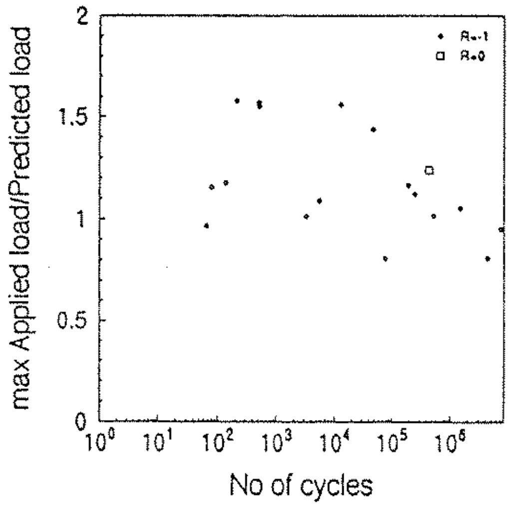
The test results indicate that failure is sudden, with no significant loss of stiffness occurring before approximately the final 5% of the life.

The series of tests summarised in Database 4.7.8 and presented in Figure 4.7.10 are more relevant to repair/strengthening clamps as the specimens are formed from split sleeves. The results in Figure 4.7.10 are presented in a similar format to that used for continuous sleeves above. A comparison of Figures 4.7.9 and 4.7.10 reveals the following:

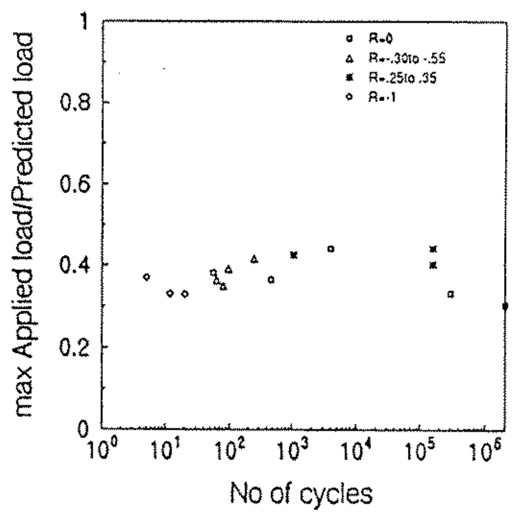
- The ratios of max applied load to predicted static failure load are approximately the same for continuous sleeves and split sleeves. This suggests that as far as fatigue performance is concerned, the data can be



a) HSE

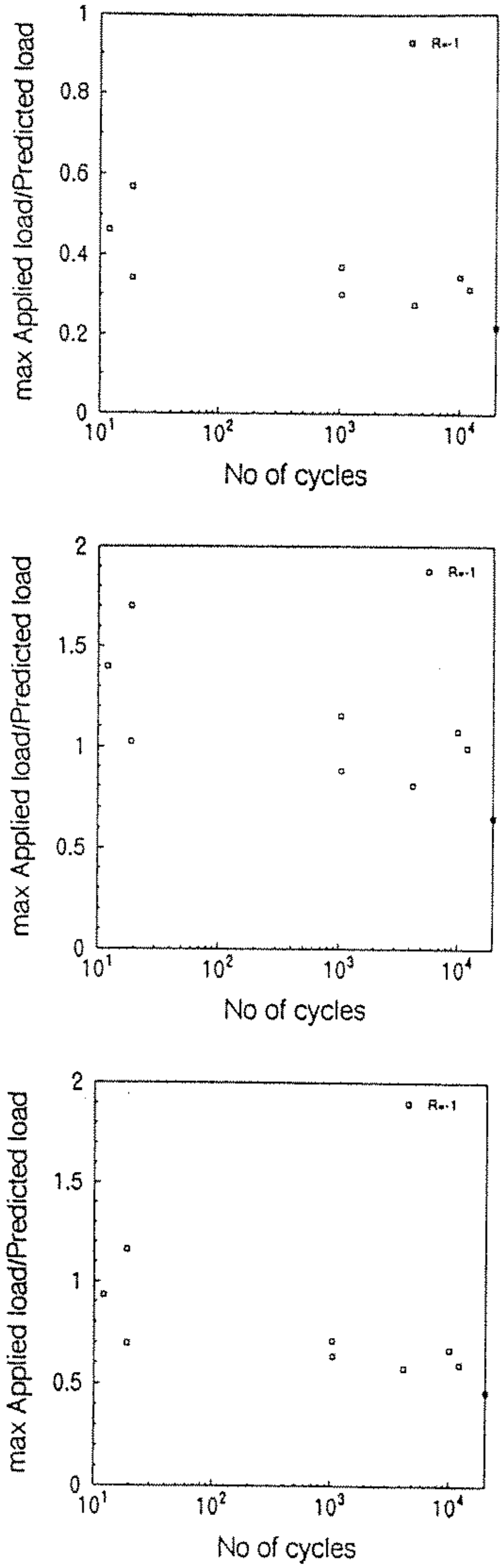


b) API



c) DnV

Figure 4.7.9: Fatigue data for weld bead continuous sleeve connections



a) HSE

b) API

c) DnV

Figure 4.7.10: Fatigue data for weld bead split sleeve connections

assigned to having come from the same population. This conclusion is at variance with that drawn in the original paper[4.28].

- The lower bound lines for the results plotted with HSE predicted strengths fall above 0.17, ie. the design static strength is below the fatigue sensitive region. This confirms that fatigue does not have to be considered with current factors of safety.

- Some results lie below the unity line for the API code. With the reduction factor of 0.7 proposed for application to split sleeve static strength using the API code, the fatigue performance may be considered unsatisfactory and specific consideration should be given to this in design.

- The split sleeve results appear better than the continuous sleeve data in the case of DNV. The DNV code requires a specific consideration of fatigue performance and the stated equation is conservative for the present results.

In summary, the use of API RP2A for split sleeve grouted connection design requires a caveat with respect to fatigue loads. The factor of safety used in the HSE Guidance Notes or the existing requirement to check for fatigue performance within the DNV regulations suggest that fatigue provision is adequate for these two codes. However, the aspects noted above for plain pipe connections (ie. geometry range, length effect and interaction between early age cycling and fatigue) also require investigating for weld bead connections.

IV 4.8 RECOMMENDATIONS

The recommendations set out below are a summary of recommendations given throughout this Section IV. They concern the static and fatigue strength of clamps and connections. Recommendations are also given for further work, mainly with respect to increasing the size of the experimental databases.

There is a general recommendation that the static capacity of a clamp is established as the product of interface slip strength (per unit area) and interface area. This avoids possible ambiguity in the meaning to be assigned to coefficients of friction (see Section IV 4.1.2.1) for stressed mechanical clamps and stressed elastomer-lined clamps. It also increases the consistency, across all types of clamps, in design.

IV 4.8.1 Stressed Mechanical Clamps

The existing formulation for the slip strength of stressed mechanical clamps has been confirmed as appropriate and the recommended equation and validity limits are given in Section IV 4.2.7.

The clamp steelwork should be assessed for fatigue performance; the fatigue performance of the steel-to-steel interface is good and not an issue that need be considered in the design.

IV 4.8.2 Unstressed Grouted Clamps/Sleeve Connections

The following recommendations are based on the recognition that unstressed grouted clamps/sleeve connections have been developed from pile/sleeve connection technology. Indeed, for continuous sleeve connections, one would expect the same slip strength equation to apply whether the application was for pile/sleeve connections or for a repair situation.

There are presently three codified design rules for unstressed grouted clamps/connections. These are the UK Health and Safety Executive's Guidance Notes[4.2], American Petroleum Institute's API RP2A[4.4] and the Det norske Veritas Classification Rules[4.6]. Only the DNV rules account for split sleeves, as may be used in repair situations. The three codified formulations are given in Section IV 4.3.3. Any of the above rules may be used for designing repair clamps/connections. However, in the case of weld bead split sleeves designed to HSE or API rules, it is recommended that a reduction factor of 0.75 or 0.70, respectively, is applied to the code result.

Variant forms of shear keys may be considered for repairs and recommendations relating to these are given in Section IV 4.3.6. Essentially the strength of connections having these other forms of shear keys may be calculated from the above existing guidance but using equivalent weld bead properties. It is further recommended that the above reduction factors are again utilised in the case of split sleeves.

It is important to prevent the two halves of a split sleeve connection opening up during service due to the action of applied loads. In the case of axial loading the relative sliding that occurs near the failure load between grout and weld beads imparts large hoop stresses to the sleeve. It is recommended that the bolts at each split line are designed to precompress the abutting faces of the split sleeve to a value of at least 0.95 times the axial force in the member. (More precompression may be required to prevent moments opening up the clamp.)

It is recommended that relative movement between the member and sleeve should be restricted to 0.6mm during the grout setting and early curing period. This should not affect clamps placed around nodes with no severed members or addmembers but does have implications for telescopic (tube to tube) types of connections.

With current levels of factors of safety, fatigue of the grout or grout interface need not be specifically considered for plain pipe or continuous sleeve connections.

IV 4.8.3 Stressed Grouted Clamps

A new formulation for the slip strength of stressed grouted clamps is recommended in Section IV 4.4.9 along with validity limits.

With current levels of factors of safety, fatigue of the grout or grout interface need not be specifically considered for stressed grouted clamps.

IV 4.8.4 Elastomer-Lined Clamps

Little data is available for the design of these clamps and it is recommended that the current HSE guidance is maintained.

IV 4.8.5 Further Work

Areas requiring further investigation have been identified throughout this chapter, and particularly in Sections IV 4.3.7 (unstressed grouted connections) and IV 4.4.8 (stressed grouted clamps).

Most of the areas are concerned with generating further experimental data, especially with respect to:

- studbolt load distribution
- studbolt losses
- slip strength of elastomer-lined clamps
- early age movement effects on both static and fatigue subsequent performance
- fatigue of split sleeve, weld bead, unstressed grouted connections
- SCF determination in repaired joints and clamp sleeves
- surface condition effects
- size and length effects
- effect of torsional loads.

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Ref. No.	Specimen No.	Clamp Type	Surface Condition	Ds (mm)	Ts (mm)	D (mm)	T (mm)	Dstud (mm)	No. of Studs	L (mm)	Ds/Ts	D/T	Estud/ Esteel	L/D	1000Kstud	Pressure (N/mm^2)	Slip Strength (N/mm^2)	Coeff. of Friction	Comments
	1/1-1	Stressed Studbolt	Shot Blasted	366.0	19.0	324.0	16.0	32.0	4.0	400.0	19.3	20.3	1.00	1.23	6.09	5.72	2.17	0.38	
	1/1-2	Stressed Studbolt	Shot Blasted	366.0	19.0	324.0	16.0	32.0	4.0	400.0	19.3	20.3	1.00	1.23	6.09	5.81	2.21	0.38	
	1/1-3	Stressed Studbolt	Shot Blasted	366.0	19.0	324.0	16.0	32.0	4.0	400.0	19.3	20.3	1.00	1.23	6.09	5.79	2.18	0.38	
	2/1-1	Stressed Studbolt	Shot Blasted	366.0	19.0	324.0	9.5	32.0	4.0	400.0	19.3	34.1	1.00	1.23	6.09	3.13	0.91	0.29	
	2/1-2	Stressed Studbolt	Shot Blasted	366.0	19.0	324.0	9.5	32.0	4.0	400.0	19.3	34.1	1.00	1.23	6.09	2.82	0.84	0.30	
	2/1-3	Stressed Studbolt	Shot Blasted	366.0	19.0	324.0	9.5	32.0	4.0	400.0	19.3	34.1	1.00	1.23	6.09	2.82	0.76	0.27	
	3/1-1	Stressed Studbolt	Shot Blasted	366.0	19.0	324.0	6.4	32.0	4.0	400.0	19.3	50.6	1.00	1.23	6.09	0.65	0.18	0.28	
	3/1-2	Stressed Studbolt	Shot Blasted	366.0	19.0	324.0	6.4	32.0	4.0	400.0	19.3	50.6	1.00	1.23	6.09	0.61	0.16	0.27	
	3/1-3	Stressed Studbolt	Shot Blasted	366.0	19.0	324.0	6.4	32.0	4.0	400.0	19.3	50.6	1.00	1.23	6.09	0.61	0.16	0.26	
	4/1-1	Stressed Studbolt	Shot Blasted	366.0	19.0	324.0	9.5	32.0	8.0	800.0	19.3	34.1	1.00	2.47	6.09	8.54	2.49	0.29	
	4/1-2	Stressed Studbolt	Shot Blasted	366.0	19.0	324.0	9.5	32.0	8.0	800.0	19.3	34.1	1.00	2.47	6.09	8.41	2.48	0.29	
	4/1-3	Stressed Studbolt	Shot Blasted	366.0	19.0	324.0	9.5	32.0	8.0	800.0	19.3	34.1	1.00	2.47	6.09	8.38	2.57	0.31	
4.1	5/1-1	Stressed Studbolt	Shot Blasted	366.0	19.0	324.0	9.5	32.0	2.0	200.0	19.3	34.1	1.00	0.62	6.09	9.68	3.08	0.32	
	5/1-2	Stressed Studbolt	Shot Blasted	366.0	19.0	324.0	9.5	32.0	2.0	200.0	19.3	34.1	1.00	0.62	6.09	9.34	2.60	0.28	
	5/1-3	Stressed Studbolt	Shot Blasted	366.0	19.0	324.0	9.5	32.0	2.0	200.0	19.3	34.1	1.00	0.62	6.09	8.80	2.21	0.25	
	6/1-1	Stressed Studbolt	Shot Blasted	366.0	19.0	324.0	9.5	20.0	4.0	400.0	19.3	34.1	1.00	1.23	2.38	3.01	0.76	0.25	
	6/1-2	Stressed Studbolt	Shot Blasted	366.0	19.0	324.0	9.5	20.0	4.0	400.0	19.3	34.1	1.00	1.23	2.38	2.84	0.64	0.23	
	6/1-3	Stressed Studbolt	Shot Blasted	366.0	19.0	324.0	9.5	20.0	4.0	400.0	19.3	34.1	1.00	1.23	2.38	2.82	0.66	0.24	
	7/1-1	Stressed Studbolt	Shot Blasted	366.0	19.0	324.0	9.5	40.0	4.0	400.0	19.3	34.1	1.00	1.23	9.52	2.06	0.72	0.35	
	7/1-2	Stressed Studbolt	Shot Blasted	366.0	19.0	324.0	9.5	40.0	4.0	400.0	19.3	34.1	1.00	1.23	9.52	2.10	0.70	0.34	
	7/1-3	Stressed Studbolt	Shot Blasted	366.0	19.0	324.0	9.5	40.0	4.0	400.0	19.3	34.1	1.00	1.23	9.52	1.98	0.63	0.32	
	8/1-1	Stressed Studbolt	Mill Scale	366.0	19.0	324.0	9.5	32.0	4.0	400.0	19.3	34.1	1.00	1.23	6.09	2.29	0.61	0.27	
	8/1-2	Stressed Studbolt	Mill Scale	366.0	19.0	324.0	9.5	32.0	4.0	400.0	19.3	34.1	1.00	1.23	6.09	2.25	0.58	0.26	
	8/1-3	Stressed Studbolt	Mill Scale	366.0	19.0	324.0	9.5	32.0	4.0	400.0	19.3	34.1	1.00	1.23	6.09	2.25	0.61	0.27	
	9/1-1	Stressed Studbolt	Coal Tar Epoxy	366.0	19.0	324.0	9.5	32.0	4.0	400.0	19.3	34.1	1.00	1.23	6.09	1.68	0.29	0.18	
	9/1-2	Stressed Studbolt	Coal Tar Epoxy	366.0	19.0	324.0	9.5	32.0	4.0	400.0	19.3	34.1	1.00	1.23	6.09	1.68	0.30	0.18	
	9/1-3	Stressed Studbolt	Coal Tar Epoxy	366.0	19.0	324.0	9.5	32.0	4.0	400.0	19.3	34.1	1.00	1.23	6.09	1.68	0.29	0.18	
	10/1-1	Stressed Studbolt	Mill Scale	366.0	19.0	324.0	9.5	32.0	8.0	800.0	19.3	34.1	1.00	2.47	6.09	19.05	4.18	0.22	
	10/1-2	Stressed Studbolt	Blasted Cleaned	366.0	19.0	324.0	9.5	32.0	8.0	800.0	19.3	34.1	1.00	2.47	6.09	16.39	3.84	0.23	
	10/1-3	Stressed Studbolt	Blasted Cleaned	366.0	19.0	324.0	9.5	32.0	8.0	800.0	19.3	34.1	1.0	2.47	6.09	21.72	4.67	0.22	

Coefficient of Friction = Slip Strength (N/mm^2) / Pressure (N/mm^2)

Ref. No.	Specimen No.	Clamp Type	Vertical Load (KN)	Horizontal Load (KN)	Coeff. of Friction	Comments	
						Lower Surface	Upper Surface
4.1	1A	Flat Plate	421.9	167.9	0.40	Blast Cleaned	Same as lower surface
	2A	Flat Plate	611.7	247.8	0.41	Blast Cleaned	Same as lower surface
	3A	Flat Plate	801.6	426.0	0.53	Blast Cleaned	Same as lower surface
	4A	Flat Plate	232.0	97.1	0.42	Blast Cleaned	Same as lower surface
	5A	Flat Plate	421.9	199.9	0.47	Blast Cleaned	Same as lower surface
	6A	Flat Plate	611.7	258.1	0.42	Blast Cleaned	Same as lower surface
	7A	Flat Plate	801.6	396.9	0.50	Blast Cleaned	Same as lower surface
	8A	Flat Plate	991.4	468.9	0.47	Blast Cleaned	Same as lower surface
	9A	Flat Plate	991.4	523.5	0.53	Blast Cleaned	Same as lower surface
	10A	Flat Plate	327.0	163.3	0.50	Blast Cleaned	Same as lower surface
	9B	Flat Plate	649.0	342.0	0.53	Grit Blast and 48 hrs salt water	Same as lower surface
	10B	Flat Plate	649.0	361.0	0.56	Grit Blast and 48 hrs salt water	Same as lower surface
	11B	Flat Plate	1180.0	622.0	0.53	Grit Blast and 48 hrs salt water	Same as lower surface
	12B	Flat Plate	1180.0	636.0	0.54	Grit Blast and 48 hrs salt water	Same as lower surface
	13B	Flat Plate	649.0	352.0	0.54	Grit Blast and 8 hrs salt water	Same as lower surface
	14B	Flat Plate	649.0	365.0	0.56	Grit Blast and 8 hrs salt water	Same as lower surface
	15B	Flat Plate	1180.0	597.0	0.51	Grit Blast and 8 hrs salt water	Same as lower surface
	16B	Flat Plate	1180.0	653.0	0.55	Grit Blast and 8 hrs salt water	Same as lower surface
	17B	Flat Plate	230.0	132.0	0.57	1st quality grit blast and 48 hrs salt air	Same as lower surface
	18B	Flat Plate	226.0	124.0	0.55	1st quality grit blast and 120 hrs salt air + light	Same as lower surface
	19B	Flat Plate	2775.0	1250.0	0.45	1st quality grit blast and spray of salt water	Same as lower surface
	20B	Flat Plate	2775.0	1350.0	0.49	1st quality grit blast and 72 hrs salt air	Same as lower surface
	21B	Flat Plate	237.0	108.0	0.46	1st quality grit blast and 24 hrs salt air	Same as lower surface
	22B	Flat Plate	232.0	125.0	0.54	1st quality grit blast and 168 hrs salt water	Same as lower surface
	23B	Flat Plate	932.0	369.0	0.40	1st quality grit blast and 168 hrs salt water	Same as lower surface
	24B	Flat Plate	932.0	438.0	0.47	1st quality grit blast	Same as lower surface
	25B	Flat Plate	587.0	302.0	0.51	1st quality grit blast and 24 hrs salt air	2nd quality grit blast
	26B	Flat Plate	587.0	288.0	0.49	1st quality grit blast and 168 hrs salt water	168 hrs salt water
	27B	Flat Plate	2888.0	1460.0	0.51	1st quality grit blast	Same as lower surface
	28B	Flat Plate	2888.0	1495.0	0.52	1st quality grit blast	Same as lower surface
	29B	Flat Plate	591.0	228.0	0.39	1st quality grit blast	2nd quality grit blast
	30B	Flat Plate	579.0	236.0	0.41	24 hrs salt air and 168 hrs salt water	336 hrs salt water
	31B	Flat Plate	2812.0	885.0	0.31	24 hrs salt air and 168 hrs salt water	Same as lower surface
	32B	Flat Plate	2888.0	979.0	0.34		Same as lower surface
	33B	Flat Plate	234.0	89.0	0.38	1st quality grit blast	Same as lower surface
	34B	Flat Plate	230.0	101.0	0.44	24 hrs salt air	Same as lower surface
	35B	Flat Plate	932.0	348.0	0.37	168 hrs salt water	Same as lower surface
	36B	Flat Plate	932.0	232.0	0.25	168 hrs salt water	Same as lower surface

Database 4.2.2: Slip strength database for flat plates

Ref. No.	Specimen No.	Ds (mm)	Ts (mm)	D (mm)	T (mm)	Dg (mm)	Tg (mm)	L (mm)	Ds/Ts	D/T	Dg/Tg	L/D	K	Sigma_cu (N/mm^2)	Sigma_s (N/mm^2)	Comments
	M/1	508.0	6.00	457.2	19.05	496.0	19.40	914.0	84.67	24.00	25.57	2.0	0.0123	73.16	1.199	
	M/2	508.0	6.00	457.2	19.05	496.0	19.40	914.0	84.67	24.00	25.57	2.0	0.0127	82.94	1.264	
	M/3	508.0	6.00	457.2	19.05	496.0	19.40	914.0	84.67	24.00	25.57	2.0	0.0126	80.34	0.764	
	M/4	508.0	6.00	457.2	19.05	496.0	19.40	914.0	84.67	24.00	25.57	2.0	0.0124	73.82	0.934	
	M/5	508.0	6.00	457.2	19.05	496.0	19.40	914.0	84.67	24.00	25.57	2.0	0.0127	82.94	0.664	
	M/6	508.0	6.00	457.2	19.05	496.0	19.40	914.0	84.67	24.00	25.57	2.0	0.0121	67.96	1.058	
	N/1	508.0	6.00	457.2	19.05	496.0	19.40	914.0	84.67	24.00	25.57	2.0	0.0096	2.44	0.342	
	N/2	508.0	6.00	457.2	19.05	496.0	19.40	914.0	84.67	24.00	25.57	2.0	0.0102	19.37	0.805	
	N/3	508.0	6.00	457.2	19.05	496.0	19.40	914.0	84.67	24.00	25.57	2.0	0.0113	46.05	1.424	
	N/4	508.0	6.00	457.2	19.05	496.0	19.40	914.0	84.67	24.00	25.57	2.0	0.0096	4.55	0.496	
	N/5	508.0	6.00	457.2	19.05	496.0	19.40	914.0	84.67	24.00	25.57	2.0	0.0111	42.10	1.197	
	N/6	508.0	6.00	457.2	19.05	496.0	19.40	914.0	84.67	24.00	25.57	2.0	0.0103	20.74	0.938	
	N/7	508.0	6.00	457.2	19.05	496.0	19.40	914.0	84.67	24.00	25.57	2.0	0.0102	17.67	0.731	
	N/8	508.0	6.00	457.2	19.05	496.0	19.40	914.0	84.67	24.00	25.57	2.0	0.0099	11.15	0.534	
	N/9	508.0	6.00	457.2	19.05	496.0	19.40	914.0	84.67	24.00	25.57	2.0	0.0113	46.85	0.963	
4.1	N/10	508.0	6.00	457.2	19.05	496.0	19.40	914.0	84.67	24.00	25.57	2.0	0.0109	37.13	1.180	
&	A1	340.0	11.18	220.0	6.35	317.6	48.82	440.0	30.41	34.65	6.51	2.0	0.0241	50.00	1.758	
4.3	A4	502.0	9.53	356.0	7.94	482.9	63.47	712.0	52.68	44.84	7.61	2.0	0.0177	50.00	1.903	
	A5	657.0	11.11	508.0	19.05	634.8	63.39	1016.0	59.14	26.67	10.01	2.0	0.0173	50.00	1.248	
	A6	1067.0	38.10	838.0	25.40	990.8	76.40	1676.0	28.01	32.99	12.97	2.0	0.0208	50.00	1.655	
	A7	1524.0	12.70	1372.0	50.80	1498.6	63.30	2744.0	120.00	27.01	23.67	2.0	0.0092	50.00	0.352	
	A8	343.0	12.70	248.0	12.70	317.6	34.80	496.0	27.01	19.53	9.13	2.0	0.0277	50.00	1.999	
	A9	686.0	12.70	610.0	25.40	660.6	25.30	1220.0	54.02	24.02	26.11	2.0	0.0150	50.00	0.724	
	A10	1575.0	38.10	1372.0	50.80	1498.8	63.40	2744.0	41.34	27.01	23.64	2.0	0.0170	50.00	1.027	
	E1.1	508.0	6.00	457.0	19.00	496.0	19.50	914.0	84.67	24.05	25.44	2.0	0.0129	88.32	1.048	
	E1.2	508.0	6.00	457.0	19.00	496.0	19.50	914.0	84.67	24.05	25.44	2.0	0.0124	75.83	1.110	
	E2.1	508.0	6.00	457.0	19.00	496.0	19.50	914.0	84.67	24.05	25.44	2.0	0.0137	107.28	1.041	
	E2.2	508.0	6.00	457.0	19.00	496.0	19.50	914.0	84.67	24.05	25.44	2.0	0.0138	110.42	1.090	

Database 4.3.1: Plain pipe continuous sleeve unstressed grouted connections (continued..)

Ref. No.	Specimen No.	Ds (mm)	Ts (mm)	D (mm)	T (mm)	Dg (mm)	Tg (mm)	L (mm)	Ds/Ts	D/T	Dg/Tg	L/D	K	Sigma_cu (N/mm^2)	Sigma_s (N/mm^2)	Comments
	X1	838.0	12.70	762.0	15.90	812.6	25.30	1524.0	65.98	47.92	32.12	2.0	0.0103	42.50	0.306	
	X2	838.0	12.70	762.0	32.00	812.6	25.30	1524.0	65.98	23.81	32.12	2.0	0.0127	42.50	0.438	
	X3	838.0	12.70	762.0	15.90	812.6	25.30	1524.0	65.98	47.92	32.12	2.0	0.0103	42.50	0.968	
	X4	838.0	12.70	762.0	32.00	812.6	25.30	1524.0	65.98	23.81	32.12	2.0	0.0127	42.50	1.039	
	X5	838.0	12.70	762.0	15.90	812.6	25.30	1524.0	65.98	47.92	32.12	2.0	0.0103	42.50	0.825	
	X6	838.0	12.70	762.0	32.00	812.6	25.30	1524.0	65.98	23.81	32.12	2.0	0.0127	42.50	0.897	
	X7	838.0	12.70	762.0	15.90	812.6	25.30	1524.0	65.98	47.92	32.12	2.0	0.0103	42.50	0.805	
	X8	838.0	12.70	762.0	32.00	812.6	25.30	1524.0	65.98	23.81	32.12	2.0	0.0127	42.50	1.070	
4.1	X9	838.0	12.70	762.0	15.90	812.6	25.30	1524.0	65.98	47.92	32.12	2.0	0.0103	42.50	0.815	
&	X10	838.0	12.70	762.0	15.90	812.6	25.30	1524.0	65.98	47.92	32.12	2.0	0.0103	42.50	1.110	
4.3	X11	838.0	12.70	762.0	32.00	812.6	25.30	1524.0	65.98	23.81	32.12	2.0	0.0127	42.50	0.897	
	X12	838.0	12.70	762.0	32.00	812.6	25.30	1524.0	65.98	23.81	32.12	2.0	0.0127	42.50	0.917	
	X13	854.0	20.70	762.0	15.90	812.6	25.30	1524.0	41.26	47.92	32.12	2.0	0.0127	42.50	0.672	
	X14	854.0	20.70	762.0	15.90	812.6	25.30	1524.0	41.26	47.92	32.12	2.0	0.0127	42.50	0.621	
	X15	854.0	20.70	762.0	28.00	812.6	25.30	1524.0	41.26	27.21	32.12	2.0	0.0161	42.50	0.581	
	X16	854.0	20.70	762.0	28.00	812.6	25.30	1524.0	41.26	27.21	32.12	2.0	0.0161	42.50	0.937	
	X17	854.0	20.70	762.0	15.90	812.6	25.30	1524.0	41.26	47.92	32.12	2.0	0.0127	42.50	0.642	
	X18	854.0	20.70	762.0	15.90	812.6	25.30	1524.0	41.26	47.92	32.12	2.0	0.0127	42.50	0.703	
	X19	854.0	20.70	762.0	15.90	812.6	25.30	1524.0	41.26	47.92	32.12	2.0	0.0127	42.50	0.734	
	X20	854.0	20.70	762.0	28.00	812.6	25.30	1524.0	41.26	27.21	32.12	2.0	0.0161	42.50	1.243	
	OB-1	425.0	13.00	350.0	13.50	399.0	24.50	1400.0	32.69	25.93	16.29	4.0	0.0199	39.50	1.090	
	OB-2	425.0	13.00	350.0	13.50	399.0	24.50	1400.0	32.69	25.93	16.29	4.0	0.0196	34.80	0.780	
4.20	OB-3	425.0	13.00	350.0	13.50	399.0	24.50	1400.0	32.69	25.93	16.29	4.0	0.0198	38.40	0.800	
	OB-4	425.0	13.00	350.0	13.50	399.0	24.50	1400.0	32.69	25.93	16.29	4.0	0.0200	41.40	1.110	
	OB-5	425.0	13.00	350.0	13.50	399.0	24.50	1400.0	32.69	25.93	16.29	4.0	0.0196	35.10	0.930	
	OB-6	425.0	13.00	350.0	13.50	399.0	24.50	1400.0	32.69	25.93	16.29	4.0	0.0195	33.40	0.910	
	B1	568.8	4.98	508.0	25.00	558.8	25.40	1016.0	114.21	20.32	22.00	2.0	0.0105	61.10	0.510	
4.21	B2	568.8	4.98	508.0	16.00	558.8	25.40	1016.0	114.21	31.75	22.00	2.0	0.0100	62.30	0.270	
	B3	568.8	4.98	508.0	12.50	558.8	25.40	1016.0	114.21	40.64	22.00	2.0	0.0098	67.00	0.470	

Database 4.3.1: Plain pipe continuous sleeve unstressed grouted connections (..continued)

Ref. No.	Specimen No.	Ds (mm)	Ts (mm)	D (mm)	T (mm)	Dg (mm)	Tg (mm)	L (mm)	Ds/Ts	D/T	Dg/Tg	L/D	K	Sigma_cu (N/mm^2)	Sigma_s (N/mm^2)	Comments
	SSA-1	267.4	6.60	216.3	8.20	254.2	18.95	50.0	40.52	26.38	13.41	0.2	0.0169	19.91	4.591	All specimens had
	SSA-2	267.4	6.60	216.3	8.20	254.2	18.95	50.0	40.52	26.38	13.41	0.2	0.0169	19.91	4.866	expansive grout
	SSA-3	267.4	6.60	216.3	8.20	254.2	18.95	50.0	40.52	26.38	13.41	0.2	0.0169	19.91	5.170	
	SSB-1	267.4	6.60	216.3	8.20	254.2	18.95	250.0	40.52	26.38	13.41	1.2	0.0169	19.91	3.178	
	SSB-2	267.4	6.60	216.3	8.20	254.2	18.95	250.0	40.52	26.38	13.41	1.2	0.0169	19.91	4.738	
	SSB-3	267.4	6.60	216.3	8.20	254.2	18.95	250.0	40.52	26.38	13.41	1.2	0.0169	19.91	4.522	
	SSC-1	267.4	6.60	216.3	8.20	254.2	18.95	500.0	40.52	26.38	13.41	2.3	0.0169	19.91	2.786	
	SSC-2	267.4	6.60	216.3	8.20	254.2	18.95	500.0	40.52	26.38	13.41	2.3	0.0169	19.91	3.689	
	SSC-3	267.4	6.60	216.3	8.20	254.2	18.95	500.0	40.52	26.38	13.41	2.3	0.0169	19.91	3.031	
	SSD-1	267.4	6.60	216.3	8.20	254.2	18.95	750.0	40.52	26.38	13.41	3.5	0.0169	19.91	2.796	
	SSD-2	267.4	6.60	216.3	8.20	254.2	18.95	750.0	40.52	26.38	13.41	3.5	0.0169	19.91	3.012	
	SSD-3	267.4	6.60	216.3	8.20	254.2	18.95	750.0	40.52	26.38	13.41	3.5	0.0169	19.91	2.119	
	SSE-1	267.4	6.60	216.3	8.20	254.2	18.95	1000.0	40.52	26.38	13.41	4.6	0.0169	19.91	1.776	
	SSE-2	267.4	6.60	216.3	8.20	254.2	18.95	1000.0	40.52	26.38	13.41	4.6	0.0169	19.91	2.158	
	SSE-3	267.4	6.60	216.3	8.20	254.2	18.95	1000.0	40.52	26.38	13.41	4.6	0.0169	19.91	2.246	
	SSF	267.4	6.60	216.3	8.20	254.2	18.95	50.0	40.52	26.38	13.41	0.2	0.0169	19.91	3.002	
	SSG	267.4	6.60	216.3	8.20	254.2	18.95	250.0	40.52	26.38	13.41	1.2	0.0169	19.91	2.139	
	SSH	267.4	6.60	216.3	8.20	254.2	18.95	500.0	40.52	26.38	13.41	2.3	0.0169	19.91	1.687	
4.19	SSI	267.4	6.60	216.3	8.20	254.2	18.95	50.0	40.52	26.38	13.41	0.2	0.0169	19.91	1.903	
	SSJ	267.4	6.60	216.3	8.20	254.2	18.95	250.0	40.52	26.38	13.41	1.2	0.0169	19.91	2.011	
	SSK	267.4	6.60	216.3	8.20	254.2	18.95	500.0	40.52	26.38	13.41	2.3	0.0169	19.91	1.550	
	SSL-1	267.4	6.60	216.3	8.20	254.2	18.95	50.0	40.52	26.38	13.41	0.2	0.0160	7.65	2.374	
	SSL-2	267.4	6.60	216.3	8.20	254.2	18.95	50.0	40.52	26.38	13.41	0.2	0.0160	7.65	2.462	
	SSL-3	267.4	6.60	216.3	8.20	254.2	18.95	50.0	40.52	26.38	13.41	0.2	0.0160	7.65	2.021	
	SSM-1	267.4	6.60	216.3	8.20	254.2	18.95	250.0	40.52	26.38	13.41	1.2	0.0160	7.65	1.344	
	SSM-2	267.4	6.60	216.3	8.20	254.2	18.95	250.0	40.52	26.38	13.41	1.2	0.0160	7.65	1.393	
	SSM-3	267.4	6.60	216.3	8.20	254.2	18.95	250.0	40.52	26.38	13.41	1.2	0.0160	7.65	2.031	
	SSN-1	267.4	6.60	216.3	8.20	254.2	18.95	500.0	40.52	26.38	13.41	2.3	0.0160	7.65	1.256	
	SSN-2	267.4	6.60	216.3	8.20	254.2	18.95	500.0	40.52	26.38	13.41	2.3	0.0160	7.65	2.090	
	SSN-3	267.4	6.60	216.3	8.20	254.2	18.95	500.0	40.52	26.38	13.41	2.3	0.0160	7.65	1.511	
	SLA-1	711.2	11.10	606.9	10.30	689.0	41.05	100.0	64.07	58.92	16.78	0.2	0.0097	19.91	3.630	
	SLA-2	711.2	11.10	606.9	10.30	689.0	41.05	100.0	64.07	58.92	16.78	0.2	0.0097	19.91	2.796	
	SLB-1	711.2	11.10	606.9	10.30	689.0	41.05	500.0	64.07	58.92	16.78	0.8	0.0097	19.91	2.610	
	SLB-2	711.2	11.10	606.9	10.30	689.0	41.05	500.0	64.07	58.92	16.78	0.8	0.0097	19.91	3.375	
	SLC-1	711.2	11.10	606.9	10.30	689.0	41.05	1000.0	64.07	58.92	16.78	1.6	0.0097	19.91	2.168	
	SLC-2	711.2	11.10	606.9	10.30	689.0	41.05	1000.0	64.07	58.92	16.78	1.6	0.0097	19.91	1.805	
	SLD-1	711.2	11.10	606.9	10.30	689.0	41.05	1500.0	64.07	58.92	16.78	2.5	0.0097	19.91	1.815	
	SLD-2	711.2	11.10	606.9	10.30	689.0	41.05	1500.0	64.07	58.92	16.78	2.5	0.0097	19.91	1.442	

Database 4.3.2: Plain pipe continuous sleeve unstressed grouted connections (expansive grout)

Ref. No.	Specimen No.	Ds (mm)	Ts (mm)	D (mm)	T (mm)	Dg (mm)	Tg (mm)	L (mm)	Ds/Ts	D/T	Dg/Tg	L/D	K	h/s	D/s	Sigma_cu (N/mm^2)	Sigma_s (N/mm^2)	Comments
	O/1	366.0	4.90	323.8	10.30	356.2	16.20	323.0	74.69	31.44	21.99	1.0	0.0098	0.0120	3.0000	2.65	0.5464	
	O/2	366.0	4.90	323.8	10.30	356.2	16.20	323.0	74.69	31.44	21.99	1.0	0.0099	0.0120	3.0000	3.69	0.6676	
	O/3	366.0	4.90	323.8	10.30	356.2	16.20	647.0	74.69	31.44	21.99	2.0	0.0099	0.0120	3.0000	3.52	0.8783	
	O/4	366.0	4.90	323.8	10.30	356.2	16.20	647.0	74.69	31.44	21.99	2.0	0.0099	0.0120	3.0000	4.71	0.9649	
	O/5	366.0	4.90	323.8	10.30	356.2	16.20	1295.0	74.69	31.44	21.99	4.0	0.0101	0.0120	3.0000	8.26	1.5510	
	O/6	366.0	4.90	323.8	10.30	356.2	16.20	1295.0	74.69	31.44	21.99	4.0	0.0101	0.0120	3.0000	7.72	1.4520	
	O/7	366.0	4.90	323.8	10.30	356.2	16.20	2591.0	74.69	31.44	21.99	8.0	0.0099	0.0120	3.0000	3.48	0.8695	
	O/8	366.0	4.90	323.8	10.30	356.2	16.20	2591.0	74.69	31.44	21.99	8.0	0.0099	0.0120	3.0000	3.73	1.0500	
	O/9	366.0	4.90	323.8	10.30	356.2	16.20	3886.0	74.69	31.44	21.99	12.0	0.0098	0.0120	3.0000	2.22	0.6900	
	O/10	366.0	4.90	323.8	10.30	356.2	16.20	3886.0	74.69	31.44	21.99	12.0	0.0099	0.0120	3.0000	3.93	0.9390	
	P/1	574.8	8.00	508.0	25.40	558.8	25.38	1016.0	71.84	20.00	22.02	2.0	0.0137	0.0120	3.0000	54.21	2.7891	
	P/2	574.8	8.00	508.0	25.40	558.8	25.38	1016.0	71.84	20.00	22.02	2.0	0.0134	0.0120	3.0000	47.59	2.9331	
	P/3	568.8	5.00	508.0	25.40	558.8	25.38	1016.0	113.75	20.00	22.02	2.0	0.0102	0.0120	3.0000	53.42	2.2835	
	P/4	568.8	5.00	508.0	25.40	558.8	25.38	1016.0	113.75	20.00	22.02	2.0	0.0101	0.0120	3.0000	50.37	2.2132	
	P/5	566.9	4.00	508.0	25.40	558.9	25.47	1016.0	141.73	20.00	21.95	2.0	0.0092	0.0120	3.0000	59.61	1.8621	
	P/6	566.9	4.00	508.0	25.40	558.9	25.47	1016.0	141.73	20.00	21.95	2.0	0.0088	0.0120	3.0000	51.27	1.7005	
	P/7	574.8	8.00	508.0	15.88	558.8	25.38	1016.0	71.84	31.99	22.02	2.0	0.0123	0.0120	3.0000	53.08		
	P/8	574.8	8.00	508.0	15.88	558.8	25.38	1016.0	71.84	31.99	22.02	2.0	0.0124	0.0120	3.0000	53.26	2.7048	
	P/9	568.8	5.00	508.0	15.88	558.8	25.38	1016.0	113.75	31.99	22.02	2.0	0.0096	0.0120	3.0000	53.57	2.0728	
4.1	P/10	568.8	5.00	508.0	15.88	558.8	25.38	1016.0	113.75	31.99	22.02	2.0	0.0095	0.0120	3.0000	52.06	2.1395	
&	P/11	566.9	4.00	508.0	15.88	558.9	25.47	1016.0	141.73	31.99	21.95	2.0	0.0087	0.0120	3.0000	58.87	1.7918	
4.3	P/12	566.9	4.00	508.0	15.88	558.9	25.47	1016.0	141.73	31.99	21.95	2.0	0.0086	0.0120	3.0000	55.60	1.8621	
	P/13	574.8	8.00	508.0	12.70	558.8	25.38	1016.0	71.84	40.00	22.02	2.0	0.0115	0.0120	3.0000	49.91	2.4224	
	P/14	574.8	8.00	508.0	12.70	558.8	25.38	1016.0	71.84	40.00	22.02	2.0	0.0117	0.0120	3.0000	54.37	2.4064	
	P/15	568.8	5.00	508.0	12.70	558.8	25.38	1016.0	113.75	40.00	22.02	2.0	0.0094	0.0120	3.0000	57.47	2.1430	
	P/16	568.8	5.00	508.0	12.70	558.8	25.38	1016.0	113.75	40.00	22.02	2.0	0.0092	0.0120	3.0000	52.68	2.0552	
	P/17	566.9	4.00	508.0	12.70	558.9	25.47	1016.0	141.73	40.00	21.95	2.0	0.0084	0.0120	3.0000	57.14	1.7743	
	P/18	566.9	4.00	508.0	12.70	558.9	25.47	1016.0	141.73	40.00	21.95	2.0	0.0082	0.0120	3.0000	53.42	1.8902	
	P/19	508.0	6.00	457.2	19.05	496.0	19.40	914.4	84.67	24.00	25.57	2.0	0.0115	0.0120	3.0000	51.20	1.8024	
	P/20	508.0	6.00	457.2	19.05	496.0	19.40	914.4	84.67	24.00	25.57	2.0	0.0115	0.0120	3.0000	53.08	3.0010	
	P/21	583.8	12.50	508.0	25.40	558.8	25.40	1016.0	46.70	20.00	22.00	2.0	0.0163	0.0120	3.0000	21.20	2.3770	
	P/22	583.8	12.50	508.0	25.40	558.8	25.40	1016.0	46.70	20.00	22.00	2.0	0.0161	0.0120	3.0000	18.60	2.5820	
	P/23	583.8	12.50	508.0	15.80	558.8	25.40	1016.0	46.70	32.15	22.00	2.0	0.0139	0.0120	3.0000	20.30	2.7460	
	P/24	583.8	12.50	508.0	15.80	558.8	25.40	1016.0	46.70	32.15	22.00	2.0	0.0140	0.0120	3.0000	23.10	2.6640	
	P/25	583.8	12.50	508.0	12.70	558.8	25.40	1016.0	46.70	40.00	22.00	2.0	0.0133	0.0120	3.0000	31.80	2.7860	
	P/26	583.8	12.50	508.0	12.70	558.8	25.40	1016.0	46.70	40.00	22.00	2.0	0.0127	0.0120	3.0000	18.20	2.1310	
	P/27	598.8	20.00	508.0	25.40	558.8	25.40	1016.0	29.94	20.00	22.00	2.0	0.0213	0.0120	3.0000	22.40	2.6640	
	P/28	598.8	20.00	508.0	25.40	558.8	25.40	1016.0	29.94	20.00	22.00	2.0	0.0212	0.0120	3.0000	19.10	2.7860	
	P/29	598.8	20.00	508.0	15.88	558.8	25.40	1016.0	29.94	31.99	22.00	2.0	0.0174	0.0120	3.0000	20.90	2.6640	
	P/30	598.8	20.00	508.0	15.88	558.8	25.40	1016.0	29.94	31.99	22.00	2.0	0.0175	0.0120	3.0000	23.10	3.3600	
	P/31	598.8	20.00	508.0	12.70	558.8	25.40	1016.0	29.94	40.00	22.00	2.0	0.0155	0.0120	3.0000	20.70	2.7910	
	P/32	598.8	20.00	508.0	12.70	558.8	25.40	1016.0	29.94	40.00	22.00	2.0	0.0158	0.0120	3.0000	26.90	2.5690	

Database 4.3.3: Weld beaded continuous unstressed sleeve grouted connections (continued..)

Ref. No.	Specimen No.	Ds (mm)	Ts (mm)	D (mm)	T (mm)	Dg (mm)	Tg (mm)	L (mm)	Ds/Ts	D/T	Dg/Tg	L/D	K	h/s	D/s	Sigma_cu (N/mm^2)	Sigma_s (N/mm^2)	Comments
	Q/1	568.8	5.00	508.0	15.88	558.8	25.38	508.0	113.75	31.99	22.02	1.0	0.0086	0.0120	3.0000	31.76	2.1430	
	Q/2	568.8	5.00	508.0	15.88	558.8	25.38	508.0	113.75	31.99	22.02	1.0	0.0088	0.0120	3.0000	36.29	2.0728	
	Q/3	568.8	5.00	508.0	15.88	558.8	25.38	508.0	113.75	31.99	22.02	1.0	0.0085	0.0240	6.0000	30.10	3.3018	
	Q/4	568.8	5.00	508.0	15.88	558.8	25.38	508.0	113.75	31.99	22.02	1.0	0.0084	0.0240	6.0000	27.61	2.9155	
	Q/5	568.8	5.00	508.0	15.88	558.8	25.38	508.0	113.75	31.99	22.02	1.0	0.0088	0.0320	8.0000	36.26	3.8145	
	Q/6	568.8	5.00	508.0	15.88	558.8	25.38	508.0	113.75	31.99	22.02	1.0	0.0087	0.0320	8.0000	33.50	3.8987	
	R/1	568.8	5.00	508.0	15.88	558.8	25.40	1016.0	113.76	31.99	22.00	2.0	0.0089	0.0060	3.0000	38.15	1.7220	
	R/2	568.8	5.00	508.0	15.88	558.8	25.40	1016.0	113.76	31.99	22.00	2.0	0.0093	0.0060	3.0000	45.89	1.8620	
	R/3	568.8	5.00	508.0	15.88	558.8	25.40	1016.0	113.76	31.99	22.00	2.0	0.0088	0.0180	3.0000	35.56	2.4940	
	R/4	568.8	5.00	508.0	15.88	558.8	25.40	1016.0	113.76	31.99	22.00	2.0	0.0090	0.0180	3.0000	39.35	2.5990	
	R/5	543.4	5.00	508.0	15.88	533.4	12.68	1016.0	108.67	31.99	42.06	2.0	0.0081	0.0120	3.0000	33.94	2.2132	
	R/6	543.4	5.00	508.0	15.88	533.4	12.68	1016.0	108.67	31.99	42.06	2.0	0.0084	0.0120	3.0000	45.89	2.3540	
	R/7	619.6	5.00	508.0	15.88	609.6	50.78	1016.0	123.91	31.99	12.00	2.0	0.0100	0.0120	3.0000	35.79	2.4941	
	R/8	619.6	5.00	508.0	15.88	609.6	50.78	1016.0	123.91	31.99	12.00	2.0	0.0107	0.0120	3.0000	44.53	2.5117	
	R/9	568.8	5.00	508.0	15.88	558.8	25.38	1016.0	113.75	31.99	22.02	2.0	0.0093	0.0120	3.0000	46.75	2.3540	
	R/10	568.8	5.00	508.0	15.88	558.8	25.38	1016.0	113.75	31.99	22.02	2.0	0.0095	0.0120	3.0000	50.83	2.0730	
	R/11	568.8	5.00	508.0	15.88	558.8	25.38	1016.0	113.75	31.99	22.02	2.0	0.0086	0.0240	3.0000	30.50	2.4170	
	R/12	568.8	5.00	508.0	15.88	558.8	25.38	1016.0	113.75	31.99	22.02	2.0	0.0085	0.0240	3.0000	29.40	2.3030	
	R/13	568.8	5.00	508.0	15.88	558.8	25.38	1016.0	113.75	31.99	22.02	2.0	0.0082	0.0300	3.0000	23.70	3.0320	
4.1	R/14	568.8	5.00	508.0	15.88	558.8	25.38	1016.0	113.75	31.99	22.02	2.0	0.0082	0.0300	3.0000	22.30	2.6220	
&	R/15	568.8	5.00	508.0	15.88	558.8	25.38	1016.0	113.75	31.99	22.02	2.0	0.0083	0.0360	3.0000	24.30	3.3190	
4.3	R/16	568.8	5.00	508.0	15.88	558.8	25.38	1016.0	113.75	31.99	22.02	2.0	0.0081	0.0360	3.0000	21.30	3.3190	
	B1.1	502.0	3.75	457.0	14.27	494.5	18.75	914.0	133.87	32.03	26.37	2.0	0.0066	0.0162	3.6560	8.68	1.0140	
	B1.2	502.0	3.75	457.0	14.27	494.5	18.75	914.0	133.87	32.03	26.37	2.0	0.0066	0.0162	3.6560	8.68	1.0690	
	B2.1	502.0	3.75	457.0	14.27	494.5	18.75	914.0	133.87	32.03	26.37	2.0	0.0074	0.0162	3.6560	30.22	1.4480	
	B2.2	502.0	3.75	457.0	14.27	494.5	18.75	914.0	133.87	32.03	26.37	2.0	0.0074	0.0162	3.6560	30.22	1.4620	
	B3.1	502.0	3.75	457.0	14.27	494.5	18.75	914.0	133.87	32.03	26.37	2.0	0.0088	0.0162	3.6560	65.26	2.0690	
	B3.2	502.0	3.75	457.0	14.27	494.5	18.75	914.0	133.87	32.03	26.37	2.0	0.0086	0.0162	3.6560	60.66	2.0830	
	C1.1	502.0	3.75	457.0	14.27	494.5	18.75	914.0	133.87	32.03	26.37	2.0	0.0067	0.0162	3.6560	9.94	0.8800	
	C1.2	502.0	3.75	457.0	14.27	494.5	18.75	914.0	133.87	32.03	26.37	2.0	0.0067	0.0162	3.6560	9.94	0.9900	
	C2.1	502.0	3.75	457.0	14.27	494.5	18.75	914.0	133.87	32.03	26.37	2.0	0.0075	0.0162	3.6560	32.32	1.4500	
	C2.2	502.0	3.75	457.0	14.27	494.5	18.75	914.0	133.87	32.03	26.37	2.0	0.0075	0.0162	3.6560	32.32	1.4500	
	C3.1	502.0	3.75	457.0	14.27	494.5	18.75	914.0	133.87	32.03	26.37	2.0	0.0088	0.0162	3.6560	65.59	2.1200	
	C3.2	502.0	3.75	457.0	14.27	494.5	18.75	914.0	133.87	32.03	26.37	2.0	0.0087	0.0162	3.6560	64.10	2.4300	
	C4.1	502.0	3.75	457.0	14.27	494.5	18.75	914.0	133.87	32.03	26.37	2.0	0.0084	0.0162	3.6560	55.34	2.3900	
	C4.2	502.0	3.75	457.0	14.27	494.5	18.75	914.0	133.87	32.03	26.37	2.0	0.0091	0.0162	3.6560	73.66	2.2600	
	D1.1	502.0	3.75	457.0	14.27	494.5	18.75	914.0	133.87	32.03	26.37	2.0	0.0078	0.0162	3.6560	41.34	1.5900	
	D1.2	502.0	3.75	457.0	14.27	494.5	18.75	914.0	133.87	32.03	26.37	2.0	0.0078	0.0162	3.6560	41.34	1.5200	
	D2.1	502.0	3.75	457.0	14.27	494.5	18.75	914.0	133.87	32.03	26.37	2.0	0.0084	0.0162	3.6560	54.96	2.2100	
	D2.2	502.0	3.75	457.0	14.27	494.5	18.75	914.0	133.87	32.03	26.37	2.0	0.0084	0.0162	3.6560	54.96	2.2200	
	D3.1	502.0	3.75	457.0	14.27	494.5	18.75	914.0	133.87	32.03	26.37	2.0	0.0097	0.0162	3.6560	90.37	2.5300	
	D3.2	502.0	3.75	457.0	14.27	494.5	18.75	914.0	133.87	32.03	26.37	2.0	0.0093	0.0162	3.6560	79.71	2.4600	
	D4.1	502.0	3.75	457.0	14.27	494.5	18.75	914.0	133.87	32.03	26.37	2.0	0.0098	0.0162	3.6560	92.69	2.5000	
	D4.2	502.0	3.75	457.0	14.27	494.5	18.75	914.0	133.87	32.03	26.37	2.0	0.0097	0.0162	3.6560	89.04	2.5900	

Database 4.3.3: Weld beaded continuous unstressed sleeve grouted connections (..continued..)

Ref. No.	Specimen No.	Ds (mm)	Ts (mm)	D (mm)	T (mm)	Dg (mm)	Tg (mm)	L (mm)	Ds/Ts	D/T	Dg/Tg	L/D	K	h/s	D/s	Sigma_cu (N/mm^2)	Sigma_s (N/mm^2)	Comments
	ABP.1								93.20	33.90		2.0	0.0096	0.0364		4.66	1.6360	
	ABP.2								93.20	33.90		2.0	0.0096	0.0364		16.38	3.0930	
	ABP.3								93.20	33.90		2.0	0.0096	0.0364		7.46	1.7860	
	ABP.4								93.20	33.90		2.0	0.0096	0.0364		19.00	3.7120	
	ABP.5								93.20	33.90		2.0	0.0096	0.0364		27.95	4.9380	
4.1	ABP.6								93.20	33.90		2.0	0.0096	0.0364		13.53	2.9150	
	ABP.7								93.20	33.90		2.0	0.0096	0.0364		34.38	4.5810	
	ABP.8								93.20	33.90		2.0	0.0096	0.0364		43.36	5.3540	
	ABP.9								93.20	33.90		2.0	0.0096	0.0364		55.50	5.2350	
	ABP.10								93.20	33.90		2.0	0.0096	0.0364		61.50	5.0570	
	ABP.11								93.20	33.90		2.0	0.0096	0.0364		56.97	5.1160	
	ABP.12								93.20	33.90		2.0	0.0096	0.0364		73.75	5.1820	
4.9	3	725.0	8.00	610.0	12.70	709.0	49.50	1220.0	90.63	48.03	14.32	2.00	0.0021	0.0330	4.0060	21.10	2.7370	
	A1	568.0	4.98	508.0	25.00	558.0	25.02	1016.0	114.06	20.32	22.30	2.0	0.0109	0.0120	3.0000	71.30	2.2300	S. Ten./P. Ten.
	A2	568.0	4.98	508.0	16.00	558.0	25.02	1016.0	114.06	31.75	22.30	2.0	0.0101	0.0120	3.0000	66.10	2.0100	S. Ten./P. Ten.
	A3	568.0	4.98	508.0	12.50	558.0	25.02	1016.0	114.06	40.64	22.30	2.0	0.0099	0.0120	3.0000	70.60	2.0100	S. Ten./P. Ten.
4.21	C1	689.4	6.03	508.0	16.00	677.3	84.67	1016.0	114.33	31.75	8.00	2.0	0.0129	0.0120	3.0000	41.50	3.0300	S. Com./P. Ten.
	D1	560.8	4.90	508.0	16.00	551.0	21.48	1016.0	114.44	31.75	25.65	2.0	0.0075	0.0420	3.0000	9.52	2.0600	S. Com./P. Ten.
	E1	598.8	20.00	508.0	30.00	558.8	25.40	1016.0	29.94	16.93	22.00	2.0	0.0228	0.0120	3.0000	25.40	3.3800	S. Com./P. Ten.
	F1	598.8	20.00	508.0	9.50	558.8	25.40	1016.0	29.94	53.47	22.00	2.0	0.0148	0.0120	3.0000	54.60	2.7900	S. Com./P. Ten.
	C1	354.0	4.00	324.0	9.50	346.0	11.00	293.0	88.50	34.11	31.45	0.9	0.0089	0.0364	7.0000	15.20	2.3600	S. Com./P. Ten.
	C2	354.0	4.00	324.0	9.50	346.0	11.00	293.0	88.50	34.11	31.45	0.9	0.0091	0.0364	7.0000	22.90	4.3300	S. Com./P. Ten.
	C3	354.0	4.00	324.0	9.50	346.0	11.00	293.0	88.50	34.11	31.45	0.9	0.0098	0.0364	7.0000	43.90	5.3400	S. Com./P. Ten.
	Z1	354.0	4.00	324.0	9.50	346.0	11.00	293.0	88.50	34.11	31.45	0.9	0.0086	0.0727	7.0000	8.70	3.3100	S. Com./P. Ten.
	Z2	354.0	4.00	324.0	9.50	346.0	11.00	293.0	88.50	34.11	31.45	0.9	0.0089	0.0727	7.0000	17.30	4.5900	S. Com./P. Ten.
4.22	Z3	354.0	4.00	324.0	9.50	346.0	11.00	293.0	88.50	34.11	31.45	0.9	0.0101	0.0727	7.0000	52.90	7.8900	S. Com./P. Ten.
	Z4	354.0	4.00	324.0	9.50	346.0	11.00	293.0	88.50	34.11	31.45	0.9	0.0090	0.1094	7.0000	19.70	3.5600	S. Com./P. Ten.
	Z5	354.0	4.00	324.0	9.50	346.0	11.00	293.0	88.50	34.11	31.45	0.9	0.0087	0.1094	7.0000	11.30	2.6700	S. Com./P. Ten.
	Z6	354.0	4.00	324.0	9.50	346.0	11.00	293.0	88.50	34.11	31.45	0.9	0.0102	0.1094	7.0000	57.70	5.8000	S. Com./P. Ten.
	Z7	354.0	4.00	324.0	9.50	346.0	11.00	293.0	88.50	34.11	31.45	0.9	0.0094	0.1448	7.0000	31.90	3.4700	S. Com./P. Ten.
	Z8	354.0	4.00	324.0	9.50	346.0	11.00	293.0	88.50	34.11	31.45	0.9	0.0103	0.1448	7.0000	61.10	6.1100	S. Com./P. Ten.
	W1	360.0	5.00	300.0	8.00	350.0	25.00	315.0	72.00	37.50	14.00	1.1	0.0132	0.0171	4.2857	50.00	2.2740	S. Com./P. Ten.
	W2	360.0	5.00	300.0	8.00	350.0	25.00	315.0	72.00	37.50	14.00	1.1	0.0139	0.0257	4.2857	60.50	2.4250	S. Com./P. Ten.
4.23	W3	360.0	5.00	300.0	8.00	350.0	25.00	315.0	72.00	37.50	14.00	1.1	0.0133	0.0171	4.2857	51.50	1.9700	S. Com./P. Ten.
	W4	360.0	5.00	300.0	8.00	350.0	25.00	315.0	72.00	37.50	14.00	1.1	0.0132	0.0257	4.2857	50.10	2.5610	S. Com./P. Ten.
	W5	360.0	5.00	300.0	8.00	350.0	25.00	315.0	72.00	37.50	14.00	1.1	0.0141	0.0171	4.2857	63.00	2.0800	S. Com./P. Ten.
	W6	360.0	5.00	300.0	8.00	350.0	25.00	315.0	72.00	37.50	14.00	1.1	0.0142	0.0257	4.2857	63.90	2.7590	S. Com./P. Ten.
	IA1								36.00	18.66	6.48		0.0211	0.0270		11.70	2.9800	S. Com./P. Ten.
	IA1/2								36.00	18.66	6.48		0.0234	0.0270		26.40	4.5400	S. Com./P. Ten.
	IA2								36.00	18.66	6.48		0.0232	0.0270		25.00	4.6500	S. Com./P. Ten.
	IB1								36.00	18.66	6.48		0.0208	0.0270		9.60	1.1600	S. Com./P. Ten.
4.24	IB1/2								36.00	18.66	6.48		0.0234	0.0270		26.20	3.3400	S. Com./P. Ten.
	IB3								36.00	18.66	6.48		0.0243	0.0270		32.00	3.6500	S. Com./P. Com.
	IC1								36.00	18.66	6.48		0.0224	0.0270		19.70	6.7200	S. Com./P. Com.
	IC2								36.00	18.66	6.48		0.0228	0.0270		22.40	6.8100	S. Com./P. Com.
	ID1								36.00	18.66	6.48		0.0263	0.0270		45.10	7.7100	S. Com./P. Com.
	ID2								36.00	18.66	6.48		0.0226	0.0270		21.10	4.4500	S. Com./P. Com.

Database 4.3.3: Weld beaded continuous unstressed sleeve grouted connections (..continued)

Ref. No.	Specimen No.	Ds (mm)	Ts (mm)	D (mm)	T (mm)	Dg (mm)	Tg (mm)	L (mm)	Ds/Ts	D/T	Dg/Tg	L/D	K	Sigma_cu (N/mm^2)	Sigma_s (N/mm^2)	Comments
	S1	254.0	5.00	229.0	6.30	244.0	7.50	510.0	50.80	36.35	32.53	2.23	0.0123	21.00	0.940	
	S1A	254.0	5.00	229.0	6.30	244.0	7.50	519.0	50.80	36.35	32.53	2.27	0.0132	48.70	1.300	
	S2	254.0	5.00	229.0	6.30	244.0	7.50	520.0	50.80	36.35	32.53	2.27	0.0126	31.30	0.570	
	S15	250.0	3.00	229.0	6.30	244.0	7.50	510.0	83.33	36.35	32.53	2.23	0.0093	23.20	0.750	
	S16	250.0	3.00	229.0	6.30	244.0	7.50	520.0	83.33	36.35	32.53	2.27	0.0095	29.10	0.800	
	S23	254.0	5.00	229.0	6.30	244.0	7.50	970.0	50.80	36.35	32.53	4.24	0.0126	29.60	0.920	
	S23A	254.0	5.00	229.0	6.30	244.0	7.50	970.0	50.80	36.35	32.53	4.24	0.0130	43.50	0.770	
	S24	254.0	5.00	229.0	6.30	244.0	7.50	975.0	50.80	36.35	32.53	4.26	0.0129	41.30	0.810	
	S31	508.0	10.00	457.0	12.70	488.0	15.50	970.0	50.80	35.98	31.48	2.12	0.0131	42.60	0.730	
	S32	508.0	10.00	457.0	12.70	488.0	15.50	920.0	50.80	35.98	31.48	2.01	0.0135	55.20	0.940	
4.1	1.2.3	254.0	5.00	229.0	6.30	244.0	7.50	458.0	50.80	36.35	32.53	2.00	0.0133	54.30	0.600	Under Water Cleaning
&	1.2.4	254.0	5.00	229.0	6.30	244.0	7.50	458.0	50.80	36.35	32.53	2.00	0.0137	65.30	0.540	Under Water Cleaning
4.3																Pressure Grouting psi/days
	1.5.1	254.0	5.00	229.0	6.30	244.0	7.50	460.0	50.80	36.35	32.53	2.01	0.0134	57.50	0.620	0/7
	1.5.2	254.0	5.00	229.0	6.30	244.0	7.50	460.0	50.80	36.35	32.53	2.01	0.0138	70.40	0.960	0/7
	1.5.3	254.0	5.00	229.0	6.30	244.0	7.50	460.0	50.80	36.35	32.53	2.01	0.0136	60.90	0.730	50/7
	1.5.4	254.0	5.00	229.0	6.30	244.0	7.50	460.0	50.80	36.35	32.53	2.01	0.0136	61.90	0.910	50/180
	1.5.5	254.0	5.00	229.0	6.30	244.0	7.50	370.0	50.80	36.35	32.53	1.62	0.0132	48.20	0.140	200/7
	1.5.6	254.0	5.00	229.0	6.30	244.0	7.50	370.0	50.80	36.35	32.53	1.62	0.0139	73.00	0.140	200/180
	1.5.7	254.0	5.00	229.0	6.30	244.0	7.50	460.0	50.80	36.35	32.53	2.01	0.0137	64.40	0.140	1000/7
	1.5.8	254.0	5.00	229.0	6.30	244.0	7.50	460.0	50.80	36.35	32.53	2.01	0.0137	65.10	0.960	1000/180

Database 4.3.4: Plain pipe split sleeve unstressed grouted connections (axial tension)

Ref. No.	Specimen No.	Ds (mm)	Ts (mm)	D (mm)	T (mm)	Dg (mm)	Tg (mm)	L (mm)	Ds/Ts	D/T	Dg/Tg	L/D	K	Sigma_cu (N/mm^2)	Sigma_s (N/mm^2)	Comments
	S3	254.0	5.00	229.0	6.30	244.0	7.50	515.0	50.80	36.35	32.53	2.25	0.0124	25.00	1.210	
	S4	254.0	5.00	229.0	6.30	244.0	7.50	515.0	50.80	36.35	32.53	2.25	0.0126	29.30	1.330	
	S17	250.0	3.00	229.0	6.30	244.0	7.50	480.0	83.33	36.35	32.53	2.10	0.0093	24.00	1.120	
4.1	S17A	250.0	3.00	229.0	6.30	244.0	7.50	510.0	83.33	36.35	32.53	2.23	0.0101	49.20	2.000	
	S18	250.0	3.00	229.0	6.30	244.0	7.50	510.0	83.33	36.35	32.53	2.23	0.0092	21.90	1.230	
	S25	254.0	5.00	229.0	6.30	244.0	7.50	970.0	50.80	36.35	32.53	4.24	0.0126	29.00	1.210	
	S26	254.0	5.00	229.0	6.30	244.0	7.50	970.0	50.80	36.35	32.53	4.24	0.0125	25.90	1.090	

Database 4.3.5: Plain pipe split sleeve unstressed grouted connections (axial compression)

Ref. No.	Specimen No.	Ds (mm)	Ts (mm)	D (mm)	T (mm)	Dg (mm)	Tg (mm)	L (mm)	Ds/Ts	D/T	Dg/Tg	L/D	K	Sigma_cu (N/mm^2)	Sigma_s (N/mm^2)	Comments
4.1	S5	254.0	5.00	229.0	6.30	244.0	7.50	510.0	50.80	36.35	32.53	2.23	0.0123	19.80		Pure Bending(91.5kNm)+No Failure
	S6	254.0	5.00	229.0	6.30	244.0	7.50	510.0	50.80	36.35	32.53	2.23	0.0118	5.00		Pure Bending(89.6kNm)+No Failure
	S7	254.0	5.00	229.0	6.30	244.0	7.50	252.0	50.80	36.35	32.53	1.10	0.0118	3.20		Pure Bending(54.5kNm)+No Failure
	S8	254.0	5.00	229.0	6.30	244.0	7.50	510.0	50.80	36.35	32.53	2.23	0.0130	42.50		Pure Bending(115.0kNm)+No Failure
	S19	250.0	3.00	229.0	6.30	244.0	7.50	510.0	83.33	36.35	32.53	2.23	0.0087	2.90		Pure Bending(86.0kNm)+No Failure
	S20	250.0	3.00	229.0	6.30	244.0	7.50	510.0	83.33	36.35	32.53	2.23	0.0089	10.10		Pure Bending(89.8kNm)+No Failure
	S11	254.0	5.00	229.0	6.30	244.0	7.50	514.0	50.80	36.35	32.53	2.24	0.0131	46.70	1.080	sigma_b/sigma_a=1
	S12	254.0	5.00	229.0	6.30	244.0	7.50	512.0	50.80	36.35	32.53	2.24	0.0129	38.20	1.030	sigma_b/sigma_a=1
	S21	250.0	3.00	229.0	6.30	244.0	7.50	507.0	83.33	36.35	32.53	2.21	0.0097	38.00	1.100	sigma_b/sigma_a=1
	S22	250.0	3.00	229.0	6.30	244.0	7.50	515.0	83.33	36.35	32.53	2.25	0.0098	41.70	1.080	sigma_b/sigma_a=1
	S29	254.0	5.00	229.0	6.30	244.0	7.50	970.0	50.80	36.35	32.53	4.24	0.0130	43.90	0.920	sigma_b/sigma_a=1
	S30	254.0	5.00	229.0	6.30	244.0	7.50	964.0	50.80	36.35	32.53	4.21	0.0130	43.50	0.770	sigma_b/sigma_a=1
	S9	254.0	5.00	229.0	6.30	244.0	7.50	518.0	50.80	36.35	32.53	2.26	0.0129	39.60	1.170	sigma_b/sigma_a=2
	S10	254.0	5.00	229.0	6.30	244.0	7.50	514.0	50.80	36.35	32.53	2.24	0.0131	44.90	1.080	sigma_b/sigma_a=2
	S13	254.0	5.00	229.0	6.30	244.0	7.50	520.0	50.80	36.35	32.53	2.27	0.0131	47.60	1.170	sigma_b/sigma_a=0.5
	S14	254.0	5.00	229.0	6.30	244.0	7.50	520.0	50.80	36.35	32.53	2.27	0.0135	58.80	1.120	sigma_b/sigma_a=0.5

Database 4.3.6: Plain pipe split sleeve unstressed grouted connections (bending and combined bending and tension)

Ref. No.	Specimen No.	Ds (mm)	Ts (mm)	D (mm)	T (mm)	Dg (mm)	Tg (mm)	L (mm)	Ds/Ts	D/T	Dg/Tg	L/D	K	h/s	D/s	Sigma_cu (N/mm^2)	Sigma_s (N/mm^2)	Comments
	T1	254.0	5.00	229.0	6.30	244.0	7.50	500.0	50.80	36.35	32.53	2.18	0.0133	0.0212	3.0500	53.00	3.3000	
	T2	254.0	5.00	229.0	6.30	244.0	7.50	500.0	50.80	36.35	32.53	2.18	0.0131	0.0212	3.0500	45.00	3.4000	
	T15	250.0	3.00	229.0	6.30	244.0	7.50	500.0	83.33	36.35	32.53	2.18	0.0097	0.0212	3.0500	37.30	3.2600	
	T16	250.0	3.00	229.0	6.30	244.0	7.50	515.0	83.33	36.35	32.53	2.25	0.0100	0.0212	3.0500	47.30	3.6100	
	T23	254.0	5.00	229.0	6.30	244.0	7.50	970.0	50.80	36.35	32.53	4.24	0.0130	0.0212	3.0500	42.00	2.7600	
	T24	254.0	5.00	229.0	6.30	244.0	7.50	960.0	50.80	36.35	32.53	4.19	0.0126	0.0212	3.0500	31.50	2.6400	
	T31	254.0	5.00	229.0	6.30	244.0	7.50	515.0	50.80	36.35	32.53	2.25	0.0123	0.0424	6.1000	21.20	3.1300	
	T32	254.0	5.00	229.0	6.30	244.0	7.50	500.0	50.80	36.35	32.53	2.18	0.0126	0.0424	6.1000	29.20	3.8800	
	T41	254.0	5.00	229.0	6.30	244.0	7.50	229.0	50.80	36.35	32.53	1.00	0.0124	0.0640	16.0000	22.00	4.3200	
	T42	254.0	5.00	229.0	6.30	244.0	7.50	229.0	50.80	36.35	32.53	1.00	0.0124	0.0640	16.0000	23.40	3.2800	
	T42A	254.0	5.00	229.0	6.30	244.0	7.50	229.0	50.80	36.35	32.53	1.00	0.0124	0.0640	16.0000	21.80	4.1500	
	T43	254.0	5.00	229.0	6.30	244.0	7.50	229.0	50.80	36.35	32.53	1.00	0.0121	0.0960	24.0000	13.80	2.9400	
	T44	254.0	5.00	229.0	6.30	244.0	7.50	229.0	50.80	36.35	32.53	1.00	0.0123	0.0960	24.0000	19.00	3.4600	
	T45	254.0	5.00	229.0	6.30	244.0	7.50	229.0	50.80	36.35	32.53	1.00	0.0121	0.0960	16.0000	14.50	2.9400	
	T46	254.0	5.00	229.0	6.30	244.0	7.50	229.0	50.80	36.35	32.53	1.00	0.0123	0.0960	16.0000	18.80	3.2800	
	T47	254.0	5.00	229.0	6.30	244.0	7.50	229.0	50.80	36.35	32.53	1.00	0.0124	0.0552	8.0000	22.10	3.4600	
	T48	254.0	5.00	229.0	6.30	244.0	7.50	229.0	50.80	36.35	32.53	1.00	0.0123	0.0552	8.0000	21.10	3.6300	
4.1	T49	254.0	5.00	229.0	6.30	244.0	7.50	229.0	50.80	36.35	32.53	1.00	0.0124	0.1104	16.0000	22.80	3.8000	
	T50	254.0	5.00	229.0	6.30	244.0	7.50	229.0	50.80	36.35	32.53	1.00	0.0123	0.1104	16.0000	20.80	3.6300	
	T51	261.0	3.00	240.0	4.00	255.0	7.50	480.0	87.00	60.00	34.00	2.00	0.0076	0.0212	3.0000	19.40	1.3000	
	T52	261.0	3.00	240.0	4.00	255.0	7.50	480.0	87.00	60.00	34.00	2.00	0.0082	0.0212	3.0000	41.60	2.6700	
	T52A	261.0	3.00	240.0	4.00	255.0	7.50	480.0	87.00	60.00	34.00	2.00	0.0081	0.0212	3.0000	37.20	1.8900	
	T53	265.0	5.00	240.0	4.00	255.0	7.50	480.0	53.00	60.00	34.00	2.00	0.0100	0.0212	3.0000	34.00	2.0500	
	T54	265.0	5.00	240.0	4.00	255.0	7.50	480.0	53.00	60.00	34.00	2.00	0.0101	0.0212	3.0000	35.50	2.2800	
	1.1.1	250.0	3.00	229.0	3.20	244.0	7.50	519.8	83.33	71.56	32.53	2.27	0.0084	0.0212	3.0500	55.60	2.0000	
	1.1.3	250.0	3.00	229.0	3.20	244.0	7.50	519.8	83.33	71.56	32.53	2.27	0.0082	0.0212	3.0500	51.10	2.2200	
	1.1.2	228.0	9.50	193.7	8.00	209.0	7.65	515.2	24.00	24.21	27.32	2.66	0.0229	0.0212	2.5800	52.20	3.6800	
	1.1.4	228.0	9.50	193.7	8.00	209.0	7.65	515.2	24.00	24.21	27.32	2.66	0.0228	0.0212	2.5800	48.80	3.7300	
	1.2.1	254.0	5.00	229.0	6.30	244.0	7.50	460.0	50.80	36.35	32.53	2.01	0.0133	0.0212	3.0500	54.30	2.7800	Under Water Cleaning
	1.2.2	254.0	5.00	229.0	6.30	244.0	7.50	460.0	50.80	36.35	32.53	2.01	0.0134	0.0212	3.0500	56.60	2.6000	Under Water Cleaning
	A1	355.0	5.00	305.0	6.30	345.0	20.00	660.0	71.00	48.41	17.25	2.16	0.0095	0.0333	4.0600	12.26	1.9400	
	A2	355.0	5.00	305.0	6.30	345.0	20.00	655.0	71.00	48.41	17.25	2.15	0.0097	0.0333	4.0600	15.32	2.2200	
	A3	355.0	5.00	305.0	6.30	345.0	20.00	660.0	71.00	48.41	17.25	2.16	0.0089	0.0333	4.0600	3.11	0.9000	
	A7.2	355.0	5.00	305.0	6.30	345.0	20.00	635.0	71.00	48.41	17.25	2.08	0.0098	0.0333	4.0600	18.54	2.5700	

Database 4.3.7: Weld beaded split sleeve unstressed grouted connections (axial tension)

Ref. No.	Specimen No.	Ds (mm)	Ts (mm)	D (mm)	T (mm)	Dg (mm)	Tg (mm)	L (mm)	Ds/Ts	D/T	Dg/Tg	L/D	K	h/s	D/s	Sigma_cu (N/mm^2)	Sigma_s (N/mm^2)	Comments
4.1	T4	254.0	5.00	229.0	6.30	244.0	7.50	520.0	50.80	36.35	32.53	2.27	0.0133	0.0212	3.0500	52.40	2.7900	
	T17	250.0	3.00	229.0	6.30	244.0	7.50	520.0	83.33	36.35	32.53	2.27	0.0100	0.0212	3.0500	47.10	2.8900	
	T18	250.0	3.00	229.0	6.30	244.0	7.50	515.0	83.33	36.35	32.53	2.25	0.0102	0.0212	3.0500	54.20	3.0000	
	T25	254.0	5.00	229.0	6.30	244.0	7.50	960.0	50.80	36.35	32.53	4.19	0.0124	0.0212	3.0500	22.60	2.4700	
	T26	254.0	5.00	229.0	6.30	244.0	7.50	970.0	50.80	36.35	32.53	4.24	0.0128	0.0212	3.0500	35.10	2.6100	
	T33	254.0	5.00	229.0	6.30	244.0	7.50	535.0	50.80	36.35	32.53	2.34	0.0121	0.0424	6.1000	12.90	2.4400	
	T34	254.0	5.00	229.0	6.30	244.0	7.50	550.0	50.80	36.35	32.53	2.40	0.0123	0.0424	6.1000	20.10	3.1700	
4.21	G1	445.0	5.00	406.0	12.50	435.0	14.50	812.0	89.00	32.48	30.00	2.00	0.0099	0.0210	3.0000	43.71	2.1100	
	G2	445.0	5.00	406.0	12.50	435.0	14.50	812.0	89.00	32.48	30.00	2.00	0.0097	0.0210	3.0000	36.34	2.4300	
	G3	445.0	5.00	406.0	12.50	435.0	14.50	812.0	89.00	32.48	30.00	2.00	0.0106	0.0210	3.0000	64.44	2.9700	
	H1	445.0	5.00	406.0	12.50	435.0	14.50	812.0	89.00	32.48	30.00	2.00	0.0101	0.0220	3.0000	50.59	2.4900	
	H2	445.0	5.00	406.0	12.50	435.0	14.50	812.0	89.00	32.48	30.00	2.00	0.0099	0.0220	3.0000	44.11	2.4300	
	H3	445.0	5.00	406.0	12.50	435.0	14.50	812.0	89.00	32.48	30.00	2.00	0.0102	0.0220	3.0000	52.66	3.1100	
	J1	445.0	5.00	406.0	12.50	435.0	14.50	812.0	89.00	32.48	30.00	2.00	0.0101	0.0230	3.0000	48.65	2.0500	
	J2	445.0	5.00	406.0	12.50	435.0	14.50	812.0	89.00	32.48	30.00	2.00	0.0099	0.0230	3.0000	42.40	2.4300	
	J3	445.0	5.00	406.0	12.50	435.0	14.50	812.0	89.00	32.48	30.00	2.00	0.0105	0.0230	3.0000	60.47	2.9700	

Database 4.3.8: Weld beaded split sleeve unstressed grouted connections (axial compression)

Ref. No.	Specimen No.	Ds (mm)	Ts (mm)	D (mm)	T (mm)	Dg (mm)	Tg (mm)	L (mm)	Ds/Ts	D/T	Dg/Tg	L/D	K	h/s	D/s	Sigma_cu (N/mm^2)	Sigma_s (N/mm^2)	Comments
4.1	T9	254.0	5.00	229.0	6.30	244.0	7.50	520.0	50.80	36.35	32.53	2.27	0.0130	0.0212	3.0500	43.20	3.2500	sigma_b/sigma_a=2
	T10	254.0	5.00	229.0	6.30	244.0	7.50	515.0	50.80	36.35	32.53	2.25	0.0136	0.0212	3.0500	61.30	3.2800	sigma_b/sigma_a=2
	T11	254.0	5.00	229.0	6.30	244.0	7.50	520.0	50.80	36.35	32.53	2.27	0.0133	0.0212	3.0500	52.20	3.2500	sigma_b/sigma_a=1
	T12	254.0	5.00	229.0	6.30	244.0	7.50	510.0	50.80	36.35	32.53	2.23	0.0127	0.0212	3.0500	33.60	3.4100	sigma_b/sigma_a=1
	T21	250.0	3.00	229.0	6.30	244.0	7.50	520.0	83.33	36.35	32.53	2.27	0.0097	0.0212	3.0500	35.60	3.3500	sigma_b/sigma_a=1
	T22	250.0	3.00	229.0	6.30	244.0	7.50	530.0	83.33	36.35	32.53	2.31	0.0096	0.0212	3.0500	34.90	3.0900	sigma_b/sigma_a=1
	T29	254.0	5.00	229.0	6.30	244.0	7.50	965.0	50.80	36.35	32.53	4.21	0.0124	0.0212	3.0500	23.30	2.1900	sigma_b/sigma_a=1
	T30	254.0	5.00	229.0	6.30	244.0	7.50	960.0	50.80	36.35	32.53	4.19	0.0134	0.0212	3.0500	54.90	>2.6300	sigma_b/sigma_a=1
	T37	254.0	5.00	229.0	6.30	244.0	7.50	520.0	50.80	36.35	32.53	2.27	0.0124	0.0212	6.1000	24.40	>2.2300	sigma_b/sigma_a=1
	T38	254.0	5.00	229.0	6.30	244.0	7.50	520.0	50.80	36.35	32.53	2.27	0.0121	0.0212	6.1000	12.40	>2.5400	sigma_b/sigma_a=1
	T13	254.0	5.00	229.0	6.30	244.0	7.50	505.0	50.80	36.35	32.53	2.21	0.0127	0.0212	3.0500	34.60	3.4500	sigma_b/sigma_a=5
	T14	254.0	5.00	229.0	6.30	244.0	7.50	520.0	50.80	36.35	32.53	2.27	0.0125	0.0212	3.0500	27.70	2.4400	sigma_b/sigma_a=5
	A4	355.0	5.00	305.0	6.30	345.0	20.00	623.0	71.00	48.41	17.25	2.04	0.0090	0.0333	4.0600	3.90	1.0800	sigma_b/sigma_a=0.416
	A5	355.0	5.00	305.0	6.30	345.0	20.00	623.0	71.00	48.41	17.25	2.04	0.0095	0.0333	4.0600	12.20	>2.1600	sigma_b/sigma_a=0.416
	A6	355.0	5.00	305.0	6.30	345.0	20.00	620.0	71.00	48.41	17.25	2.03	0.0090	0.0333	4.0600	3.27	0.9000	sigma_b/sigma_a=0.416
	A7.1	355.0	5.00	305.0	6.30	345.0	20.00	635.0	71.00	48.41	17.25	2.08	0.0098	0.0333	4.0600	18.54	>2.1800	sigma_b/sigma_a=0.416
	A8	355.0	5.00	305.0	6.30	345.0	20.00	640.0	71.00	48.41	17.25	2.10	0.0095	0.0333	4.0600	13.36	2.5300	sigma_b/sigma_a=0.416
	A9	355.0	5.00	305.0	6.30	345.0	20.00	630.0	71.00	48.41	17.25	2.07	0.0094	0.0333	4.0600	10.29	2.1400	sigma_b/sigma_a=0.416

Database 4.3.9: Weld beaded split sleeve unstressed grouted connections (combined bending and tension)

Ref. No.	Specimen No.	Ds (mm)	Ts (mm)	D (mm)	T (mm)	Dg (mm)	Tg (mm)	L (mm)	Ds/Ts	D/T	Dg/Tg	L/D	K	h/s	D/s	% Bead	Sigma_cu (N/mm^2)	Sigma_s (N/mm^2)	Comments
	1.4.1	254.0	5.00	229.0	6.30	244.0	7.50	458.0	50.80	36.35	32.53	2.00	0.0139	0.0212	3.0500	87.50	70.80	2.6800	
	1.4.2	254.0	5.00	229.0	6.30	244.0	7.50	458.0	50.80	36.35	32.53	2.00	0.0131	0.0212	3.0500	87.50	47.21	2.7900	
	1.4.3	254.0	5.00	229.0	6.30	244.0	7.50	458.0	50.80	36.35	32.53	2.00	0.0132	0.0212	3.0500	75.00	50.00	2.5500	
4.1	1.4.4	254.0	5.00	229.0	6.30	244.0	7.50	458.0	50.80	36.35	32.53	2.00	0.0134	0.0212	3.0500	75.00	56.10	2.6800	
	1.4.5	254.0	5.00	229.0	6.30	244.0	7.50	458.0	50.80	36.35	32.53	2.00	0.0137	0.0212	3.0500	62.50	64.20	2.2900	
	1.4.6	254.0	5.00	229.0	6.30	244.0	7.50	458.0	50.80	36.35	32.53	2.00	0.0138	0.0212	3.0500	62.50	67.90	2.2900	
	1.4.7	254.0	5.00	229.0	6.30	244.0	7.50	458.0	50.80	36.35	32.53	2.00	0.0136	0.0212	3.0500	50.00	61.20	1.9600	
	1.4.8	254.0	5.00	229.0	6.30	244.0	7.50	458.0	50.80	36.35	32.53	2.00	0.0137	0.0212	3.0500	25.00	64.40	1.4800	

Database 4.3.10: Discontinuously weld beaded split sleeve unstressed grouted connections (axial tension)

Ref. No.	Specimen No.	Ds (mm)	Ts (mm)	D (mm)	T (mm)	Dg (mm)	Tg (mm)	L (mm)	Ds/Ts	D/T	Dg/Tg	L/D	K	(Phi*hs) /(Sh*Si)	Sigma_cu (N/mm^2)	Sigma_s (N/mm^2)	Comments
	1.4.1	292.0	5.00	229.0	6.30	282.00	26.50	458.0	58.40	36.35	10.64	2.00	0.0112	0.0213	50.60	5.7100	Wall Buckled
4.1	1.4.5	292.0	5.00	229.0	6.30	282.00	26.50	458.0	58.40	36.35	10.64	2.00	0.0112	0.0426	51.30	5.2800	Bond Slip
	1.4.6	292.0	5.00	229.0	6.30	282.00	26.50	458.0	58.40	36.35	10.64	2.00	0.0112	0.0426	49.00	5.4500	Bond Slip
	1.4.7	292.0	5.00	229.0	6.30	282.00	26.50	458.0	58.40	36.35	10.64	2.00	0.0112	0.0213	40.70	3.7100	Stud Yield

Specimen No.	Phi (mm)	h ds (mm)	S dh (mm)	S dl (mm)
1.4.1	12.00	12.00	90.00	75.00
1.4.5	8.45	8.45	45.00	75.00
1.4.6	8.45	8.45	45.00	75.00
1.4.7	6.00	12.00	45.00	75.00

Phi = Stud diameter
h ds = Stud height
S dh = Stud hoop spacing
S dl = Stud longitudinal spacing

Note:Studs on brace member,split sleeve had weld beads with h/s = 0.021

Database 4.3.11: Studded split sleeve unstressed grouted connections

Ref. No.	Specimen No.	Ds (mm)	Ts (mm)	D (mm)	T (mm)	Dg (mm)	Tg (mm)	L (mm)	Ds/Ts	D/T	Dg/Tg	L/D	K	h/s	D/s	Sigma_cu (N/mm^2)	Sigma_s (N/mm^2)	Comments
4.9	1	725.0	8.00	610.0	12.70	709.00	49.50	1220.0	90.63	48.03	14.32	2.00	0.0022	0.0330	4.0660	22.00	3.336	
	2	725.0	8.00	610.0	12.70	709.00	49.50	1220.0	90.63	48.03	14.32	2.00	0.0023	0.0330	4.0660	23.00	3.000	

Database 4.3.12: Stud/strap continuous sleeve connections

Ref. No.	Spec. No.	Clamp Type	Surface Condition	Ds (mm)	Ts (mm)	D (mm)	T (mm)	Dg (mm)	Tg (mm)	L (mm)	Ds/Ts	D/T	Dg/Tg	L/D	fcu (N/mm^2)	Pressure (N/mm^2)	Mod. Press. (N/mm^2)	Slip Strength (N/mm^2)
4.1	1/1	Stressed Studbolt	Shot Blasted	402.0	19.0	324.0	16.0	364.0	20.0	355.0	21.2	20.3	18.2	1.10	50.0	9.14	9.47	5.45
	2/1	Stressed Studbolt	Shot Blasted	402.0	19.0	324.0	9.5	364.0	20.0	355.0	21.2	34.1	18.2	1.10	50.0	9.75	10.10	4.81
	3/1	Stressed Studbolt	Shot Blasted	402.0	19.0	324.0	6.4	364.0	20.0	355.0	21.2	50.6	18.2	1.10	50.0	1.33	1.38	1.10
	4/1	Stressed Studbolt	Shot Blasted	402.0	19.0	324.0	9.5	364.0	20.0	755.0	21.2	34.1	18.2	2.33	50.0	11.39	11.80	4.88
	6/1	Stressed Studbolt	Shot Blasted	402.0	19.0	324.0	9.5	364.0	20.0	355.0	21.2	34.1	18.2	1.10	50.0	4.23	4.38	2.84
	7/1	Stressed Studbolt	Shot Blasted	402.0	19.0	324.0	9.5	364.0	20.0	355.0	21.2	34.1	18.2	1.10	50.0	2.25	2.33	1.82
	8/1	Stressed Studbolt	Shot Blasted	402.0	19.0	324.0	9.5	364.0	20.0	355.0	21.2	34.1	18.2	1.10	78.0	3.22	3.34	2.14
	9/1	Stressed Studbolt	Shot Blasted	402.0	19.0	324.0	9.5	364.0	20.0	355.0	21.2	34.1	18.2	1.10	78.0	3.92	4.06	2.68
	10/1	Stressed Studbolt	Shot Blasted	402.0	19.0	324.0	9.5	364.0	20.0	355.0	21.2	34.1	18.2	1.10	78.0	3.78	3.92	2.43
4.11	C1	Pressurised Grout	Shot Blasted	244.5	7.1	168.3	9.5	230.3	31.0	300.0	34.4	17.7	7.4	1.8		8.10	8.10	4.20
	F1	Pressurised Grout	Shot Blasted	244.5	7.1	168.3	9.5	230.3	31.0	300.0	34.4	17.7	7.4	1.8		17.60	17.60	6.80
	A2	Pressurised Grout	Shot Blasted	910.0	30.0	750.0	30.0	850.0	50.0	690.0	30.3	25.0	17.0	0.9	63.4	15.20	15.20	7.27
	B2	Pressurised Grout	Shot Blasted	910.0	30.0	750.0	30.0	850.0	50.0	1410.0	30.3	25.0	17.0	1.9	64.5	18.00	18.00	5.83
	C2	Pressurised Grout	Shot Blasted	900.0	25.0	750.0	15.0	850.0	50.0	690.0	36.0	50.0	17.0	0.9	64.2	8.70	8.70	4.35
	D2	Pressurised Grout	Shot Blasted	900.0	25.0	750.0	15.0	850.0	50.0	1410.0	36.0	50.0	17.0	1.9	63.5	9.20	9.20	3.28
4.25	2	Pressurised Grout	Shot Blasted	244.5	7.1	168.3	9.5	230.3	31.0	365.0	34.4	17.7	7.4	2.2	70.0	7.20	7.20	3.42
	3	Pressurised Grout	Shot Blasted	244.5	7.1	168.3	9.5	230.3	31.0	365.0	34.4	17.7	7.4	2.2	70.0	14.00	14.00	5.50
	4	Pressurised Grout	Shot Blasted	244.5	7.1	168.3	9.5	230.3	31.0	365.0	34.4	17.7	7.4	2.2	70.0	16.80	16.80	6.75
	5	Pressurised Grout	Shot Blasted	244.5	7.1	168.3	9.5	230.3	31.0	365.0	34.4	17.7	7.4	2.2	70.0	17.30	17.30	6.87

Database 4.4.1: Stressed grouted connections

REF No.	TEST ID	JOINT TYPE	D (mm)	T (mm)	d (mm)	t (mm)	Theta (degrees)	LOAD TYPE	LOCATION	MEASURED SCF Original	MEASURED SCF Clamped	SCF RATIO Clamped/Original	BETA	GAMMA	TAU	ROSS-SECTIONAL ARE BRACE (mm u2)	ROSS-SECTIONAL ARE ACE+CLA (mm u2)	AREA RATIO	2nd MOMENT OF AREA BRACE (mm u4)	2nd MOMENT OF AREA ACE+CLA (mm u4)	2nd MOMENT OF AREA RATIO
4.1		DT	609.60	12.70	323.90	9.50	90.0	Tens	CS	29.59	1.97	0.067	0.53	24.0	0.75	9.39E+03	7.11E+04	7.571			
		DT	609.60	12.70	323.90	9.50	90.0	Comp	CS	29.59	1.97	0.067	0.53	24.0	0.75	9.39E+03	7.11E+04	7.571			
		T	660.40	9.50	609.60	12.70	90.0	Tens	CS	31.08	6.18	0.199	0.92	34.8	1.34	2.38E+04	1.11E+05	4.649			
		T	660.40	9.50	609.60	12.70	90.0	Tens	BS	15.90	2.10	0.132	0.92	34.8	1.34	2.38E+04	1.11E+05	4.649			
		T	660.40	9.50	609.60	12.70	90.0	Comp	CS	31.20	6.78	0.217	0.92	34.8	1.34	2.38E+04	1.11E+05	4.649			
		T	660.40	9.50	609.60	12.70	90.0	Comp	BS	16.50	2.46	0.149	0.92	34.8	1.34	2.38E+04	1.11E+05	4.649			
		DT	609.60	12.70	323.90	9.50	90.0	IPB	CC	4.00	0.52	0.130	0.53	24.0	0.75				1.16E+08	1.61E+09	13.823
		DT	609.60	12.70	323.90	9.50	90.0	OPB	CS	12.65	0.58	0.046	0.53	24.0	0.75				1.16E+08	3.61E+09	31.034
		T	660.40	9.50	609.60	12.70	90.0	IPB	CC	8.04	1.60	0.199	0.92	34.8	1.34				1.06E+09	6.45E+09	6.085
		T	660.40	9.50	609.60	12.70	90.0	OPB	CS	38.12	6.32	0.166	0.92	34.8	1.34				1.06E+09	7.85E+09	7.406
		T	660.40	9.50	609.60	12.70	90.0	OPB	BS	18.62	5.00	0.269	0.92	34.8	1.34				1.06E+09	7.85E+09	7.406

NOTES

1. LOCATION

CS = Chord Saddle
CC = Chord Crown
BS = Brace Saddle
BC = Brace Crown

Database 4.7.1: SCFs for joints with stressed mechanical clamps

REF No.	TEST ID	JOINT TYPE	D (mm)	T (mm)	d (mm)	t (mm)	Theta (degrees)	LOAD TYPE	LOCATION	MEASURED SCF Original	MEASURED SCF Clamped	SCF RATIO Clamped/Original	BETA	GAMMA	TAU	ROSS-SECTIONAL ARE BRACE (mm^2)	ROSS-SECTIONAL ARE ACE+CLA (mm^2)	AREA RATIO	2nd MOMENT OF AREA BRACE (mm^4)	2nd MOMENT OF AREA ACE+CLA (mm^4)	2nd MOMENT OF AREA RATIO
	2Y	Y	508	12.7	273	9.3	45.0	Tens	CS	6.85	4.62	0.674	0.54	20.0	0.73	7.70E+05	1.76E+04	2.280			
		Y	508	12.7	273	9.3	45.0	Tens	BS	4.31	2.54	0.589	0.54	20.0	0.73	7.70E+05	1.76E+04	2.280			
		Y	508	12.7	273	9.3	45.0	Comp	CS	8.77	3.18	0.363	0.54	20.0	0.73	7.70E+05	1.76E+04	2.280			
		Y	508	12.7	273	9.3	45.0	Comp	BS	5.16	2.39	0.463	0.54	20.0	0.73	7.70E+05	1.76E+04	2.280			
	2T	T	508	12.7	273	9.3	90.0	Tens	CS	12.23	6.47	0.528	0.54	20.0	0.73	7.70E+05	1.76E+04	2.280			
		T	508	12.7	273	9.3	90.0	Tens	BS	7.91	4.24	0.536	0.54	20.0	0.73	7.70E+05	1.76E+04	2.280			
		T	508	12.7	273	9.3	90.0	Comp	CS	11.09	4.31	0.389	0.54	20.0	0.73	7.70E+05	1.76E+04	2.280			
		T	508	12.7	273	9.3	90.0	Comp	BS	8.22	3.04	0.370	0.54	20.0	0.73	7.70E+05	1.76E+04	2.280			
	2K	K	406	10	219	6	45.0	Bal Axial	CS	3.26	1.41	0.433	0.54	20.3	0.60	4.01E+05	9.05E+03	2.254			
		K	406	10	219	6	45.0	Bal Axial	BS	3.98	1.29	0.324	0.54	20.3	0.60	4.01E+05	9.05E+03	2.254			
		K	406	10	219	6	45.0	Tens	CS	4.54	1.44	0.317	0.54	20.3	0.60	4.01E+05	9.05E+03	2.254			
		K	406	10	219	6	45.0	Tens	BS	4.78	1.36	0.285	0.54	20.3	0.60	4.01E+05	9.05E+03	2.254			
		K	406	10	219	6	45.0	Comp	CS	4.54	2.08	0.458	0.54	20.3	0.60	4.01E+05	9.05E+03	2.254			
		K	406	10	219	6	45.0	Comp	BS	4.82	1.81	0.376	0.54	20.3	0.60	4.01E+05	9.05E+03	2.254			
		K_o	406	10	219	6	45.0	Bal Axial	CS	1.93	0.88	0.456	0.54	20.3	0.60	4.01E+05	9.05E+03	2.254			
		K_o	406	10	219	6	45.0	Bal Axial	BS	2.61	0.52	0.199	0.54	20.3	0.60	4.01E+05	9.05E+03	2.254			
		K_o	406	10	219	6	45.0	Tens	CS	6.32	2.13	0.337	0.54	20.3	0.60	4.01E+05	9.05E+03	2.254			
		K_o	406	10	219	6	45.0	Tens	BS	5.44	1.89	0.347	0.54	20.3	0.60	4.01E+05	9.05E+03	2.254			
		K_o	406	10	219	6	45.0	Comp	CS	6.26	2.23	0.359	0.54	20.3	0.60	4.01E+05	9.05E+03	2.254			
		K_o	406	10	219	6	45.0	Comp	BS	5.02	1.52	0.303	0.54	20.3	0.60	4.01E+05	9.05E+03	2.254			
	7X	X	305	6.3	305	6.3	72.0	Tens	CS	2.3	2	0.870	1.00	24.2	1.00	5.91E+05	1.14E+04	1.930			
4.1		X	305	6.3	305	6.3	72.0	Tens	BS	6.15	2.8	0.455	1.00	24.2	1.00	5.91E+05	1.14E+04	1.930			
		X	305	6.3	305	6.3	72.0	Comp	CS	2.3	1.6	0.696	1.00	24.2	1.00	5.91E+05	1.14E+04	1.930			
		X	305	6.3	305	6.3	72.0	Comp	BS	6.15	2	0.325	1.00	24.2	1.00	5.91E+05	1.14E+04	1.930			
	2Y	Y	508	12.7	273	9.3	45.0	IPB	CC	2.85	0.61	0.214	0.54	20.0	0.73				6.71E+07	1.89E+08	2.814
		Y	508	12.7	273	9.3	45.0	IPB	BC	3.44	0.39	0.113	0.54	20.0	0.73				6.71E+07	1.89E+08	2.814
		Y	508	12.7	273	9.3	45.0	OPB	CS	5.45	1.47	0.270	0.54	20.0	0.73				6.71E+07	1.89E+08	2.814
		Y	508	12.7	273	9.3	45.0	OPB	BS	3.73	0.81	0.217	0.54	20.0	0.73				6.71E+07	1.89E+08	2.814
		T	508	12.7	273	9.3	90.0	IPB	CC	3.24	0.38	0.117	0.54	20.0	0.73				6.71E+07	1.89E+08	2.814
		T	508	12.7	273	9.3	90.0	IPB	BC	1.72	0.31	0.180	0.54	20.0	0.73				6.71E+07	1.89E+08	2.814
		T	508	12.7	273	9.3	90.0	OPB	CS	7.91	3.19	0.403	0.54	20.0	0.73				6.71E+07	1.89E+08	2.814
		T	508	12.7	273	9.3	90.0	OPB	BS	5.45	2.24	0.411	0.54	20.0	0.73				6.71E+07	1.89E+08	2.814
	2K	K	406	10	219	6	45.0	Bal IPB	CC	1.94	0.22	0.113	0.54	20.3	0.60				2.28E+07	6.77E+07	2.970
		K	406	10	219	6	45.0	Bal IPB	CC	1.24	0.16	0.129	0.54	20.3	0.60				2.28E+07	6.77E+07	2.970
		K	406	10	219	6	45.0	Bal IPB	BC	1.43	0.29	0.203	0.54	20.3	0.60				2.28E+07	6.77E+07	2.970
		K	406	10	219	6	45.0	Bal IPB	BC	1.98	0.21	0.106	0.54	20.3	0.60				2.28E+07	6.77E+07	2.970
		K	406	10	219	6	45.0	IPB	CC	1.7	0.17	0.100	0.54	20.3	0.60				2.28E+07	6.77E+07	2.970
		K	406	10	219	6	45.0	IPB	CC	1.38	0.16	0.116	0.54	20.3	0.60				2.28E+07	6.77E+07	2.970
		K	406	10	219	6	45.0	IPB	BC	1.1	0.14	0.127	0.54	20.3	0.60				2.28E+07	6.77E+07	2.970
		K	406	10	219	6	45.0	IPB	BC	2.01	0.07	0.035	0.54	20.3	0.60				2.28E+07	6.77E+07	2.970
		K	406	10	219	6	45.0	Bal OPB	CS	4.97	1.48	0.298	0.54	20.3	0.60				2.28E+07	6.77E+07	2.970
		K	406	10	219	6	45.0	Bal OPB	BS	4.1	1.17	0.285	0.54	20.3	0.60				2.28E+07	6.77E+07	2.970
		K	406	10	219	6	45.0	OPB	CS	3.65	1.05	0.288	0.54	20.3	0.60				2.28E+07	6.77E+07	2.970
		K	406	10	219	6	45.0	OPB	BS	2.98	0.78	0.262	0.54	20.3	0.60				2.28E+07	6.77E+07	2.970
		K	406	10	219	6	45.0	OPB	CS	3.94	1.1	0.279	0.54	20.3	0.60				2.28E+07	6.77E+07	2.970
		K	406	10	219	6	45.0	OPB	BS	3.29	0.97	0.295	0.54	20.3	0.60				2.28E+07	6.77E+07	2.970
		K_o	406	10	219	6	45.0	Bal IPB	CC	1.14	0.11	0.096	0.54	20.3	0.60				2.28E+07	6.77E+07	2.970
		K_o	406	10	219	6	45.0	Bal IPB	BC	2.79	0.14	0.050	0.54	20.3	0.60				2.28E+07	6.77E+07	2.970
		K_o	406	10	219	6	45.0	Bal IPB	BC	1.71	0.11	0.064	0.54	20.3	0.60				2.28E+07	6.77E+07	2.970
		K_o	406	10	219	6	45.0	IPB	CC	1.98	0.2	0.101	0.54	20.3	0.60				2.28E+07	6.77E+07	2.970
		K_o	406	10	219	6	45.0	IPB	BC	1.94	0.22	0.113	0.54	20.3	0.60				2.28E+07	6.77E+07	2.970
		K_o	406	10	219	6	45.0	IPB	BC	2.35	0.15	0.064	0.54	20.3	0.60				2.28E+07	6.77E+07	2.970
		K_o	406	10	219	6	45.0	IPB	CC	1.7	0.2	0.118	0.54	20.3	0.60				2.28E+07	6.77E+07	2.970
		K_o	406	10	219	6	45.0	IPB	BC	1.22	0.08	0.066	0.54	20.3	0.60				2.28E+07	6.77E+07	2.970
		K_o	406	10	219	6	45.0	IPB	BC	1.78	0.14	0.079	0.54	20.3	0.60				2.28E+07	6.77E+07	2.970
		K_o	406	10	219	6	45.0	Bal OPB	CS	10.45	1.87	0.179	0.54	20.3	0.60				2.28E+07	6.77E+07	2.970
		K_o	406	10	219	6	45.0	Bal OPB	BS	4.31	0.35	0.081	0.54	20.3	0.60				2.28E+07	6.77E+07	2.970
		K_o	406	10	219	6	45.0	OPB	CS	4.62	0.86	0.186	0.54	20.3	0.60				2.28E+07	6.77E+07	2.970
		K_o	406	10	219	6	45.0	OPB	BS	2.5	0.46	0.184	0.54	20.3	0.60				2.28E+07	6.77E+07	2.970
		K_o	406	10	219	6	45.0	OPB	CS	4.24	0.96	0.226	0.54	20.3	0.60				2.28E+07	6.77E+07	2.970
		K_o	406	10	219	6	45.0	OPB	BS	2.99	0.71	0.237	0.54	20.3	0.60				2.28E+07	6.77E+07	2.970

NOTES

1. LOCATION: CS = Chord Saddle; CC = Chord Crown; BS = Brace Saddle; BC = Brace Crown

2. JOINT TYPE: K do = Overlap K Joint

Database 4.7.2: SCFs for joints with unstressed grouted clamps

T-4-7-3.XLS

REF No.	TEST ID	JOINT TYPE	D (mm)	T (mm)	d (mm)	t (mm)	Theta (degrees)	LOAD TYPE	LOCATION	MEASURED SCF Original	MEASURED SCF Clamped	SCF RATIO Clamped/Original	BETA	GAMMA	TAU	ROSS-SECTIONAL ARE BRACE (mm u2)	ROSS-SECTIONAL ARE ACE+CLA (mm u2)	AREA RATIO	2nd MOMENT OF AREA BRACE (mm u4)	2nd MOMENT OF AREA ACE+CLA (mm u4)	2nd MOMENT OF AREA RATIO
		DT	609.6	12.7	323.9	9.5	90.0	Tens	CS	29.59	2.2	0.074	0.53	24.0	0.75	9.39E+03	7.43E+04	7.912			
		DT	609.6	12.7	323.9	9.5	90.0	Tens	BS	19.33	1.58	0.082	0.53	24.0	0.75	9.39E+03	7.43E+04	7.912			
4.1		DT	609.6	12.7	323.9	9.5	90.0	Comp	CS	29.59	2.17	0.073	0.53	24.0	0.75	9.39E+03	7.43E+04	7.912			
		DT	609.6	12.7	323.9	9.5	90.0	Comp	BS	17.75	1.38	0.078	0.53	24.0	0.75	9.39E+03	7.43E+04	7.912			
		DT	609.6	12.7	323.9	9.5	90.0	OPB	CS	12.65	0.78	0.062	0.53	24.0	0.75				1.16E+08	3.80E+09	32.702
		DT	609.6	12.7	323.9	9.5	90.0	OPB	BS	8.58	0.6	0.070	0.53	24.0	0.75				1.16E+08	3.80E+09	32.702
	CHORD	Y	711.2	19.05	406.4	12.7	68.8	Tens	CS	7.39	3.35	0.453	0.57	18.7	0.67	1.57E+04	7.57E+04	4.822			
	LOAD	Y	711.2	19.05	406.4	12.7	68.8	Tens	CS	7.14			0.57	18.7	0.67	1.57E+04	7.57E+04	4.822			
	=	Y	711.2	19.05	406.4	12.7	68.8	Tens	BS	5.61	2.24	0.399	0.57	18.7	0.67	1.57E+04	7.57E+04	4.822			
	2030KN	Y	711.2	19.05	406.4	12.7	68.8	Tens	BS	5.14	1.79	0.348	0.57	18.7	0.67	1.57E+04	7.57E+04	4.822			
		Y	711.2	19.05	406.4	12.7	68.8	Tens	CC	3.49	2.28	0.653	0.57	18.7	0.67	1.57E+04	7.57E+04	4.822			
		Y	711.2	19.05	406.4	12.7	68.8	Tens	CC		0.59		0.57	18.7	0.67	1.57E+04	7.57E+04	4.822			
		Y	711.2	19.05	406.4	12.7	68.8	Tens	BC	2.35	1.45	0.617	0.57	18.7	0.67	1.57E+04	7.57E+04	4.822			
		Y	711.2	19.05	406.4	12.7	68.8	Tens	BC	0.78	0.24	0.308	0.57	18.7	0.67	1.57E+04	7.57E+04	4.822			
		Y	711.2	19.05	406.4	12.7	68.8	Comp	CS	-6.16			0.57	18.7	0.67	1.57E+04	7.57E+04	4.822			
		Y	711.2	19.05	406.4	12.7	68.8	Comp	CS	-8.08			0.57	18.7	0.67	1.57E+04	7.57E+04	4.822			
		Y	711.2	19.05	406.4	12.7	68.8	Comp	BS	-4.83	-1.84	0.381	0.57	18.7	0.67	1.57E+04	7.57E+04	4.822			
		Y	711.2	19.05	406.4	12.7	68.8	Comp	BS	-5.97	-1.27	0.213	0.57	18.7	0.67	1.57E+04	7.57E+04	4.822			
		Y	711.2	19.05	406.4	12.7	68.8	Comp	CC	-2.2	1.77	-0.805	0.57	18.7	0.67	1.57E+04	7.57E+04	4.822			
		Y	711.2	19.05	406.4	12.7	68.8	Comp	CC	-2.67	-0.31	0.116	0.57	18.7	0.67	1.57E+04	7.57E+04	4.822			
		Y	711.2	19.05	406.4	12.7	68.8	Comp	BC	-1.65	-1.06	0.642	0.57	18.7	0.67	1.57E+04	7.57E+04	4.822			
		Y	711.2	19.05	406.4	12.7	68.8	Comp	BC	-1.26	-0.2	0.159	0.57	18.7	0.67	1.57E+04	7.57E+04	4.822			
		Y	711.2	19.05	406.4	12.7	68.8	IPB+ve	CS	0.72			0.57	18.7	0.67				3.05E+08	7.51E+09	24.623
4.17		Y	711.2	19.05	406.4	12.7	68.8	IPB+ve	CS	0.51	-0.18	-0.353	0.57	18.7	0.67				3.05E+08	7.51E+09	24.623
		Y	711.2	19.05	406.4	12.7	68.8	IPB+ve	BS	0.82	-0.1	-0.122	0.57	18.7	0.67				3.05E+08	7.51E+09	24.623
		Y	711.2	19.05	406.4	12.7	68.8	IPB+ve	BS	0.76	-0.28	-0.368	0.57	18.7	0.67				3.05E+08	7.51E+09	24.623
		Y	711.2	19.05	406.4	12.7	68.8	IPB+ve	CC	-3.15	-0.64	0.203	0.57	18.7	0.67				3.05E+08	7.51E+09	24.623
		Y	711.2	19.05	406.4	12.7	68.8	IPB+ve	CC	2.17	-0.1	-0.046	0.57	18.7	0.67				3.05E+08	7.51E+09	24.623
		Y	711.2	19.05	406.4	12.7	68.8	IPB+ve	BC	-2.04	-0.31	0.152	0.57	18.7	0.67				3.05E+08	7.51E+09	24.623
		Y	711.2	19.05	406.4	12.7	68.8	IPB+ve	BC	1.8	0.04	0.022	0.57	18.7	0.67				3.05E+08	7.51E+09	24.623
		Y	711.2	19.05	406.4	12.7	68.8	IPB-ve	CS	-0.57	0.19	-0.333	0.57	18.7	0.67				3.05E+08	7.51E+09	24.623
		Y	711.2	19.05	406.4	12.7	68.8	IPB-ve	CS	-0.54			0.57	18.7	0.67				3.05E+08	7.51E+09	24.623
		Y	711.2	19.05	406.4	12.7	68.8	IPB-ve	BS	-0.72	0.13	-0.181	0.57	18.7	0.67				3.05E+08	7.51E+09	24.623
		Y	711.2	19.05	406.4	12.7	68.8	IPB-ve	BS	-0.76	0.28	-0.368	0.57	18.7	0.67				3.05E+08	7.51E+09	24.623
		Y	711.2	19.05	406.4	12.7	68.8	IPB-ve	CC	3.09	0.59	0.191	0.57	18.7	0.67				3.05E+08	7.51E+09	24.623
		Y	711.2	19.05	406.4	12.7	68.8	IPB-ve	CC	-2.05	0.1	-0.049	0.57	18.7	0.67				3.05E+08	7.51E+09	24.623
		Y	711.2	19.05	406.4	12.7	68.8	IPB-ve	BC	2.04	0.3	0.147	0.57	18.7	0.67				3.05E+08	7.51E+09	24.623
		Y	711.2	19.05	406.4	12.7	68.8	IPB-ve	BC	-1.8	-0.08	0.044	0.57	18.7	0.67				3.05E+08	7.51E+09	24.623
		Y	711.2	19.05	406.4	12.7	68.8	OPB	CS	-7.05	-0.46	0.065	0.57	18.7	0.67				3.05E+08	1.00E+10	32.787
		Y	711.2	19.05	406.4	12.7	68.8	OPB	CS		0.48		0.57	18.7	0.67				3.05E+08	1.00E+10	32.787
		Y	711.2	19.05	406.4	12.7	68.8	OPB	BS	-5.17	-0.21	0.041	0.57	18.7	0.67				3.05E+08	1.00E+10	32.787
		Y	711.2	19.05	406.4	12.7	68.8	OPB	BS	5.66	0.21	0.037	0.57	18.7	0.67				3.05E+08	1.00E+10	32.787
		Y	711.2	19.05	406.4	12.7	68.8	OPB	CC	0.4	0.13	0.325	0.57	18.7	0.67				3.05E+08	1.00E+10	32.787
		Y	711.2	19.05	406.4	12.7	68.8	OPB	CC	0.25	0.05	0.200	0.57	18.7	0.67				3.05E+08	1.00E+10	32.787
		Y	711.2	19.05	406.4	12.7	68.8	OPB	BC	0.46	0.06	0.130	0.57	18.7	0.67				3.05E+08	1.00E+10	32.787
		Y	711.2	19.05	406.4	12.7	68.8	OPB	BC	0.33	0.07	0.212	0.57	18.7	0.67				3.05E+08	1.00E+10	32.787

NOTES

1. LOCATION

CS = Chord Saddle
CC = Chord Crown

BS = Brace Saddle
BC = Brace Crown

Database 4.7.3: SCFs for joints with stressed grouted clamps (continued..)

MSL

REF No.	TEST ID	JOINT TYPE	D (mm)	T (mm)	d (mm)	t (mm)	Theta (degrees)	LOAD TYPE	LOCATION	MEASURED SCF Original	MEASURED SCF Clamped	SCF RATIO Clamped/Original	BETA	GAMMA	TAU	ROSS-SECTIONAL ARE BRACE (mm u2)	ROSS-SECTIONAL ARE ACE+CLA (mm u2)	AREA RATIO	2nd MOMENT OF AREA BRACE (mm u4)	2nd MOMENT OF AREA ACE+CLA (mm u4)	2nd MOMENT OF AREA RATIO
	CHORD	Y	711.2	19.05	406.4	12.7	68.8	Tens	CS	6.67	3.82	0.573	0.57	18.7	0.67	1.57E+04	7.57E+04	4.822			
	LOAD	Y	711.2	19.05	406.4	12.7	68.8	Tens	CS	6.59	4.13	0.627	0.57	18.7	0.67	1.57E+04	7.57E+04	4.822			
	-	Y	711.2	19.05	406.4	12.7	68.8	Tens	BS	4.59	2.1	0.458	0.57	18.7	0.67	1.57E+04	7.57E+04	4.822			
	4530KN	Y	711.2	19.05	406.4	12.7	68.8	Tens	BS	5.02	1.59	0.317	0.57	18.7	0.67	1.57E+04	7.57E+04	4.822			
		Y	711.2	19.05	406.4	12.7	68.8	Tens	CC	3.41	2.16	0.633	0.57	18.7	0.67	1.57E+04	7.57E+04	4.822			
		Y	711.2	19.05	406.4	12.7	68.8	Tens	CC	1.92	0.59	0.307	0.57	18.7	0.67	1.57E+04	7.57E+04	4.822			
		Y	711.2	19.05	406.4	12.7	68.8	Tens	BC	2.2	1.22	0.555	0.57	18.7	0.67	1.57E+04	7.57E+04	4.822			
		Y	711.2	19.05	406.4	12.7	68.8	Tens	BC	0.59	0.24	0.407	0.57	18.7	0.67	1.57E+04	7.57E+04	4.822			
		Y	711.2	19.05	406.4	12.7	68.8	Comp	CS	-5.89	-3	0.509	0.57	18.7	0.67	1.57E+04	7.57E+04	4.822			
		Y	711.2	19.05	406.4	12.7	68.8	Comp	CS	-7.85			0.57	18.7	0.67	1.57E+04	7.57E+04	4.822			
		Y	711.2	19.05	406.4	12.7	68.8	Comp	BS	-4.83			0.57	18.7	0.67	1.57E+04	7.57E+04	4.822			
		Y	711.2	19.05	406.4	12.7	68.8	Comp	BS	-5.81			0.57	18.7	0.67	1.57E+04	7.57E+04	4.822			
		Y	711.2	19.05	406.4	12.7	68.8	Comp	CC	-1.92	-1.61	0.839	0.57	18.7	0.67	1.57E+04	7.57E+04	4.822			
		Y	711.2	19.05	406.4	12.7	68.8	Comp	CC	-2.67	-0.31	0.116	0.57	18.7	0.67	1.57E+04	7.57E+04	4.822			
		Y	711.2	19.05	406.4	12.7	68.8	Comp	BC	-1.65	-1.06	0.642	0.57	18.7	0.67	1.57E+04	7.57E+04	4.822			
		Y	711.2	19.05	406.4	12.7	68.8	Comp	BC	-1.22	0	0.000	0.57	18.7	0.67	1.57E+04	7.57E+04	4.822			
		Y	711.2	19.05	406.4	12.7	68.8	IPB+ve	CS	0.66			0.57	18.7	0.67				3.05E+08	7.51E+09	24.623
		Y	711.2	19.05	406.4	12.7	68.8	IPB+ve	CS	0.55	-0.18	-0.327	0.57	18.7	0.67				3.05E+08	7.51E+09	24.623
		Y	711.2	19.05	406.4	12.7	68.8	IPB+ve	BS	2.77	-0.1	-0.036	0.57	18.7	0.67				3.05E+08	7.51E+09	24.623
		Y	711.2	19.05	406.4	12.7	68.8	IPB+ve	BS	0.78	-0.57	-0.474	0.57	18.7	0.67				3.05E+08	7.51E+09	24.623
4.17		Y	711.2	19.05	406.4	12.7	68.8	IPB+ve	CC	-3.04	-0.63	0.207	0.57	18.7	0.67				3.05E+08	7.51E+09	24.623
		Y	711.2	19.05	406.4	12.7	68.8	IPB+ve	CC	2.36	-0.1	-0.042	0.57	18.7	0.67				3.05E+08	7.51E+09	24.623
		Y	711.2	19.05	406.4	12.7	68.8	IPB+ve	BC	-2.01	-0.31	0.154	0.57	18.7	0.67				3.05E+08	7.51E+09	24.623
		Y	711.2	19.05	406.4	12.7	68.8	IPB+ve	BC	1.78	0.04	0.022	0.57	18.7	0.67				3.05E+08	7.51E+09	24.623
		Y	711.2	19.05	406.4	12.7	68.8	IPB-ve	CS	-0.56	0.16	-0.286	0.57	18.7	0.67				3.05E+08	7.51E+09	24.623
		Y	711.2	19.05	406.4	12.7	68.8	IPB-ve	CS	-0.57	0.42	-0.737	0.57	18.7	0.67				3.05E+08	7.51E+09	24.623
		Y	711.2	19.05	406.4	12.7	68.8	IPB-ve	BS	0.72	0.11	0.153	0.57	18.7	0.67				3.05E+08	7.51E+09	24.623
		Y	711.2	19.05	406.4	12.7	68.8	IPB-ve	BS	-0.78			0.57	18.7	0.67				3.05E+08	7.51E+09	24.623
		Y	711.2	19.05	406.4	12.7	68.8	IPB-ve	CC	3.06	0.6	0.196	0.57	18.7	0.67				3.05E+08	7.51E+09	24.623
		Y	711.2	19.05	406.4	12.7	68.8	IPB-ve	CC	-1.96	0.09	-0.046	0.57	18.7	0.67				3.05E+08	7.51E+09	24.623
		Y	711.2	19.05	406.4	12.7	68.8	IPB-ve	BC	1.99	0.28	0.141	0.57	18.7	0.67				3.05E+08	7.51E+09	24.623
		Y	711.2	19.05	406.4	12.7	68.8	IPB-ve	BC	-1.78	-0.09	0.051	0.57	18.7	0.67				3.05E+08	7.51E+09	24.623
		Y	711.2	19.05	406.4	12.7	68.8	OPB	CS	-7.27	-0.5	0.069	0.57	18.7	0.67				3.05E+08	1.00E+10	32.787
		Y	711.2	19.05	406.4	12.7	68.8	OPB	CS	7.35	0.48	0.065	0.57	18.7	0.67				3.05E+08	1.00E+10	32.787
		Y	711.2	19.05	406.4	12.7	68.8	OPB	BS	-5.25	-0.25	0.048	0.57	18.7	0.67				3.05E+08	1.00E+10	32.787
		Y	711.2	19.05	406.4	12.7	68.8	OPB	BS	5.21	0.23	0.044	0.57	18.7	0.67				3.05E+08	1.00E+10	32.787
		Y	711.2	19.05	406.4	12.7	68.8	OPB	CC	0.35	0.12	0.343	0.57	18.7	0.67				3.05E+08	1.00E+10	32.787
		Y	711.2	19.05	406.4	12.7	68.8	OPB	CC	0.22	0.07	0.318	0.57	18.7	0.67				3.05E+08	1.00E+10	32.787
		Y	711.2	19.05	406.4	12.7	68.8	OPB	BC	0.44	0.04	0.091	0.57	18.7	0.67				3.05E+08	1.00E+10	32.787
		Y	711.2	19.05	406.4	12.7	68.8	OPB	BC	0.39	0.1	0.256	0.57	18.7	0.67				3.05E+08	1.00E+10	32.787

NOTES

1. LOCATION

CS = Chord Saddle
CC = Chord Crown

BS = Brace Saddle
BC = Brace Crown

Database 4.7.3: SCFs for joints with stressed grouted clamps (..continued..)

REF No.	TEST ID	JOINT TYPE	D (mm)	T (mm)	d (mm)	t (mm)	Theta (degrees)	LOAD TYPE	LOCATION	MEASURED SCF		SCF RATIO	BETA	GAMMA	TAU	ROSS-SECTIONAL ARE		AREA RATIO	2nd MOMENT OF AREA		2nd MOMENT OF AREA RATIO
										Original	Clamped	Clamped/Original				BRACE (mm u2)	ACE+CLA (mm u2)		BRACE (mm u4)	ACE+CLA (mm u4)	
	CHORD	Y	711.2	19.05	406.4	12.7	68.8	Tens	CS	6.67	3.68	0.552	0.57	18.7	0.67	1.57E+04	7.57E+04	4.822			
	LOAD	Y	711.2	19.05	406.4	12.7	68.8	Tens	CS	6.44	3.75	0.582	0.57	18.7	0.67	1.57E+04	7.57E+04	4.822			
	-	Y	711.2	19.05	406.4	12.7	68.8	Tens	BS	4.43	2.24	0.506	0.57	18.7	0.67	1.57E+04	7.57E+04	4.822			
	7030KN	Y	711.2	19.05	406.4	12.7	68.8	Tens	BS	5.06	1.83	0.362	0.57	18.7	0.67	1.57E+04	7.57E+04	4.822			
		Y	711.2	19.05	406.4	12.7	68.8	Tens	CC	3.14	1.84	0.586	0.57	18.7	0.67	1.57E+04	7.57E+04	4.822			
		Y	711.2	19.05	406.4	12.7	68.8	Tens	CC	2.32	0.47	0.203	0.57	18.7	0.67	1.57E+04	7.57E+04	4.822			
		Y	711.2	19.05	406.4	12.7	68.8	Tens	BC	2.28	1.02	0.447	0.57	18.7	0.67	1.57E+04	7.57E+04	4.822			
		Y	711.2	19.05	406.4	12.7	68.8	Tens	BC	0.59	0.24	0.407	0.57	18.7	0.67	1.57E+04	7.57E+04	4.822			
		Y	711.2	19.05	406.4	12.7	68.8	Comp	CS	-6.16	-3.06	0.497	0.57	18.7	0.67	1.57E+04	7.57E+04	4.822			
		Y	711.2	19.05	406.4	12.7	68.8	Comp	CS	-7.34	-2.82	0.384	0.57	18.7	0.67	1.57E+04	7.57E+04	4.822			
		Y	711.2	19.05	406.4	12.7	68.8	Comp	BS	-4.83	-2.07	0.429	0.57	18.7	0.67	1.57E+04	7.57E+04	4.822			
		Y	711.2	19.05	406.4	12.7	68.8	Comp	BS	-5.43	-1.5	0.276	0.57	18.7	0.67	1.57E+04	7.57E+04	4.822			
		Y	711.2	19.05	406.4	12.7	68.8	Comp	CC	-1.77	-1.61	0.910	0.57	18.7	0.67	1.57E+04	7.57E+04	4.822			
		Y	711.2	19.05	406.4	12.7	68.8	Comp	CC	-2.9	-0.2	0.069	0.57	18.7	0.67	1.57E+04	7.57E+04	4.822			
		Y	711.2	19.05	406.4	12.7	68.8	Comp	BC	-1.65	-1.06	0.642	0.57	18.7	0.67	1.57E+04	7.57E+04	4.822			
		Y	711.2	19.05	406.4	12.7	68.8	Comp	BC		0		0.57	18.7	0.67	1.57E+04	7.57E+04	4.822			
		Y	711.2	19.05	406.4	12.7	68.8	IPB+ve	CS	0.68	-0.18	-0.265	0.57	18.7	0.67				3.05E+08	7.51E+09	24.623
		Y	711.2	19.05	406.4	12.7	68.8	IPB+ve	CS	0.59			0.57	18.7	0.67				3.05E+08	7.51E+09	24.623
		Y	711.2	19.05	406.4	12.7	68.8	IPB+ve	BS	0.78	-0.1	-0.128	0.57	18.7	0.67				3.05E+08	7.51E+09	24.623
		Y	711.2	19.05	406.4	12.7	68.8	IPB+ve	BS	0.8	-0.5	-0.625	0.57	18.7	0.67				3.05E+08	7.51E+09	24.623
4.17		Y	711.2	19.05	406.4	12.7	68.8	IPB+ve	CC	-3.07	-0.66	0.215	0.57	18.7	0.67				3.05E+08	7.51E+09	24.623
		Y	711.2	19.05	406.4	12.7	68.8	IPB+ve	CC	3.64	-0.13	-0.036	0.57	18.7	0.67				3.05E+08	7.51E+09	24.623
		Y	711.2	19.05	406.4	12.7	68.8	IPB+ve	BC	-2.08	-0.31	0.149	0.57	18.7	0.67				3.05E+08	7.51E+09	24.623
		Y	711.2	19.05	406.4	12.7	68.8	IPB+ve	BC	1.77	0.04	0.023	0.57	18.7	0.67				3.05E+08	7.51E+09	24.623
		Y	711.2	19.05	406.4	12.7	68.8	IPB-ve	CS	-0.52	0.14	-0.269	0.57	18.7	0.67				3.05E+08	7.51E+09	24.623
		Y	711.2	19.05	406.4	12.7	68.8	IPB-ve	CS	-0.57	0.18	-0.316	0.57	18.7	0.67				3.05E+08	7.51E+09	24.623
		Y	711.2	19.05	406.4	12.7	68.8	IPB-ve	BS	-0.69	0.08	-0.116	0.57	18.7	0.67				3.05E+08	7.51E+09	24.623
		Y	711.2	19.05	406.4	12.7	68.8	IPB-ve	BS	-0.81	0.015	-0.019	0.57	18.7	0.67				3.05E+08	7.51E+09	24.623
		Y	711.2	19.05	406.4	12.7	68.8	IPB-ve	CC	3.41	0.59	0.173	0.57	18.7	0.67				3.05E+08	7.51E+09	24.623
		Y	711.2	19.05	406.4	12.7	68.8	IPB-ve	CC	-1.96	0.07	-0.036	0.57	18.7	0.67				3.05E+08	7.51E+09	24.623
		Y	711.2	19.05	406.4	12.7	68.8	IPB-ve	BC	1.96	0.27	0.138	0.57	18.7	0.67				3.05E+08	7.51E+09	24.623
		Y	711.2	19.05	406.4	12.7	68.8	IPB-ve	BC	-1.78	-0.08	0.045	0.57	18.7	0.67				3.05E+08	7.51E+09	24.623
		Y	711.2	19.05	406.4	12.7	68.8	OPB	CS	-6.6	-0.5	0.076	0.57	18.7	0.67				3.05E+08	1.00E+10	32.787
		Y	711.2	19.05	406.4	12.7	68.8	OPB	CS	6.67	0.48	0.072	0.57	18.7	0.67				3.05E+08	1.00E+10	32.787
		Y	711.2	19.05	406.4	12.7	68.8	OPB	BS	-5.25	-0.25	0.048	0.57	18.7	0.67				3.05E+08	1.00E+10	32.787
		Y	711.2	19.05	406.4	12.7	68.8	OPB	BS	5.32			0.57	18.7	0.67				3.05E+08	1.00E+10	32.787
		Y	711.2	19.05	406.4	12.7	68.8	OPB	CC	0.48	-0.12	-0.250	0.57	18.7	0.67				3.05E+08	1.00E+10	32.787
		Y	711.2	19.05	406.4	12.7	68.8	OPB	CC	0.17	-0.07	-0.412	0.57	18.7	0.67				3.05E+08	1.00E+10	32.787
		Y	711.2	19.05	406.4	12.7	68.8	OPB	BC	0.5	-0.01	-0.020	0.57	18.7	0.67				3.05E+08	1.00E+10	32.787
		Y	711.2	19.05	406.4	12.7	68.8	OPB	BC	0.28			0.57	18.7	0.67				3.05E+08	1.00E+10	32.787

NOTES

1. LOCATION

CS = Chord Saddle
CC = Chord Crown

BS = Brace Saddle
BS = Brace Crown

Database 4.7.3: SCFs for joints with stressed grouted clamps (..continued)

MSL

Ref No.	TEST No.	JOINT TYPE	D (mm)	T (mm)	d (mm)	t (mm)	Theta (degrees)	GAP (mm)	LOAD TYPE	LOCATION	SCF MEASURED	SCF PREDICTED	SCF JOINT PREDICTED / SCF JOINT MEASURED	SECTION PROPERTY RATIO AREA	SECTION PROPERTY RATIO MODULUS (IPB)	SECTION PROPERTY RATIO MODULUS (OPB)	SCF SLEEVE FLANGE(IPB) RELATIVE TO BRACE NOMINAL STRESS	Comments
4.1	2Y	Y	558.00	12.70	324.00	10.00	45.00	0.00	TEN	CS	3.30	9.28	2.812	2.864				
	2Y	Y	558.00	12.70	324.00	10.00	45.00	0.00	TEN	BS	3.07	4.85	1.580	2.864				
	2T	T	558.00	12.70	324.00	10.00	90.00	0.00	TEN	CS	6.93	16.18	2.335	2.864				
	2T	T	558.00	12.70	324.00	10.00	90.00	0.00	TEN	BS	4.70	10.37	2.207	2.864				
	2K (NON)	K	457.00	10.00	273.00	6.00	45.00	110.00	TEN	CS	1.26	8.17	6.480	3.374				
	2K (NON)	K	457.00	10.00	273.00	6.00	45.00	110.00	TEN	BS	1.78	5.73	3.219	3.374				
	2K	K	457.00	10.00	273.00	6.00	45.00	-255.00	TEN	CS	1.57	8.17	5.201	3.374				
	2K	K	457.00	10.00	273.00	6.00	45.00	-255.00	TEN	BS	2.69	5.73	2.130	3.374				
	2A	T	558.00	12.70	229.00	6.30	90.00	0.00	TEN	CS	3.35	9.75	2.910	3.398				
	2A	T	558.00	12.70	229.00	6.30	90.00	0.00	TEN	BS	5.28	8.56	1.621	3.398				
	U	Y	558.00	12.70	229.00	6.30	90.00	0.00	TEN	CS	2.71	9.75	3.597	3.398				
	U	Y	558.00	12.70	229.00	6.30	90.00	0.00	TEN	BS	2.35	8.56	3.642	3.398				
	7X	X	361.00	8.00	355.00	5.00	72.00	0.00	TEN	CS	1.40	5.84	4.169	2.691				
	7X	X	361.00	8.00	355.00	5.00	72.00	0.00	TEN	BS	2.20	4.09	1.859	2.691				
	CONOCO	X	361.00	8.00	355.00	5.00	72.00	0.00	TEN	CS	1.11	5.84	5.259	2.691				
	CONOCO	X	361.00	8.00	355.00	5.00	72.00	0.00	TEN	BS	1.88	4.09	2.175	2.691				
	2Y	Y	558.00	12.70	324.00	10.00	45.00	0.00	COMP	CS	2.62	9.28	3.542	2.864				
	2Y	Y	558.00	12.70	324.00	10.00	45.00	0.00	COMP	BS	2.62	4.85	1.851	2.864				
	2T	T	558.00	12.70	324.00	10.00	90.00	0.00	COMP	CS	4.70	16.18	3.443	2.864				
	2T	T	558.00	12.70	324.00	10.00	90.00	0.00	COMP	BS	3.39	10.37	3.059	2.864				
	2K (NON)	K	457.00	10.00	273.00	6.00	45.00	110.00	COMP	CS	2.18	8.17	3.745	3.374				
	2K (NON)	K	457.00	10.00	273.00	6.00	45.00	110.00	COMP	BS	1.78	5.73	3.219	3.374				
	2K	K	457.00	10.00	273.00	6.00	45.00	-255.00	COMP	CS	1.59	8.17	5.135	3.374				
	2K	K	457.00	10.00	273.00	6.00	45.00	-255.00	COMP	BS	2.70	5.73	2.122	3.374				
	2A	T	558.00	12.70	229.00	6.30	90.00	0.00	COMP	CS	2.64	9.75	3.693	3.398				
	2A	T	558.00	12.70	229.00	6.30	90.00	0.00	COMP	BS	4.16	8.56	2.057	3.398				
	U	Y	558.00	12.70	229.00	6.30	90.00	0.00	COMP	CS	2.47	9.75	3.947	3.398				
	U	Y	558.00	12.70	229.00	6.30	90.00	0.00	COMP	BS	1.50	8.56	5.706	3.398				
	7X	X	361.00	8.00	355.00	5.00	72.00	0.00	COMP	CS	1.40	5.84	4.169	2.691				
	7X	X	361.00	8.00	355.00	5.00	72.00	0.00	COMP	BS	2.00	4.09	2.045	2.691				
	CONOCO	X	361.00	8.00	355.00	5.00	72.00	0.00	COMP	CS	1.36	5.84	4.292	2.691				
	CONOCO	X	361.00	8.00	355.00	5.00	72.00	0.00	COMP	BS	1.93	4.09	2.119	2.691				
	2K (NON)	K	457.00	10.00	273.00	6.00	45.00	110.00	Bal Axial	CS	1.24	4.35	3.507	3.374				
	2K (NON)	K	457.00	10.00	273.00	6.00	45.00	110.00	Bal Axial	BS	1.40	3.16	2.258	3.374				
	2K	K	457.00	10.00	273.00	6.00	45.00	-255.00	Bal Axial	CS	0.80	2.50	3.124	3.374				
	2K	K	457.00	10.00	273.00	6.00	45.00	-255.00	Bal Axial	BS	1.45	3.26	2.248	3.374				

NOTES

1. LOCATION
   CS = Chord Saddle
   BS = Brace Saddle
2. Predicted SCFs based on Efthymiou Parametric Equations.
3. 4 No. 75x15 flanges at all splits.

Database 4.7.4: SCFs for sleeves in unstressed grouted clamps (continued..)

MSL

Ref No.	TEST No.	JOINT TYPE	D (mm)	T (mm)	d (mm)	t (mm)	Theta (degrees)	GAP (mm)	LOAD TYPE	LOCATION	SCF MEASURED	SCF PREDICTED	SCF JOINT PREDICTED / SCF JOINT MEASURED	SECTION PROPERTY RATIO AREA	SECTION PROPERTY RATIO MODULUS (IPB)	SECTION PROPERTY RATIO MODULUS (OPB)	SCF SLEEVE FLANGE(IPB) RELATIVE TO BRACE NOMINAL STRESS	Comments
	2Y	Y	558.00	12.70	324.00	10.00	45.00	0.00	IPB	CC	0.57	3.50	6.135		4.649		0.76	
	2Y	Y	558.00	12.70	324.00	10.00	45.00	0.00	IPB	BC	0.60	3.37	5.612		4.649			
	2T	T	558.00	12.70	324.00	10.00	90.00	0.00	IPB	CC	0.52	4.46	8.571		4.649		0.95	
	2T	T	558.00	12.70	324.00	10.00	90.00	0.00	IPB	BC	0.42	3.50	8.333		4.649			
	2A	T	558.00	12.70	229.00	6.30	90.00	0.00	IPB	CC	0.22	3.04	13.827		7.748		0.51	
	2A	T	558.00	12.70	229.00	6.30	90.00	0.00	IPB	BC	0.35	3.20	9.149		7.748			
	U	Y	558.00	12.70	229.00	6.30	90.00	0.00	IPB	CC		3.04			7.748		0.58	
	U	Y	558.00	12.70	229.00	6.30	90.00	0.00	IPB	BC		3.20			7.748			
	CONOCO	X	361.00	8.00	355.00	5.00	72.00	0.00	IPB	CC	0.43	2.59	6.028		4.693		1.0	
	CONOCO	X	361.00	8.00	355.00	5.00	72.00	0.00	IPB	BC	0.96	2.48	2.583		4.693			
	2K (NON)	K	457.00	10.00	273.00	6.00	45.00	110.00	Bal IPB	CC	0.25				7.252		0.42	
	2K (NON)	K	457.00	10.00	273.00	6.00	45.00	110.00	Bal IPB	BC	0.20				7.252			
	2K	K	457.00	10.00	273.00	6.00	45.00	-255.00	Bal IPB	CC	0.09				7.252		0.45	
	2K	K	457.00	10.00	273.00	6.00	45.00	-255.00	Bal IPB	BC	0.21				7.252			
	2K (NON)	K	457.00	10.00	273.00	6.00	45.00	110.00	1 B IPB	CC	0.17	2.82	16.600		7.252		0.29	
4.1	2K (NON)	K	457.00	10.00	273.00	6.00	45.00	110.00	1 B IPB	BC	0.20	3.11	15.560		7.252			
	2K	K	457.00	10.00	273.00	6.00	45.00	-255.00	1 B IPB	CC	0.11	2.82	25.655		7.252		0.24	
	2K	K	457.00	10.00	273.00	6.00	45.00	-255.00	1 B IPB	BC	0.40	3.11	7.780		7.252			
	2Y	Y	558.00	12.70	324.00	10.00	45.00	0.00	OPB	CS	1.45	7.67	5.286			2.376		
	2Y	Y	558.00	12.70	324.00	10.00	45.00	0.00	OPB	BS	1.79	5.43	3.032			2.376		
	2T	T	558.00	12.70	324.00	10.00	90.00	0.00	OPB	CS	3.44	13.35	3.880			2.376		
	2T	T	558.00	12.70	324.00	10.00	90.00	0.00	OPB	BS	2.85	9.45	3.315			2.376		
	2A	T	558.00	12.70	229.00	6.30	90.00	0.00	OPB	CS	2.03	6.85	3.372			2.279		
	2A	T	558.00	12.70	229.00	6.30	90.00	0.00	OPB	BS	4.23	6.85	1.619			2.279		
	2K (NON)	K	457.00	10.00	273.00	6.00	45.00	110.00	Bal OPB	CS	1.56					2.394		
	2K (NON)	K	457.00	10.00	273.00	6.00	45.00	110.00	Bal OPB	BS	2.46					2.394		
	2K	K	457.00	10.00	273.00	6.00	45.00	-255.00	Bal OPB	CS	1.67					2.394		
	2K	K	457.00	10.00	273.00	6.00	45.00	-255.00	Bal OPB	BS	2.89					2.394		
	2K (NON)	K	457.00	10.00	273.00	6.00	45.00	110.00	1 B OPB	CS	1.08	6.20	5.736			2.394		
	2K (NON)	K	457.00	10.00	273.00	6.00	45.00	110.00	1 B OPB	BS	1.11	5.02	4.524			2.394		
	2K	K	457.00	10.00	273.00	6.00	45.00	-255.00	1 B OPB	CS	0.73	5.37	7.352			2.394		
	2K	K	457.00	10.00	273.00	6.00	45.00	-255.00	1 B OPB	BS	1.56	4.35	2.788			2.394		

NOTES

1. LOCATION

CS = Chord Saddle BS = Brace Saddle
CC = Chord Crown BS = Brace Crown

2. Predicted SCFs based on Efthymiou Parametric Equations.

3. 4 No. 75x15 flanges at all splits.

Database 4.7.4: SCFs for sleeves in unstressed grouted clamps (..continued)

Reference No..	Specimen Type	Steel type	No of nuts	Forming method	Thread series	Tightening (Nm)	Tightening Turn	Size (mm)	Fy (N/mm^2)	Loading type	P (kN)	M (kN-m)	Stress Range (N/mm^2) ASME(1)	Stress Range (N/mm^2) ISO(2)	Cycles to failure	Failure mode	Comments
4.29	A	A193 Gr B7	1	cut	8N			35	794	Tension	54.91		69.00	68.49	2.00E+06	first cut thread	
	A	A193 Gr B7	1	cut	8N			35	794	Tension	54.91		69.00	68.49	1.20E+07	first cut thread	
	A	A193 Gr B7	1	cut	8N			35	794	Tension	109.82		138.00	136.97	1.80E+05	first cut thread	
	A	A193 Gr B7	1	cut	8N			35	794	Tension	109.82		138.00	136.97	1.90E+05	first cut thread	
	A	A193 Gr B7	1	cut	8N			35	794	Tension	164.73		207.00	205.46	8.00E+04	first cut thread	
	A	A193 Gr B7	1	cut	8N			35	794	Tension	164.73		207.00	205.46	8.50E+04	first cut thread	
	A	A193 Gr B7	1	cut	8N			35	794	Tension	49.34		62.00	61.54	2.10E+06	first cut thread	
	A	A193 Gr B7	1	cut	8N			35	794	Tension	54.91		69.00	68.49	1.30E+07	first cut thread	
	A	A193 Gr B7	1	cut	8N			35	794	Tension	54.91		69.00	68.49	1.50E+07	first cut thread	
	A	A193 Gr B7	1	cut	8N			35	794	Tension	54.91		69.00	68.49	1.80E+07	first cut thread	
	A	A193 Gr B7	1	cut	8N			35	794	Tension	115.39		145.00	143.92	1.70E+05	first cut thread	
	A	A193 Gr B7	1	cut	8N			35	794	Tension	120.80		151.80	150.67	1.35E+05	first cut thread	
	A	A193 Gr B7	1	cut	8N			35	794	Tension	120.96		152.00	150.87	1.80E+05	first cut thread	
	A	A193 Gr B7	1	cut	8N			35	794	Tension	164.73		207.00	205.46	5.50E+04	first cut thread	
	A	A193 Gr B7	1	cut	8N			35	794	Tension	175.71		220.80	219.16	8.00E+04	first cut thread	
	B	A193 Gr B7	1	cut	8N			44	759	Tension	92.69		69.00	68.60	1.40E+06	first cut thread	
	B	A193 Gr B7	1	cut	8N			44	759	Tension	92.69		69.00	68.60	1.50E+06	first cut thread	
	B	A193 Gr B7	1	cut	8N			44	759	Tension	92.69		69.00	68.60	2.20E+06	first cut thread	
	B	A193 Gr B7	1	cut	8N			44	759	Tension	185.38		138.00	137.21	1.50E+05	first cut thread	
	B	A193 Gr B7	1	cut	8N			44	759	Tension	203.92		151.80	150.93	1.90E+05	first cut thread	
	B	A193 Gr B7	1	cut	8N			44	759	Tension	254.89		189.75	188.66	6.25E+04	first cut thread	
	B	A193 Gr B7	1	cut	8N			44	759	Tension	315.14		234.60	233.25	6.25E+04	first cut thread	
	B	A193 Gr B7	1	cut	8N			44	759	Tension	92.69		69.00	68.60	1.20E+06	first cut thread	
	B	A193 Gr B7	1	cut	8N			44	759	Tension	92.69		69.00	68.60	1.80E+06	first cut thread	
	B	A193 Gr B7	1	cut	8N			44	759	Tension	185.38		138.00	137.21	2.00E+05	first cut thread	
	B	A193 Gr B7	1	cut	8N			44	759	Tension	185.38		138.00	137.21	3.00E+05	first cut thread	
	B	A193 Gr B7	1	cut	8N			44	759	Tension	278.07		207.00	205.81	7.00E+04	first cut thread	
	B	A193 Gr B7	2	cut	8N	270		44	759	Tension	185.38		138.00	137.21	9.00E+05	first cut thread	
	B	A193 Gr B7	2	cut	8N	270		44	759	Tension	185.38		138.00	137.21	3.10E+05	first cut thread	
	B	A193 Gr B7	2	cut	8N	270		44	759	Tension	278.07		207.00	205.81	4.80E+04	first cut thread	
	B	A193 Gr B7	2	cut	8N	270		44	759	Tension	278.07		207.00	205.81	5.80E+04	first cut thread	
	B	A193 Gr B7	2	cut	8N	270		44	759	Tension	278.07		207.00	205.81	7.20E+04	first cut thread	
	B	A193 Gr B7	2	cut	8N		1/6	44	759	Tension	185.38		138.00	137.21	2.20E+05	first cut thread	
	B	A193 Gr B7	2	cut	8N		1/3	44	759	Tension	92.69		69.00	68.60	2.20E+06	along thread	Specimens that failed in the first engaged thread of the interior nut are included in the along the thread failure classification.
	B	A193 Gr B7	2	cut	8N		1/3	44	759	Tension	92.69		69.00	68.60	4.00E+06	along thread	
	B	A193 Gr B7	2	cut	8N		1/3	44	759	Tension	92.69		69.00	68.60	5.00E+06	along thread	
	B	A193 Gr B7	2	cut	8N		1/3	44	759	Tension	101.96		75.90	75.46	4.00E+06	along thread	
	B	A193 Gr B7	2	cut	8N		1/3	44	759	Tension	185.38		138.00	137.21	8.00E+05	along thread	
	B	A193 Gr B7	2	cut	8N		1/3	44	759	Tension	185.38		138.00	137.21	9.00E+05	along thread	
	B	A193 Gr B7	2	cut	8N		1/3	44	759	Tension	278.07		207.00	205.81	1.00E+05	along thread	
	B	A193 Gr B7	2	cut	8N		1/3	44	759	Tension	278.07		207.00	205.81	1.25E+05	along thread	
	BB	A193 Gr B7	2	cut	8N			35	794	Bending		0.44	138.00	136.47	4.50E+05	first cut thread	Short threaded bolts. Thread termination lies within the zone of high bending moment
	BB	A193 Gr B7	2	cut	8N			35	794	Bending		0.44	138.00	136.47	6.00E+05	first cut thread	
	BB	A193 Gr B7	2	cut	8N			35	794	Bending		0.66	207.00	204.70	2.10E+05	first cut thread	
	BB	A193 Gr B7	2	cut	8N			35	794	Bending		0.66	207.00	204.70	2.50E+05	first cut thread	
	BB	A193 Gr B7	2	cut	8N			35	794	Bending		0.87	276.00	272.94	7.00E+04	first cut thread	
	BB	A193 Gr B7	2	cut	8N			35	794	Bending		0.87	276.00	272.94	8.00E+04	first cut thread	
	BG	A193 Gr B7	2	cut	8N			35	794	Bending		0.55	172.50	170.59	3.50E+05	along thread	Extended thead bolts. Thread continues along the bolt length terminating in the zone of zero bending moment
	BG	A193 Gr B7	2	cut	8N			35	794	Bending		0.59	186.30	184.23	2.50E+05	along thread	
	BG	A193 Gr B7	2	cut	8N			35	794	Bending		0.72	227.70	225.17	4.00E+05	along thread	
	BG	A193 Gr B7	2	cut	8N			35	794	Bending		0.79	248.40	245.64	4.10E+05	along thread	
	BG	A193 Gr B7	2	cut	8N			35	794	Bending		0.87	276.00	272.94	1.00E+05	along thread	
	BG	A193 Gr B7	2	cut	8N			35	794	Bending		0.87	276.00	272.94	2.00E+05	along thread	

Notes

1) Tensile Stress Area = PI/4*(D-0.9743/n)^2
where: D = basic major diameter; n = number of threads per inch; units = inches

2) Tensile Stress Area = PI/4*[(d2+d3)/2]^2
where: d2 = basic pitch diameter; d3 = minor diameter = d1-(H/6); d1 = basic minor diameter
H = height of the fundamental triangle for the thread; units = mm

Database 4.7.5: Fatigue behaviour of B7 studbolts

Ref No.	Spec No	D ds (mm)	T ds (mm)	D (mm)	T (mm)	L (mm)	D ds /T ds	D/T	D dg /T dg	L/D	K	h (mm)	s (mm)	h/s	sigma dcu (N/mm u2) START	END	Applied load (kN) min	max	range	No cycles	FREQ (Hz)	Comments
	3A1	407.00	13.70	353.00	13.70	762.48	29.71	25.77	28.54	2.16	0.0200	0.00	-	-	26.8	31.0	22.58	752.56	729.99	2000000	1.0	Permanent 26.80
	3A2	407.00	13.20	355.00	13.50	710.00	30.83	26.30	29.73	2.00	0.0194	0.00	-	-	27.1	27.1	20.43	653.74	633.31	10	1.0	Relative 29.50
	3A3	407.00	13.50	355.00	13.60	731.30	30.15	26.10	30.40	2.06	0.0196	0.00	-	-	26.9	36.7	11.91	595.38	583.48	2000000	1.0	Displ(mm): 2.90
	3B1	407.00	13.80	357.00	14.50	774.69	29.49	24.62	33.88	2.17	0.0201	0.00	-	-	32.6	34.2	17.90	724.88	706.99	2000000	1.0	4.33
4.26	3B2	407.00	13.60	356.00	13.70	761.84	29.93	25.99	31.92	2.14	0.0196	0.00	-	-	48.1	50.5	21.30	862.70	841.40	2000000	1.0	8.11
	3B3	406.00	13.00	357.00	14.00	756.84	31.23	25.50	33.04	2.12	0.0193	0.00	-	-	39.8	43.0	19.52	800.45	780.93	2000000	1.0	9.44
	3C1	407.00	12.90	356.00	13.90	694.20	31.55	25.61	30.25	1.95	0.0193	0.00	-	-	44.4	49.6	-369.41	738.82	1108.23	2000000	1.0	17.50
	3D1	452.00	6.90	358.00	14.50	748.22	65.51	24.69	10.93	2.09	0.0162	0.00	-	-	27.8	31.2	156.33	402.16	265.83	2000000	1.0	1.97
	3D2	453.00	6.90	357.00	13.70	749.70	65.65	26.06	10.69	2.10	0.0161	0.00	-	-	31.6	37.7	146.30	438.91	292.61	2000000	1.0	2.20
	3D3	452.00	7.00	358.00	14.30	744.64	64.57	25.03	10.95	2.08	0.0162	0.00	-	-	36.7	39.7	167.67	510.62	342.95	2000000	1.0	4.80
	V10	366.00	5.00	324.00	10.00	650.00	73.20	32.40	22.25	2.01	0.0120	0.00	-	-	73.6		-100.00	100.00	200.00	239000	0.1	No deterioration
																	-160.00	160.00	320.00	670	0.1	No deterioration
																	-200.00	200.00	400.00	6322	0.1	No deterioration
																	-240.00	240.00	480.00	660	0.1	No deterioration
4.27																	-280.00	280.00	560.00	34000	0.1	No deterioration
																	-350.00	350.00	700.00	6780	0.1	No deterioration
																	-400.00	400.00	800.00	16000	0.1	No deterioration
																	-450.00	450.00	900.00	55000	0.1	No deterioration
																73.6	0.00	714.55	714.55	-	-	Static load test
	6.1	368.00	10.00	324.00	10.00	325.00	36.80	32.40	29.00	1.00	0.0164	0.00	-	-	71.8	86.4	-245.00	245.00	490.00	3260000	3.0	No deterioration
																	-350.00	350.00	700.00	5680	3.0	Failure
	6.2	368.00	10.00	324.00	10.00	345.00	36.80	32.40	29.00	1.06	0.0164	0.00	-	-	77.3	85.7	-250.00	250.00	500.00	6920	3.0	Failure
	6.5	368.00	10.00	324.00	10.00	325.00	36.80	32.40	29.00	1.00	0.0164	0.00	-	-	80.4	101.2	-235.00	235.00	470.00	3570000	3.0	Steelwork failure
	6.6	368.00	10.00	324.00	10.00	325.00	36.80	32.40	29.00	1.00	0.0164	0.00	-	-	73.5		-180.00	180.00	360.00	2320000	3.0	No deterioration
4.1																	-225.00	225.00	450.00	1330000	3.0	No deterioration
																	-260.00	260.00	520.00	1570000	3.0	No deterioration
																	-300.00	300.00	600.00	247000	3.0	No deterioration
																	-350.00	350.00	700.00	346000	3.0	No deterioration
																	-400.00	400.00	800.00	162000	2.0	No deterioration
																85.8	-450.00	450.00	900.00	492000	1.0	Failure
	6.9	368.00	10.00	324.00	10.00	325.00	36.80	32.40	29.00	1.00	0.0164	0.00	-	-	65.2	67.4	-210.00	210.00	420.00	4430	1.0	Failure

Notes: Specimens 6.5, 6.6 and 6.9 were combined bending/tension (f db /f ds =1.0,1.0 and 2.0 respectively).

Database 4.7.6: Fatigue of plain pipe continuous sleeve unstressed grouted connections

Ref No.	Spec No.	D_s (mm)	T_s (mm)	D (mm)	T (mm)	L (mm)	D_s/T_s	D/T	D_g/T_g	L/D	K	h (mm)	s (mm)	h/s	sigma_ (N/mm²)		Applied load (kN)			Nr cycles	FREQ (Hz)	Comments	
															START	END	min	max	range			Displ(mm)	Cycles for delta=35 mm
	4A1	406.00	12.60	353.80	13.70	711.14	32.22	25.82	28.21	2.01	0.0192	2.20	340.00	0.0379	38.6	41.7	94.22	2002.15	1907.93	345699	1.0	70.00	345287
	4A2	406.00	12.80	357.00	13.30	714.00	31.72	26.84	32.51	2.00	0.0188	2.20	340.00	0.0379	43.1	42.8	24.74	2301.22	2276.47	3950	1.0	66.00	3
	4A3	406.50	12.80	354.00	13.90	761.10	31.76	25.47	28.52	2.15	0.0194	2.10	340.00	0.0362	44.1	44.1	50.28	2086.55	2036.27	56	1.0	46.00	23
	4B1	407.30	13.30	354.30	13.00	733.40	30.62	27.23	28.84	2.07	0.0192	2.25	340.00	0.0388	43.8	43.8	-634.69	1878.69	2513.38	79	1.0	35.00	79
	4B2	407.50	13.50	354.50	12.80	730.27	30.19	27.70	29.27	2.06	0.0192	2.25	340.00	0.0388	43.1	43.1	-633.40	1955.08	2588.48	63	1.0	35.00	63
	4B3	406.50	12.70	355.70	13.00	729.19	32.01	27.36	30.01	2.05	0.0187	2.20	340.00	0.0379	43.4	43.4	-682.02	2020.80	2702.82	96	1.0	35.00	96
	4C1	405.50	13.60	355.50	12.50	733.66	29.82	28.44	33.18	2.12	0.0188	2.25	340.00	0.0388	46.1	46.1	-1154.58	2282.31	3436.89	250	1.0	43.50	126
	4C2	406.00	13.50	356.00	13.80	704.88	30.07	25.80	32.96	1.98	0.0196	2.30	340.00	0.0397	36.9	40.0	-1124.10	2202.31	3326.41	250	1.0	77.00	1
4.26	4C3	406.80	13.40	354.80	14.10	702.50	30.36	25.16	30.16	1.98	0.0199	1.90	340.00	0.0328	42.3	42.3	-1222.01	2252.33	3474.34	96	1.0	66.00	17
	4D1	405.70	12.50	357.00	13.00	714.00	32.46	27.46	32.13	2.00	0.0184	2.25	340.00	0.0388	44.1	44.6	526.36	2080.36	1554.00	160000	1.0	2.00	160000
	4D2	408.80	12.30	358.20	13.20	730.73	33.24	27.14	29.55	2.04	0.0184	2.25	340.00	0.0388	44.6	45.4	723.27	2331.23	1605.96	160000	1.0	8.00	160000
	4D3	409.10	12.90	356.10	13.40	733.57	31.71	26.57	28.18	2.06	0.0191	2.25	340.00	0.0388	45.9	45.9	706.83	2329.92	1623.09	1050	1.0	35.00	1050
	4E1	408.70	13.40	356.30	14.00	730.42	30.50	25.45	29.84	2.05	0.0197	2.25	340.00	0.0388	45.8	45.8	-1845.95	1897.95	3743.91	20	1.0	17.50	-
	4E2	409.00	13.20	357.70	13.00	726.13	30.98	27.52	30.73	2.03	0.0189	2.25	340.00	0.0388	46.6	46.6	-1990.68	1990.68	3981.37	5	1.0	35.00	5
	4E3	408.80	13.20	356.90	12.40	728.08	30.97	28.78	29.99	2.04	0.0186	2.25	340.00	0.0388	46.9	46.9	-1708.61	1708.61	3417.22	12	1.0	35.00	12
	4F1	406.50	13.50	354.50	13.00	698.37	30.10	27.27	30.59	1.97	0.0192	2.37	340.00	0.0409	34.1	45.4	23.02	1611.53	1588.51	2000000	1.0	19.70	2000000
	4F2	406.50	13.00	356.00	13.30	694.20	31.27	26.77	31.06	1.95	0.0190	2.28	340.00	0.0393	45	44.5	24.38	1804.04	1779.66	2000000	1.0	17.00	2000000
	4F3	406.50	13.00	356.30	13.50	694.79	31.27	26.39	31.45	1.95	0.0191	2.45	340.00	0.0422	42.5	42.5	0.00	2118.01	2118.01	322	1.0	35.00	322
	4G1	406.80	13.00	354.30	13.20	712.14	31.29	26.84	28.74	2.01	0.0191	2.28	340.00	0.0393	35.6	39	22.75	1592.46	1569.71	2000000	1.0	25.00	2000000
	4G2	406.00	13.00	357.00	13.70	749.70	31.23	26.06	35.04	2.10	0.0191	2.10	340.00	0.0362	38.3	38.9	23.46	1947.10	1923.64	462	1.0	45.00	180
	4G3	406.30	12.90	354.80	14.10	816.04	31.51	25.16	29.40	2.30	0.0195	2.30	340.00	0.0397	39.1	39.1	33.48	2085.87	2052.38	300000	1.0	43.00	373
	V1	366.00	5.00	324.00	10.00	324.00	73.20	32.40	22.25	1.00	0.0120	1.30	108.00	0.0224	75	75	-500.00	500.00	1000.00	80	0.1	Failure at 80	
	v2	366.00	5.00	324.00	10.00	324.00	73.20	32.40	22.25	1.00	0.0120	1.30	108.00	0.0224	76.5		-240.00	240.00	480.00	107	3.0	No deterioration	
																92.1	-500.00	500.00	1000.00	66	0.1	Failure at 66	
	v3	366.00	5.00	324.00	10.00	324.00	73.20	32.40	22.25	1.00	0.0120	1.30	108.00	0.0224	69.1		0.00	500.00	500.00	440000	3.0	No deterioration	
																	0.00	600.00	600.00	290000	3.0	No deterioration	
																	0.00	800.00	800.00	140000	3.0	No deterioration	
																	0.00	1005.00	1005.00	-	-	Static test failure	
																	0.00	800.00	800.00	2850000	3.0	No deterioration	
																73.1	0.00	950.00	950.00	-	-	Static test failure	
4.27	v4	366.00	5.00	324.00	10.00	400.00	73.20	32.40	22.25	1.23	0.0120	1.30	108.00	0.0224	77.6	77.6	-600.00	600.00	1200.00	3461	0.1	Failure at 3461	
	v5	366.00	5.00	324.00	10.00	390.00	73.20	32.40	22.25	1.20	0.0120	1.30	108.00	0.0224	79	79	-640.00	640.00	1280.00	140	0.1	Failure at 140	
	v6	366.00	5.00	324.00	10.00	366.00	73.20	32.40	22.25	1.13	0.0120	1.30	108.00	0.0224	95.2		-265.00	265.00	530.00	1600000	3.0	No deterioration	
																95.2	-530.00	530.00	1060.00	1424	0.1	Failed at 1424	
	V9	366.00	5.00	324.00	10.00	370.00	73.20	32.40	22.25	1.14	0.0120	1.30	108.00	0.0224	70.6	70.6	-475.00	475.00	950.00	3282	0.1	Failure at 3282	
	V11	366.00	5.00	324.00	10.00	324.00	73.20	32.40	22.25	1.00	0.0120	1.30	108.00	0.0224	72	77.3	-360.00	360.00	720.00	4240000	3.0	Failure	
	V12	366.00	5.00	324.00	10.00	330.00	73.20	32.40	22.25	1.02	0.0120	1.30	108.00	0.0224	75		-420.00	420.00	840.00	7010000	3.0	No deterioration	
																	-460.00	460.00	920.00	730000	3.0	No deterioration	
																	-500.00	500.00	1000.00	210000	3.0	No deterioration	
																	-550.00	550.00	1100.00	1978	3.0		
																75	0.00	1186.00	1186.00	-	-	Static test failure	
	V13	366.00	5.00	324.00	10.00	319.00	73.20	32.40	22.25	0.98	0.0120	1.30	108.00	0.0224	73.3		-440.00	440.00	880.00	1490000	3.0	No deterioration	
																	-490.00	490.00	980.00	200000	3.0	No deterioration	
																81.5	-710.00	710.00	1420.00	520	0.1	Failure at 520	
	6.3	368.00	10.00	324.00	10.00	323.00	36.80	32.40	29.00	1.00	0.0164	1.00	83.00	0.0172	77.1		-400.00	400.00	800.00	250000	2.0	No deterioration	
																	-450.00	450.00	900.00	650000	2.0	No deterioration	
																96.3	-500.00	500.00	1000.00	191000	3.0	Failure	
	6.4	368.00	10.00	324.00	10.00	323.00	36.80	32.40	29.00	1.00	0.0164	1.00	83.00	0.0172	73.9	92.1	-420.00	420.00	840.00	530000	3.0	Failure	
4.1	6.7	368.00	10.00	324.00	10.00	323.00	36.80	32.40	29.00	1.00	0.0164	1.00	83.00	0.0172	72	74.8	-545.00	545.00	1090.00	310	1.0	Failure	
	6.8	368.00	10.00	324.00	10.00	323.00	36.80	32.40	29.00	1.00	0.0164	1.00	83.00	0.0172	76	76	-506.00	506.00	1012.00	46440	1.5	Failure	
	6.11	368.00	10.00	324.00	10.00	323.00	36.80	32.40	29.00	1.00	0.0164	1.00	83.00	0.0172	79.8	79.8	-578.00	578.00	1156.00	210	1.0	Failure	
	6.12	368.00	10.00	324.00	10.00	323.00	36.80	32.40	29.00	1.00	0.0164	1.00	83.00	0.0172	58.3	58.3	-444.00	444.00	888.00	12850	1.0	Failure	
	6.15	368.00	10.00	324.00	10.00	323.00	36.80	32.40	29.00	1.00	0.0164	1.00	83.00	0.0172	48.8	-	-200.00	200.00	400.00	76210	0.1	No deterioration	

Note: Specimens 6.7, 6.8, 6.11, 6.12 and 6.15 were combined bending/tension (f_b/f_t=1.0,1.0,2.0,2.0 and respectively).

Database 4.7.7: Fatigue of weld beaded continuous sleeve unstressed grouted connections

Ref No.	Spec No.	D_s (mm)	T_s (mm)	D (mm)	T (mm)	L (mm)	D_s/T_s	D/T	D_s/T_g	L/D	K	h (mm)	s (mm)	h/s	sigma$_c$ (N/mm^2)		Applied load (kN)			No. cycles	FREQ (Hz)	Comments
															START	END	min	max	range			
4.28	K1	366.00	5.00	324.00	10.00	325.00	73.20	32.40	22.25	1.00	0.0120	1.50	125.00	0.012	84.7	84.7	-400.00	400.00	800.00	12	0.03	
	L1	366.00	5.00	324.00	10.00	325.00	73.20	32.40	22.25	1.00	0.0120	1.50	125.00	0.012	87.1	87.1	-300.00	300.00	600.00	19	0.03	
	M1	366.00	5.00	324.00	10.00	325.00	73.20	32.40	22.25	1.00	0.0120	1.50	125.00	0.012	87.7	87.7	-500.00	500.00	1000.00	19	0.03	
	K2	366.00	5.00	324.00	10.00	325.00	73.20	32.40	22.25	1.00	0.0120	1.50	125.00	0.012	94.5	94.5	-275.00	275.00	550.00	1035	0.1	
	L2	366.00	5.00	324.00	10.00	325.00	73.20	32.40	22.25	1.00	0.0120	1.50	125.00	0.012	93.6	93.6	-250.00	250.00	500.00	4205	0.1	
	M2	366.00	5.00	324.00	10.00	325.00	73.20	32.40	22.25	1.00	0.0120	1.50	125.00	0.012	93.2	93.2	-200.00	200.00	400.00	20000	0.3	No failure tested to 20000
	K3	366.00	5.00	324.00	10.00	325.00	73.20	32.40	22.25	1.00	0.0120	1.50	125.00	0.012	72.5	72.5	-252.00	252.00	504.00	12000	0.3	No failure tested to 12000
	L3	366.00	5.00	324.00	10.00	325.00	73.20	32.40	22.25	1.00	0.0120	1.50	125.00	0.012	75.5	75.5	-302.00	302.00	604.00	1026	0.1	
	M3	366.00	5.00	324.00	10.00	325.00	73.20	32.40	22.25	1.00	0.0120	1.50	125.00	0.012	76.4	76.4	-284.00	284.00	568.00	10000	0.3	No failure tested to 10000
	N	366.00	5.00	324.00	10.00	325.00	73.20	32.40	22.25	1.00	0.0120	1.50	125.00	0.012	93.2	93.2	0.00	980.00	980.00	-	-	Static load test

Database 4.7.8: Fatigue of weld beaded split sleeve unstressed grouted connections

IV 5 GROUTED MEMBERS

IV 5.1 INTRODUCTION

This Section is concerned with available data on fully and partially grout-filled damaged and undamaged tubular members. It also critically examines design methods for such members and presents the detailed derivation of the design approach presented in Sections III 5.2.2 and III 5.3.2 in Part III.

IV 5.2 AVAILABLE DATA

Test results and findings on fully and partially grouted damaged and undamaged tubulars have been reported by a number of workers[5.1 - 5.9]. The data have been entered into databases classified according to extent of fill (full or partial), whether damaged or not, and whether load primarily is compression or a combination of compression and flexure through eccentric application of the axial load. The databases are presented in Databases 5.2.1 to 5.2.6 at the end of this Chapter as follows:

Database 5.2.1: Fully grouted and damaged and subjected to axial compression

Database 5.2.2: Fully grouted and damaged and subjected to combined compression and moment

Database 5.2.3: Fully grouted and undamaged and subjected to axial compression

Database 5.2.4: Fully grouted and undamaged and subjected to combined compression and moment

Database 5.2.5: Partially grouted and damaged and subjected to axial compression

Database 5.2.6: Partially grouted and undamaged and subjected to axial compression.

In each database, the source reference is indicated together with the model identification. The model identifications are the same as in the source documents. Apart from the data extracted from Reference 5.8, all the data have been determined from publications of the originating workers. However, because some of the results are presented in non-dimensional form, cross-checks have been conducted where possible in the development of the database which are presented entirely in terms of basic variables. The report by Loh[5.8] has proved invaluable in this respect.

Under each database are notes which summarise the non-numeric facts relating to the models and their testing. These seek to provide a comprehensive summary of each series, including its weaknesses and its strengths. A review of these indicates that some tests are not necessarily conducted to required standards. Also, there is a lack of consistency with the execution of several facets of the test programmes. For example, grout strengths are determined from cylinders and cubes, the latter of varying sizes. The difference between cylinder and cube-based strengths is recognised when conducting comparisons with predictions. The original values, however, are listed in the databases. Tensile tests are conducted both statically and dynamically whereas the model tests are performed statically. Again,the difference between static and dynamic yield strengths is recognised in the assessment of data. Dynamic yield stresses are considered more appropriate as these are the values generally available to the designer. The yield strength of heat-treated models can vary significantly dependent on oven layout and on heating, holding and cooling durations. Thus yield stresses should be determined for every tubular tested. Strain gauges are not always used to check the correct alignment at initial loading or the continued correct pin-ended or eccentrically loaded conditions as loading approaches its ultimate value.

Reporting standards also vary. In some cases, few basic details are given, for example, diameter and thickness, and instead only derived values are presented, for example, D/t ratios. For reference, all basic details should be presented. General conclusions of the tests include:

- grouting needs to be complete to guarantee slip between the grout and the inside of the tubular will not occur.
- grout has little direct effect on strength when the tubular is of high slenderness although it contributes fully to stiffness.
- at low slenderness, grouting inhibits local buckling in undamaged tubulars. To date, no upper D/t ratio limit has been determined beyond which this benefit is negligible.
- at low slenderness, grouting inhibits the growth of dent damage and for low levels of damage may enable the original tubular strength to be realised or even exceeded.
- these benefits at low slenderness are realised even for partial grout filling provided failure can be determined to occur within the grouted region.

IV 5.3 DESIGN APPROACHES FOR FULLY GROUTED MEMBERS

A number of approaches are available for the design of intact composite beam-columns including infill cylindrical members. Two of particular relevance are those presented in BS5400: Part 5[5.10] and AISC-LRFD[5.11]. Loh has examined both of these and exploited them in the development of internal EPR technical guidance[5.8].

In the case of damaged grout-filled members, only one procedure is widely quoted[5.1]. This has been used relatively frequently by other workers seeking to conduct comparisons between their experimental findings and a theoretical prediction. This approach has some limitations, as discussed below, but it has played a unique role in the development of techniques for determining the strength of dent/bow damaged grout-filled tubulars.

The Parsanejad method[5.1] will be considered first followed by that of BS5400: Part 5 together with modifications proposed by Loh in order that it can be applied to damaged rather than intact grouted tubulars. The design method presented in Part III, Section III 5.2.2, is then developed in detail building on both Parsanejad's approach, BS5400: Part 5 and the API RP2A column curve[5.12].

IV 5.3.1 Parsanejad

Parsanejad begins by considering the geometry of the locally dented grout-filled tubular section. The properties in the case of the undamaged section are as shown in Figure 5.3.1. Parsanejad, however, uses the diameter to mid-thickness when determining the grout area A_g - see equation 8 of Reference 5.1 -rather than the grout diameter itself. (Loh[5.8] also observed this.)

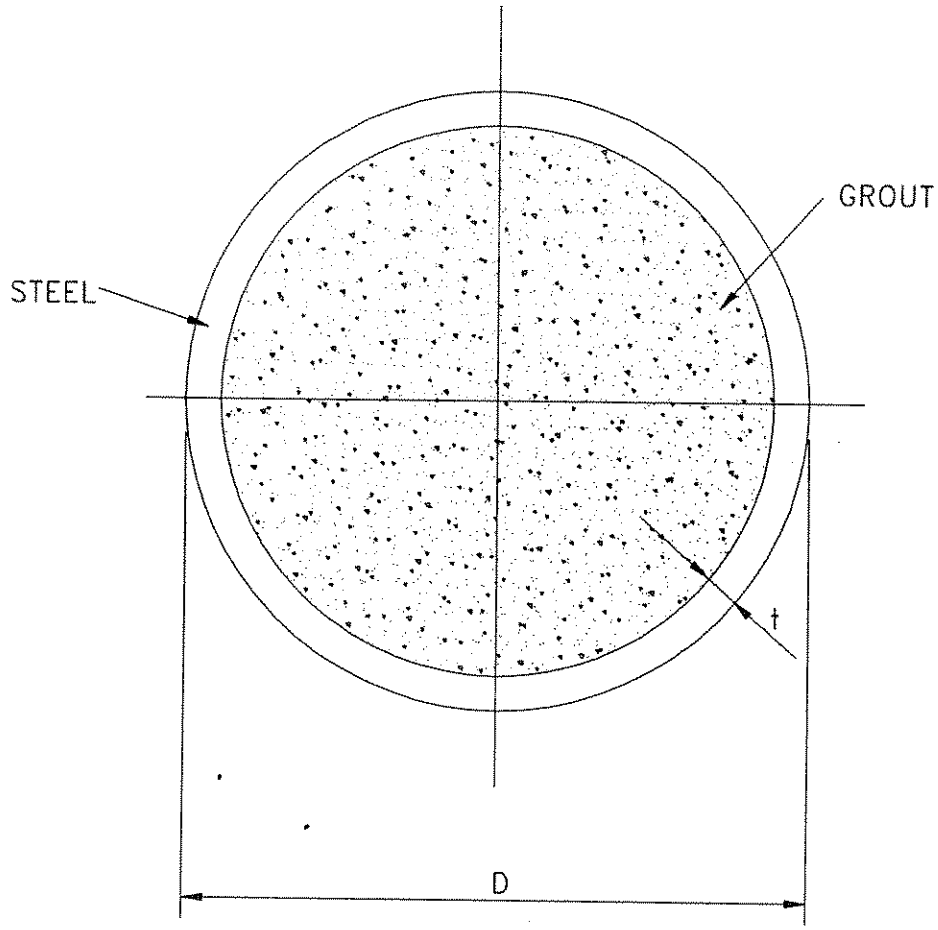
In the case of the damaged section, Figure 5.3.2, Loh has presented a similar set of equations but which differ slightly from those of Parsanejad: those of Loh are preferred here. The parameters sought are the transformed area A_{tr} and the second moment of area I_{tr} of the grouted damaged cross-section. The term 'transformed' is used to describe the result of the process of converting the grout cross-section expressed in grout material properties to an equivalent one for use with steel material properties. For example, in the case of cross-sectional area, this proceeds as follows. If P is the total axial load on a steel-grout section, it is equal to the sum of the individual loads in the steel and the grout. Assuming full bond between the grout and steel:

$$\begin{aligned} P &= P_s + P_g \\ &= f_s A_s + f_g A_g \end{aligned}$$

As the strain in both the steel and grout is the same:

$$\epsilon = f_s/E_s = f_g/E_g, \text{ then } f_g = \epsilon E_g.$$

$$\begin{aligned} \text{Substituting } P &= f_s A_s + \epsilon E_g A_g \\ &= f_s A_s + (f_s/E_s) E_g A_g \\ &= f_s A_s + f_s (E_g/E_s) A_g \\ &= f_s (A_s + A_g/m) \\ &= f_s A_{tr} \end{aligned}$$



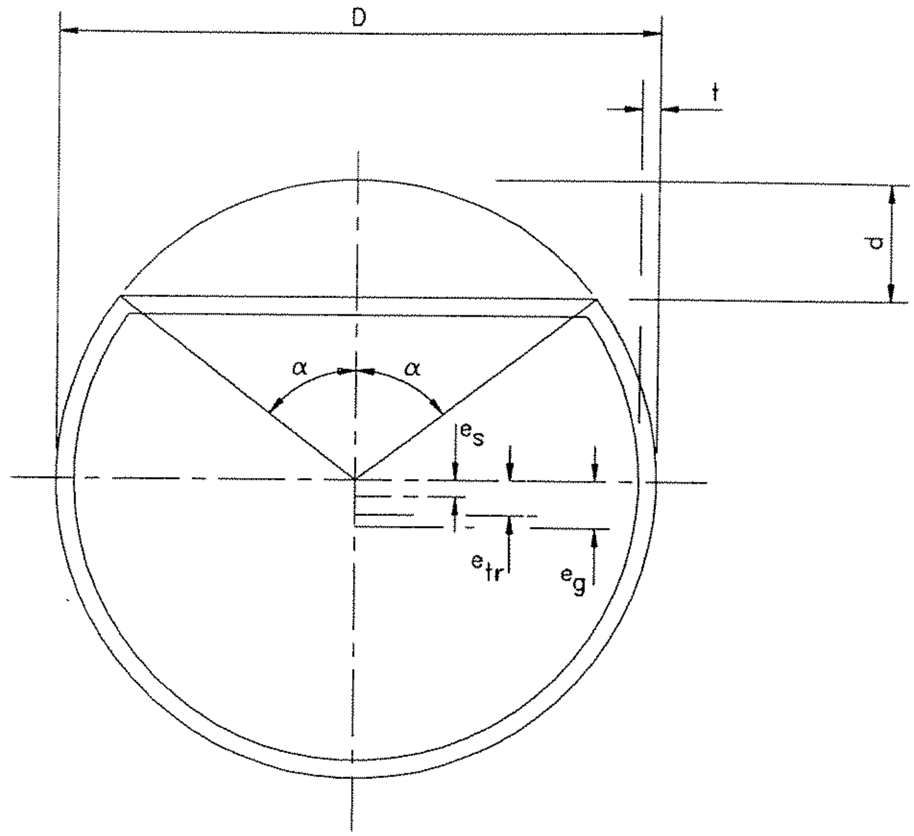
A_s = steel cross-sectional area = π (D-t)t

A_g = grout cross-sectional area = $\frac{\pi}{4}$ $(D-2t)^2$

I_s = steel second moment of area = $\frac{\pi}{64}\left[D^4 - (D-2t)^4\right]$

I_g = grout second moment of area = $\frac{\pi}{64}\left[(D-2t)^4\right]$

Figure 5.3.1: **Cross-sectional properties of a grout-filled undamaged member**



Fully Grouted

$m \quad = E_s / E_g$

$\alpha \quad = \cos^{-1}(1 - 2d/D)$

$D' \quad = D\text{-}t$

$A_s \quad = \pi D' t - D' t(\alpha - \sin\alpha)$

$A_g \quad = (D' - t)^2 (\pi - \alpha + ½ \sin 2\alpha) / 4$ *

$A_{tr} = A_s + A_g / m$

$e_s \quad = ½ D'^2 t \sin\alpha (1 - \cos\alpha) / A_s$

$e_g \quad = (D' - t)^3 \sin^3\alpha / 12A_g$ *

$e_{tr} = (A_s e_s + A_g e_g / m) / A_{tr}$ (not required if the approximate form of I_{tr} below is used)

Figure 5.3.2: Design methodology for fully and partially grouted damaged and undamaged tubular members

(continued...)

$$I_s = D'^3 t(\pi - \alpha - ½ \sin 2\alpha + 2 \sin\alpha \cos^2\alpha)/8 - A_s e_s^2$$

$$I_g = (D' - t)^4 (\pi - \alpha + ¼ \sin 4\alpha)/64 - A_g e_g^2 \quad *$$

$$I_{tr} = I_s + I_g/m + A_s(e_{tr} - e_s)^2 + A_g(e_{tr} - e_g)^2/m$$
$$\approx I_s + I_g/m$$

$$P_u = A_s f_y + 0.67 A_g f_{cu}$$

$$P_E = \pi^2 E_s I_{tr}/L^2$$

$$\lambda = \sqrt{(P_u/P_E)}$$

$$K_1 = 1 - \lambda^2/4 \quad \lambda < \sqrt{2}$$
$$= 1/\lambda^2 \quad \lambda \geq \sqrt{2}$$

$$P_{col} = K_1 P_u$$

$$M_p = ((D' + t)^3 - (D' - t)^3) f_y/6$$

$$M_{pd} = M_p(1 - 0.5d/D' - 1.6(d/D')^2)$$

$$\rho = 0.6 f_{cu}/f_y \quad *$$

$$n' = 5.5(M_{pd}/M_p)(\rho D'/t)^{0.66}$$

$$M_u = M_{pd}(1 + 0.01 n')$$

$$\phi = 0.02(25 - L/D') \quad \geq 0$$

$$\delta = 0.25(25 - L/D') \quad \geq 0$$

$$C_2 = (1 + \phi + \phi^2)^{-0.5}$$

$$C_1 = 4\phi\delta C_2$$

$$\gamma = 0.67 A_g(f_{cu} + C_1 f_y t/D')/P_u$$

$$K_{20} = 0.9\gamma^2 + 0.2 \quad \leq 0.75$$

$$K_{30} = 0.04 - \gamma/15 \quad \geq 0$$

β = ratio of smaller to larger end moments

Figure 5.3.2: Design methodology for fully and partially grouted damaged and undamaged tubular members

(...continued...)

$K_2 \quad = K_{20}(115 - 30(2\beta - 1)(1.8 - \gamma) - 100\lambda)/(50(2.1 - \beta)) \geq 0 \text{ and } \leq K_{20}$

$K_3 \quad = K_{30} + \lambda((0.5\beta + 0.4)(\gamma^2 - 0.5) + 0.15)/(1 + \lambda^3)$ *

$T_2 \quad = 4K_3/K_1$

$T_1 \quad = 1 - K_2/K_1 - T_2$

$\Delta \quad = d_o + e$

$P \quad = \tfrac{1}{2}(-M_u^2/P_{col}T_2\Delta^2 - T_1M_u/T_2\Delta \pm ((-M_u^2/P_{col}T_2\Delta^2 - T_1M_u/T_2\Delta)^2 + 4M_u^2/T_2\Delta^2)^{0.5})$

<u>Partially Grouted</u>

The terms above marked with a * should be set equal to zero.

$P \quad = 1/(1/P_{col} + T_1\Delta/M_u)$

Figure 5.3.2: Design methodology for fully and partially grouted damaged and undamaged tubular members

(...continued)

where m is the modular ratio E_s/E_g. $A_{tr} = A_s + A_g/m$ is the transformed area which can now be multiplied by E_s to obtain the axial stiffness for the entire cross-section without independent consideration of the steel and grout contributions.

By considering a curved section of composite member, and assuming the grout in tension does not crack, the relevant expression for second moment of area can readily be derived[5.8].

For a column subjected to end loading P applied with eccentricity e with respect to the undeformed cross-section, the total eccentricity at the assumed mid-height position of the dent damage is given by:

$$e_t = e + d_1 + e_{tr}$$

where d_1 is the out-of-straightness (O-O-S) or bow of the column. The moment at the damaged section is Pe_t and the corresponding stress Pe_t/z_{tr} where z_{tr} is the section modulus to the damaged face of the transformed section. The total stress at this face is the sum of the axial and bending stresses, the latter which has to be factored by the first order column buckling amplification term $1/(1 - P/P_E)$ to account for the effects of column buckling. Here P_E is the column Euler load given by $\pi^2 E_s I_{tr}/L^2$, L being the (effective) column length.

Assuming full composite interaction, linear elastic material behaviour, and that the response of the entire damaged member is represented by the bending stiffness at the damaged section for loading up to failure, the sum of the stresses at the damaged face is:

$$\frac{P}{A_{tr}} + \frac{Pe_t/z_{tr}}{1 - P/P_E} \qquad \ldots\ 5.3.1$$

Adopting the failure criterion that the ultimate strength of the column $P_u = \sigma_u A^*_{tr}$ (where A^*_{tr} is the undamaged transformed area) is reached when this summation achieves yield stress f_y, Equation 5.3.1 becomes:

$$\frac{\sigma_u A^*_{tr}}{A_{tr}} + \frac{\sigma_u A^*_{tr} e_t/z_{tr}}{1 - \sigma_u A^*_{tr}/P_E} = f_y \qquad \ldots\ 5.3.2$$

By introducing some normalising parameters, this can be expressed in the form preferred by Parsanejad[5.1].

$$\left[\frac{\sigma_u}{\sigma_y}\right]^2 - \left[\frac{1 + k}{\lambda^2} + q\right] \frac{\sigma_u}{\sigma_y} + \frac{q}{\lambda^2} = 0 \qquad \ldots\ 5.3.3$$

where $k = A_{tr}e_t/z_{tr}$, $q = A_{tr}/A^*_{tr}$ and $\lambda = \sqrt{f_y A_{tr}/P_E} = (L/\pi\, r_{tr})\sqrt{f_y/E}$ is the column non-dimensional slenderness parameter, and r_{tr} is the radius of gyration of the transformed section.

Parsanejad demonstrated[5.1] that this equation provided reasonable estimates of the failure loads for his own tests. However, as observed by Loh[5.8], the strength at low slenderness ($\lambda < 0.7$) is limited to $f_y\, A_{tr}$ which, compared with the strength of short grouted members calculated in accordance with BS5400: Part 5, ignores the significant benefits which accrue due to the triaxial containment of the infill. This containment effect is a function of the circumferential stiffness of the tubular and thus is inversely proportional to the diameter to thickness ratio. For the range of typical geometries for offshore braces in particular, this limitation is not likely to prove significant.

More importantly, Equation 5.3.3 limits the flexural strength to first yield in bending albeit on the transformed section. For undamaged steel sections of suitable proportions[5.12], the full plastic moment capacity is 27.3% greater than the first yield moment. As the presence of the grout inhibits the development of local buckles in undamaged sections and the growth of dents in damaged sections, the full plastic capacity is available for D/t well beyond the usual limits. Consequently, it is expected that Equation 5.3.3 will under-estimate strength where flexure is an important proportion of the total stress pattern.

Further, the assumption that the grout remains linear and elastic up to ultimate conditions is questioned by Loh. He suggests that the grout cracks under tension and so only that portion in compression is effective. In Reference 5.7, models were reported to be cut through at the damage section after testing and no break-down of the grout was observed. It is possible that cracks to the tension face could have been missed with this form of investigation.

In Table 5.3.1, ratios of the experimental strengths of data listed in Databases 5.2.1 to 5.2.6 to their strengths as predicted by this method are listed together with a statistical evaluation of these ratios for damaged members alone, undamaged members alone, and for both damaged and undamaged members. No differentiation is made between members subjected to axial compression alone and combined compression and bending because the combined loading case can be interpreted as axial compression eccentrically applied which is treated directly by this method. Indeed, most reported combined loading cases actually relate to eccentrically loaded members, and even in cases of nominally axial compressive loading, the load is frequently applied with a small degree of eccentricity so in essence are combined loading cases anyway.

Reference	Model ID	Parsanejad [5.1] eqn (5.3.3)	BS5400/ API I	BS5400/ API II	Recommended Approach	Recommended only pin-ended models	Recommended only pin-ended models & excl. M2
5.7	A1	0.788	0.814	0.728	1.292	1.292	1.292
	A2	1.184	1.141	0.981	0.970	0.970	0.970
	A3	1.022	1.039	0.909	0.893	0.893	0.893
	A4	1.201	1.146	0.946	0.933	0.933	0.933
	A5	1.427	1.400	1.170	1.159	1.159	1.159
	B1	1.010	1.038	0.933	0.917	0.917	0.917
	B2	1.089	1.057	0.918	0.906	0.906	0.906
	B3	0.924	0.936	0.822	0.804	0.804	0.804
	B4	1.009	0.961	0.808	0.793	0.793	0.793
	C1	1.212	1.244	1.154	1.149	1.149	1.149
	C2	1.299	1.324	1.260	1.255	1.255	1.255
	D1	1.248	1.188	0.982	0.965	0.965	0.965
	D2	1.414	1.341	1.110	1.090	1.090	1.090
	F1	1.653	1.668	1.412	1.397	1.397	1.397
	F2	1.476	1.492	1.262	1.249	1.249	1.249
	G1	1.227	1.183	1.071	1.056	1.056	1.056
	G2	1.374	1.335	1.182	1.166	1.166	1.166
	H1	1.205	1.095	1.022	1.006	1.006	1.006
	H2	1.233	1.128	1.044	1.028	1.028	1.028
	J1	1.427	1.483	1.232	1.228	1.228	1.228
	J2	1.455	1.549	1.245	1.242	1.242	1.242
	K1	1.475	1.449	1.181	1.174	1.174	1.174
	K2	1.324	1.265	1.075	1.068	1.068	1.068
	L1	1.336	1.196	1.045	1.035	1.035	1.035
	L2	1.315	1.169	1.036	1.026	1.026	1.026
	M1	1.243	1.266	1.074	1.066	1.066	1.066
	M2	1.899	1.925	1.640	1.627	1.627	
	N1	1.263	1.231	1.049	1.037	1.037	1.037
	N2	1.367	1.321	1.140	1.127	1.127	1.127
	O1	1.398	1.280	1.139	1.124	1.124	1.124
	O2	1.210	1.087	1.007	0.991	0.991	0.991
	Q1	1.551	1.323	1.112	1.095	1.095	1.095
	Q2	1.804	1.582	1.302	1.285	1.285	1.285
	Q3	1.327	1.173	1.001	0.991	0.991	0.991
	Q4	1.629	1.431	1.235	1.222	1.222	1.222
5.1	A2	1.153	1.222	1.113	1.109	1.109	1.109
	B1	1.219	1.230	1.125	1.113	1.113	1.113
	B2	1.205	1.442	1.200	1.193	1.193	1.193
	C2	1.017	1.105	0.971	0.967	0.967	0.967
	D1	1.099	1.171	0.988	0.982	0.982	0.982
	D2	1.167	1.381	1.121	1.119	1.119	1.119
	E1	1.119	1.256	1.045	1.037	1.037	1.037
	E2	1.267	1.224	1.093	1.074	1.074	1.074
5.2	G1	0.986	1.091	1.075	1.066	1.066	1.066
	G2	1.299	1.342	1.220	1.193	1.193	1.193
	G3	1.239	1.381	1.216	1.202	1.202	1.202
	F1	1.433	1.540	1.278	1.264	1.264	1.264
	H1	1.283	1.303	1.219	1.196	1.196	1.196

Table 5.3.1: Ratios of test to predicted strength for fully grouted damaged and undamaged tubulars

(continued...)

Reference	Model ID	Parsanejad [5.1] (eqn 5.3.3)	BS 5400/ API I	BS 5400/ API II	Recommended Approach	Recommended - only pin-ended models	Recommended - only pin-ended models * excl. M2
5.5	PF15	1.256	1.283	1.147	1.169	1.169	1.169
	PF16	1.485	1.515	1.338	1.353	1.353	1.353
5.9	A3	1.320	1.372	1.287	1.278	1.278	1.278
	B3	1.049	1.092	1.008	1.003	1.003	1.003
	C3	0.954	1.110	0.899	0.899	0.899	0.899
Damaged members	Aver	1.275	1.270	1.105	1.105	1.105	1.095
	sd	0.212	0.197	0.159	0.153	0.153	0.136
	COV	0.166	0.155	0.144	0.138	0.138	0.124
5.8	Wimpey	0.774	0.853	0.761	0.761		
	Wimpey	0.756	0.855	0.742	0.742		
	Grigory	0.601	0.724	0.670	0.670		
5.7	C3	1.218	1.306	1.180	1.180	1.180	1.180
	P1	0.983	1.054	0.953	0.953	0.953	0.953
5.1	A1	1.024	1.092	1.022	1.022	1.022	1.022
	C1	0.951	1.106	0.958	0.958	0.958	0.958
5.3	1	1.821	1.825	1.155	1.154	1.154	1.154
	2	1.582	1.612	1.131	1.130	1.130	1.130
	3	1.155	1.127	0.985	0.984	0.984	0.984
	4	1.264	1.133	1.095	1.092	1.092	1.092
	5	0.759	0.834	0.703	0.702	0.702	0.702
	6	0.895	0.821	0.799	0.796	0.796	0.796
Undamaged members	Aver	1.060	1.103	0.935	0.934	0.997	0.997
	sd	0.348	0.322	0.181	0.181	0.155	0.155
	COV	0.328	0.292	0.194	0.194	0.156	0.156
Damaged undamaged members	Aver	1.233	1.237	1.072	1.072	1.088	1.079
	sd	0.256	0.234	0.176	0.172	0.157	0.142
	COV	0.208	0.189	0.164	0.160	0.144	0.132

Table 5.3.1: Ratios of test to predicted strength for fully grouted damaged and undamaged tubulars

The statistics presented are average (or mean, or bias), standard deviation (sd), and coefficient of variation (COV). The last is the normalised form of sd given by sd/average and is a direct measure of the accuracy of an approach. Good prediction models have COVs around 0.05, and reasonable models around 0.12. Models with COVs of 0.25 and greater, whilst in isolation might be acceptable, when used in a reliability evaluation will tend to completely dominate the evaluation and not provide a very meaningful insight into the results. "Average" on the other hand can be interpreted in two ways. If based on empirically derived factors, it means these need to be re-evaluated to determine more suitable values. In such cases, an average of unity can be realised and the model is then described as unbiased. In contrast, if the method is theoretically based, the greater its departure from unity, the less appropriate is the basic theory.

Table 5.3.1 shows that Parsanejad's model is of reasonable accuracy for damaged tubulars, inaccurate for undamaged tubulars, and reasonable to poor accuracy overall. The bias is such that in principle it should be possible to reduce it closer to unity with the introduction of suitable empirical constants.

IV 5.3.2 <u>BS5400 / API - I and II</u>

Seeking an improved model for containment, Loh[5.8] pursued the application of BS5400 : Part 5[5.10], the background of which is documented elsewhere[5.13]. Loh simplified the process for determining the ultimate moment capacity of grout filled tubulars before modifying this to account for the presence of damage. He then sought to invoke the use of the AISC-LRFD equation[5.11] for composite columns as well as the API RP2A-LRFD column curve[5.12], and the corresponding compression-moment interaction equations. His evaluation, based on a database not dissimilar to that presented in Databases 5.2.1 to 5.2.6, indicated that the BS5400: Part 5 methodology provided the most accurate basis. On the basis of this, the methodology described in this section was developed although initially the API RP2A-LRFD compression-moment interaction approach was examined to see whether it could be exploited in some way.

The equation for basic grout-filled tubular compressive strength proposed by Loh was:

$$P_u \quad = \quad A_s f_y + 0.67 A_g f_{cu} \qquad \text{.... 5.3.4}$$

The 0.67 factor is required to convert cube strength into an appropriate grout design strength. This conversion reflects both scale effects and strain-rate effects and is widely accepted in concrete design. The steel and concrete areas can be readily modified should the section be damaged - see Figure 5.3.2.

For determining the full plastic moment capacity based on rectangular stress blocks (the grout is ignored in tension), BS5400 presents a semi-graphical solution. Loh simplified this by curve-fitting to derive an explicit approximation. This was further modified to account for changes to the compression face of the cross-section when damage is present. The resulting expression (n′ in Figure 5.3.2) enables the ultimate moment capacity for a damaged (or undamaged) grout-filled tubular section to be rapidly evaluated.

Calculating λ as $\sqrt{P_u/P_E}$ where the Euler load P_E is based on the fully transformed section I_{tr}, and substituting into the API RP2A column and compression-moment interaction expressions, enables the strength of damaged (and undamaged) grout filled tubulars to be calculated. The accuracy of this approach as demonstrated in Table 5.3.1 (see column headed BS5400/API-I) is only a slight improvement on the original Parsanejad approach.

An examination of the compression-moment interaction equations implicit in the BS5400 and API RP2A expressions shows these to differ significantly. This was considered to be the main source of inaccuracy in this methodology so the BS5400 form of interaction was examined. The resulting accuracy (BS5400/API-II, Table 5.3.1) shows a reasonable improvement on the preceding approach together with a major reduction in the bias. This BS5400/API-II approach was adopted to form the basis of the proposed method described in the following section.

IV 5.3.3 Recommended Approach

Based on the BS5400/API-II method, several refinements were considered in an effort to further improve the accuracy. These were raised by Loh as requiring consideration. AISC-LRFD recommends a knock-down factor of 0.8 be applied to the grout flexural stiffness when calculating the Euler buckling load. This is ostensibly to account for a reduction in the effectiveness of grout in tension. Introducing the 0.8 factor has no practical effect on the accuracy of the BS5400/API-II methodology but the bias increased by 0.02 suggesting the model is now less good at modelling physical response. Consequently, the full grout flexural stiffness was retained in the calculation of P_E.

Loh suggests that the term e_{tr} is not appropriate at ultimate load conditions. This is because the term was derived assuming fully effective grout and steel properties yet at failure the grout on only the compression face may be contributing. This was examined by introducing a knock-down factor on e_{tr} and varying it across a practical range. The smallest COV of 0.162 was obtained for a knock-down factor of -0.5: the corresponding bias was 1.034. For a factor of 1.0, the corresponding COV and bias were 0.175 and 1.146. For a factor of 0.0, the values were 0.164 and 1.072. Loh's proposition appears to be justified. However, a possibly more appropriate approach would be to examine the strain distribution at ultimate conditions and re-evaluate the relevant cross-sectional properties depending on the extent of tension present. This would require an iterative approach which at present does not appear justified. A knock-down factor of 0.0 is considered appropriate, in spite of the apparently improved modelling that results from adopting a value of -0.5. This is, on the basis of this evaluation, approximately 3.7% conservative. However, as discussed in the next paragraph, the term e_{tr} is not calculated where the second moment of area is approximated.

Along similar lines, Loh indicates that the second moment of area can be found approximately from $I_s + I_g/m$. This omits the terms involving shifts of the centroids of the steel and grout areas between the undamaged and damaged sections. Including the complete set of terms makes no difference at all to the bias or accuracy of the method. It is possible that the dent sizes considered are too small to influence this evaluation. The largest d/D ratio considered is 0.16. This is relatively small compared with that found in jacket members as a result of underwater explosions of 0.5 and 0.66[5.6]. However, until further relevant test data are available, this aspect cannot be examined further.

When considering the bow distortion, Loh suggests that this should be reduced by an amount corresponding to the length/1000 as this equates to the tolerance adopted in API/AISC specifications upon which the column curves are ostensibly based. Setting column strength equal to the Euler buckling strength at $\lambda = \sqrt{2}$, as these specifications do, implies that for this slenderness and beyond the column is perfectly straight. This is clearly not the case. In practice, initial O-O-S usually approximates length/1500 although, where reported, test model values are around length/5000. This implication suggests that the full value of any O-O-S should be considered together with any eccentricity in the applied loading: this has been adopted here. In the cases where no initial O-O-S was reported, nor no bow damage introduced, an O-O-S of length or effective length/1500 was adopted.

In preparing the final set of equations, some further minor refinements were introduced such as the correct equation for calculating the plastic moment capacity of steel sections rather than the simplification generally adopted of D^2Tf_y. These resulted in small changes to the bias with little effect on accuracy.

In Loh's assessment of design approaches, one particular model was identified as unsuitable for comparison. This is model Grigory 4 listed under Reference 5.8 in Database 5.3. The reason for its exclusion was that it was manufactured from ERW tube with a pronounced 'rounded' stub-column stress-strain curve which is not compatible with the stub-column curves exhibited by fabricated tubulars. Since in the development of the database, few published works have been found to present stub-column test results, it does not seem entirely justified to exclude this result on this reason alone. There would appear to be a separate reason for excluding this result, and, in the process, two others, the Wimpey 3.1 and 3.2 models listed under the same reference, based on the fact that they are non pin-ended models. Theoretically, it is possible to treat such members in a similar fashion by use of the effective length approach. This was done and produced the lowest ratio of actual to predicted in the case of the Grigory model, and relatively low ratios for the two Wimpey models - see Table 5.3.1. If these three models are excluded from the statistical evaluation, as shown in

Table 5.3.1, a considerable improvement in both COV (0.194 to 0.156) and mean bias (0.934 to 0.997) results for the undamaged models. Overall, for both damaged and undamaged models combined, the larger improvement is in COV (0.160 to 0.144).

One further model was considered for exclusion. This is model M2 of Reference 5.7 which is seen from Table 5.3.1 to have the largest ratio of P_{ex}/P_{pred} by a considerable margin. For the entire database, the result approximates to the mean plus 3.5 standard deviations, or 1 in 4000. For the database of pin-ended models, it represents the mean plus 3.7 standard deviations, or 1 in 8700. In reviewing Reference 5.7, it is apparent as noted by the authors that the result for this model is inconsistent with the remaining results from this source. The effect on the statistical evaluation of deleting this results is shown in Table 5.3.1. Again little effect on the mean but a further improvement in COV is realised (0.144 to 0.132).

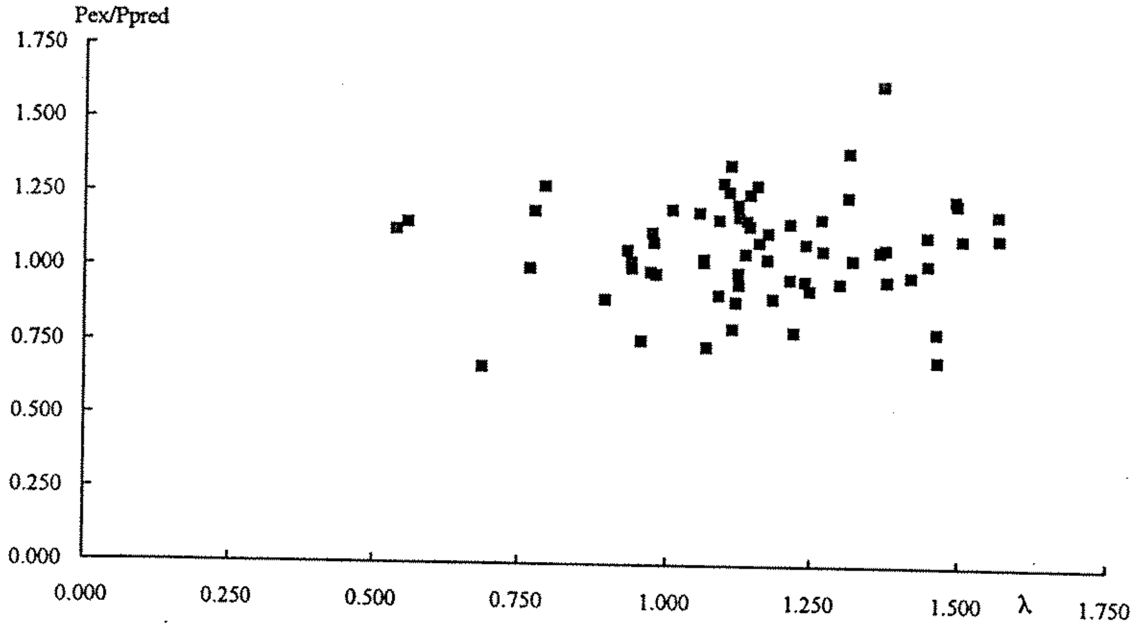
The impact of omitting these four models is more clearly appreciated by comparison of the plots presented in Figures 5.3.3(a) and (b). The former presents a plot of the P_{ex}/P_{pred} values versus the slenderness parameter λ for the full database and the latter with the four models excluded.

The final COV realised of 0.132 (or even 0.160 for the entire database) is good considering that a composite structure is under consideration and making allowances for a number of weaknesses found in the testing techniques adopted by some of the workers as discussed in Section IV 5.2. The proposed approach is thus recommended for use in the assessment of dent/bow damaged members where complete grout infilling is proposed as a repair method, and in the design of grouted intact members where additional strength/stiffness is required without any additional wave-load penalties.

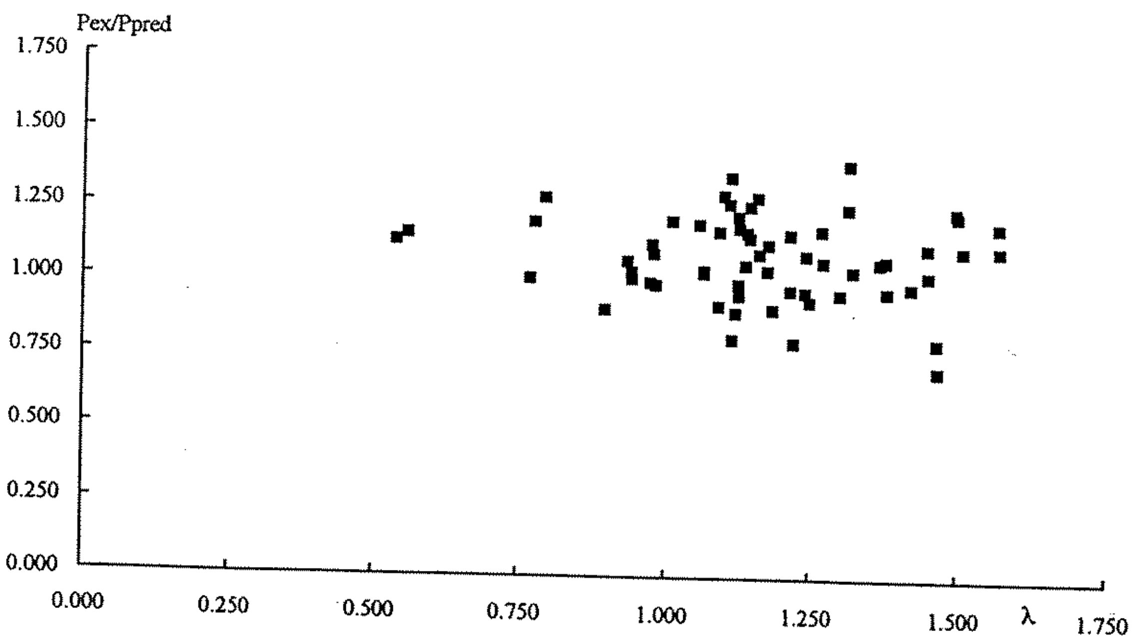
IV 5.3.4 <u>Scale (Size) Effects</u>

One of the concerns when exploiting model results for full scale application is the effect of scale. For steel only construction, provided the same form of manufacture is used, non-dimensionalisation of the results has long been established in the case of ductile failures such as yielding and buckling. With failure by fracture, tearing or cracking, such non-dimensionalisation is less obvious to achieve.

According to Reference 5.7, upon cutting open the specimens following testing, the grout was observed to be fully intact. This is consistent with a ductile form of failure suggesting it should be possible to non-dimensionalise the results. However, the same reference also claims that a scale effect is apparent, the larger the scale the stronger the models relatively.



(a) Entire database



(b) Pin-ended models only and excluding M2

Figure 5.3.3: Fully grouted members - Ratios of test to predicted strength for recommended design approach versus slenderness parameter

An examination for the basis of this claim is found to lie in the use of the Parsanejad method in the determination of theoretical strength. This method was found (Table 5.3.1) to be the least accurate of the four considered and therefore possibly not the best basis for drawing conclusions concerning the effects of scaling, or of any parameter. Nevertheless, an examination of the variation of P_{ex}/P_{pred} with diameter, where P_{pred} is in accordance with Parsanejad, (Figure 5.3.4) demonstrates how this conclusion will have arisen. In the lower plot, only the results of Reference 5.7 are presented. From this a trend for strength to increase with increasing diameter is apparent. However, this finding is not supported when the entire database is examined as in the upper plot.

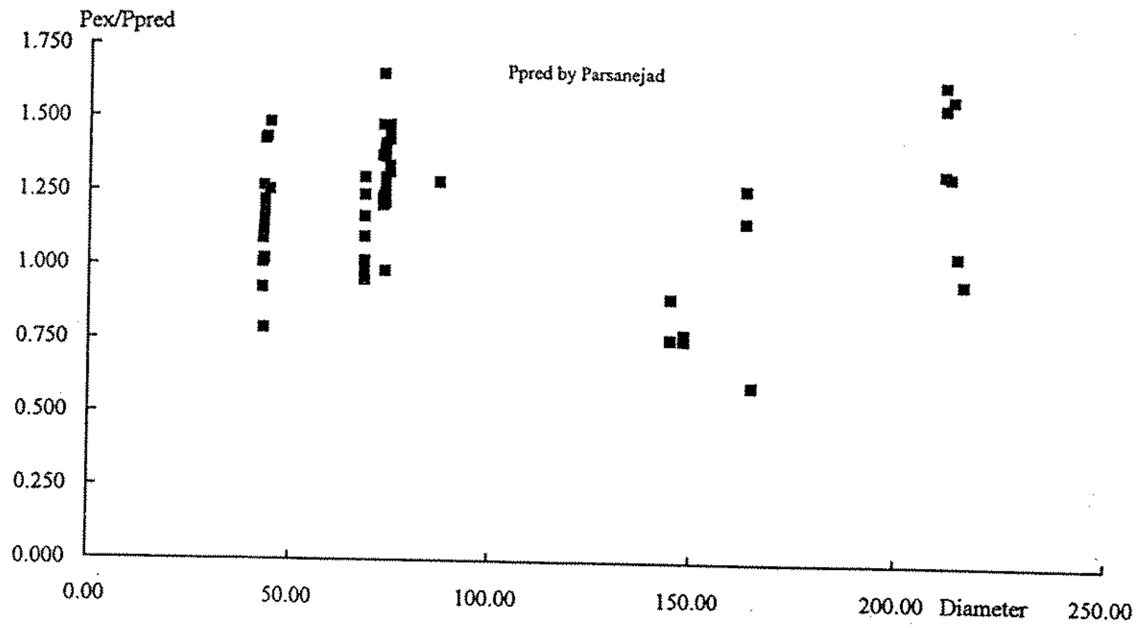
The same plots are presented in Figure 5.3.5 but with P_{pred} in accordance with the current proposal. It is clear, particularly from the lower plot, that a scale effect is not present.

IV 5.4 DESIGN APPROACH FOR PARTIALLY GROUTED MEMBERS

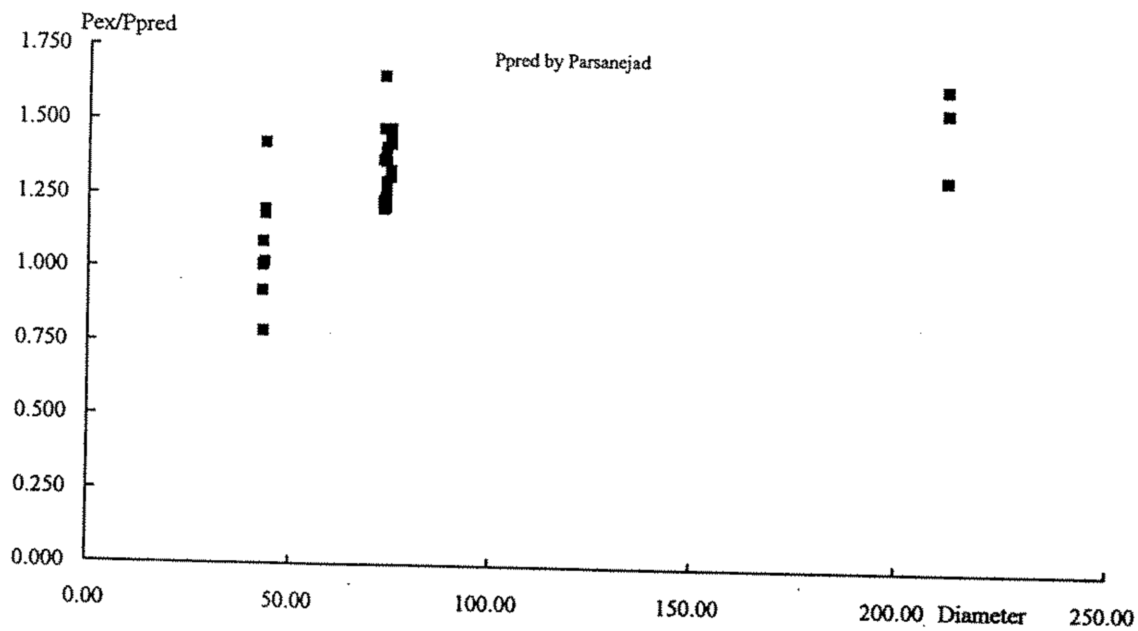
Test observations of partially grouted tubulars indicate the grout does not act compositely with the steel but still supports the steel in the same manner as for the fully grouted counterparts. On this basis, the equations in the recommended approach for fully grouted members can be simply modified by setting the contribution of the grout to zero. The relevant terms are indicated in Figure 5.3.2 by an asterisk.

Use of the remaining equations to estimate the strength of the partially grouted models listed in Databases 5.2.5 and 5.2.6 produced a very significant under-estimate of experimental strength. An examination of the results indicated that the K_3 term which controls the influence of the second-order term in the strength equation, was always negative. This implied a concave interaction curve was being invoked for steel sections when a convex one should be in place particularly for small (zero) levels of damage as reflected in Reference 5.12. As K_3 primarily reflected the grout contribution, it also was set to zero thereby adopting a linear form of compression-moment interaction.

Linearity was introduced to realise a considerable improvement in the comparison. Three results however were still not well modelled, namely, Wimpey 3.3 and Grigory 5 of Reference 5.8, both undamaged, and I1 of Reference 5.2. For the first two models, the ungrouted length was not specified, the incomplete filling apparently relating to a void at one end of the models. In the case of Wimpey 3.3, failure was precipitated by local buckling at the void, the small size of this helping to increase the strength by controlling the local buckling mode. A similar event appears to have taken place in relation to Grigory 5 although the rounded stub-column curve discussed earlier apparently resulted in a lower strength than would have been expected based on the steel properties above.

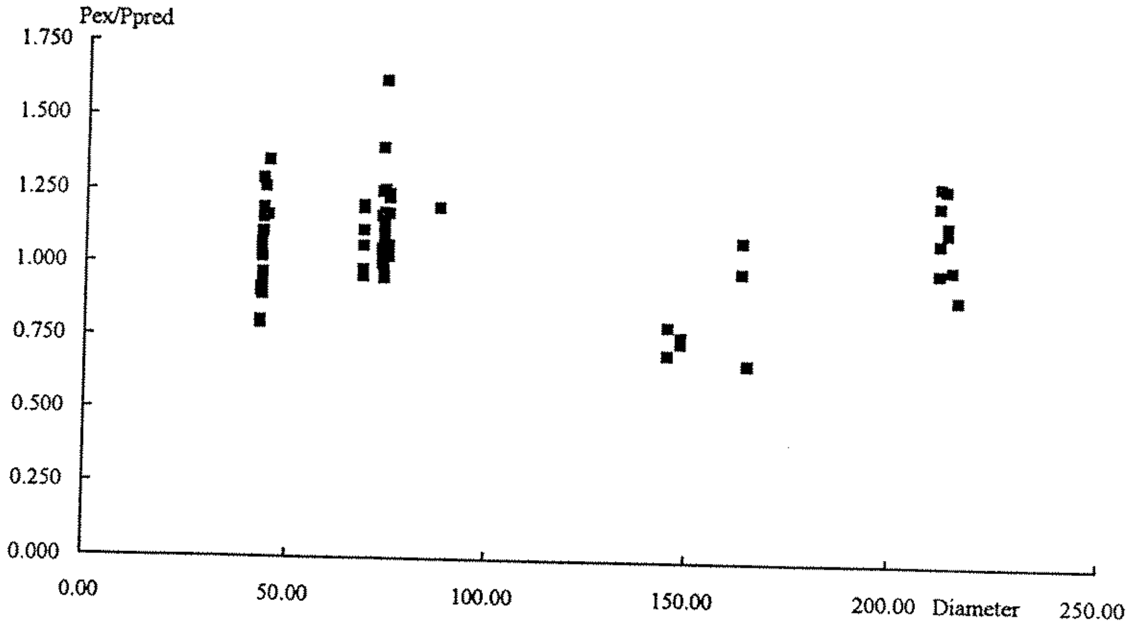


(a) Entire database

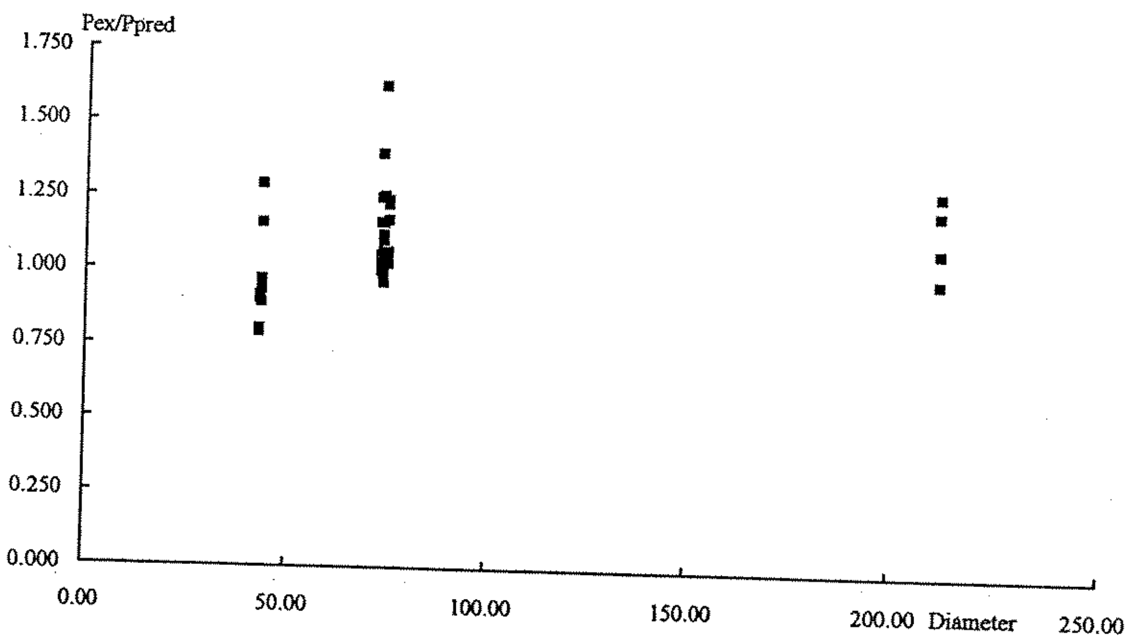


(b) Data from Reference 5.7

Figure 5.3.4: Effect of scale on fully grouted members - Variation of test to Parsanejad[5.1] predicted strengths with diameter



(a) Entire database



(b) Data from Reference 5.7

Figure 5.3.5: Effect of scale on fully grouted members - Variation of test to recommended predicted strengths with diameter

Model I1 was found to fail in the grouted region but at a location adjacent to one end of the grouted length. Thus the relevant cross-sectional properties were those of the tubular as undented, and the magnitude of the initial bow is less than the maximum value initially used. The appropriate value of bow at the end of the grouted length was reported[5.7]. Eliminating the dent and introducing the appropriate value of initial bow led to the ratio of P_{ex}/P_{pred} (1.292) shown in Table 5.4.1. This is presented together with the comparisons of the remaining models. The statistical evaluation is conducted including and excluding the undamaged models. Based on the latter, good accuracy has been realised with a slightly conservative model bias. This bias is only marginally greater than that found for the best of the fully-grouted member evaluations (1.094 compared with 1.079). Figure 5.4.1 presents the corresponding plot.

Reference	Model ID	Recommended Approach	Recommended only pin-ended models
5.2	I1	1.292	1.292
	I2	1.106	1.106
	I3	1.015	1.015
	H2	1.047	1.047
		0.000	0.000
5.4	J1	0.932	0.932
	J2	1.026	1.026
	J3	0.914	0.914
	J4	1.022	1.022
	J5	1.054	1.054
	J6	1.063	1.063
	K1	1.070	1.070
	K2	0.999	0.999
	K3	1.289	1.289
	K4	1.134	1.134
		0.000	0.000
5.5	PF1	1.055	1.055
	PF2	1.066	1.066
	PF3	1.037	1.037
	PF4	1.129	1.129
	PF5	1.099	1.099
	PF6	1.130	1.130
	PF7	1.121	1.121
	PF8	1.118	1.118
	PF9	1.120	1.120
	PF10	1.189	1.189
	PF11	1.094	1.094
	PF12	1.140	1.140
	PF13	1.232	1.232
	PF14	1.148	1.148
5.8	Wimpey 3.3	1.949	
	Grigory 5	1.429	
	Average	1.134	1.094
	sd	0.186	0.088
	COV	0.164	0.081

Table 5.4.1: Ratios of test to predicted strengths for partially grouted damaged and undamaged tubulars

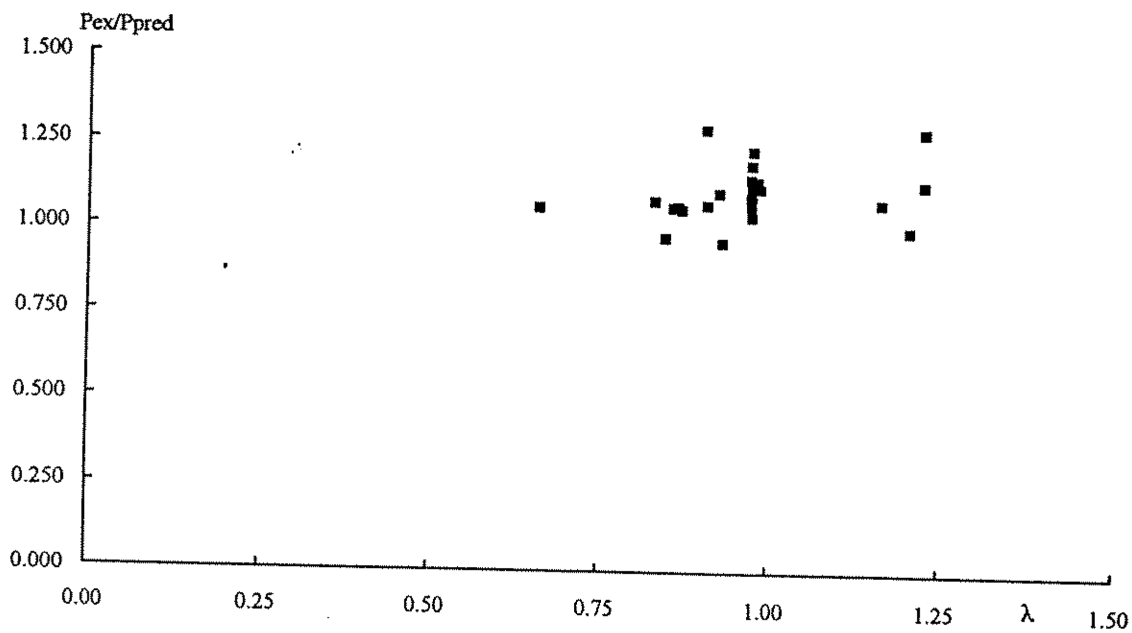


Figure 5.4.1: **Partially grouted members - Ratios of test to predicted strengths for recommended design approach versus slenderness parameter**

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Reference	Model ID	Outer Diam D' (mm)	Thickness t (mm)	Effective length facto K	Length L (mm)	Grout Lgth Lg (mm)	Yield Stress fy (N/mm^2)	Elast. Mod. Es (kN/mm^2)	Grout Str. fcu (N/mm^2)	Elast. Mod. Eg (kN/mm^2)	Initial O-O-R (%)	Initial O-O-S (%)	Dent Depth dd (mm)	Bow do (mm)	Ldg Ecc. e (mm)	Ult. Load Pu (kN)	Comments
5.7	A1	44.45	1.27		900	900	519	205	43	11	-	-	4.54	1.18	-	56.5	
	A2	44.45	1.24		900	900	519	205	74	11	-	-	4.62	0.93	-	85.2	
	A3	44.45	1.24		900	900	519	205	43	11	-	-	5.62	1.78	-	63.9	
	A4	44.45	1.23		900	900	519	205	74	11	-	-	6.15	1.83	-	72	
	A5	44.45	1.24		900	900	519	205	74	11	-	-	4.59	1.58	-	96.5	
	B1	44.45	1.59		900	900	517	205	43	11	-	-	4.71	1.28	-	83.2	
	B2	44.45	1.59		900	900	517	205	74	11	-	-	4.7	1.08	-	91.6	
	B3	44.45	1.6		900	900	517	205	43	11	-	-	5.9	2.23	-	65.8	
	B4	44.45	1.58		900	900	517	205	74	11	-	-	6.37	1.9	-	70.9	
	C1	76.25	2.64		1560	1560	502.8	205	74	11	-	-	3.34	0.55	-	375	
	C2	76.25	2.64		1560	1560	502.8	205	74	11	-	-	3.32	0.28	-	416	
	D1	76.25	2.63		1560	1560	502.8	205	74	11	-	-	11.89	4.7	-	224.3	
	D2	76.25	2.63		1560	1560	502.8	205	74	11	-	-	11.99	4.35	-	256.7	
	F1	76.25	3.2		1810	1810	464.1	205	74	11	-	-	7.42	2.18	-	371	
	F2	76.25	3.2		1810	1810	464.1	205	74	11	-	-	7.35	2.25	-	331	
	G1	76.25	3.29		1550	1550	486	205	74	11	-	-	7.1	1.1	-	365	
	G2	76.25	3.29		1550	1550	486	205	74	11	-	-	7.31	1.75	-	388	
	H1	76.25	3.25		1290	1290	471.5	205	74	11	-	-	7.43	1.35	-	381	
	H2	76.25	3.25		1290	1290	471.5	205	74	11	-	-	7.28	1.68	-	383	
	J1	76.25	1.6		1850	1850	510.2	205	74	11	-	-	7.79	1.85	-	197	
	J2	76.25	1.6		1850	1850	510.2	205	74	11	-	-	7.59	3.28	-	190	
	K1	76.25	1.59		1580	1580	499.7	205	74	11	-	-	7.32	2.35	-	239.5	
	K2	76.25	1.59		1580	1580	499.7	205	74	11	-	-	7.54	1.2	-	229	
	L1	76.25	1.59		1320	1320	501.3	205	74	11	-	-	7.64	1.4	-	263	
	L2	76.25	1.59		1320	1320	501.3	205	74	11	-	-	7.55	1.12	-	265	
	M1	76.25	2.72		1820	1820	495.7	205	74	11	-	-	7.32	1.75	-	259	
	M2	76.25	2.72		1820	1820	495.7	205	74	11	-	-	7.44	1.6	-	397	
	N1	76.25	2.63		1560	1560	488	205	74	11	-	-	7.61	2.75	-	282	
	N2	76.25	2.63		1560	1560	488	205	74	11	-	-	7.71	2.03	-	317	
	O1	76.25	2.7		1300	1300	481.2	205	74	11	-	-	7.84	2.78	-	353	
	O2	76.25	2.7		1300	1300	481.2	205	74	11	-	-	7.63	1.2	-	340	
	Q1	219.5	8		4910	4910	319.4	205	74	11	-	-	29.56	7.6	-	1966.8	
	Q2	219.5	8.09		4910	4910	319.4	205	74	11	-	-	28.9	13.25	-	2118.1	
	Q3	219.5	8.09		4910	4910	319.4	205	74	11	-	-	20.54	6.83	-	1913.9	
	Q4	219.5	8.15		4910	4910	319.4	205	74	11	-	-	20.77	5.6	-	2420.7	

Models A to P heat-treated cold-drawn seamless tube.
Models Q hot finished pipe.
Each diam. and thick. measured.
Grout of Oilwell B cement, w/c ratio = 0.36.
Coupons tested dynamically but limited in number.

Es, Eg probably not measured - standard values quoted.
Damage by 60° indentor - saddle support - limited measurement details.
Models tested at 14 and 28 days. Tests conducted statically.
No check on pinned ended conditions.
Calculated undamaged transformed area not equal to quoted value.

Database 5.2.1: Fully grouted and damaged and subjected to axial compression (continued..)

Reference	Model ID	Outer Diam D' (mm)	Thickness t (mm)	Effective length facto K	Length L (mm)	Grout Lgth Lg (mm)	Yield Stress fy (N/mm^2)	Elast. Mod. Es (kN/mm^2)	Grout Str. fcu (N/mm^2)	Elast. Mod. Eg (kN/mm^2)	Initial O-O-R (%)	Initial O-O-S (%)	Dent Depth dd (mm)	Bow do (mm)	Ldg Ecc. e (mm)	Ult. Load Pu (kN)	Comments
5.1	A2	44.57	1.38		1254	1254	588	198	32	16.5	0.06	0.023	3.217655	0	0.3048	69.66335	
	B1	44.593	1.393		1254	1254	565	198	32	16.5	0.075	0.015	6.25536	-0.32604	0.6096	61.53953	
	B2	44.59	1.39		1254	1254	578	197	32	16.5	0.052	0.02	5.67216	5.68062	2.2352	47.33891	
	C2	69.869	1.419		1860	1860	536	197	32	16.5	0.14	0.02	5.250115	-0.1302	0.9398	123.9429	
	D1	69.878	1.398		1860	1860	494	199	32	16.5	0.15	0.014	10.1008	0.7812	0.9906	103.9043	
	D2	69.857	1.407		1860	1860	519	200	32	16.5	0.14	0.03	10.43178	7.8678	2.0574	88.08665	
	E1	44.475	1.465		1254	1254	369	192	29.6	15.7	0.31	0.017	4.890237	6.49572	0.4826	39.03861	
	E2	44.48	1.48		1254	1254	352	183	29.6	15.7	0.29	0.02	6.8456	0.1881	0.2032	54.25953	
Primarily analytical paper. Parsanejad et al [5.2] provides some model details, eg, tubes cold drawn, Grout area based on diam. to mid-thickness of steel.								Some equations presented contain errors. These have been corrected by Loh [5.8].									
5.2	G1	69.922	1.392		1905	1905	219	206	15.7	14.9	0.18	0.068	4.893042	0.09525	-7.832979	90.93515	
	G2	69.855	1.395		1905	1905	232	181	15.7	14.9	0.17	0.06	10.07047	1.12395	19.65487	86.74235	
	G3	69.854	1.404		1905	1905	231	196	15.7	14.9	0.42	0.047	8.782135	8.13435	24.25868	63.28595	
	F1	44.433	0.723		1337	1337	179	172	15.7	14.9	0.1	0.023	6.289869	2.21942	10.86194	24.33648	
	H1	88.799	1.559		1905	1905	224	215	15.7	14.9	0.24	0.036	12.70214	4.21005	23.46756	128.7482	
Models F, G heat-treated cold-drawn seamless, model H welded. Detailed initial surveys including O-O-R and O-O-S. Yield measured from both ends.								Grout of w/c ratio = 0.45, Calgrout additive. Properties from 50.8 mm cylinders. Damage by 60° indentor, applied in beam mode. Reverse bending required. Only damage ratios quoted. Tests at 31 and 32 days. Both yield stress & tests conducted dynamically. Pinned end configuration produced eccentricity as models deflected. Eccenticity determined but later corrected by Loh [5.8].									
5.6	5A	273	7.8		4800		-	-	42.8	-	-	-	141	44.2	-	1680	
	5B	273	7.8		4800		-	-	42.8	-	-	-	140	50.4	-	1730	
	6B	273	7.8		4800		-	-	42.8	-	-	-	175.5	83.5	-	1257	
Damage only representative of explosions whilst all other references simulate vessel impact. Models of hot-finished pipe. No geometry or material property measurements reported. Denting carefully introduced. Detailed measurements provided.								Grout of ordinary Portland cement, w/c ratio = 0.44. tested at 22 days. One end fixed, other pinned.									

Database 5.2.1: Fully grouted and damaged and subjected to axial compression (..continued)

Reference	Model ID	Outer Diam D' (mm)	Thickness t (mm)	Effective length facto K	Length L (mm)	Grout Lgth Lg (mm)	Yield Stress fy (N/mm^2)	Elast. Mod. Es (kN/mm^2)	Grout Str. fcu (N/mm^2)	Elast. Mod. Eg (kN/mm^2)	Initial O-O-R (%)	Initial O-O-S (%)	Dent Depth dd (mm)	Bow do (mm)	Ldg Ecc. e (mm)	Ult. Load Pu (kN)	Comments
5.9	A3	219.075	6.2738		4541.52	4541.52	240	200.4897	30.17241	17.11133	0.05	0.01	21.7678	2.724912	43.815	849.759	
	B3	219.3036	4.7498		4544.06	4544.06	230.3448	211.8207	26.7931	15.75576	0.32	0.06	21.9202	4.998466	43.86072	520.533	
	C3	219.5322	3.429		4574.54	4574.54	271.7241	212.4138	47.54483	23.65544	2.64	0.02	21.844	4.117086	43.90644	542.778	

Hot rolled ERW pipe ASTM A53 Grade B. Models carefully surveyed.
Static and dynamic yield stress. Three specimens per D/t group.
Compressive yield stress measured via stub-column tests.
Residual stresses determined by sectioning.

Grout of API Class A Portland cement, densified microsilica, w/c ratio = 0.60-0.65.
50 mm cubes, tested up to and including 28 days. Models tested at 28 days.
Damage by knife-edged indentor - saddle supported.
Tests pin-ended eccentrically loaded. Models strain gauged along length.

Database 5.2.2: Fully grouted and damaged and subjected to combined compression and moment

Reference	Model ID	Outer Diam D' (mm)	Thickness t (mm)	Effective length facto K	Length L (mm)	Grout Lgth Lg (mm)	Yield Stress fy (N/mm^2)	Elast. Mod. Es (kN/mm^2)	Grout Str. fcu (N/mm^2)	Elast. Mod. Eg (kN/mm^2)	Initial O-O-R (%)	Initial O-O-S (%)	Dent Depth dd (mm)	Bow do (mm)	Ldg Ecc. e (mm)	Ult. Load Pu (kN)	Comments
5.8	Wimpey 3.1	152.8064	4.7752	0.58	6096	6096	251.7241	207	84.13793	37.86207	-	-	-	-	0	802.1547	
	Wimpey 3.2	152.8064	4.7752	0.65	6096	6096	257.931	207	79.31034	35.72414	-	-	-	-	0	702.0522	
	Grigory 4	168.9862	3.937	0.51	5059.68	5059.68	341.3793	207	93.7931	42.2069	-	-	-	-	0	1138.944	
5.7	C3	76.25	2.64	-	1560	1560	502.8	205	74	11	-	-	-	-	0	384	
	P1	76.25	2.64	-	1560	1560	502.8	205	74	11	-	-	-	-	0	310	
5.1	A1		1.397	-	1254	1254	584	199	32	16.5	0.022	0.009	-	-	0	71.47405	
	C1		1.406	-	1860	1860	505	198	32	16.5	0.11	0.02	-	-	0.85625	129.0046	

Notes to reference 5.8:

Models 3.1, 3.2 seamless pipe. Grout of Oilwell B cement, w/c ratio = 0.34.
Grout strength from 75 mm cubes. Grout modulus determined by Loh [5.8] from BS5400:Part 5.
Model 4 of ERW pipe with significant through-thick.residual stresses
Grout of ASTM Type I Portland cement, w/c ratio = 0.34.
Grout strength from 50 mm cubes. Grout modulus determined by Loh [5.8] from BS5400:Part 5.
Es assumed 207 kN/mm^2. Effective lengths for 3.1, 3.2 determined from strain gauge measurements.
For model 4, determined from measured bending curvatures.
Yield stress probably dynamic. Model tests probably static.

Note to reference 5.7:

For commentary, see Database 5.2.1

Note to reference 5.1:

For commentary, see Database 5.2.1

Database 5.2.3: Fully grouted and undamaged and subjected to axial compression

Reference	Model ID	Outer Diam D' (mm)	Thickness t (mm)	Effective length facto K	Length L (mm)	Grout Lgth Lg (mm)	Yield Stress fy (N/mm^2)	Elast. Mod. Es (kN/mm^2)	Grout Str. fcu (N/mm^2)	Elast. Mod. Eg (kN/mm^2)	Initial O-O-R (%)	Initial O-O-S (%)	Dent Depth dd (mm)	Bow do (mm)	Ldg Ecc. e (mm)	Ult. Load Pu (kN)	Comments
5.3	1	216	2.8		2160	2160	418	204.0156	70	15.648	-	-	-	-	36.76623	1350	
	2	216	2.8		2160	2160	418	204.4736	55.8	12.7796	-	-	-	-	190.6287	334	
	3	168.3	5		3360	3360	404	204.3788	58.29	13.28258	-	-	-	-	32.64945	665	
	4	168.3	5		3360	3360	404	204.4528	56.33	12.88666	-	-	-	-	327.9174	144	
	5	154.8	9.9		4460	4460	489	204.1833	64.11	14.45822	-	-	-	-	30.51372	582	
	6	154.8	9.9		4460	4460	489	204.2257	62.76	14.18552	-	-	-	-	305.6921	180	

At least some models were fabricated but some possibly seamless because of size.
Grout of Oilwell B cement, w/c ratio = 0.34.
Grout strength from 75 mm cubes. Modulus not measured but calculated from
E_g = (1508 + 202 fcu) N/mm^2 in accordance with Billington & Lewis, OTC 3083 1978.
Modular ratio from same source = 10000/(67 + 10 fcu) so Es found as 204 kN/mm^2
but Es from stress-strain curves =198410 N/mm^2.

Coupons tested dynamically and curves presented but quoted yield stresses modified in accordance with RP 2A local buckling equation.
Thick. & diam. measured but diam.not quoted. Maxm O-O-R = 0.7%. Maxm O-O-S = 1 mm but no details.
Tests conducted statically. Strain ratios used to check correct eccentricity applied. Since composite action occurs, this is assumed to be based on composite section and not steel section as interpreted by others. Improved comparison with predictions if composite action assumed.

Database 5.2.4: Fully grouted and undamaged and subjected to combined compression and moment

Reference	Model ID	Outer Diam D' (mm)	Thickness t (mm)	Effective length facto K	Length L (mm)	Grout Lgth Lg (mm)	Yield Stress fy (N/mm^2)	Elast. Mod. Es (kN/mm^2)	Grout Str. fcu (N/mm^2)	Elast. Mod. Eg (kN/mm^2)	Initial O-O-R (%)	Initial O-O-S (%)	Dent Depth dd (mm)	Bow do (mm)	Ldg Ecc. e (mm)	Ult. Load Pu (kN)	Comments
5.2	I1	69.856	1.326		1925.5	900	231	181	18.9	14.9	0.46	0.087	6.99006	5.3914	0.524255	61.39539	
	I2	69.959	1.339		1925.5	600	230	190	18.9	14.9	0.26	0.041	6.58752	5.96905	0.387703	48.33262	
	I3	69.918	1.378		1925.5	300	232	201	18.9	14.9	0.66	0.024	6.854	6.1616	0.422206	46.05293	
	H2	89.31	1.53		1927	300	219	237	15.7	14.9	0.79	0.05	12.81588	1.3489	0.174682	80.85168	
For main commentary, see Database 5.2.1 Models I heat-treated cold-drawn seamless.									Models I2, I3, H2 failed outside grouted region.								
5.4	J1	70.005	1.429		1630	901	254	179	35.8	13.7	0.2	0.02	6.308992	5.053	0.150867	51.21878	
	J2	69.967	1.419		1630	698	263	187	35.8	13.7	0.124	0.034	4.935456	6.52	0.123386	56.49866	
	J3	69.99	1.42		1630	486	305	174	35.8	13.7	0.153	0.039	5.27989	6.031	0.150854	56.53857	
	J4	69.992	1.435		1630	305	261	174	35.8	13.7	0.113	0.04	4.936104	6.194	0.150825	56.30533	
	J5	69.94	1.403		1630	886	255	177	35.8	13.7	0.187	0.022	8.087366	0.326	0.047976	64.01383	
	J6	69.968	1.407		1630	495	259	181	35.8	13.7	0.038	0.034	7.884515	0.2771	0.102842	66.01109	
	K1	69.977	1.416		2280	890	254	181	35.8	13.7	0.143	0.092	5.142075	9.12	0.418222	45.00908	
	K2	69.968	1.397		2280	496	266	178	35.8	13.7	0.152	0.072	5.554251	8.664	0.329141	42.50724	
	K3	69.965	1.414		2280	1286	266	181	35.8	13.7	0.133	0.048	7.883365	0.171	0.130247	63.74842	
	K4	70.03	1.42		2280	502	261	177	35.8	13.7	0.769	0.079	7.82154	0.5016	0.130359	54.96099	
Commentary as for Reference 5.2, Database 5.2.1 All models heat-treated cold-drawn seamless. Grout included expansive additive, w/c ratio = 0.45.									Grout cylinders and models tested at 13 days. Slip occurred during pre-peak loading. No post-peak response.								
5.5	PF1	46.13	1.63		900	50	527	205	63.8		-	-	4.43	1.26	-	90.68	
	PF2	46.19	1.65		900	50	527	205	75.42		-	-	4.46	1.65	-	91.48	
	PF3	46.15	1.65		900	100	527	205	63.8		-	-	4.42	1.15	-	90.68	
	PF4	46.13	1.65		900	100	527	205	75.42		-	-	4.38	0.79	-	100.14	
	PF5	46.15	1.64		900	150	527	205	63.8		-	-	4.44	0.95	-	96.3	
	PF6	46.13	1.64		900	150	527	205	75.42		-	-	4.42	0.81	-	99.5	
	PF7	46.17	1.65		900	200	527	205	63.8		-	-	4.46	1.59	-	96.3	
	PF8	46.17	1.67		900	200	527	205	75.42		-	-	4.44	0.85	-	100.14	
	PF9	46.11	1.64		900	275	527	205	63.8		-	-	5.06	0.73	-	97.9	
	PF10	46.05	1.61		900	275	527	205	75.42		-	-	4.31	1.22	-	101.1	
	PF11	46.16	1.65		900	400	527	205	63.8		-	-	4.36	0.68	-	97.58	
	PF12	46.15	1.65		900	400	527	205	75.42		-	-	4.91	0.85	-	100.14	
	PF13	46.1	1.6		900	600	527	205	63.8		-	-	4.47	1.54	-	102.72	
	PF14	46.18	1.65		900	600	527	205	75.42		-	-	4.38	1.02	-	101.1	
	PF15	46.15	1.65		900	780	527	205	63.8		-	-	4.45	1.11	-	128.4	
	PF16	46.1	1.61		900	780	527	205	75.42		-	-	4.44	1.11	-	154.06	

Models heat-treated cold-drawn seamless. Yield determined dynamically from 3 coupons from 1 tube.
Grout of Oilwell B cement, w/c ratio = 0.36. 100 mm cubes and models tested at 14 & 28 days.
Cylinder stress-strain curves demonstrate two phases of behaviour.
Coupon stress-strain curves exhibit significant rounding so 0.2% offset value used.
Major discrepancies in presented strain measurements. Quoted modulus values seriously in error.

Damage by 60° indentor, saddle support.
Models statically loaded. Quoted ultimate loads are not steady-state values. Relevant values determined from load-shortening tables.
All models failed at indent.

Database 5.2.5: Partially grouted and damaged and subjected to axial compression

Reference	Model ID	Outer Diam D' (mm)	Thickness t (mm)	Effective length facto K	Length L (mm)	Grout Lgth Lg (mm)	Yield Stress fy (N/mm^2)	Elast. Mod. Es (kN/mm^2)	Grout Str. fcu (N/mm^2)	Elast. Mod. Eg (kN/mm^2)	Initial O-O-R (%)	Initial O-O-S (%)	Dent Depth dd (mm)	Bow do (mm)	Ldg Ecc. e (mm)	Ult. Load Pu (kN)	Comments
5.8	Wimpey 3.3	152.8064	4.7752	0.65	6096	****	253.1034	207	95.86207	43.17241	-	-	-	-	0	621.9702	
	Grigory 5	168.9862	3.937	0.59	5059.68	****	336.5517	207	93.7931	42.2069	-	-	-	-	0	667.35	

Commentary generally as for Database 5.2.1.
Ungrouted length not defined. Void at one end.

Failure at voids.

Database 5.2.6: Partially grouted and undamaged and subjected to axial compression

MSL

IV 6 GROUT FILLED TUBULAR JOINTS

IV 6.1 GENERAL

The following subsections presents a review and appraisal of grouted-filled tubular joint technology. The strength of a tubular joint is enhanced by grout-filling, as it reinforces the chord wall and restricts local bending and section ovalisation. Both static and fatigue endurance may be increased due to the composite action of the chord steel shell and the confined cementitious grout.

IV 6.2 DESCRIPTION

A grout-filled tubular joint is one in which the chord is filled with an unreinforced cementitious material. The chord may be completely filled (grouted joint) or, in the case of a piled leg, the annulus between the tubulars may be filled (double-skin joint). A grouted joint and the associated notation is shown in Figure 6.2.1.

IV 6.3 REVIEW OF EXISTING GUIDANCE

IV 6.3.1 General

The provisions of major design codes on any aspect of grouted joints are summarised below. In almost all cases no guidance is given.

IV 6.3.2 Static Strength

The HSE Guidance Notes[6.1], API RP2A[6.2], NPD[6.3] and DNV[6.4] all state that capacity may be established by testing and/or analytical methods.

IV 6.3.3 Stress Concentration Factors (SCFs)

With the exception of Lloyds[6.5], no specific guidance on the determination of stresses is given. Lloyds recommend the determination of an effective thickness which by itself would give the same moment of inertia as that calculated from treating the chord shell and pile as a composite section, but neglecting grout. Lloyds recommend that the effective thickness calculated on this basis is limited to 1.75T. The calculated thickness can thereafter be used in conjunction with parametric SCF equations developed for simple unreinforced joints. This Lloyds' recommendation is intended to cover double-skin joints only; no guidance is provided for grouted joints which, within the context of strengthening and repair of existing installations, is the popular option.

All codes state that stresses may be established by testing and/or analytical methods. DNV adds a caution as follows:-

$\alpha = 2L/D$

$\beta = d/D$

$\gamma = D/2T$

$\tau = t/T$

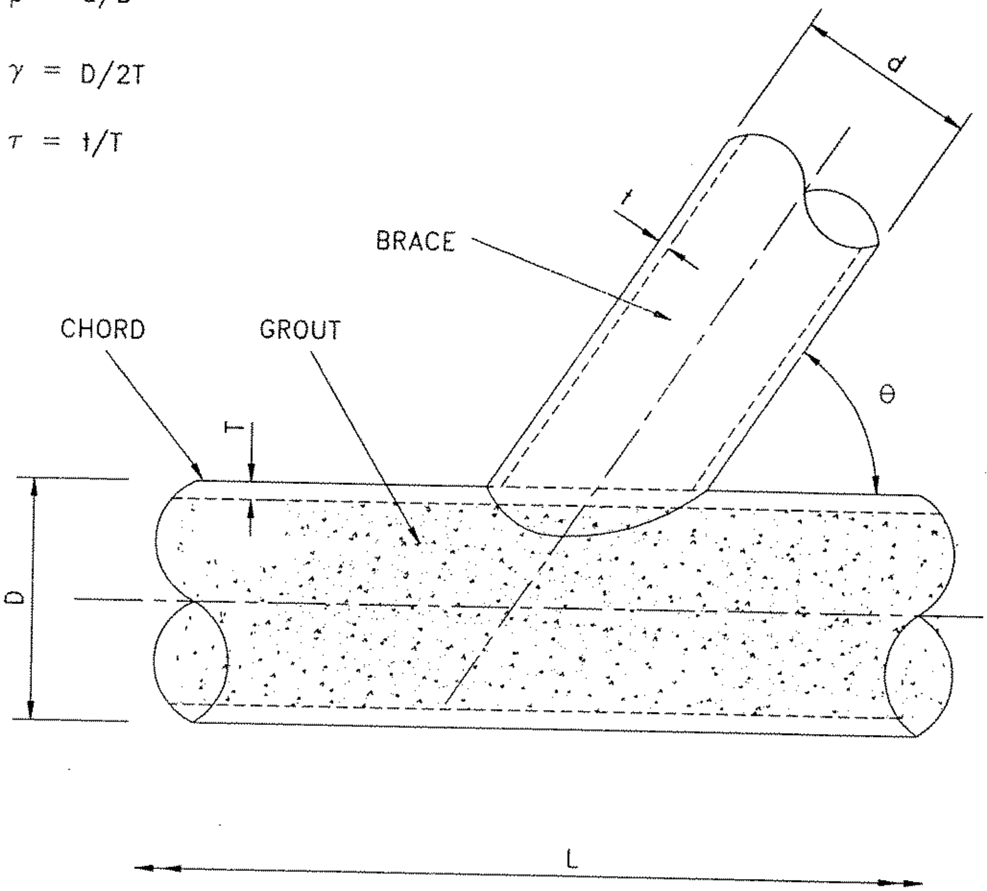

Figure 6.2.1: **Typical chord grout-filled joint and notation**

"In the case of grout-reinforced joints and joints with doubler plates, proper account is to be taken of the dependence of SCF's on load and load history. Finite-element analysis is to take account of all relevant non-linearities, including the steel to grout interface, as relevant."

IV 6.3.4 Fatigue

No specific guidance is given for grouted joints.

IV 6.3.5 Other Issues

The HSE Guidance Notes specifically allows the use of grout filling for joints on existing installations. The Guidance Notes states:

"Filling of members with grout to improve the strength of the member or joint may be adopted. When grouting is undertaken, the effects on the member or joint stiffness characteristics should be allowed for in the overall structure re-analysis."

"On certain structures the annulus between the jacket and pile may be filled with grout to enhance the joint or leg member strength. Stresses before and after grouting should be considered for both pile and leg. The possible re-distribution of foundation loads should also be considered."

IV 6.4 REVIEW OF OTHER PUBLISHED INFORMATION

IV 6.4.1 General

The following principal research and development programmes have been carried out to date to investigate the behaviour of grouted joints:-

(i) EEC Composite Jacket Project. The results from this work are confidential, although general trends noted from the findings are described in Reference 6.6. A series of elastic, ultimate strength and fatigue tests on either double-skin or fully grouted joints have been carried out. It is understood, however, that a significant portion of the experimental programme concentrated on thin chord sections, giving γ ratios between approximately 50 and 100, well in excess of the γ ratios for joints in existing offshore installations.

(ii) Veritec 'Double Skin Grout Reinforced Joints'. This joint industry project addressed the elastic, ultimate strength and fatigue response of double skin grouted joints through both experimental and numerical means[6.7]. The original reports have kindly been released to this project by Mobil.

(iii) BP/ADMA tests. This test programme examined the elastic and ultimate strength behaviour of double skin grouted joints of T configurations, where the inside of the pile was also grouted. Reference 6.8 presents the findings from this investigation.

(iv) Amoco tests. The results from this work are reported in Reference 6.9. A number of elastic tests and an ultimate load test on a grouted non-overlapping K joint were carried out.

(v) Occidental tests. Original test reports released by Occidental to the JIRRP project[6.10] in the early 1980s have been sourced[6.11, 6.12]. The results from a series of tests on T and DT joints are reported.

(vi) Reference 6.13 presents an interpretive discussion of the behaviour of double-skin grouted K joints, using data and information generated from finite element analyses[6.14], elastic tests[6.15] and a fatigue test[6.16].

(vii) The results from a series of axial fatigue tests on grouted YT joints are reported in Reference 6.17.

(viii) Elastic tests on two grouted T joints have been carried out for HSE and EE Caledonia, and the results are presented in Reference 6.18. Further information regarding these tests have kindly been made available by HSE.

(ix) Reference 6.19 describes the use of three-dimensional isoparametric elements in the finite element analysis of a K joint.

(x) The fatigue analysis of grouted joints on Shell Oil's Cognac platform is addressed in Reference 6.20.

It becomes clear from study of the available literature that much of the work reported in the public domain is of an ad hoc nature and addresses technology specific to an identified problem. There is a paucity of data and information to generate comprehensive design methods in this area to cover all cases which are likely to occur in practice. This situation will prevail even if the confidential data from the EEC composite jacket project are added to the public domain knowledge base which currently exists, for the reason noted above. In recognition of this status, a number of operators have commissioned joint tests specific to their needs (eg. BP[6.8], Amoco[6.9] and Occidental[6.11, 6.12]), thereby generating back-up information and data to justify the use of chord grout-filling as a viable technical option. In other cases, estimates of the reductions in SCFs or increases in capacity for grouted joints have been made using approximate methods based on engineering statics, assumptions and the available public domain information. Each of the above referenced documents are reviewed below.

IV 6.4.2 EEC Composite Jacket Project[6.6]

The paper reports on a testing programme conducted at Wimpey Laboratories on composite joints. The paper contains a set of five graphs produced in the preliminary stages of this programme. These are shown in Figures 6.4.1 to 6.4.5. Four of the five figures refer to grouted joints and the fifth refers to a double-skin joint. The predicted as-welded (ie. ungrouted) strengths in the paper relate to the recommendations contained in the UEG design guide on tubular joints[6.21]. A considerable enhancement in strength above the predicted as-welded strengths is evident.

The paper concludes that:

a. Significant improvements in strength and substantial reductions of SCFs can be achieved through chord grout-filling, with reduction factors (RFs) of the order of 0.45, 0.18, 0.90 and 0.35 for tension, compression, IPB and OPB loading, respectively. (Note that RF is defined as the ratio of grouted joint SCF to ungrouted joint SCF, at the hot spot location.)

b. Up to 20-fold increases in fatigue life have been observed in tests.

c. Grouting of previously ungrouted piles within jacket legs, or other chord members within a structure, will lead to an improved performance and an increased fatigue life.

d. The effects of local joint flexibility can be significant, and should be included in a reappraisal of jacket structures. If grouting is being considered, reduced local joint flexibility should be accounted for in the analysis.

IV 6.4.3 Veritec Double Skin Grout Reinforced Joints Project[6.7]

This project concentrated on double-skin joints only; grouted joints were not considered. Seventeen specimens were fabricated, namely nine X joints, four T joints, two Y joints and two K joints. All specimens were subjected to axial load SCF tests in the as-welded and double-skin conditions. Three X joints and one K joint were subsequently subjected to axial fatigue loads. Further, four specimens (one X joint and three T joints) were subjected to static tension loads to failure.

Although grouted joints were not covered in this programme, the double-skin results reported are considered important within the context of grouted joint behaviour and, therefore, these results are reported and assessed in detail in a later section.

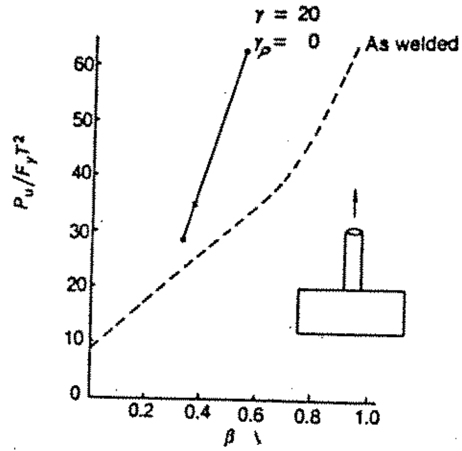

Figure 6.4.1: **Grouted joint under axial tension: Ultimate strength results given in Reference 6.6**

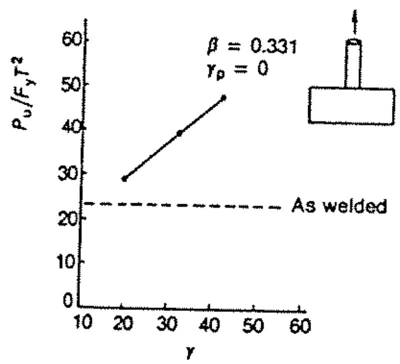

Figure 6.4.2: **Grouted joint under axial tension: Ultimate strength results given in Reference 6.6**

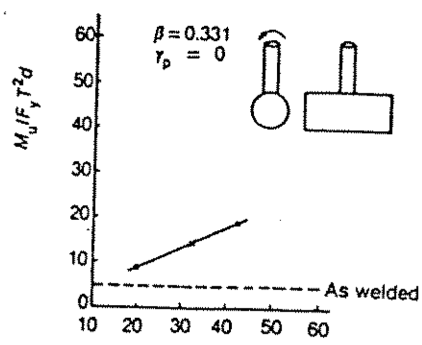

Figure 6.4.3: **Grouted joint under out-of-plane moment: Ultimate strength results given in Reference 6.6**

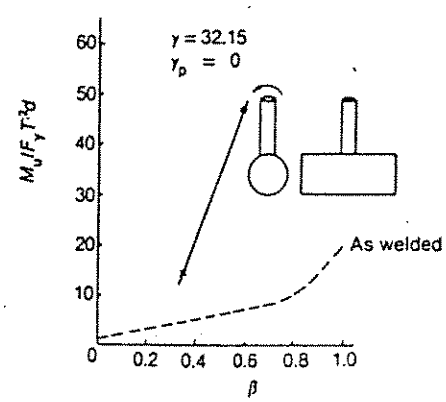

Figure 6.4.4: **Grouted joint under out-of-plane moment: Ultimate strength results given in Reference 6.6**

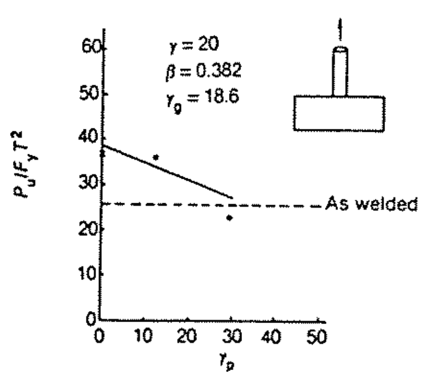

Figure 6.4.5: **Double-skin joint under axial tension: Ultimate results given in Reference 6.6**

IV 6.4.4 BP/ADMA Tests[6.8]

This paper summarises the results of ten T joint tests, in which the chords of five specimens were reinforced with a grouted pile, ie. grout was placed in the annulus and in the pile.

The joint dimensions of the specimens are as follows:

$D = 508mm$	T not given	$d = 168mm$
t not given	$D_p = 318mm$	T_p not given

Unfortunately, no thickness values for the joints are given in the paper, and full interpretation is therefore not possible. One tension, two compression and two in-plane moment ultimate load tests were carried out. SCFs and stress distributions around the periphery of the intersection were also measured. It is concluded in the paper that:

a. For fully grouted joints of the geometry considered, a punching shear failure is not possible if the brace stresses are wholly compressive.

b. Grouting the chord member of tubular joints reduces the strain levels around the joint resulting in a significant reduction in maximum stresses. This can be expected to result in an improved fatigue performance of such joints over that of ungrouted joints, with possible enhancements in design fatigue lives of 20 to 750 fold.

The paper recommends a design approach for static loads. Two separate methods for compressive and tensile stresses are recommended as follows:

a. Compressive brace stresses only

   For this case the paper suggests that punching shear need not be considered for a 'fully-grouted' joint. The design of the joint will be controlled either by the permissible brace stresses or by the fatigue performance.

b. Partially or wholly tensile brace stresses

   For these cases the paper suggests that the joint should be designed against 'punching shear'. For joints having the same geometry as the specimens tested, the paper recommends that a 60% enhancement be applied for design purposes.

IV 6.4.5 Amoco Tests[6.9]

This paper reports on a series of elastic tests (axial and bending) and an ultimate balanced axial load test on a non-overlapping grouted K joint. The information on the results provided in this paper is complete and, therefore, the results are extracted and appraised in detail in a later section.

IV 6.4.6 Occidental Tests[6.11, 6.12]

These papers report on a series of elastic tests (axial and bending) on a grouted T joint. The information contained in the papers is complete and, therefore, the results are extracted and appraised in detail in a later section.

IV 6.4.7 Marshall[6.13]

This document reviews SCF formulations for simple, steel reinforced and double-skin/grouted joints. Marshall proposes that the γ value in SCF equations can be modified to take account of the additional stiffness for grouted joints. The effective thickness is defined as:

$$T_e = ((T^3 + T_p^3)/T)^{1/2} \qquad \text{.... 6.4.1}$$

The effective thickness is limited to 2T, compared with the Lloyds' limitation of 1.75T.

IV 6.4.8 Bouwkamp et al[6.17]

This paper reports on a series of fatigue tests carried out on thin-walled YT joints, where the chord member is filled with a cement grout. The tests were carried out in air under cyclic loading conditions. The object of the tests was to assess the fatigue life of this type of joint under complete axial load reversal (ie. R = -1).

The joint dimensions are given as follows:

Brace No.	θ	γ	β	ζ
1	90°	19.32	0.436	0.568
2	45°	19.32	0.436	0.568

After initial plastic deformation of the joint, the test specimens were observed to act in an essentially linear manner. The nature of this linear behaviour is shown to be different for tensile and compressive load cases. For tensile loads, the load transfer is concentrated towards the saddle point of the connecting weld. For compressive loads the transfer of forces is more uniform.

Bouwkamp explains that under compressive loading conditions there is an effective load transfer between the chord wall and the grout annulus. Under tensile loading conditions the main function of the grouted annulus is to prevent the deformation of the chord wall, which results in a lesser load being carried by the grout.

The report concludes that the cement grout fill significantly increases the overall fatigue life when compared with ungrouted joints. Typically, a fatigue life of 1200 cycles as opposed to 60 cycles is reported, which represents a 20-fold increase in fatigue life.

IV 6.4.9 HSE/EE Caledonia Tests[6.18]

This paper reports on a series of elastic tests (axial and bending) and fatigue tests on two grouted T joints. The information contained in this paper, and the additional information made available by HSE, is complete and, therefore, the results are extracted and appraised in detail in a later section.

IV 6.4.10 Liew et al[6.19]

This paper describes the use of three-dimensional isoparametric elements in the analysis of complex joints, taking a double-skin K joint as an example. A description of these elements is first given, and the advantage over two-dimensional plate elements is stated. Liew suggests that a 3D isoparametric element can be described as one in which node points are defined in both the inner and outer surfaces of a shell, as shown in Figure 6.4.6. The paper continues with a discussion of the general problems concerning finite elements, and discusses their use in the analysis of complex joints.

IV 6.4.11 Kinra et al[6.20]

This paper deals with the fatigue analysis of the Cognac platform. A brief discussion of the effect of grout on fatigue lives of tubular joints is presented. Two separate analyses were undertaken to assess the lives with grout in the bonded and disbonded cases. These were carried out using techniques described in Reference 6.19 reviewed above. It is shown that the ratios of fatigue lives with and without effective grout range from 1.01 to 1.72.

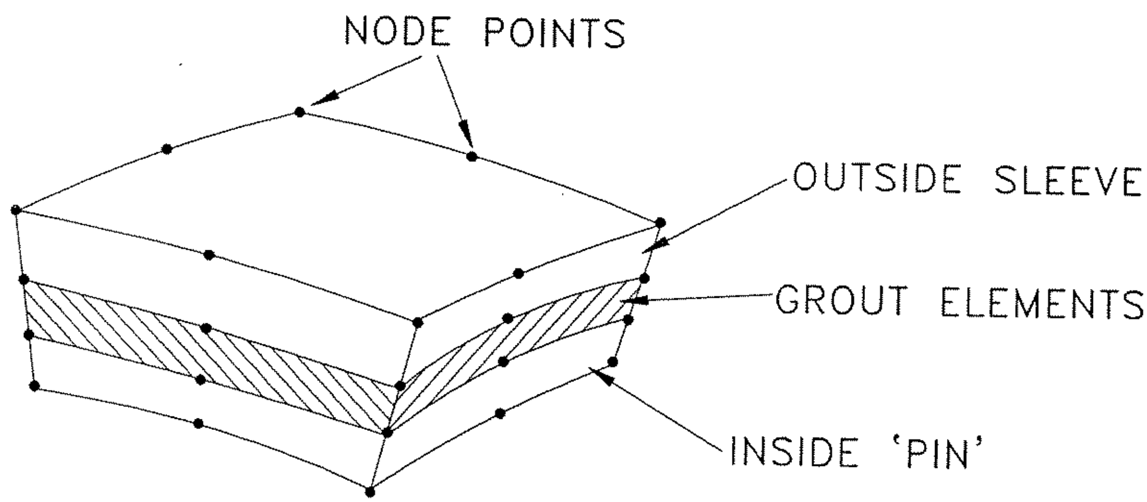

Figure 6.4.6: **Elements for three-dimensional modelling of double-skin joints as proposed in Reference 6.19**

IV 6.5 REVIEW OF AVAILABLE TEST DATA

IV 6.5.1 General

The following sections review available data on grouted and double-skin joints. Filling the tubular chord (and/or brace) members with a cementitious material results in the efficient use of materials in applications most suited to their mechanical properties. The cementitious material is contained and therefore greater strength and ductility is achieved. The steel tubular is the containment medium and is, therefore, predominantly subjected to hoop loads, and the cementitious filling minimises any tendency for buckling of the steel shell. Grouting technology is well proven and offshore grouting works can be executed with confidence.

A number of technical benefits can be demonstrated through grout-filling of tubular joint chords, viz:

- The presence of the grout increases the radial stiffness of the chord member. The grout restricts local chord wall deformations, which leads to a reduction of deformation-induced bending stresses and associated SCFs.
- Any reduction in SCF implies an enhancement in fatigue life.
- The chord member bending stiffness is increased, resulting in a reduction of stress at crown locations which are driven by the α ratio. The increased chord bending stiffness also implies that the capacity of large β ratio, grouted T/Y joints, subjected to axial loads, may not be limited by chord failure in the beam-bending sense.
- The grout severely restricts ovalisation of the chord cross-section, which indicates an increase in the capacity of grouted joints when compared with corresponding ungrouted cases.

IV 6.5.2 Failure Modes

Grouted joints can fail in a number of modes depending on load type, infill material type and joint configuration. Failure modes under static and fatigue loads are discussed below.

(a) Static loads

Typical load-deformation curves for axially loaded grouted and ungrouted specimens are given in Figure 6.5.1[6.21]. The increase in strength under compression loading and tension loading is governed by the load transfer mechanism. Compression loaded joints show a very large strength enhancement due to load transfer in bearing to the grout through the chord wall at the brace/chord intersection. For some compression loaded T joints an overall bending failure of the grouted chord may occur.

For tension loaded joints a smaller enhancement in strength is obtained, as shown in Figure 6.5.1. Here the grout acts as a filler, preventing ovalisation of the chord. The increase in strength is again due to load transfer, in bearing, to the grout. However, this transference of load does not occur under the brace member. Failure usually occurs when local deformation of the chord wall under the loaded brace takes place, leading to shear failure of the chord shell around the brace periphery. Examination of failed specimens shows that a gap between the chord and the grout has usually developed.

For moment loaded joints the failure mechanism is modified from that of the simple joint type. Inward deformation of the chord wall is prevented and the centre of rotation at failure is displaced by half the brace diameter from the position expected for ungrouted joints. The mechanism of failure is therefore that of yield followed by extensive cracking of the chord wall, propagating from the tension side of the joint. For double-skin joints, ovalisation failure is possible.

(b) Fatigue loads

Fatigue-induced failures occur in the form of cracks in the steelwork which initiate and propagate from 'hot-spot' locations.

IV 6.5.3 Static Strength

IV 6.5.3.1 Database

Few published data exist on the static strength of grouted or double-skin joints. Five documented test results on intact joints are available. The geometric parameters and measured strengths are presented as Database 6.5.1. The screening criteria adopted in the preparation of the database are as follows:

- The minimum chord diameter is 291.2mm; there has, therefore, been no need to impose a size restriction as API RP2A and the HSE Guidance Notes adopt 140mm and 125mm, respectively, as lower limits to restrict size/scale effects.
- All results reported relate to measured yield strength values.
- All test results have been sourced from original reports.
- Test results where insufficient information is presented have been rejected. The data rejected are described in Section IV 6.4.

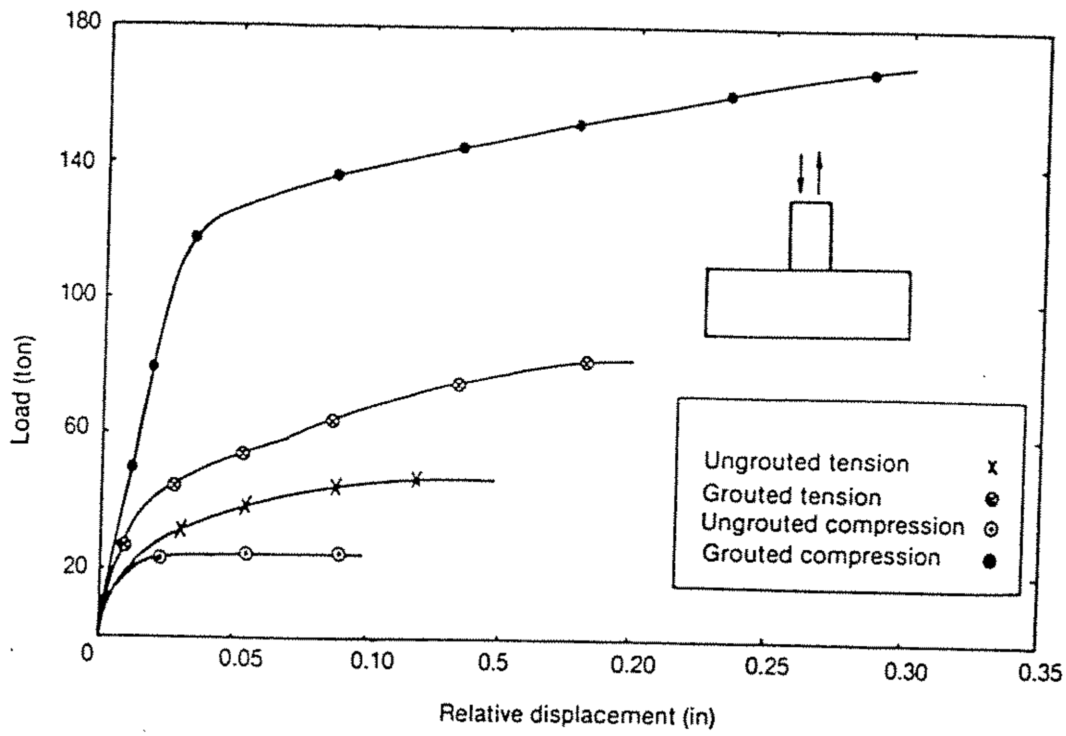

Figure 6.5.1: **Typical load-deformation curves for axially loaded grouted joints given in Reference 6.21**

- All tests results have been generated using adequate testing procedures.

The only results rejected on the basis of the above relate to data for which insufficient information is available, eg. τ, T not given. Nevertheless, these rejected data have been reviewed in detail in Section IV 6.4.

IV 6.5.3.2 Grouted joints

One screened test result is available. This test result concerns a grouted K joint subjected to balanced axial load (see Database 6.5.1). Failure was through local buckling of the compression brace near the loaded end; joint failure did not occur. In the following sub-sections, a new approach to grouted joint design is developed, for different joint types and load cases, based on the principles of engineering mechanics, and on the evidence of behaviour/failure characteristics described in the literature (Section IV 6.4 refers, together with associated References).

Axial Tension

T/Y Joints

No screened data are available. However, examination of the failure modes (Section IV 6.5.2) and the observations cited in References 6.6 and 6.8 (Section IV 6.4) indicates that failure is associated with a shear pull-out of the chord plug within the brace footprint. This is not unexpected, as the grout prevents chord ovalisation and, therefore, collapse of the chord is not possible under these conditions.

Given the above observations, it is not unreasonable to take the capacity of grouted T/Y joints subjected to tension loads to be equivalent to the load required to cause shear pull-out of the chord plug. On this basis, failure of the joint occurs when the acting stress on the chord from the brace load equals the shear yield for the chord, ie.

$$\frac{P_u \sin\theta}{\text{chord plug shear area}} = \frac{F_y}{\sqrt{3}} \qquad \text{.... 6.5.1}$$

The chord plug shear area is given by:

$$\pi\, d\, K'_a\, T \qquad \text{.... 6.5.2}$$

where K'_a is the exact intersection length factor.

Combining the above and rearranging gives the following equation:

$$\frac{P_u \sin\theta}{F_y T^2} = 3.6\, K'_a\, \beta\, \gamma \qquad \text{.... 6.5.3}$$

The exact intersection length factor can conservatively be replaced by the approximate length factor defined in the HSE Guidance Notes, ie.

$$\frac{P_u \sin\theta}{F_y T^2} = 3.6\, K_a\, \beta\, \gamma \qquad \text{.... 6.5.4}$$

where $K_a = (1 + 1/\sin\theta)/2$

Equation 6.5.4 can be considered to represent the mean strength of tension loaded T/Y joints assuming that adequate brace capacity is available. In order to establish a 'characteristic' or 'lower bound' strength, and in light of the lack of relevant data, it appears not unreasonable to assume that the relationship between characteristic and mean strength is the same as that observed for ungrouted T/Y joints subjected to tension loads, as the failure mode in both cases (ungrouted and grouted) involves fracture of the chord wall under shear loads. In Reference 6.22, which presents the background to the current HSE Guidance Notes recommendations on tubular joints, the relation for T/Y joints subjected to tension loads is:

characteristic strength = 0.69 x mean strength 6.5.5

Therefore, the 'characteristic' strength for grouted T/Y joints subjected to tension loads can be described as follows:

$$\frac{P_k \sin\theta}{F_y T^2} = 2.5\, K_a\, \beta\, \gamma \qquad \text{.... 6.5.6}$$

DT/X Joints

No data are available for this case. However, in light of the full constraint to ovalisation that is afforded by the grout, Equation 6.5.6 would be expected to be equally applicable to DT/X joints. This is likely to hold true for all β ratios, although some kind of interaction would be expected at $\beta = 1.0$. Note that the 0.69 factor used for T/Y joints is conservatively carried through for DT/X joints.

Axial Compression

No data are available for T/Y and DT/X joints. The K joint result contained in Database 6.5.1 reflects failure of the compression brace with no apparent distress of the joint. The observations cited in Section IV 6.4, and associated references, indicate that joint failure does not occur. This observation is not unexpected as the grout, under confined triaxial stress conditions, exhibits substantial load carrying capability. The maximum sustainable load for the joint would therefore be expected to be limited by brace capacity. Failure would be expected to involve compression brace buckling, perhaps local to the intersection and accompanied by 'elephant foot' type deformations; this assumes that, for K joints, both braces have the same dimensions and similar material yield stress properties. In a more general sense, the two intersections for K joints should be treated separately, with the compression brace intersection capacity restricted to brace capacity and the tension brace intersection capacity restricted to the lesser of either the shear pull-out of the chord plug (Equation 6.5.6) or the tension brace capacity.

In-plane Moment

T/Y Joints

No data are available for in-plane moment loaded grouted joints. In light of the observed failure mechanisms for this load case (Sections IV 6.4 and IV 6.5.2), the capacity would be expected to be a function of a proportion of the shear capacity of the intersection acting over a certain lever arm. Under in-plane moments loads, the failure mode for grouted joints involves cracking and fracture on the tension side of the cross section, together with local brace buckling on the compression side. Note that buckling of the chord wall on the compression side does not occur due to the restraint provided by the grout.

On the basis that one half of the shear capacity of the intersection is available acting over a lever arm of $d/\sin\theta$, and (conservatively) assuming no increase in intersection length for $< 90°$ joints (ie. $K'_a = 1.0$), the following capacity equation has been derived:

$$\frac{M_u \sin\theta}{0.5\pi d^2 T} = \frac{F_y}{\sqrt{3}} \qquad \ldots\ 6.5.7$$

Rearranging Equation 6.5.7 gives the following non-dimensional relationship:

$$\frac{M_u \sin\theta}{F_y T^2 d} = 1.8\,\beta\,\gamma \qquad \ldots\ 6.5.8$$

Adopting the same basis for 'characteristic' strength definition as tension loaded joints,

$$\text{characteristic strength} = 0.82 \times \text{mean strength} \qquad \ldots\ 6.5.9$$

The 0.82 factor is extracted from Reference 6.22. Therefore, the 'characteristic' strength of in-plane moment loaded T/Y joints can be described as follows:

$$\frac{M_k \sin\theta}{F_y T^2 d} = 1.5\,\beta\,\gamma \qquad \text{.... 6.5.10}$$

DT/X Joints

No data are available for DT/X joints. Examination of the failure modes for ungrouted DT/X joints[6.22] reveals that the behaviour of T/Y joints and DT/X joints subjected to IPB loads is similar. Failure is concentrated at the crown locations, and little interaction would be expected between the failure mechanisms at crown positions on either side of the chord for DT/X joints. It is therefore reasonable to expect that the IPB capacity of grouted DT/X joints can be estimated using Equation 6.5.10 which relates to T/Y joints.

K/YT Joints

No data are available for this case. In light of the observations noted in References 6.1, 6.2 and 6.22 for ungrouted joints (ie. the IPB capacity for K/YT joints is the same as T/Y joints), the capacity of in-plane moment loaded, grouted, K/YT joints would be expected to be adequately predicted using Equation 6.5.10 which relates to T/Y joints. However, it should be noted that some interaction of stress fields for small-gap K joints would be expected, even for ungrouted K joints, although none of the present day codes allow for this interaction due to lack of substantiated evidence.

Out-of-Plane Moment

No data are available for out-of-plane moment loaded grouted joints. Examination of the available ungrouted joint guidance[6.1, 6.2] suggests that IPB strength is greater than OPB strength for the same geometry. This observation is within expectations, and is a result of the lower resistance available at chord saddle locations for OPB due to the lower stiffness. Under grouted conditions, the saddle location stiffnesses are substantially increased, with the likely result that the OPB grouted strength can be estimated on the same basis as developed for IPB grouted strength. The lever arm appropriate for OPB is simply d. For Y joints, the brace OPB (M) is resolved at the joint to M$\sin\theta$ and a torsional component. This then leads to the following equation for estimating the OPB capacity:

$$\frac{M_k \sin\theta}{F_y T^2 d} = 1.5\,\beta\,\gamma \qquad \text{.... 6.5.11}$$

Note, however, that for large β ratios, Equation 6.5.11 is likely to prove conservative because the moment is transferred by chord membrane forces. It may thus prove possible to increase the OPB quoted capacity given by Equation 6.5.11 for large β; it might be proposed that this could be catered for by multiplying the right hand side by the function Q_β. Another joint industry project is currently underway by MSL Engineering to investigate this possibility.

Equation 6.5.11 can be considered to be applicable for T/Y and K/YT grouted joints. Observations for ungrouted DT/X joints[6.22] indicate significant stress field interaction at saddle locations for DT/X joints, to the extent that capacity is downgraded by up to 34% at $\beta = 1.0$. Assuming, conservatively, that the same extent of detrimental interaction occurs for grouted DT/X joints, the following formulation can be postulated to cover these joint configurations:

$$\frac{M_k \sin\theta}{F_y T^2 d} = 1.5\,\beta\,\gamma\,/\,\sqrt{Q_\beta} \qquad \text{.... 6.5.12}$$

IV 6.5.3.3 <u>Double-skin joints</u>

Four screened test results are available (3 T joints and 1 X joint, see Database 6.5.1). All these joints have been subjected to axial tension loads, and failure occurred either through brace yielding or rupture of the brace at the intersection. From this standpoint, the joint failure mechanisms exhibited are identical to those noted for grouted joints. However, as mentioned in Section IV 6.5.2, double-skin joints may fail by chord ovalisation (ie. chord collapse), depending on the stiffness of the inner pile. The strength of a double-skin joint may therefore be limited by:

- brace yield failure
- brace rupture failure
- chord bending failure
- chord ovalisation failure.

The first three limitations are common for both grouted and double-skin joints. A resistance model is developed below, to enable the potential for chord ovalisation failure to be assessed.

For ungrouted joints, ovalisation failure is the most common form of failure, as the chord deforms under increasing brace loads. Given the lack of data on the strength of double-skin joints, an approach based on equivalent thickness is developed herein, to enable the present-day practice for the design of ungrouted joints to be extrapolated to cover double-skin joints. This is the only option available, pending the generation of suitable data on double-skin joints in the future.

Ovalisation failure occurs as the chord deforms under applied brace loads. The chord deforms in the radial direction, as chord circumferential bending stresses are generated and increased. Roark[6.23] formulations are used to develop the following equation for T_e, the effective thickness for double-skin joints:

$$T_e = ((T^3 + T_p^3)/T)^{1/2} \qquad \text{.... 6.5.13}$$

Equation 6.5.13 has been developed on the basis of calculating an equivalent thickness which gives the same local bending strength as the steel-grout-steel sandwich. However, the grout is assumed to act only as a filler, and no composite action is allowed to develop. A simplified form of Equation 6.5.13 can be obtained as follows:-

$$T_e = (T^2 + T_p^2)^{1/2} \qquad \text{.... 6.5.14}$$

Equation 6.5.14 has been used by Veritec in their work on double-skin joints[6.7].

Table 6.5.1 presents a comparison of the available data with various predictive models.

Specimen No.	Joint Type	Chord F_y (N/mm^2)	Measured Strength (kN)	Brace Yield Capacity* (kN)	Ovalisation Capacity** (kN)		Rupture Capacity*** (kN)
					(a)	(b)	
PP1 - 5	T	397	2109	2654	5236	4405	2157
PP2 - X3	X	368	981	1380	1493	1279	1000
PP2 - T1	T	352	618	524	1029	939	362
PP2 - T3	T	377	2394	2653	1534	1652	1923

(i) * Calculated as π d t (F_y of brace)

(ii) ** (a) = using Equation 6.5.13 and HSE Guidance Notes mean[6.22]
(b) = using Equation 6.5.14 and HSE Guidance Notes mean[6.22]

(iii) *** Using Equation 6.5.4.

Table 6.5.1: Comparison of measured and predicted strengths for double-skin joints

The following comments are worthy of note from an examination of Table 6.5.1:

- The limiting capacities all reflect best estimates, and can be considered to represent 'mean' strength.
- The simpler T_e formulation (Equation 6.5.14) performs adequately when compared with Equation 6.5.13. Typically, Equation 6.5.14 will give a lower capacity when compared with Equation 6.5.13, provided $T < T_p$.
- A comparison of the minimum predicted capacity with measured values reveals the following:

Specimen No.	Measured Strength (kN)	Predicted Strength (kN)	Ratio of Measured/Predicted	Limiting Criteria
PP1 - 5	2109	2157	0.98	Rupture
PP2 - X3	981	1000	0.98	Rupture
PP2 - T1	618	362	1.71	Rupture
PP2 - T3	2394	1652	1.45	Ovalisation*
* Using Equation 6.5.14				

- The ratios above indicate that the approach described herein for double-skin joints give predictions of capacity equal to or less than measured values.

IV 6.5.4 Stress Concentration Factors

IV 6.5.4.1 Database

A number of SCF results exist for grouted and double-skin joints. Four grouted joint specimens (1 K joint, 1 DT joint and 2 T joints) have been subjected to elastic SCF tests[6.9, 6.11, 6.12, 6.18]. For double-skin joints, Reference 6.7 contains a number of axial SCFs measured on T, DT and K joints.

Database 6.5.2 contains the collated hot spot SCF data. Axial SCF RFs relate to tension loading. The screening criteria adopted in the preparation of the database are as follows:

- Test results where insufficient information is presented have been rejected. The data rejected are described in Section IV 6.4.

- All test results reported in Database 6.5.2 have been generated using adequate procedures.

- A geometric limitation has not been imposed, as the smallest chord diameter is 289.5mm.

- All results relate to steel model specimens.

The results rejected on the basis of the above criteria concern data where insufficient information is reported, ie. missing τ, T values.

IV 6.5.4.2 General trends

A study of all available information has been carried out, and the following comments apply to the general trends identified in respect of the effects of chord grout-filling on stress distributions and SCFs:-

- Substantial reductions in SCFs have been reported in the literature.

- Typically, the higher the SCFs at the saddle location for joints in the as-welded (ie. non grouted) condition, the larger the reduction (in absolute terms) through chord grout-filling. This is in line with expectations since the high SCFs in the as-welded condition are generally the result of local chord wall bending of thin-walled chords (high γ ratios).

- The reductions in SCF, if any, depend on the joint configuration, geometry and load case. It proves instructive at this stage to examine the SCF distributions associated with typical simple as-welded joints:-

  - The SCF distributions are different for different joint configurations, even if the same load-case and nominally identical joint geometry conditions are considered. These differences are a result of the manner in which the chord resists the in-coming brace loads.

  - The dominant parameters which govern the SCF of a given joint are α ratio, β ratio, γ ratio, ζ ratio, τ ratio and θ.

  - The 'hot spot' SCF (ie. highest SCF) is located at or near the saddle locations for axial and out-of-plane moment loaded T/Y and DT/X joints, and at or near the crown locations for the same joints subjected to in-plane moment loads. One possible exception to this behaviour concerns the SCF at crown locations for axially loaded T/Y joints; the chord length effect (α ratio dominance) can be pronounced due to the overall beam bending of the chord. It can be shown that the highest SCF occurs at these locations if the α ratio (chosen on the basis of distance between chord end restraints or points of contraflexure) is sufficiently high.

  - For 'hot-spot' SCFs located at saddle positions (ie. axial or OPB loaded T/Y or DT/X joints), the following comments apply:-

    a) The effect of α ratio is not significant.

    b) SCFs increase as the β ratio increases, up to an approximate value of $\beta = 0.7$, beyond which the SCFs decrease. At high β ratios, the joint stiffness is significantly increased, and load transfer mechanisms local to the saddle positions begin to shift from chord wall bending to chord wall membrane action.

    c) SCFs increase with increasing γ ratios and joint stiffness is significantly decreased.

d) SCFs increase with increasing τ ratios. However, note that the τ ratio and γ ratio effects are interlinked by virtue of the non-dimensionalisation using the chord wall thickness term T.

e) SCFs decrease with decreasing θ angles, as the brace loads are increasingly transmitted through weld shear in a direction parallel to the chord longitudinal axis. This correctly implies a β ratio dependency, which is reflected in the parametric equations contained in the Appendix in Part III of this document.

- For 'hot-spot' SCFs located at crown positions, (ie. IPB loaded T/Y or DT/X joints), the following comments apply:-

a) The effect of α ratio is not significant, in the sense that increasing α ratio does not result in a significant increase in SCF.

b) Little change in SCFs can be observed with increase in β ratios.

c) The above comments in relation to γ and τ ratio effects apply equally in these cases.

- In general terms, the 'hot-spot' SCFs due to in-plane moment loads are appreciably lower than those associated with brace axial or out-of-plane loads, for T/Y or DT/X joints. This is a result of differences in the longitudinal and radial stiffness at the crown and saddle locations, respectively.

- In light of the characteristics noted above for as-welded joints, and on the basis of engineering mechanics and trends described in the literature, the following general trends for grouted joints would be expected:-

  - SCFs for grouted joints follow the same response trends noted above for as-welded joints, with a number of exceptions.

  - For axially loaded T/Y joints, the presence of grout increases the bending stiffness of the chord member and, hence, the influence of α ratio on chord crown SCFs would be expected to diminish.

  - For axially or OPB loaded joints, where the 'hot-spot' SCFs are located at or near the saddle positions, the reduction in SCFs from chord grout-filling, in relative percentage terms, would be expected to be less at high β ratios. For $\beta = 1.0$ joints in particular, the brace loads are transmitted through the chord by predominant membrane action, with little local wall bending. As the grout restricts local chord wall deformations, the benefit through grout-filling is greater for joints where through wall bending is the dominant load transfer mechanism. A similar characteristic would be expected at low γ ratios, where load transfer is achieved increasingly by shear through the chord wall rather than local bending. For specific conditions of 'stiff' as-welded joints, ie. joints with $\beta = 1.0$ and low γ ratios (say $\gamma \leq 12$), it may be expected that the benefit from chord grout-filling is minimal.

- Typically, for IPB loaded joints, where 'hot-spot' SCFs are located at or near the crown locations, the reduction in SCFs through chord grout-filling is expected to be significantly lower than for the equivalent load-cases of axial or OPB, due to the differential stiffness of the chord crown and saddle locations.

- A more uniform distribution of stresses around the joint periphery would be expected for grouted joints. In some cases, and usually at non-critical locations at the intersection, increases in stresses over the as-welded condition may occur, by virtue of the more uniform redistribution.

- Given the increase in the radial stiffness due to the presence of grout, a lesser sensitivity of SCFs to variations in the angle θ would be expected.

It follows from the above observations and comments, and the restricted database which is available, that an approximate method for estimating grouted joint SCFs, based on suitably modified existing parametric formulations for as-welded joints, may be appropriate. In this manner,

- the effects of joint type, geometry and load-case can be accommodated, on the basis of joint response for as-welded connections,

- specific modifications can be introduced to reflect the expected response characteristics for grouted and double-skin joints, where these are considered to be significantly different from the behaviour of as-welded joints.

The presence of grout has the single fundamental effect of constraining ovalisation of the chord cross-section. This effect is similar to the increase in radial stiffness that results as the γ ratio of the chord reduces, ie. as the chord wall thickness is increased. In Section IV 6.5.4.3 below, the basis for calculation of an effective chord wall thickness (and hence effective γ ratio) to account for the presence of grout is presented.

IV 6.5.4.3 Development of effective thickness model

Consider a tubular chord member, with grout in-fill. The grout core has an outside diameter D_g, and a thickness $D_g/2$. The grout core can be reduced to an equivalent steel tubular, of Diameter D_g, and of thickness T_{gs}. The formulation for T_{gs} is determined on the basis of radial stiffness considerations, ie. the value for T_{gs} which ensures that the radial stiffness of the grout core and the equivalent steel tubular are the same.

The radial stiffness of the grout core is obtained using thick-walled theory (Table 32, Case 1c, Roark[6.23]). The change in radius under uniform external radial pressure q is given by:

$$- 0.4q\ (D-2T)/E_g \qquad \text{.... 6.5.15}$$

For an equivalent steel tubular, using thin-wall theory (Table 28, Case 1b, Roark[6.23]), the change in radius under uniform external radial pressure q is given by:

$$- q\ (D-2T)^2/4\ E_s\ T_{gs} \qquad \text{.... 6.5.16}$$

where T_{gs} = equivalent steel thickness

For equal radial displacement, and hence equal radial stiffness, Equation 6.5.15 is set equal to Equation 6.5.16. Re-arrangement of this equality gives the following formulation for T_{gs}:

$$T_{gs} = \frac{(D-2T)}{2} \cdot \frac{1}{0.8} \cdot \frac{E_g}{E_s} \qquad \text{.... 6.5.17}$$

Taking the modular ratio (E_s/E_g) as 18 for standard grout mixes (other values may be appropriate for low-heat grouts) as recommended in the HSE Guidance Notes:

$$T_{gs} = \frac{D-2T}{2} \times \frac{1}{14.4} \qquad \text{.... 6.5.18}$$

Note that the γ ratio for the equivalent steel tubular which has the same radial stiffness as the grout core is always 14.4. The larger the diameter of the chord member to be grout-filled, the larger the thickness of the equivalent steel tubular for the grout core. The calculation for T_{gs} as described by Equation 6.5.18 is valid for thin-shell/thick-shell theory since $\frac{D-2T}{2T_{gs}}$ is always 14.4 (ie. greater than the limit of 10 as defined by Roark[6.23]).

Having developed a formulation for T_{gs}, the next step is to determine an equivalent thickness, T_e, which gives the same inertia as the combined (composite) action of the chord shell and the equivalent steel member representing the grout core. This equality leads to the following equation for T_e:

$$T_e = (5D + 134T)/144 \qquad \text{.... 6.5.19}$$

Note that $T_e/T = 0.07\gamma + 0.93$, and the formulation is active from $\gamma = 1.0$ (solid steel cylinder) upwards.

IV 6.5.4.4 <u>Calibration of effective thickness model</u>

It can be observed from an examination of Database 6.5.2 that significantly reduced benefits may be obtained through grouting for IPB loading. The reduced benefits for IPB loading under grouted conditions can be accounted for by introducing a modified γ_e term for this case, as follows:

$$\gamma_e = \frac{D}{2T^{0.9}T_e^{0.1}} \qquad \text{.... 6.5.20}$$

In this manner, the benefits through grouting for IPB can be downgraded in line with the observation noted above. The Equivalent Thickness Model with modified γ_e (T replaced by T_e for axial/OPB loadcase, and the above formulation for IPB) has been used to predict SCF Reduction Factors (RFs) for the joints contained in Database 6.5.2. A comparison of predicted RFs with measured RFs is presented in Tables 6.5.2, 6.5.3 and 6.5.4, for axial tension, IPB and OPB cases, respectively. The following comments are worthy of note from an examination of these tables:

(1) The effective thickness approach performs reasonably for axial and OPB SCFs not withstanding the likely scatter (in test data), relatively confined database and accuracy of Efthymiou formulations which have been used as a basis for the model.

(2) The modified T_e term for IPB performed satisfactory, reflecting limited benefit on SCFs from grouting in these instances.

Figures 6.5.2, 6.5.3 and 6.5.4 present histograms of the ratios of prediction to test results for axial tension, IPB and OPB load cases, respectively. The summary statistics are also noted on these figures.

Specimen No.	Joint Type	Brace (Saddle)			Chord (Saddle)		
		Pred.	Test	Pred/Test	Pred.	Test	Pred./Test
PP1-1	X	0.46	0.39	1.179	0.42	0.36	1.167
PP1-2	X	0.42	0.34	1.235	0.38	0.31	1.226
PP1-3	X	0.47	0.55	0.855	0.44	0.59	0.746
PP1-4	X	0.48	0.39	1.231	0.44	0.36	1.222
PP1-5	T	0.48	0.43	1.116	0.42	0.46	0.913
PP2-X1	X	0.54	0.50	1.080	0.50	0.49	1.020
PP2-X2	X	0.45	0.43	1.047	0.40	0.43	0.930
PP2-X3	X	0.46	0.36	1.278	0.42	0.35	1.200
PP2-X4	X	0.33	0.24	1.375	0.30	0.21	1.429
PP2-X5	X	0.58	0.58	1.00	0.43	0.71	0.606
PP2-T1	T	0.51	0.52	0.981	0.43	0.51	0.843
PP2-T3	T	0.42	0.38	1.105	0.36	0.44	0.818
PP2-T4	T	0.54	0.56	0.964	0.46	0.56	0.821
PP2-K1	K	0.73	0.54	1.352	0.66	0.74	0.892
PP2-K2	K	0.66	0.87	0.759	0.59	0.87	0.678
PP2-Y32	Y	0.66	0.76	0.868	0.47	0.72	0.653
PP2-Y60	Y	0.53	0.45	1.178	0.45	0.48	0.938
PP0-T2	T	0.57	0.49	1.163	0.49	0.47	1.043
PP0-X2	X	0.46	0.35	1.314	0.42	0.33	1.273
1	K	0.72	0.67	1.075	0.62	0.72	0.861
1	T	0.48	-	-	0.38	0.54	0.704
2	X	0.42	-	-	0.39	0.41	0.951
1	T	0.72	0.62	1.161	0.61	0.63	0.968

Note: The last four specimens are fully grouted, all others are double-skin.

Table 6.5.2: Grouted/double-skin joint test data on SCFs - measured and predicted axial tension SCF RFs

Specimen No.	Joint Type	Brace (Saddle)			Chord (Saddle)		
		Pred.	Test	Pred/Test	Pred.	Test	Pred./Test
PP0-T2	T	0.96	-	-	0.95	0.93	1.022
1	K	0.99	0.66	1.500	0.96	0.91	1.055
1	T	0.97	-	-	0.96	0.95	1.011
2	X	0.95	-	-	0.94	0.76	1.237
1	T	0.97	1.00	0.970	0.96	0.98	0.980

Table 6.5.3: Grouted/double-skin joint test data on SCFs - measured and predicted IPB SCF RFs

Specimen No.	Joint Type	Brace (Saddle)			Chord (Saddle)		
		Pred.	Test	Pred/Test	Pred.	Test	Pred./Test
PP0-T2	T	0.51	0.81	0.630	0.44	0.77	0.636
1	K	0.43	0.30	1.433	0.41	0.13	3.154
1	T	0.47	-	-	0.44	0.33	1.333
2	X	0.43	-	-	0.41	0.27	1.519
1	T	0.57	0.84	0.679	0.55	0.72	0.764

Table 6.5.4: Grouted/double-skin joint test data on SCFs - measured and predicted OPB SCF RFs

IV 6.5.5 Fatigue

Few documented fatigue test results are available for grouted or double-skin joints. In Reference 6.7, the results of four fatigue tests, subjected to constant amplitude cyclic axial loads, are reported. A strain ratio of $R \simeq -1$ was adopted. These results are tabulated in Database 6.5.3. A comparison of measured life (using measured 'hot spot' strain range) with life predicted using the HSE Guidance Notes is presented in Table 6.5.5. The hot spot strain range noted in Database 6.5.3 has been converted to a hot spot stress range using a factor of 1.2 E[6.21]. It can be observed that the measured fatigue lives are well in excess of predictions. Although increased fatigue endurance can be noted, the paucity of data precludes this enhancement to be fully qualified. However, the data provide sufficient evidence that the HSE Guidance Notes S-N curve (T curve - developed on the basis of ungrouted joints) can be used for the fatigue life assessment of grouted and double-skin joints, provided the hot-spot stress range concept is adopted. This observation reduces the fatigue assessment

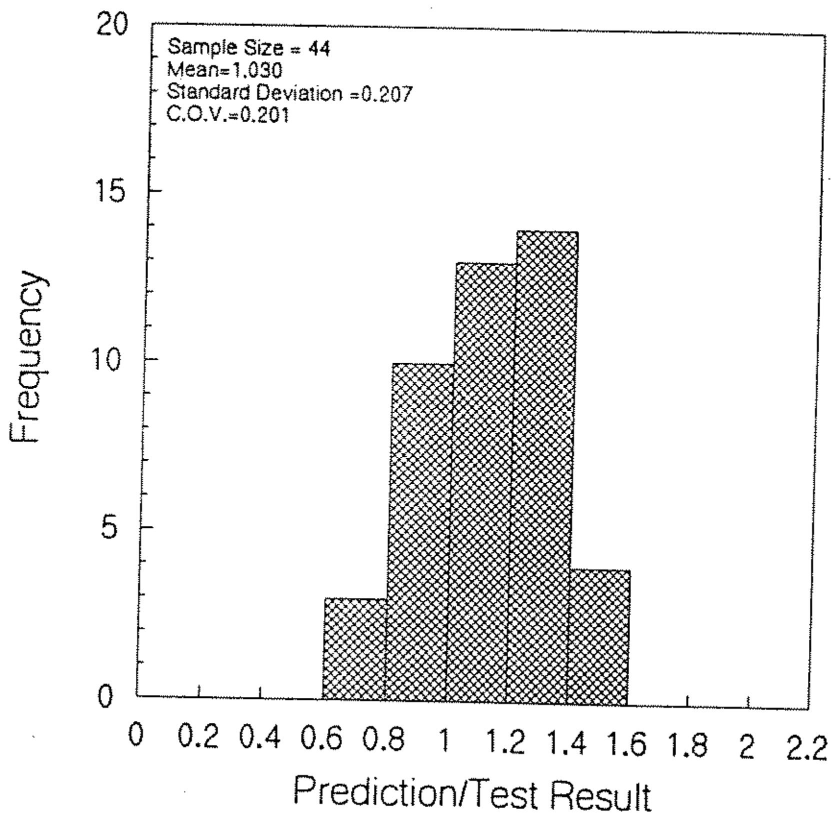

Figure 6.5.2: **Histogram for axial tension loaded joints - SCF RFs**

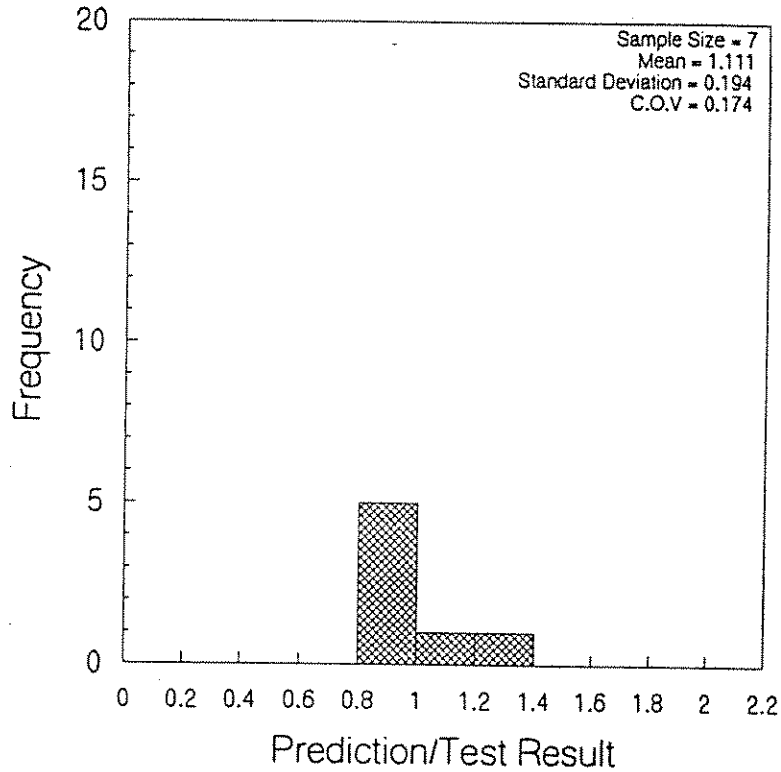

Figure 6.5.3: Histogram for IPB loaded joints - SCF RFs

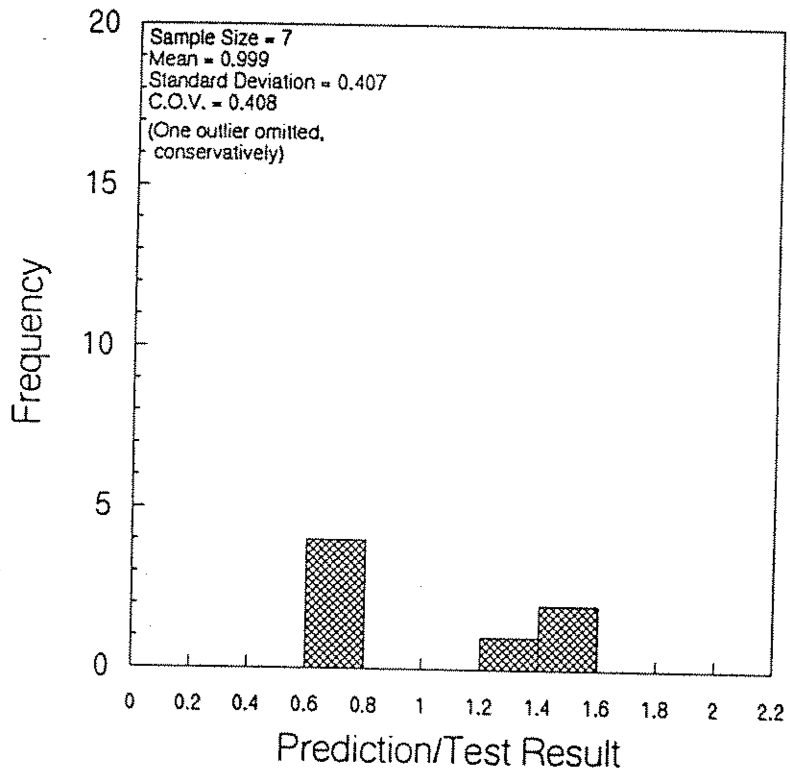

Figure 6.5.4: Histogram for OPB loaded joints - SCF RFs

problem for grouted/double-skin joints to SCF estimations.

Specimen No	Predicted Life (cycles)	Measured Life (cycles)	Prediction/Measured
PP1 - 2	0.04×10^6	11×10^6	0.0036
PP2 - X1	0.04×10^6	$> 15 \times 10^6$	< 0.0027
PP2 - X2	0.02×10^6	$> 5 \times 10^6$	< 0.0040
PP2 - K1	0.02×10^6	0.225×10^6	0.0889

Table 6.5.5: Comparison of predicted and measured fatigue lives for double-skin joints

IV 6.5.6 Local Joint Flexibility

References 6.6, 6.7 and 6.21 presents outline data on local joint flexibility for grouted/double-skin joints. The results indicate that the assumption of rigid connections for global analysis purposes is appropriate.

IV 6.6 DEVELOPMENT OF RECOMMENDED PRACTICE

Section IV 6.5 above contains an exhaustive appraisal of all available data and information on grouted/double-skin joints. Resistance models have been prepared and calibrated to the available data and information. There are significant gaps in the data and this has been recognised in the preparation of the Recommended Practice in Part III of this document. Grout-filling arguably represents the most cost-effective and mechanically efficient technique available at the present time, and the Recommended Practice in Part III is expected to be of significant assistance to the designer, given the present-day complete lack of suitable guidance in design codes or published literature. Pending the generation of new data and information, and further calibration and enhancement of the Recommended Practice, it will be necessary to seek specialist advice on a case-by-case basis, to judge the direct relevance of the recommendations contained herein to the case at hand and, therefore, to define the need for, and extent of, back-up experimental/numerical studies, if any.

In the development of the Recommended Practice in Part III, the findings reported in Section IV 6.1 to Section IV 6.5 have been used. In addition, a number of other issues have been addressed, to ensure completeness of the Recommended Practice. These issues are described and discussed below:

Static Strength

- The Q_f factor in API RP2A and the HSE Guidance Notes accounts for de-rating of joint capacities due to the presence of chord loads. The reductions in capacity of as-welded joints stem from detrimental amplification by the chord loads of the stress levels imposed at the joint intersection through ovalisation from applied brace loads. The presence of grout in chord members essentially constrains ovalisation of the joint. The effects of chord loads in this instance would be expected to be reduced. However, due to a complete lack of suitable data to cover this case, it is considered appropriate to conservatively adopt the existing Q_f term.

- The recommended brace interaction curve for grouted joints is the same as that for as-welded joints in the HSE Guidance Notes. The load interaction curve requires, as input, the allowable loads for unidirectional axial, IPB and OPB cases. The failure modes for grouted/double-skin joints suggests that adoption of the as-welded interaction curve is directionally correct, i.e. beneficial interaction for axial/IPB loads, and detrimental interaction for axial/OPB loads.

SCFs and Fatigue

- The present-day design practice for as-welded joints does not consider or recognise the status of SCF formulae. Some designers believe their calculated SCFs give totally reliable estimates of 'hot spot' stresses, others recognise that scatter is likely (perhaps of the order of plus or minus 20% in nominal SCF) whilst others believe that the formulae give an upper bound and therefore provide a safe design. If a truly probabilistic approach were adopted, it would be necessary to consider four primary variables in the calculation of the probability of failure by fatigue. These variables are:

  (1) loading spectrum

  (2) accuracy of SCF

  (3) scatter in fatigue life for known hot spot stress range

  (4) relationship of stresses at point of consideration to calculated SCF.

  The treatment of multi-variable probabilistic analysis is quite complex and, although it is possible, it is not appropriate in the current context. For chord grout-filling application, the concept of Assessment SCF is introduced.

Assessment SCF is given by the relationship:

$$ASCF = \frac{SCF}{\Gamma}$$

where ASCF = Assessment stress concentration factor

SCF = SCF calculated in the basis of Section IV 6.5.4

Γ = partial factor selected to give the appropriate level of confidence.

Depending on the approach taken on a case-by-case basis, Γ can be modified to give a level of confidence appropriate to the case in question.

- Although adequately substantiated evidence for the effects of grouting for high β, low γ joints is not available, it is understood that these 'stiff' joints benefit little from chord grout-filling, for reasons noted in Section IV 6.5.4.2. The benefits for joints with $\beta = 1.0$ and $\gamma \leq 12.0$ should, therefore, be given special consideration.

- For T/Y joints subjected to brace axial loads:

  (i) there is evidence that a more uniform distribution of stresses occur around the joint periphery, leading in some cases to an increase in SCFs at the crown locations.

  (ii) it is recognised that the bending stiffness of the chord increases, thereby reducing α-induced stresses, in the beam-bending sense, at the crown location.

  The load taken in bending by the grout can be shown to be represented by the following:

  $$(\gamma - 1)^4 / (18\,\gamma^4 - 17\,(\gamma - 1)^4) \qquad \text{.... 6.6.1}$$

  It can also be shown that (for $\gamma = 12$ joints) the load taken is 11.8%. For higher γ ratios, the grout will take an increasing share of the chord load.

  This assumes simple conditions, with no cap end (i.e. chord end) effects (conservative). Therefore, either the above expression or a factor of 0.88 can be applied (conservatively) to the α part of the Efthymiou equations for chord and brace crown SCFs for axially loaded T/Y joints.

IV 6.7 CLOSURE

This Section IV 6 has covered an appraisal of grouted/double-skin technology. The findings contained herein are used in the development of the Recommended Practice described in Part III of this document.

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Ref. No.	Specimen No.	Joint type	D (g) (mm)	gamma	beta	tau	theta	alpha	tau_g	tau_p	$gamma_g$	$gamma_p$	Chord F_y (N/mm^2)	Brace F_y (N/mm^2)	Measured load (kN)	Comments
6.7	PP1-5	T	508.0	20.7	0.482	0.772	90	13.00	4.88	1.300	4.23	11.1	397	370	2109	Yield load
	PP2-X3	X	291.2	21.0	0.746	0.741	90	12.30	4.21	1.270	4.99	12.4	368	401	981	Yield load
	PP2-T1	T	300.0	19.8	0.252	0.793	90	17.30	4.35	1.160	4.56	12.4	352	375	618	Yield load
	PP2-T3	T	589.3	30.3	0.493	0.826	90	9.48	6.41	0.821	4.72	27.8	377	368	2394	Rupture
6.9	1*	K	457.2 (55.1)	24.0	0.889	1.000	60	17.50	23.00	-	1.00	-	367	-	4510	Compression brace failure

Notes:

(i) * Full grout case; all others double-skin

(ii) All joints subjected to axial tension, apart from K joint which was subjected to balanced axial load

(iii) Subscripts g and p refer to grout and pile respectively

Database 6.5.1: Grouted/double-skin joints test data on static strength

Ref. No.	Specimen No.	Joint type	D (g) (mm)	gamma	mma$_g$/gam Ax, OPB (IPB)	beta	tau	theta	alpha	tau_g	tau_p	$gamma_g$	$gamma_p$	SCF RF**					
														Axial Tension		IPB		OPB	
														Chord Saddle	Brace Saddle	Chord Crown	Brace Crown	Chord Saddle	Brace Saddle
6.7	PP1-1	X	297.2	20.7	0.422	0.507	0.685	90	12.10	4.49	1.150	4.61	13.20	0.36	0.39	-	-	-	-
	PP1-2	X	297.2	24.2	0.383	0.506	0.800	90	12.10	5.41	1.300	4.47	13.70	0.31	0.34	-	-	-	-
	PP1-3	X	297.2	19.7	0.435	0.507	0.887	90	12.10	1.32	1.060	14.90	16.30	0.59	0.55	-	-	-	-
	PP1-4	X	294.6	19.3	0.440	0.514	0.632	90	12.20	4.30	1.050	4.50	13.70	0.36	0.39	-	-	-	-
	PP1-5	T	508.0	20.7	0.422	0.482	0.772	90	13.00	4.88	1.300	4.23	11.10	0.46	0.43	-	-	-	-
	PP2-X1	X	289.6	15.3	0.502	0.522	0.756	90	12.40	2.62	0.929	5.66	12.40	0.49	0.50	-	-	-	-
	PP2-X2	X	289.6	22.5	0.401	0.250	0.713	90	12.40	5.53	1.360	4.97	12.40	0.43	0.43	-	-	-	-
	PP2-X3	X	292.1	21.0	0.419	0.746	0.741	90	12.30	4.21	1.270	4.99	12.40	0.35	0.36	-	-	-	-
	PP2-X4	X	403.9	34.7	0.299	0.496	0.601	90	11.90	5.91	1.030	5.88	27.00	0.21	0.24	-	-	-	-
	PP2-X5	X	269.2	20.0	0.431	1.000	0.705	90	12.00	4.40	0.997	4.54	14.70	0.71	0.58	-	-	-	-
	PP2-T1	T	300.0	19.8	0.434	0.252	0.793	90	17.30	4.35	1.160	4.56	12.40	0.51	0.52	-	-	-	-
	PP2-T3	T	589.3	30.3	0.330	0.493	0.826	90	9.48	6.41	0.821	4.72	27.80	0.44	0.38	-	-	-	-
	PP2-T4	T	294.6	19.8	0.434	0.755	0.822	90	8.41	4.39	1.000	4.51	14.50	0.56	0.56	-	-	-	-
	PP2-K1	K	599.4 (45.0)	19.7	0.435	0.425	0.850	60	6.80	3.85	1.050	5.12	14.30	0.74	0.54	-	-	-	-
	PP2-K2	K	322.6 (36.8)	28.4	0.344	0.401	0.764	60	12.60	5.66	0.952	5.02	23.90	0.87	0.87	-	-	-	-
	PP2-Y32	Y	320.0	19.2	0.442	0.506	0.782	32	14.60	4.15	0.971	4.62	14.60	0.72	0.76	-	-	-	-
	PP2-Y60	Y	322.6	19.5	0.438	0.503	0.783	60	12.60	4.23	0.981	4.61	14.60	0.48	0.45	-	-	-	-
	PPO-T2	T	508.0	15.9	0.491 (0.931)	0.481	0.625	90	13.00	3.75	1.000	4.23	11.10	0.47	0.49	0.93	-	0.77	0.81
	PPO-X2	X	294.6	21.0	0.419	0.511	0.704	90	12.10	4.59	1.000	4.36	15.40	0.33	0.35	-	-	-	-
6.9	1*	K	457.2 (55.1)	24.0	0.385 (0.908)	0.889	1.000	60	17.50	23.00	-	1.00	-	0.72	0.67	0.91	0.66	0.13	0.30
6.11,6.12	1*	T	660.4	34.7	0.299 (0.885)	0.923	1.300	90	7.70	33.70	-	1.00	-	0.54	-	0.95	-	0.33	-
	2*	X	609.6	24.0	0.385 (0.908)	0.531	0.750	90	8.30	23.00	-	1.00	-	0.41	-	0.76	-	0.27	-
6.18	1*	T	914.4	14.3	0.520 (0.937)	0.500	0.500	90	5.00	13.30	-	1.00	-	0.63	0.62	0.98	1.00	0.72	0.84

Notes:
(i) * Full grout cases; all others double-skin
(ii) ** RF= SCF Reduction Factor (grouted SCF/ungrouted SCF)
(iii) Reference 6.18 data represent an average from two specimens
(iv) Subscripts g and p refer to grout and pile respectively

Database 6.5.2: Grouted/double-skin joint test data on SCFs

Ref. No.	Specimen No.	Joint type	D (mm)	alpha	beta	gamma	tau	theta	zeta	Hot spot strain range (microstrain)	Life to through thickness crack (cycles)
6.7	PP1-2	X	297.2	12.1	0.506	24.2	0.800	90	-	1500	11000000
	PP2-X1	X	289.6	12.4	0.522	15.3	0.756	90	-	1500	> 15000000
	PP2-X2	X	289.6	12.4	0.250	22.5	0.713	90	-	1800	> 5000000
	PP2-K1	K	599.4	6.8	0.425	19.7	0.850	60	0.075	1800	225000

Database 6.5.3: Double-skin test data on fatigue

REFERENCES

7.1 Godfrey, P. ERA Conference, London, 1992.

7.2 Mays, G. C. and Hutchinson, A. R.. "Adhesives in Civil Engineering". Cambridge University Press, UK, 1992.

7.3 Sharp, J. V., Bowditch, M. R. and Clarke, J. D. "Adhesive Based Repair Methods for Steel Offshore Structures" IRM '86, Aberdeen, Nov 1986.

7.4 Cao, Y. M., Julien, J. F., Renault J. P. and Lazare, F.. " Mechanical Behaviour of a Resin-Assembled Steel T-joint. OTC 6656, OTC conference Houston, Texas, 1991.

7.5 Private correspondence from Elf to project team.

7.6 Private correspondence from Mobil to project team.

7.7 Clarke, J, Peel J. W. and Lowes, J. M.. "Development of a Swaged Pile/Sleeve Connection System for Application on a North Sea Jacket" OTC 5772, OTC Conference, Houston, Texas, 1988.

IV 7 OTHER TECHNIQUES

IV 7.1 INTRODUCTION

This section deals with non-standard joining techniques which may have potential for use in repair and strengthening work. The items discussed in this section are not featured in detail in Volume III because they are not yet established in offshore engineering practice. In some instances there may be scope for their inclusion in current project work.

Three jointing techniques will be considered in detail. They are :

- Adhesives - using a glue, normally epoxy resin, to stick a repair to the structure.
- Swaging - forming a localised plastic flow region in a metal tube which creates an interference lock joint with another concentric tube.
- Mechanical Connections - using grab, twist and or indentation devices to achieve the mechanical locking of two component parts.

All three possible jointing techniques may be of interest for underwater repair work, and in most instances, some prototype work has been undertaken.

IV 7.2 ADHESIVES

The term adhesives encompasses a broad range of jointing operations which are generally performed either at ambient temperature or involve curing at temperatures which are generally below 100°C.

Epoxy resins are the most commonly used structural glues. This range of materials encompasses a gamut of one- and two-part resins, films and pastes. The materials have had extensive use in marine craft and, in particular are used in the construction of hulls in minesweepers (Naval vessels). Godfrey[7.1] gives a detailed breakdown of the types of adhesives that are used in offshore applications.

The epoxy materials may have fillers added to improve the thixotropy of the glue, so as to prevent it flowing off vertical surfaces, or to increase strength and alter the permeability of the resulting matrix. Glass fibres improve tensile strength and reduce the exothermic reaction. Plate-like inclusions are included in one particular brand of metal coating so as to lengthen the leak path and hence improve the impermeable quality of the finished product.

Glass reinforced plastics have been used in offshore structures for over a decade. Current uses include fittings and partitions in accommodation blocks, fire protection of risers, for cladding and bulkheads, and as a hard-wearing surface/fire protection for deck plates. More recently, carbon-fibre reinforced laminates have been used for strengthening blast walls, and has great potential for repair of components. The strength and stiffness of carbon-fibre laminates can surpass those of steel, but at a fraction of the weight. A joint industry project is currently underway at MSL Engineering to enable these materials to be exploited. They are not considered further here.

IV 7.2.1 Concrete Work

Adhesive repairs are a practical proposition for a number of applications. Repairs to concrete structures using adhesives are more common than those for steel because of the comparable strength of the adhesive and concrete, with the advantage that adhesives exhibit predictable tensile strength. Epoxy resin is commonly used to anchor steel bolts and bars in reinforced and plain concrete. It is also used to protect re-bars which have been exposed and may suffer corrosion before a final repair can be made.

IV 7.2.2 Joint Design

Epoxy glues need to harden and make take up to 15 days to cure at 5°C. Some epoxies will not set below this temperature which can be a problem in the North Sea. In general, the curing time will be halved for each 10°C rise in temperature[7.2].

Strength data for many adhesives can only be obtained from the manufacturer but the following data (from Reference 7.2) may be used for guidance:

Material	Tensile strength N/mm^2	Shear strength N/mm^2	Tensile Modulus N/mm^2	Flex. Modulus N/mm^2	Poisson's Ratio
Epoxy polyamide	12	15	2000	2100	0.40
Aromatic Epoxy Polyamide	16	28	2800	3100	0.38

Use of a particular epoxy for offshore applications needs a partnership approach with client, supplier and designer taking responsibilities for various aspects of the project.

For joint repair work, a non-linear analysis technique may be required to take account of creep behaviour of the epoxy material. Trials may be required to prove curing time, strength and fatigue endurance. S-N data are available for a limited range of circumstances. Techniques for joint design are described in Reference 7.2

Sharp et al[7.3] have reported development work on epoxy glues that will set underwater. The work describes sleeve design technique, fatigue, shear and creep behaviour. Tests include 8-year specimen immersion in seawater.

Guidelines

When structural design with adhesives is considered, it will be important to consider the following key points:

- How can the adhesive be held in place whilst it sets?
- What is the curing time, and associated expansion and shrinkage effects. Can the heat of formation be controlled in the joint during setting?
- How is the joint filled, and how long is there to set the joint before setting occurs? Can the joint be held tight whilst the adhesive sets?
- What provisions can be made for quality assurance?
- Can the long term integrity be guaranteed?
- Can the repair be removed if something goes wrong with the installation?

Applications

Repairs to structural joints have been proposed by Cao et al[7.4] and it is understood that a prototype repair was installed on a West African platform[7.5]. Trials are currently underway to examine the efficacy of using epoxy-filled joints as a permanent bolt bonding technique[7.6].

Adhesives obviously represent one specialised repair and strengthening option which may see increased usage in the next decade of offshore SMR work.

IV 7.3 SWAGING

Swaging is a technique for joining metals used extensively in manufacturing processes to achieve the fast, low temperature connection of metals. In the offshore industry, swaging is used to join two concentric pipes. The inner pipe is expanded radially, by internal pressure, into grooves machined on the inside of the outer pipe. Elastic and plastic deformations take place such that, when the pressure is released, a permanent connection results.

Swaging tools are in common use for well operations and this experience was initially translated into subsea template installation where the technique was used for pile load transfer.

Details of a proprietary pile connection system have been described by Clarke et al[7.7]. This and other commercial systems may be covered by patent protection.

IV 7.4 MECHANICAL CONNECTIONS

Several specialised mechanical connections have been devised by supply companies for use in pipelines and in down-hole and subsea installations.

Key elements of mechanical connections are:

- Remote operation, often several hundred feet from the power source.
- Concerned largely with creating a shear link and a pressurised containment.
- Downhole tools are all geared to latch / unlatch under simple control operations such as push, pull, turn to right, turn to left.
- The tooling often involves threaded connections.

Some mechanised connections may be of value to those concerned with pipeline or subsea repairs. Several systems are the subject of patent protection. Individual options may be developed in conjunction with the relevant equipment supply company.

REFERENCES

7.1 Godfrey, P. ERA Conference, London, 1992.

7.2 Mays, G. C. and Hutchinson, A. R.. "Adhesives in Civil Engineering". Cambridge University Press, UK, 1992.

7.3 Sharp, J. V., Bowditch, M. R. and Clarke, J. D. "Adhesive Based Repair Methods for Steel Offshore Structures" IRM '86, Aberdeen, Nov 1986.

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CONTROLLED DOCUMENT

STRENGTHENING, MODIFICATION AND REPAIR OF OFFSHORE INSTALLATIONS

PART V - CLAMP STUDBOLT LOAD VARIATION

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STRENGTHENING, MODIFICATION AND REPAIR OF OFFSHORE INSTALLATIONS

PART V - CLAMP STUDBOLT LOAD VARIATION

STRENGTHENING, MODIFICATION AND REPAIR OF OFFSHORE INSTALLATIONS
PART V - CLAMP STUDBOLT LOAD VARIATION

CONTENTS

NOMENCLATURE

B	transverse distance between studbolt rows
D	internal diameter of saddle
E	Young's modulus
e	eccentricity
F	horizontal resultant acting on clamp half
F_i	product of F_1 and F_2
F_0	product of F_3 and F_4
F_1 to F_4	reduction factors (see Section V 6)
H	height of clamp
J	torsional constant
K	spring stiffness
L	length of enclosed tubular
ℓ	length of clamp
ℓ_1, ℓ_2	lengths of tubulars within clamp
M	applied moment
P	preload; total load
P_m	studbolt load variation obtained from Assessment Model
P_1, P_2	point loads acting on clamp half
Q	value of distributed load acting on both clamp halves
q_1, q_2	maximum values of distributed loads acting on clamp half
R	internal radius of saddle plate; relative stiffness ratio
s	studbolt spacing
T_H	component of torque due to horizontal forces
T_V	component of torque due to vertical forces
T_1, T_2	internal torques
w_1, w_2	maximum values of distributed loading taken by studbolt pairs
x	distance between end of clamp and a point midway between first and second pair of studbolts
Γ	partial factor of safety

β	circumferential angle
Δ	displacement
σ_r	radial stress
σ_v	vertical component of radial stress

V 1 INTRODUCTION

In the design of clamps, there are two main criteria which have to be considered in establishing the magnitude of the total studbolt load. These are:

- to provide sufficient resistance against slippage of the clamp
- to prevent the two halves of the clamp being prised apart due to the action of applied loadings (i.e. a bolt pull-off check).

In approximately half of clamp designs, it is the second of these which governs the studbolt preload that is specified. The studbolt load, in turn, affects the studbolt size and, in conjunction with the capacity of the tubular member to resist hoop compression, the studbolt spacing.

It is apparent, therefore, that design assumptions with respect to the load variations induced in the studbolts as a result of applied loading in the clamped tubular member, have a fundamental effect on clamp sizing. Despite this observation, there are little available data on studbolt load variation and necessarily conservative design assumptions have to be made. Since clamping technology represents one of the major options for conducting strengthening and repair work, the Project Steering Committee accepted MSL Engineering's proposal that additional Participant funding should be used to explore the robustness of the relevant design recommendations in Part III of the Design Manual. It was further agreed that this should be through laboratory testing and FE modelling.

This report presents the results and the assessment of a series of ten tests on a neoprene-lined clamp and seven on a stressed grouted clamp. In addition, an FE analysis programme was conducted which simulated a variety of stressed grouted clamps. Section V 2 describes the objectives and the scope of the test programme, and the test hardware. Section V 3 presents the results of the tests. Section 4 presents the FE programme. An assessment of the results is given in Section V 5 and a suitable design methodology is developed in Section V 6. Section V 7 presents the conclusions of the work.

The matrices for both the test programme and the FE studies were designed by MSL. The testing work was conducted by City University, London, and the FE analyses were performed by British Gas plc. The results were assessed, and the recommendations were formulated by MSL.

V 2 TEST PROGRAMME DESCRIPTION

V 2.1 OBJECTIVES

The overall objective of the test programme was to provide experimental data on the variation of studbolt loads due to externally applied loading. Data was generated for both neoprene-lined and stressed grouted clamps. Secondary objectives were to explore the effects of studbolt preload, the position of the abutting ends of the tubular members within the clamp, studbolt stiffness and to study the effect of a missing studbolt on results.

V 2.2 SCOPE OF TESTS

The tests essentially comprise a specimen, consisting of the clamp and two abutting tubular members (as shown in Figure 2.3), under four-point bending (pure bending along clamp). Seventeen tests were conducted as specified in Table 2.2.1 below.

Test Number	Clamp Type	Tubular Position Within Clamp	Studbolt Size	Orientation of Studbolts to Plane of Bending (°)*	Studbolt Preload (kN)	Maximum Applied Bending Moment (kNm)
1	Neoprene	25/75	M30	0	180	95
2	Neoprene	25/75	M30	90	180	95
3	Neoprene	37.5/62.5	M30	0	90	70
4	Neoprene	37.5/62.5	M30	90	90	70
5	Neoprene	37.5/62.5	M30	0	180	140
6	Neoprene	37.5/62.5	M30	30	180	140
7	Neoprene	37.5/62.5	M30	60	180	140
8	Neoprene	37.5/62.5	M30	90	180	140
9	Neoprene	37.5/62.5	M24	0	180	140
10	Neoprene	37.5/62.5	M24	90	180	140
11	Grouted	37.5/62.5	M24	0	180	140
12	Grouted	37.5/62.5	M24	90	180	140
13	Grouted	37.5/62.5	M30	0	180	140
14	Grouted	37.5/62.5	M30	90	180	140
15	Grouted	37.5/62.5	M30	0	180	140
16	Grouted	37.5/62.5	M30	90	180	140
17	Grouted	37.5/62.5	M30	270	180	140

* Plane of bending is vertical; 0° = vertical studbolts (IPB), 90° = horizontal studbolts (OPB)

Note: Tests 15 to 17 were with a missing studbolt, see text

Table 2.2.1: Details of test programme

In Table 2.2.1, two tubular positions are indicated, namely 37.5/62.5 and 25/75. The numbers refer to the percentage of the clamp length that encloses each tubular. The purpose of varying the tubular positions within the clamp was to investigate L/D effects (L/D varies from 1.24 to 3.71 given the geometry described in Section V 2.3).

The maximum applied moment, given in Table 2.2.1, was calculated on the basis that the bolts within the portion of the clamp enclosing the shorter tubular length resist the full applied moment and that the bolt load so calculated must be less than the bolt preload.

The tests were conducted in two series (i.e. Tests 1 to 8 and Tests 9 to 17 respectively) with an assessment period in between. The decision was made during the assessment period to investigate different studbolt sizes and to remove the neoprene liner (after Test 10) and fill the annulus so created with grout. Tests 15 to 17 were designed to study the effect of a missing studbolt. Tests 16 and 17 differed with respect to whether the missing studbolt would have lain above or below the tubular members.

All tests were performed in the elastic region of material behaviour, albeit the neoprene or grout may have behaved non-linearly.

V 2.3 DESCRIPTION OF CLAMP SPECIMEN

The specimen consisted of a clamp and two tubular members.

Initially, use was made of a single neoprene-lined clamp that was designed and detailed to be within the constraints of the testing apparatus. A neoprene-lined clamp was first selected in preference to a stressed grouted clamp for ease of re-use, and economy, between parts of the test programme. However, as mentioned in Section V 2.2 above, Tests 9 to 17 were conducted with the liner replaced by a grout annulus.

The clamp had an overall length of 1600 mm and was provided with 8 pairs of studbolts at 200mm spacing. Figures 2.1 and 2.2 give details of the clamp dimensions. The neoprene liner was 10 mm nominal thickness of hardness 65 IRHD to BS903 Part A26, and was cold bonded to the clamp saddle plate. The grout annulus in later tests was also of 10 mm thickness.

The M24 and M30 studbolts were to ASTM A193 Gd B7 specification and the nuts to ASTM A194 Gd 2H.

The two tubular members (tube A and tube B) were cut from a single length of pipe of nominal dimensions 324 mm diameter by 9.5 mm thickness. Actual thickness measurements ranged from 9.30 mm to 9.86 mm with a mean of 9.49 for tube A and 9.65 mm for tube B. The seamless tubular members were to API Spec. 5L and were of sufficient length to accommodate the different connection lengths within the test apparatus. The average yield and ultimate strengths of longitudinal tensile specimens taken from the tubular were 344 N/mm^2 and 485 N/mm^2 respectively (mill certificate values).

V 2.4 DESCRIPTION OF TEST RIG

Two similar self-reacting four-point bending rigs were used for applying the moments. For both rigs the saddle supports were at 5640 mm centres and the loading actuators at 3640 mm centres. The clamp was positioned symmetrically within the pure moment loaded region. The actuators were provided with swivel bearings, load cells and saddle plates.

By suitably orientating the specimen within the rig, pure IPB, pure OPB or any combination of IPB and OPB could be achieved.

V 2.5 INSTRUMENTATION

All studbolts were provided with a pair of strain gauges. These were located on machined flats at opposite ends of a diameter at the mid-length of each studbolt. The studbolts were individually calibrated. For convenience, each studbolt was assigned a letter as indicated in Figure 2.3.

One end of each tubular is strain gauged as indicated in Figure 2.4. The majority of the strain gauges were located at the split line of the clamp (i.e. at the 3 and 9 o'clock positions) to avoid potential damage on specimen assembly/strip-out.

Strain gauge readings were recorded directly through a Compulog data acquisition system.

Deflections were monitored by dial gauges. The position of the gauges are shown in Figure 2.5.

The outputs from load cells under the actuators was also recorded by the Compulog system.

V 2.6 TEST PROCEDURE

The bottom half of the clamp was placed in the test rig and supported. The tubulars were then positioned in the clamp. The top half of the clamp was then positioned together with the studbolts. Spherical washers were used with the studbolts. A hydraulic tensioning system was used to tension all the bolts simultaneously to the required load.

The specimen was rotated to the required plane of loading and the dial gauges were set up. The strain gauges were connected to the logger and initialised for the start of the test.

The load was applied in increments and readings taken after an interval of 5 minutes. Readings were taken at each increment.

V 3 TEST RESULTS

V 3.1 GENERAL

The complete set of test data is tabulated in Appendix A. Bolt loads (based on the paired strain gauges in conjunction with the calibration factors - see Section V 2.5), strain gauge outputs and deflection readings are presented for each test and at each load increment.

In the subsections below, pertinent information is extracted and presented graphically. In general, only results pertaining to the maximum applied moment in each test are shown. However, aspects dealing with non-linearity of the data (due mainly, it is thought, to the nature of internal bearing load distributions) are addressed in Section V 3.2.

V 3.2 LOADS IN STUDBOLTS

V 3.2.1 Series 1 Tests (Tests 1 to 8)

Figures 3.1 to 3.8 illustrate the distribution of bolt load variation (about the preload identified in Table 2.2.1) along the length of the clamp for the maximum applied moment in Tests 1 to 8 respectively. Bolts A to H were along one side of the clamp and bolts I to P were the corresponding partners in each pair of bolts, see Figure 2.3. In addition, the distribution of the average load variation in each pair of studbolts is shown.

The distribution patterns are more easily compared by referring to Figure 3.9 which is a composite of the previous eight figures. One pattern broadly applied to five tests, namely Tests 2, 4, 6, 7 and 8. In these tests the direction of the studbolt axes is not in the plane of bending. It is remarkable that the angle between the studbolt axes and the plane of bending (i.e. the orientation angle in Figure 3.9) does not have to be large to achieve pattern similarity; e.g. the pattern of Test 6 at an orientation of only 30° is almost identical to those of Tests 7 and 8 at orientations of 60° and 90° respectively whilst all three tests are very different to Test 5 at 0° orientation. It is appropriate to explain the patterns in the series of Tests 5 to 8, where the clamp orientation is steadily increased from 0° to 90° in 30° increments. The maximum studbolt load in Test 5, at 0° orientation, is approximately 4 kN whereas in Test 8, at 90° orientation, it is more than an order greater at 45 kN. Thus, at intermediate orientations, studbolt loads are dominated by the resolved moment corresponding to the 90° orientation. The maximum studbolt loads for Tests 6 (30°) and 7 (60°) can be well predicted from the results for Tests 5 (0°) and 8 (90°) by resolving the applied moment:

Test 6: $P = 45 \sin 30° + 4 \cos 30° = 22.5 + 3.5 = 26$ kN

Test 7: $P = 45 \sin 60° + 4 \cos 60° = 39.0 + 2.0 = 41$ kN.

The calculated values, 26 kN and 41 kN, can be compared with the test results of 24 kN and 39 kN respectively.

Tests 1, 3 and 5, with the studbolt axes in the direction of the plane of bending, display different distributions to each other. It is to be particularly noted that Tests 3 and 5, which had the tubulars at the same position (37.5/62.5) within the clamp, and the same ratio of bolt preload to applied moment, present very different bolt load patterns. For any of the three tests, it may be observed that the distributions along each side cross over each other. This indicates that a degree of twisting occurred in the clamp (one studbolt in any one particular pair loads, whilst the other unloads).

It is relevant for the purposes of design to enquire about the degree of bending in the studbolts. In series 1 tests, no measurements were made of the bending induced as a result of preloading the studbolts. However, studbolt bending can be deduced due to the applied moment from the detailed strain gauge output data. Table 3.1 below gives the difference in strain readings, as a percentage of the average of the paired gauges, for the studbolt in each test having the greatest load variation.

Test Number	Designation of Studbolt with Maximum Axial Strain Variation	Bending in Studbolt (±%)
1	A	6.5
2	I	2.2
3	K	10.7
4	I	10.8
5	A	9.8
6	I	11.2
7	I	10.8
8	I	11.1

Table 3.2.1: Studbolt bending strain as a percentage of the axial strain arising from applied moment to the clamp (Tests 1 to 8)

The great majority of studbolts behaved reasonably linearly with applied moments. Those that did not were associated with Tests 1, 3 and 5, where the studbolt axes were parallel to the plane of bending, and were located in that portion of the clamp enclosing the shorter length of tubular. Figure 3.10 shows the variation of studbolt load with applied moment for that studbolt in each test displaying the greatest load variation. The increased non-linearity of studbolt behaviour in Tests 1, 3 and 5 over that in other tests can clearly be seen. Adjacent bolts in Tests 1, 3 and 5 also exhibited non-linear behaviour.

V 3.2.2 Series 2 Tests (Tests 9 to 17)

Figures 3.11 to 3.19 show the distributions of bolt load variation (about the preload value) along the length of the clamp for the maximum applied moment in Tests 9 to 17 respectively.

Tests 9 and 10, with the neoprene lined clamps are identical to Tests 5 and 8 respectively except for a change of studbolt size. Whereas Tests 8 (Figure 3.8) and 10 (Figure 3.12), for studbolts orthogonal to the plane of bending, give very similar distribution patterns, those for Tests 5 (Figure 3.5) and 9 (Figure 3.11) are quite different. The Test 9 distribution pattern is, however, similar to Test 3 (Figure 3.3) which was tested under similar conditions to Test 5 but for a lower preload and applied moment (moment/preload ratio is constant).

Tests 11 to 17 relate to the stressed grouted clamp. An examination of the studbolt loads show that they are significantly less than those associated with the neoprene lined clamp. This is a direct result of the increased clamp stiffness such that the internal clamp loads are more disposed to relieve the clamp precompression rather than straining the studbolts. Nevertheless, the studbolt load distribution patterns for the two clamp types are broadly similar, e.g. compare the equivalent Tests 5 and 13 (Figures 3.5 and 3.15) or Tests 8 and 14 (Figures 3.8 and 3.16).

The effect of having different studbolt sizes can be seen by comparing Tests 11 and 13 (Figures 3.13 and 3.15) or Tests 12 and 14 (Figures 3.14 and 3.16). Similar patterns of distribution are evident; but larger studbolts with their greater stiffness usually attract more load.

The results of the tests with studbolt A removed are compared with equivalent tests with all studbolts in place in Figures 3.20 to 3.22. The equivalent test result to compare with Test 17 has been derived from a reflection of the result of Test 14 (in effect this rotates the clamp through 180°, and 90° + 180° = 270°, the angle at which Test 17 was conducted). It may be observed that the loss of a single studbolt has had most effect for Test 15 (Figure 3.20) when the studbolts are aligned parallel to the plane of bending. However, the maximum studbolt load variation is still small, less than 1 kN. The studbolt preloads are changed by removing studbolt A, see Figure 3.23. These preload changes are additive to the load variations caused by the applied moments (Figures 3.17 to 3.19).

Figure 3.24 confirms observations made with respect to the first series of tests (Figure 3.10). That is the studbolts behave reasonably linearly with applied moments when the studbolts are orthogonal to the plane of bending, but not otherwise.

V 3.3 STRAINS IN TUBULARS

Strains gauges were attached to the outside surfaces of the tubulars as indicated in Figure 2.4. The outermost sets of gauges, at the extreme right and left ends of the diagram, may or may not be within the extent of the clamp depending on its position with respect to the abutting ends of the tubulars.

Figures 3.25 to 3.41 illustrate the distribution of longitudinal strains along the tubulars at the maximum applied moment for Tests 1 to 17 respectively. Table 3.3.1 gives the calculated maximum strains on the outside surface of the tubulars at the various o'clock positions (see Figure 2.4) taking into account clamp orientation and the applied moment. These values are calculated on a simple elastic beam basis using the nominal dimensions of the tubulars and nominal applied moment.

Test Number	Calculated Maximum Strain (x10^6)			
	3 o'clock	6 o'clock	9 o'clock	12 o'clock
1	0	646	0	-646
2	646	0	-646	0
3	0	476	0	-476
4	476	0	-476	0
5	0	952	0	-952
6	476	825	-476	-825
7	825	476	-825	-476
8	952	0	-952	0
9	0	952	0	-952
10	952	0	-952	0
11	0	952	0	-952
12	952	0	-952	0
13	0	952	0	-952
14	952	0	-952	0
15	0	952	0	-952
16	952	0	-952	0
17	-952	0	952	0

Table 3.3.1: Calculated maximum longitudinal strains in tubulars at various circumferential positions (see Figure 2.4)

There is broad agreement between the above tabulated values and the maximum strains recorded in Figures 3.25 to 3.41. As expected, the strains at locations which are diametrically opposite are of equal magnitude but of opposite sign, and the strain distributions decay to zero towards the ends of the tubulars.

A comparison of strain distribution patterns for the neoprene lined clamp in the first series of tests (Tests 1 to 8) can be made from the composite Figure 3.42. It may be observed that when the axes of the studbolts are parallel to the plane of bending (0° orientation - Tests 1, 3 and 5), very linear distributions result. This is indicating that the section of each tubular enclosed by the clamp is acting as a cantilever, with the inference that the bending moment is fed from each tubular into the clamp by reasonably localised reaction forces at the end of the tubular and where the tubular first enters the clamp. By the time the clamp is rotated through 90°, i.e. as in Tests 2,4 and 8, the distributions exhibit some non-linearity, with the corresponding inference that internal reactions between the tubulars and the clamp are more diffuse in nature.

There are insufficient cross gauges to plot hoop strain distributions and, indeed, this was never the intention. Rather, a limited number of gauges were provided to assess the degree of hoop straining that might be sustained. Inspection of the relevant data in Appendix A shows that significant hoop strains can occur; the maximum recorded surface strain being at gauge 37 in test 8 (= -632 microstrain). It is presumed that the majority of this is due to hoop bending, rather than hoop extensional effects, arising from member ovalisation. (The adjacent gauge 43 at the same section gives a reading of 31 microstrain.) There is a strong tendency for the maximum hoop (surface) strain to occur at a section near the end of clamp rather than at the end of the tubular.

V 3.4 DEFLECTIONS

Dial gauges were used to measure overall deflections of the specimen and to guard against possible opening of the two clamp halves, see Figure 2.5. In general, deflections were linearly related to the applied moment. The readings at the maximum applied moment for each test in the first series are given in Table 3.4.1 below.

Test Number	Dial Gauge Reading (mm)					
	1	2	3	4	5	6
1	19.5	26.7	16.8	0.25	0.10	-
2	19.2	26.1	18.1	-0.55	-0.22	26.2
3	14.5	18.9	12.5	0.01	0.0	-
4	14.0	17.4	13.4	-0.23	-0.18	0
5	27.0	33.4	21.5	0.01	0.0	33.0
6	25.8	34.7	21.8	-0.22	-0.07	33.7
7	25.3	32.0	21.5	-0.29	-0.18	32.8
8	26.0	32.6	21.7	-0.34	-0.28	33.0

Table 3.4.1: Maximum deflections (see Figure 2.5)

An examination of the data given in Table 3.4.1 reveals the following:

- The readings for dial gauges 1 and 3 under the loading points are not the same indicating the system is not behaving symmetrically. The only geometric asymmetry relates to the position of the abutting ends of the tubulars. It would be erroneous to conclude that the stiffness of the clamp/tubular system is significantly less than that of an intact tubular because a check on beam curvature deflections (i.e. dial gauge 2 minus average of dial gauges 1 and 3) correlates reasonably well with calculated values assuming an intact tubular with no clamp:

Test Number	1	2	3	4	5	6	7	8
Calculated Deflection	6.45	6.45	4.75	4.75	9.51	9.51	9.51	9.51
Measured Deflection	8.55	7.45	4.95	3.70	9.15	10.9	8.60	8.75

  Test 1, perhaps, indicates a significant difference between calculated and measured values, and this may be due to the relative short enclosed length of one of the tubulars. However, in general, it is believed that most of the discrepancies between the readings for dial gauges 1 and 3 is due to rig deformation/settlement of supports.

- Dial gauges 4 and 5 give a measure of the relative movement (i.e. separation) of the two clamp halves. In tests 3 and 5, for which the studbolt axes are parallel to the plane of bending (i.e. 0° orientation), very little relative movement occurred. Test 1, also at 0° orientation, gave some movement, this presumably being due to the greater internal reaction forces associated with the shorter lever arm length of enclosed tubular. Relative movement occurred for all the tests which were not at 0° orientation, the movement becoming greater for increasing angle of orientation and for shorter enclosed length of tubular.

- A studbolt load can be calculated from the relative movement of the clamp halves assuming that it is taken up by straining over the effective length of the studbolt (assumed as 600 mm) and then converting this strain to a load using the studbolt calibration tests. On this basis, a relative movement of 0.25 mm gives a studbolt load of 45 kN. Although of the right order, the calculated values do not agree very well with measured studbolt loads, the former usually being somewhat greater than the latter. It is considered that bending in the flange plate could account for the discrepancy; it may be recalled that bending of the studbolt was observed in the tests (see Table 3.2.1).

Similar observations, i.e. non symmetry of results and non separation of clamp halves, can be inferred for the second series of tests from the results presented in Appendix A. For the grouted clamp tests, no separation appears to have occurred.

V 4 FE PROGRAMME

V 4.1 OBJECTIVE

The objective of conducting Finite Element (FE) analyses was to extend the geometries used in the test programme to give a more extensive database than that provided in the experimental work alone. In this manner the recommendations derived in Section V 6 will have a wider application and install confidence in their use.

V 4.2 SCOPE OF ANALYSES

A total of 12 clamp geometries were modelled, each analysed under both in-plane and out-of-plane bending. Advantage was taken of symmetry and 24 half models were generated - 12 for IPB about the vertical plane and 12 for OPB about the horizontal plane. The geometries are based on the stressed grouted clamp used in the test programme. Table 4.2.1 gives details of the cross-sectional geometries; in other respects the models resembled the test clamp. Thus Model 1 is nominally identical to the condition of Tests 13 and 14 in the experimental programme.

Model 1 was intended to serve as the basis of calibration to test data. Models 2 and 3, with Model 1, explore the effect of variation in studbolt size (stiffness). Model 10 differs from all others with respect to studbolt spacing. The effect of changing the position of the ends of the tubulars within the clamp is the basis for Model 11. The remaining models were designed to investigate changing the stiffnesses of various components of the clamp/grout/tubular system. In some cases this was simply effected by changing the modulus for the component, e.g. the clamp flange stiffness (and hence the overall bending rigidity of the clamp) in Models 6 and 7 (to compare with Model 1).

Changing the clamp saddle, grout annulus or tubular geometry presents a little difficulty. This is because it is not clear what are the fundamental governing parameters affecting studbolt loads. Changing the tubular D/T value, for instance, alters both its bending rigidity and its radial stiffness but not in proportion to each other. It would therefore not be possible to establish whether D/T, bending rigidity or radial stiffness was the important parameter from results pertaining to different D/T values. Summary calculations are presented in Appendix B which illustrate how bending and radial stiffnesses are related to D/T, and the logic behind the selection of the rather precise tube geometries (Models 8,9 and 12) and moduli (Models 4 and 5). Additionally, the modulus of the grout in Model 9 was lowered to preserve the radial stiffness of the base case (Model 1).

Model Number	L/D %	Studbolt Size	Studbolt Spacing (mm)	Flange Thickness (mm)	Saddle diameter/ thickness (mm)	Grout Annulus dia/thi (mm)	Grout Annulus D/T ratio	Tubular diameter/ thickness (mm)	Tubular D/T ratio	Additional Modifications of selected E values (N/mm^2)
1	37.5/62.5	M30	200	12	368/12	344/10	34.4	324/9.5	34.1	
2	37.5/62.5	M20.	200	12	368/12	344/10	34.4	324/9.5	34.1	
3	37.5/62.5	M42	200	12	368/12	344/10	34.4	324/9.5	34.1	
4	37.5/62.5	M30	200	12	368/12	344/10	34.4	324/9.5	34.1	E(tube) = 130750
5	37.5/62.5	M30	200	12	368/12	344/10	34.4	324/9.5	34.1	E(tube) = 367837
6	37.5/62.5	M30	200	12	368/12	344/10	34.4	324/9.5	34.1	E(flange) = 310800
7	37.5/62.5	M30	200	12	368/12	344/10	34.4	324/9.5	34.1	E(flange) = 447300
8	37.5/62.5	M30	200	12	368/12	344/25	13.8	294/7.855	37.4	
9	37.5/62.5	M30	200	12	368/25	318/12	26.5	294/7.855	37.4	E(grout)= 0.691
10	37.5/62.5	M30	150	12	368/12	344/10	34.4	324/9.5	34.1	
11	25/75	M30	200	12	368/12	344/10	34.4	324/9.5	34.1	
12	37.5/62.5	M30	200	12	368/12	344/25	13.8	294/13.35	22.0	

Notes:
1. All diameters refer to outside diameters
2. E(grout) refers to the reduction from the calibrated value
3. Model 1 corresponds to tests 13 and 14 and was the base case for calibration

Table 4.2.1: FE Modelling Programme Geometry Table

V 4.3 MODELLING

An illustration of the FE model used for in-plane bending analysis is given in Figure 4.1. Symmetry was used to assign boundary conditions at the centreline. The model for out-of-plane bending was very similar but was cut at the horizontal plane.

The model was generated by a programme written in the Patran Command Language for use with Patran 2.5. The analysis was done using the ABAQUS package. The tubular and the clamp were modelled in shell elements (8 node quad elements), the grout layer was modelled with solid elements (20 node brick elements), and the bolts were modelled with beam elements. Building the model from all solid elements was considered, a brief investigation suggested that this would be the most accurate method. The large number of nodes (more than 100000) which would be generated ruled out using solid elements for the full model.

A key modelling assumption was that there would be no separation of the grout from the clamp or the tubular and that there would be no slipping. Thus, no contact elements were used.

V 4.4 CALIBRATION

Model 1 of the FE programme, under in-plane and out-of-plane bending, simulates Tests 13 and 14 of the test programme. The FE/test results comparison for these loading cases are shown in Figures 4.2 and 4.3 respectively. It can be seen that the FE model captures the test results well, particularly for the out-of-plane bending case. The test results for the in-plane bending case are subject to error because of the very low magnitudes of loads involved.

Apart from meshing details, the only sensible variable quantity in the FE model is the grout modulus. The results in Figures 4.2 and 4.3 were obtained with a grout modulus of 14,000 N/mm^2, corresponding to a steel/grout modulus ratio of 14.6.

The global deflection behaviour has also been well represented, as shown by Table 4.4.1 below.

Gauge Position	Displacement (mm)			
	Test 13	FE Model	Test 14	FE Model
1	10.68	11.64	10.49	11.74
2	17.62	18.03	18.13	18.28
3	11.49	11.64	12.25	11.74
6	16.04	17.71	17.52	17.18

Table 4.4.1: Comparison of FE and test displacements

V 4.5 RESULTS

The variation of studbolt loads to applied moment obtained from the FE analyses are tabulated in Appendix C. Here, comparative distribution plots are presented, according to the parameter under investigation.

<u>Bolt Stiffness</u>

Models 1,2 and 3 incorporated M30, M20 and M42 studbolts respectively, but are identical in all other respects. Figures 4.4 and 4.5 show the studbolt distribution pattern for in-plane and out-of-plane bending respectively. As expected, the larger and the stiffer a studbolt is, the greater is the load attracted to it. The studbolt loads are not proportional to the studbolt stiffness as shown below:

Model/Studbolt	Relative Stiffness	Relative load IPB	OPB
Model 1/M30	1	1	1
Model 2/M20	0.44	0.76	0.74
Model 3/M42	2.00	1.24	1.24

This is because the studbolt stiffness only constitutes a relatively small part of the total (radial) stiffness of the clamp/tubular/studbolt system and, therefore, a significant change in studbolt stiffness does not affect total stiffness as much. This aspect is addressed further in Section V 5.

Clamp Rigidity

Figures 4.6 and 4.7 show the comparison of results of Models 1, 6 and 7 which differ in the flange stiffness value, for in-plane and out-of-plane bending respectively. It should be noted that the flange stiffness was altered in the FE analysis by specifying different E values, and not by changing flange thickness. The latter may have affected the side plate height which in turn would have affected clamp radial stiffness. The effective thickness of the flange (based on a constant E value) and the relative ratio of the second moment of area (for in-plane bending) of the two clamp halves acting as a composite member are as follows:

Model	Flange effective thickness	2nd moment of area ratio
1	12	1.0
6	20	1.48
7	30	2.13

Changing the flange stiffness, and by inference the overall bending rigidity of the clamp, appears to be a negligible parameter for in-plane bending, see Figure 4.6.

The corresponding plots for out-of-plane bending, Figure 4.7, shows some effect though of not great significance. There are two possible reasons for this effect. It is argued in Section V 5 that out-of-plane bending induces torsional components within the clamp. Firstly, changing the flange stiffness alters the position of the flexural centre of each clamp half and thus the eccentricity of the line of action of the out-of-plane force acting on each clamp half (i.e. the torsional component changes). Secondly, the torsional stiffness of the clamp steelwork (and in particular the closed section formed by the flange, side plates and saddle plate) is also modified. Both of these reasons are further addressed in Section V 5.2.2.

Saddle, Grout Annulus and Tubular Rigidity Ratios

Figures 4.8 to 4.19 show various plots where one or more ratio is changing; the even-numbered figures relate to in-plane bending and the odd-numbered figures to out-of-plane bending. The choice of which model results to show on the same plot is partly related to keeping either the radial or bending stiffness constant, see Appendix B.

In all cases, the in-plane bending results show a greater sensitivity to changing a stiffness parameter than do the out-of-plane bending data.

Position of Tubulars

Figures 4.20 and 4.21 respectively relate to the in-plane and out-of-plane bending results for different tubular positions.

Surprisingly, Model 11 (at the 25/75 position) maintains a high degree of symmetry about the clamp mid-length. Although the test results for grouted clamps (at the 37.5/62.5 position) show a greater degree of symmetry than the corresponding neoprene-lined clamps, it is considered that the degree of symmetry in the FE results is unreasonably high and may indicate a modelling deficiency (e.g. use of grouted nodes tied to steel nodes; no gap or sliding elements).

Bolt Spacing and Clamp Length

Model 10 relates to a shorter clamp (L = 1200mm) with a proportionate decrease in studbolt spacing (s = 150mm). The in-plane and out-of-plane bending results are given in Figures 4.22 and 4.23 respectively where they are compared with the results from Model 1. The substantial increased loads in the shorter clamp are approximately in accordance with expectations, see Section V 6.

V 4.6 CLOSURE

Greater attention is given to test results than the FE results, and particularly of the neoprene-lined clamp test results, in arriving at an understanding of clamp behaviour (Section V 5) and the subsequent development of the design methodology set out in Section V 6. Nevertheless, the good correlation of FE Model 1 with Tests 13 and 14, see Section V 4.4, gives sufficient confidence to use the FE results to test the design methodology, see Section V 6.4.

V 5 ASSESSMENT OF RESULTS

V 5.1 GENERAL

In the present context, there are two main issues that a designer has to address. Firstly, a minimum studbolt preload has to be specified to ensure that a clamp does not open up (i.e. separation of clamp halves) under applied loads. Secondly, for fatigue assessment of the studbolts, the load variation in the studbolts have to be estimated.

In order to examine these issues in the light of the results of this programme, it is useful to start with an analogy, i.e. the load behaviour of two Tee pieces bolted together. The analogy is illustrated in Figure 5.1 where it is assumed that the Tee pieces are so rigid that the classic prying forces normally associated with this type of connection can be neglected. If the bolts are not preloaded, then they take the full applied load as soon as it is applied. However, if the bolts are preloaded (to a value P), the applied load is initially resisted by both the bolts (which go into tension) and the faying surface (the precompression relaxing). During this regime, the studbolt load variation due to the applied load is very much less than the non preloaded case, the variation being dependent on the system stiffness (see Figure 5.1). This, of course, is why preloaded bolts are used in fatigue situations. When the precompression at the faying surface becomes totally relaxed, separation occurs and the bolts take all further increments of applied load.

A stressed (grouted or neoprene-lined) clamp under IPB behaves in a similar manner as the Tee connection, although the pertinent loads in this case are more obscure. Rather than thinking about the applied loads (i.e. the tubular bending moments) directly, it is necessary to consider the clamp internal load distributions resulting from the applied loads. Internal load distributions can be predicted by assumed mechanisms of clamp behaviour, and several of these (termed assessment models) are explored below. Substantially different internal load distribution effects apply for IPB and OPB load cases, as the test and FE results have indicated. Conceptually, it is easier to hypothesise about the mechanisms for generating and resisting the internal loads due to the IPB load case, and thus assessment models are developed for this case first.

Once internal load distributions resulting from the application of an external moment have been found, the apportionment between studbolt load variation and relaxation of the clamp precompression can be estimated. By analogy with the behaviour of the bolted Tee joint, Figure 5.1, this partitioning can be expected to be a function of the studbolt and clamp radial stiffnesses. This aspect is addressed in Section V 5.3 where the predictive capability of the assessment models are examined.

One point that should be clarified at this stage is the means by which the bending moment in the tubular member is transferred to the clamp. There are two mechanisms: by interface shear and by bearing forces, see Figure 5.2. It is only that part of the moment which is transferred by bearing that may be effective in causing variations in studbolt loads. In the discussion that follows, it is argued that for IPB, only a proportion of the bearing loads is effective in opening the clamp, whilst for OPB all bearing loads lead to studbolt load variation.

It may be gathered from this preliminary introduction that the behaviour of the clamp under IPB and OPB is substantially different and that there are a number of steps to follow through in understanding the mechanisms which give rise to studbolt load variation. It may therefore be helpful for the reader to turn first to the summary given in Section V 5.4 to gain an overall picture before going into the details in Sections V 5.2 and V 5.3.

V 5.2 ASSESSMENT MODELS

The assessment models which follow are intended to provide both an understanding of clamp behaviour and a basis for establishing design methodologies. Successive models are increasingly complex and, although of increasing accuracy, may not be suitable for design purposes.

Assessment Models 1 to 3 were conceptually developed for IPB (studbolt axes parallel to the plane of bending) and Assessment Models 4 and 5 for OPB.

V 5.2.1 <u>Models for IPB</u>

The essence of the IPB models is the estimation of internal distributed loads within the clamp which tend to separate the two clamp halves apart. The calculated distributed loads for each model are integrated over a bolt spacing and then halved (recognising that a pair of studbolts is contained within the spacing) to give resulting load values that can be assigned to a single studbolt. These values are presented in this Section V 5.2.1. It should be noted that the values will not be experienced by preloaded studbolts because of load sharing between the studbolt and clamp, and because a proportion of the applied moments are transferred by interface shear rather than by bearing. It is possible to consider the actual studbolt load variation to be given by:

$$P = F_1 F_2 P_M$$

where:

P = actual studbolt load variation

P_M = studbolt load variation calculated from an assessment model assuming full applied moment

F_1 = a factor representing that proportion of the applied moment which is transferred to the clamp by bearing forces and which tend to open the clamp up

F_2 = a factor accounting for load sharing between studbolt and clamp.

The factors F_1 and F_2 above are discussed further in Section V 5.3. In this Section V 5.2.1, all that needs to be noted is the shape of the studbolt load distributions and that, when the halves of the clamp separate, both F_1 and F_2 become unity.

Assessment Model Number 1

In this model, an internal load distribution in the clamp, in equilibrium with the applied moment, is assumed. The load in any given studbolt may then be calculated by integrating the internal load distribution over that length of clamp associated with the studbolt. This would normally be carried out for the critical studbolt pair at the end of the clamp for which the integration length is from the end of the clamp to a point midway between the first and second pairs of studbolts. This model is one traditional approach used for designing clamps.

The choice of the shape of the internal load distribution is somewhat subjective but a commonly assumed distribution is illustrated in Figure 5.3. The distribution and the resulting studbolt load is usually taken to apply to both IPB and OPB moments.

The calculated bolt load variation for each test, for the extreme bolt at the end of the clamp enclosing the shorter length of tubular, is given in Table 5.2.1 below. Also shown is the maximum measured studbolt variation (note in the tests, this did not necessarily correspond to the extreme studbolt). The results for Tests 6 and 7 are not included, as these may be obtained from moment resolution (see Section V 3.2.1) and neither are the tests with a missing studbolt.

As can be seen, the calculated studbolt load variation is very conservative. The model performs worst for tests where the studbolt axes are parallel to the plane of bending (IPB), even though, conceptually, the model should be more applicable. This conservatism is mainly due to the fact that the factors F_1 and F_2 are unity at this stage. This aspect is discussed more fully in Section V 5.3.

Test Number (and clamp type, load type)	Maximum Studbolt Load Variation (kN)	
	Calculated*	Measured
Neoprene - lined, IPB:		
1	178	6.7
3	78	3.0
5	156	3.9
9	156	3.1
Neoprene - lined, OPB:		
2	178	47.0
4	78	23.3
8	156	45.3
10	156	32.1
Stressed grouted, IPB:		
11	156	0.9
13	156	0.8
Stressed grouted, OPB:		
12	156	3.7
14	156	4.7

* P_M - see Section V 5.2

Table 5.2.1: Calculated (Assessment Model 1) and measured studbolt load variation

<u>**Assessment Model Number 2**</u>

As a refinement to Model 1, the bending stiffness of the clamp may be considered. The ratios of the second moments of area of the clamp to that of the tubular member are 9.4 and 7.5 for 0° and 90° clamp orientations respectively. For the purposes of developing the model, the clamp can initially be assumed to be so stiff compared to the tubular that each clamp half can be considered rigid. The effect of assuming the clamp to be rigid is that any relative movement between the clamp halves and thus the corresponding bolt extensions will vary linearly along the length of the clamp. The studbolt loads are therefore more favourably distributed compared to Model 1.

Two variants of the model (termed Model 2A and Model 2B), differing with respect to the assumed internal load distributions, are explored here. Model 2A is illustrated in Figure 5.4. For ease in developing the model, the point loads from the studbolts have been smeared to give a linearly varying distributed load. It is further assumed that the tubulars bear onto the clamp at discrete points. Four equilibrium equations can be written for the clamp/tubular system:

$$M = P_1 \ell_1 = P_2 \ell_2$$

$$P_1 + P_2 = \tfrac{1}{2}(w_1 + w_2)\, \ell$$

$$P_2\, \ell = w_2\, \ell^2 / 2 + (w_1 - w_2)\, \ell^2 / 6$$

$$(P_1 + P_2)\, \ell_1 = w_2\, \ell^2 / 2 + (w_1 - w_2)\, \ell^2 / 6$$

where:

$\ell = \ell_1 + \ell_2$ and all other symbols are defined in Figure 5.4.

These equilibrium equations can be solved to give explicit expressions for w_1 and w_2 in terms of the applied moment, M:

$$w_1 = \frac{2M}{\ell}\left[\frac{2}{\ell_1} - \frac{1}{\ell_2}\right]$$

$$w_2 = \frac{2M}{\ell}\left[\frac{2}{\ell_2} - \frac{1}{\ell_1}\right]$$

The distributed loading can be integrated to give studbolt point loads as in the previous model.

Test Number	Maximum Studbolt Load Variation (kN)	
	Calculated*	Measured
Neoprene - lined:		
1	45.8	6.7
3	19.3	3.0
5	38.6	3.9
9	38.6	3.1
Stressed grouted:		
11	38.6	0.9
13	38.6	0.8

* P_M - see Section V 5.2

Table 5.2.2: Calculated (Assessment Model 2) and measured studbolt load variation

Model 2B is illustrated in Figure 5.5 and is similar to Model 2A except that the loads applied to the clamp halves from the tubulars are not so localised. Similar equilibrium equations can be written and solved to give:

$$w_1 = \frac{3M}{\ell}\left[\frac{2}{\ell_1} - \frac{1}{\ell_2}\right]$$

$$w_2 = \frac{3M}{\ell}\left[\frac{2}{\ell_2} - \frac{1}{\ell_1}\right]$$

Inspection of these expressions for w_1 and w_2 with those for Model 2A above show that the magnitudes are increased by 50%, the pattern of studbolt loads remaining the same.

It may be observed that the tubular-to-clamp loads in Model 2B are assumed to exist only for compressive bearing forces. However, for a preloaded clamp, bearing forces essentially of a tensile nature could develop due to relaxation of the compressive preload. A third model variant (Model 2C) may therefore be suggested in which the distributed load from each tubular on each clamp half (q loads) consists of two triangular distributions - one tensile and one compressive. However, solving the associated equilibrium equations shows that both w_1 and w_2 are zero for this case. This result is generalised for clamps having finite stiffness under Model 3 and therefore this aspect is further discussed there.

Better agreement between calculated and measured maximum studbolt load variations are obtained with Model 2 than with Model 1. However, as far as the pattern of the distribution along the length of the clamp is concerned, that predicted (or rather assumed) in Model 1 is, on the whole, preferred to that predicted by Model 2.

There is still undue conservatism in the model with respect to the IPB tests. As noted in Model 1, this could be partly addressed by apportioning the distributed load between studbolt stretching and relief of preload. Again, this is an aspect discussed more fully in Section V 5.3.

Assessment Model Number 3

It is apparent from a comparison of the results given by Model 1 and Model 2 that clamp rigidity, and by inference bolt rigidity, have an effect on both the studbolt loads and the distribution patterns. In order to understand the behaviour of the clamp, it was considered necessary to include the effects of the rigidities into a model. Three variants of this model, differing with respect to the assumed internal load distribution between the tubular members and the clamp, may be considered. These variants are numbered 3A, 3B and 3C and are depicted in Figure 5.6.

Model 3C assumes that the bending moment in each tubular is transferred to each clamp half by linearly varying distributed loads. Furthermore, it is assumed that the distributed load is split equally between the upper and lower clamp halves by virtue of further compression or relaxation of the annular material. The resulting deflected profiles of the two clamp halves are identical and thus the distance between the halves remains constant along the length of the clamp. It may therefore be concluded that the assumed load distribution of Model 3C does not lead to a variation of studbolt load.

It was observed in Section V 3.3 that the strains in the tubulars were quite linear with applied moment, at least for the IPB tests. It was inferred that the distributed loads were localised (i.e. point loads would be a fair approximation). However, the distributed loads assumed in Model 3C (and Model 3B) lead to a moment decay in the tubular member which can be closely approximated by the linear decay obtained from point loads, see Figure 5.7. The closeness of the two moment decay curves makes it difficult to ascertain from test data the proportion of IPB transferred by point loads and the proportion by distributed loads, and thus how much of the IPB is actually effective in tending to separate the two clamp halves.

Further discussion is now confined to Models 3A and 3B, see Figure 5.6, i.e. the case where the IPB is transferred to the clamp by point or locally distributed loads. Although in principle the models could be analysed by hand calculation (consideration can be given to smearing the bolt stiffness and using 'beams on elastic foundation' methods), analysis was done by FE modelling. The total stiffness assigned to each pair of M30 studbolts was 384 kN/mm and the second moment of area of one clamp half, about its neutral axis parallel to the split, was calculated as 74.5 x 10^6 mm^4.

It was confirmed numerically that the same spring loads are derived if the ratio of spring stiffness (K) to clamp bending stiffness (EI) is preserved. Defining the ratio R (=K/EI) as unity with the above stiffnesses taken in arbitrary units, additional analyses were undertaken over a range of R values from 0.2 to 10. This enable the sensitivity of the results to R to be explored as well as to facilitate the discussions in Section V 5.3. Additionally, the case of a completely rigid clamp (R = 0.0), corresponding to Models 2A and 2B, can be compared with the numerical results.

It should be noted at this stage that the spring stiffness corresponds to the summation of studbolt stiffness and clamp internal stiffness (due to preload, see Figure 5.1). Thus R = 1.0 relates to the loading regime after clamp half separation has occurred. Before then, the effective R will be somewhat higher than unity and the resulting load variations shown in Figures 5.8 to 5.12 should be shared between studbolts and clamp preload relaxation. The load variation in these figures have been integrated over a distance corresponding to the bolt spacing and then halved to give forces that can nominally be assigned to individual studbolts (plus associated clamp region).

Figure 5.8 shows the calculated load variations, along the length of clamp, for applied point loading (Model 3A) when the tubulars are at the 37.5/62.5 position. The corresponding plots for the more diffuse applied loading (Model 3B) are given in Figure 5.9. Comparing these two figures, it can be seen that the effect of the R ratio is diminished in the case of Model 3B, although the maximum load variation is greater, particularly as R tends to zero. The load variation patterns are broadly similar for the two load cases, and which may be described as a shallow lop-sided 'w' distribution.

Figure 5.10 and 5.11 show the equivalent plots to Figure 5.8 (i.e. applied point loading, Model 3A) for tubular positions 25/75 and 50/50 respectively. Taking the figures in order of increasing symmetry of tubular position (i.e. Figure 5.10, Figure 5.8 and Figure 5.11), it can be seen that the effect of R increases and that the w-shape becomes better defined. For the 50/50 tubular position, Figure 5.11, the critical studbolt positions still correspond to the outermost pairs.

The effect of R and tubular position on the maximum load variation is summarised in Figure 5.12. Clearly the tubular position is the more important parameter but this will be defined by the clamp configuration. As noted above, R will be unity (for M30 studbolts) and the load variation will be the studbolt load variation only after the clamp halves have separated. Before separation, R will be greater than unity and the load variation has to be shared between studbolts and change of clamp preload.

The comparison of Model 3 with test data is given in Section V 5.3.1.

V 5.2.2 Models for OPB

Assessment Model Number 4

This model deals exclusively with clamps having studbolt axes orthogonal to the plane of bending, i.e. clamps under OPB moments.

The relevant test data indicate that the studbolts in any given pair have equal but opposite loads. This is indicative of torsional effects rather than the prising effects associated with clamps of 0° orientation. Figure 5.13 illustrates how torsion on each clamp half may arise. Firstly an assumed radial stress pattern can be assumed, Figure 5.13(a). (This assumption is fully justified for neoprene lined clamps because the radial compressive stiffness of the neoprene is very much greater than its circumferential shear stiffness.) The vertically resolved stress components (as drawn) are equal but of opposite sense in each quadrant and, with the calculated lever arm, give rise to one component of torsion, Figure 5.13(b). A second torsion component is found from the eccentricity of the horizontal resultant with respect to the shear (or flexural) centre of the clamp half. (The shear centre is calculated as the centroid of the flexural stiffnesses of the steelwork components; thus the distance between an arbitrary datum and the shear centre is $\Sigma EIy/\Sigma EI$.) Calculations based on the above approach reveal that the distributed torque per unit length on each clamp half is:

$$T = 0.25\,DQ$$

where

Q is the total (both clamp halves) internal distributed bearing loading per unit length. For the point load case, only the section at which the point loads are applied should be considered.

D is the internal diameter of the saddle plate.

Note the above torque is only valid for the specific test clamp geometry considered herein. The torque is resisted by torsion of the clamp half, circumferential shear stresses induced in the neoprene or grout, and by couples from studbolt pairs. Supplementary calculations show that the resistance afforded by studbolt pairs is much greater than the clamp torsional resistance but less than the resistance from circumferential shear stressing.

A mechanistic model of how the clamp works is illustrated in Figure 5.14(a). For clarity, only one studbolt pair and associated flange plate area is shown. Likewise, the internal torques are shown as being applied at discrete locations and not distributed. A consideration of this model would indicate that the behaviour of the two clamp halves are mirrored, and therefore the tubular members are not subject to any torque and do not twist relative to each other. It follows that the tubular members may just as well be continuous in respect of providing a reaction base for the neoprene/grout circumferential stresses. It may also be observed that the torque is resisted by springs which act in parallel, which may therefore be combined. The mechanistic model therefore suggests that the simplified FE model shown in Figure 5.14(b) should be adequate. In this model the tubular member is treated as a rigid base, the torsional springs represent the combined stiffness arising from the studbolts and circumferential straining of the annular material, the single line represents the torsional stiffness of one clamp half, and the magnitudes and the disposition (point or distributed) of torques T_1 and T_2 are calculated from internal bearing loads as above.

In order to test the above reasoning, analyses were conducted with the FE model consisting of a line of torsional elements with torsional springs at the nodes. The torsional elements were assigned a torsional constant calculated from the enclosed compartment within the half clamp ($J = 35.6 \times 10^6$ mm^4). In a similar manner to the IPB Model 3, a spring to clamp stiffness ratio R (=K/GJ) may be defined. For OPB, the ratio R is defined as unity for the case of the torsional spring stiffness derived from M30 studbolts and the above torsional constant (the stiffnesses being taken in arbitrary units).

Figure 5.15 compares the calculated load variation (assignable to a single studbolt and equivalent associated clamp region) for the cases where the OPB moment (and hence the applied torque) is assumed to be either distributed or applied at points. As can be seen, there is little difference between the two. However, the test data would indicate that the smoother results of the distributed load assumption is to be preferred and thus this is used hereafter.

The calculated load variations for clamp/tubular position of 25/75 and 50/50 are shown in Figures 5.16 and 5.17 respectively. The maximum load variation occurs at the extreme studbolt position on the shorter tubular.

The effect of changing the spring/clamp torsional rigidity ratio R for the 37.5/62.5 position is indicated in Figure 5.18. The load variation does not change significantly except by a scalar quantity, the pattern remaining the same. The maximum load variation is plotted against R in Figure 5.19. The curve has been taken back to the origin because at zero spring stiffness, all the torque is resisted by torsion of the clamp half. The load variation reaches a plateau at large values of R, and is more horizontal than is the case for IPB Model 3.

<u>**Assessment Model Number 5**</u>

In Model 4, it was observed that as R varies, the pattern of the load distribution pattern remains fairly constant and that the magnitude of the load variation quickly reaches a plateau as R increases. This suggests that a suitable approximation may be found by ignoring the clamp torsional stiffness (i.e. equivalent to letting R tend to infinity in Model 4). The load variation is found simply by integrating the applied torque distribution curve over the studbolt spacing and calculating the studbolt force to give the necessary resisting torque. This approach results in the following expression for the maximum studbolt (plus associated clamp region) load variation:

$$P = 0.25 \times \frac{6MDs}{B\ell_1^{\,2}} \left(1 - s/\ell_1\right)$$

where

M = tubular OPB moment

D = inner diameter of clamp saddle plate

s = studbolt spacing

B = transverse distance between studbolt rows

ℓ_1 = shorter length of enclosed tubulars

The factor 0.25 is specific to the geometry of the test clamp (see discussion under Model 4 above).

The predictions of the above expression can be compared to the results of Model 4, as tabulated below:

Clamp Position	Model 4 results		Model 5 results
	R = 1.0	R = 10	
25/75	59.5	88.5	102.6
37.5/62.5	43.5	56.2	60.8
50/50	30.7	36.6	38.4

All results are for a single studbolt position, kN

As can be seen, the Model 5 results are a good approximation to the Model 4, R = 10, results. This agreement is not unexpected because a high R value implies the torsional rigidity of the clamp is small compared to the torsional resistance afforded by the studbolts and this is precisely the basis of Model 5. The Model 5 results are a conservative approximation to the Model 4, R = 1.0, results. The conservatism reduces for increasing symmetry of clamp/tubular position. This is because the effect of R reduces for increasing symmetry.

V 5.3 COMPARISON OF MODELS AND TEST DATA

The adequacy of the assessment models developed in Section V 5.2 to explain observed clamp behaviour is now examined. Again, it is appropriate to discuss IPB and OPB behaviour separately.

V 5.3.1 IPB Behaviour

V 5.3.1.1 Load variation pattern

The load variation patterns predicted by Assessment Models 1 to 3 for applied IPB are shown in Figure 5.20. The loads have been normalised to give a value of unity at the most heavily stressed studbolt. The distributed loads have been integrated (over the studbolt spacing) and then halved to present the load in a single studbolt were that studbolt to attract all the load (as opposed to sharing the load between the studbolt and relaxation of preload). This integration process results in a third line, in the vicinity of the abutting ends of the tubulars, for Model 1 (compare the studbolt distribution pattern of Figure 5.20 with the assumed internal load distribution of Figure 5.3).

The load variation patterns can be compared to test data as follows:

Pattern in Figure 5.20a : (clamp position 37.5/62.5)	Test 3 - Neoprene, M30 (Figure 3.3)
	Test 5 - Neoprene, M30 (Figure 3.5)
	Test 9 - Neoprene, M24 (Figure 3.11)
	Test 11 - Grout, M24 (Figure 3.13)
	Test 13 - Grout, M30 (Figure 3.15)
Pattern in Figure 5.20b : (clamp position 25/75)	Test 1 - Neoprene, M30 (Figure 3.1)

The load variation pattern of Model 3A captures the essential elements of the test patterns apart from Test 3 and 9. The characteristic W-shape of Tests 5,11 and 13 is reflected in the predicted results and the degree of correlation could be improved by selecting a different value of R (ratio of 'studbolt' stiffness to clamp bending rigidity), see Figure 5.8. The pattern of Test 1 is predicted well, with negative (compressive) values for the load variation being manifested in both the test and the model.

As mentioned above, rather poorer agreement is obtained for Tests 3 and 9, which are very similar to each other. The evidence would suggest that the studbolt preload had a role to play. Test 3 is similar to Test 5 but for a lower preload (and applied moment) and Test 9 is also similar to Test 5 but for a smaller size of studbolt (and hence preload again).

V 5.3.1.2 Magnitude of studbolt loads

A comparison of maximum studbolt load variations between Assessment Models 1 to 3 and test data is given in Table 5.3.1 below.

Test No.	Description (type, position, studbolt, M)	Model 1	Model 2	Model 3 R = 0.2	Model 3 R = 10	Test Data
1	Neoprene, 25/75, M30, 95	178	45.8	56.5	76.8	6.7
3	Neoprene, 37.5/62.5, M30, 70	78	19.3	23.0	33.5	3.0
5	Neoprene, 37.5/62.5, M30, 140	156	38.6	45.9	67.0	3.9
9	Neoprene, 37.5/62.5, M24, 140	156	38.6	45.9	67.0	3.1
11	Grouted, 37.5/62.5, M24, 140	156	38.6	45.9	67.0	0.9
13	Grouted, 37.5/62.5, M30, 140	156	38.6	45.9	67.0	0.8

Table 5.3.1: Calculated (no load sharing) and measured maximum studbolt load variations (kN) for IPB

The above calculated values are clearly one or two orders greater than the test values and there are three possible reasons for this:

i) A proportion of the applied moment actually enters the clamp by the shear transfer mechanism, see Figure 5.2, and this proportion does not lead to studbolt load variation in the tests.

ii) Part of internal bearing load distribution is applied equally to the two clamp halves, see Figure 5.6c, and this part again does not lead to load variation in the tests.

iii) The calculated values reflect the load that has to be shared between a single studbolt and the associated area of clamp (by relaxation of the compression induced by initial preload).

It is not possible from the experimental data, to distinguish between items (i) and (ii). Nevertheless, it would seem that item (ii) is active but forms a variable component because of the non-linear studbolt load response with applied moment, see top diagrams in Figures 3.10 and 3.24. Non-linearity does not occur in the case of OPB (lower diagrams of cited figures) as all internal bearing loads lead to studbolt load variation and therefore item (ii) does not apply.

However, it is considered that item (iii) is dominant and this may be inferred from the large relative difference between the test results for neoprene-lined clamps and grouted clamps, and from an appraisal of OPB data in which a bound can be placed on the role of item (i). Unfortunately, item (iii) is not easily amenable to hand calculation.

V 5.3.2 <u>**OPB Behaviour**</u>

V 5.3.2.1 <u>Load variation pattern</u>

The load variation patterns predicted by Assessment Models 4 and 5 for applied OPB are shown in Figure 5.21. As for IPB, loads are again normalised to give a value of unity at the most heavily stressed studbolt. The predicted patterns can be compared to test data as follows:

Pattern in Figure 5.2/a: (clamp position 37.5/62.5)	Test 4 - Neoprene, M30 (Figure 3.4)
	Test 8 - Neoprene, M30 (Figure 3.8)
	Test 10 - Neoprene, M24 (Figure 3.12)
	Test 12 - Grout, M24 (Figure 3.14)
	Test 14 - Grout, M30 (Figure 3.16)
Pattern in Figure 5.21b: (clamp position 25/75)	Test 2 - Neoprene, M30 (Figure 3.2)

In general, Model 4 gives a good prediction of the patterns obtained from the neoprene clamp tests. Even better agreement would have been obtained in certain regions had point torques, rather than distributed torques, been used to construct Figure 5.21. The difference between these applied torques is illustrated in Figure 5.15 where, for example, the slope of the pattern at the ends of tubulars for the point torque case reflects better that observed. The application of the point torque case in Model 5 (which ignores the longitudinal dispersion of torque via clamp box torsional stiffness) would lead to nonsensical results.

The predicted patterns do not accord well with the test data (or the FE results) for stressed grouted clamps. For these clamps, the internal torques are resisted almost entirely by circumferential shear straining of the grout, and only a very small proportion of the torque is left to cause studbolt load variation. The high relative stiffness of the grout compared to that of the studbolts to resist torque certainly affects the distribution pattern, as may be inferred from Figure 5.18 (e.g. compare the line slopes in the region of the ends of the tubulars). However, the poor agreement may be more germane of second order effects not considered in the Assessment Models and which only become apparent at the low levels of studbolt load variations associated with grouted clamps.

V 5.3.2.2 Magnitude of studbolt loads

A comparison of maximum studbolt load variations between Assessment Models 4 and 5 and test data is given in Table 5.3.2 below. The Model 4 results are quoted for what is considered to be the most apt value of R (R = 0.6 and 1.0 for M24 and M30 studbolts respectively), the result for R = 0.6 being found by interpolation.

Test No.	Description (type, position, studbolt, M)	Model 4		Model 5	Test Data
		R = 0.6	R = 1.0		
2	Neoprene, 25/75, M30, 95	-	40.4	69.6	47.0
4	Neoprene, 37.5/62.5, M30, 70	-	21.8	30.4	23.3
9	Neoprene, 37.5/62.5, M30, 140	-	43.5	60.8	45.3
10	Neoprene, 37.5/62.5, M24, 140	34.9	-	60.8	32.1
12	Grouted, 37.5/62.5, M24, 140	34.9	-	60.8	3.7
14	Grouted, 37.5/62.5, M30, 140	-	43.5	60.8	4.7

Table 5.3.2: Calculated (no load sharing) and measured maximum load variations (kN) for OPB

Much closer agreement is obtained between predicted and test values for OPB than IPB (Table 5.3.1). For neoprene clamps, the agreement is particularly good. Supplementary calculations indicate that the resistance to torque afforded by circumferential straining of the neoprene is small and therefore the great majority of the torque leads to studbolt load variations. This is the basis of Models 4 and 5 in that no load sharing (between the annular material and the studbolts) is taken into account. Even better agreement can be obtained if the non-symmetry in the test data is considered. For instance, the Test 2 data shown in Figure 3.2 shows the studbolts in the extreme pair have load variations of 47kN (as indicated in the above table) and 27kN, giving an average value of 37kN.

The results for the stressed grouted clamp indicate that load sharing is important, with most of the torque being resisted by circumferential straining of the grout. This can be confirmed, albeit only approximately, by calculation in which the relative torsional stiffness afforded by the annular material and the studbolts is considered.

The fact that good correlation is obtained for neoprene-lined clamps, would suggest that practically all moments are transferred by the bearing mechanism, with little by interface shear, see Figure 5.2. This may not, however, apply to stressed grouted clamps.

V 5.4 SUMMARY

The discussions in the previous subsections of this Section V 5 have identified the essential elements of clamp behaviour under IPB and OPB. There are three primary steps involved in quantifying studbolt load variation:

- Establish the moment transfer mechanism from the tubular into the clamp. In the main this will be by radial bearing forces between the tubular and clamp.

- Use a model to establish the manner by which the clamp/studbolt system resists the above bearing loads.

- Apportion load share between the studbolts and other parts of the clamp system.

Although these three steps are common to IPB and OPB, the behaviour of the clamp system is quite different under these load cases. It is therefore appropriate to deal with them separately.

V 5.4.1 Clamp Behaviour Under IPB

A flowchart summarising the interrelationship of the effects leading to variations of studbolt load for clamps under IPB is given in Figure 5.22.

Of the two ways by which the moment in the tubular is transferred to the clamp (see Figure 5.2), it is only the bearing mechanism that may lead to studbolt load variation. Furthermore, only that portion of the bearing loads which are applied differently on the two clamp halves are effective (similar loads cause the same deflected profile in the two clamp halves which, in turn, does not affect the distance between the halves and does not lead to changes of length in the studbolts). Note that when separation of the clamp halves does occur, and all the tubular moment is transferred by point loads (e.g. see Figure 5.4), all of the applied IPB becomes effective in causing studbolt load variation.

Having established the localised (point) bearing loads acting on each clamp half, one of the assessment models described in Section V 5.2 can be used to determine the resisting (or reaction) loads. The models vary in complexity according to how well the effect of clamp half bending rigidity is captured. Finally, the resisting load is shared out between studbolt loads and a variation in the clamp-to-tubular preload according to relative stiffnesses. Again, when separation of the clamp halves does occur, the studbolts alone carry the resisting load (see the analogy in Figure 5.1).

The two loading regimes, i.e. before and after clamp half separation, should be fully recognised as very different behaviour applies to each. Fortunately, it is easier to consider the behaviour after (or at the point of) separation because all of the IPB is effective and no load sharing has to be considered. This makes the estimation of studbolt load variation more certain and this is important as the maximum variation defines the minimum preload required.

V 5.4.2 Clamp Behaviour Under OPB

The corresponding flowchart for clamps under OPB is given in Figure 5.23. As in the case of IPB, only that portion of the OPB moment that is transferred to the clamp by bearing forces is effective. However, as opposed to the IPB behaviour, all bearing forces eventually lead to studbolt load variation.

Whereas bearing forces tend to separate the two clamp halves in the case of IPB, this is not so for OPB. Rather, OPB bearing forces lead to an applied torque distribution on each clamp half due to the eccentricity of the bearing forces with respect to the shear centre of the clamp half. The distributed torques are of opposite sense in the two halves (and therefore leads to tension in one studbolt and compression in the other for any one pair). The distributed torques are partly resisted by torsion of the closed box section in each clamp half and partly by the combination of circumferential straining of the annulus material and studbolt loads acting over a lever arm. The former mechanism (clamp half torsion) resists the applied torques by providing a load path longitudinally and the latter mechanism (combination of circumferential shear and studbolts) is active at each cross section. The latter mechanism is the more dominant.

It may be perceived that there is one further fundamental difference between IPB and OPB behaviour. The fact that OPB does not tend to cause clamp half separation means that studbolt load variation is always directly related to the magnitude of the OPB moment, and is not affected by preload. There is therefore only a single regime of loading for OPB as opposed to the two regimes (before and after clamp half separation) for IPB.

V 6 DESIGN RECOMMENDATIONS

V 6.1 GENERAL

The development of the design recommendations set out below is based on the understanding of clamp behaviour gained from the test and numerical data, as discussed in Section V 5. Because of fundamental differences in IPB and OPB behaviour, these are treated separately below and, in general, their effects have to be numerically summed to identify the critical studbolt load combination.

As always, IPB and OPB moments are defined here relative to the clamp split line. IPB moments cause member deflections in a plane parallel to the axes of the studbolts, i.e. orthogonal to the clamp split plane. Conversely, the plane of bending for OPB moments is the same as the clamp split plane.

Two distinct regimes of behaviour have been identified, at least as far as IPB is concerned, these relating to the behaviour before and after clamp half separation. It is important, for the design of the minimum studbolt preload, to find the point at which separation will occur as it is essential to prevent such separation during service. Fortunately, this is relatively easy to predict and the design formulation in Section V 6.2 is reasonably robust. No separation occurs in the case of OPB but, nevertheless, studbolt load variation does occur and this has to be taken into account when estimating the minimum preload. Once again, the OPB rules are robust.

The only time when it may be wished to consider the behaviour before clamp half separation is for assessing the fatigue loads on studbolts. The test results have shown that both IPB and OPB moments give rise to very small studbolt load variations (i.e. less than 1% of the studbolt yield load) for a stressed grouted clamp, and hence fatigue is most unlikely to present any problem for such clamps. Fatigue could, however, be a conceivable criterion for designing the studbolts of a stressed neoprene-lined clamp. It is again fortunate that the most dominant cause of studbolt variation is OPB moment, for which the design rule is more robust (as compared to the case of IPB before clamp half separation).

The design formulations are a balance between simplicity and accuracy. Certainly they are suitable for hand calculations. It should be noted that the actual distribution (pattern) of the variation of studbolt loads is of little interest in design; what is important is to estimate the maximum variation of all studbolts as this will determine a lower limit to the preload that is generally applied to all studbolts. Neither is great accuracy in estimating the maximum variation of any benefit in the majority of clamp designs. This is because preload also governs the slip capacity of the clamps and this criterion will now assume a greater role than hitherto given that the recommendations below suggest that the effect of clamp prying forces are not as important as previously thought.

V 6.2 STUDBOLT LOAD VARIATION DUE TO IPB

The maximum load variation of any studbolt due to IPB moments in a tube-to-tube clamp may be estimated from the following expression:

$$P = \Gamma_i F_i P_{Mi}$$

where:

Γ_i = Partial factor of safety to account for model uncertainty (=2 for static preload, = 1.0 for fatigue)

F_i = F_1F_2, where

- F_1 = A factor representing the proportion of the applied IPB moment that is transferred to the clamp by bearing forces (see Figure 5.2b) which tend to open the clamp

- F_2 = A factor accounting for load sharing between studbolts (which stretch) and the clamp body (which experiences a relaxation of preload compression)

P_{Mi} = The maximum load variation of any studbolt calculated from an Assessment Model assuming the applied IPB moment is fully transferred by bearing and ignoring any load sharing.

For estimating preload requirements for either neoprene-lined or grouted clamps, F_i should be taken as unity. It is recommended that P_{Mi} is established by assuming the clamp halves to be rigid (i.e. as in Assessment Model 2, see Section V 5.2.1). To account for the practical range of the ratio of studbolt stiffness to clamp rigidity, the partial factor of safety, Γ_i, should then be taken as 2.0, although this could be reduced following a more detailed assessment. It should not be reduced to a value below 1.5 say, to ensure that even partial separation does not occur.

For a clamp connecting two tubulars, such as used in the test programme, the following equation for P_{Mi} can be derived:

$$P_{Mi} = \frac{M_i \, x/\ell}{2} \left(\frac{4 - 3x/\ell}{\ell_1} - \frac{2 - 3x/\ell}{\ell_2} \right)$$

where:

M_i = in-plane bending moment

x = the distance from the end of the clamp to a point midway between the first and second pair of studbolts

ℓ_1 = shorter enclosed length of tubular

ℓ_2 = longer enclosed length of tubular

ℓ = $\ell_1 + \ell_2$

When assessing studbolt fatigue loads due to in-plane bending of the tubular member, the above equations apply except that the factor F_i is no longer unity. The test programme has shown that F_i is variable and is significantly different for neoprene-lined and stressed grouted clamps. The following values are conservative:

F_i = 0.2 for neoprene-lined clamps

= 0.05 for stressed grouted clamps.

Furthermore, the partial factor of safety may be taken as unity for fatigue assessment.

V 6.3 STUDBOLT LOAD VARIATION DUE TO OPB

The maximum load variation of any studbolt due to OPB moments in a tube-to-tube may similarly be estimated from the following equation:

$$P = \Gamma_0 F_0 P_{M0}$$

where:

Γ_0 = Partial factor of safety to account for model uncertainty (= 1.0 in all cases)

F_0 = F_3F_4, where:

F_3 = A factor representing the proportion of the applied OPB moment that is transferred to the clamp by bearing forces which lead to a torsional moment being applied to each clamp half

F_4 = A factor accounting for sharing of the internal torsional moment between studbolts (one studbolt stretching, the other contracting, for any given pair of studbolts) and annulus material

P_{M0} = The maximum load variation of any studbolt calculated from an Assessment Model assuming the applied OPB moment is fully transferred by bearing and ignoring any load sharing.

As opposed to the case of IPB moment, OPB moment by itself does not cause separation of the two clamp halves. (Rather, at any cross section, OPB moment causes rotation (twisting) of each half circumferentially about the tubular axis with the rotations of the halves being in opposite senses.) There is no need, therefore, to define two values of F_0 for preload and fatigue calculations respectively; assuming, of course, that the clamp is designed never to separate (under IPB moment).

For simplicity, Assessment Model 5 (see Section V 5.2.2) is chosen as a basis for formulating a design rule. This assumes all internal torsional moments are resisted by the studbolts alone and therefore it is always conservative. The expression in Section V 5.2.2 has been adjusted (by substituting D by 0.75 H) to make it more general for application to other geometries and simplified to:

$$P_{M0} = \frac{M_0 \, Hx}{B \, \ell_1^2} (1 - x/\ell_1)$$

where:

M_0	=	out-of-plane bending moment
H	=	overall height of clamp measured between outer surfaces of flange plates
B	=	transverse distance between studbolt rows
x	=	the distance from the end of the clamp to a point midway between the first and second pair of studbolts
ℓ_1	=	shorter enclosed length of tubular

The following values of F_0 are recommended:

F_0 = 1.0 for neoprene-lined clamps

= 0.2 for stressed grouted clamps.

The factor of safety, Γ_0, may be taken as unity.

V 6.4 COMPARISON OF DESIGN RECOMMENDATIONS WITH TEST AND NUMERICAL DATA

The efficacy of the above recommendations for a tube-to-tube clamp is demonstrated in the following tables. The design values have been established with the partial factors of safety set to unity and the factor F_i for the IPB case taken as appropriate to fatigue assessment. The latter is applicable because clamp half separation was not observed during the tests.

Test. No	Description	IPB		OPB		Combined	
		Test	Design	Test	Design	Test	Design
1/2	Neoprene, 25/75, M30, 95	6.7	9.2	47.0	62.1		
3/4	Neoprene, 37.5/62.5, M30, 70	3.0	3.9	23.3	27.1		
5/8	Neoprene, 37.5/62.5, M30, 140	3.9	7.7	45.3	54.2		
6	Neoprene, 37.5/62.5, M30, 140	-	6.7	-	27.1	24.4	33.8
7	Neoprene, 37.5/62.5, M30, 140	-	3.9	-	46.9	39.4	50.8
9/10	Neoprene, 37.5/62.5, M24, 140	3.1	7.7	32.1	54.2		
11/12	Grouted, 37.5/62.5, M24, 140	0.9	1.9	3.7	10.8		
13/14	Grouted, 37.5/62.5, M30, 140	0.8	1.9	4.7	10.8		
15/16	Grouted, 37.5/62.5, M30, 140	0.9	1.9	4.8	10.8		

Notes:

1) Tests 6 and 7 were conducted with combined in-plane and out-of-plane moments.
2) Tests 15 and 16 were conducted with one studbolt deliberately removed.
3) All studbolt loads are given in kN.

Table 6.4.1: Comparison of maximum studbolt load variations recorded in tests with design values for fatigue assessment

FE Model No	Description[1]	IPB		OPB	
		FE	Design	FE	Design
1	Base case[2]	0.36	1.9	4.72	10.8
2	M20 Studbolt	0.27	1.9	3.51	10.8
3	M42 Studbolt	0.44	1.9	5.83	10.8
4	E (tube) lower	0.41	1.9	4.66	10.8
5	E (tube) higher	0.30	1.9	4.63	10.8
6	E (flange) higher	0.37	1.9	5.01	10.8
7	E (flange) highest	0.37	1.9	5.50	10.8
8	Small tubular, thicker grout	0.15	1.9	4.43	10.8
9	Thicker saddle	0.17	1.9	3.86	10.8
10	Studbolt spacing = 150mm, ℓ = 1200mm	1.02	2.6	8.09	14.5
11	Tubular position 25/75	0.40	3.4	5.06	18.3
12	As No. 8 but thicker tubular	0.16	1.9	4.16	10.8

Notes:

1) For a full description of geometries, see Table 4.2.1
2) Tubular position 37.5/62.5; M30 studbolt, clamp as per tests
3) All studbolt loads are given in kN.

Table 6.4.2: Comparison of maximum studbolt load variations from FE analyses with design values for fatigue assessment

Inspection of the tables shows that the design values are always conservative. It may be thought that the IPB design values are relatively rather high compared to both the test and numerical data and indeed they are. However, they are still small in absolute terms and therefore this is not of practical significance for fatigue assessments.

Of greater importance is the establishment of preload where the IPB design values shown in the tables are to be multiplied by five for neoprene-lined clamps or 20 for stress grouted clamps (i.e. the reciprocal of the factor F_i) and then multiplied by the partial factor of safety (= 2.0 for IPB). The resulting preload values represent a significant reduction of more traditional approaches.

V 6.5 EXTENSION OF DESIGN RECOMMENDATIONS TO NODAL CLAMPS

So far, all the discussions and design recommendations have been made with respect to tube-to-tube clamps. In practice, however, clamps are more commonly employed around nodal joints of the structure. It is the purpose of this Section V 6.5 to formulate design recommendations for such nodal clamps based on the philosophy and understanding developed for tube-to-tube clamps. Whilst the experimental or numerical data do not exist to verify the recommendations below, they are believed to be sensibly conservative.

Three basic assumptions are made:

- each brace connection in a nodal clamp can be treated separately and therefore any clamp can be analysed as a number of Y joints,
- the brace to chord weld has completely failed and therefore the entire brace moment has to be transferred to the chord via the clamp,
- the values of the factor F (accounting for transfer mechanism and load sharing) and the partial factors of safety given in Sections V 6.2 and V 6.3 for tube-to-tube clamps also apply to nodal clamps.

As before, IPB and OPB moments are defined in terms of the orientation of the plane of bending relative to the studbolt axes (and not to the plane of the joint).

V 6.5.1 Clamp Under IPB

The design recommendations for a tube-to-tube clamp under IPB rely on assessing the studbolt loads from a rigid clamp model. A similar philosophy is adopted for a nodal clamp in that the clamp halves are again considered rigid, at least for the section along the centre-line of the brace.

Figure 6.1 illustrates a plan view and a cross-section along the brace centre-line for a typical nodal clamp. The dimension l_2 in this figure can be approximated as $D/\sin\theta$. Note that l_1 is not necessarily smaller than l_2 (as opposed to the tube-to-tube clamp where l_1 should always be taken as the smaller dimension).

The cross-section in Figure 6.1 shows the loads which are assumed to act on each clamp half. P_1 is the value of the point bearing loads applied by the brace due to the applied moment M_i. M_1 and M_2 are resisting torques transferred from the chord to the clamp halves and oppose the applied moment. These resisting torques are required to maintain equilibrium but do not have to be quantified in the analysis below. The distributed load (w) on each clamp half represents the smeared studbolt loads on the brace part of the clamp. It may be noticed that no advantage has been taken of the potential contribution to the distributed load w from the 'chord studbolts' adjacent to the brace part of the clamp. It is now assumed that when the clamp begins to separate, one or both halves will rotate about the chord axis. As both clamp halves are assumed to be rigid, it follows that the load w is linearly distributed with a projected value of zero at the chord axis. The relationship between w_1 and w_2 is therefore:

$$w_2 = w_1 \quad \frac{l_2}{2} \Big/ \left(\frac{l_2}{2} + l_1 \right)$$

Brace moment equilibrium and vertical equilibrium of either of the clamp halves gives:

$$M_i = P_1 l_1$$

$$P_1 = \tfrac{1}{2}\,(w_1 + w_2) l_1$$

Solving the above three equations for w_1 in terms of M_i supplies the following relationship:

$$w_1 = \frac{2M_i}{l_1^{\,2}} \left(\frac{l_1 + l_2 / 2}{l} \right)$$

where $l = l_1 + l_2$

The value of w at any point can be found by interpolation. Integrating over the distance from the brace end of the clamp to a point midway between the first and second pair of studbolts (ie. the distance x in Figure 6.1), and then halving the result (to account for there being a pair of studbolts), the following result for the load in the critical studbolt is obtained:

$$P_{Mi} = \frac{M_i\, x / l}{2 l_1^{\,2}} (l + l_1 - x)$$

All lengths in the above equation are shown in Figure 6.1. Finally, the maximum studbolt load variation, P, may be found according to the following expression:

$$P = \Gamma_i\, F_i\, P_{Mi}$$

As noted in Section V 6.3, for preload calculations, Γ_i and F_i should be taken as 2.0 and 1.0 respectively. For fatigue load calculations, Γ_i can be taken as 1.0 and F_i as either 0.2 (for neoprene-lined clamps) or 0.05 (for stressed grouted clamps).

V 6.5.2 Clamp Under OPB

The assumption of complete failure of the weld between the brace and chord, coupled with the use of Assessment Model 5 in developing the design recommendation for tube-to-tube clamps under OPB, means that exactly the same formulation in Section V 6.3 can be used for nodal clamps under OPB. The only point of clarification required is that l_1 for nodal clamps should be taken as defined in Figure 6.1.

V 7 CONCLUSIONS

Systematic tests have been conducted on neoprene-lined and stress grouted clamps intended to join tubular members end-to-end. The clamps were tested under pure applied moment loading. The studbolts and the tubular members were provided with strain gauges to determine the behaviour of the clamp/tubular systems, and particularly the response of the studbolts to the applied loading. A series of Finite Element analyses were also undertaken to extend the test range of geometries.

Simplified models of the clamp/tubular interaction have been developed and calibrated to the experimental data to provide a basis for new design methods. These new methods result in studbolt preload requirements, to prevent clamp halves separating under moment loading, much smaller than those obtained by traditional approaches.

More detailed conclusions relating to clamp/tubular behaviour are:

- The mechanisms leading to studbolt load variations are quite different for IPB (plane of bending parallel to studbolt axes) and OPB (plane of bending perpendicular to studbolt axes). The mechanisms are summarised in Figures 5.22 and 5.23 respectively.

- For both IPB and OPB, the moment transfers from the tubular into the clamp by longitudinal shear stresses at the interface and by internal radial bearing loads (see Figure 5.2). The former mechanism does not lead to studbolt load variation and can only apply if separation of the clamp halves has not occurred. The latter mechanism always leads to studbolt load variations in the case of OPB but only may in the case of IPB depending on the nature of bearing loads. The (IPB) radial bearing loads will not lead to studbolt load variation if they are reacted equally (with respect to both position and magnitude) by the two clamp halves. This is because the two halves would deflect equally with no tendency for the two halves to separate.

- For IPB, the internal bearing loads are resisted by studbolt stretching and by relaxation of the internal clamp compression induced by studbolt preload. After clamp halves separate, only studbolt stretching can remain active and this leads to two regimes of behaviour: before and after separation. Therefore, before separation, the internal bearing loads are shared between the studbolts and the rest of the clamp; load sharing is particularly important for stressed grouted clamps. A beam analogy, with the beam stiffness representing one clamp half and loads representing studbolt loads and those from contact between the clamp and tubular member, gives a satisfactory basis for modelling the system. Rigid beam assumptions prove adequate for design purposes.

- Applied OPB moment does not lead to clamp half separation. The bearing loads give rise to internal torques of opposite senses on the two clamp halves, the torque distributions varying along the length of the clamp. The internal torques are resisted by torsion of clamp box section, by circumferential shearing of the annulus material and by studbolt load variation associated with a lever arm. For design purposes, torsion of the clamp box section can be ignored as it is generally small.

- It will generally be found that, for equal IPB and OPB moments, IPB will govern preload requirements to prevent clamp half separation but OPB is more severe as far as fatigue is concerned. However, the test data indicate that fatigue of studbolts is not really an issue, particularly for stressed grouted clamps.

- The tests have confirmed that resolving moments into IPB and OPB components work well; the resulting studbolt load variations can then be numerically summed.

- The overall compliance of a neoprene-lined clamp/tubular system is similar to that of an intact member, despite the presence of the liner.

- The loss of a single studbolt leads to a redistribution of preload in other studbolts, particularly for the nearest studbolt on the same side of the clamp. The grouted clamp test data show that the tension of this studbolt increased by about 23kN, i.e. only by 13% of the original preload value with all studbolts in place. Under the action of applied moment (IPB or OPB), the distribution patterns of studbolt load variation remained similar to those of the complete system, except local to the missing studbolt location. The maximum studbolt load variations were also largely unchanged. The tests indicate the robustness of clamped repairs, even were a studbolt to go missing.

FIGURES

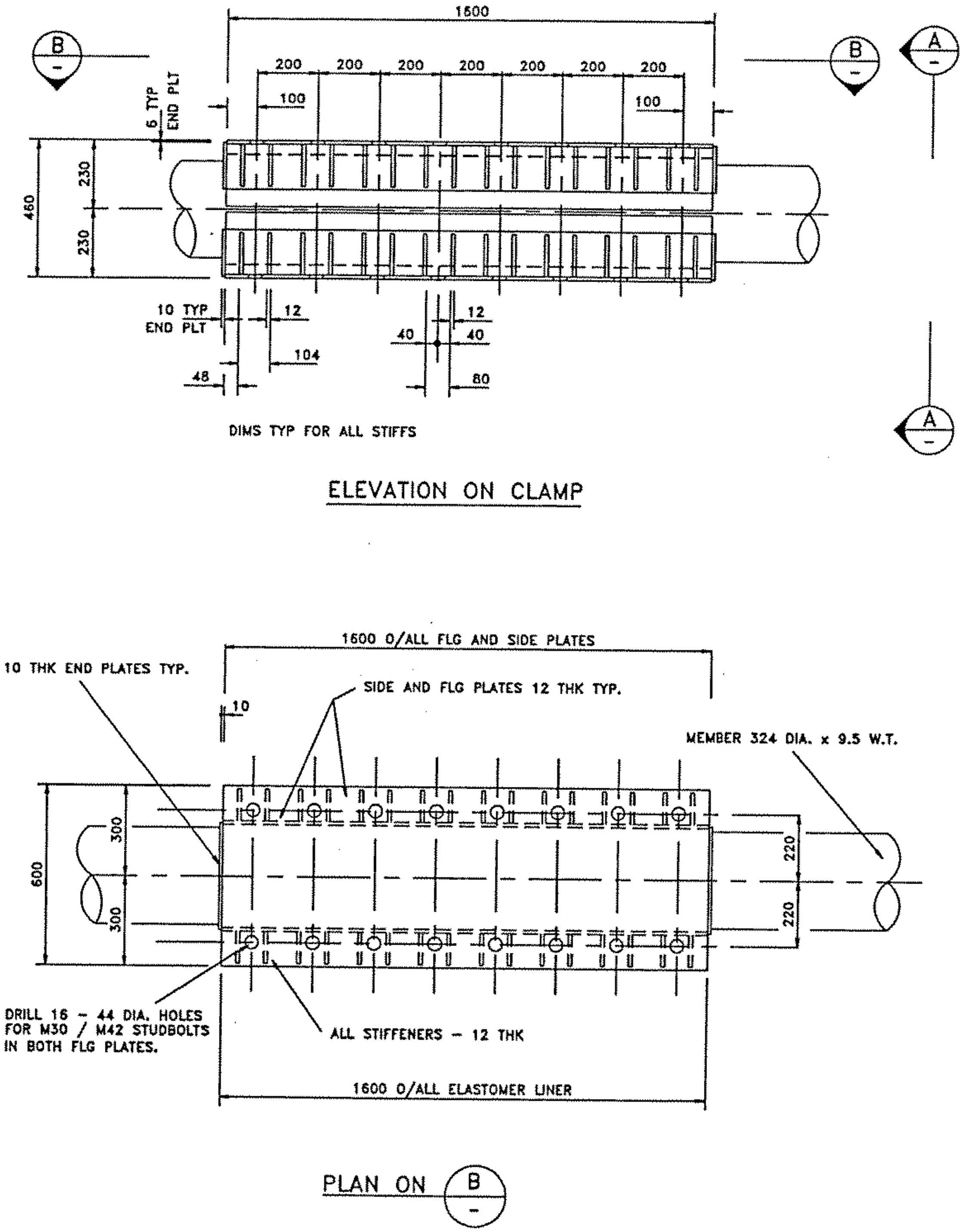

Fig. 2.1: Elevation and plan on clamp

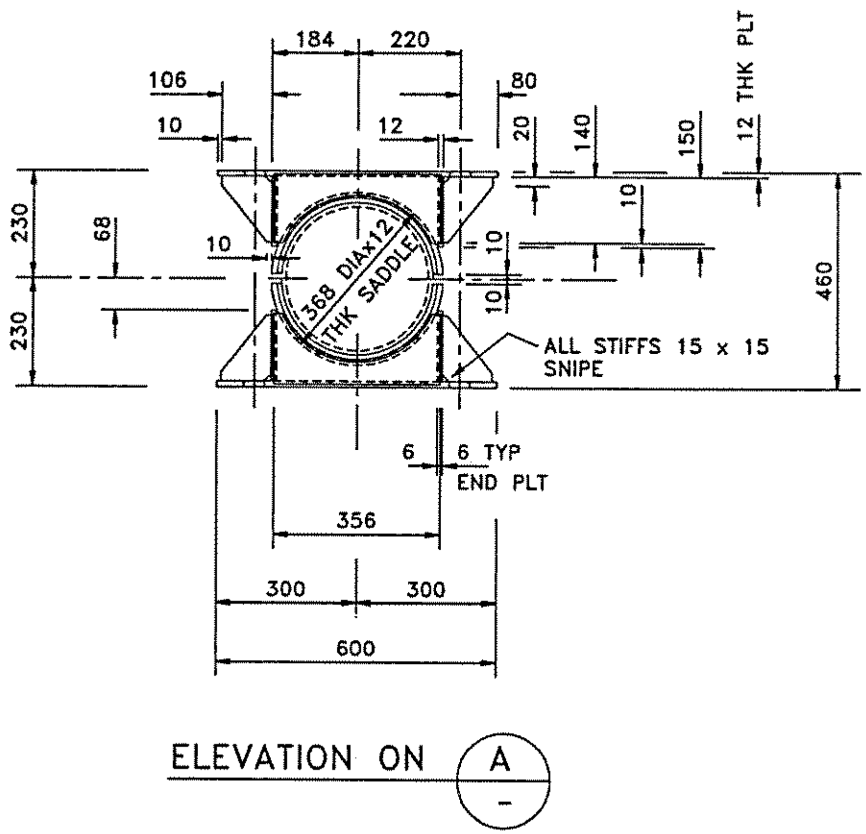

Fig. 2.2: End view on clamp

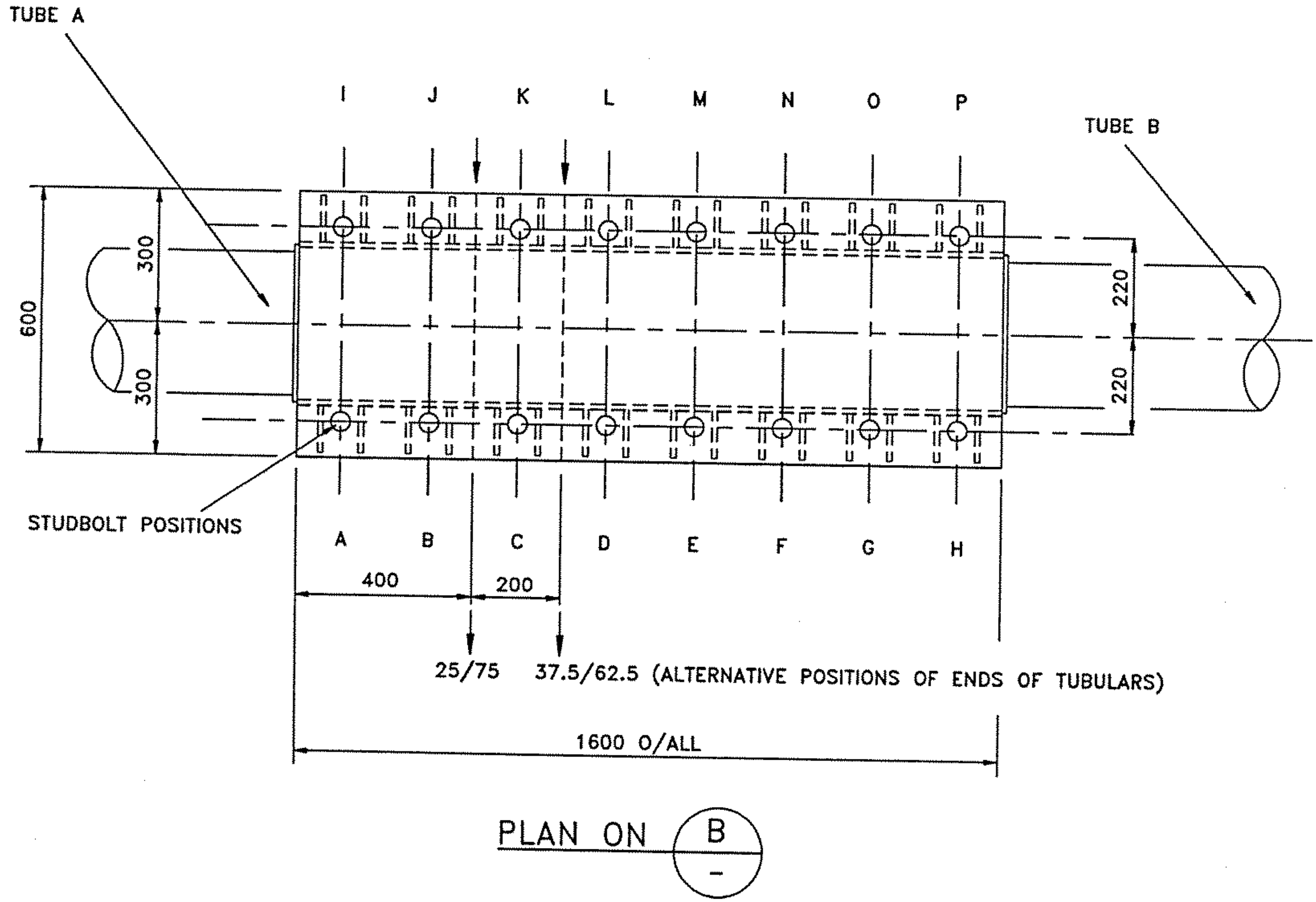

Fig. 2.3: Studbolt numbering

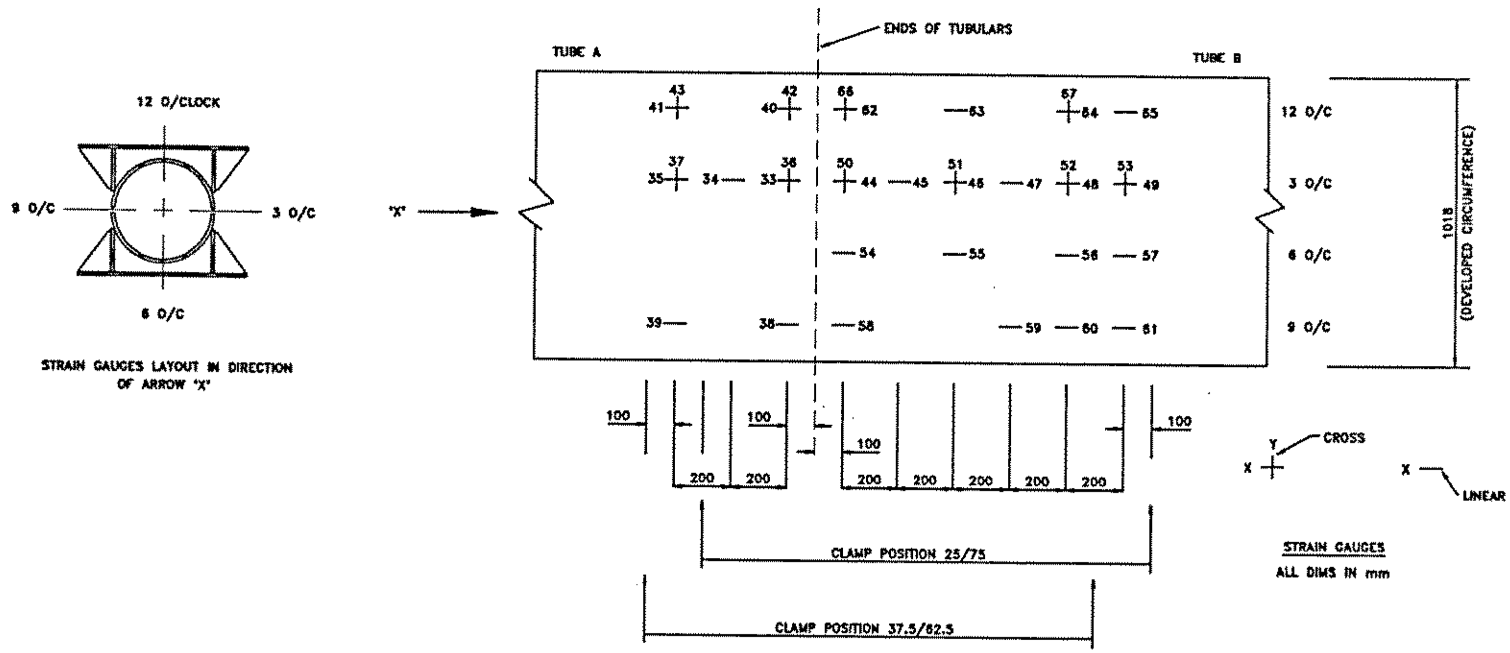

Fig. 2.4: Strain gauges and clamp positions on tubes A and B

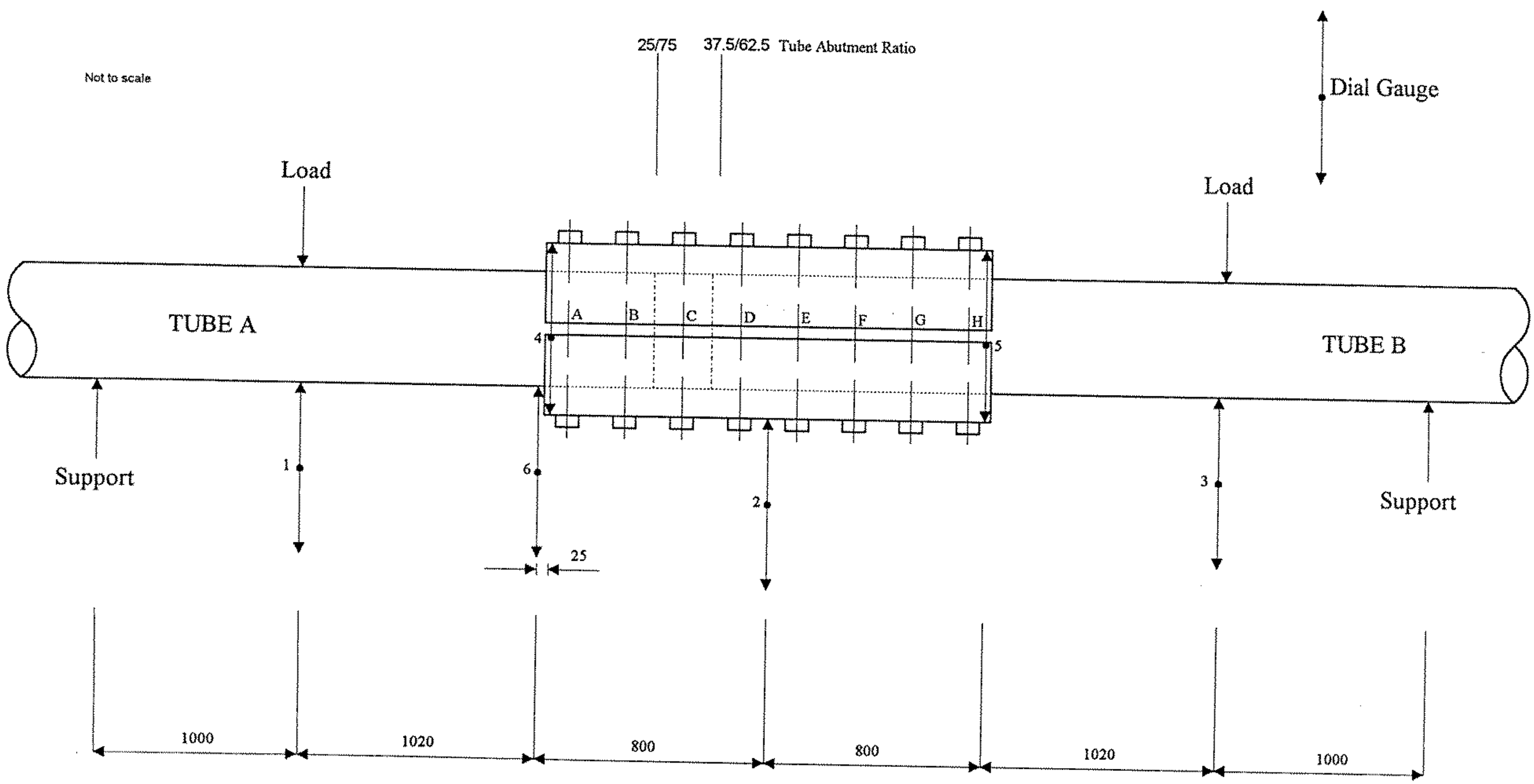

Fig. 2.5: Dial gauge positions

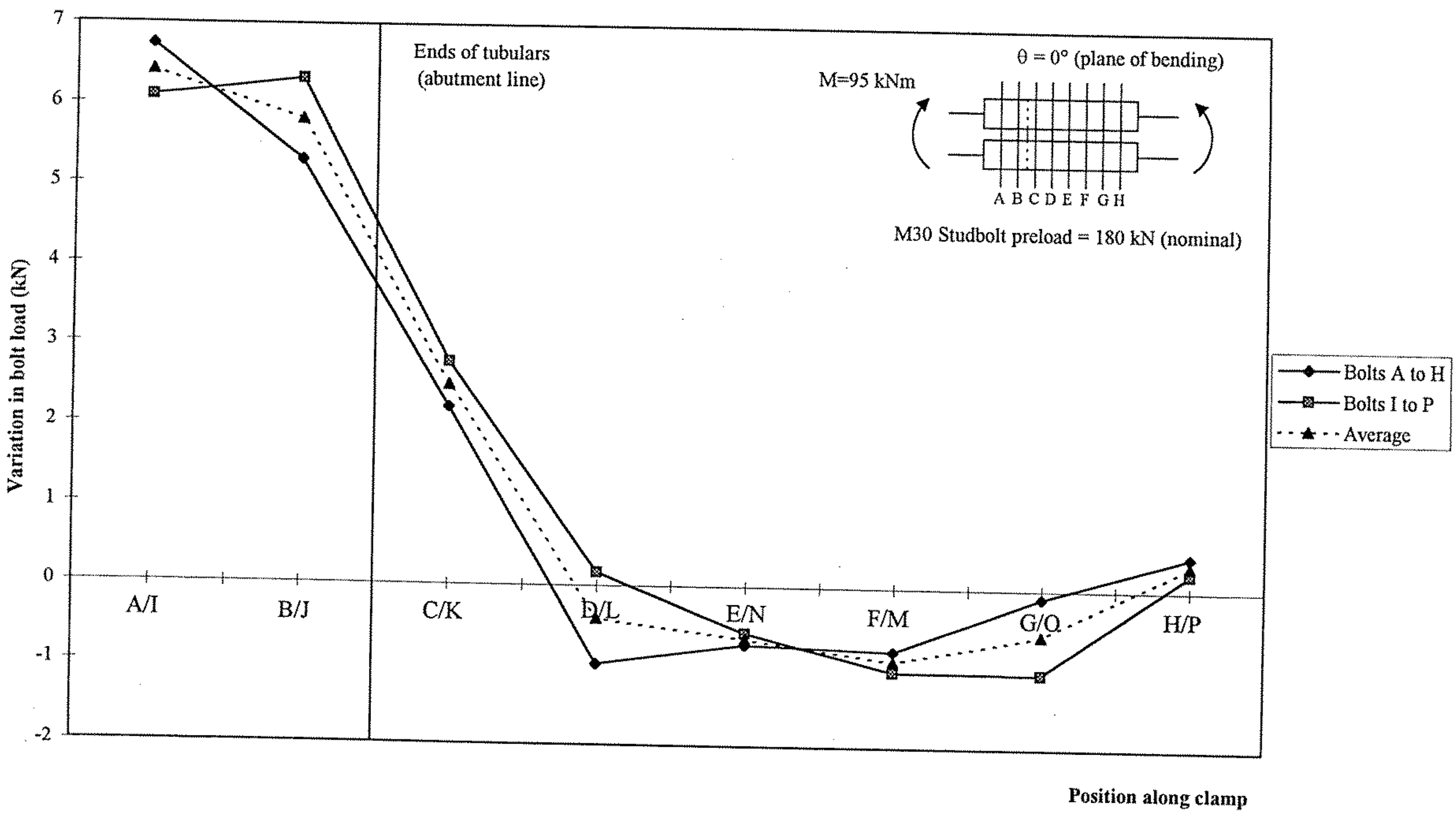

Fig 3.1 : Studbolt load distribution: Test 1 (neoprene clamp)

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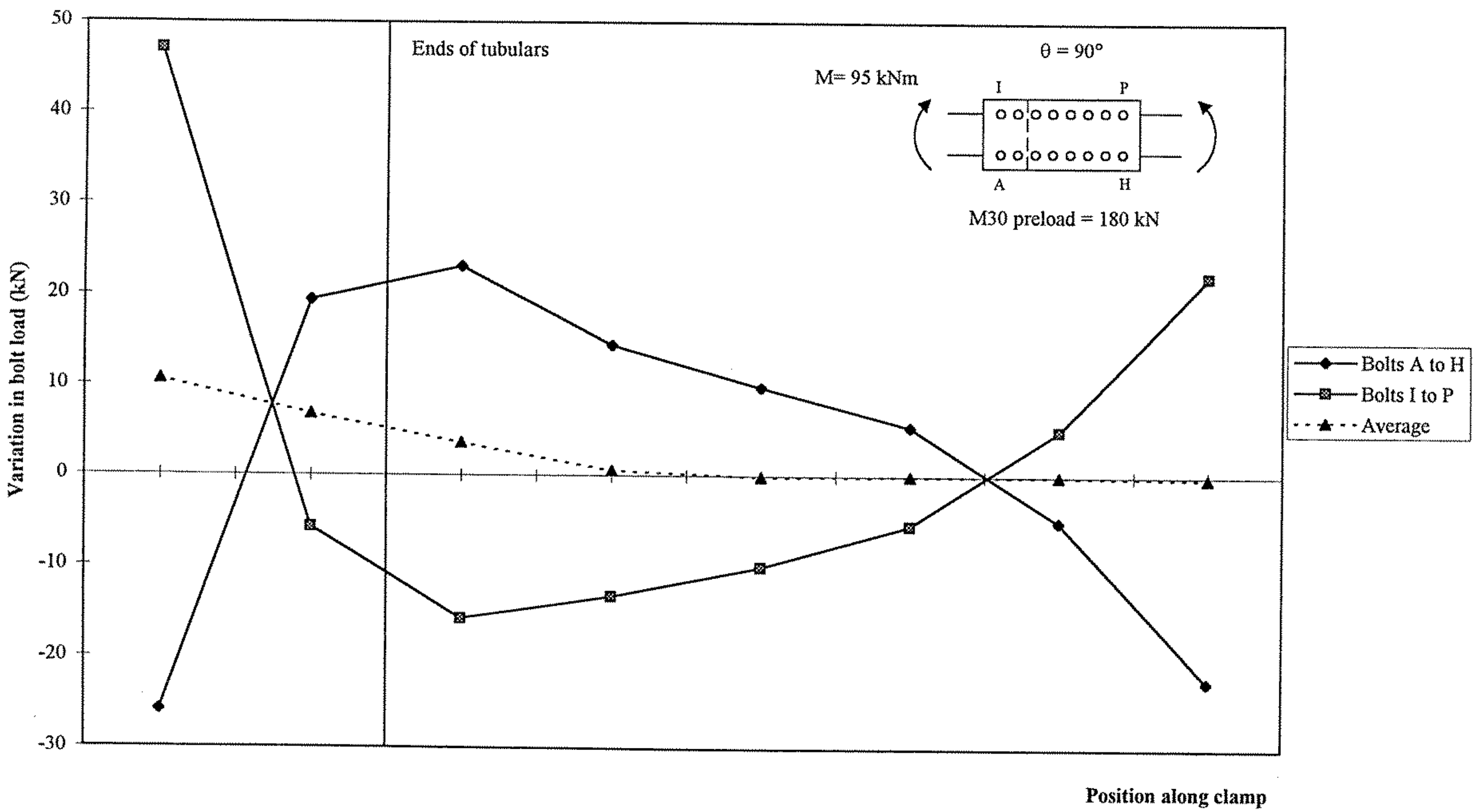

Fig 3.2 : Studbolt load distribution: Test 2 (neoprene clamp)

S M:\C m24\grout\BLTUPDT.XLS\test 2

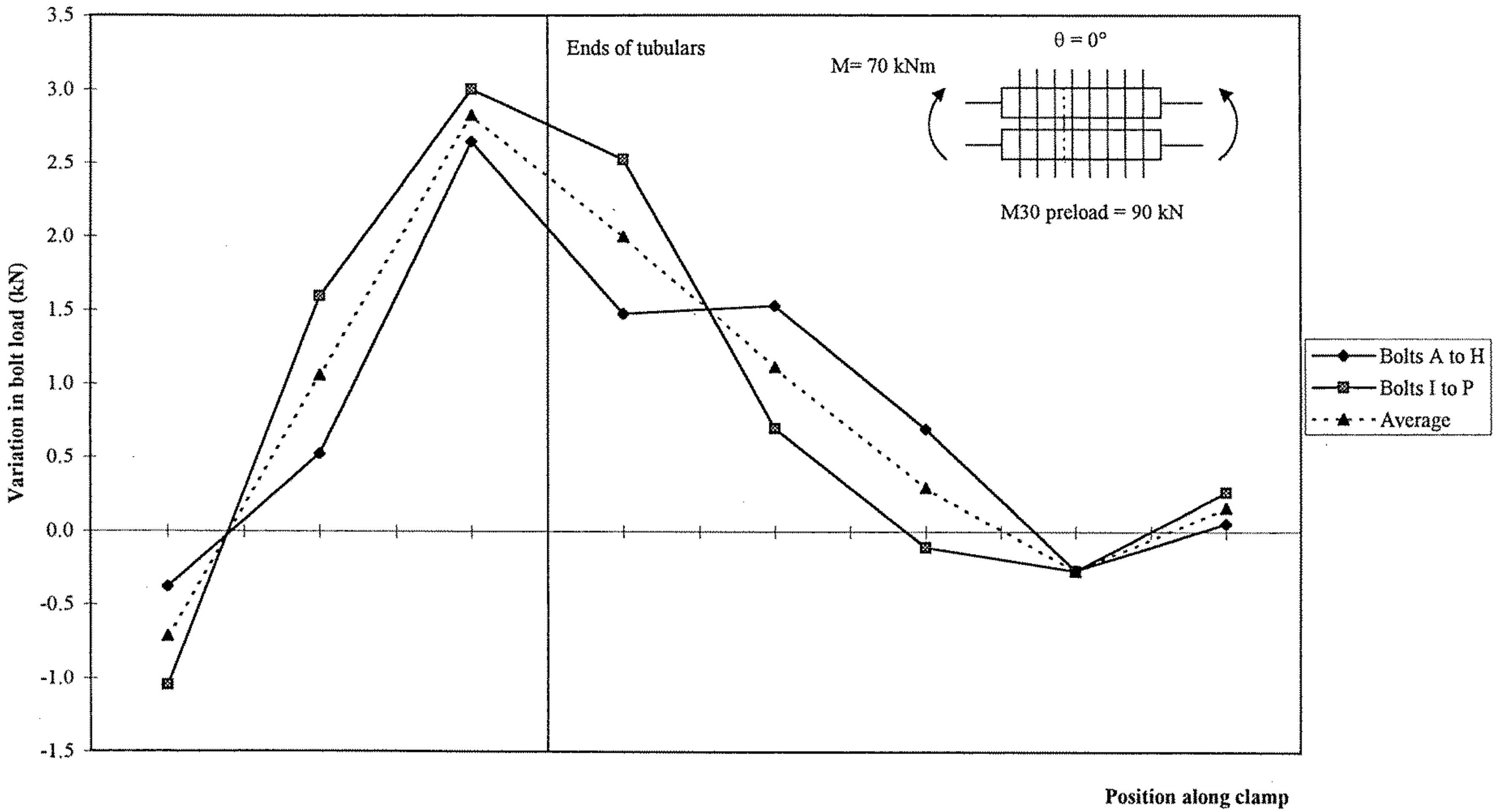

Fig 3.3 : Studbolt load distribution: Test 3 (neoprene clamp)

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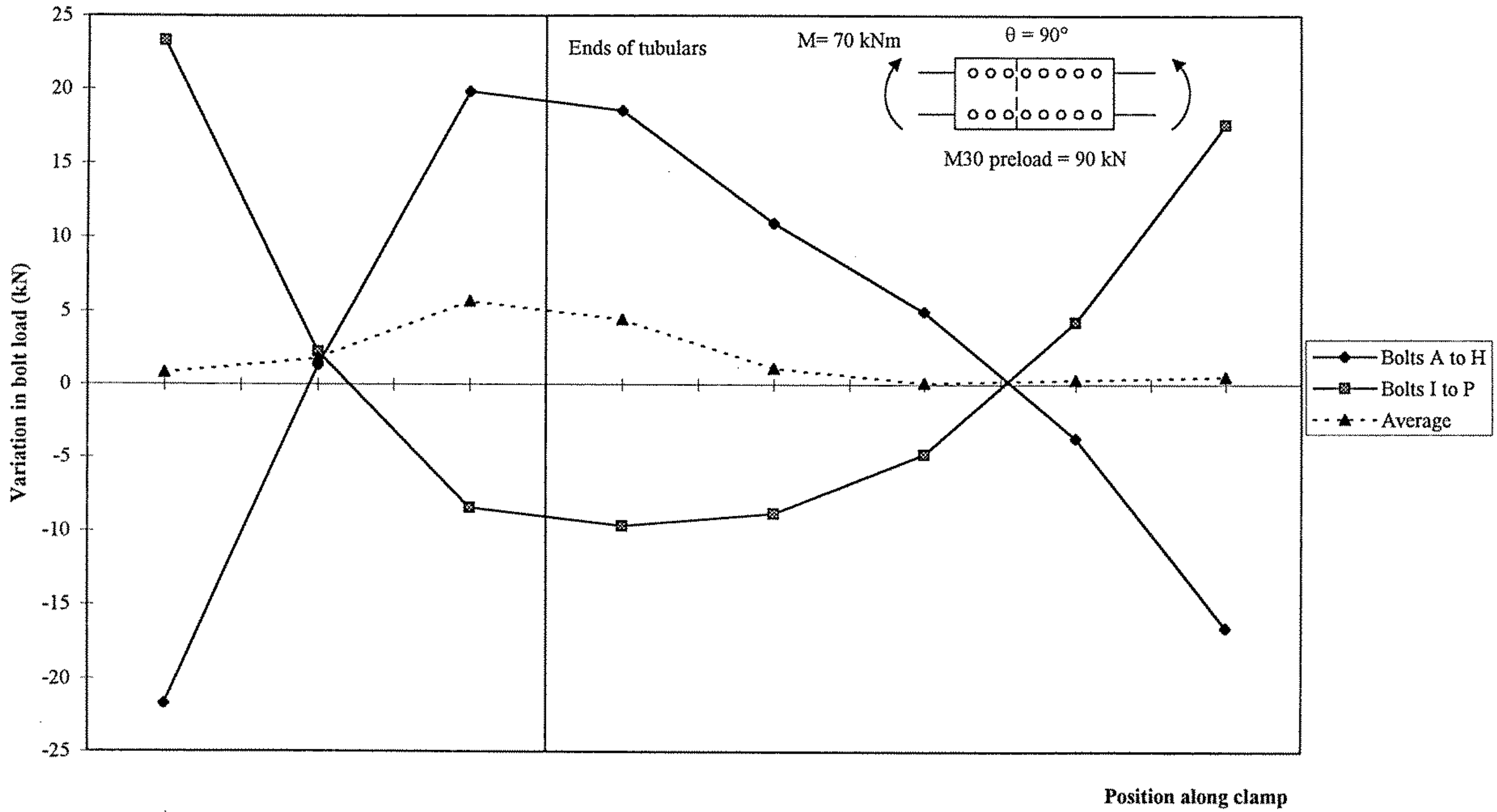

Fig 3.4 : Studbolt load distribution: Test 4 (neoprene clamp)

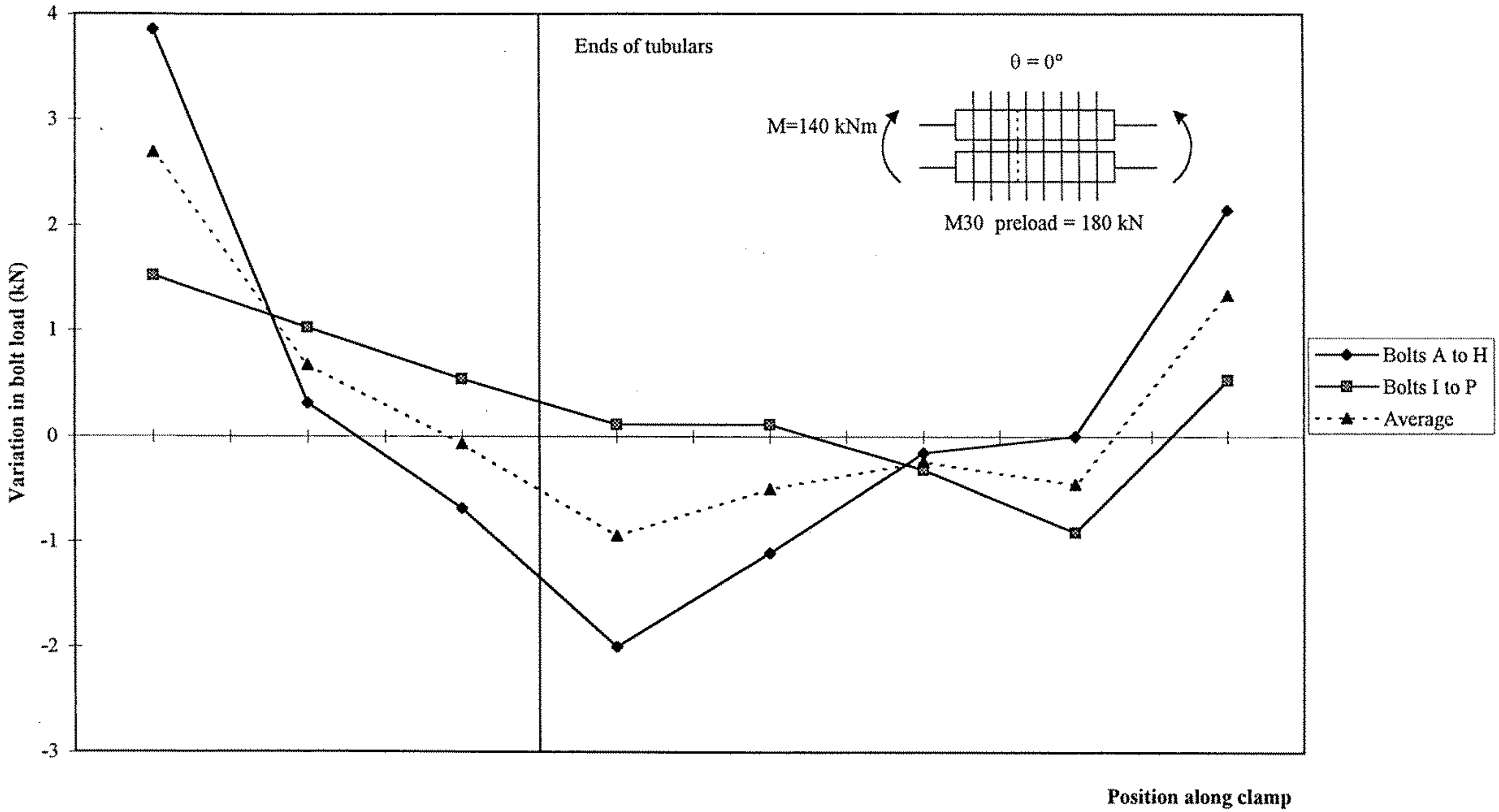

Fig 3.5 : Studbolt load distribution: Test 5 (neoprene clamp)

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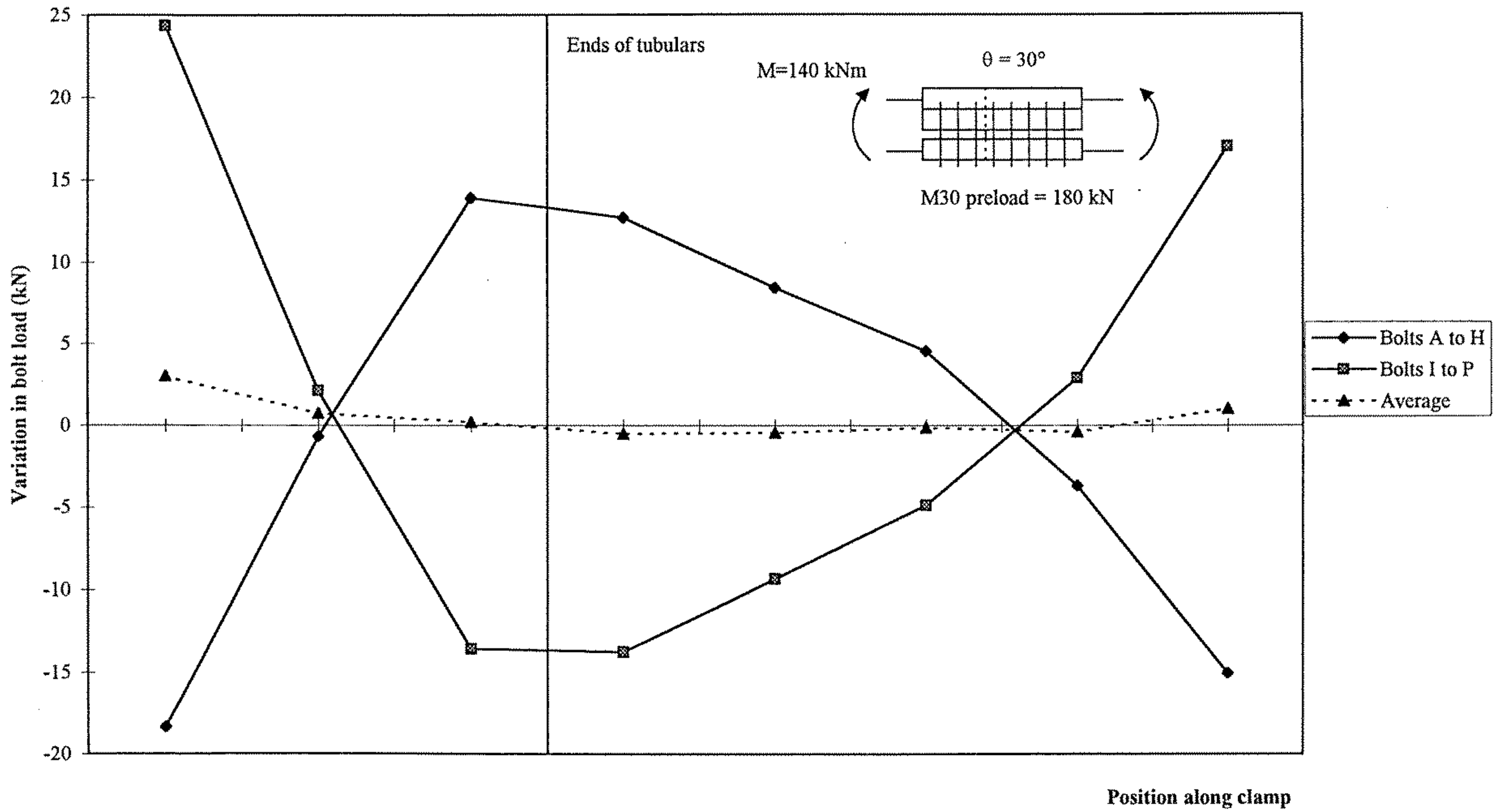


Fig 3.6 : Studbolt load distribution: Test 6 (neoprene clamp)

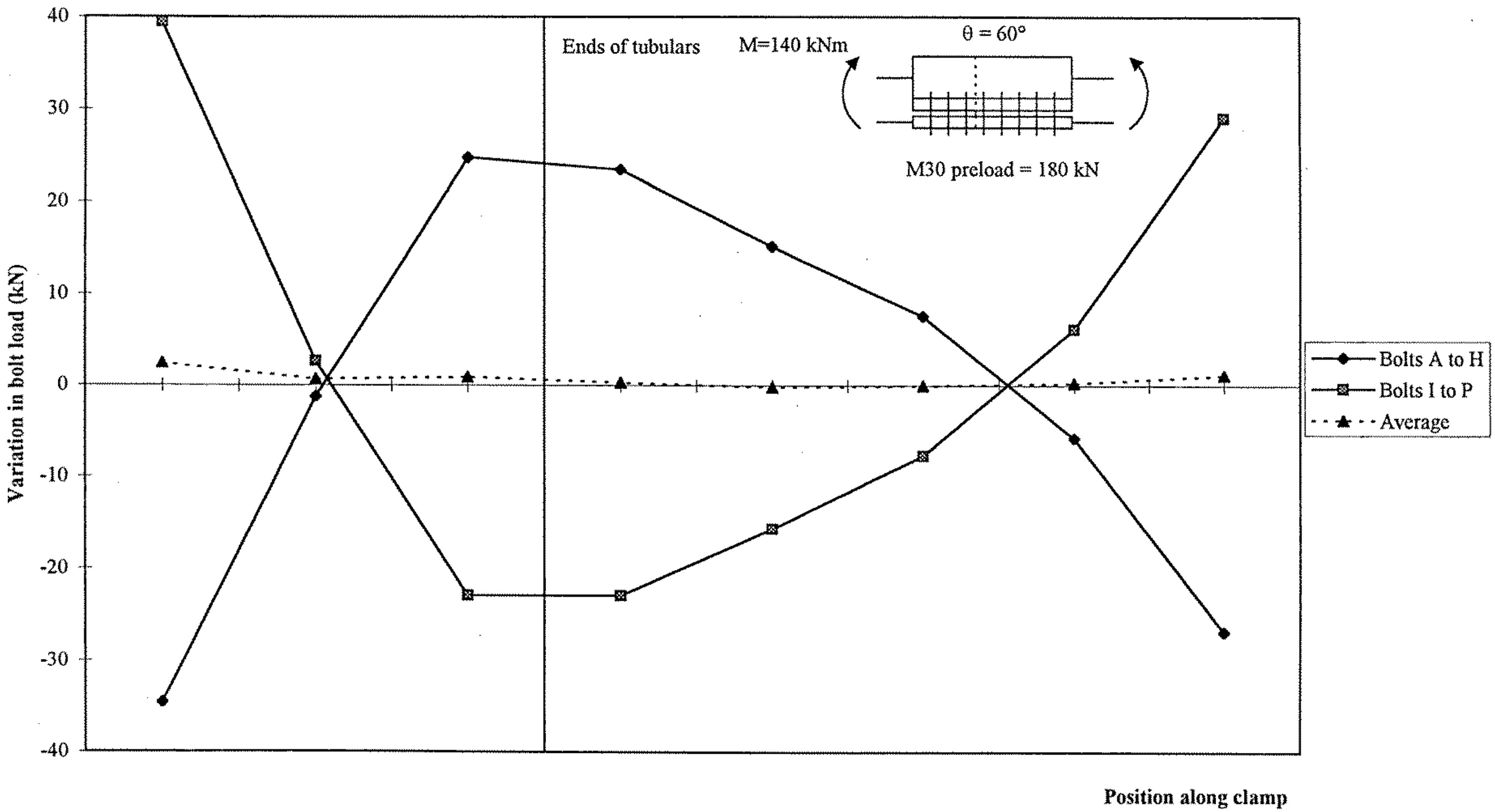


Fig 3.7 : Studbolt load distribution: Test 7 (neoprene clamp)

S M:\C167\m24\grout\BLTUPDT.XLS\test 7

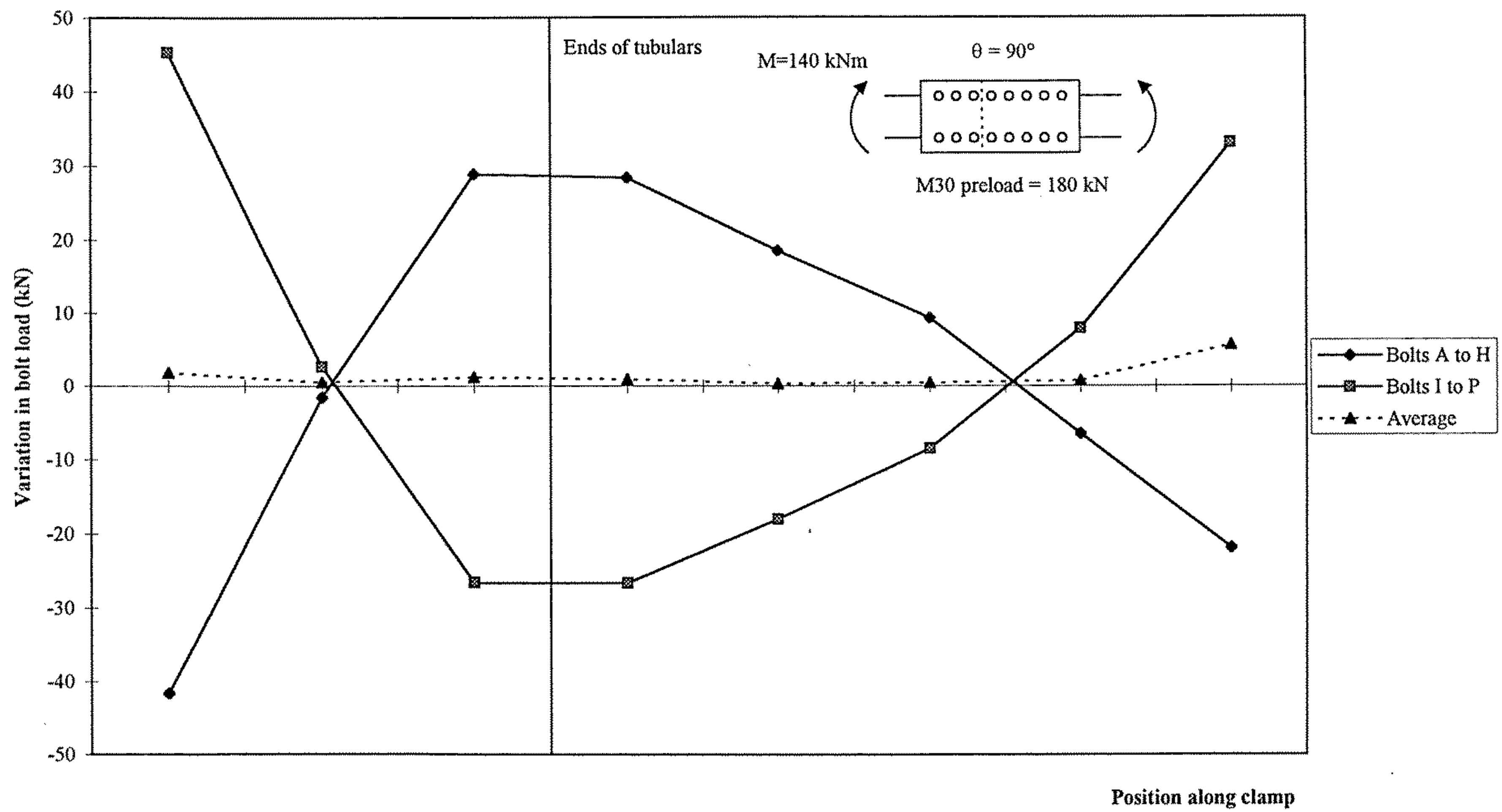


Fig 3.8 : Studbolt load distribution: Test 8 (neoprene clamp)

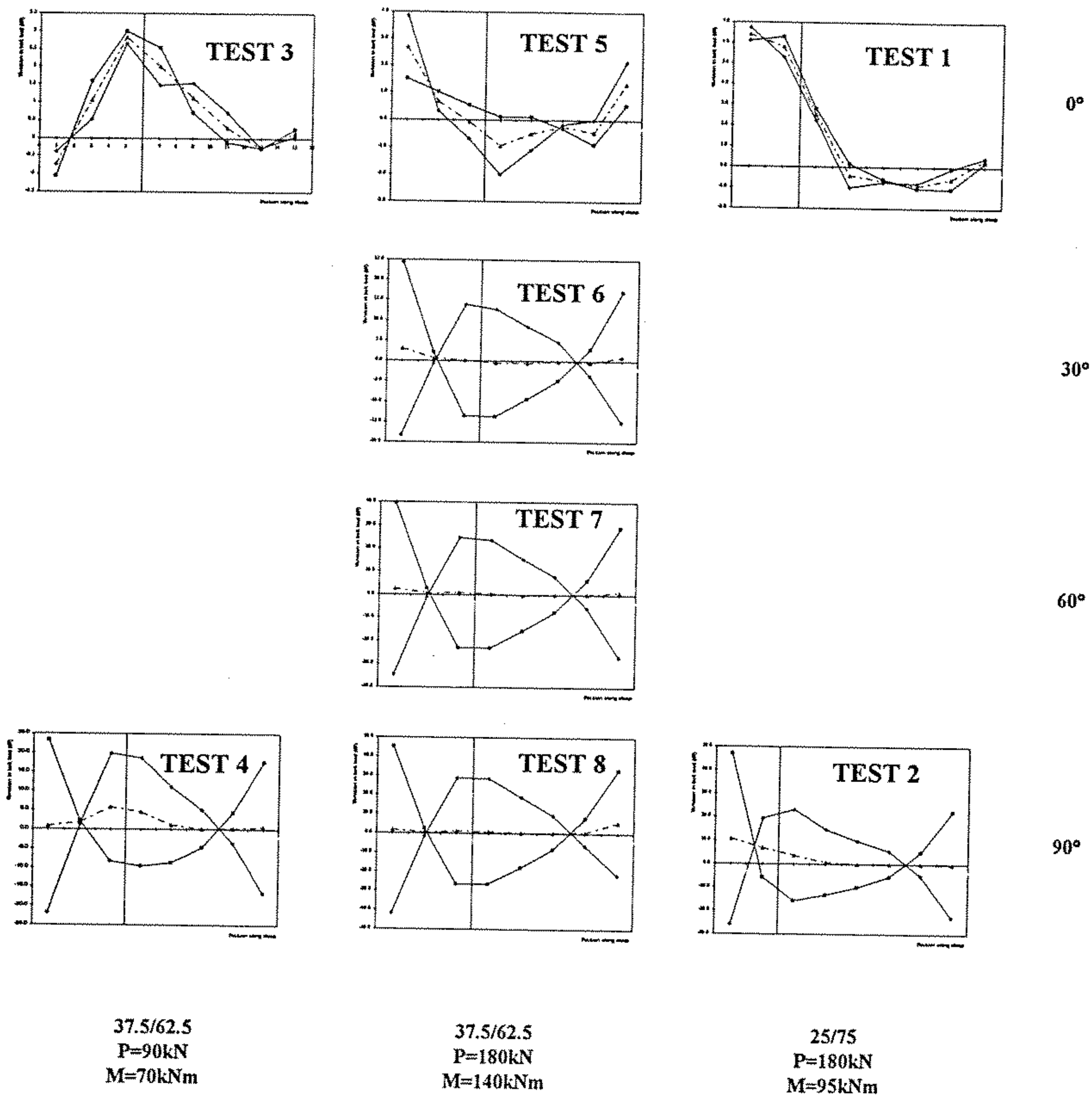
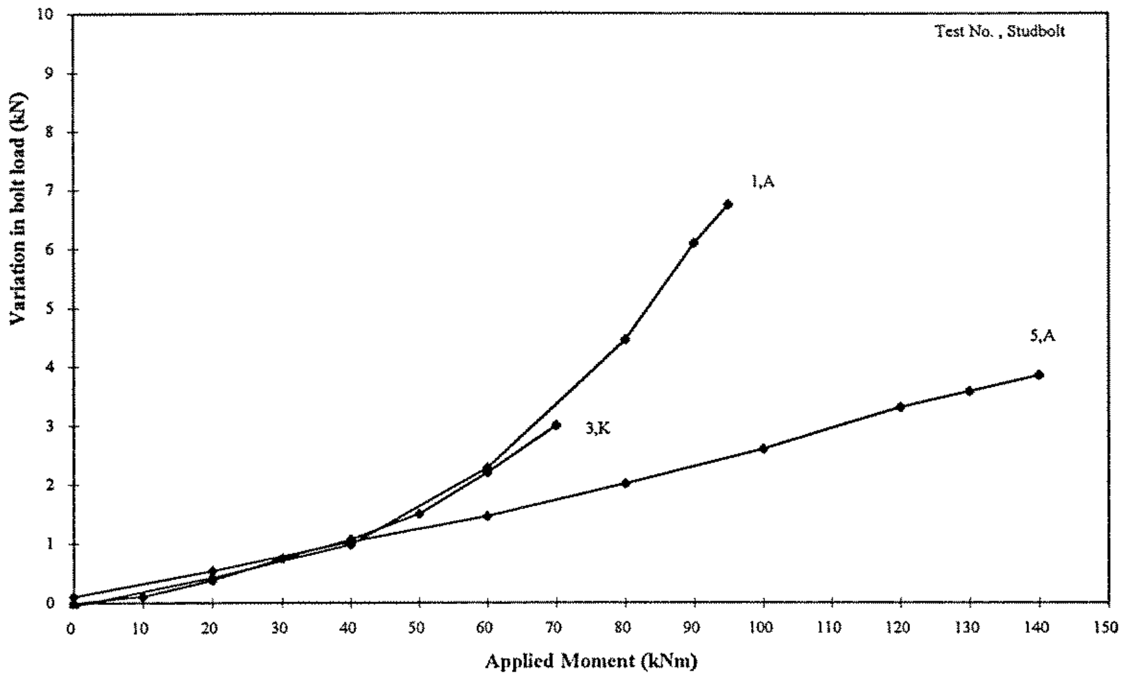
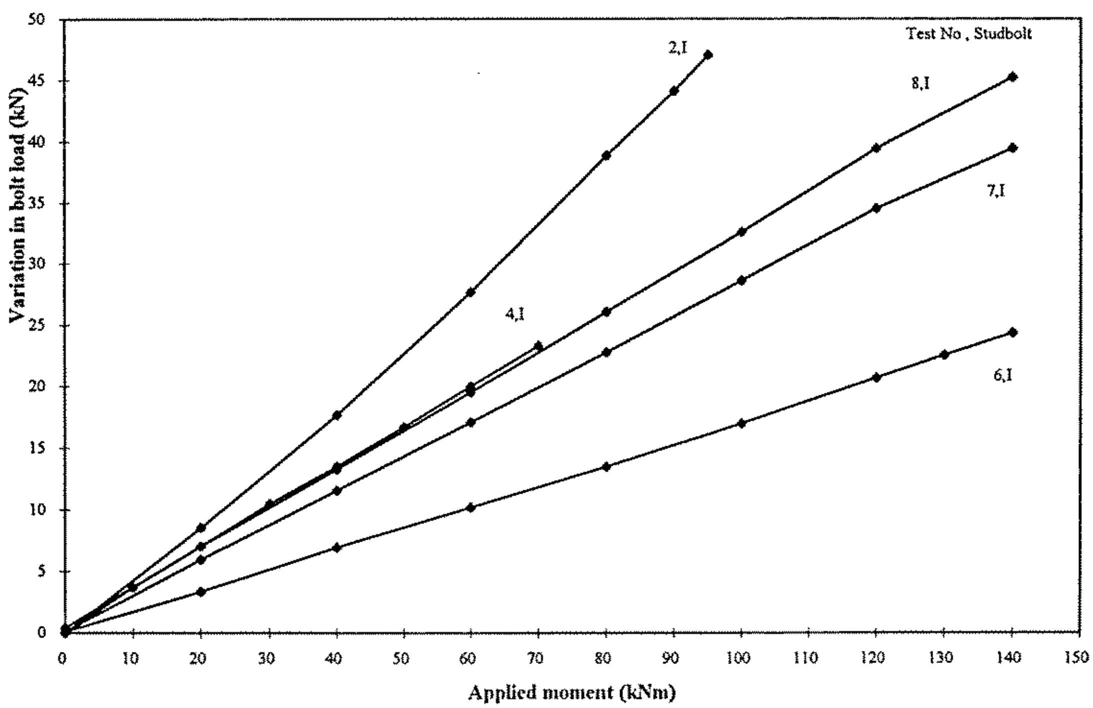


Fig 3.9 : Studbolt load distributions : all tests in Series 1



a) Tests with orientation = 0°



b) Tests with orientation other than 0°

Fig 3.10 : Variation of studbolt load with applied moment (critical studbolt in each test)

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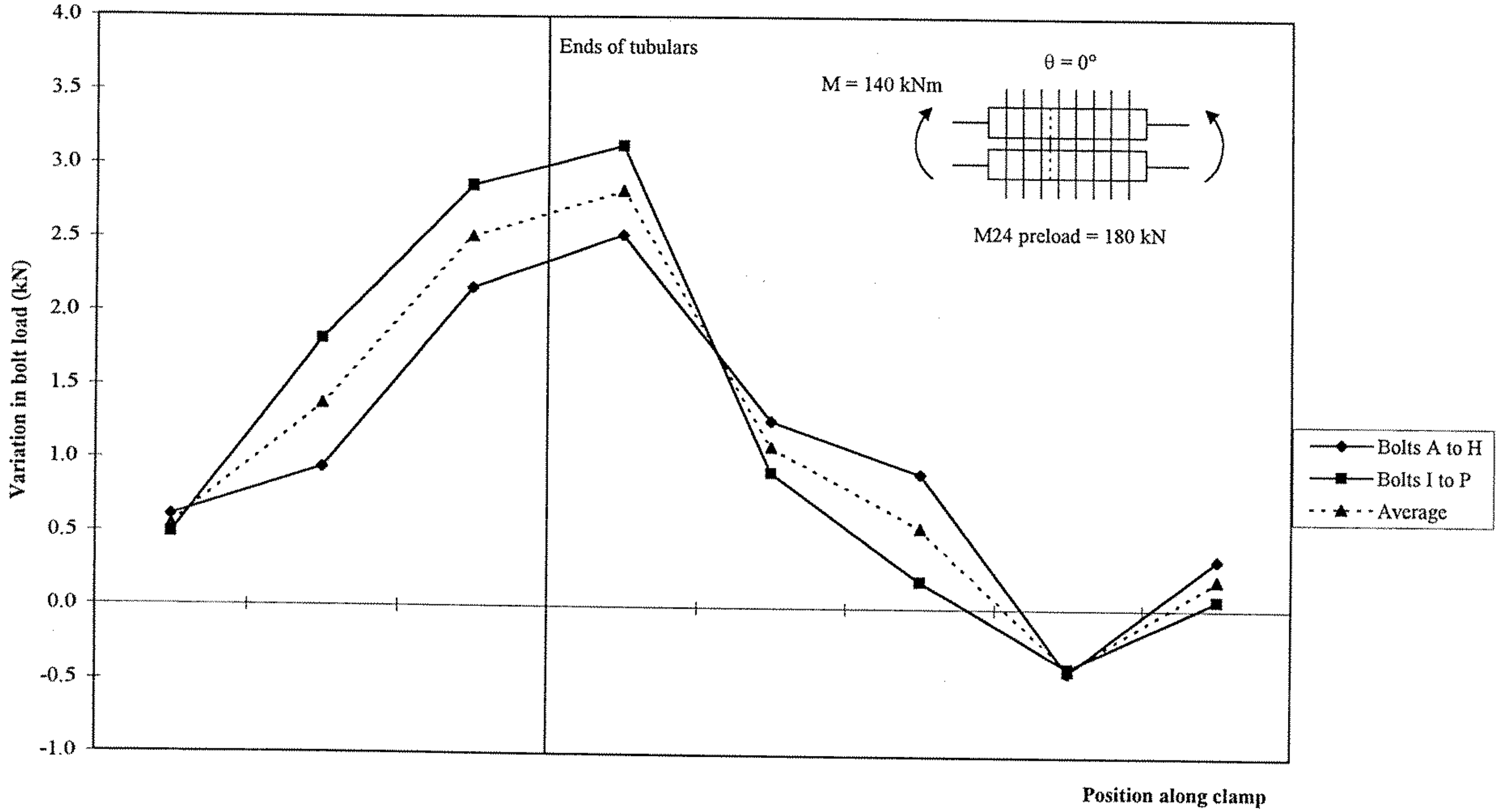

Fig 3.11 : Studbolt load distribution: Test 9 (neoprene clamp)

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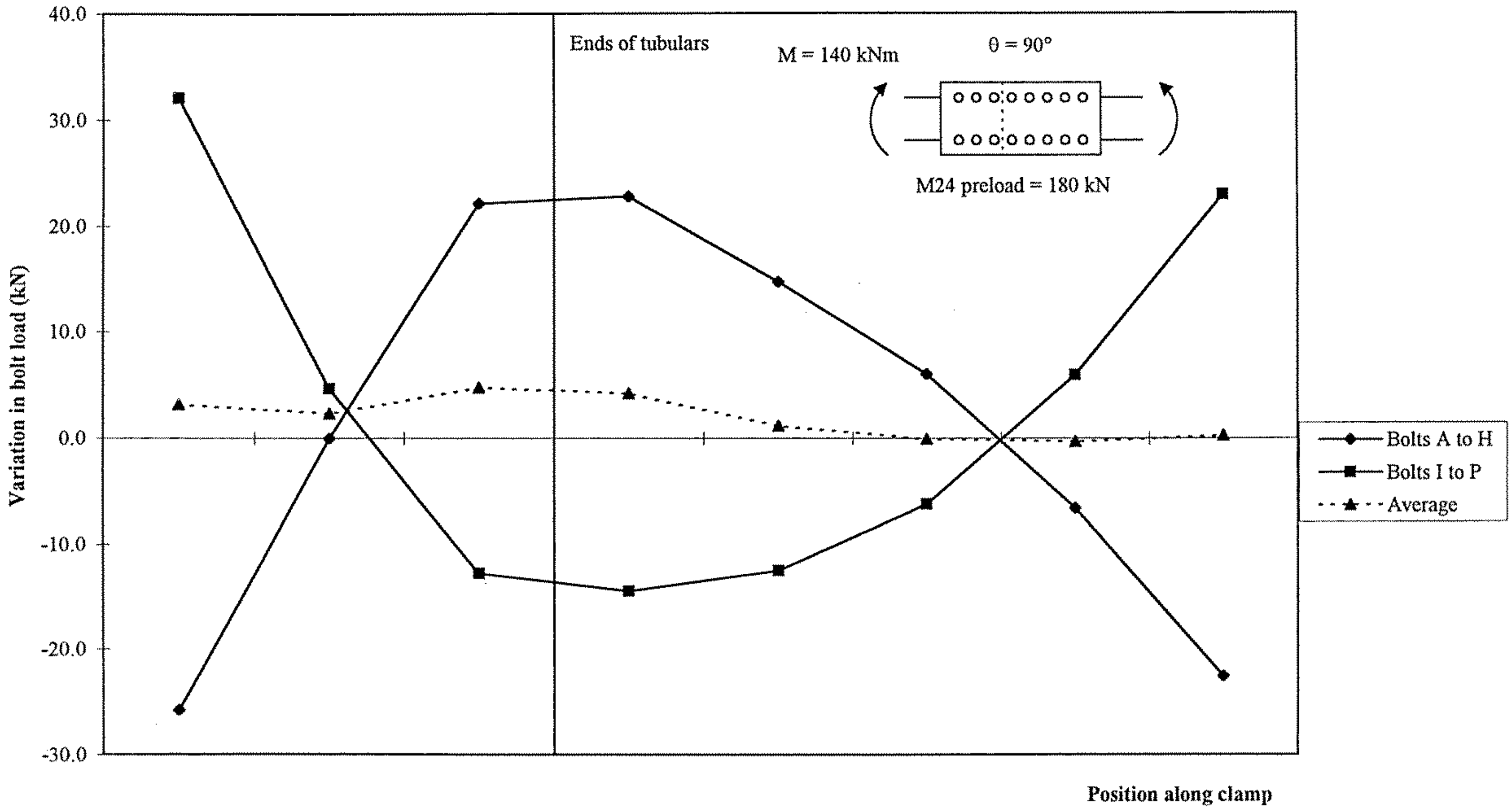

Fig 3.12 : Studbolt load distribution: Test 10 (neoprene clamp)

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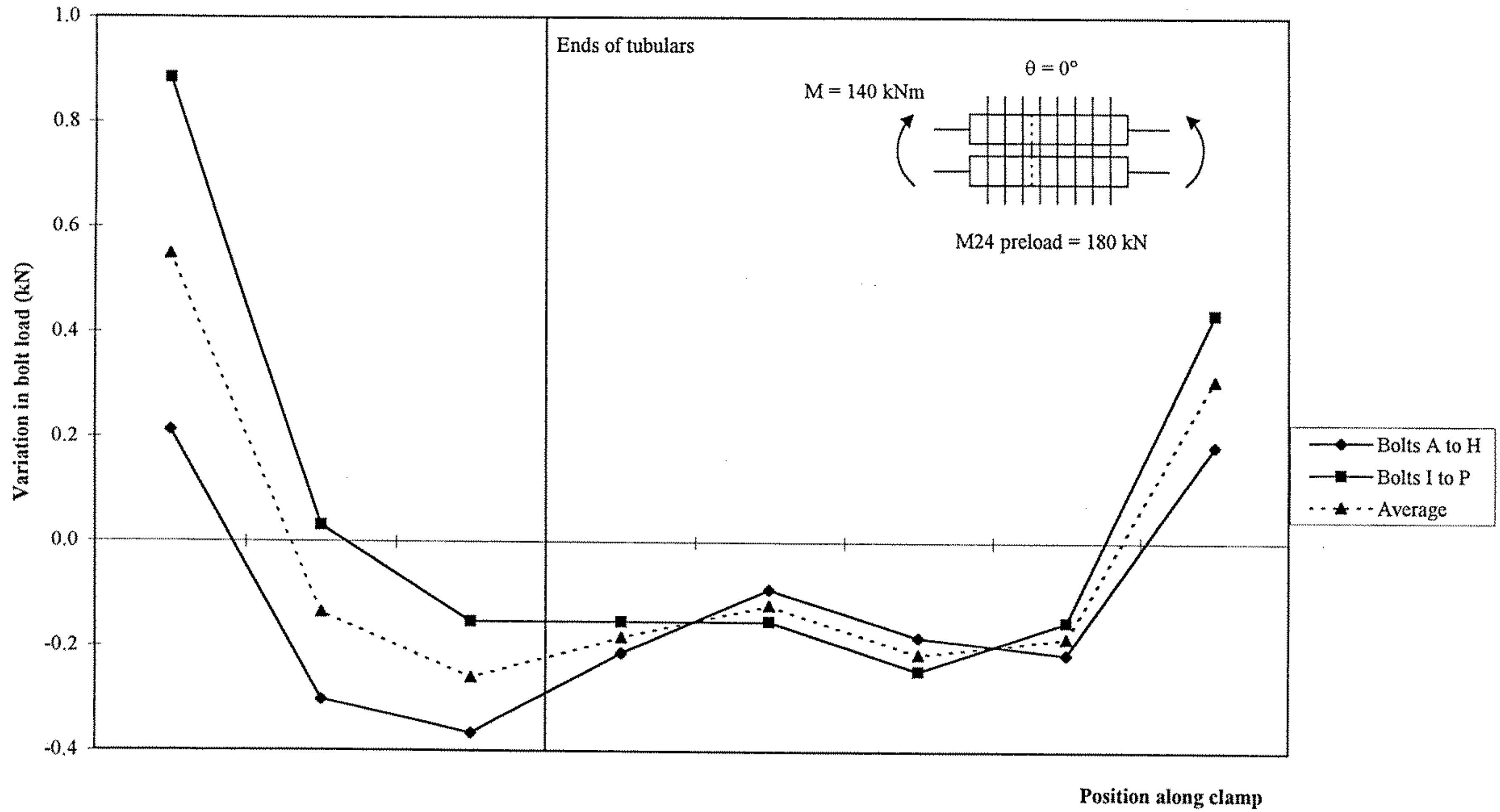

Fig 3.13 : Studbolt load distribution: Test 11 (grouted clamp)

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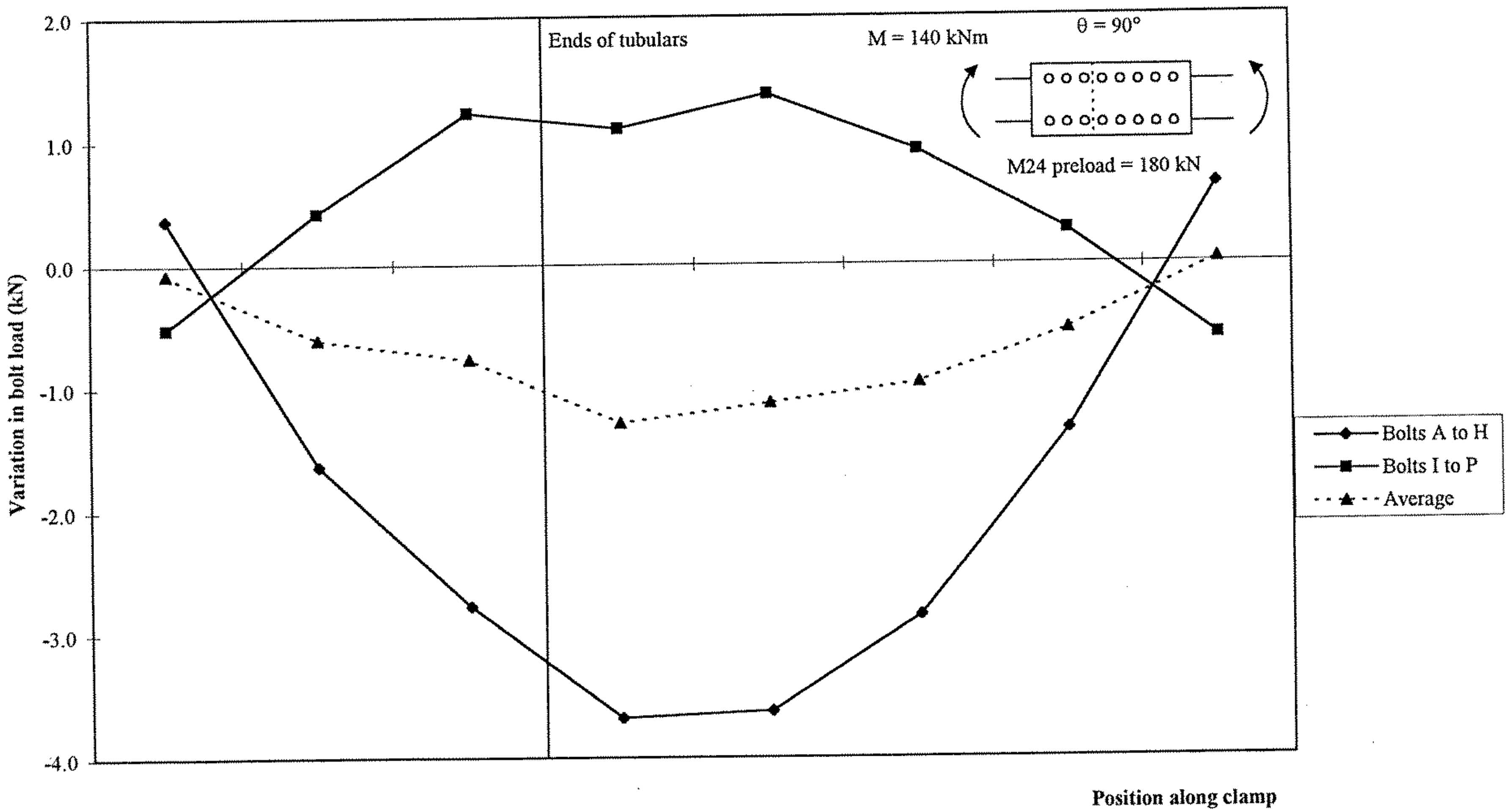

Fig 3.14 : Studbolt load distribution: Test 12 (grouted clamp)

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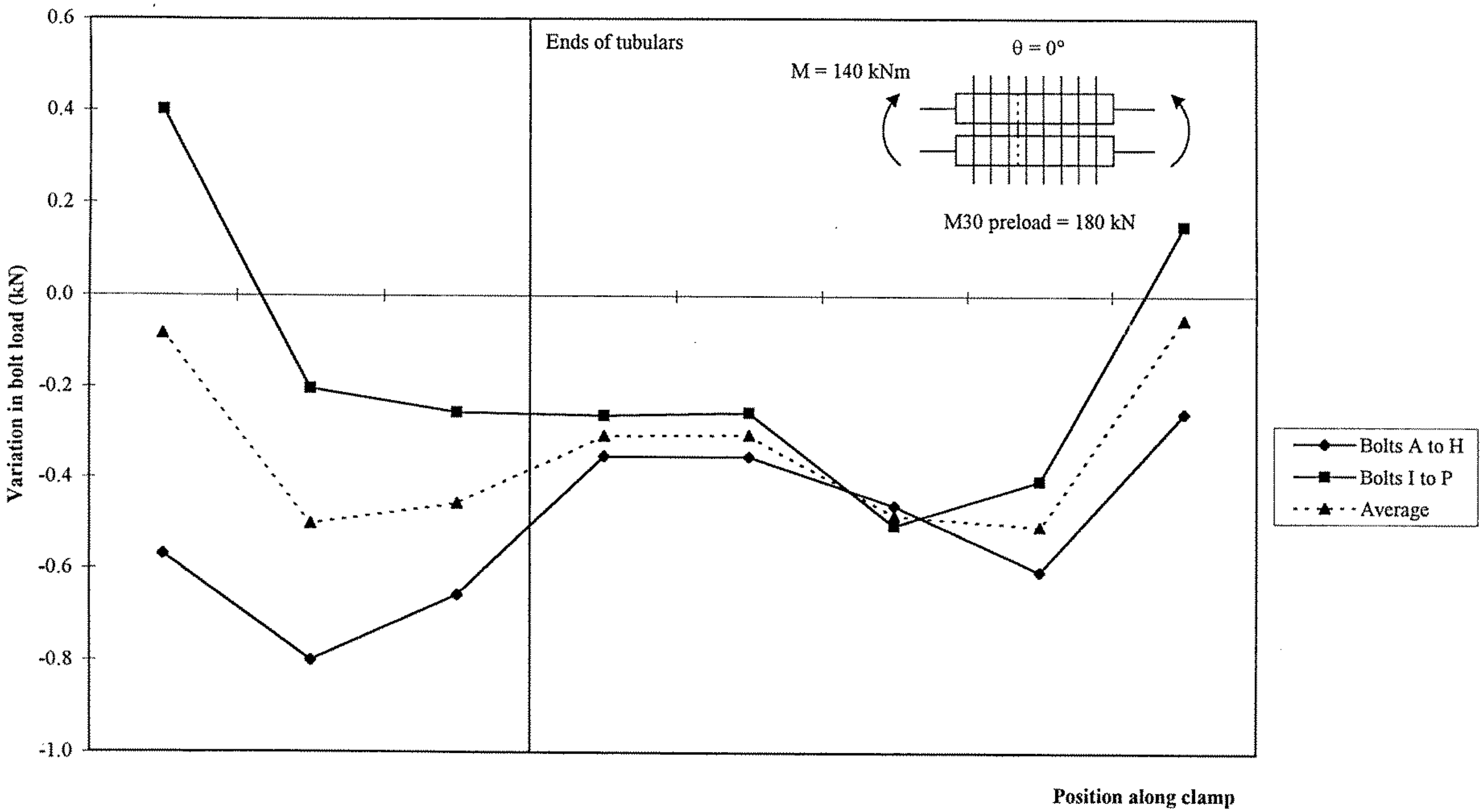

Fig 3.15 : Studbolt load distribution: Test 13 (grouted clamp)

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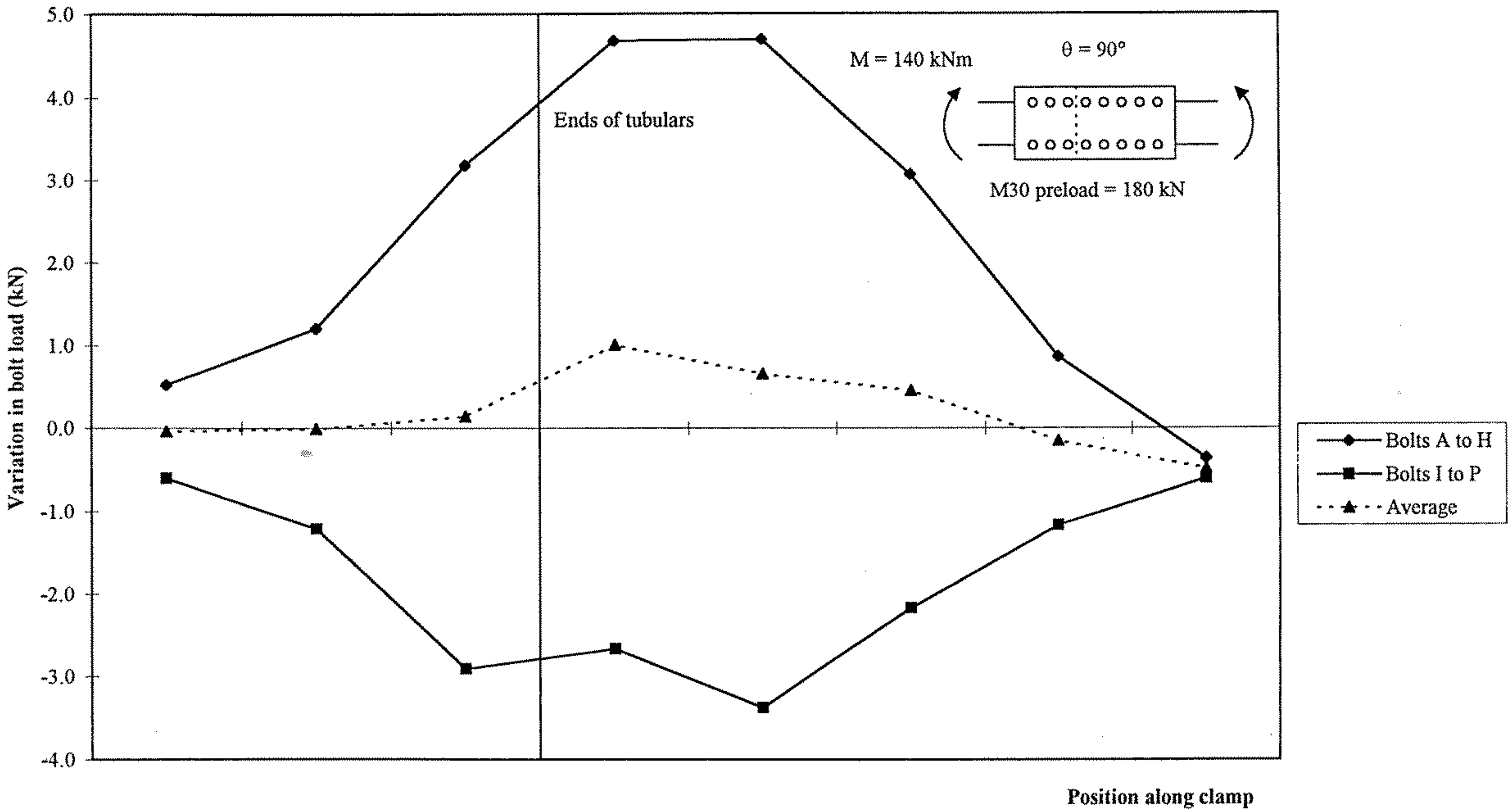

Fig 3.16 : Studbolt load distribution: Test 14 (grouted clamp)

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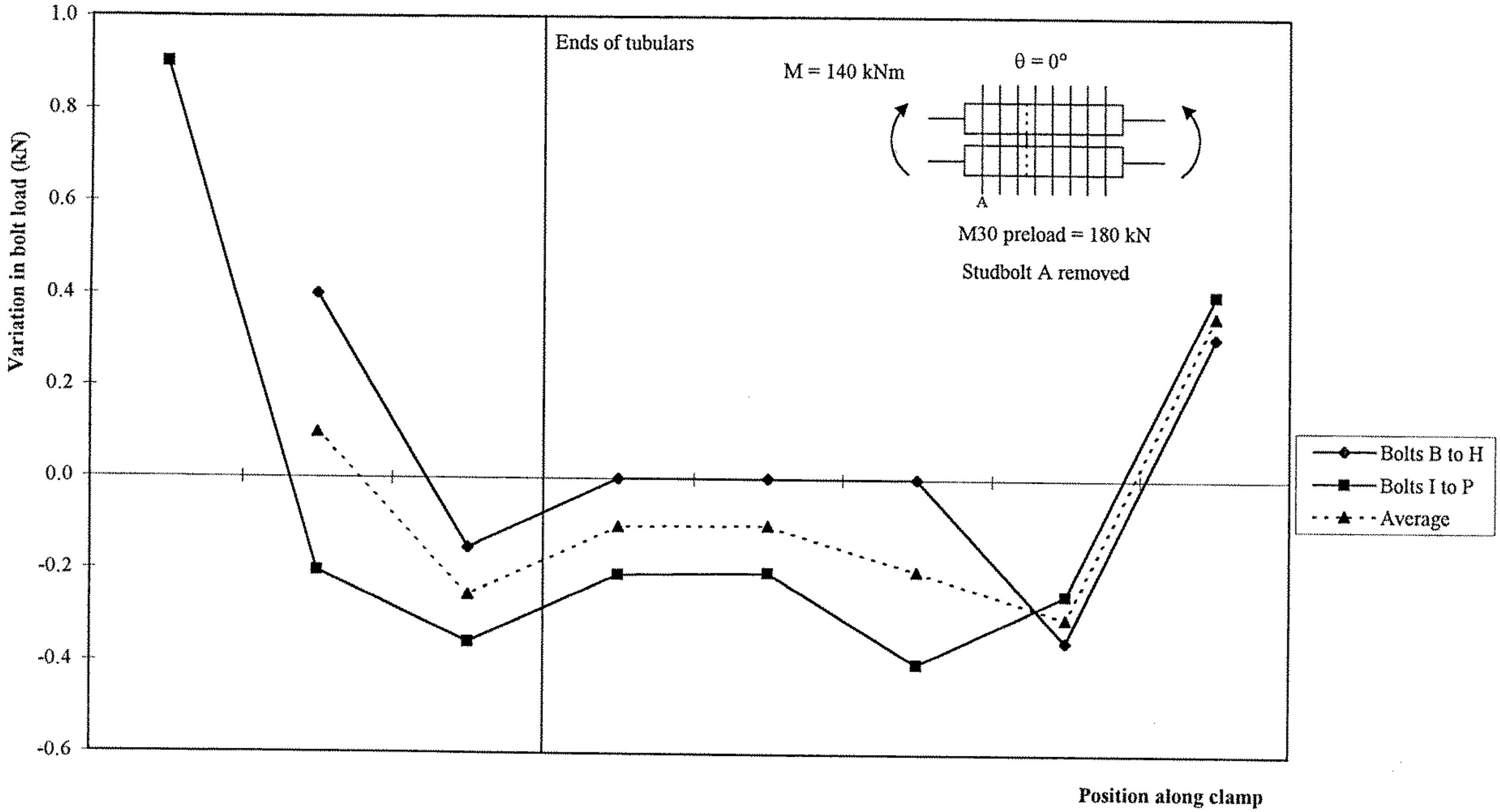

Fig 3.17 : Studbolt load distribution: Test 15 (grouted clamp)

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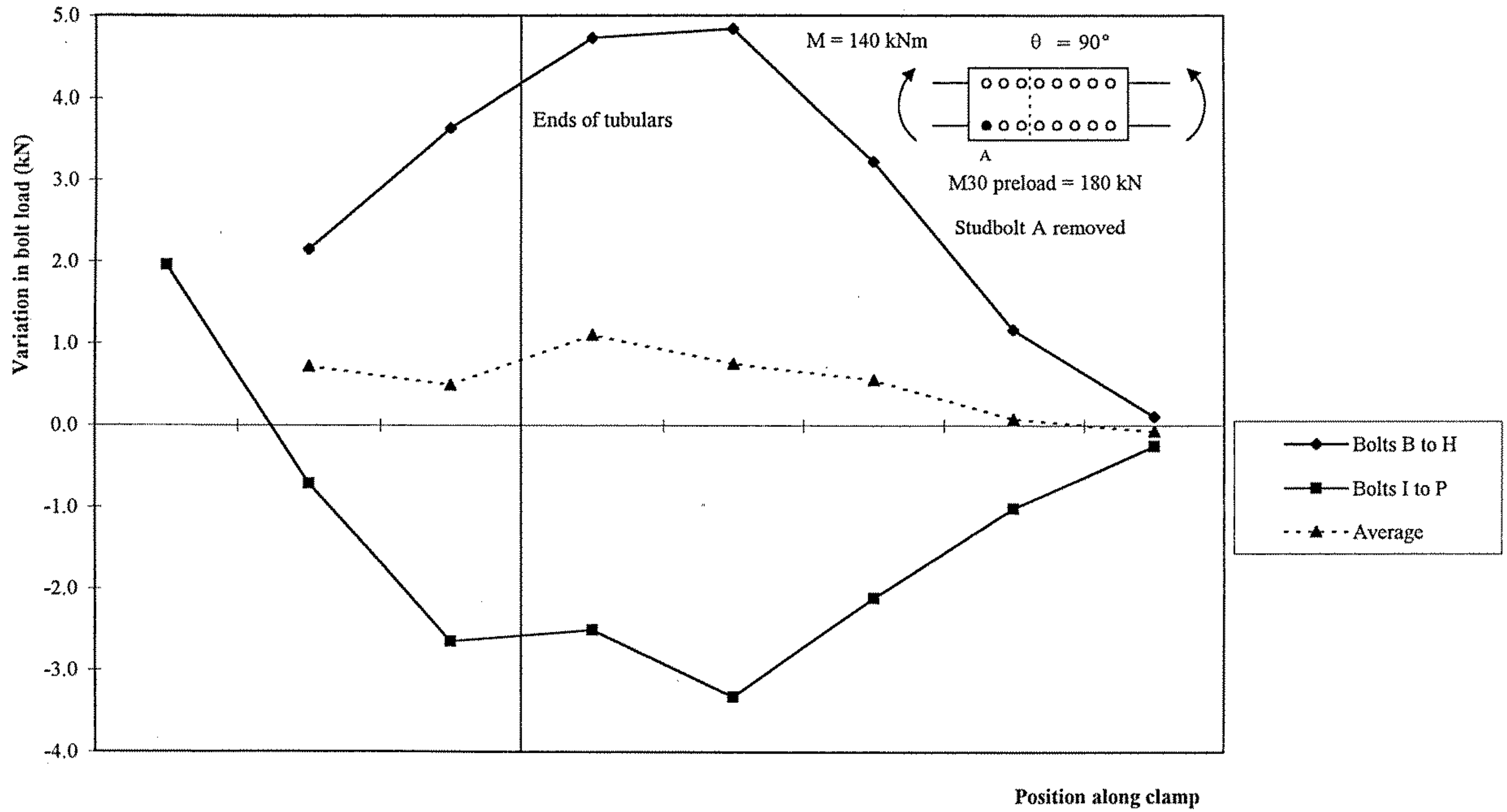

Fig 3.18 : Studbolt load distribution: Test 16 (grouted clamp)

S M:\C ... 124\GROUT\T15-17.XLS\Chart16

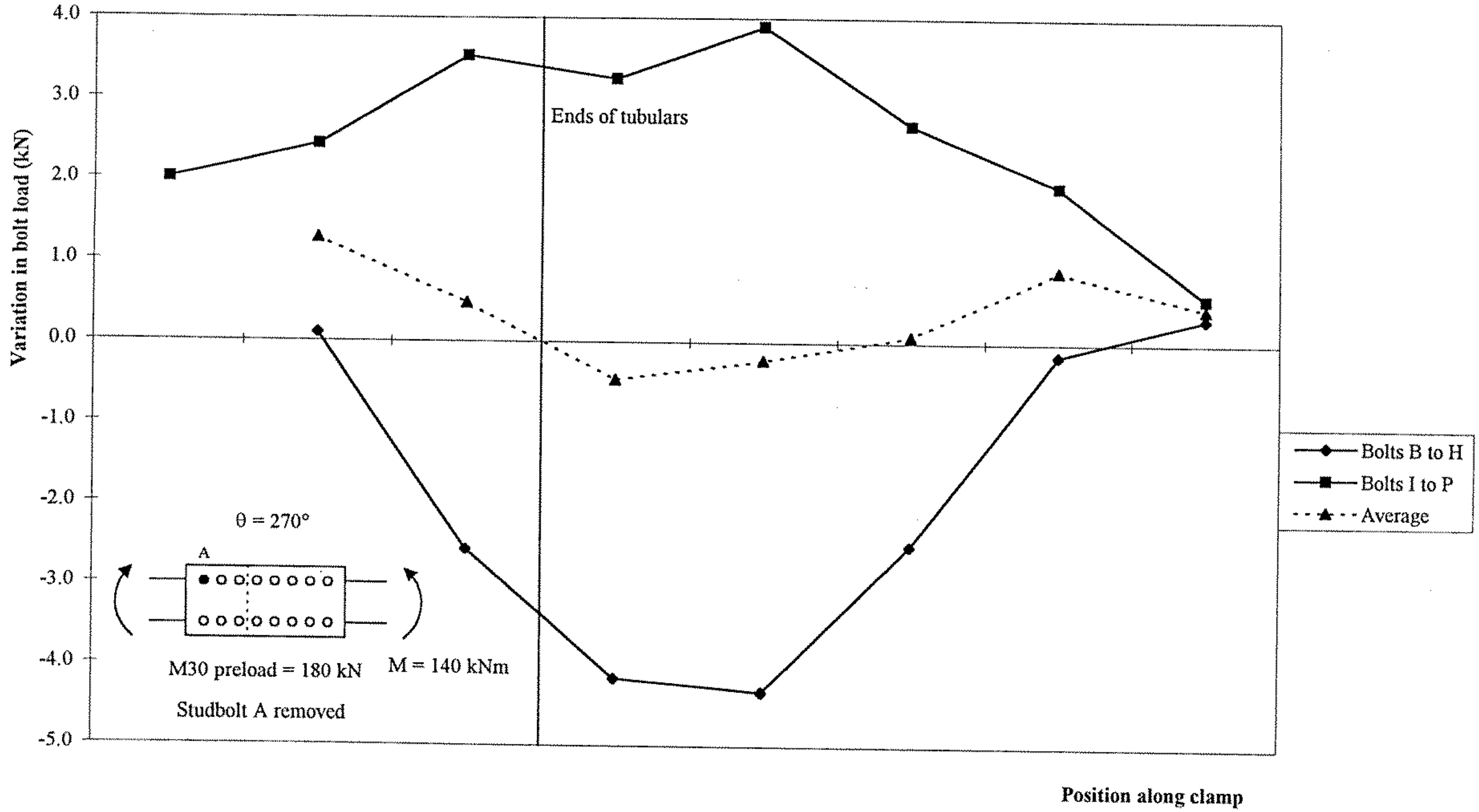

Fig 3.19 : Studbolt load distribution: Test 17 (grouted clamp)

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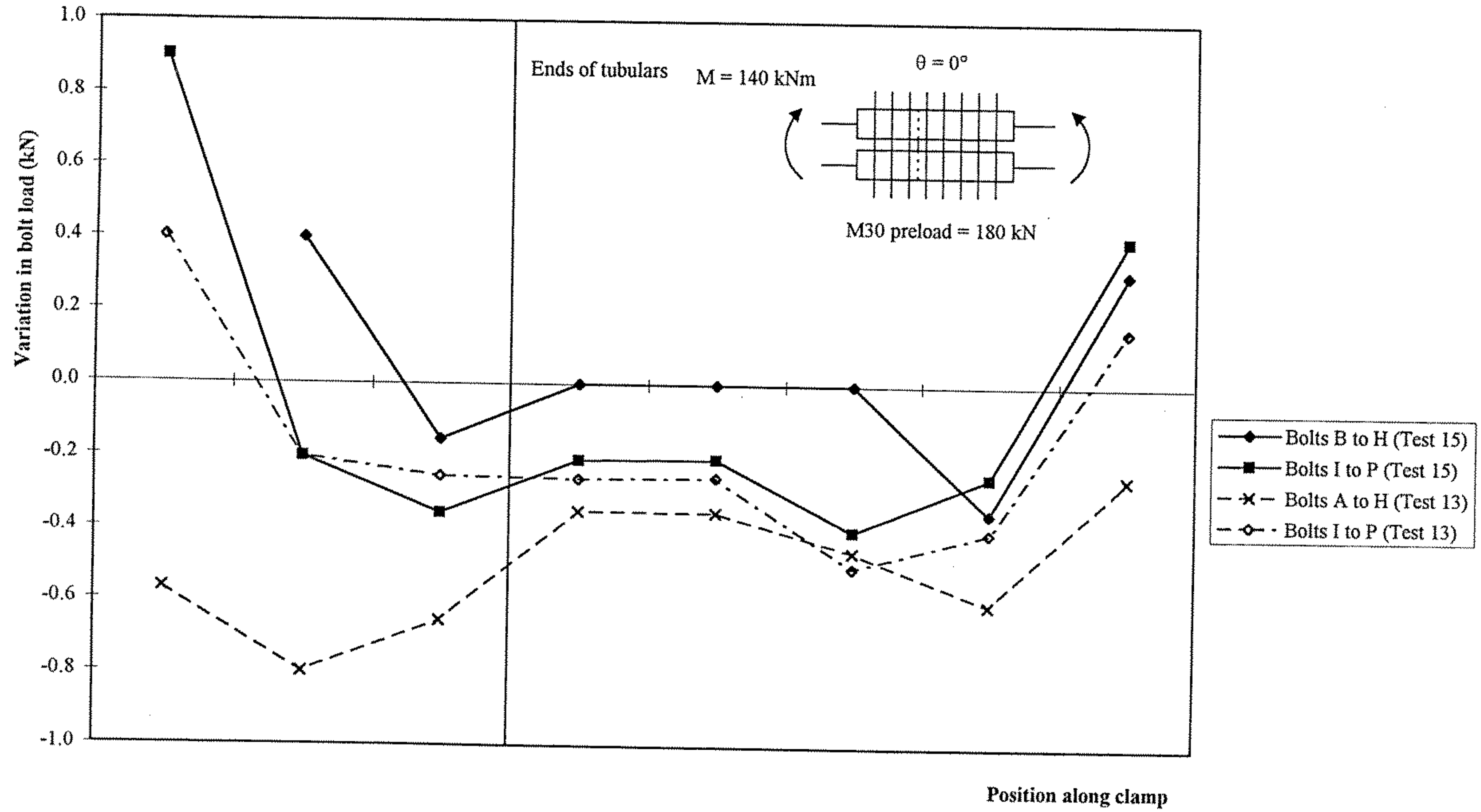

Fig 3.20 : Studbolt load distribution: Test 13 and Test 15 (grouted clamp)

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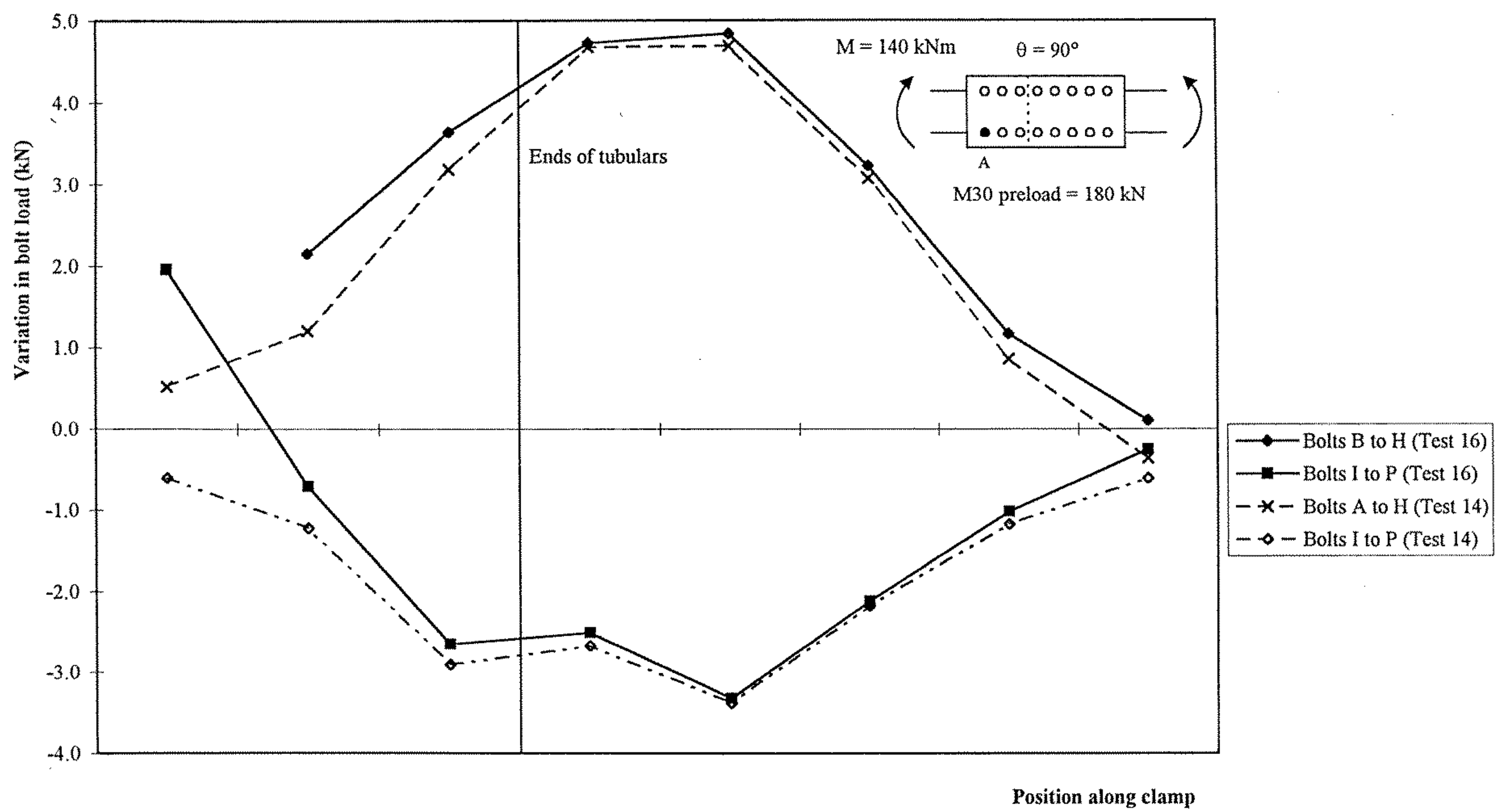

Fig 3.21 : Studbolt load distribution: Test 14 and Test 16 (grouted clamp)

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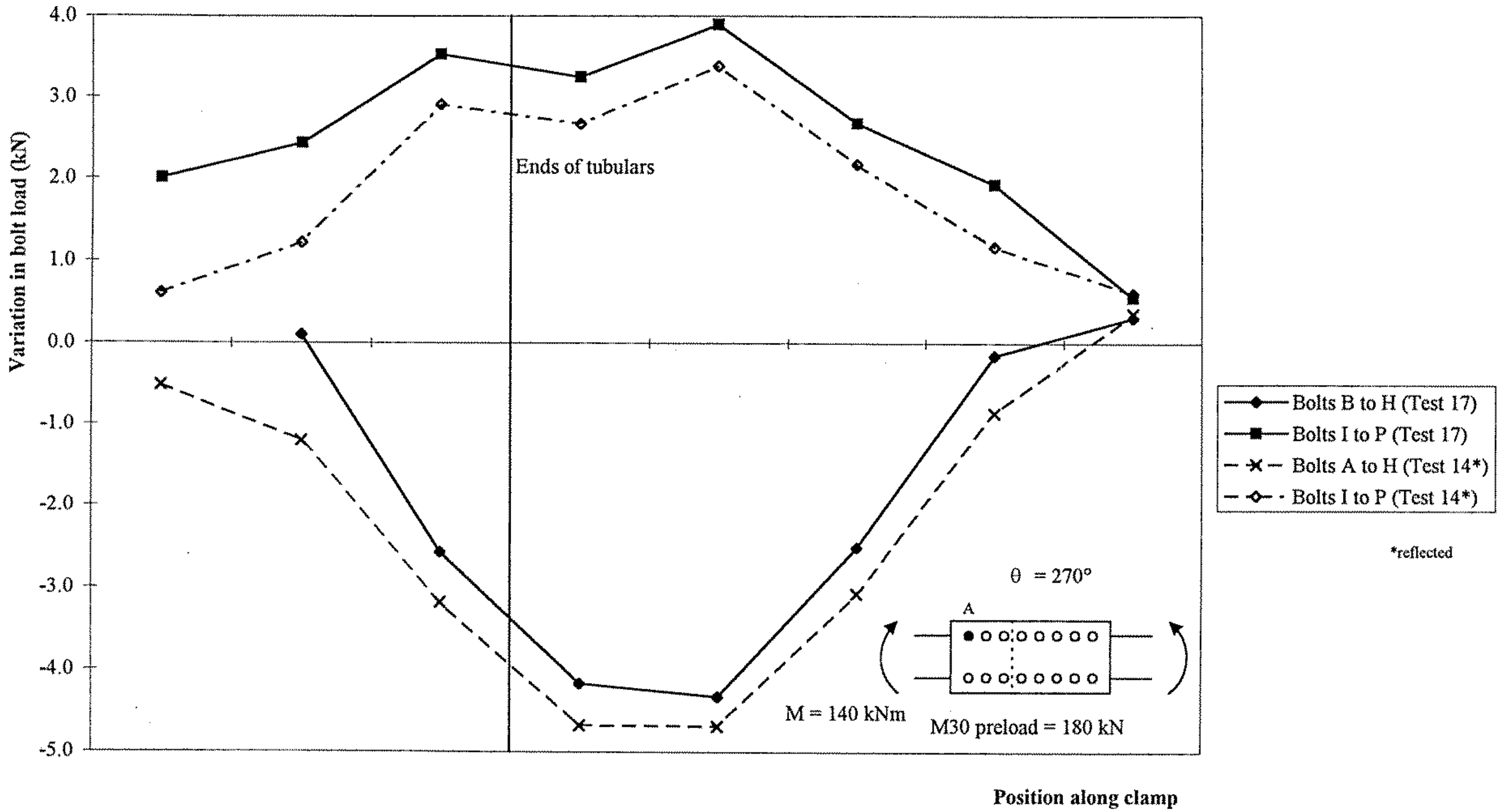


Fig 3.22 : Studbolt load distribution: Test 14 (reflected) and Test 17 (grouted clamp)

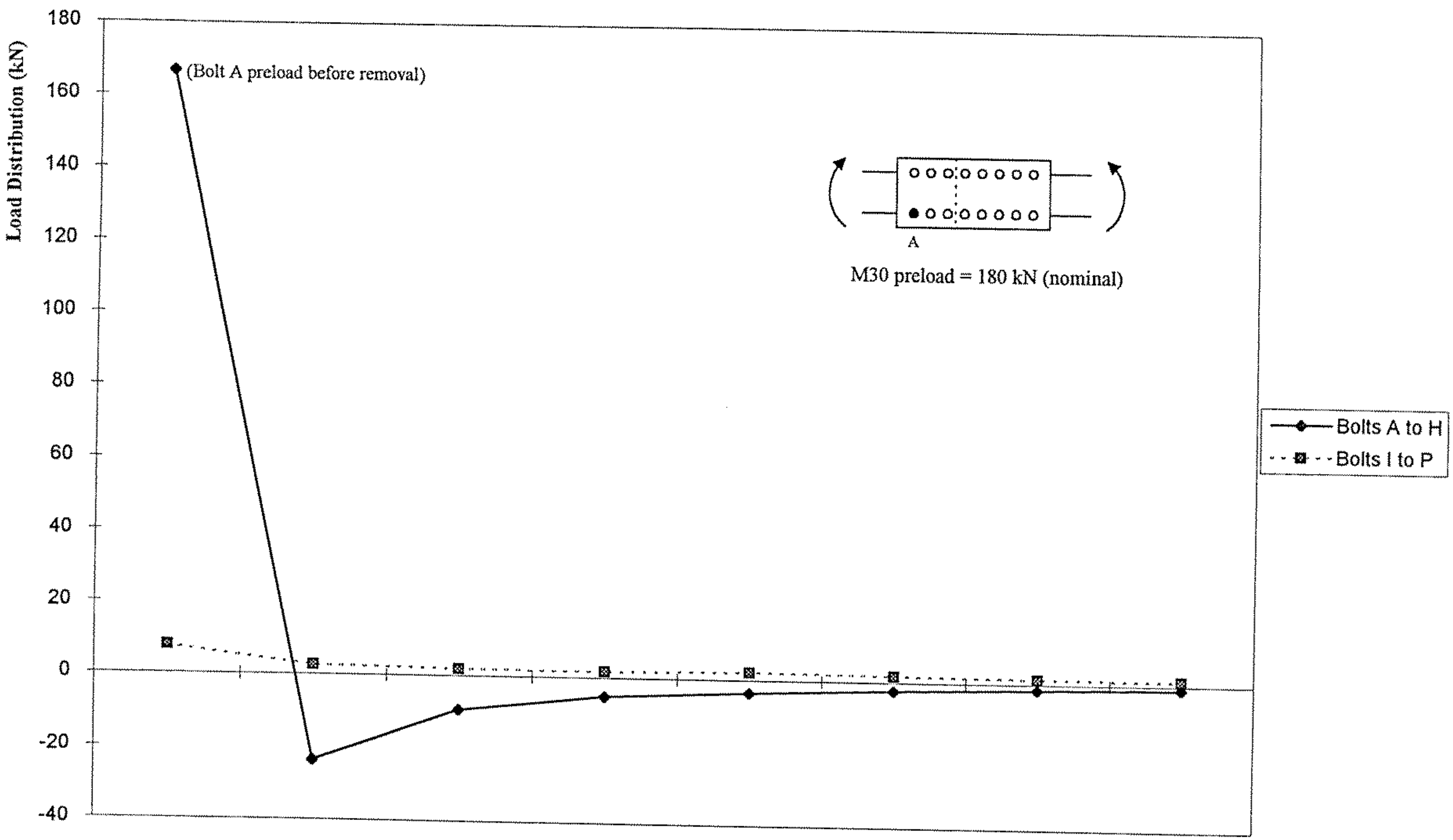
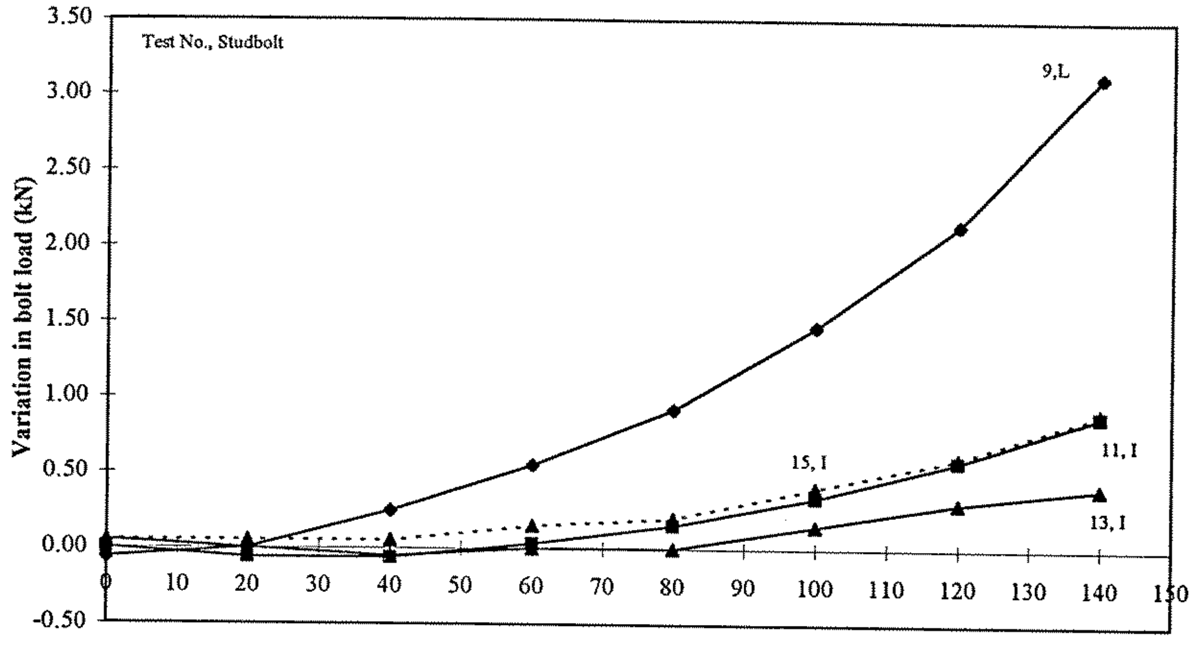
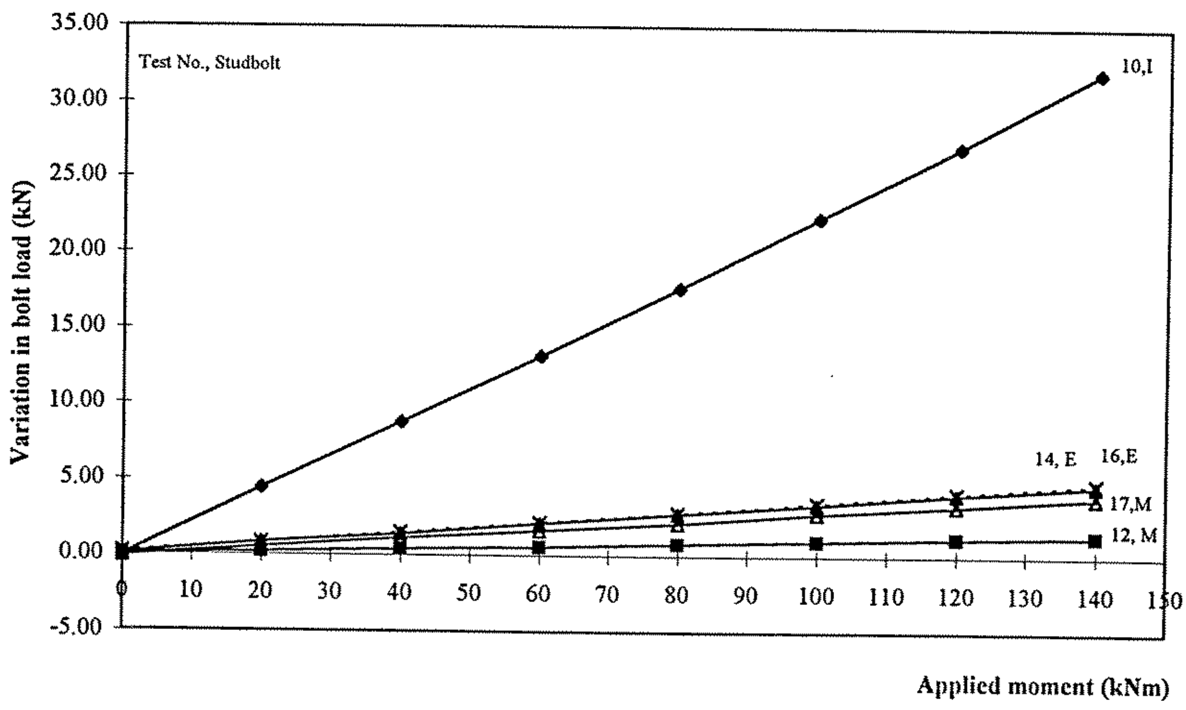


Fig 3.23 : Change in studbolt preload with removal of studbolt A

S M:\C167\m30\MSL_AOFF.XLS\Fig 3.23



Variation of studbolt load with applied moment (0° Bending)



Variation of studbolt load with applied moment 90° Bending

Fig 3.24 : Variation of studbolt load with applied moment (critical studbolt in each test)

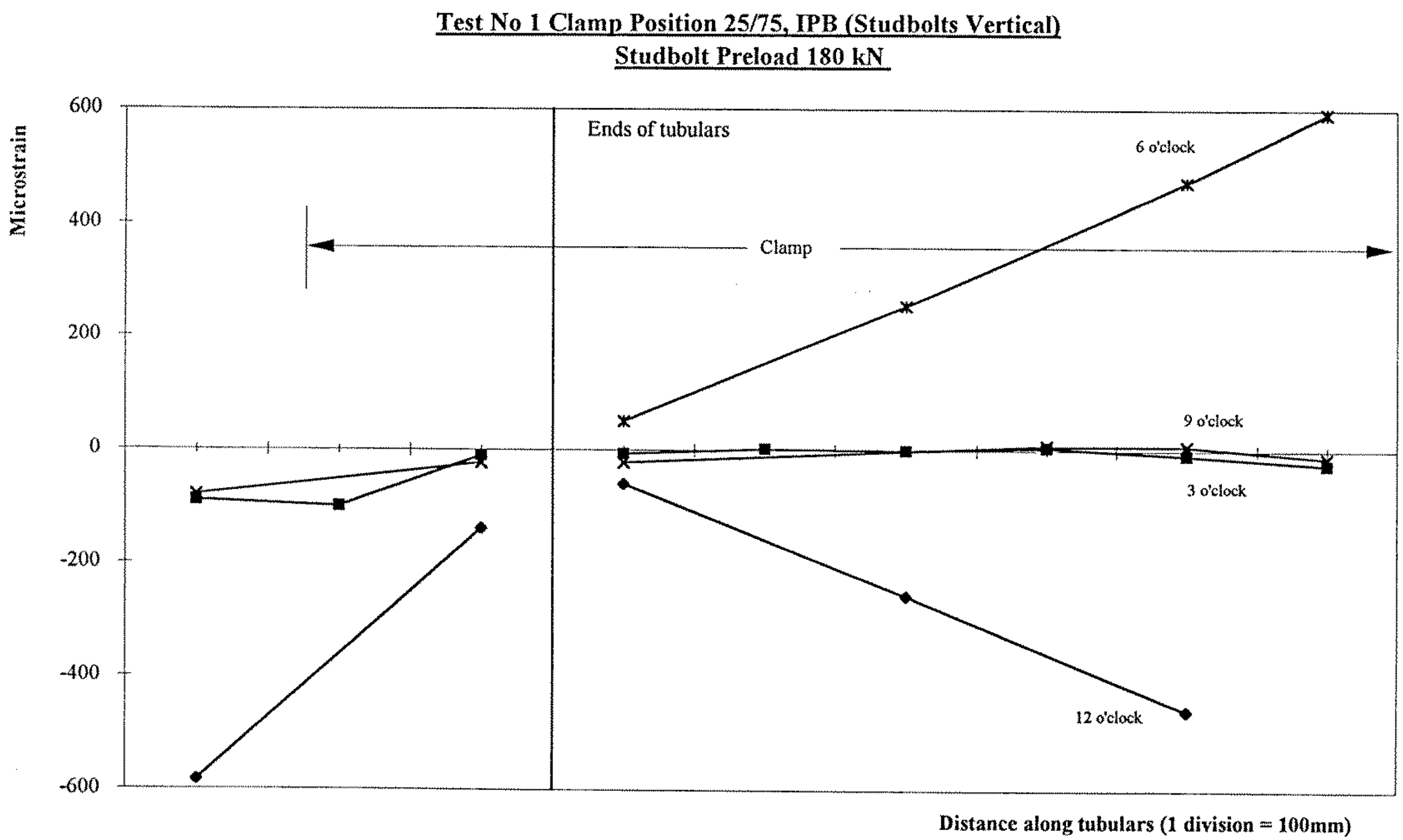


Fig 3.25 : Longitudinal strain in tubulars: Test 1

S M:\C167\tests\CITYSTR.XLS\P1

Test No 2 Clamp Position 25/75, OPB (Studbolts Horizontal)
Studbolt Preload 180 kN

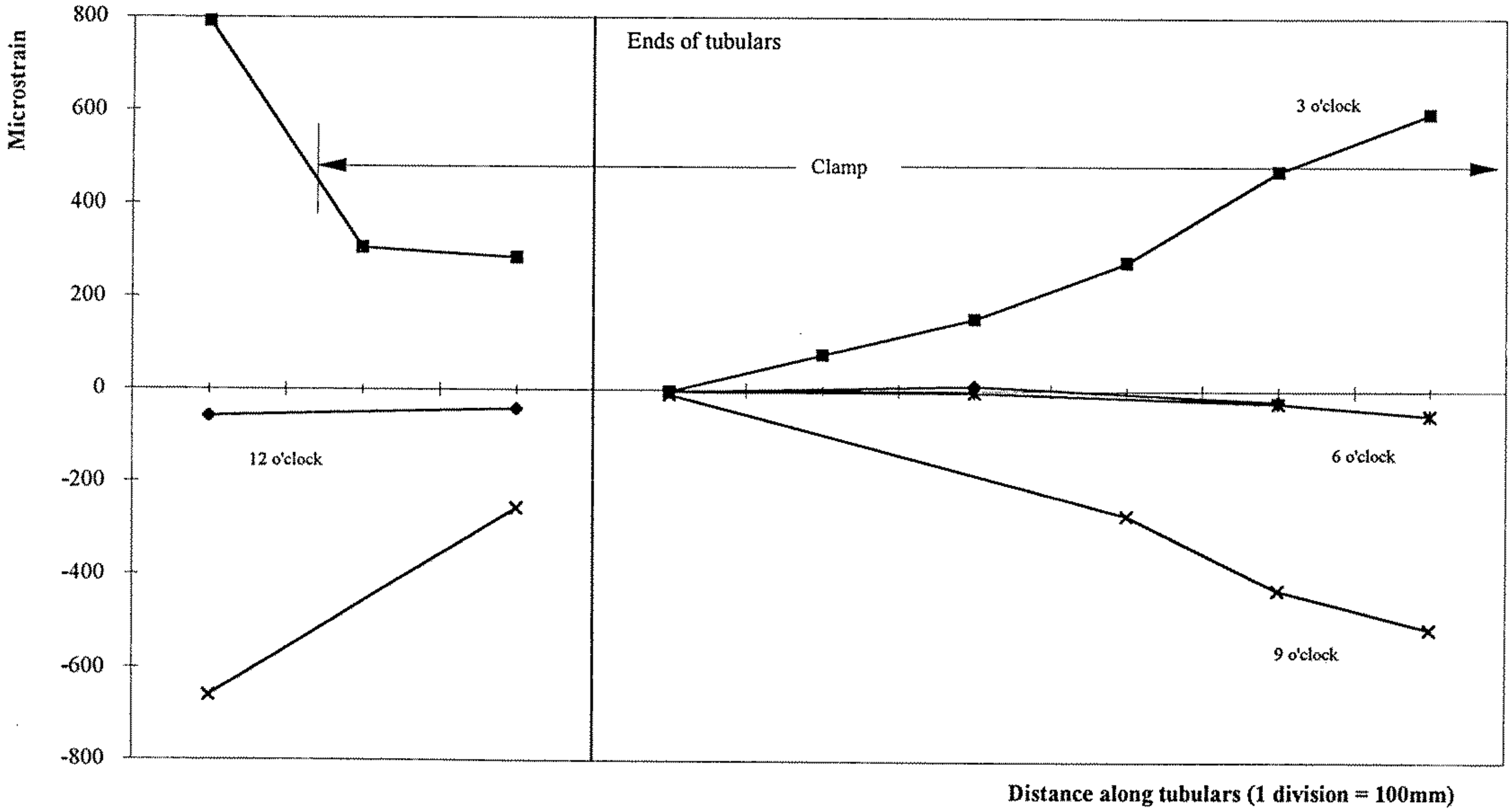


Fig 3.26 : Longitudinal strains in tubulars: Test 2

Test No 3 Clamp Position 37.5/62.5, IPB (Studbolts Vertical)
Studbolt Preload 90 kN

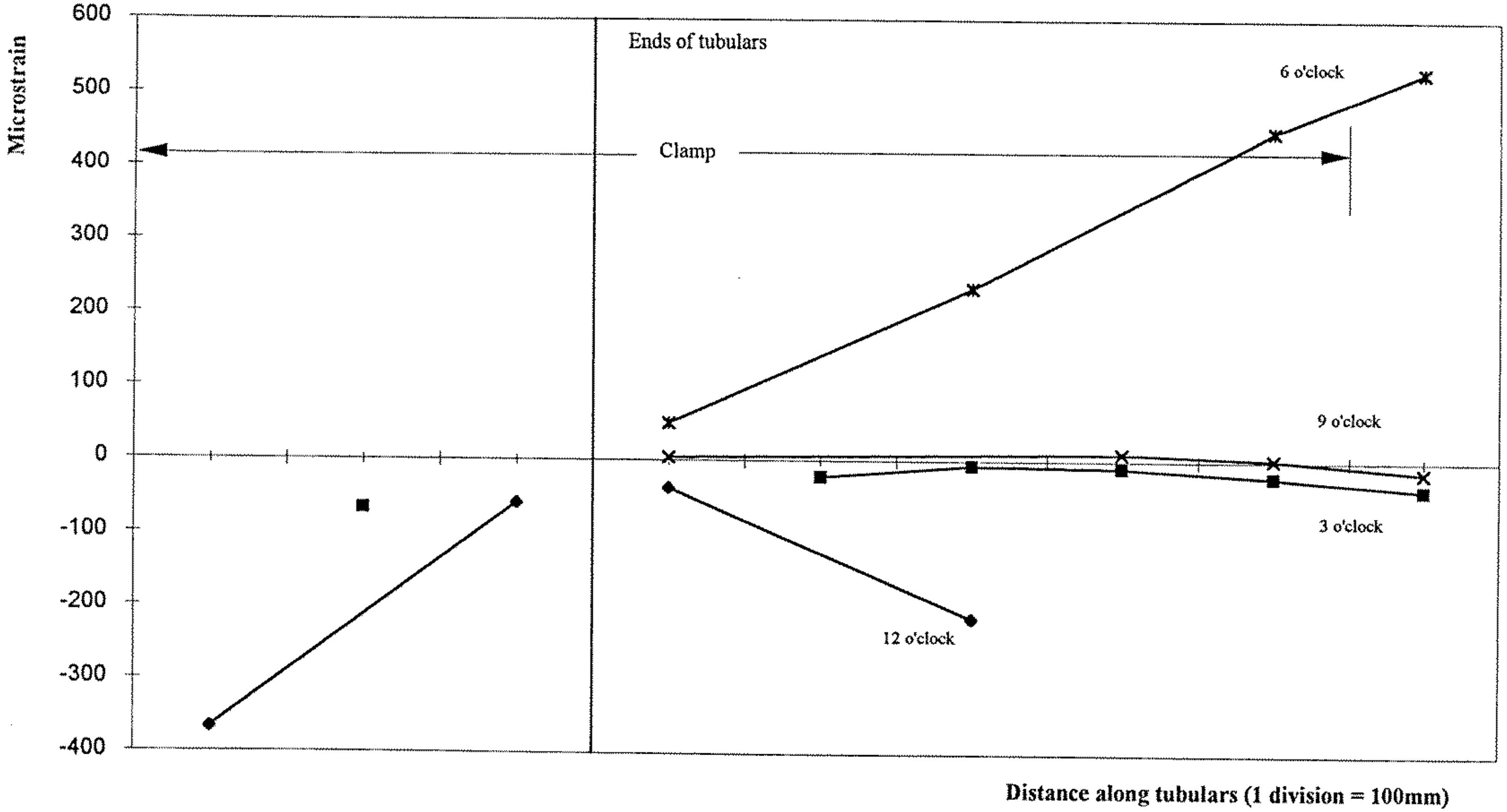


Fig 3.27 : Longitudinal strains in tubulars: Test 3

S M:\C167\tests\CITYSTR.XLS\P3

Test No 4 Clamp Position 37.5/62.5, OPB (Studbolts Horizontal)
Studbolt Preload 90 kN

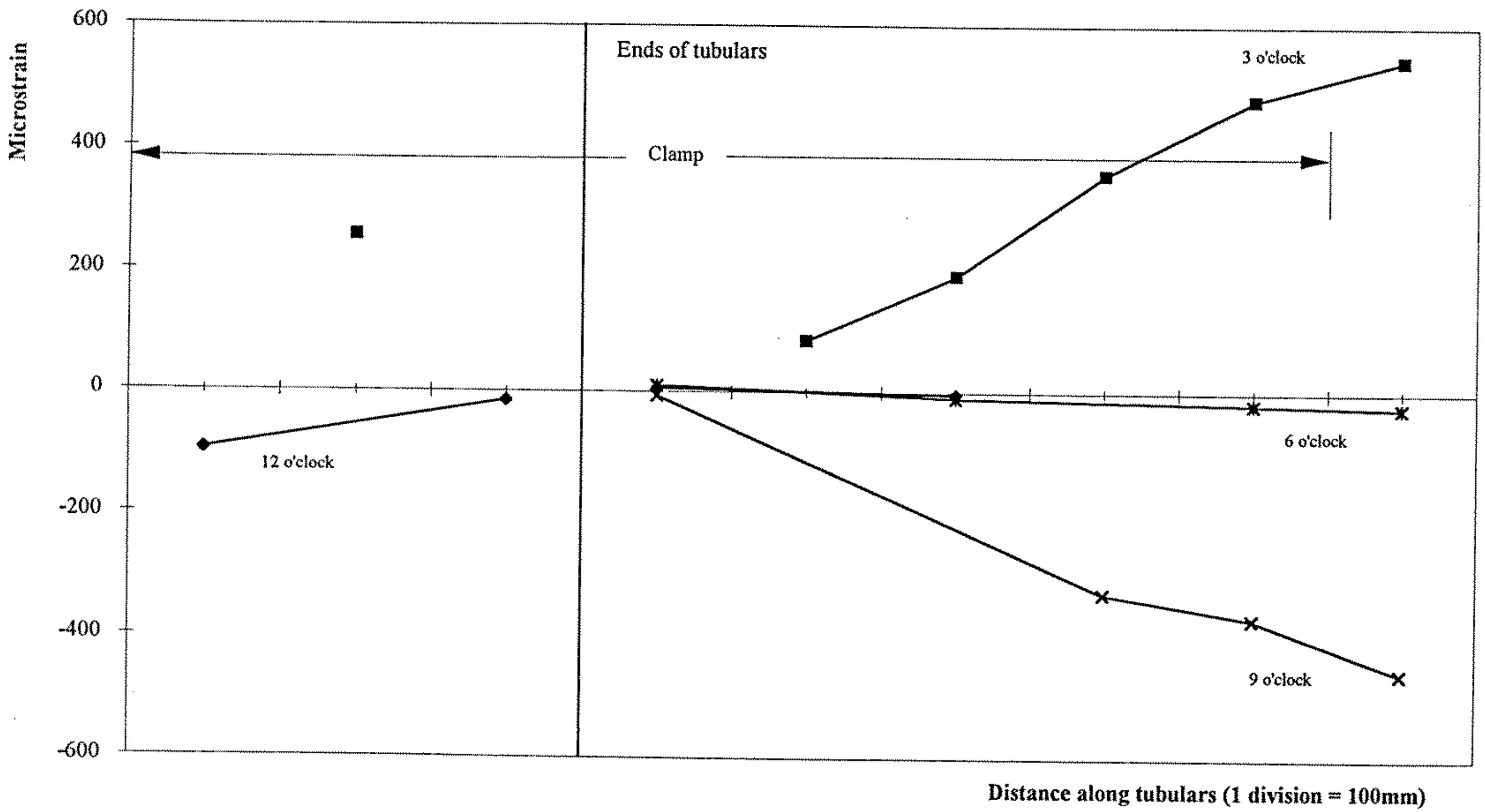


Fig 3.28 : Longitudinal strains in tubulars: Test 4

Test No 5 Clamp Position 37.5/62.5, IPB (Studbolts Vertical)
Studbolt Preload 180 kN

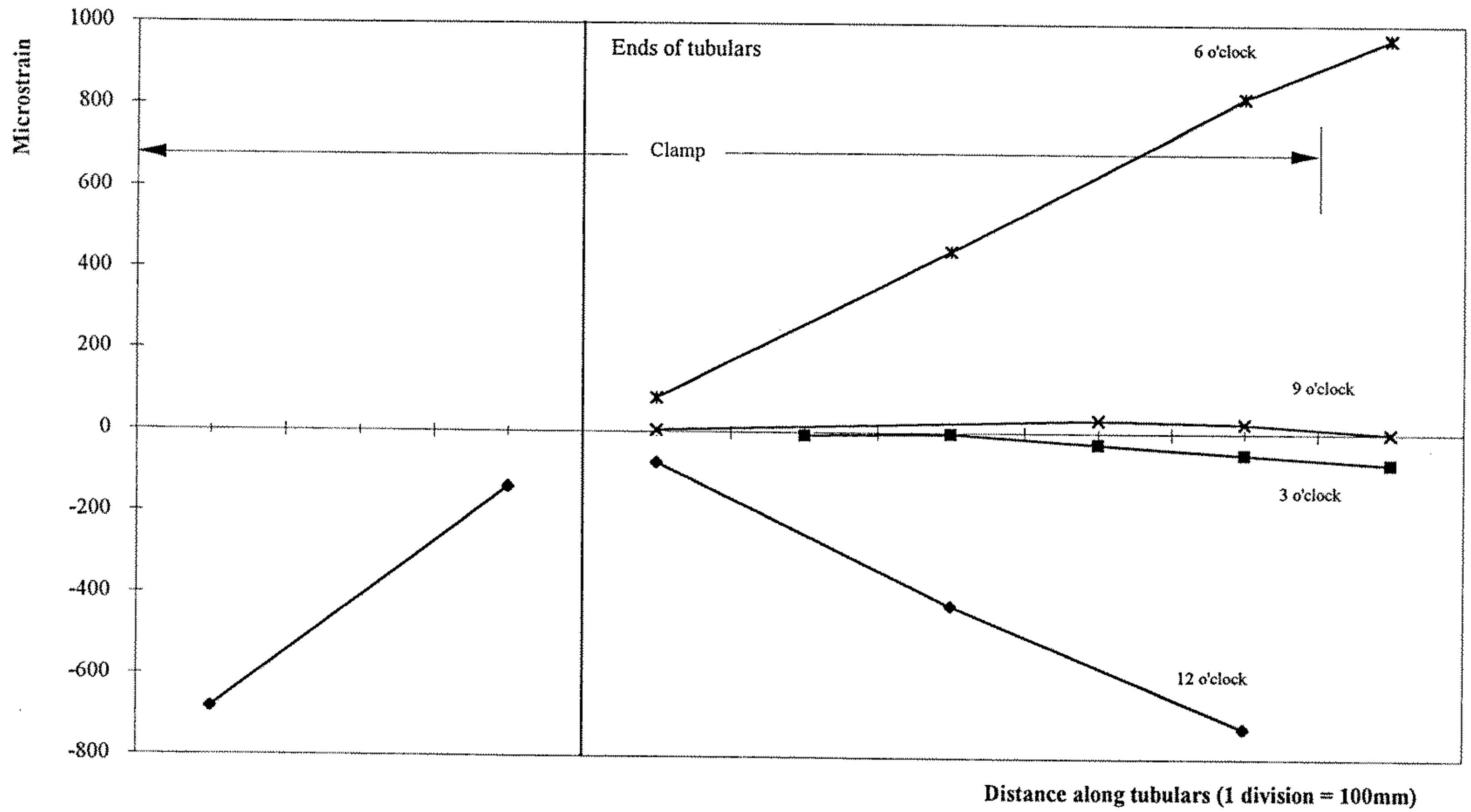


Fig 3.29 : Longitudinal strains in tubulars: Test 5

S M:\C167\tests\CITYSTR.XLS\P5

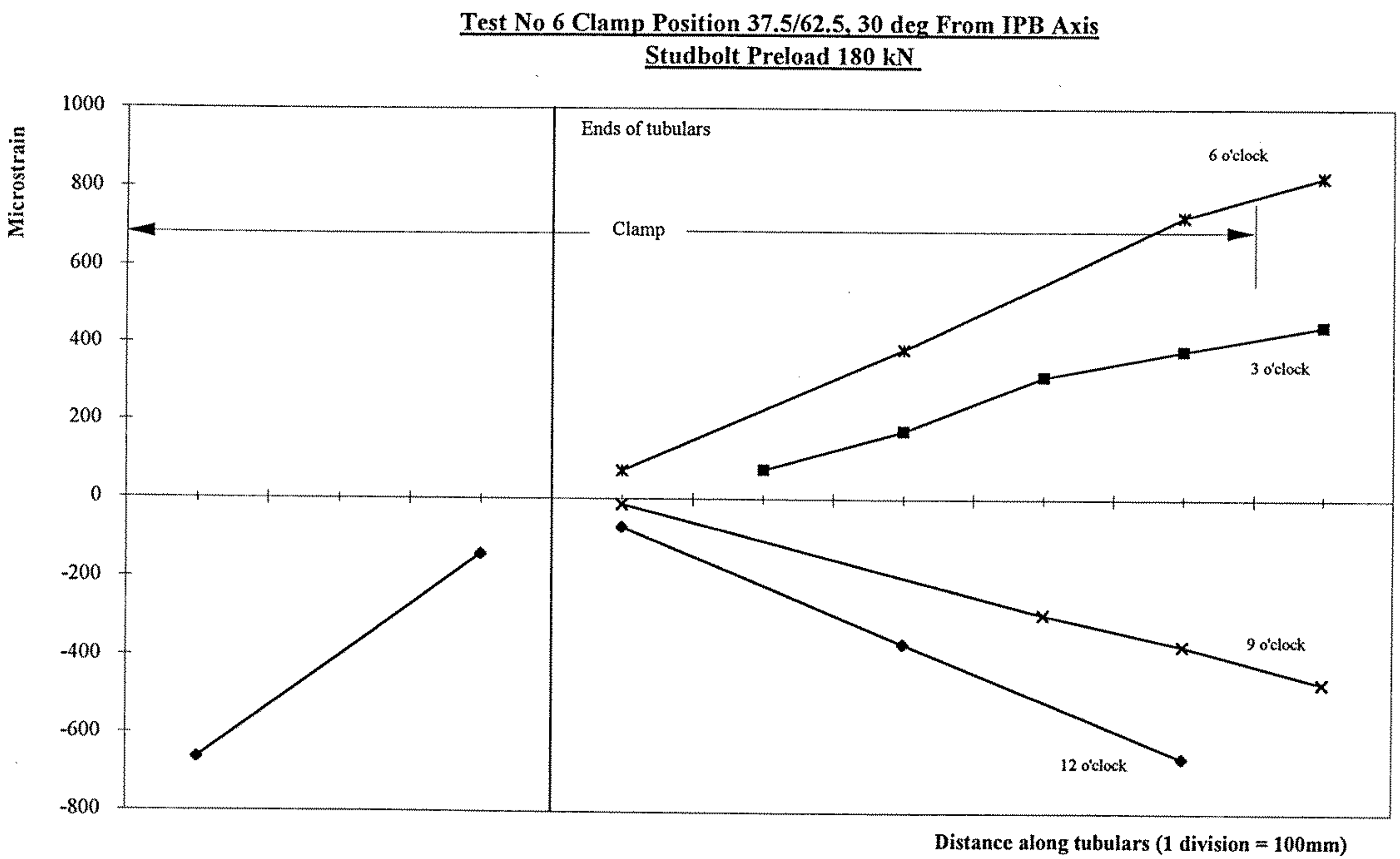


Fig 3.30 : Longitudinal strains in tubulars: Test 6

S M:\C...sts\CITYSTR.XLS\P6

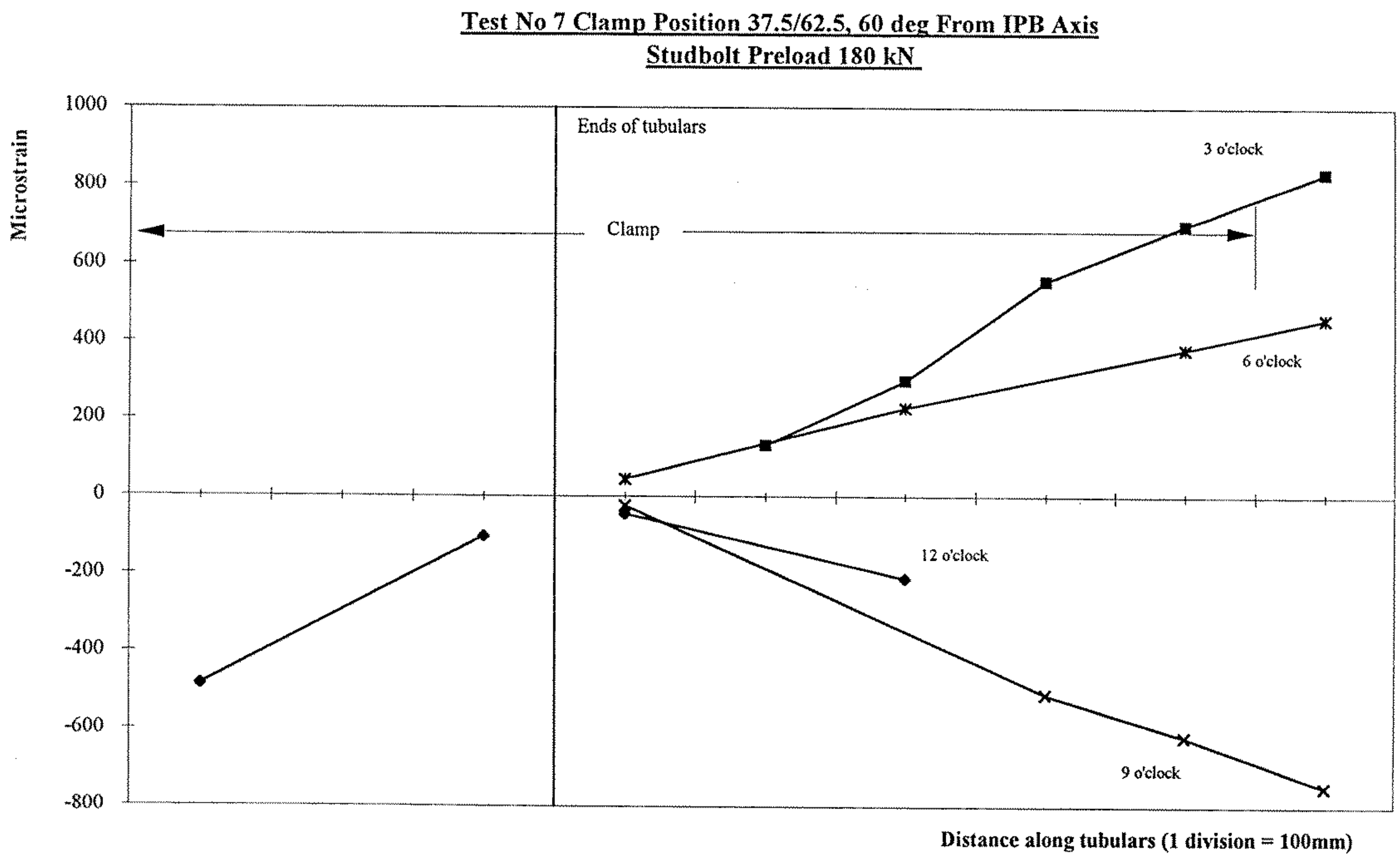


Fig 3.31 : Longitudinal strains in tubulars: Test 7

S M:\C167\tests\CITYSTR.XLS\P7

Test No 8 Clamp Position 37.5/62.5, OPB (Studbolts Horizontal)
Studbolt Preload 180 kN

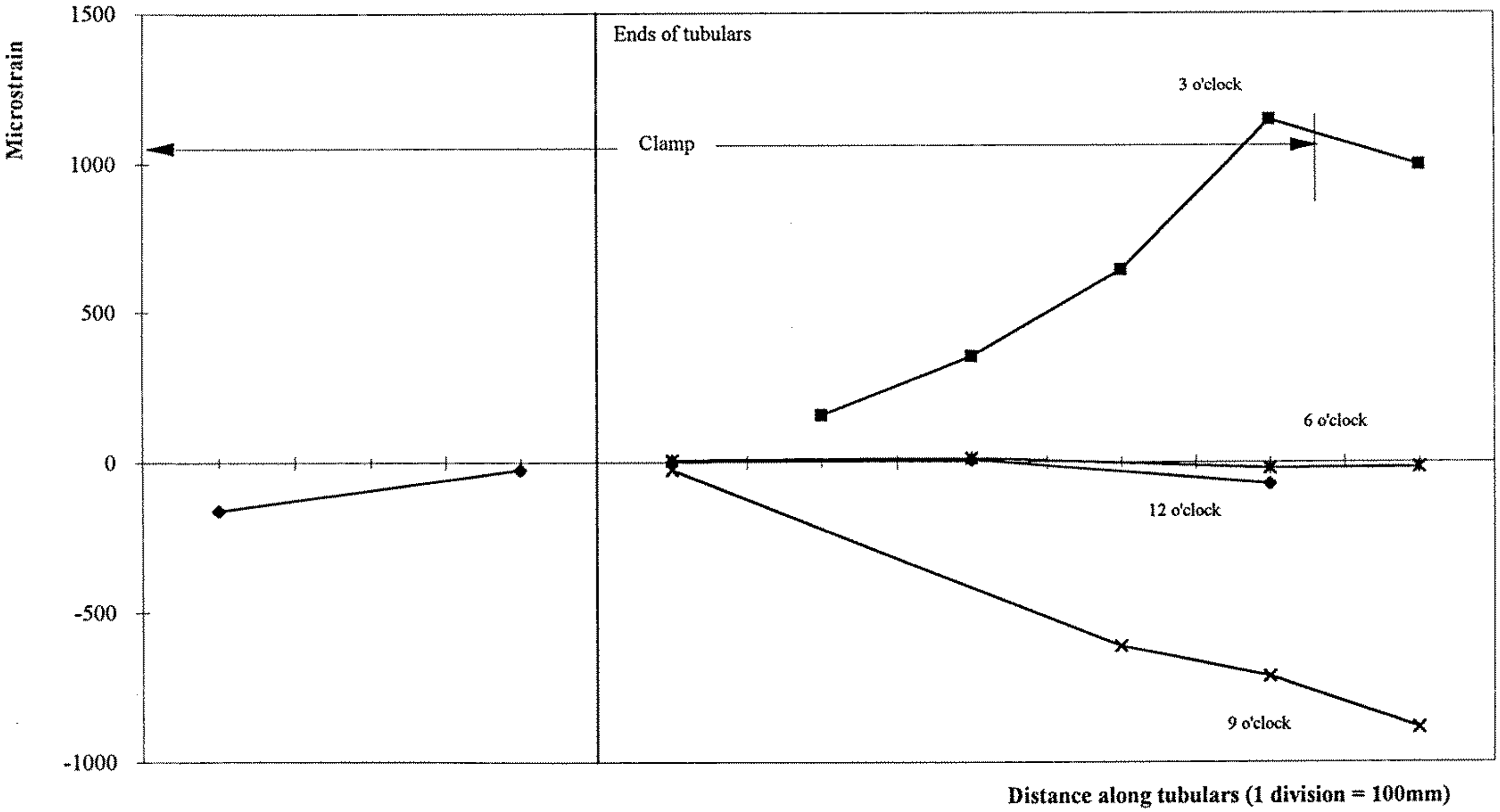


Fig 3.32 : Longitudinal strains in tubulars: Test 8

Test No 9 Clamp Position 37.5/62.5,IPB (Studbolts Vertical)
Studbolt Preload 180 kN

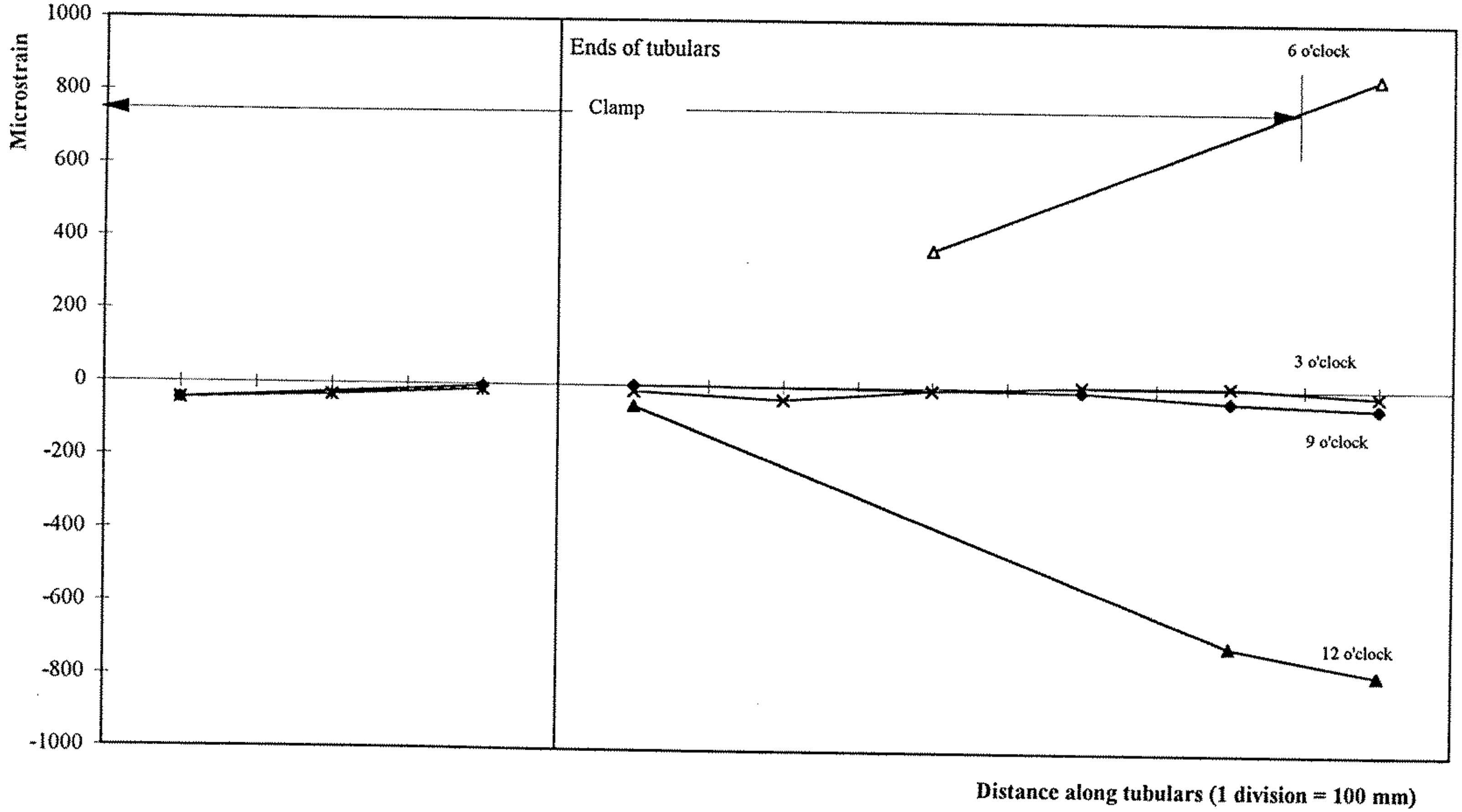


Fig 3.33 : Longitudinal strains in tubulars: Test 9

S M:\C167\M24\GROUT\TEST9.XLS\T9STRAIN

Test No 10 Clamp Position 37.5/62/5, OPB (Studbolts Horizontal)
Studbolt Prebolt 180 kN

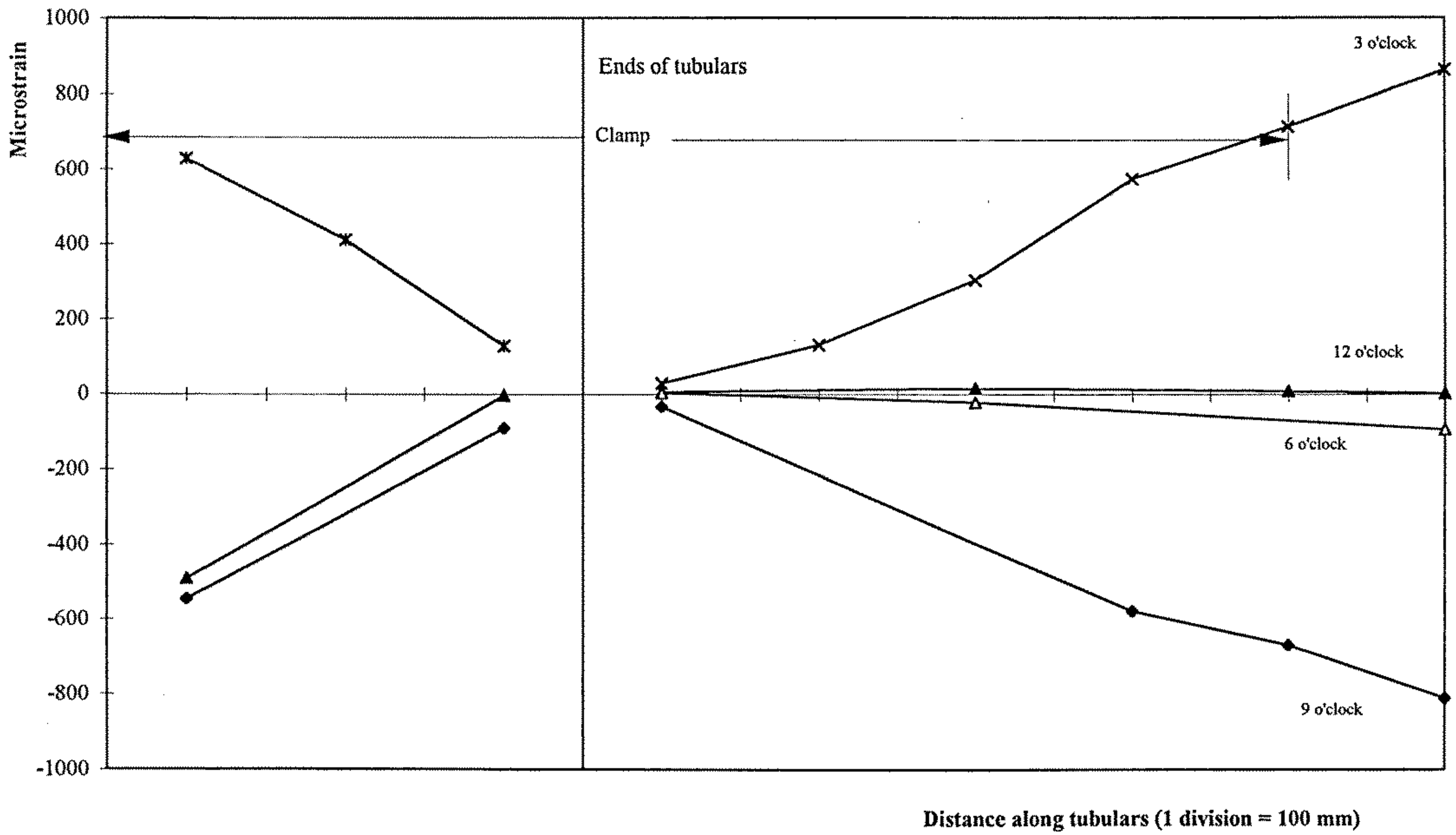


Fig 3.34 : Longitudinal strains in tubulars: Test 10

Test No 11 Clamp Position 37.5/62.5,IPB (Studbolts Vertical)
Studbolt Preload 180 kN

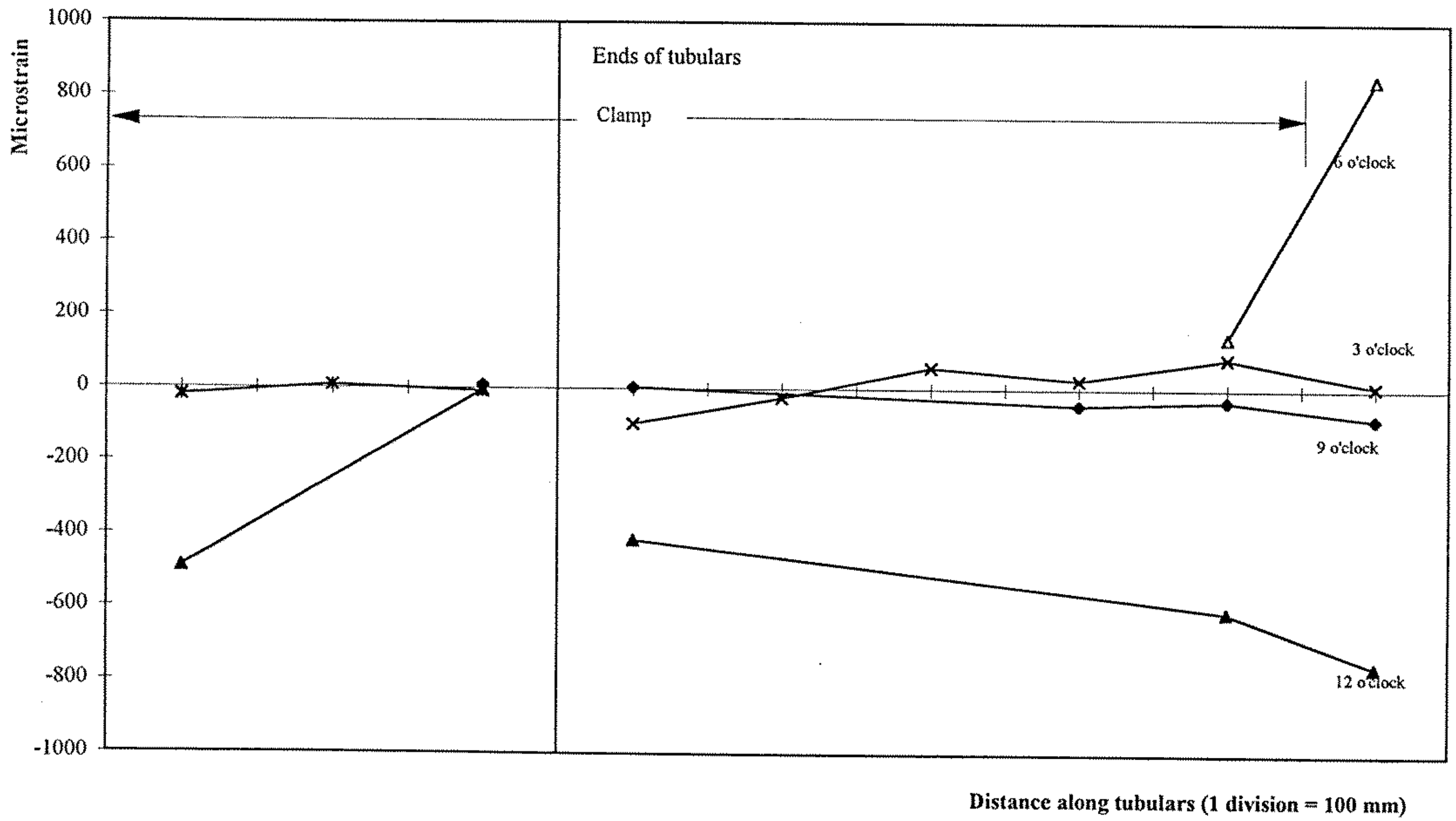


Fig 3.35 : Longitudinal strains in tubulars: Test 11

S M:\C167\M24\GROUT\TEST9.XLS\T11STRAIN

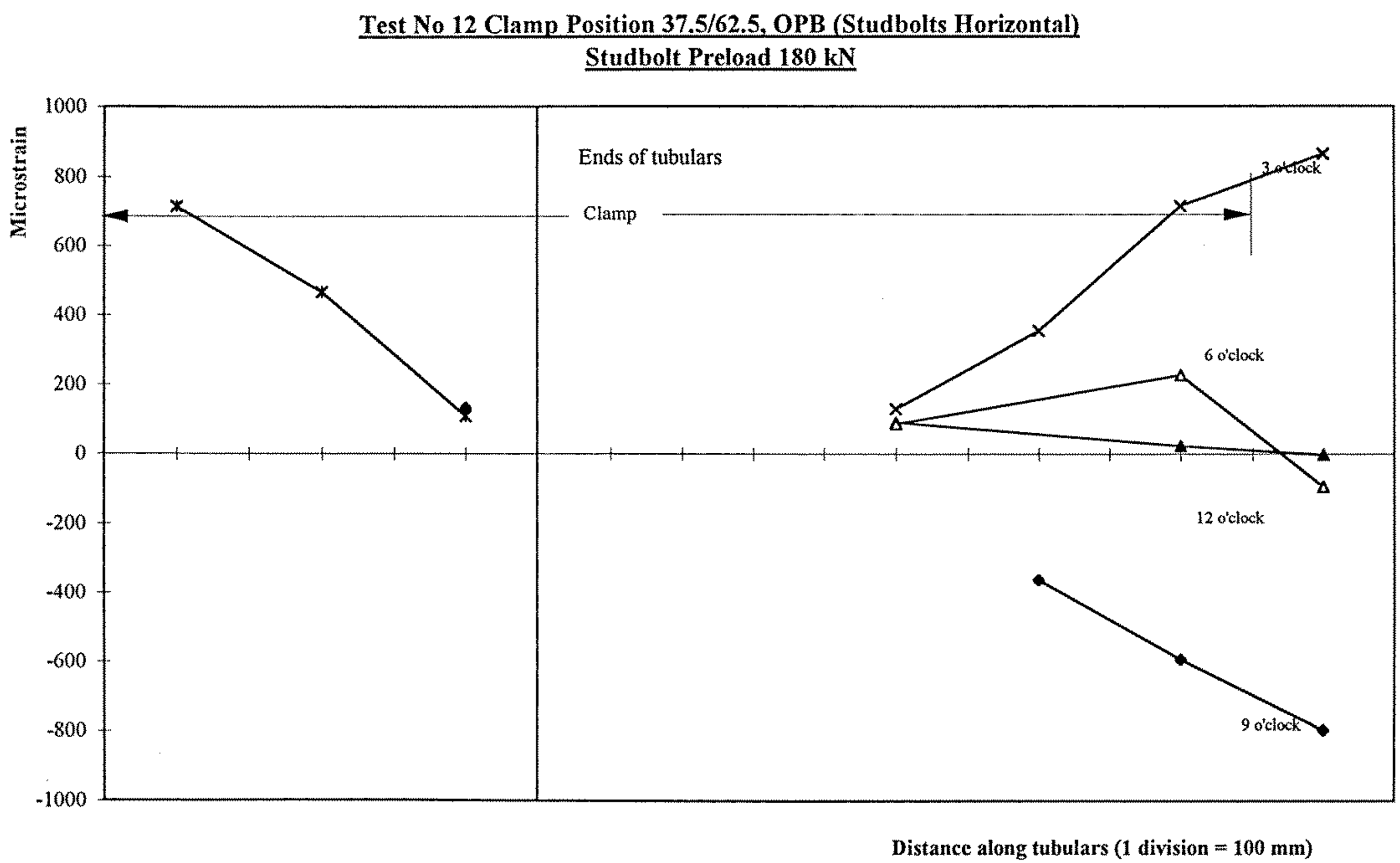


Fig 3.36 : Longitudinal strains in tubulars: Test 12

S M:\C 124\GROUT\TEST9.XLS\T12STRAIN

Test No 13 Clamp Position 37.5/62.5,IPB (Studbolt Vertical)
Studbolt Preload 180 kN

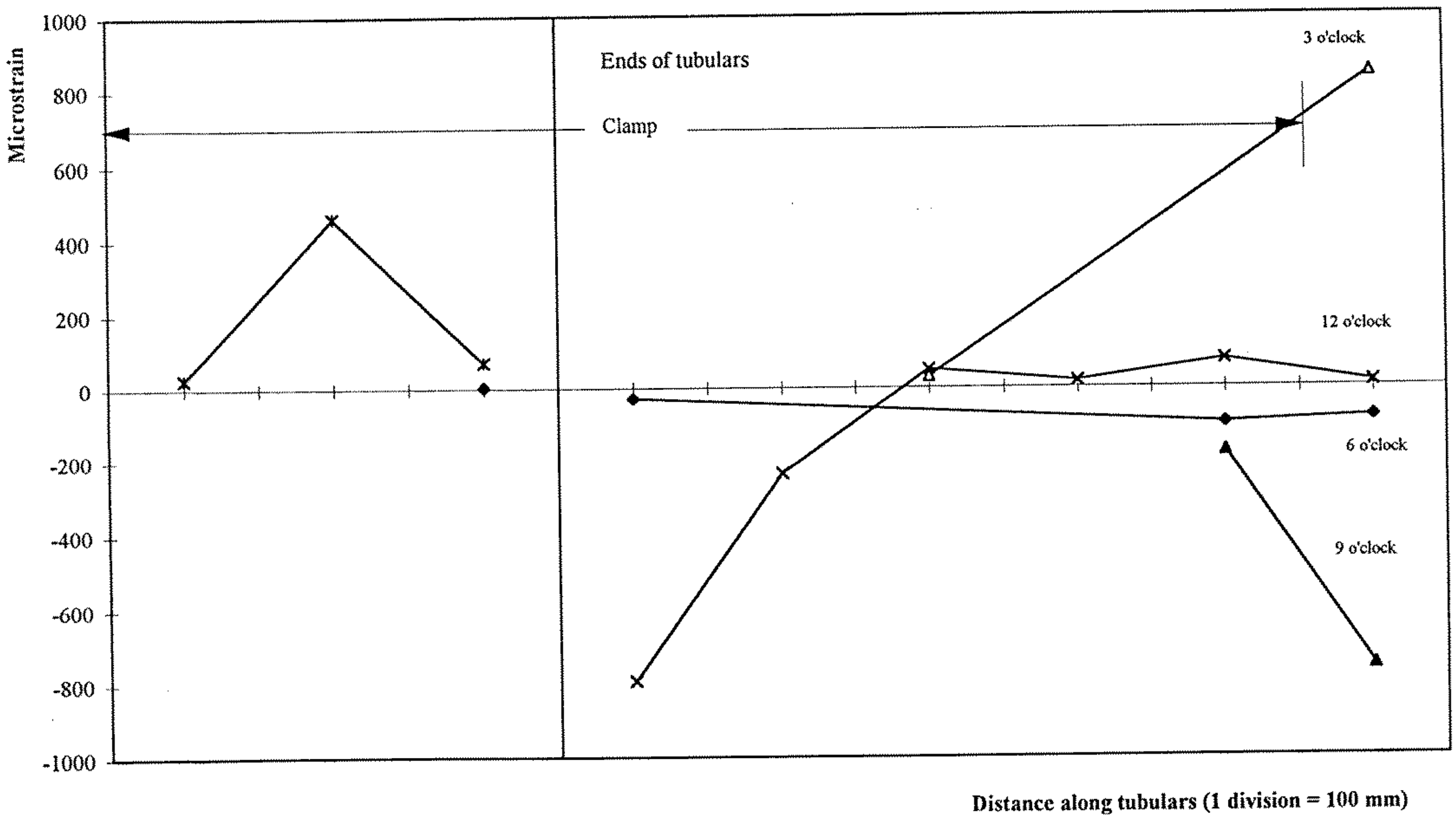


Fig 3.37 : Longitudinal strains in tubulars: Test 13

S M:\C167\M24\GROUT\TEST9.XLS\T13STRAIN

Test No 14 Clamp Position 37.5/62.5,OPB (Studbolts Horizontal)
Studbolt Preload 180 kN

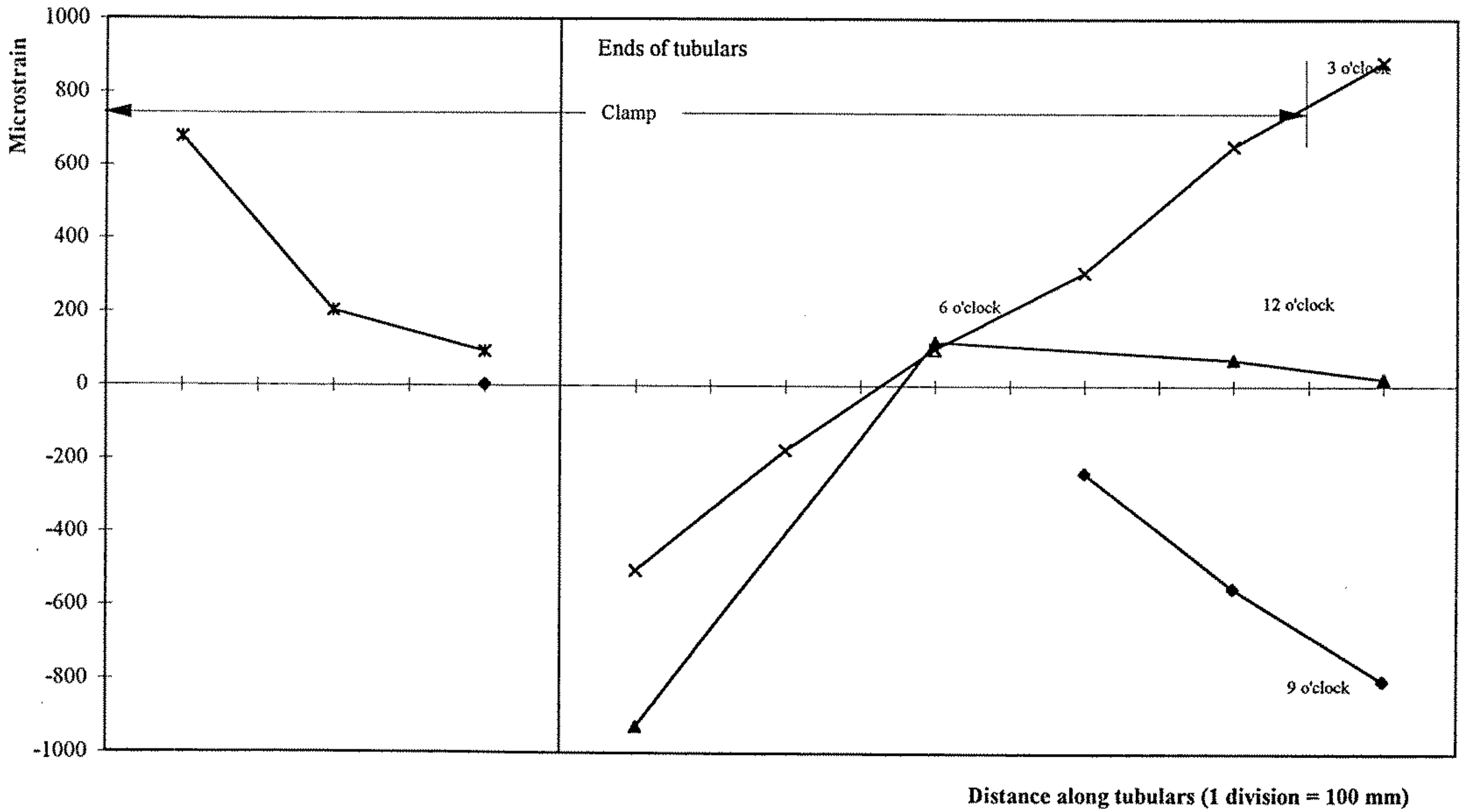


Fig 3.38 : Longitudinal strains in tubulars: Test 14

S M:\C[illegible]424\GROUT\TEST9.XLS\T14STRAIN

Test No 15 Clamp Position 37.5/62.5,IPB (Studbolts Horizontal)
Studbolt Preload 180 kN

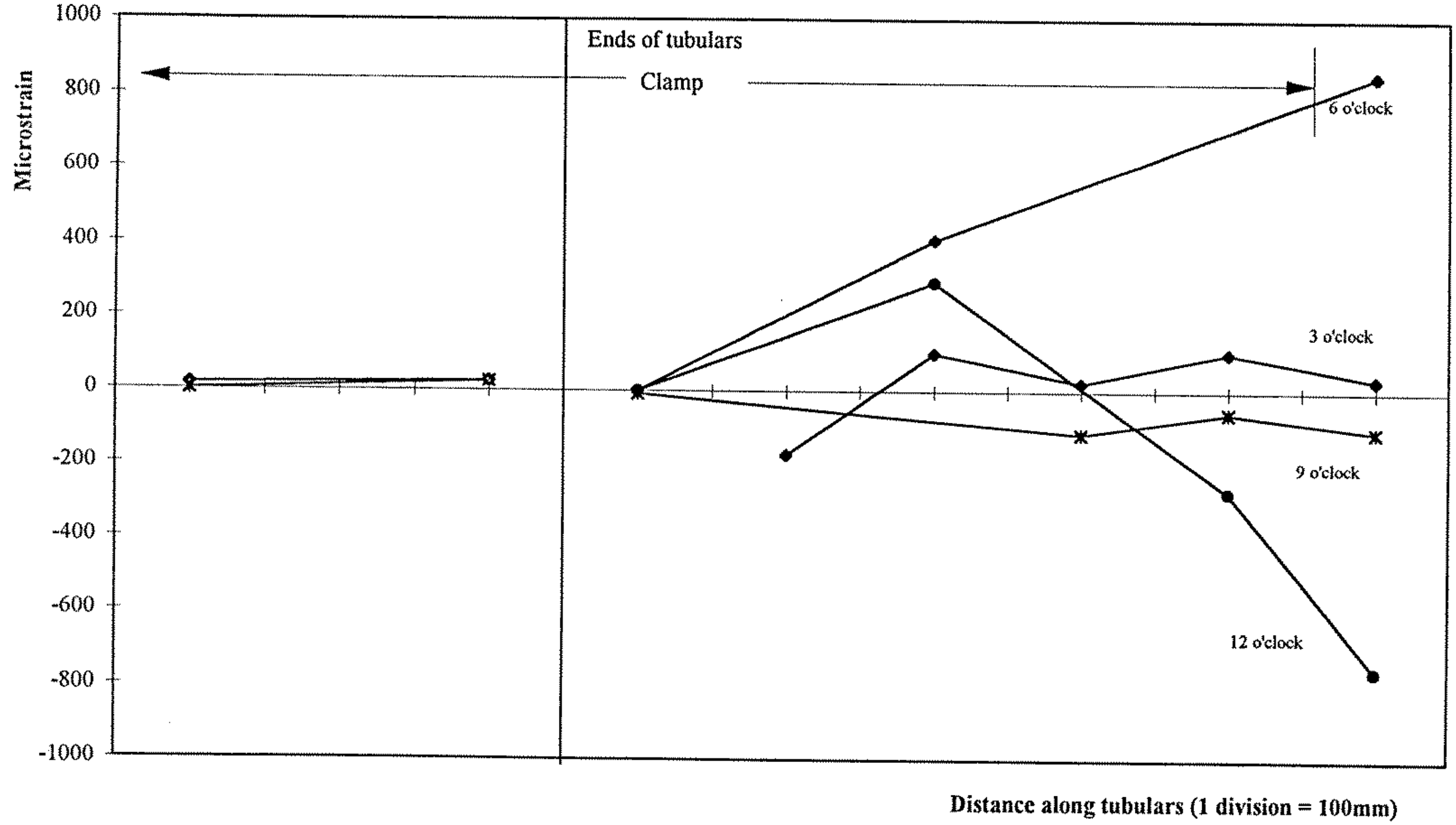


Fig 3.39 : Longitudinal strain in tubulars: Test 15

S M:\C167\M24\GROUT\T15-17.XLS\T15strain

Test No 16 Clamp Position 37.5/62.5,OPB (Studbolts Horizontal)
Studbolt Preload 180 kN

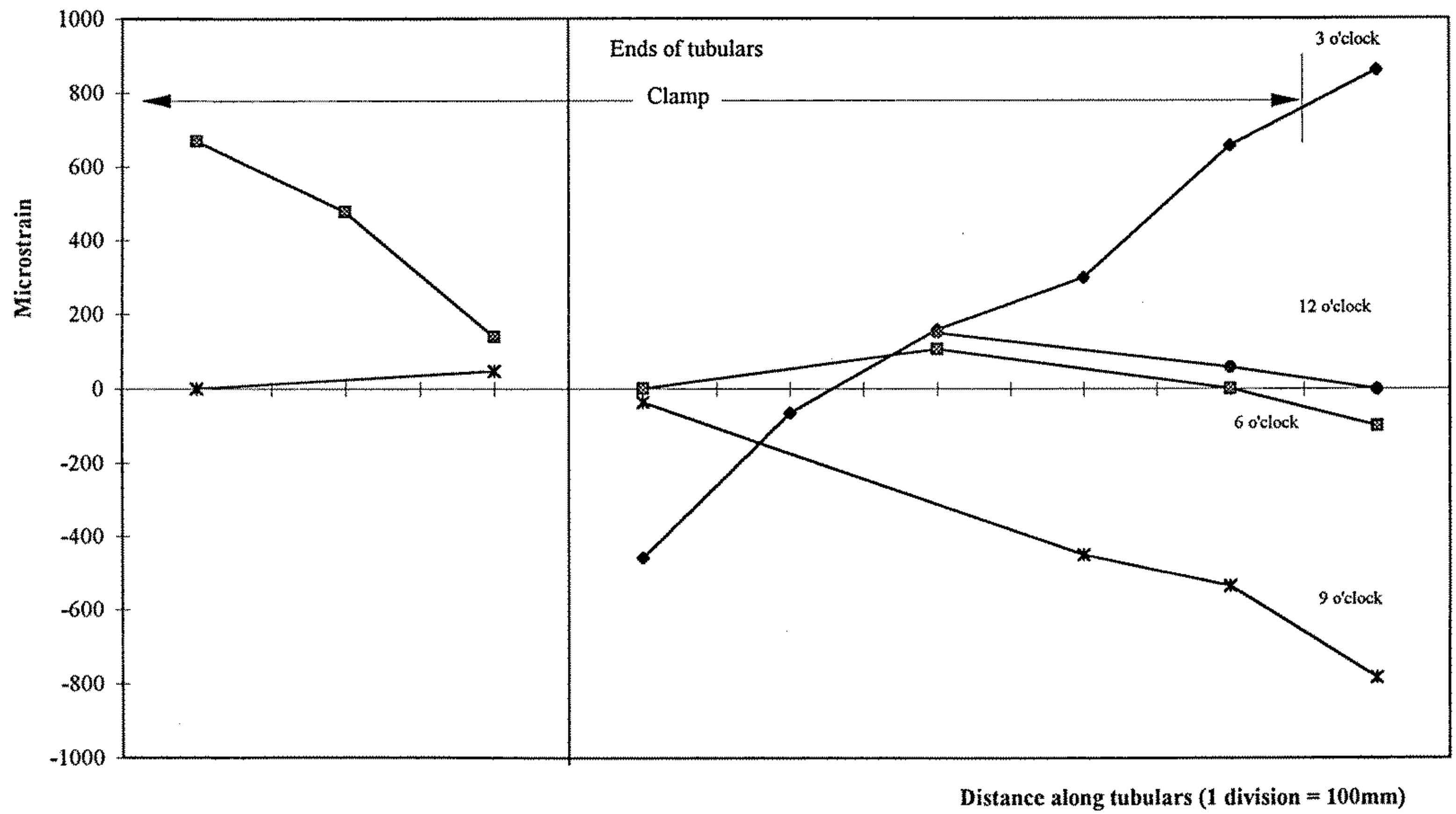


Fig 3.40 : Longitudinal strain in tubulars: Test 16

S M:\C...M24\GROUT\T15-17.XLS\T16strain

Test No 17 Clamp Position 37.5/62.5, 270 deg from IPB Axis
Studbolt Preload 180 kN

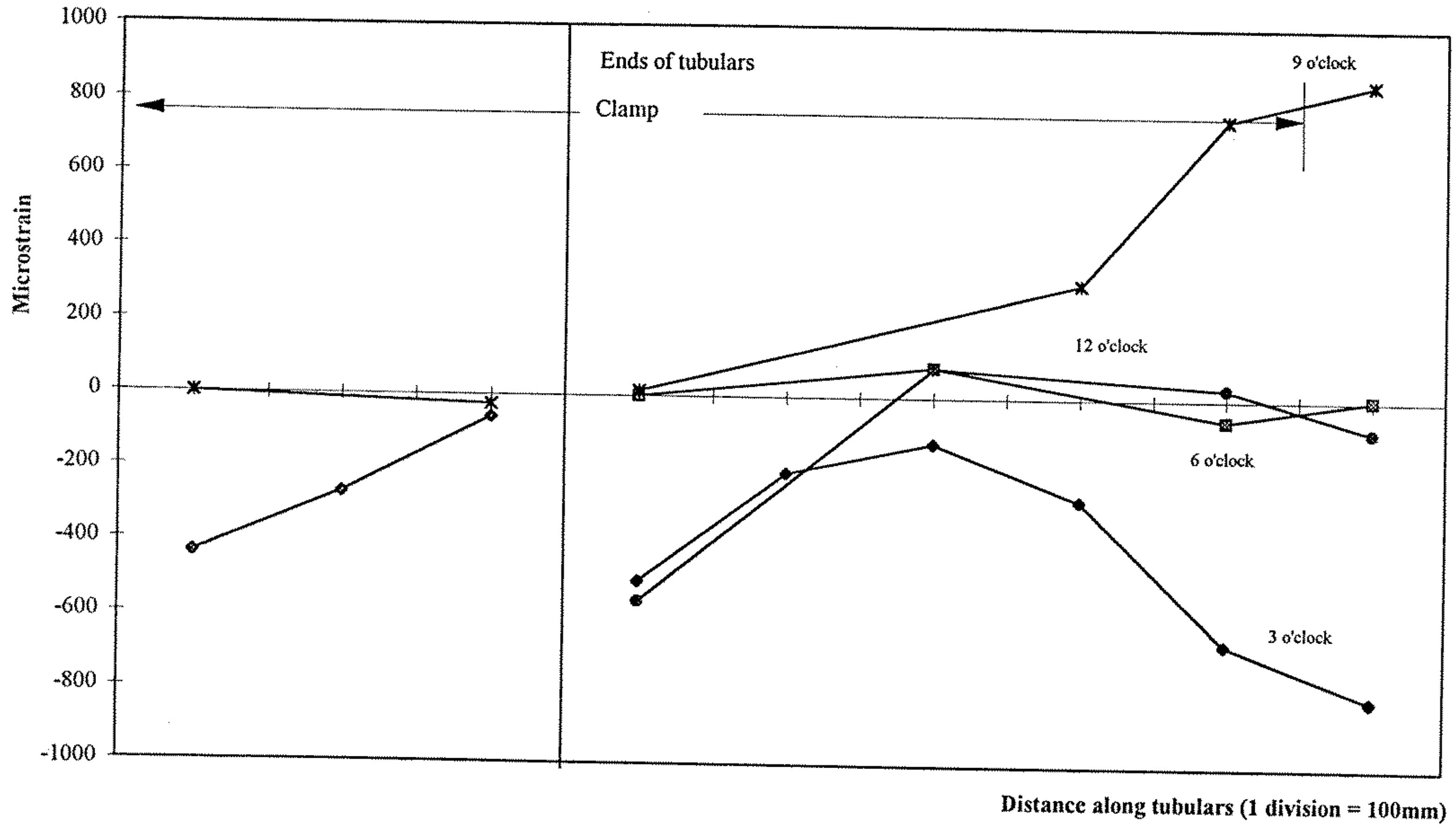


Fig 3.41 : Longitudinal strain in tubulars: Test 17

S M:\C167\M24\GROUT\T15-17.XLS\T17strain

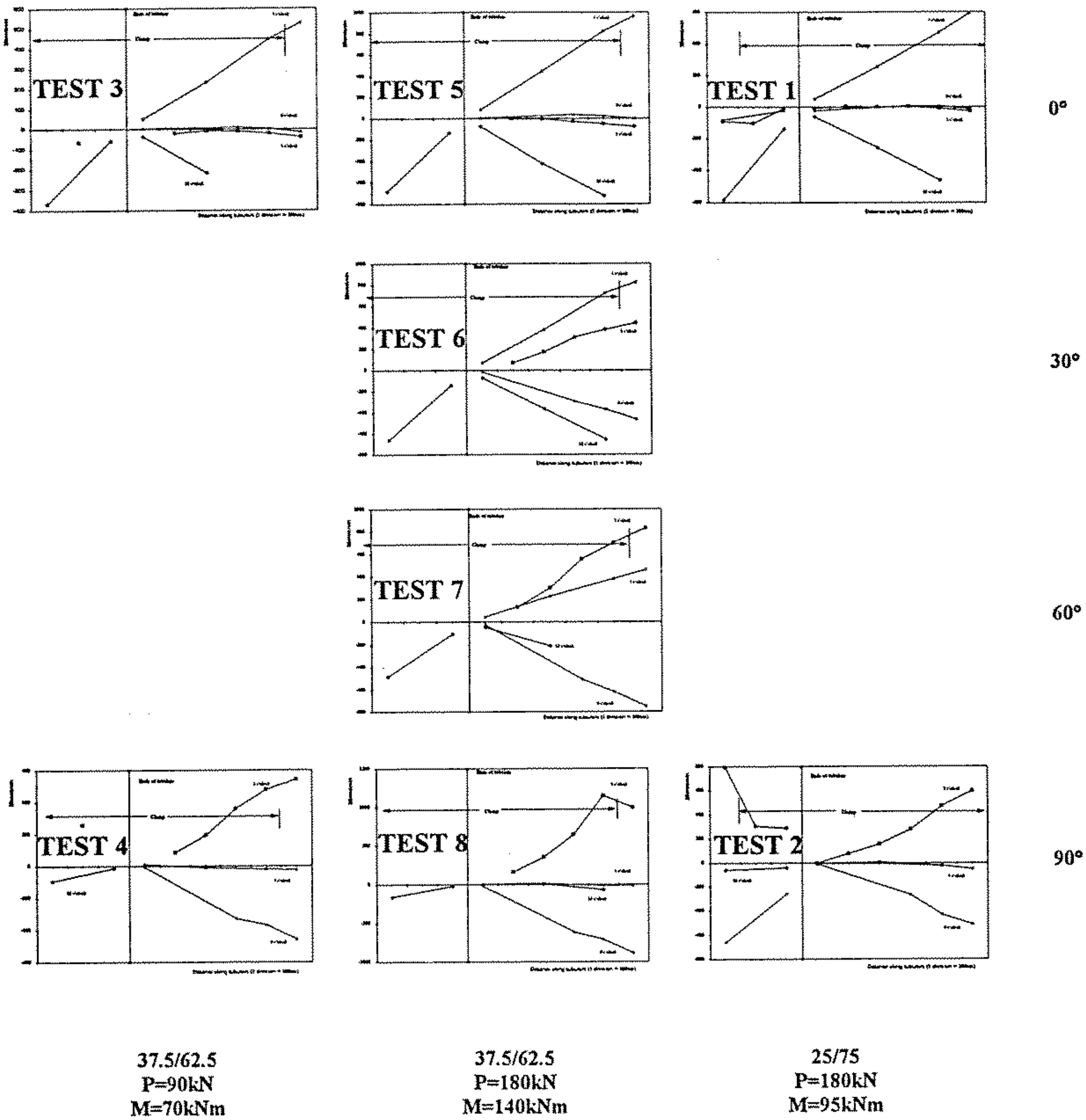


Fig 3.42 : Longitudinal strain distribution in tubulars

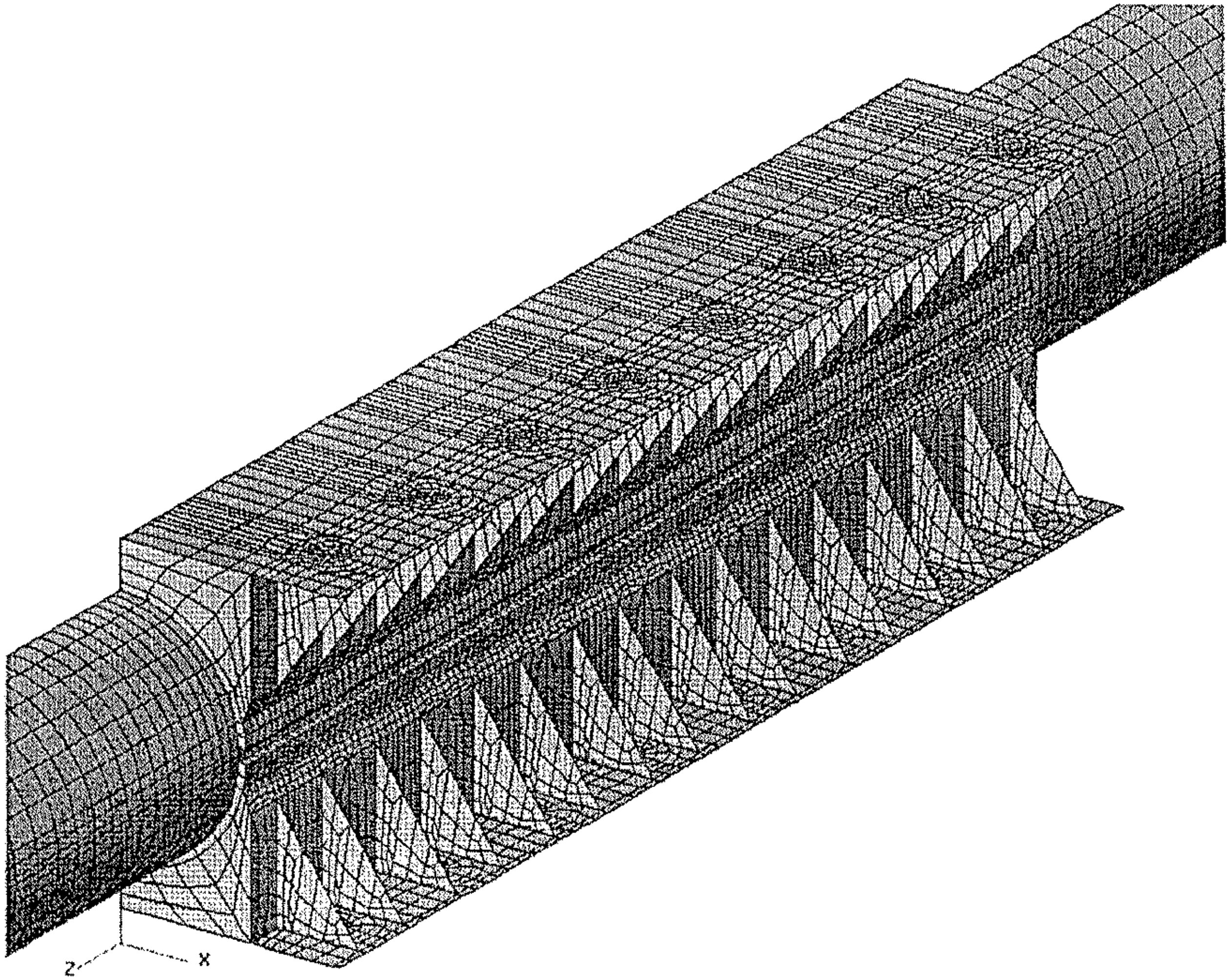


Fig. 4.1: FE Model used for in-plane bending (studbolts not shown)

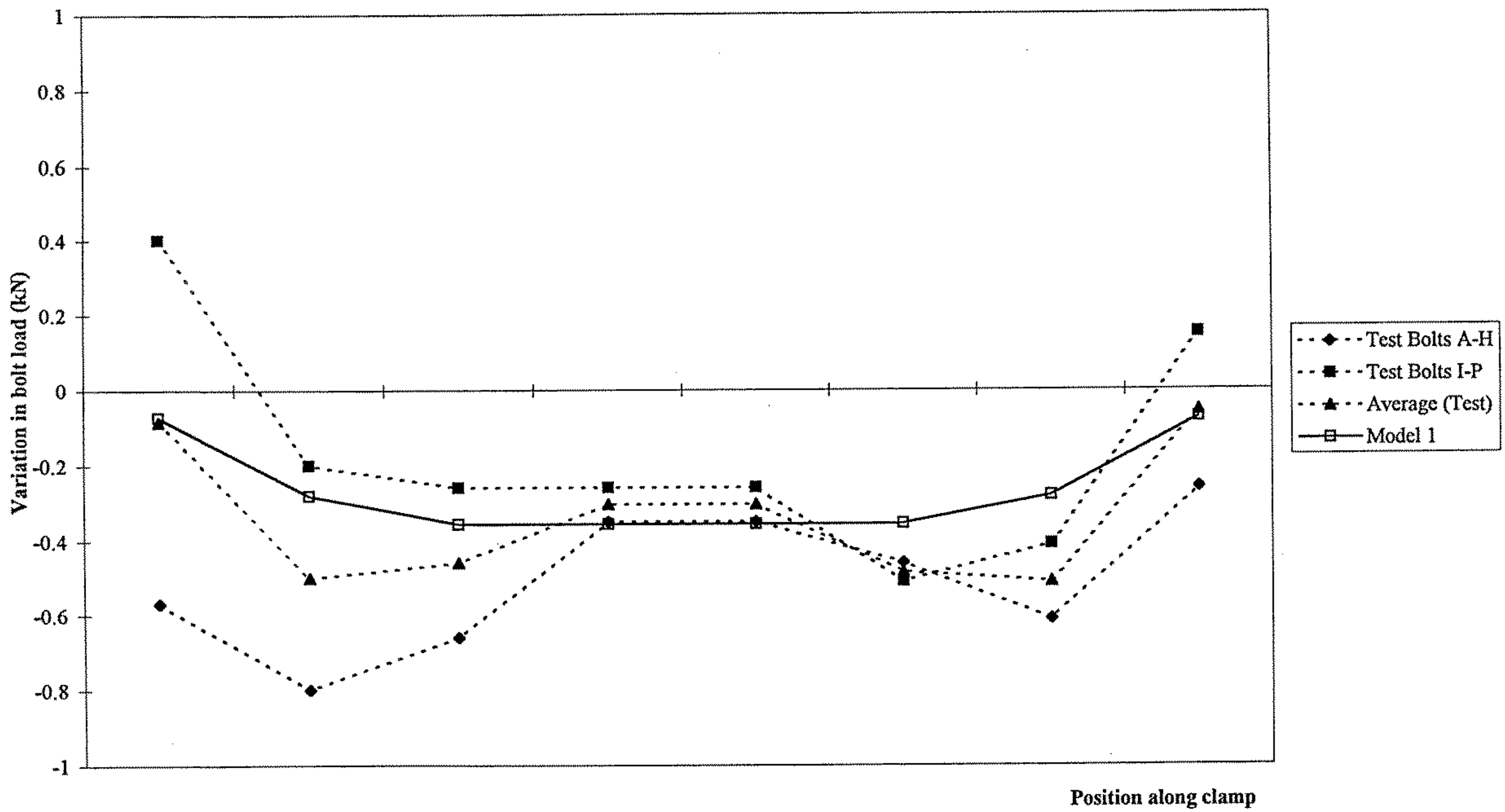


Fig. 4.2 : Comparison of FE Model 1 and Test 13: in-plane bending (0°)

S M:\C gasdata\DATA.XLS\Fig 4.2

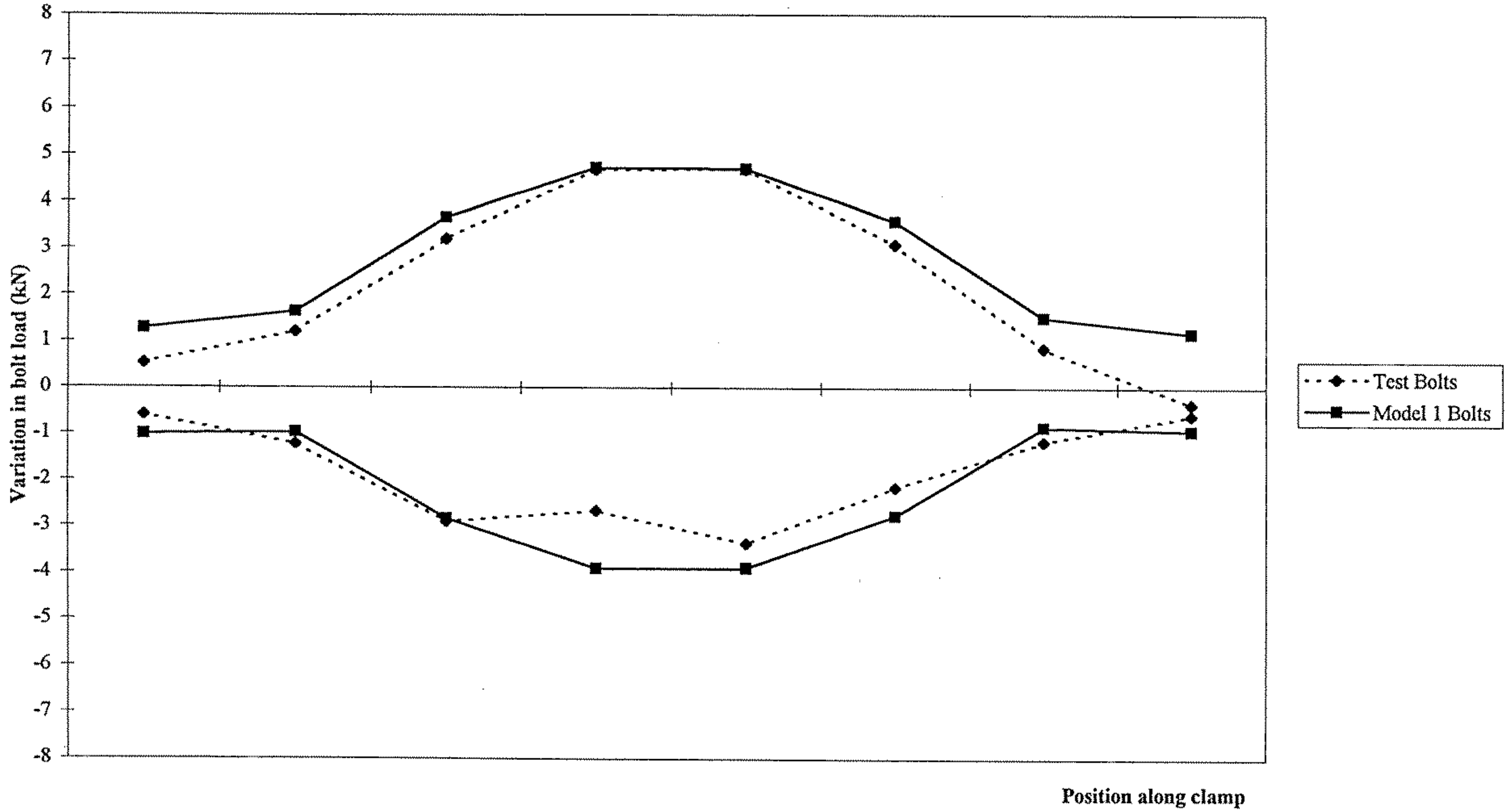


Fig. 4.3 : Comparison of FE Model 1 and Test 14: out-plane bending (90°)

S M:\C167\bgasdata\DATA.XLS\Fig 4.3

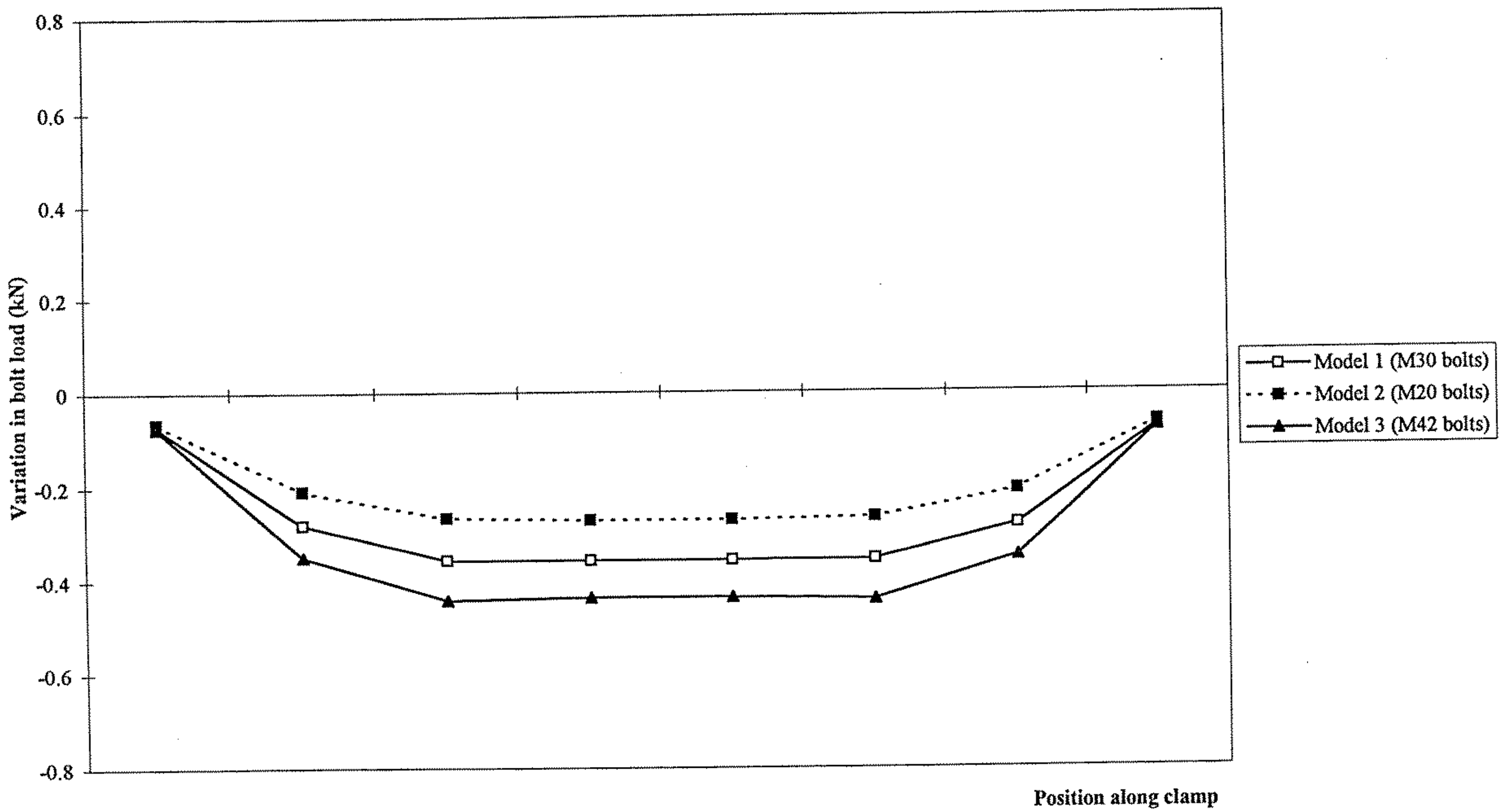


Fig. 4.4 : Effect of studbolt size on in-plane bending results

S M:\C gasdata\DATA.XLS\Fig 4.4

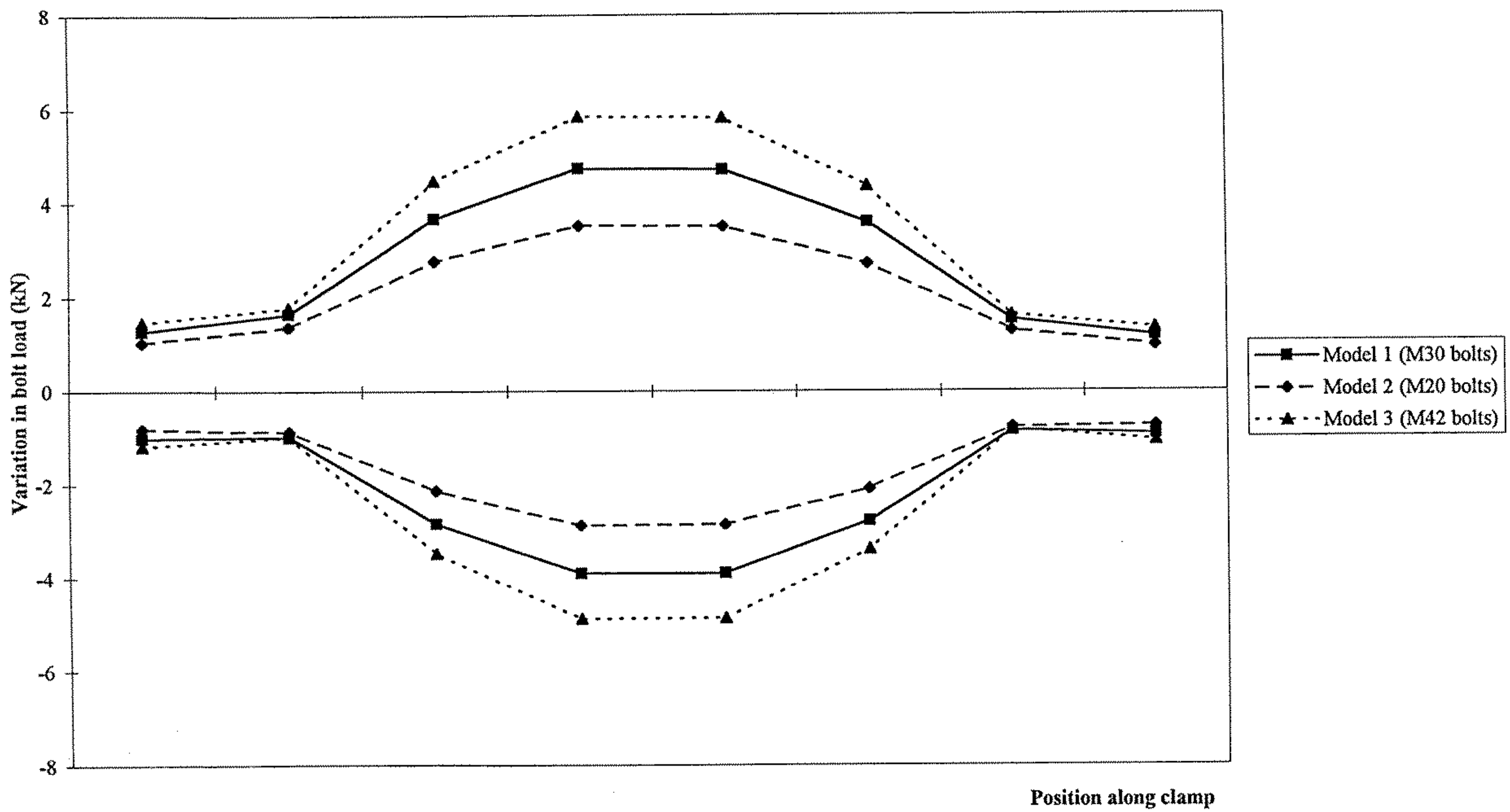


Fig. 4.5 : Effect of studbolt size on out-of-plane bending results

S M:\C167\bgasdata\DATA.XLS\Fig 4.5

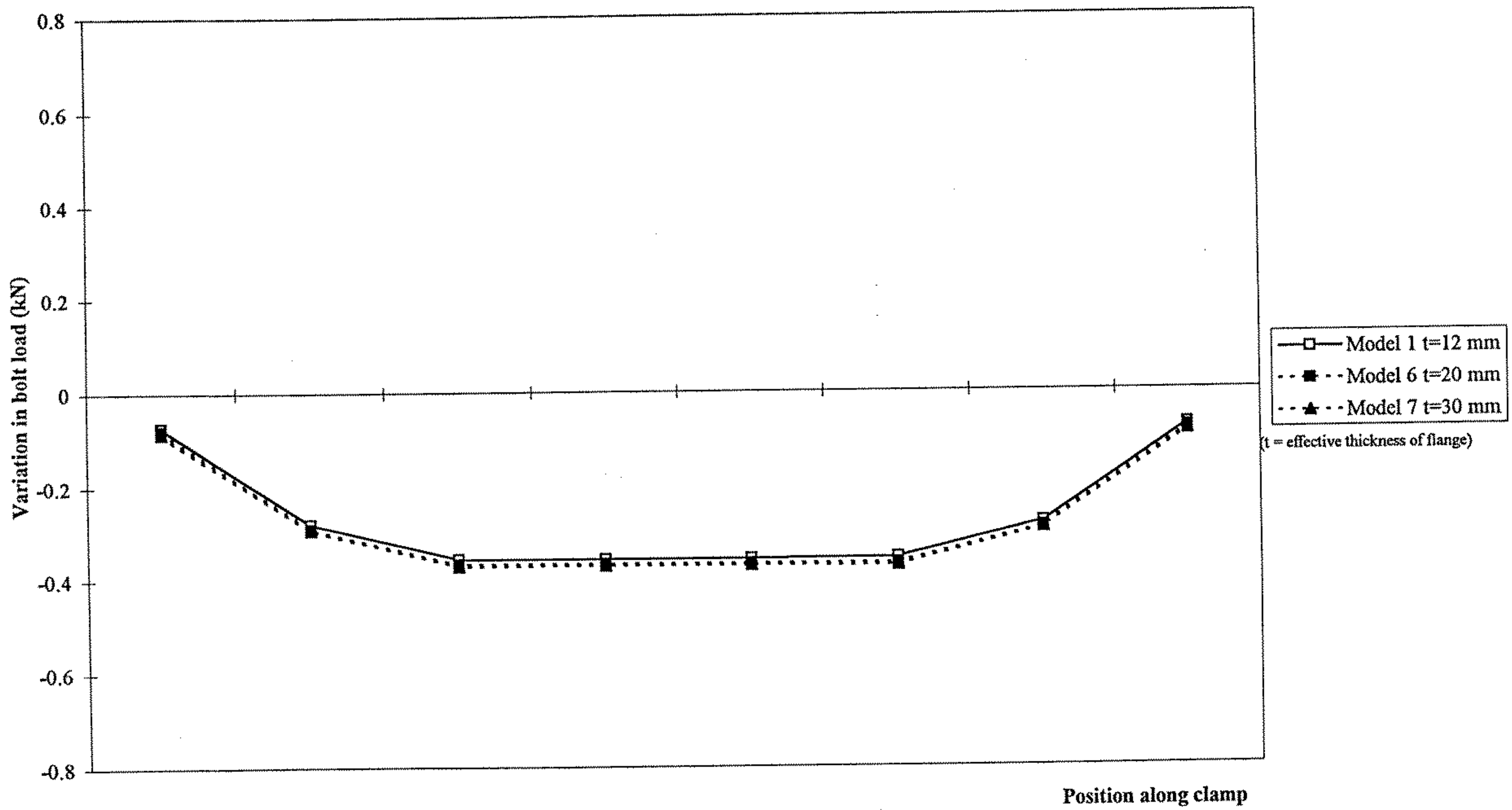


Fig. 4.6 : Effect of flange stiffness on in-plane bending results

S M:\C gasdata\DATA.XLS\Fig 4.6

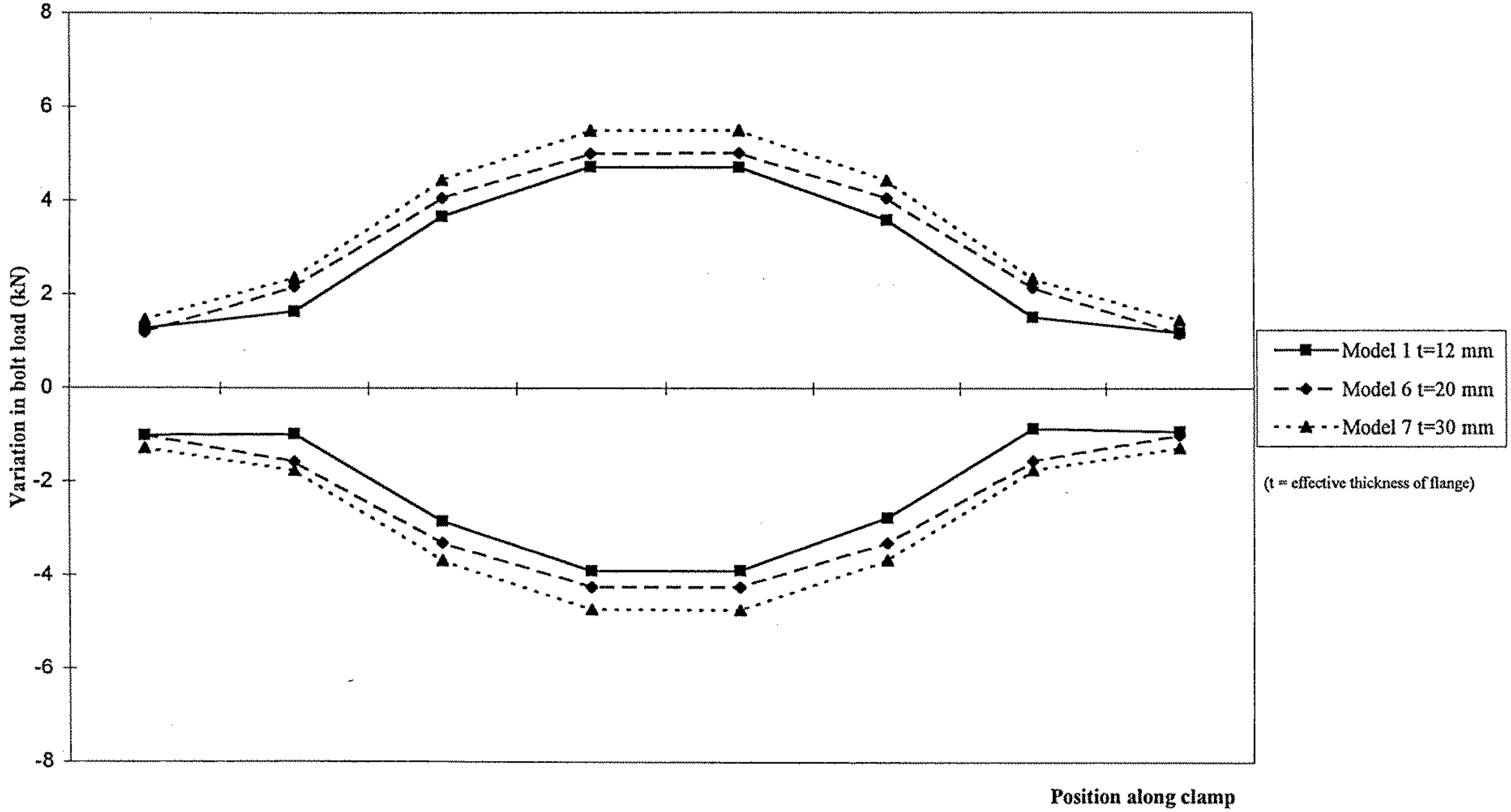


Fig. 4.7 : Effect of flange stiffness on out-of-plane bending results

S M:\C167\bgasdata\DATA.XLS\Fig 4.7

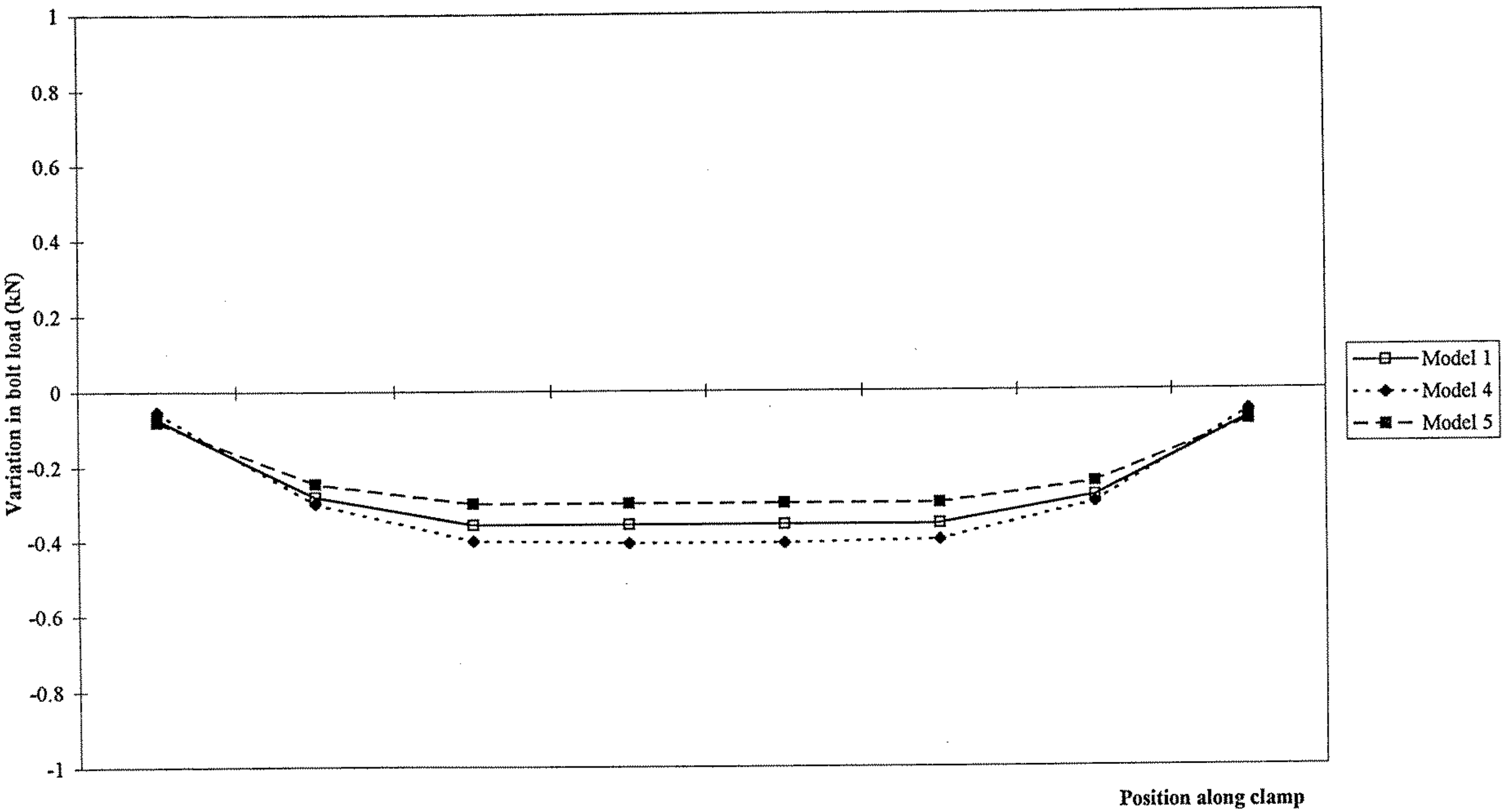


Fig. 4.8 : Comparison of FE Models 1, 4 & 5: in-plane bending (0°)

S M:\C gasdata\DATA.XLS\145ipb

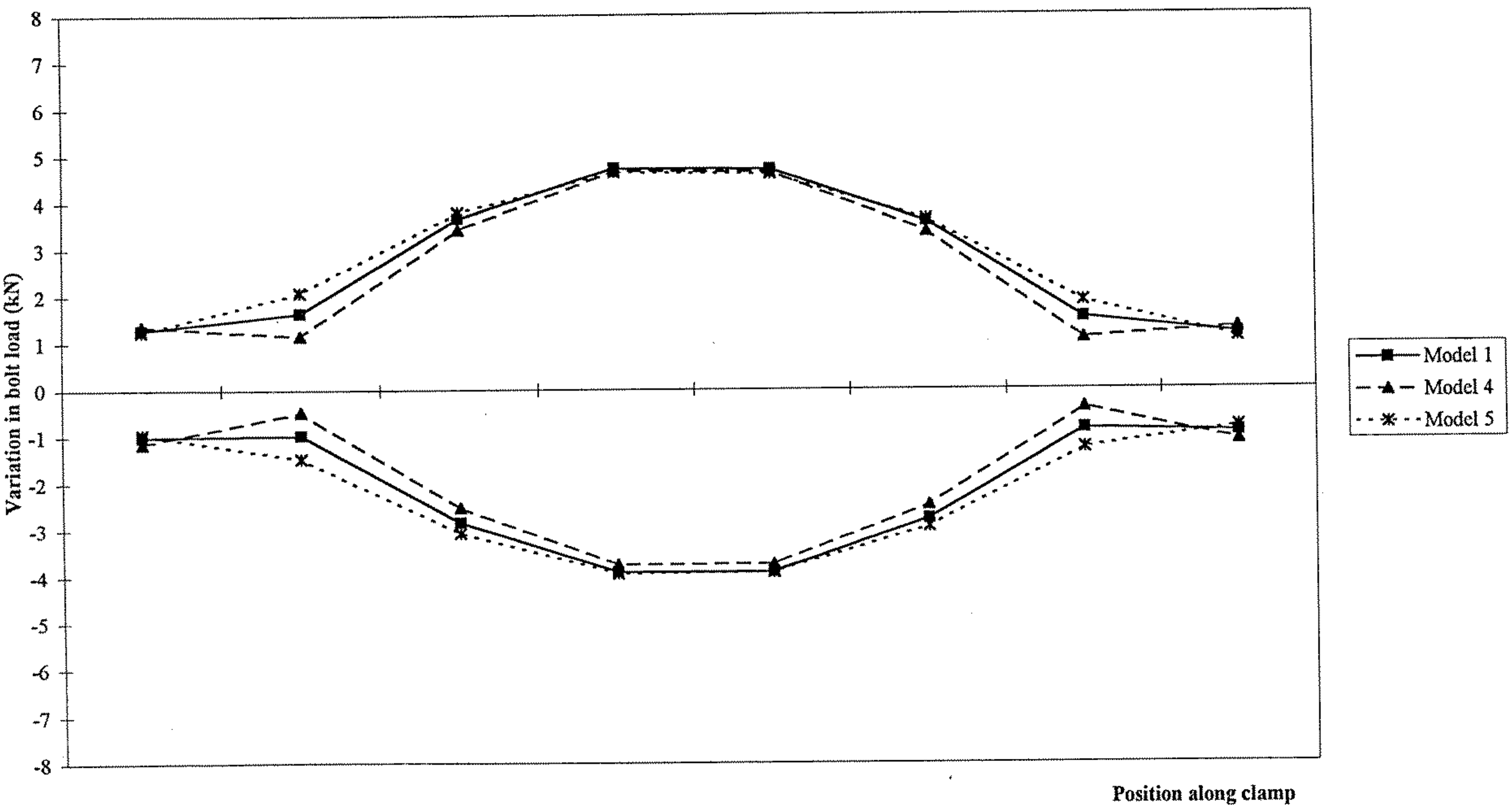


Fig. 4.9 : Comparison of FE Models 1, 4 & 5: out-of-plane bending (90°)

S M:\C167\bgasdata\DATA.XLS\145opb

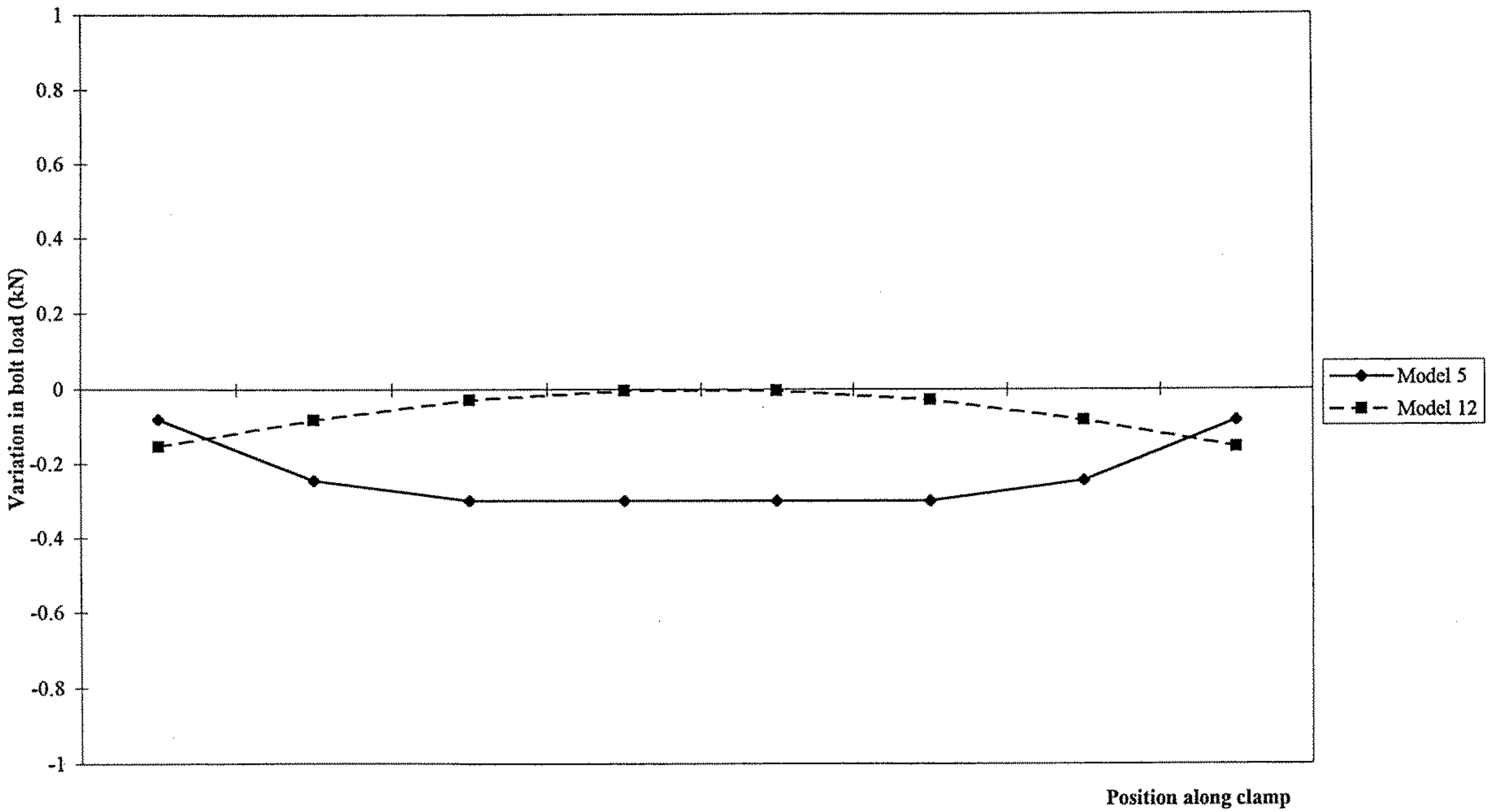


Fig. 4.10 : Comparison of FE Models 5 and 12: in-plane bending (0°)

S M:\C gasdata\DATA.XLS\5-12in

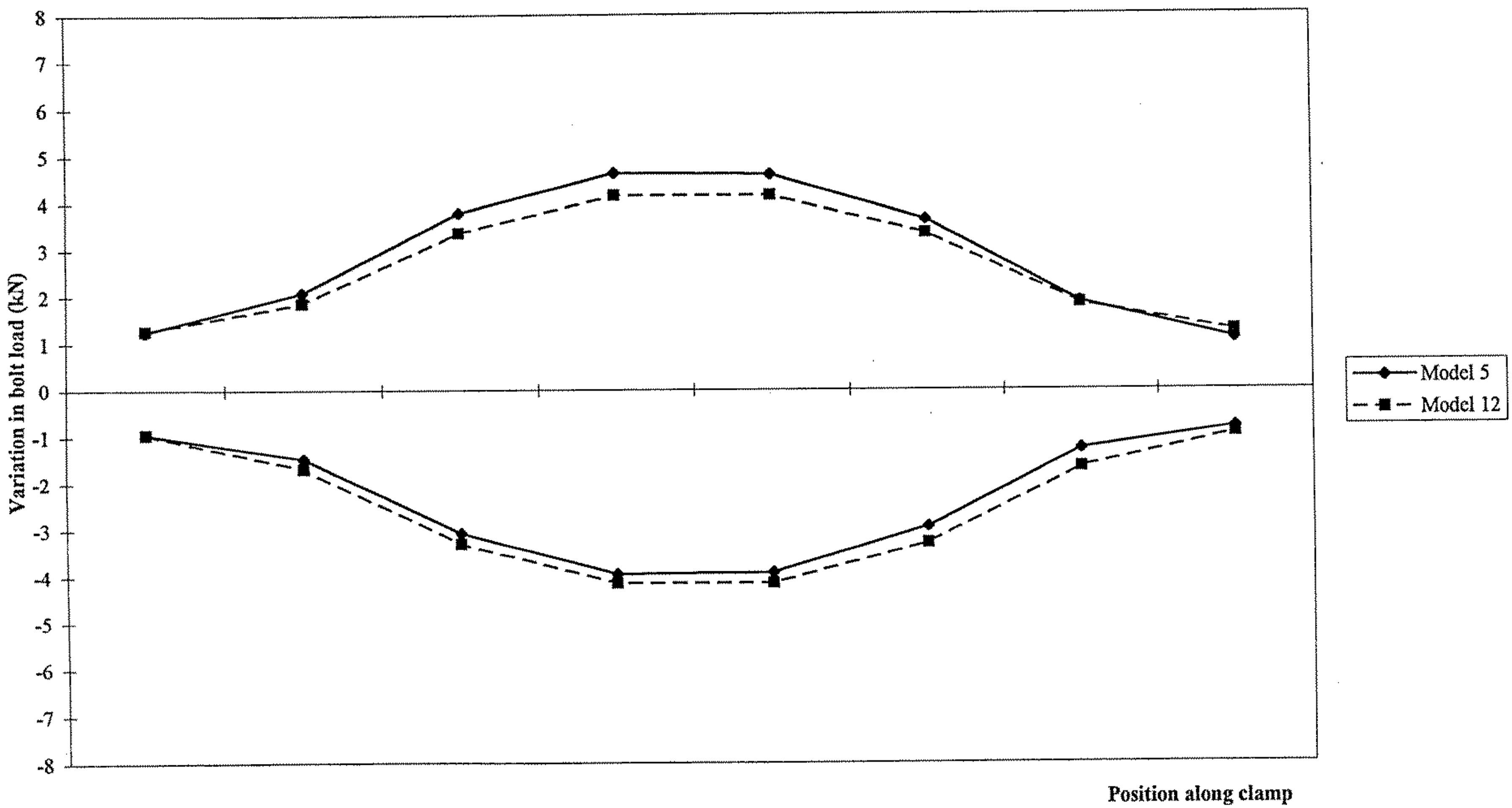


Fig. 4.11 : Comparison of FE Models 5 and 12: out-of-plane bending (90°)

S M:\C167\bgasdata\DATA.XLS\5-12out

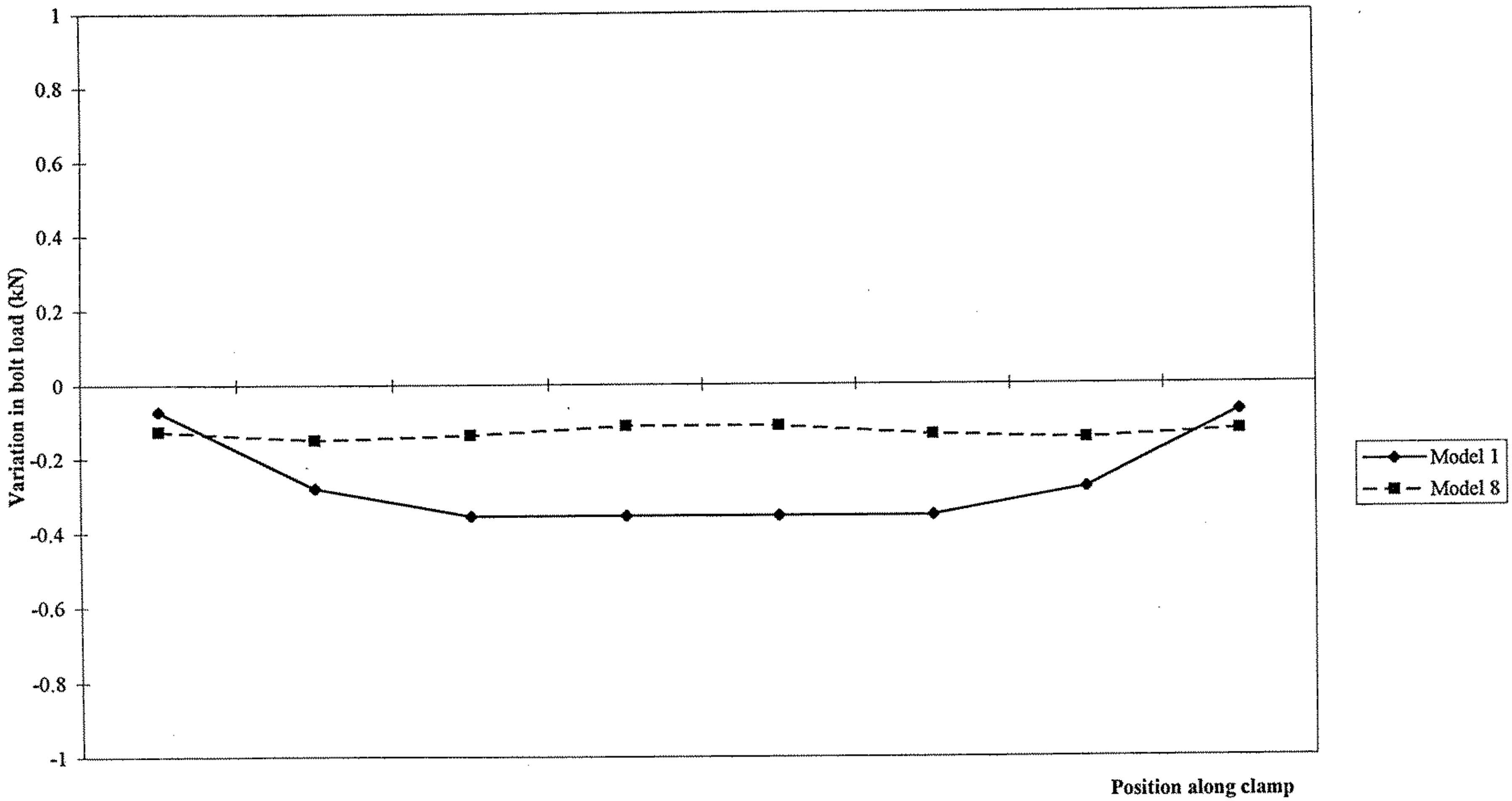


Fig. 4.12 : Comparison of FE Models 1 and 8: in-plane bending (0°)

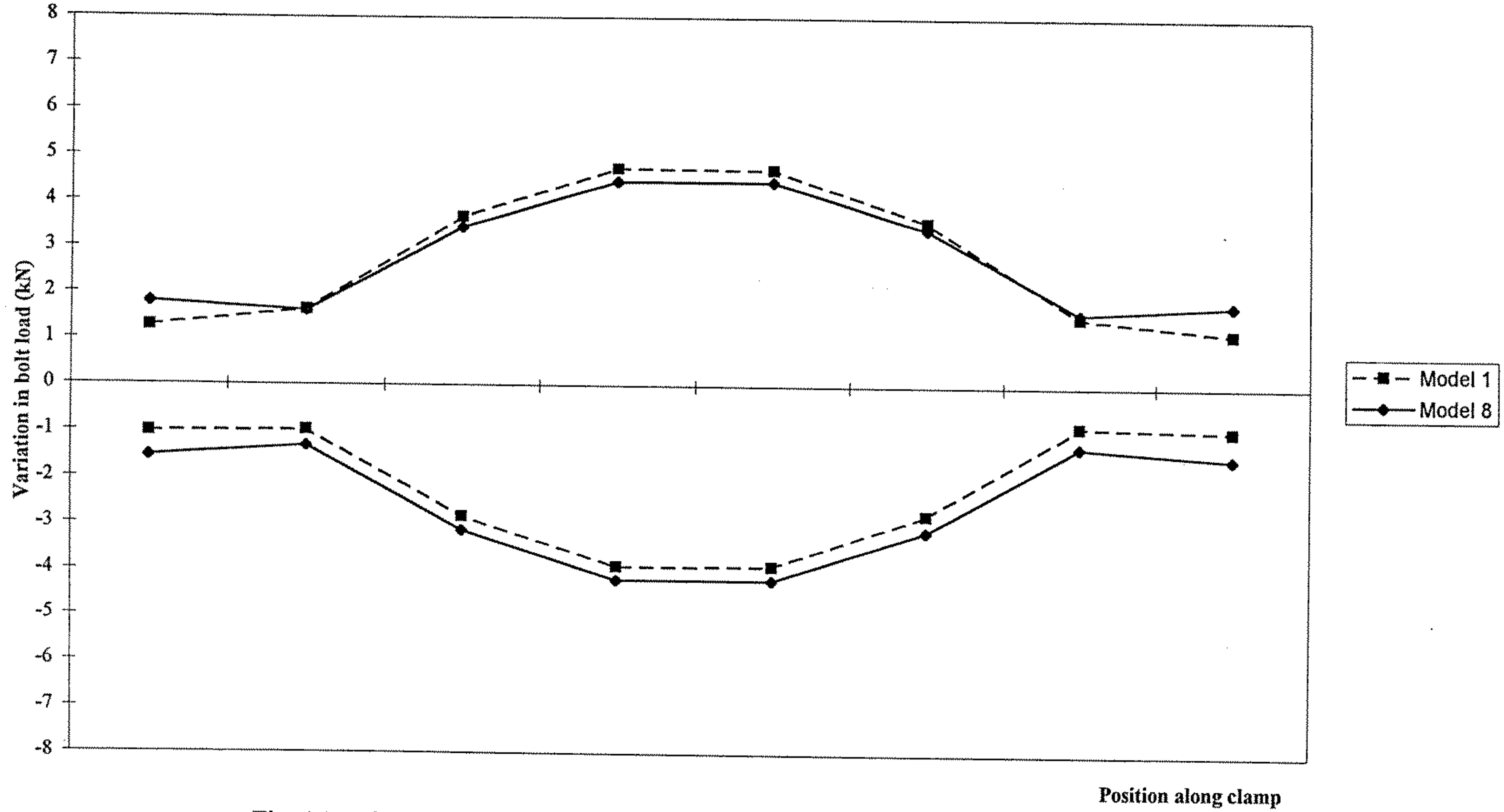


Fig. 4.13 : Comparison of FE Models 1 and 8: out-of-plane bending (90°)

S M:\C167\bgasdata\DATA.XLS\1-8out

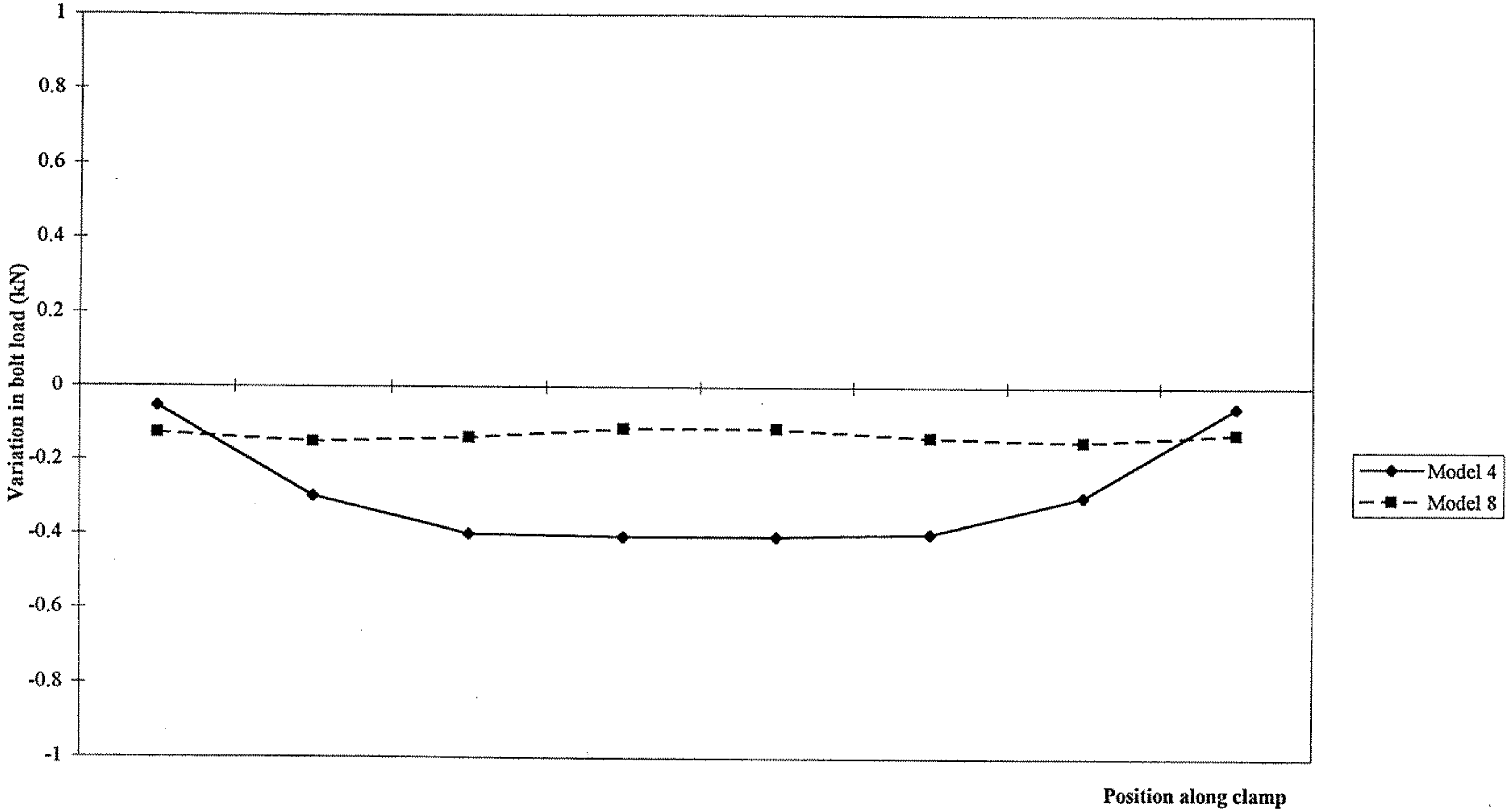


Fig. 4.14 : Comparison of FE Models 4 and 8: in-plane bending (0°)

S M:\C gasdata\DATA.XLS\4-8in

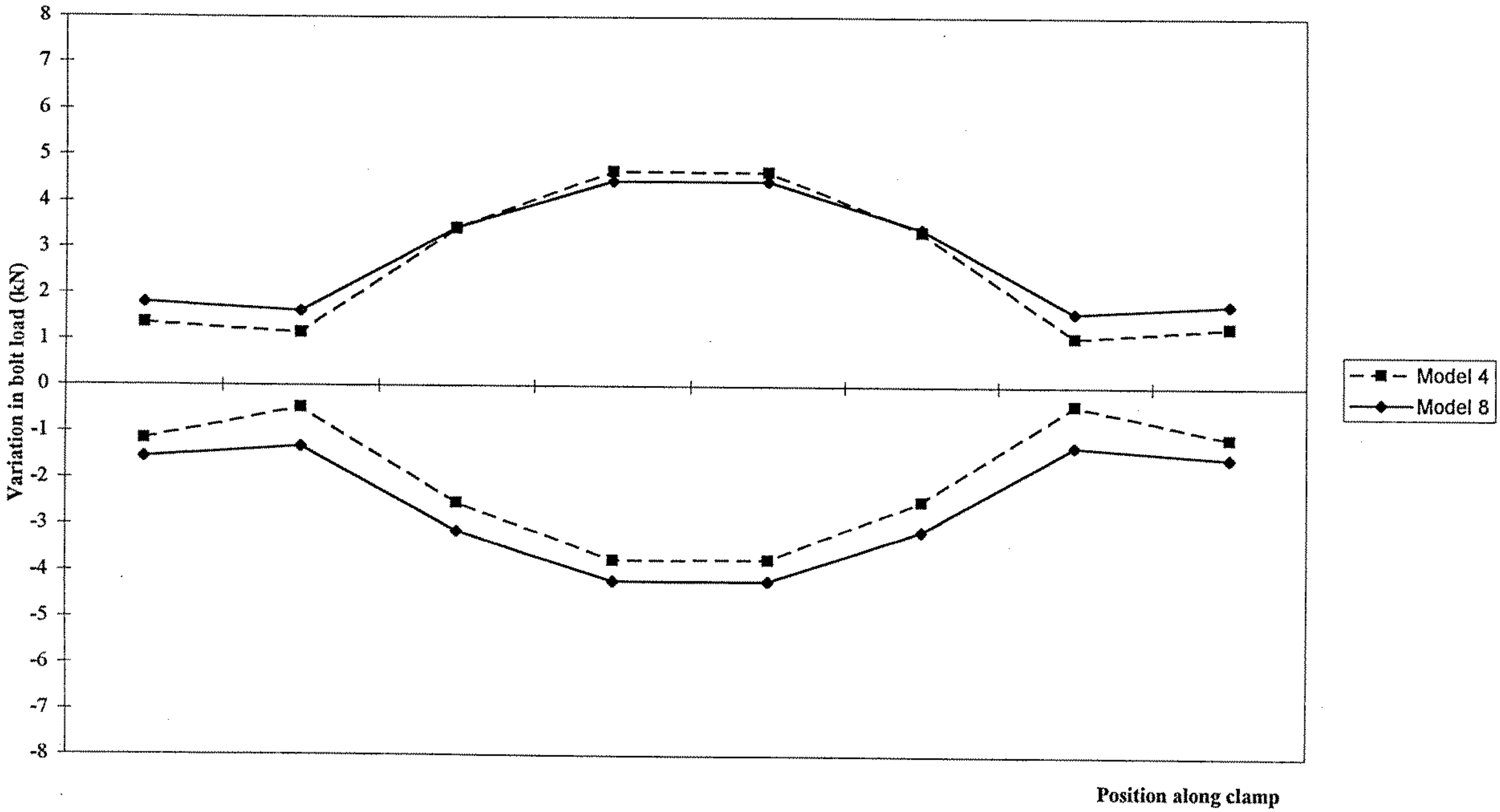


Fig. 4.15 : Comparison of FE Models 4 and 8: out-of-plane bending (90°)

S M:\C167\bgasdata\DATA.XLS\4-8out

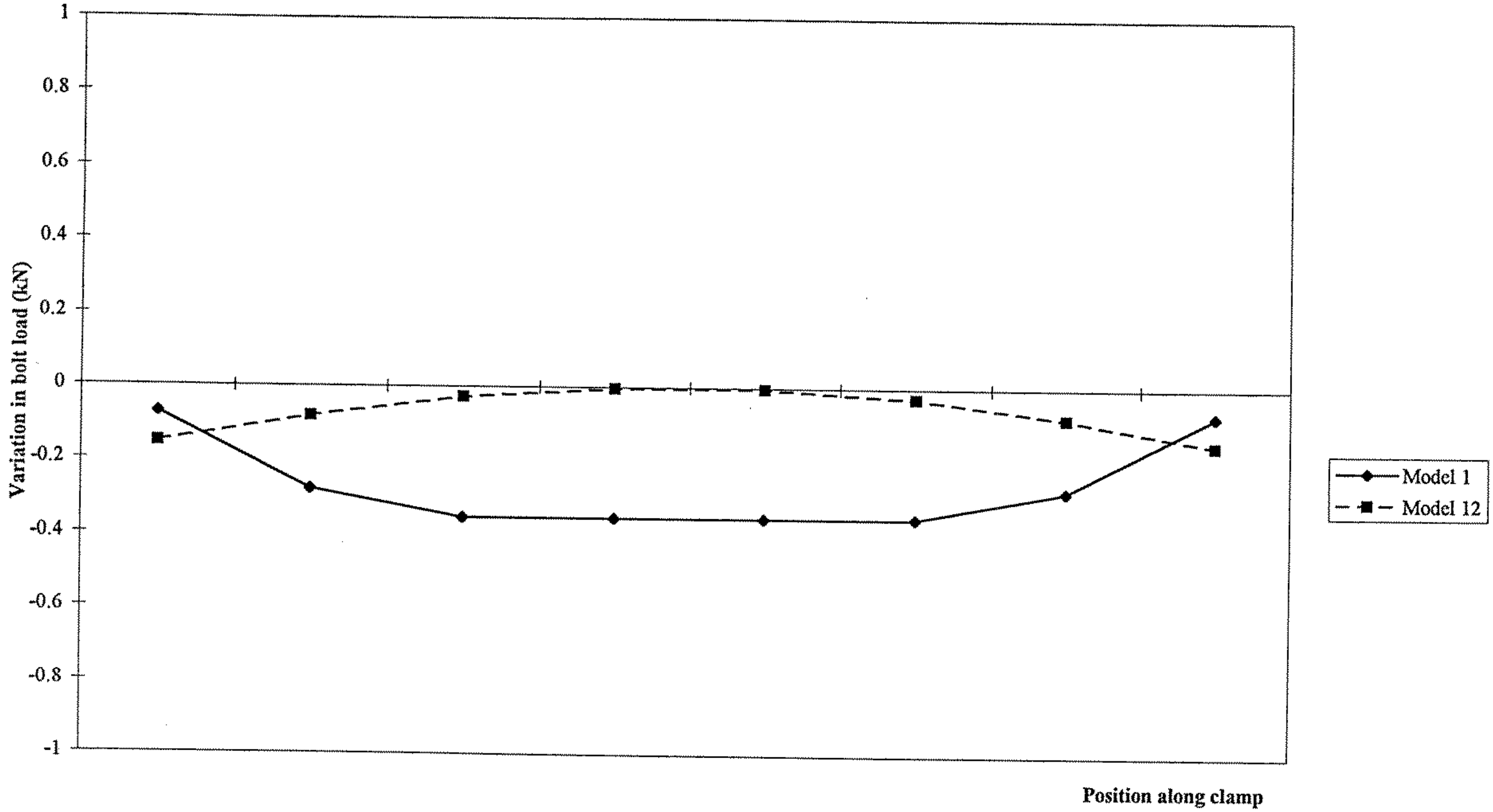


Fig. 4.16 : Comparison of FE Models 1 and 12: in-plane bending (0°)

S M:\C gasdata\DATA.XLS\1-12in

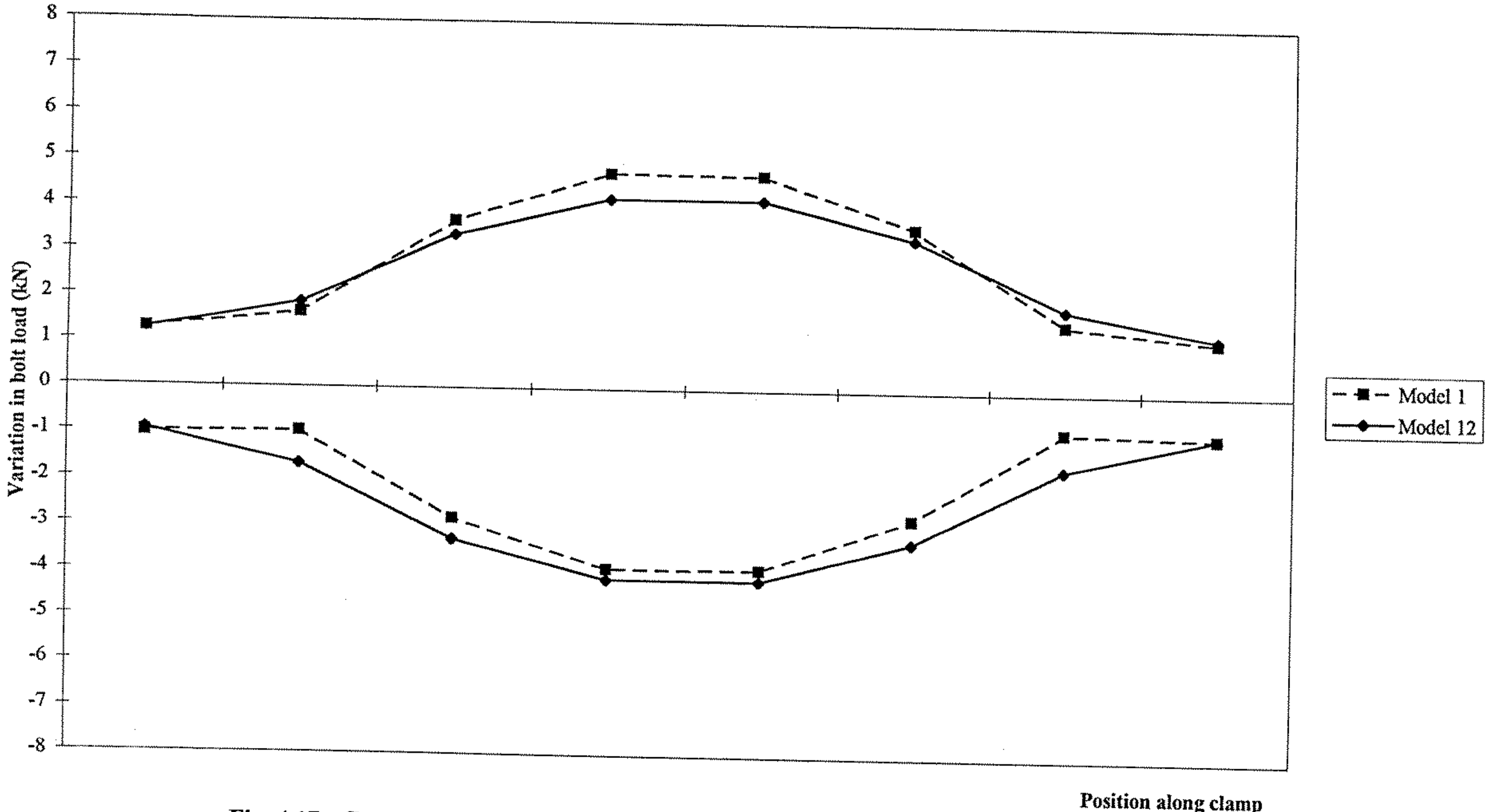


Fig. 4.17 : Comparison of FE Models 1 and 12: out-of-plane bending (90°)

S M:\C167\bgasdata\DATA.XLS\1-12out

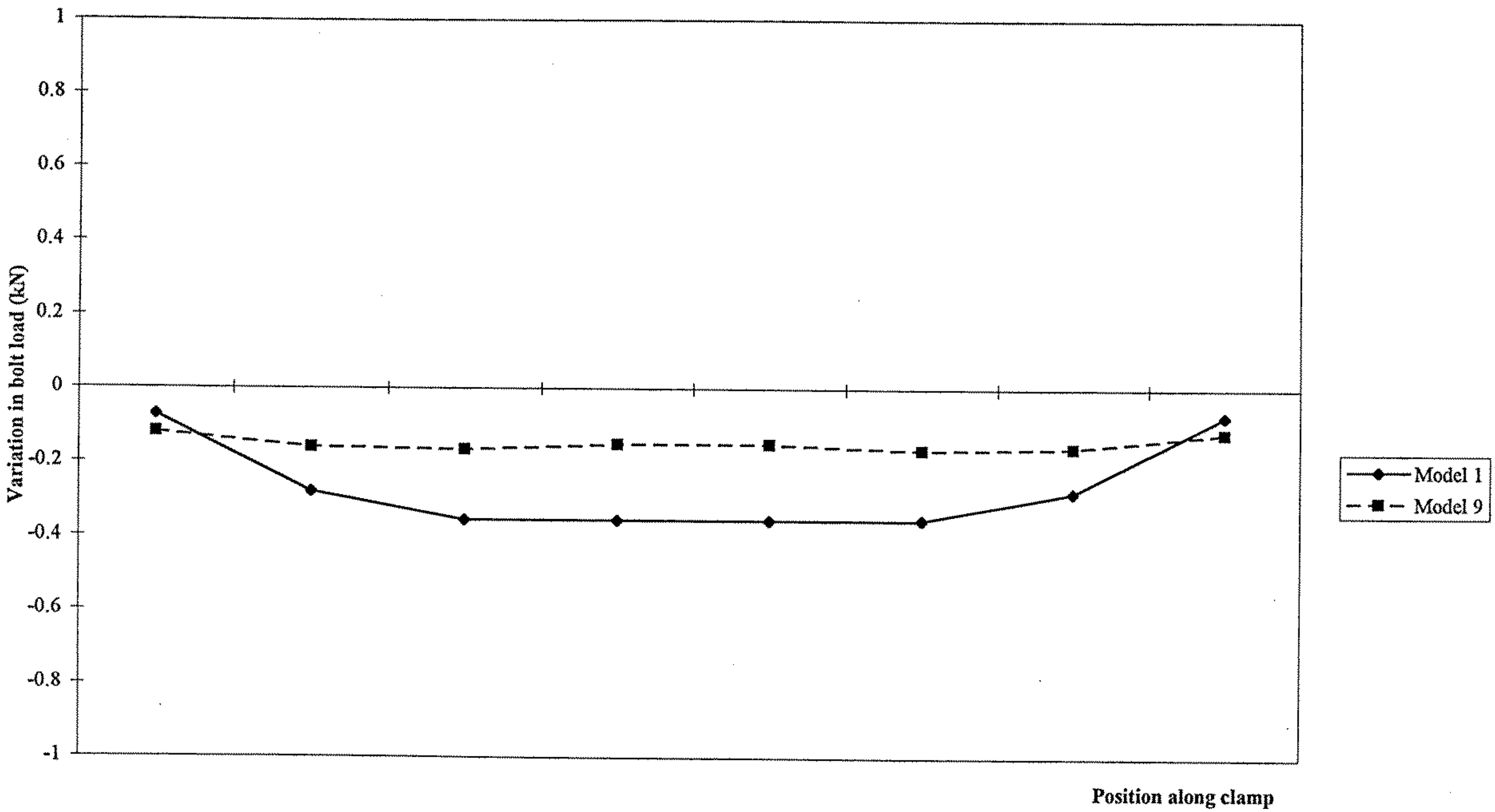


Fig. 4.18 : Comparison of FE Models 1 and 9: in-plane bending (0°)

S M:\C gasdata\DATA.XLS\1-9in

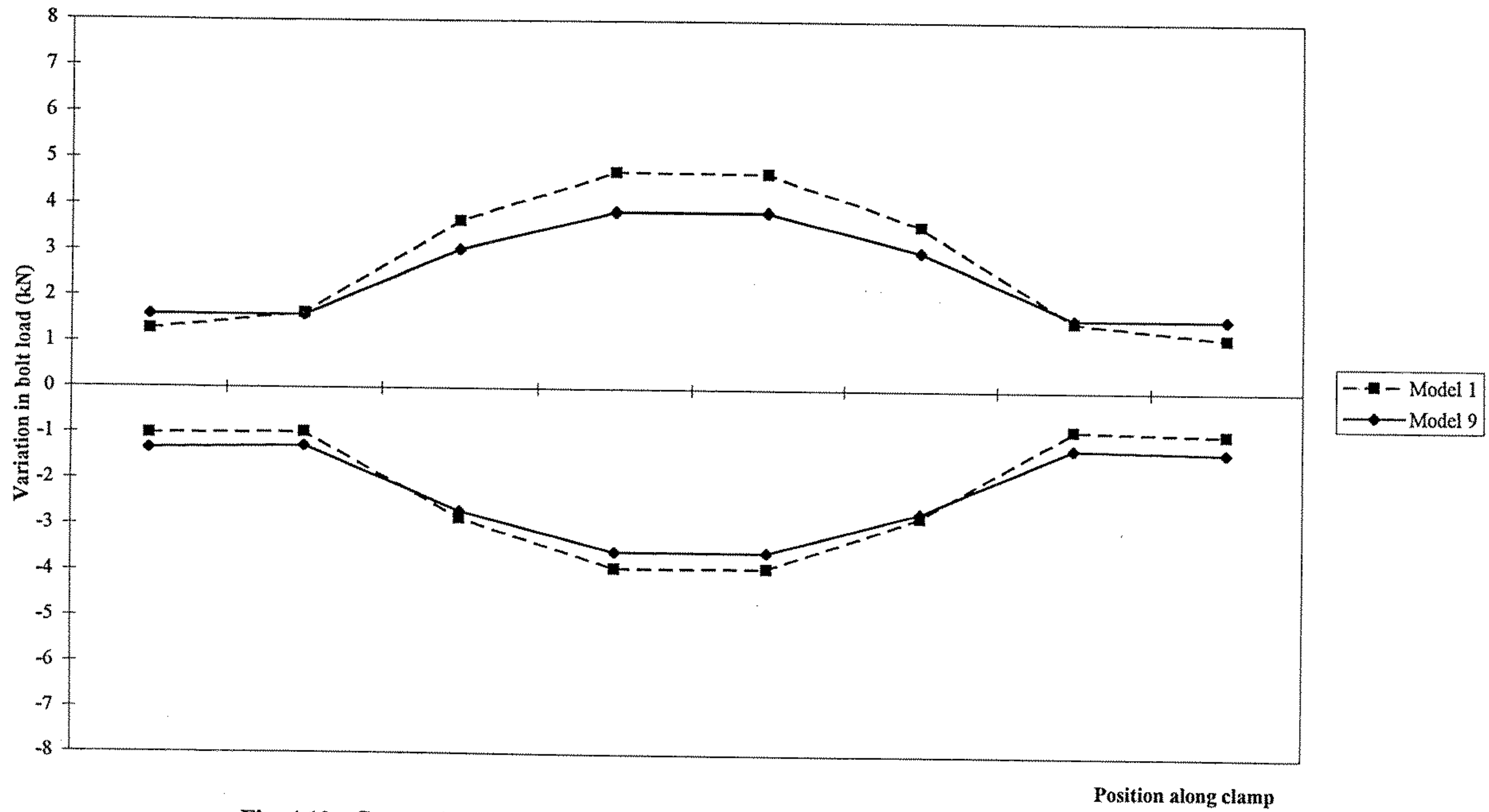


Fig. 4.19 : Comparison of FE Models 1 and 9: out-of-plane bending (90°)

S M:\C167\bgasdata\DATA.XLS\1-9out

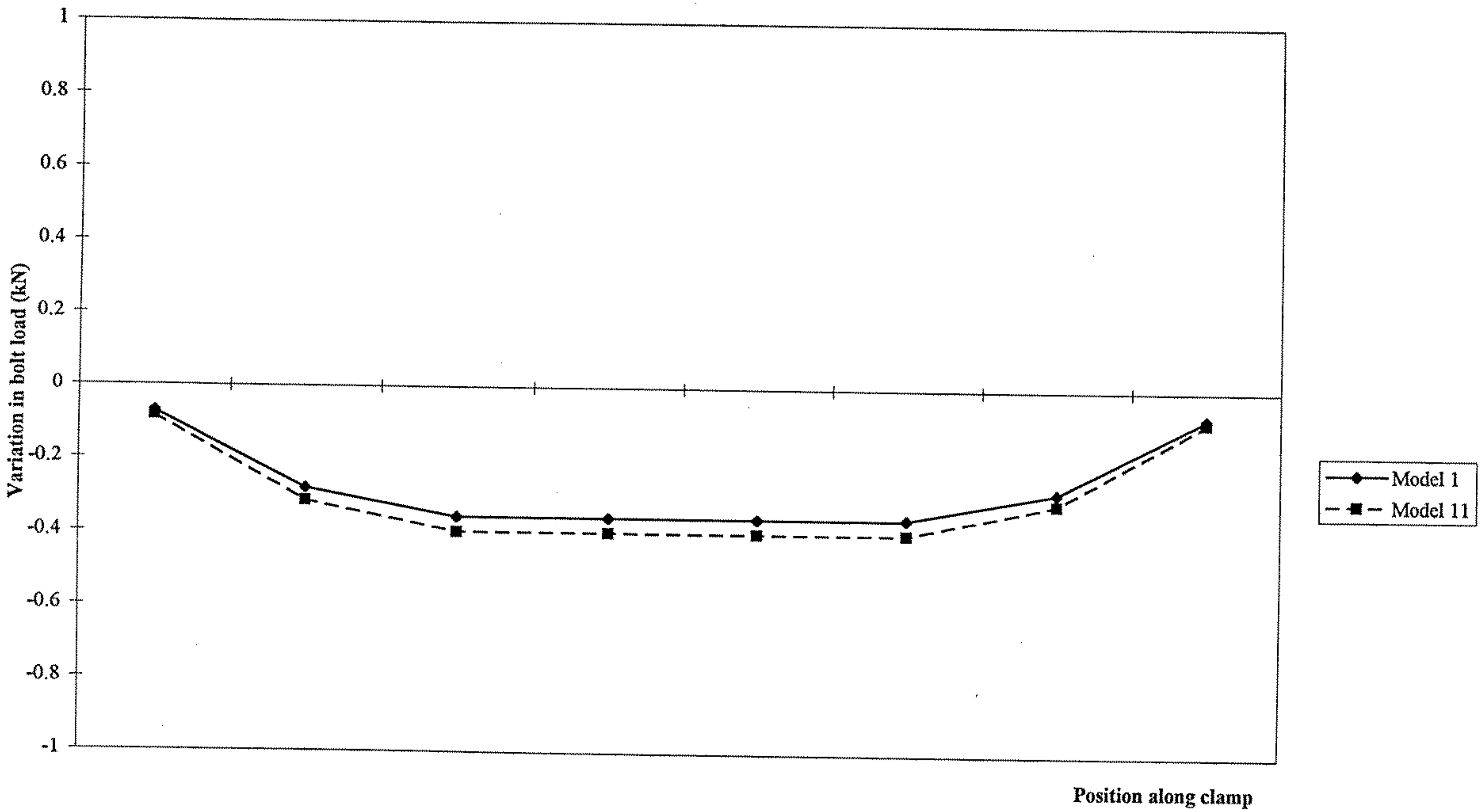


Fig. 4.20 : Comparison of FE Models 1 and 11: in-plane bending (0°)

S M:\C gasdata\DATA.XLS\1-11in

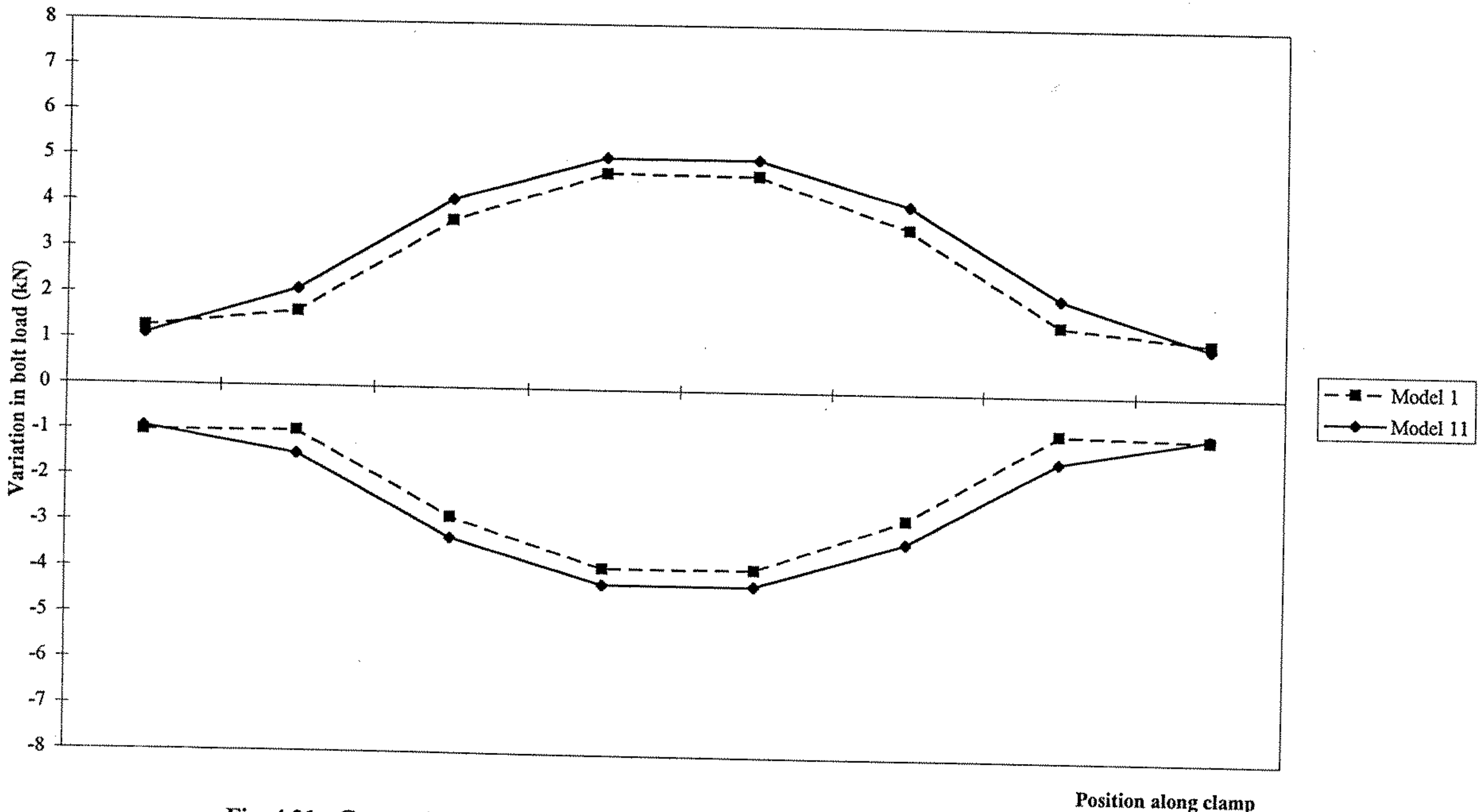


Fig. 4.21 : Comparison of FE Models 1 and 11: out-of-plane bending (90°)

S M:\C167\bgasdata\DATA.XLS\1-11out

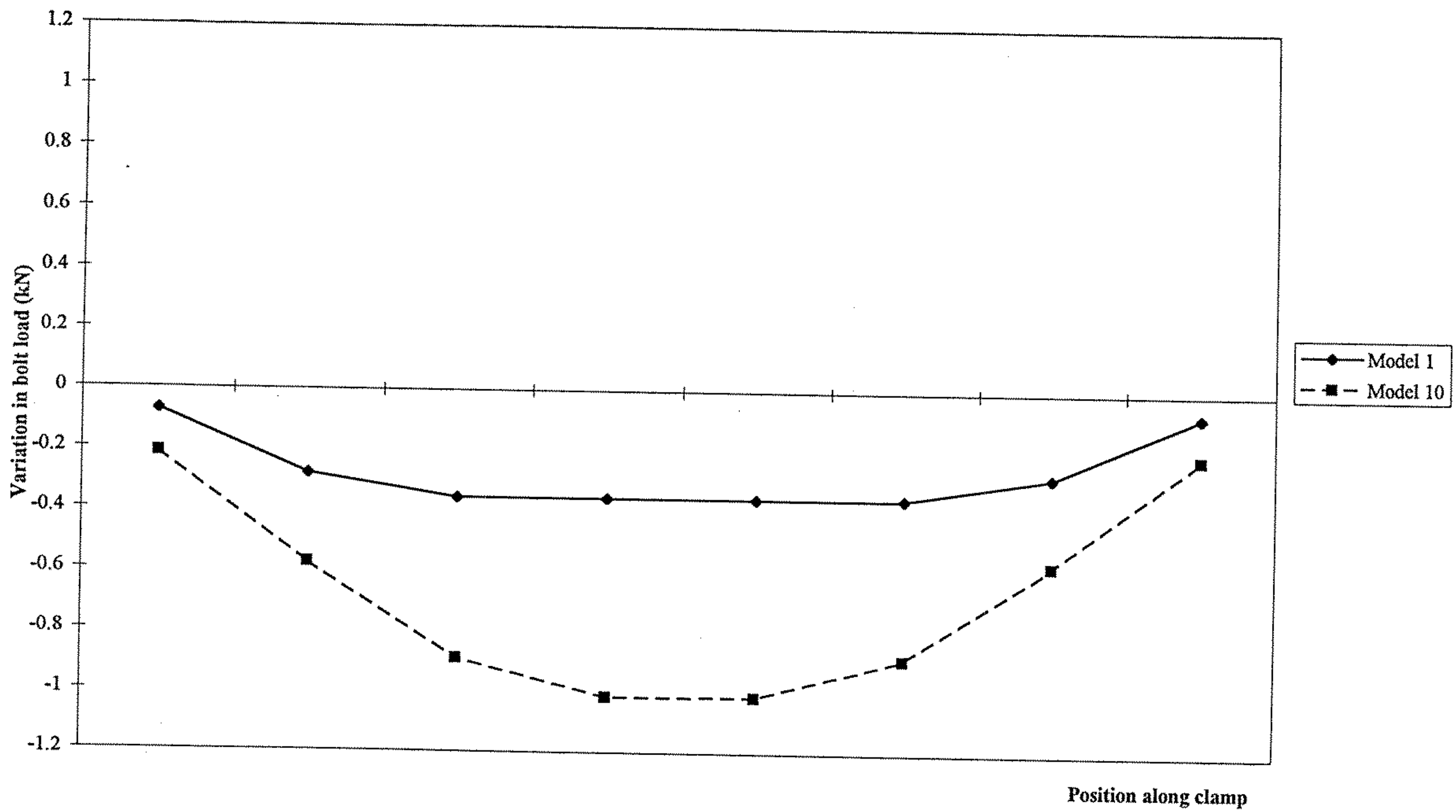


Fig. 4.22 : Comparison of FE Models 1 and 10: in-plane bending (0°)

S M:\C gasdata\DATA.XLS\1-10in

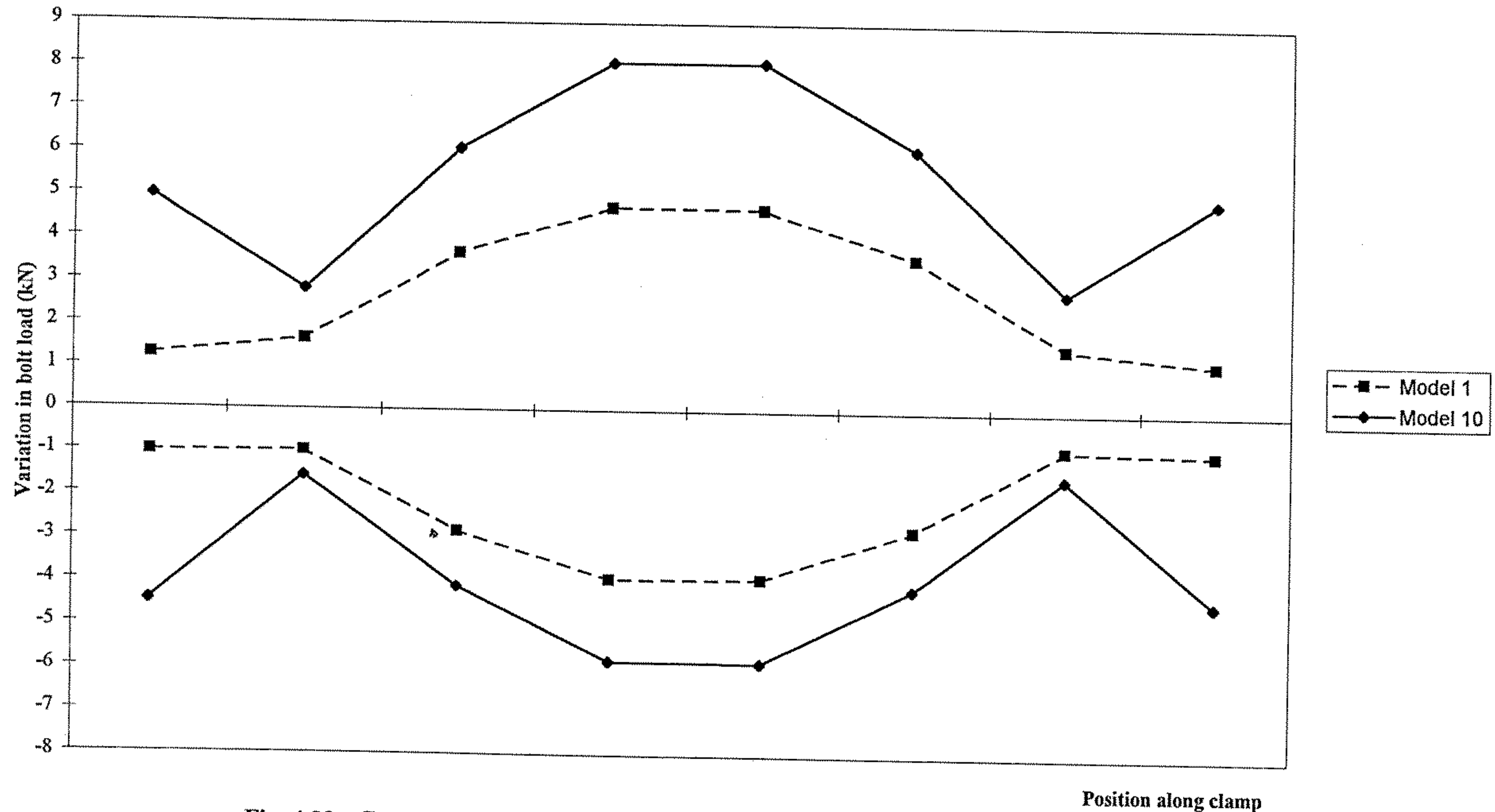


Fig. 4.23 : Comparison of FE Models 1 and 10: out-of-plane bending (90°)

S M:\C167\bgasdata\DATA.XLS\1-10out

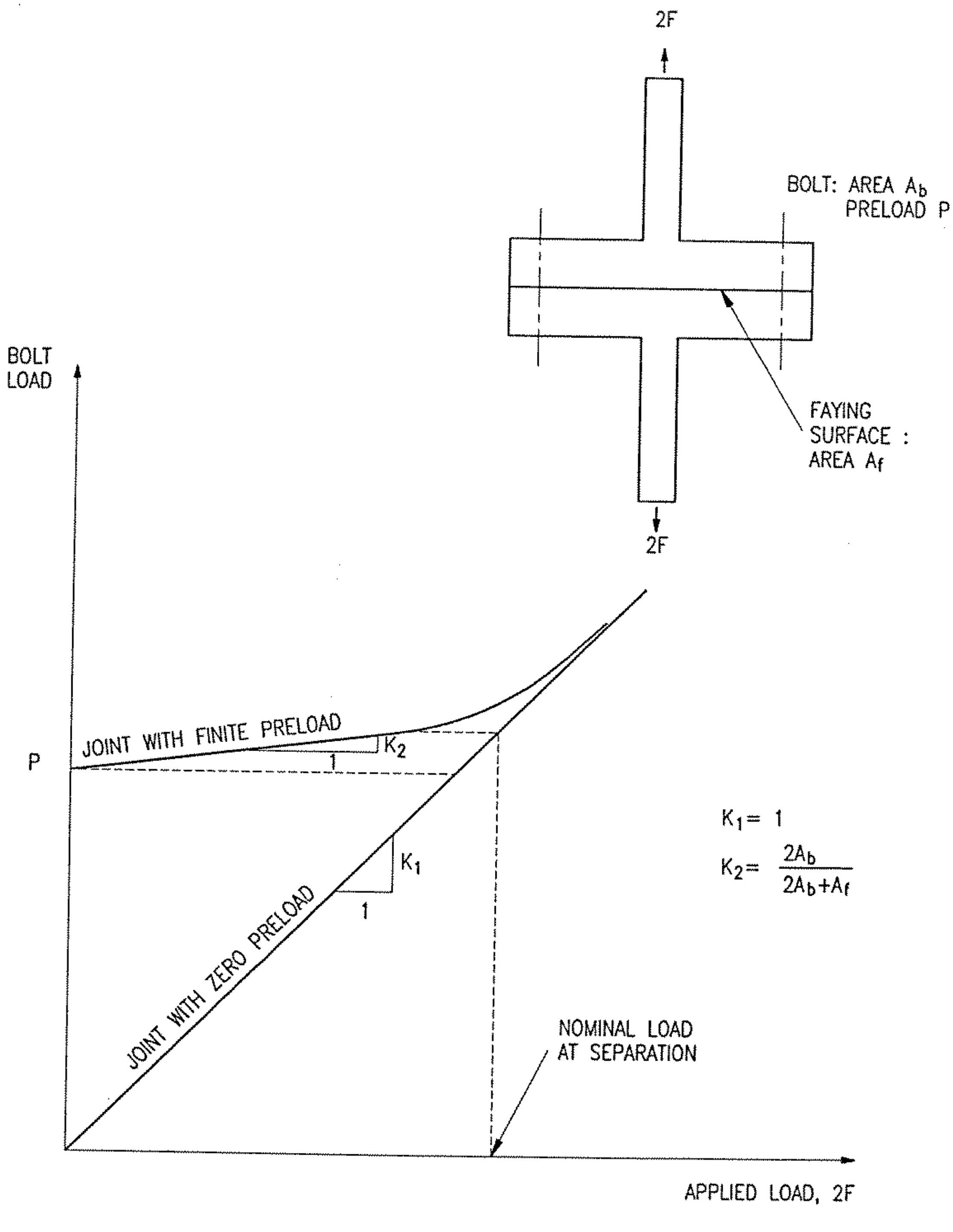
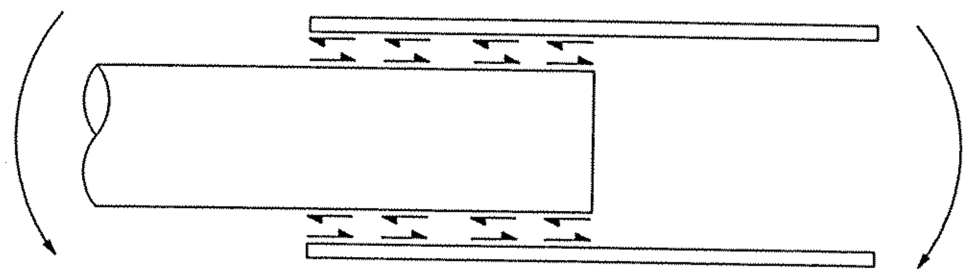
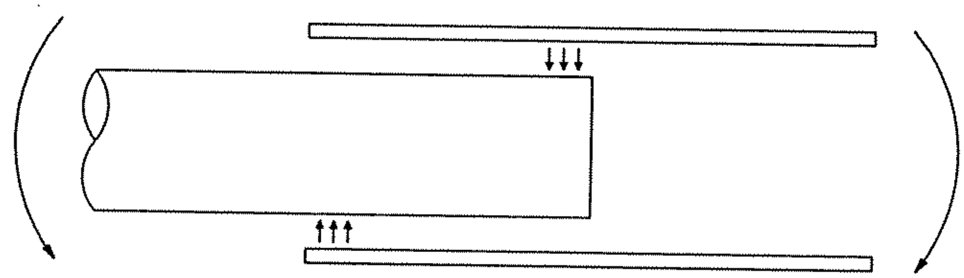


Fig. 5.1: Load behaviour of two T-pieces bolted together

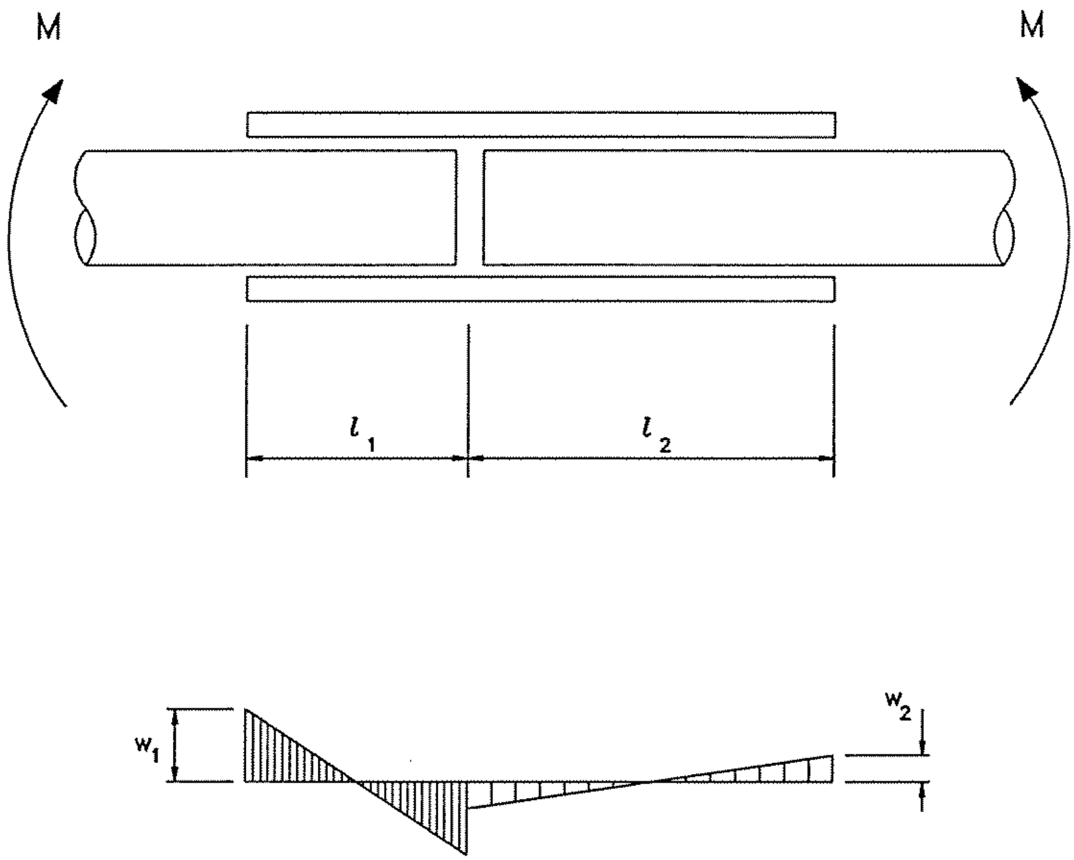


(a) TRANSFER BY INTERFACE SHEAR



(b) TRANSFER BY BEARING FORCES

Fig. 5.2: Moment transfer mechanisms



$$w_1 = \frac{6M}{l_1^{\,2}} \qquad\qquad w_2 = \frac{6M}{l_2^{\,2}}$$

Fig. 5.3: An assumed internal load distribution (Model 1)

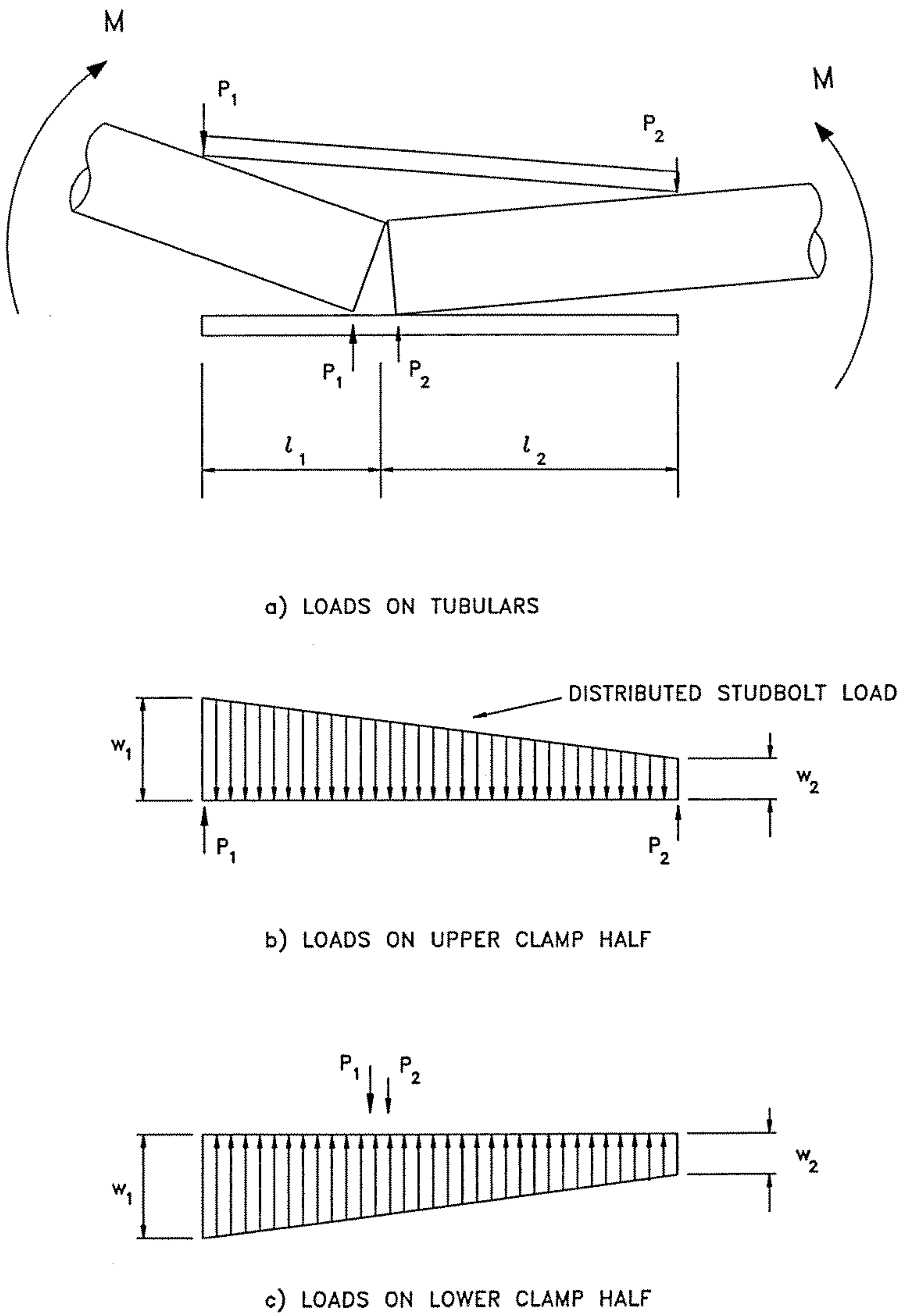


Fig. 5.4: Internal point load assuming rigid clamp (Model 2A)

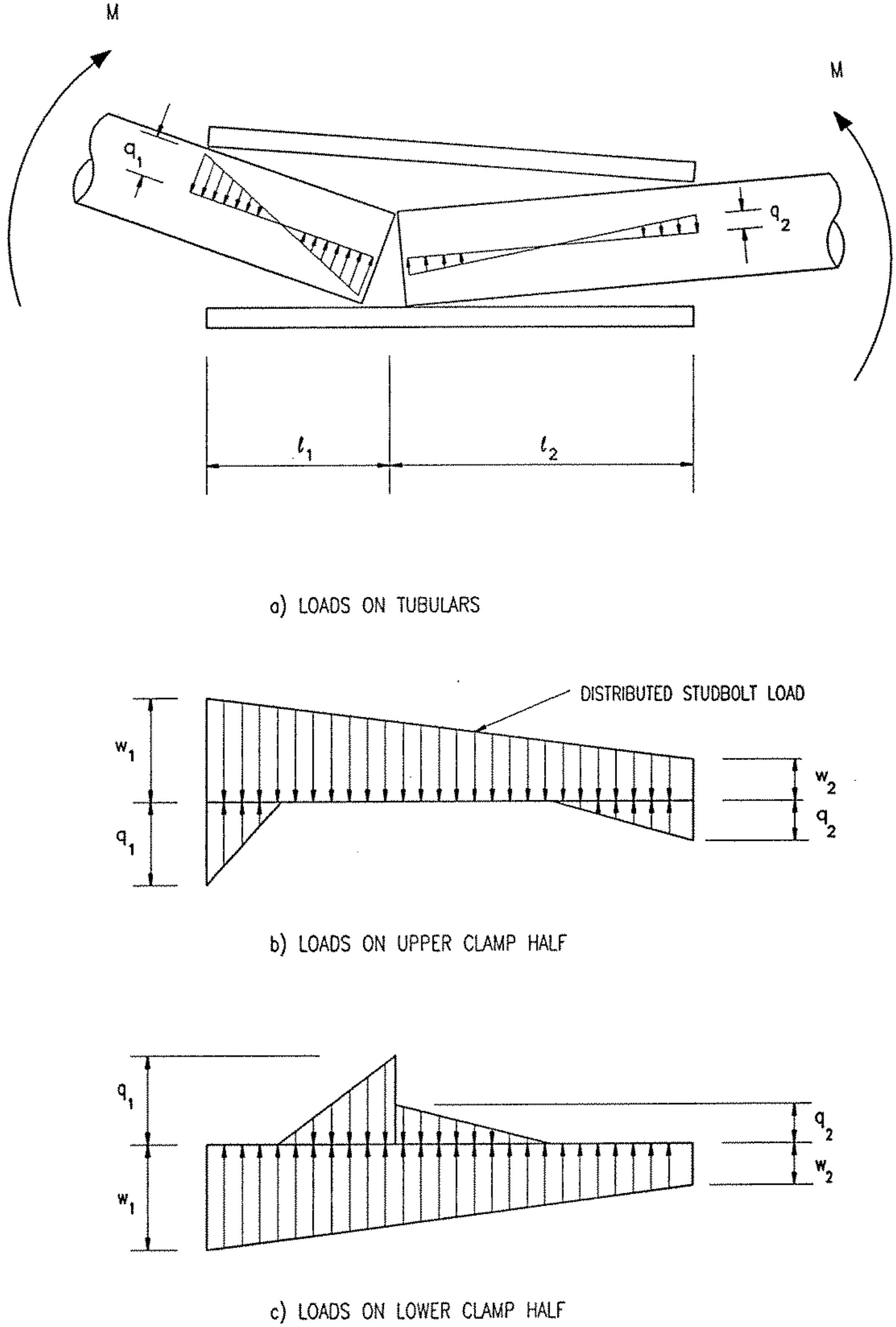


Fig. 5.5: Internal distributed load assuming rigid clamp (Model 2B)

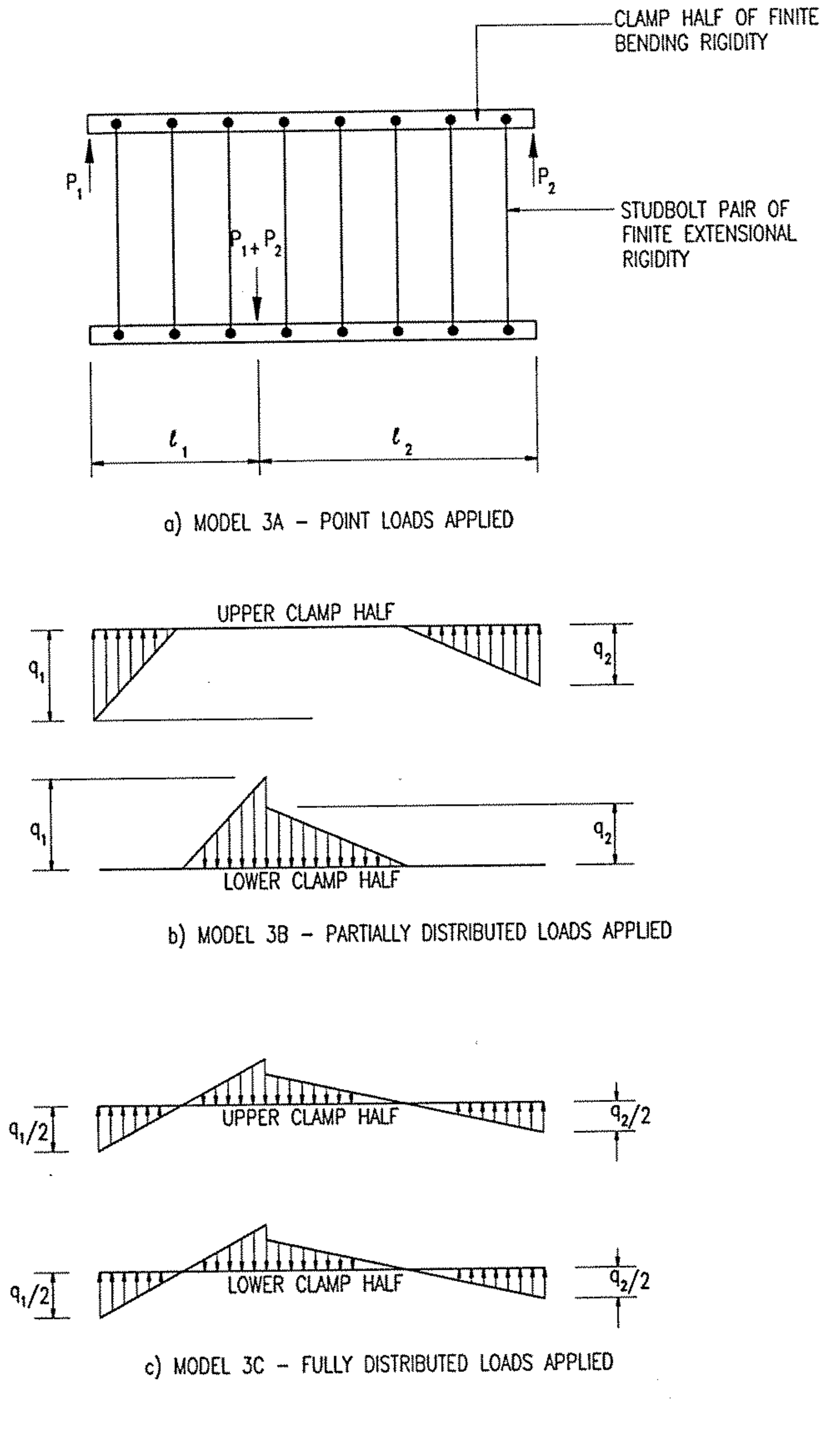


Fig. 5.6: Models 3A, 3B and 3C incorporating clamp and studbolt rigidity

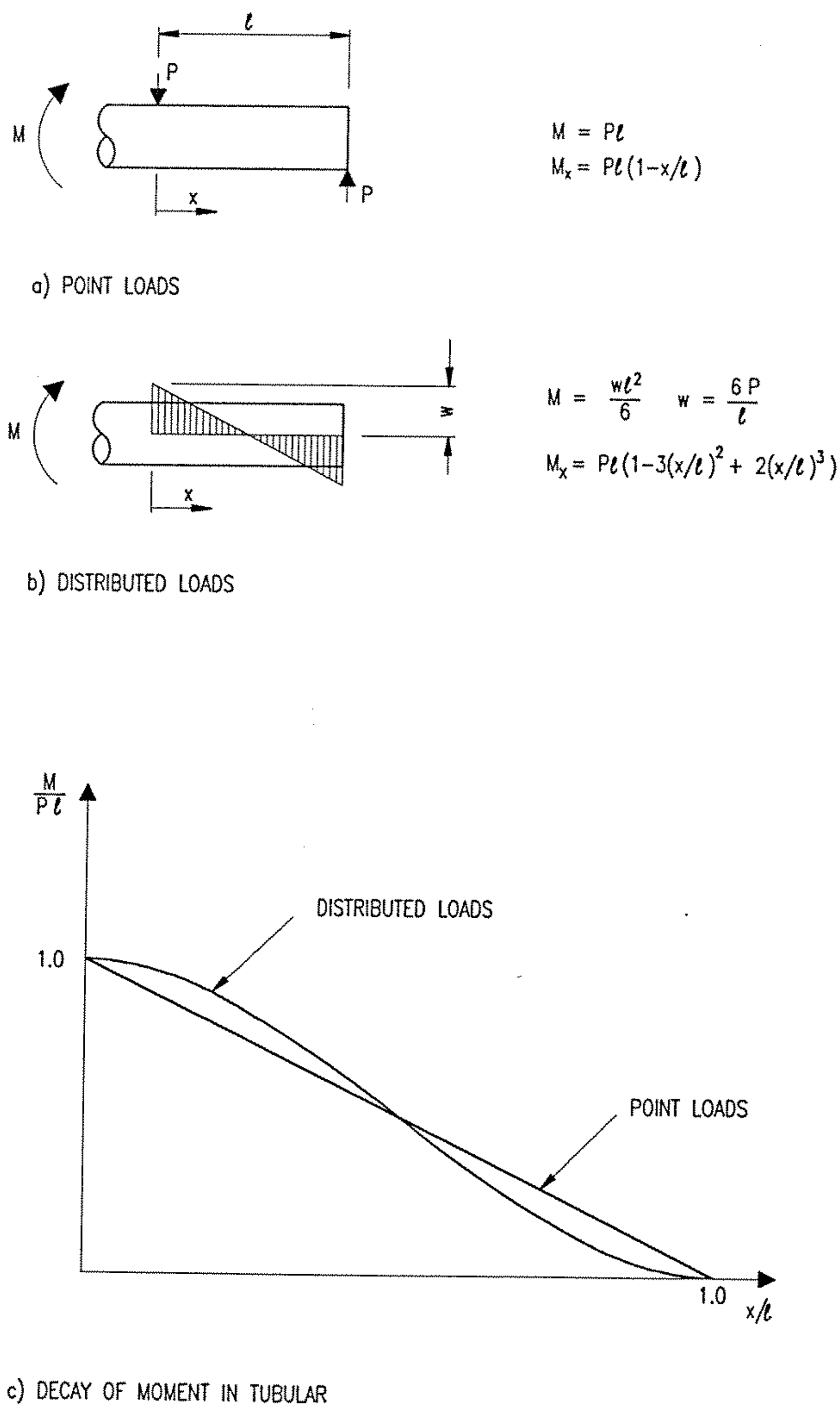


Fig. 5.7: Moment decay in tubular members

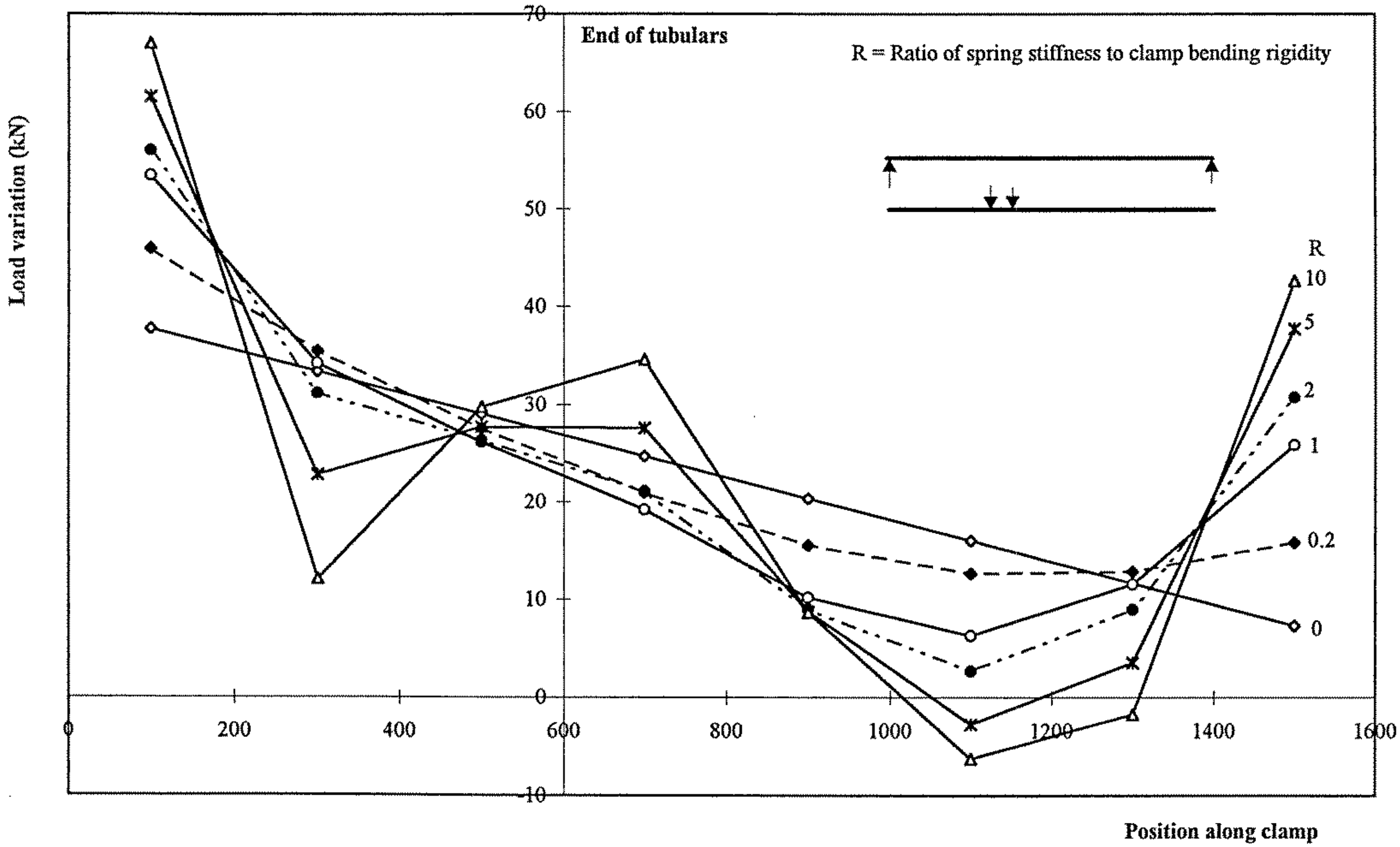


Fig. 5.8: Load variation for clamp position 37.5/62.5, M = 140 kNm, Model 3A

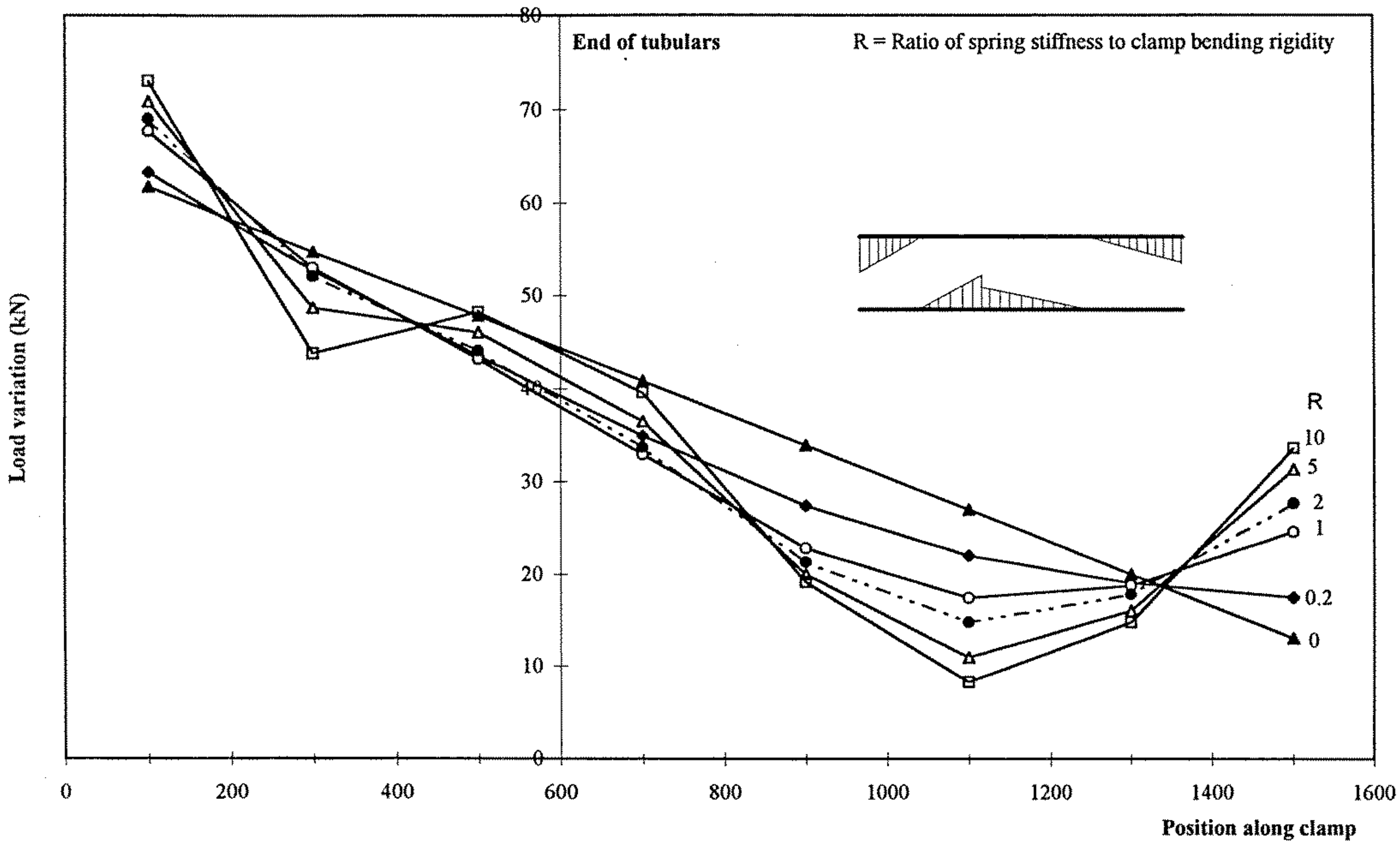


Fig. 5.9: Load variation for clamp position 37.5/62.5, M = 140 kNm, Model 3B

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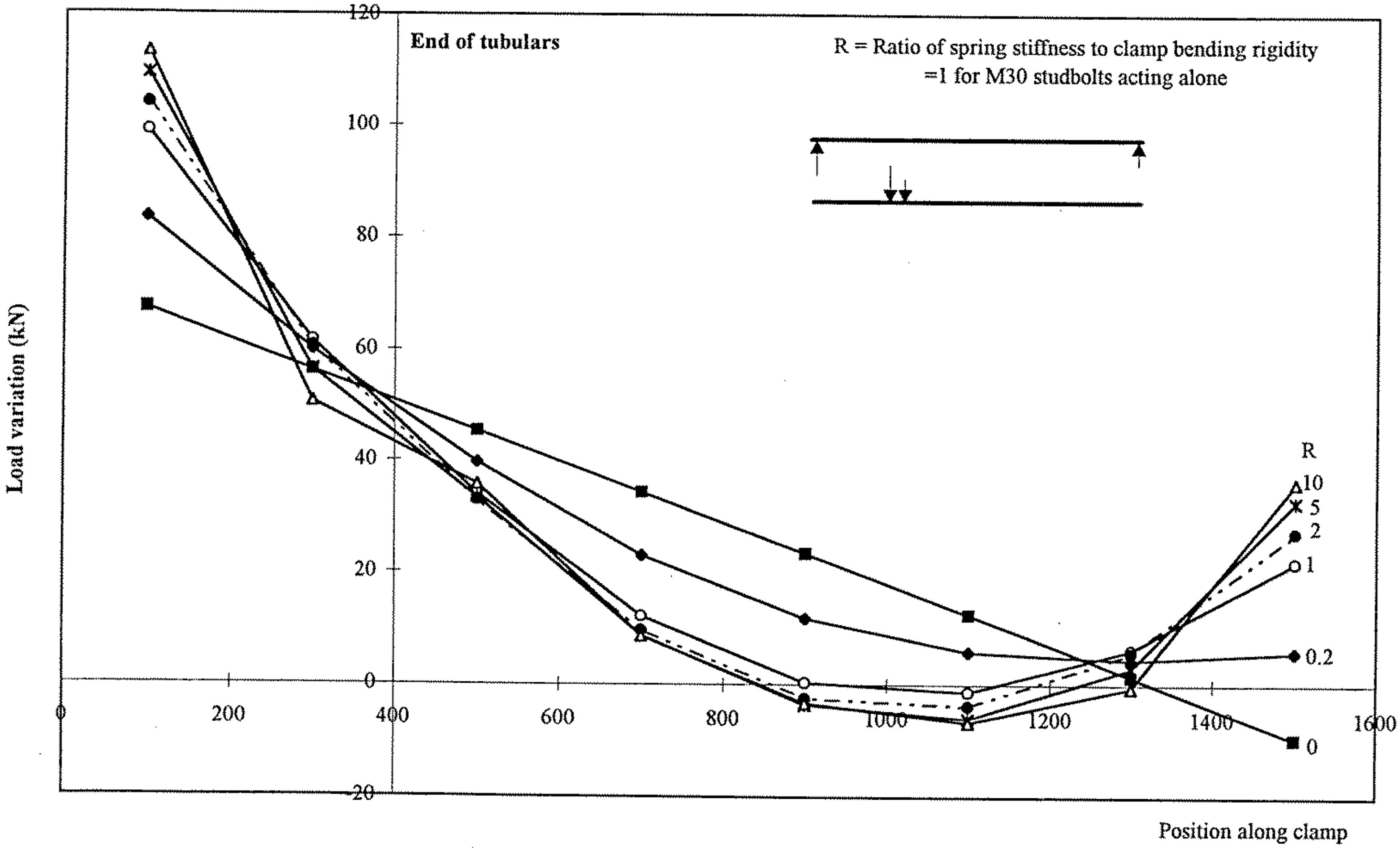

Fig. 5.10: Load variation for clamp position 25/75, M = 140 kNm, Model 3A

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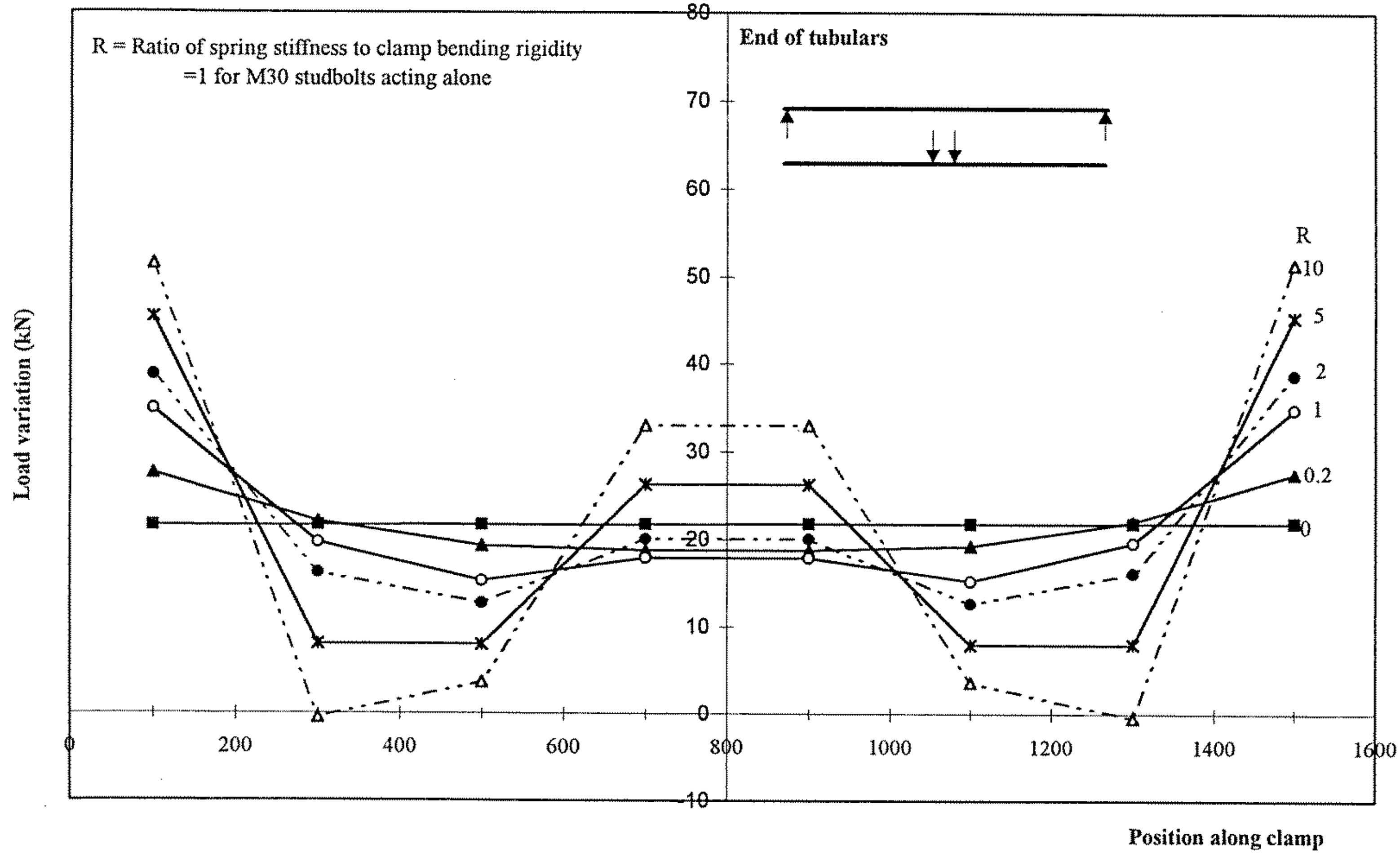

Fig. 5.11: Load variation for clamp position 50/50, M = 140 kNm, Model 3A

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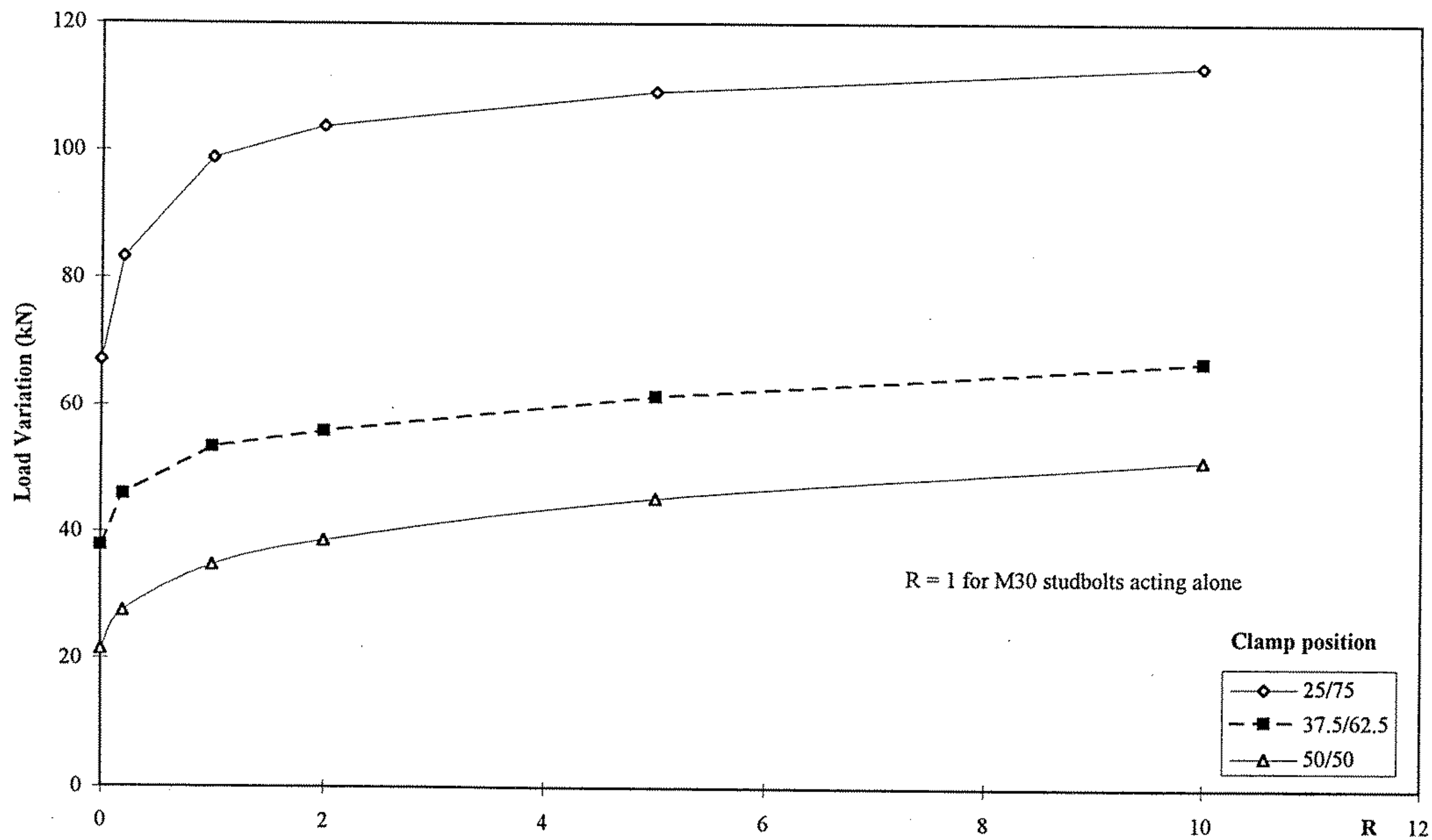

Fig. 5.12: Effect of spring stiffness to clamp bending rigidity ratio (R) on studbolt load variation (Model 3A) for M_{IPB} = 140 kNm

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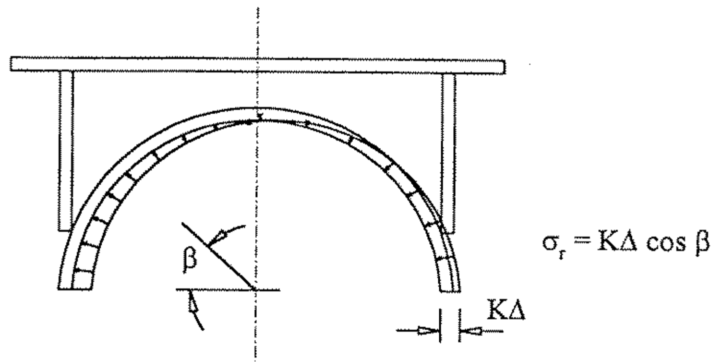
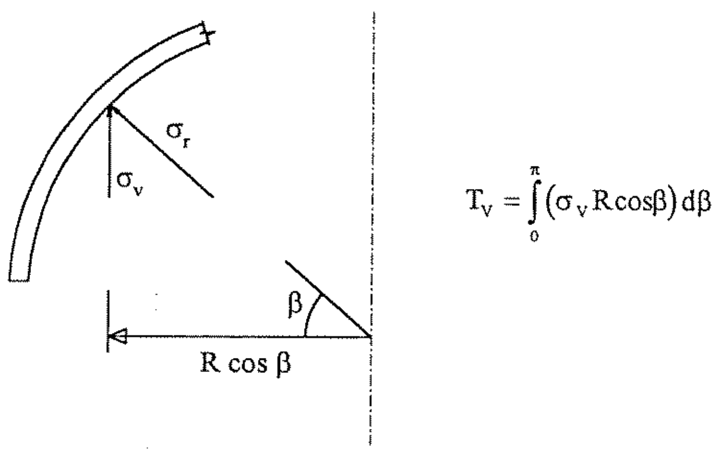
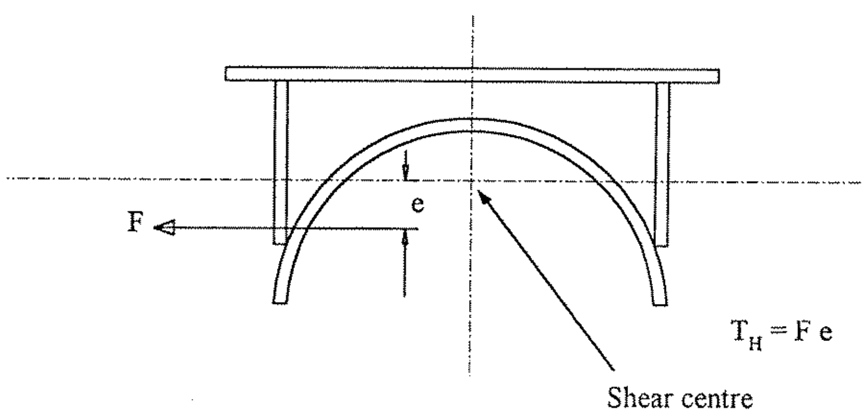


a) Assumed radial stress distribution at any given section (Typical)

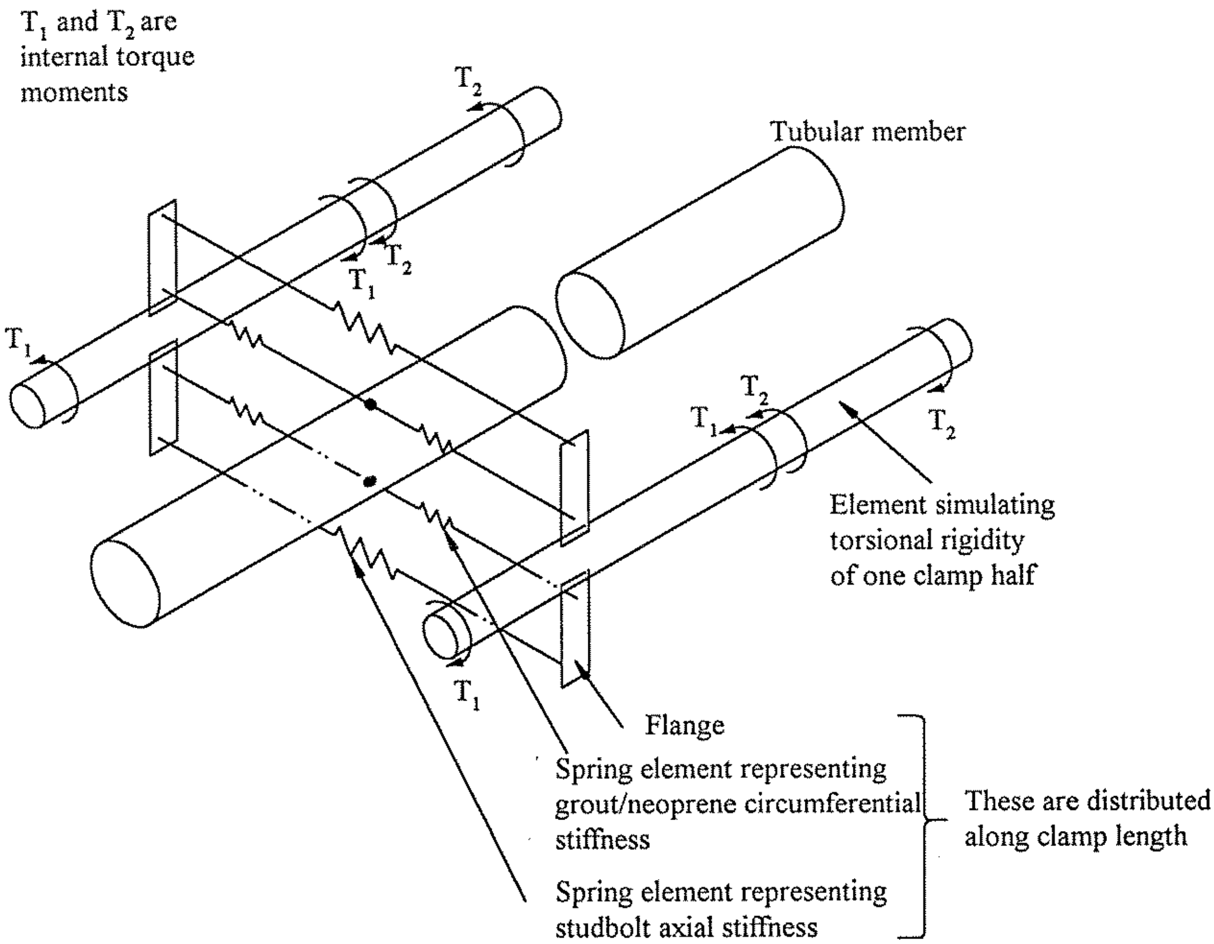


b) Torsion component due to resolved vertical stresses

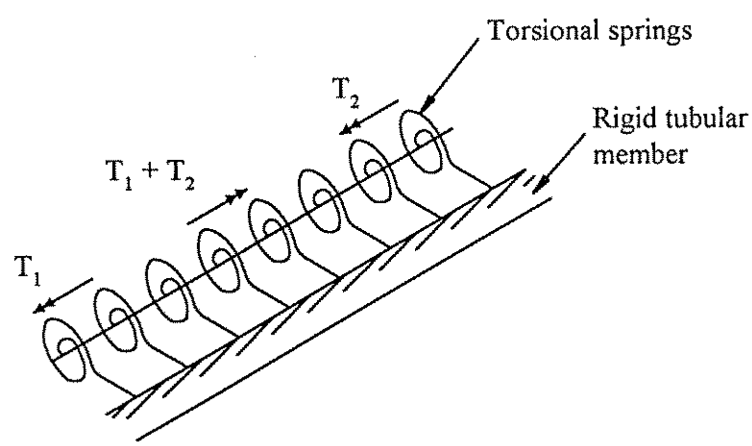


c) Torsion component due to eccentricity of horizontal resultant

Fig. 5.13: Mechanisms giving rise to torsional loading on clamp half (M_{OPB} applied)



(a) Mechanistic model



(b) Simplified FE model

Fig. 5.14: Development of model for out-of-plane bending (Model 4 - Point load case)

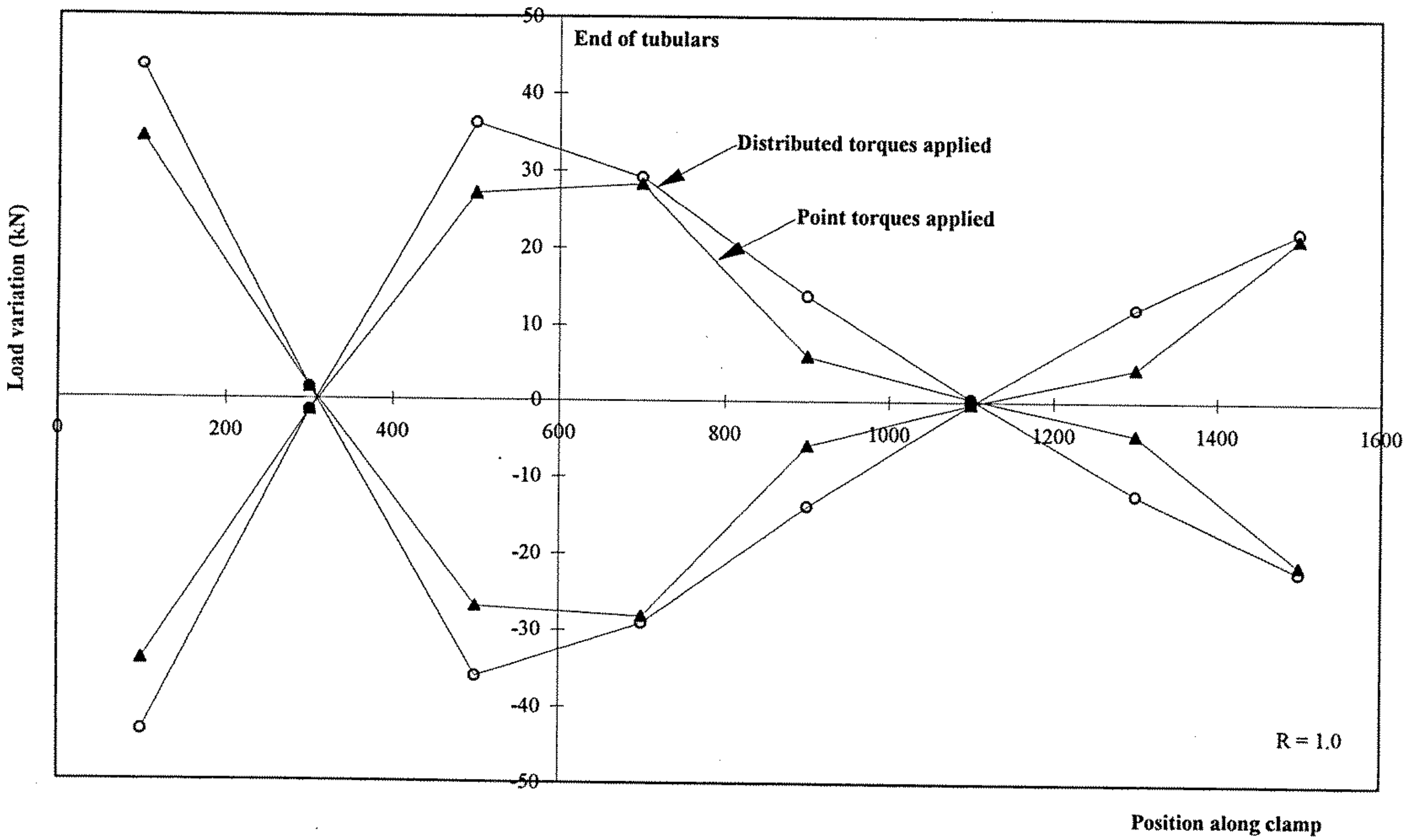


Fig. 5.15: Load variation for clamp position 37.5/62.5, M = 140 kNm, Model 4

S M:\C167\FE_NEW\OPB.XLS\Fig. 5.15

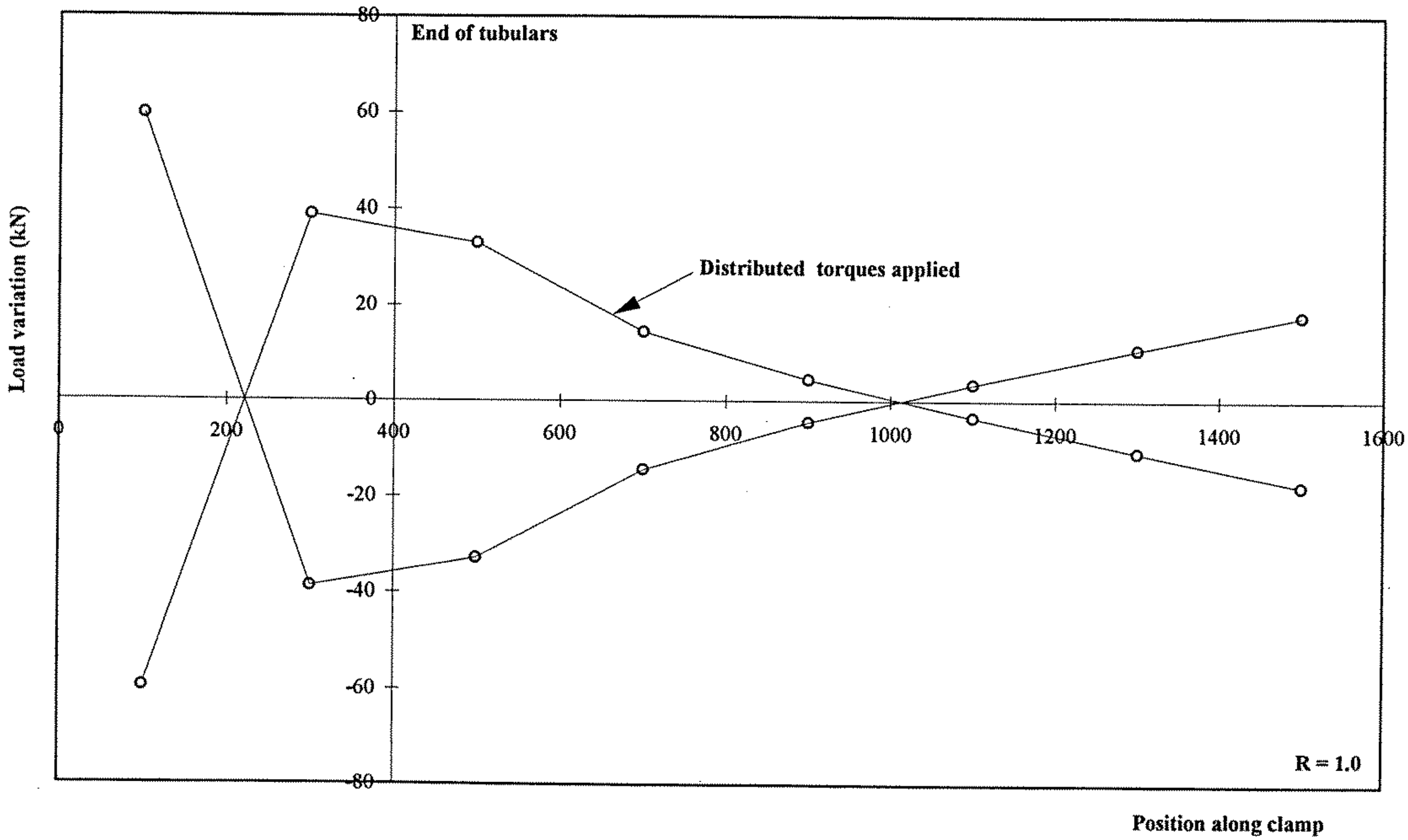


Fig. 5.16: Load variation for clamp position 25/75, M = 140 kNm, Model 4

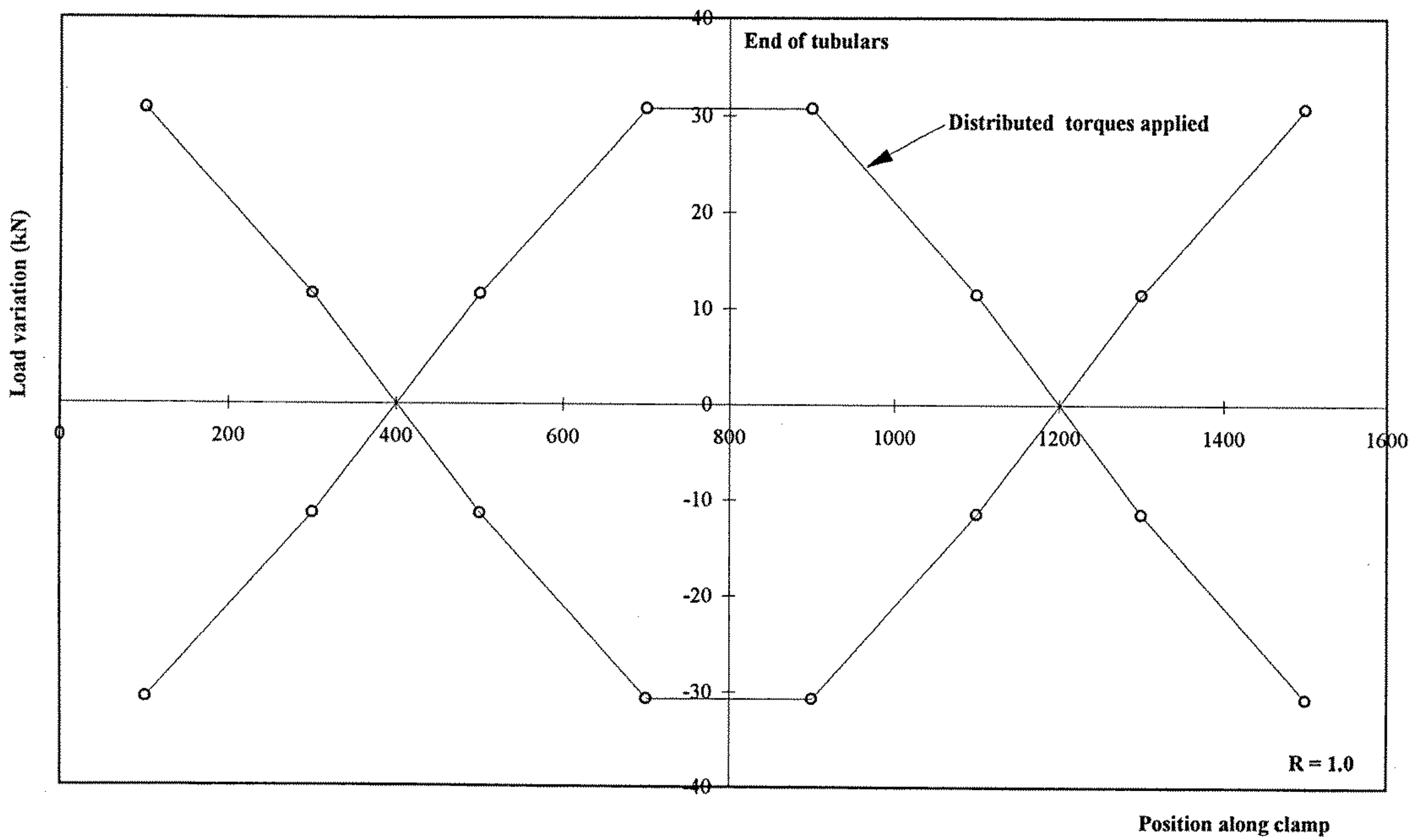


Fig. 5.17: Load variation for clamp position 50/50, M = 140 kNm, Model 4

S M:\C167\FE_NEW\OPB.XLS\Fig. 5.17

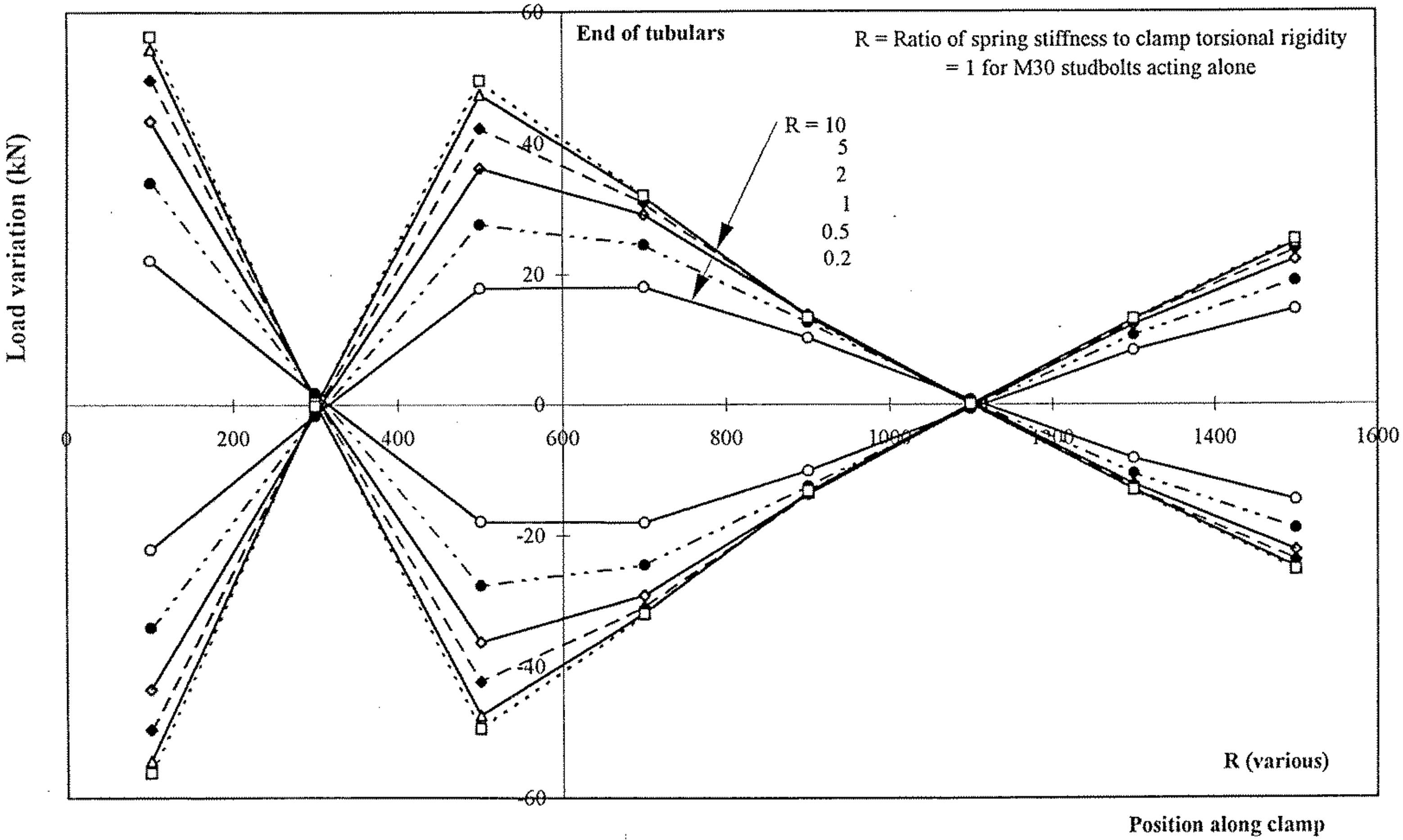


Fig. 5.18: Load variation for clamp position 37.5/62.5, M=140 kNm, Model 4

S M:\C FE-NEW\OPB.XLS\Fig. 5.18

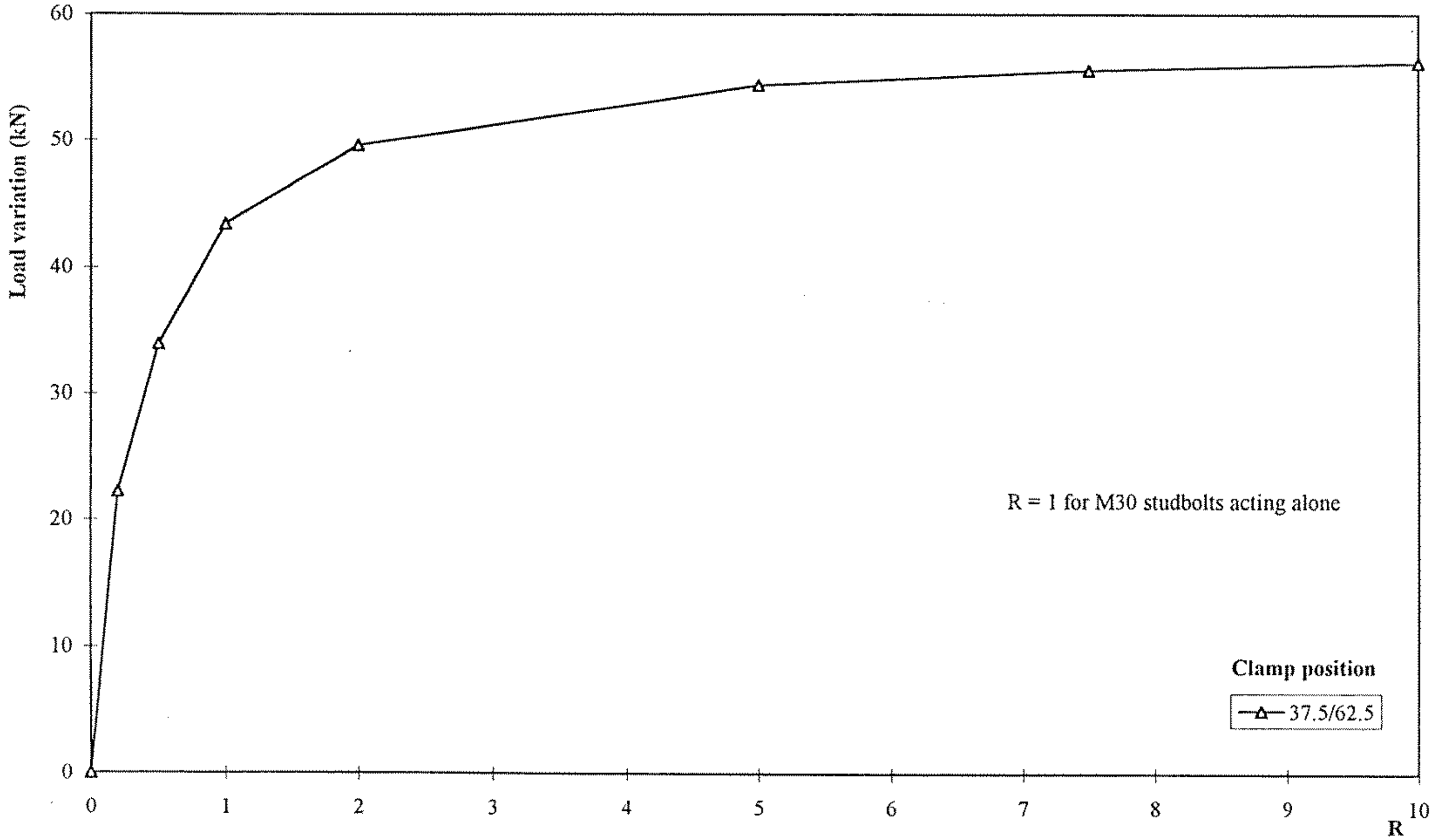
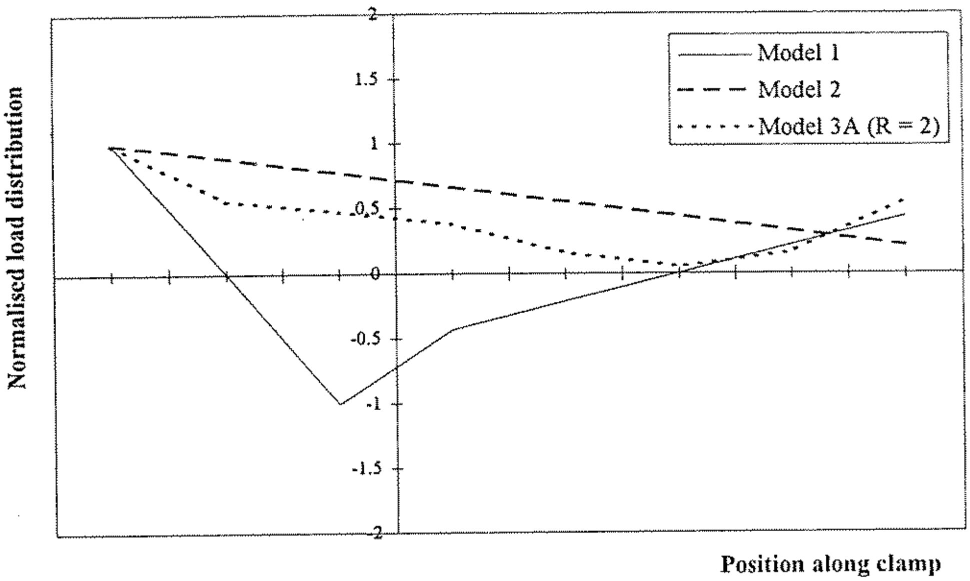
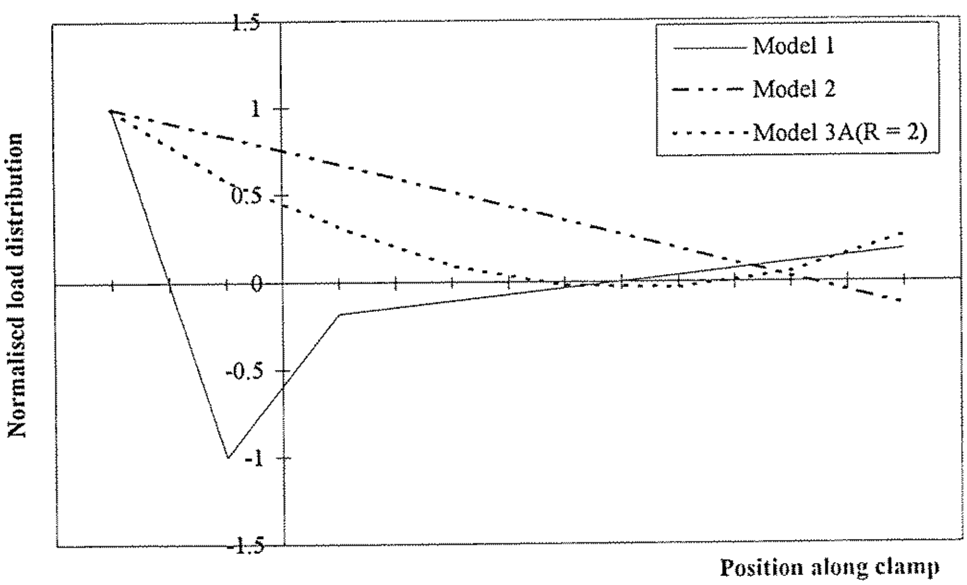


Fig. 5.19: Effect of spring stiffness to clamp torsional stiffness ratio (R) on studbolt load variation (Model 4) for M_{OPB} = 140 kNm

S M:\C167\FE_NEW\OPB.XLS\Fig. 5.19

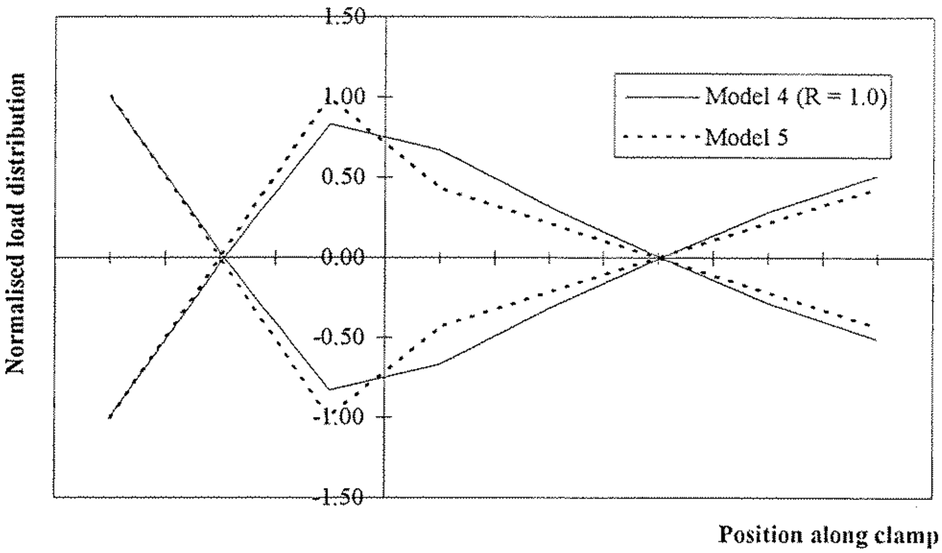


a) Clamp position 37.5/62.5

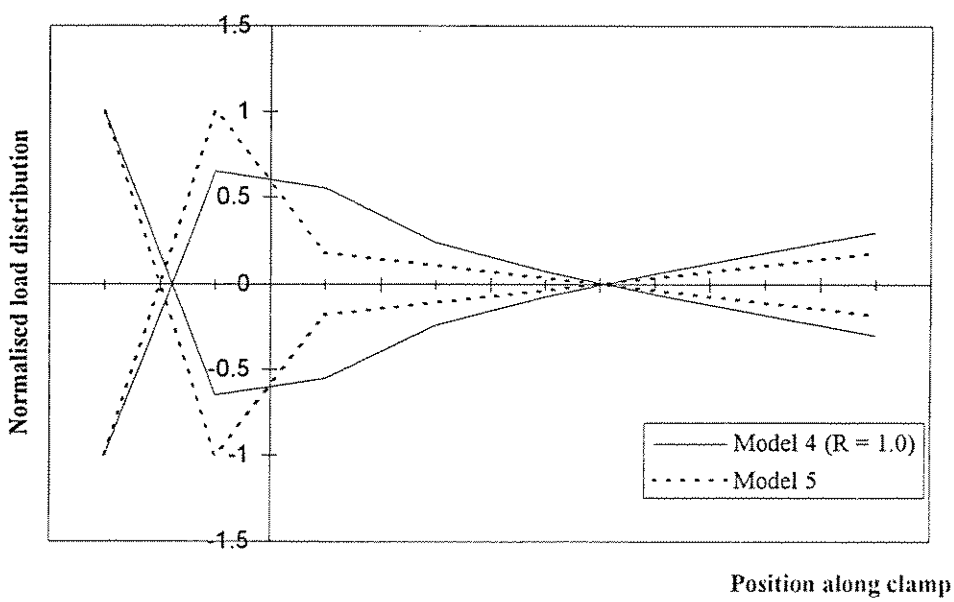


b) Clamp position 25/75

Fig. 5.20: Comparisons of studbolt load distribution patterns for IPB



a) Clamp position 37.5/62.5



b) Clamp position 25/75

Fig. 5.21: Comparisons of studbolt load distribution patterns for OPB

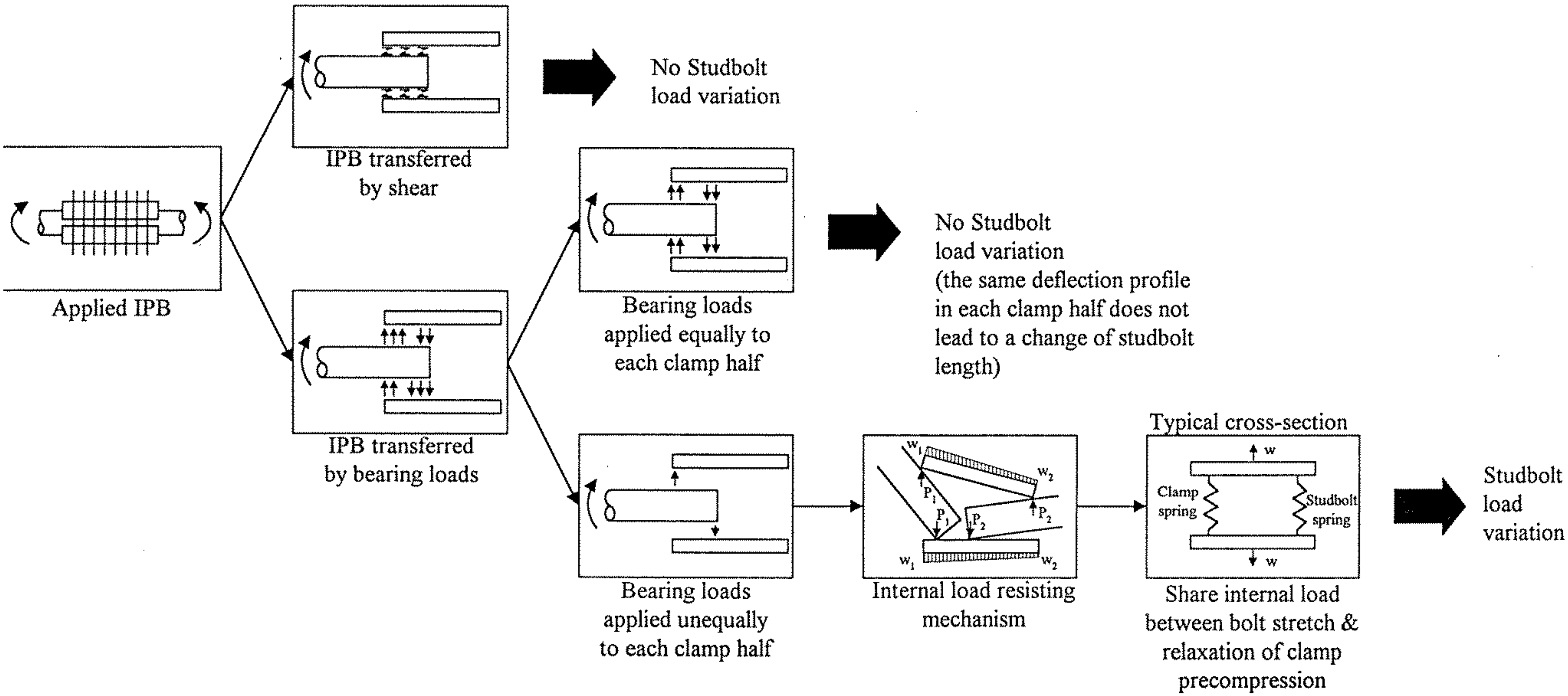


Fig 5.22 :Summary of IPB behaviour

S M:/c167/fig522.vsd

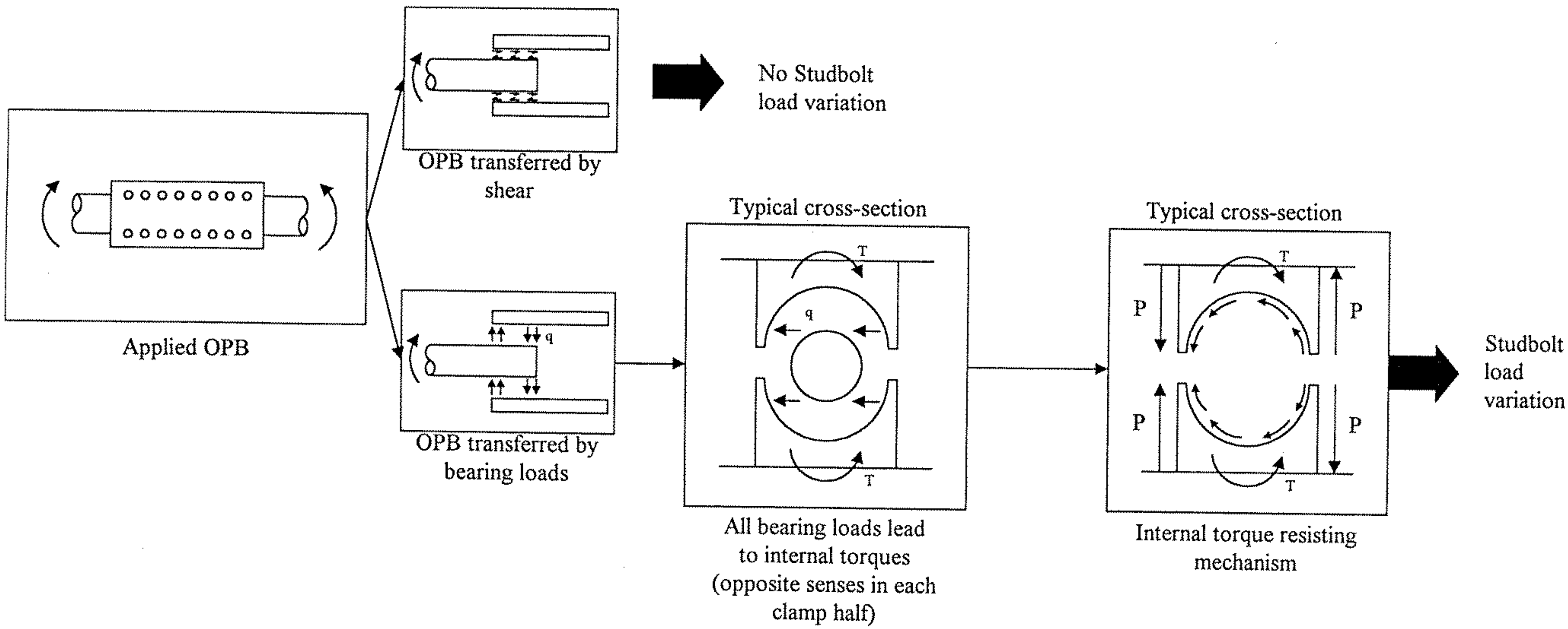
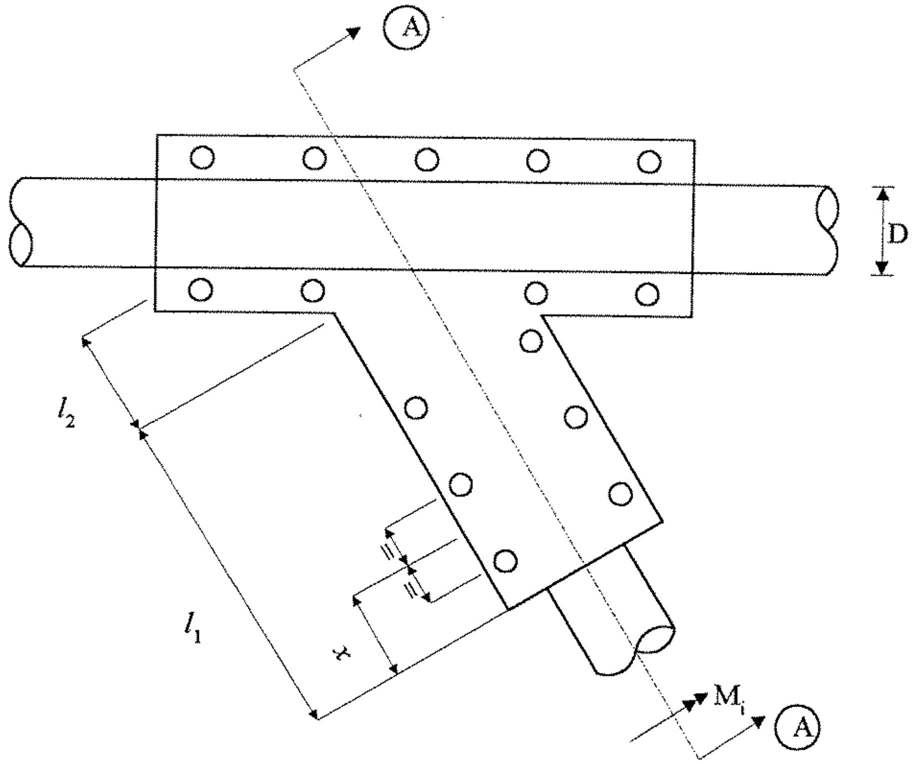
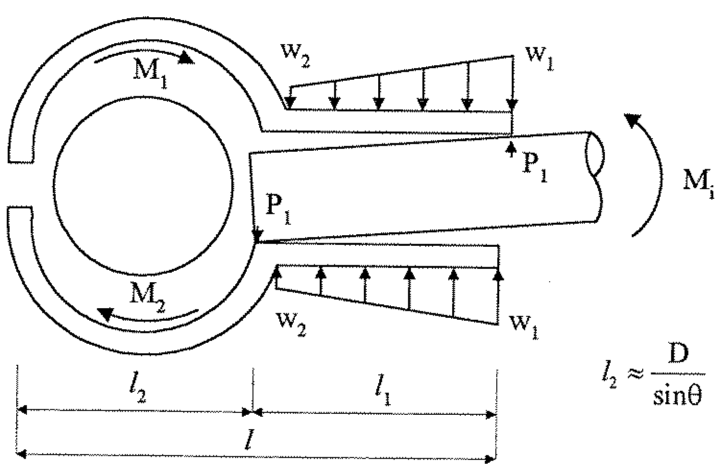


Fig 5.23 :Summary of OPB behaviour

S M:/c167/fig523.vsd



Plan



Cross-section A-A

Fig 6.1: Rigid clamp model for nodal clamps

APPENDIX A

COMPLETE SET OF TEST DATA

TEST NO 1: CLAMP POSITION 25/75, IPB (STUDBOLTS VERTICAL), STUDBOLT PRELOAD 180 kN.

MOMENT (kNm)	0	20	40	60	80	90	95	0
Studbolt	Studbolt Load Variation (kN)							
BOLT A	-0.0543	0.4344	0.9774	2.2806	4.4526	6.0816	6.7332	-0.8688
BOLT B	0	-0.1575	-0.21	0.525	2.73	4.3575	5.3025	-0.2625
BOLT C	0	-0.05295	-0.05295	0.2118	1.1649	1.9062	2.2239	-0.05295
BOLT D	0	-0.1054	-0.3162	-0.527	-0.8959	-1.0013	-1.0013	0
BOLT E	0	0.05275	-0.1055	-0.26375	-0.58025	-0.68575	-0.7385	0.15825
BOLT F	0.0536	0	-0.1072	-0.3752	-0.6968	-0.7504	-0.804	0.1072
BOLT G	0	0.0528	0.0528	0	-0.1056	-0.1056	-0.1056	0.0528
BOLT H	0	0.05365	0.1073	0.2146	0.26825	0.4292	0.4292	0
BOLT I	0	0.1575	0.5775	1.7325	3.99	5.4075	6.09	-0.7875
BOLT J	0	-0.1062	0.0531	1.062	3.5046	5.2038	6.3189	-0.2655
BOLT K	0	0.05365	0.05365	0.5365	1.55585	2.3606	2.7898	0
BOLT L	0	0.0549	0.0549	0	0	0.1098	0.1647	0.1098
BOLT M	0	0.1074	0.0537	-0.1074	-0.3759	-0.537	-0.5907	0.1074
BOLT N	0	0	-0.159	-0.424	-0.848	-1.007	-1.06	0.106
BOLT O	0	-0.1071	-0.26775	-0.58905	-0.91035	-0.9639	-1.071	0.05355
BOLT P	0	0.053	0.053	0.106	0.053	0.212	0.212	0.053
Strain Gauge No.	STRAIN (µStrain)							
33	0	-5	-9	-11	-11	-11	-11	1
34	1	-11	-22	-43	-74	-91	-100	1
35	0	-17	-33	-52	-74	-85	-90	-1
36	0	-12	-24	-32	-35	-35	-37	4
37	0	14	28	40	51	60	64	0
38	1	-2	-9	-16	-23	-23	-24	-1
39	0	-13	-25	-42	-63	-72	-80	-1
40	0	-38	-75	-98	-122	-133	-139	0
41	0	-135	-264	-385	-502	-557	-584	1
42	0	71	144	182	144	117	96	5
43	0	47	93	137	185	211	224	3
44	0	1	1	0	-4	-7	-7	2
45	0	2	3	4	2	2	2	1
46	1	0	0	-1	-2	-1	-1	2
47	0	0	1	2	3	5	5	1
48	0	-1	-3	-5	-8	-9	-9	25
49	-1	-8	-13	-19	-23	-25	-26	0
50	0	-18	-37	-58	-86	-101	-108	2
51	-1	-9	-18	-29	-43	-50	-55	0
52	0	-2	-4	-7	-11	-12	-13	0
53	0	9	18	25	32	36	39	0
54	0	13	24	34	43	48	50	0
55	-1	55	111	163	215	241	254	2
56	-2	96	200	301	398	448	472	2
57	-1	128	254	377	501	561	593	0
58	0	-2	-4	-10	-18	-21	-23	0
59	0	1	2	4	5	7	8	1
60	0	1	2	4	4	5	7	0
61	0	-4	-8	-11	-14	-14	-14	0
62	0	-14	-27	-41	-54	-58	-60	1
63	0	-55	-110	-163	-218	-244	-257	-1
64	0	-101	-196	-292	-389	-434	-460	-2
65								
66	1	-1	-3	2	18	28	34	0
67	0	39	75	112	149	168	178	3

DIAL GAUGE No.	DISPLACEMENT (mm)							
1	0	4.28	8.36	12.32	16.35	18.45	19.5	0.36
2	0	5.71	11.25	16.75	22.34	25.23	26.7	0.41
3	0	3.67	7.18	10.65	14.19	15.91	16.82	0.26
4A	0	0.1	0.18	0.23	0.24	0.26	0.25	0.02
5H	0	0.06	0.07	0.07	0.09	0.11	0.1	0.03

TEST NO 2: CLAMP POSITION 25/75, OPB (STUDBOLT HORIZONTAL), STUDBOLT PRELOAD 180 kN.

MOMENT (kNm)	0	20	40	60	80	90	95	0
Studbolt	Studbolt Load Variation (kN)							
BOLT A	0	-6.8418	-13.0863	-18.4077	-22.4802	-24.7608	-25.8468	-3.3123
BOLT B	0.1575	3.3075	6.7725	10.7625	15.435	17.85	19.3725	-1.47
BOLT C	0	4.5537	9.05445	13.8729	18.90315	21.4977	23.03325	0.05295
BOLT D	0.1054	3.2674	6.1132	9.2225	12.1737	13.5439	14.4925	0.7905
BOLT E	0.05275	2.16275	4.27275	6.27725	8.28175	9.1785	9.75875	0.633
BOLT F	0.0536	1.2864	2.4656	3.6984	4.7168	5.1456	5.4136	0.3216
BOLT G	0.0528	-1.0032	-1.9536	-2.9568	-4.0128	-4.8048	-5.0688	0
BOLT H	0.1073	-4.98945	-9.81795	-14.3782	-18.93845	-21.5673	-22.58665	-0.2146
BOLT I	0	8.5575	17.6925	27.6675	38.85	44.1	47.04	-3.2025
BOLT J	0.0531	-2.2302	-4.248	-5.4693	-6.0003	-6.0534	-5.7879	-1.4868
BOLT K	0.1073	-3.7555	-7.4037	-10.73	-13.7344	-15.2366	-15.82675	-0.26825
BOLT L	0.1098	-2.8548	-5.5998	-8.3997	-11.1996	-12.627	-13.3956	0.4941
BOLT M	0.0537	-2.0406	-4.0812	-6.2292	-8.3772	-9.6123	-10.0956	0.4833
BOLT N	0	-1.166	-2.332	-3.392	-4.558	-5.247	-5.512	0.53
BOLT O	0	1.071	2.142	3.15945	4.23045	4.7124	4.98015	0.4284
BOLT P	0	4.823	9.593	14.363	18.974	20.882	22.048	-0.477
Strain Gauge No.	STRAIN (µStrain)							
33	0	53	110	169	235	268	286	6
34	1	94	172	242	296	305	306	-46
35	1	177	343	507	669	748	790	10
36	0	42	95	158	233	303	353	144
37	1	-79	-164	-253	-350	-408	-438	-8
38	0	-47	-97	-151	-211	-245	-257	-16
39	0	-146	-288	-423	-562	-628	-660	7
40	0	-6	-17	-26	-34	-41	-43	3
41	1	-15	-27	-37	-46	-54	-59	-8
42	0	0	2	3	0	-4	-8	10
43	0	11	22	37	55	64	70	-5
44	0	1	1	1	0	-3	-4	3
45	0	17	32	48	64	70	75	3
46	1	32	64	96	129	145	155	6
47	0	57	116	173	233	263	278	8
48	0	109	205	301	396	446	472	24
49	0	130	255	377	502	562	595	-1
50	0	-2	-8	-14	-22	-30	-34	7
51	-1	-9	-15	-22	-30	-34	-35	4
52	0	-24	-49	-74	-98	-113	-119	3
53	1	-53	-108	-165	-223	-256	-272	-1
54	0	0	-1	-2	-3	-5	-5	2
55	0	-2	-2	-1	-3	-5	-5	0
56	-1	-8	-12	-17	-22	-25	-27	-2
57	0	-12	-24	-32	-43	-52	-54	-4
58	-2	-3	-4	-4	-7	-11	-11	1
59	-1	-60	-116	-169	-224	-255	-270	-8
60	-2	-96	-183	-272	-361	-407	-431	-8
61	0	-116	-225	-328	-433	-486	-513	0
62	0	0	-1	-2	-4	-6	-6	2
63	-1	0	2	6	8	8	9	3
64	0	-6	-11	-14	-19	-22	-23	5
65								
66	1	4	9	17	28	35	39	-2
67	0	2	4	7	10	9	10	-2

DIAL GAUGE No.	DISPLACEMENT (mm)							
1	0	4.14	8.11	12.03	16.04	18.16	19.2	0.57
2	0	5.15	10.03	16.16	21.6	24.54	26.06	0.86
3	0	3.71	7.22	10.9	14.93	17.03	18.06	0.43
4I	0	-0.08	-0.2	-0.32	-0.46	-0.52	-0.55	0.03
5P	0	-0.04	-0.09	-0.14	-0.19	-0.21	-0.22	-0.01
6	0	5.51	10.86	16.23	21.7	24.65	26.19	0.49

TEST NO 3: CLAMP POSITION 37.5/62.5, IPB (STUDBOLTS VERTICAL), STUDBOLT PRELOAD 90 kN.

MOMENT (kNm)	0	10	20	30	40	50	60	70	0
Studbolt	Studbolt Load Variation (kN)								
BOLT A	0.1086	0.2172	0.2715	0.2172	0.1629	0.0543	-0.1086	-0.3801	-1.9005
BOLT B	0.0525	0.105	0	0.105	0.21	0.1575	0.315	0.525	-0.735
BOLT C	0.05295	0.15885	0.2118	0.6354	1.00605	1.32375	1.95915	2.6475	-0.15885
BOLT D	0.0527	0	0	0.1581	0.4216	0.6851	1.0013	1.4756	0.0527
BOLT E	-0.05275	0	0.1055	0.3165	0.58025	0.79125	1.1605	1.52975	0.05275
BOLT F	0.1072	0.1072	0.1608	0.268	0.4288	0.4824	0.6432	0.6968	0.2144
BOLT G	0.1056	0.0528	0.0528	0	0	-0.1056	-0.2112	-0.264	0.1584
BOLT H	0.1073	0.05365	0	0.16095	0.1073	0.16095	0.16095	0.05365	0.05365
BOLT I	0	0.0525	0.0525	0	-0.1575	-0.315	-0.63	-1.05	-2.1
BOLT J	0	0.0531	0.2655	0.4779	0.7434	0.8496	1.2213	1.593	-0.5841
BOLT K	0	0.1073	0.37555	0.7511	1.073	1.5022	2.19965	3.0044	-0.05365
BOLT L	0	0.1098	0.2745	0.549	0.9333	1.3176	1.9215	2.5254	0.1098
BOLT M	0.0537	0.1074	0.1074	0.0537	0.1611	0.3759	0.4833	0.6981	0.1611
BOLT N	0	0	-0.053	0	-0.053	-0.053	-0.053	-0.106	0.106
BOLT O	0	0	-0.05355	-0.05355	-0.1071	-0.1071	-0.2142	-0.26775	0.05355
BOLT P	0.053	0.053	0.106	0.106	0.159	0.212	0.265	0.265	0.053
Strain Gauge No.	STRAIN (µStrain)								
33									
34	-4	-10	-19	-27	-35	-46	-58	-67	-12
35									
36	-4	11	24	40	56	74	93	112	2
37	4	0	0	-1	-1	-1	-2	-4	26
38									
39									
40	6	-7	-18	-28	-36	-43	-52	-59	7
41	4	-52	-107	-157	-210	-261	-314	-367	-9
42	4	21	41	61	80	97	119	140	5
43	3	6	9	12	15	16	19	21	3
44									
45	-3	-6	-8	-10	-12	-15	-18	-22	-3
46	4	2	1	0	-1	-3	-4	-7	3
47	3	0	-1	-2	-4	-6	-8	-10	3
48	4	-1	-8	-13	-17	-21	-24	-22	7
49	4	-4	-11	-16	-23	-29	-34	-39	3
50									
51	-3	-7	-10	-13	-17	-21	-25	-30	-1
52									
53	-2	3	7	12	17	21	24	28	-2
54	6	11	17	24	30	36	43	50	6
55	5	39	73	105	138	170	202	234	8
56	5	72	138	200	265	326	389	448	8
57	5	85	163	238	313	386	459	529	6
58	4	4	4	4	4	4	5	4	5
59	4	4	5	6	7	8	9	10	4
60	4	3	2	2	3	3	2	2	4
61	4	0	-3	-5	-8	-10	-12	-15	5
62	3	-3	-8	-13	-20	-25	-32	-38	3
63	2	-32	-63	-95	-125	-155	-186	-215	1
64									
65									
66	-3	-13	-25	-36	-48	-61	-74	-85	-7
67	3	14	24	33	42	51	60	67	3

DIAL GAUGE No.	DISPLACEMENT (mm)								
1	0	2.26	4.49	6.61	8.61	10.64	12.6	14.53	0.75
2	0	2.39	5.3	8.08	10.78	13.35	16.21	18.9	0.23
3	0	1.89	3.82	5.65	7.38	9.12	10.79	12.47	-0.14
4A	0	0	0	0	0	0	0.01	0.01	0
5H	0	0	0	0	0	0	0	0	0

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TEST NO 4: CLAMP POSITION 37.5/62.5, OPB (STUDBOLTS HORIZONTAL), STUDBOLT PRELOAD 90 kN.

MOMENT (kNm)	0	10	20	30	40	50	60	70	0
Studbolt	Studbolt Load Variation (kN)								
BOLT A	0.0543	-3.4209	-6.2988	-9.4482	-12.6519	-15.8013	-18.7878	-21.72	-0.5973
BOLT B	0	0.1575	0.2625	0.42	0.4725	0.6825	0.945	1.3125	-0.21
BOLT C	-0.05295	2.6475	5.03025	7.7307	10.53705	13.50225	16.6263	19.85625	0.68835
BOLT D	0	2.5296	4.8484	7.378	9.9603	12.7534	15.5992	18.5504	0.6324
BOLT E	0	1.42425	2.954	4.431	5.96075	7.64875	9.23125	10.972	0.5275
BOLT F	0.0536	0.6968	1.4472	2.144	2.7872	3.484	4.2344	4.9848	0.536
BOLT G	0	-0.4224	-0.8448	-1.3728	-1.9008	-2.5872	-3.1152	-3.696	0.1584
BOLT H	0	-2.62885	-4.88215	-7.45735	-9.7643	-12.17855	-14.3782	-16.57785	0.69745
BOLT I	0	3.675	7.035	10.5	13.4925	16.695	20.0025	23.31	-1.155
BOLT J	-0.0531	0.1593	0.3186	0.5841	0.9027	1.2744	1.6992	2.2302	0.3717
BOLT K	0	-1.87775	-3.3263	-4.7212	-5.7942	-6.81355	-7.6183	-8.42305	0.48285
BOLT L	0	-1.9764	-3.4587	-5.1057	-6.3684	-7.5762	-8.6193	-9.6624	0.3294
BOLT M	0	-1.4499	-2.8461	-4.1886	-5.2626	-6.6051	-7.6254	-8.8068	0.2685
BOLT N	0	-0.742	-1.378	-2.12	-2.809	-3.498	-4.081	-4.77	0.371
BOLT O	0	0.69615	1.1781	1.8207	2.4633	3.15945	3.6414	4.284	0.69615
BOLT P	0	2.756	5.194	7.685	10.123	12.72	15.052	17.596	0.424
Strain Gauge No.	STRAIN (μStrain)								
33									
34	-6	34	72	110	146	184	220	257	4
35									
36	-5	1	10	22	36	54	77	99	34
37	2	-36	-74	-117	-159	-206	-256	-309	-14
38									
39									
40	5	1	-3	-7	-10	-11	-13	-15	8
41	4	-11	-24	-38	-52	-66	-81	-96	3
42	4	6	8	8	5	3	0	-1	4
43	3	7	13	18	23	31	39	46	17
44									
45	-3	11	23	36	48	60	73	85	2
46	4	34	59	86	111	139	165	192	10
47	4	60	108	160	207	260	309	358	12
48	3	75	138	206	269	341	408	479	39
49	3	90	166	245	317	396	468	544	5
50									
51	-2	-7	-11	-16	-21	-27	-34	-41	-2
52									
53	-1	-33	-60	-90	-118	-149	-178	-209	-3
54	5	5	6	7	7	8	9	10	6
55	4	2	1	-1	-3	-5	-7	-10	3
56	2	-1	-5	-8	-11	-14	-16	-19	3
57	2	-2	-6	-10	-14	-18	-21	-24	2
58	1	0	-2	-4	-5	-7	-7	-7	2
59	1	-51	-96	-145	-190	-239	-284	-330	-2
60	3	-59	-111	-165	-216	-270	-320	-370	2
61	1	-73	-137	-203	-266	-332	-394	-459	-3
62	3	3	4	4	4	5	5	5	3
63	1	1	1	0	-1	-3	-3	-4	4
64									
65									
66	-3	0	3	8	12	18	25	32	2
67	4	6	8	10	10	11	12	13	7

DIAL GAUGE No.	DISPLACEMENT (mm)								
1	0	2.38	4.38	6.34	8.27	10.24	12.1	14.04	0.65
2	0	2.1	4.6	7.21	9.71	12.32	14.83	17.36	0.02
3	0	2.13	3.95	5.86	7.72	9.6	11.46	13.41	0.73
4I	0	-0.03	-0.07	-0.1	-0.13	-0.16	-0.2	-0.23	0.01
5P	0	-0.02	-0.05	-0.07	-0.1	-0.13	-0.15	-0.18	0

TEST NO 5: CLAMP POSITION 37.5/62.5, IPB (STUDBOLTS VERTICAL), STUDBOLT PRELOAD 180 kN.

MOMENT (kNm)	0	20	40	60	80	100	120	130	140	0
Studbolt	Studbolt Load Variation (kN)									
BOLT A	0.1086	0.543	1.0317	1.4661	2.0091	2.6064	3.3123	3.5838	3.8553	-0.2715
BOLT B	0.105	0.105	0.0525	0.1575	0.105	0.1575	0.21	0.2625	0.315	0.1575
BOLT C	0	-0.05295	-0.2118	-0.26475	-0.4236	-0.5295	-0.6354	-0.6354	-0.68835	-0.15885
BOLT D	0	-0.2635	-0.527	-0.7905	-1.1594	-1.4229	-1.7391	-1.8972	-2.0026	-0.2108
BOLT E	0	-0.15825	-0.3165	-0.422	-0.58025	-0.7385	-0.89675	-1.00225	-1.10775	0
BOLT F	0	0	-0.0536	-0.1072	-0.1072	-0.1608	-0.1608	-0.2144	-0.1608	0.1608
BOLT G	0	0	-0.0528	-0.0528	-0.0528	-0.0528	0	-0.0528	0	0.1584
BOLT H	0	0.3219	0.48285	0.7511	1.12665	1.5022	1.87775	1.98505	2.146	-0.05365
BOLT I	0	0.21	0.42	0.63	0.84	1.1025	1.365	1.4175	1.5225	-0.42
BOLT J	0	0.1593	0.3186	0.4248	0.5841	0.6372	0.9027	1.062	1.062	0
BOLT K	0.05365	0.1073	0.16095	0.16095	0.2146	0.26825	0.3219	0.4292	0.5365	0.2146
BOLT L	-0.0549	-0.0549	0	-0.0549	0	-0.0549	0	0.0549	0.1098	0.0549
BOLT M	0	0	0.0537	0	0.0537	0.0537	0.1074	0.0537	0.1074	0.1074
BOLT N	0.053	0	0	-0.159	-0.212	-0.265	-0.265	-0.265	-0.318	0.212
BOLT O	0.05355	-0.1071	-0.26775	-0.3213	-0.48195	-0.6426	-0.69615	-0.80325	-0.91035	0
BOLT P	0.053	0.053	0.106	0.159	0.212	0.265	0.477	0.477	0.53	-0.053
Strain Gauge No.	STRAIN (μStrain)									
33										
34										
35										
36	-3	19	39	56	70	85	98	104	110	-19
37	1	-8	-14	-19	-20	-20	-20	-12	-9	54
38										
39										
40	5	-19	-41	-62	-82	-103	-122	-129	-137	6
41	4	-106	-207	-304	-397	-492	-585	-632	-682	1
42	4	30	53	77	100	122	147	161	174	8
43	3	14	22	30	38	43	49	51	55	7
44										
45	-4	-4	-6	-6	-6	-7	-7	-8	-8	-3
46	4	2	0	-1	-2	-3	-5	-5	-5	3
47	3	-1	-7	-11	-15	-20	-24	-27	-29	-1
48	4	-7	-16	-23	-32	-40	-46	-47	-52	10
49	4	-10	-21	-32	-43	-54	-64	-70	-75	3
50										
51	-4	-8	-12	-16	-19	-23	-27	-28	-31	-5
52										
53	-2	7	12	20	28	34	42	46	50	0
54	5	18	29	40	50	61	71	77	82	5
55	4	74	136	197	259	321	383	413	444	6
56	3	129	245	358	470	585	698	745	821	20
57	3	157	293	428	561	700	832	899	966	1
58	4	3	3	2	2	2	3	3	3	8
59	4	7	10	15	19	23	28	29	32	10
60	4	7	9	11	14	16	21	22	24	6
61	3	1	-1	-2	-2	-3	-2	-1	-1	9
62	3	-11	-24	-35	-46	-58	-69	-73	-76	4
63	1	-65	-127	-187	-246	-305	-364	-394	-426	0
64	-1	-119	-223	-315	-418	-520	-620	-672	-725	-3
65										
66	-4	-5	-7	-9	-10	-13	-16	-18	-20	-6
67	3	25	43	61	78	95	112	121	129	5
DIAL GAUGE No.	DISPLACEMENT (mm)									
1	0	5.12	7.98	11.55	15.6	19.86	23.54	25.29	27.03	0.23
2	0	6.32	10.19	14.84	19.53	24.16	28.81	31.14	33.35	0.15
3	0	3.9	6.45	9.44	12.47	15.4	18.44	19.19	21.46	0.38
4I	0	0	0	0	0	0.01	0.01	0.01	0.01	0
5P	0	0	0	0	0	0	0	0	0	0
6	0	6.3	10.11	14.7	19.31	23.86	28.44	30.71	33	0.07

TEST NO 6: CLAMP POSITION 37.5/62.5, 30deg FROM IPB AXIS
STUDBOLT PRELOAD 180 kN.

MOMENT (kNm)	0	20	40	60	80	100	120	130	140	0
Studbolt	Studbolt Load Variation (kN)									
BOLT A	0.1086	-3.0951	-5.8644	-8.4165	-10.9143	-13.3578	-15.8013	-17.2674	-18.3534	-0.3801
BOLT B	0.105	-0.105	-0.21	-0.315	-0.4725	-0.63	-0.5775	-0.735	-0.6825	0.0525
BOLT C	0	2.06505	4.1301	6.0363	7.99545	9.90165	11.80785	12.86685	13.92585	0.1059
BOLT D	0	1.8972	3.7417	5.4808	7.2726	9.0117	10.7508	11.6994	12.7007	0
BOLT E	0	1.21325	2.37375	3.587	4.80025	5.908	7.0685	7.64875	8.44	0.05275
BOLT F	0.0536	0.6432	1.34	1.9832	2.5728	3.1088	3.752	4.0736	4.556	0.2144
BOLT G	0.1056	-0.6336	-1.1088	-1.584	-2.0592	-2.5872	-3.1152	-3.3264	-3.696	-0.1056
BOLT H	0.05365	-2.4679	-4.7212	-6.81355	-8.85225	-10.9446	-12.9833	-13.8417	-15.022	-0.3219
BOLT I	0.105	3.36	6.93	10.185	13.4925	17.01	20.685	22.5225	24.36	-0.21
BOLT J	0.1593	0.2124	0.4779	0.7965	1.062	1.3806	1.6461	1.9116	2.124	0.1593
BOLT K	0.16095	-2.0387	-4.02375	-5.9015	-7.9402	-9.81795	-11.6957	-12.6614	-13.57345	-0.05365
BOLT L	0.2196	-2.0313	-4.0626	-6.0939	-7.9056	-9.9369	-11.9133	-12.8466	-13.7799	-0.0549
BOLT M	0.0537	-1.5036	-2.7924	-4.0275	-5.3163	-6.6588	-8.055	-8.6994	-9.3438	0
BOLT N	0.106	-0.795	-1.484	-2.12	-2.809	-3.657	-4.293	-4.505	-4.876	0.053
BOLT O	0.16065	0.37485	0.8568	1.23165	1.6065	2.08845	2.4633	2.73105	2.8917	0
BOLT P	0.053	2.491	5.035	7.367	9.805	12.19	14.628	15.9	17.013	-0.265
Strain Gauge No.	STRAIN (µStrain)									
33										
34										
35										
36	-2	18	40	61	81	102	123	136	148	10
37	2	-37	-75	-114	-149	-185	-224	-244	-261	1
38										
39										
40	6	-19	-41	-62	-82	-102	-122	-132	-142	7
41	5	-101	-200	-293	-381	-474	-567	-615	-664	-4
42	4	25	47	67	86	106	125	135	146	8
43	4	13	23	31	37	44	53	56	60	5
44										
45	-3	7	17	27	39	50	60	66	74	-2
46	4	28	52	75	99	123	146	160	174	9
47	4	48	90	132	174	216	259	284	314	17
48	3	61	107	156	211	270	309	337	382	-15
49	4	69	131	191	250	311	373	407	445	15
50										
51	-4	-15	-22	-31	-39	-47	-56	-60	-63	-2
52										
53	-1	-23	-43	-61	-80	-99	-117	-125	-132	0
54	6	14	25	34	43	53	61	67	71	4
55	6	64	118	171	224	278	331	358	385	3
56	5	110	209	303	399	496	597	651	725	38
57	5	133	249	364	479	593	710	768	826	1
58	4	0	-4	-6	-9	-12	-14	-14	-15	6
59	4	-44	-86	-126	-167	-209	-250	-271	-293	3
60	4	-56	-109	-159	-211	-263	-317	-342	-371	0
61	3	-73	-139	-204	-268	-335	-398	-430	-467	2
62	3	-10	-21	-32	-43	-53	-64	-68	-73	4
63	3	-56	-110	-162	-213	-264	-316	-340	-368	1
64	-2	-119	-203	-287	-380	-473	-564	-610	-659	-2
65										
66	-3	-5	-6	-6	-7	-8	-9	-10	-10	-5
67	5	22	41	56	72	88	104	113	123	4

DIAL GAUGE No.	DISPLACEMENT (mm)									
1	0	4.31	8.17	11.77	15.24	18.75	22.24	23.92	25.8	0.89
2	0	5.3	10.38	15.13	19.86	24.77	29.6	32.04	34.69	0.81
3	0	3.54	6.66	9.65	12.7	15.75	18.75	20.25	21.82	0.72
4I	0	-0.01	-0.01	-0.01	-0.01	-0.01	-0.2	-0.21	-0.22	-0.05
5P	0	0	0	0	0	-0.02	-0.02	-0.04	-0.07	-0.02
6	0	5.41	10.35	15.01	19.66	24.3	28.9	31.22	33.68	0.9

TEST NO 7: CLAMP POSITION 37.5/62.5, 60deg FROM IPB AXIS
STUDBOLT PRELOAD 180 kN.

MOMENT (kNm)	0	20	40	60	80	100	120	140	0
Studbolt	Studbolt Load Variation (kN)								
BOLT A	0.155841	-5.342034	-10.320801	-15.14427	-19.760856	-24.376899	-28.992942	-34.593987	-1.50411
BOLT B	0.0756	-0.02415	-0.175875	-0.326025	-0.526575	-0.62685	-0.7266	-1.27785	-0.4263
BOLT C	0.0503025	3.7928085	7.333575	10.621241	14.009511	17.448614	20.988851	24.731357	0.353706
BOLT D	-0.075888	3.545656	6.921618	10.193234	13.363666	16.636336	19.959071	23.482593	0.377332
BOLT E	#REF!	2.3932675	4.55971	6.67604	8.7928975	10.9087	12.973863	15.190945	0.32705
BOLT F	0.077184	1.357152	2.432368	3.507048	4.633184	5.708936	6.5794	7.602624	0.12864
BOLT G	0.025344	-0.78144	-1.638912	-2.396064	-3.152688	-4.009632	-4.866576	-5.774736	-0.076032
BOLT H	-0.025752	-4.304876	-8.1993295	-11.996677	-15.629855	-19.267861	-22.906941	-26.852362	-0.409886
BOLT I	0.075075	5.991825	11.5584	17.123925	22.790775	28.5579	34.4757	39.439575	-1.429575
BOLT J	0.152397	0.45666	0.760392	1.064655	1.47087	1.825578	2.333214	2.637477	-0.152928
BOLT K	0.051504	-3.485104	-6.8672	-9.9923125	-13.272474	-16.398659	-19.677747	-22.957372	-0.103008
BOLT L	0.157014	-3.408192	-6.764778	-10.015407	-13.161726	-16.360749	-19.717335	-22.967415	-0.052704
BOLT M	0.025776	-2.282787	-4.436694	-6.591675	-8.847612	-11.053608	-13.310082	-15.670197	0.025776
BOLT N	0.15211	-1.06318	-2.07548	-3.18954	-4.25219	-5.36625	-6.47978	-7.69507	0.05088
BOLT O	0.12852	0.997101	1.866753	2.684997	3.5037765	4.3734285	5.3448255	6.163605	0.025704
BOLT P	0.15211	4.55588	8.70737	12.55411	16.60437	20.65463	24.90629	29.05725	-0.25334
Strain Gauge No.	STRAIN (µStrain)								
33									
34	-6	34							
35									
36	-1.43	16.71	32.96	47.29	62.56	76.89	94.08	117.96	13.85
37	2.86	-52.54	-106.02	-162.38	-222.55	-282.73	-348.64	-419.31	-9.55
38									
39									
40	5.74	-10.51	-26.74	-40.11	-56.35	-71.63	-86.92	-104.11	5.74
41	5.26	-74.03	-143.76	-211.57	-279.39	-344.34	-414.07	-485.7	-5.25
42	4.3	16.72	28.18	40.59	51.1	61.61	73.07	83.58	7.16
43	3.82	13.37	21.96	29.61	36.29	42.03	50.62	61.13	12.41
44									
45	-2.38	17.67	35.82	54.92	74.03	93.13	113.18	131.33	-2.38
46	4.3	49.19	90.26	131.34	172.41	212.53	255.51	298.5	4.3
47	3.34	87.4	163.81	237.36	313.77	390.19	468.51	555.44	16.71
48	3.34	140.89	227.81	309.95	403.56	499.08	591.73	700.62	17.67
49	3.34	133.25	249.77	364.39	479.97	594.59	710.17	831.47	5.26
50									
51	-3.82	-14.33	-24.84	-35.34	-44.89	-54.44	-64.95	-73.55	0
52									
53	-1.92	-46.81	-87.88	-126.08	-168.11	-208.23	-251.21	-294.2	0.95
54	5.73	11.46	18.15	23.88	29.61	34.38	39.16	43.93	2.86
55	4.78	40.12	72.59	104.11	135.63	166.21	196.77	228.29	5.74
56	5.25	64.47	117	168.58	222.08	273.65	328.09	380.63	10.99
57	4.78	74.51	139.46	201.55	265.54	325.72	390.67	458.49	11.46
58	4.3	-0.48	-4.3	-7.16	-10.99	-14.81	-18.63	-23.4	3.34
59	3.34	-75.93	-148.53	-219.21	-290.85	-360.57	-435.08	-510.54	-3.34
60	3.34	-96.95	-184.82	-270.79	-356.75	-442.71	-531.55	-620.38	2.39
61	3.82	-114.62	-221.6	-325.72	-431.74	-533.94	-639.96	-749.81	0
62	3.82	-3.82	-11.46	-18.14	-24.83	-31.52	-39.15	-45.84	3.82
63	2.39	-30.08	-60.65	-90.26	-120.83	-150.43	-181.95	-212.53	0.48
64	-2.86	-66.85	-133.72	-189.12	-253.12	-302.79			0
65									
66	-3.34	-3.34	-3.34	-3.34	-3.34	-3.34	-3.34	-1.44	-4.3
67	5.25	18.62	31.04	42.5	53	63.51	73.07	84.53	3.34

DIAL GAUGE No.	DISPLACEMENT (mm)								
1	0	4.25	8.04	11.47	14.94	18.32	21.77	25.27	0.86
2	0	4.52	8.98	13.11	17.48	21.96	26.6	32.01	0.22
3	0	3.45	6.52	9.51	12.5	15.46	18.41	21.45	0.51
4I	0	-0.04	-0.09	-0.13	-0.17	-0.21	-0.25	-0.29	0
5P	0	-0.02	-0.05	-0.07	-0.1	-0.13	-0.15	-0.18	0
6a	0	5.36	10.19	14.68	19.2	23.68	28.17	32.81	0.88

TEST NO 8: CLAMP POSITION 37.5/62.5, OPB (STUDBOLTS HORIZONTAL) STUDBOLT PRELOAD 180 kN.

MOMENT (kNm)	0	20	40	60	80	100	120	140	0
Studbolt	Studbolt Load Variation (kN)								
BOLT A	0.285618	-6.24993	-12.162657	-17.919543	-23.572716	-29.225889	-35.034903	-41.673621	-1.789185
BOLT B	0.3255	0.225225	0.024675	-0.17535	-0.4263	-0.4767	-0.777525	-1.579725	-0.777525
BOLT C	0.32829	4.62783	8.572605	12.51738	16.462155	20.559426	24.756773	28.904346	0.1265505
BOLT D	0.226083	4.505323	8.380881	12.257493	16.032394	20.109793	24.237784	28.41584	0.579173
BOLT E	0.3022575	3.0231025	5.5930825	8.061255	10.58165	13.201743	15.770668	18.592265	0.80602
BOLT F	0.358048	1.792384	3.072352	4.300328	5.631752	6.809344	7.936016	9.266904	0.717168
BOLT G	0.302544	-0.554928	-1.613568	-2.370192	-3.327984	-4.236144	-5.245152	-6.505488	0
BOLT H	0.384134	-4.6884735	-9.1982925	-13.554136	-18.16589	-22.470766	-27.08252	-21.83555	-1.153475
BOLT I	0.425775	7.0455	13.2636	19.5321	26.051025	32.56995	39.439575	45.256575	-1.9803
BOLT J	0.354708	0.607995	0.912789	1.166607	1.521315	1.877085	2.231262	2.586501	0.050445
BOLT K	0.409886	-3.6895105	-7.4814925	-11.172076	-14.963522	-18.704	-22.59899	-26.544947	-0.051504
BOLT L	0.34038	-3.74967	-7.524594	-11.300616	-15.128793	-18.799407	-22.731894	-26.66493	0.288225
BOLT M	0.307164	-2.359578	-4.872738	-7.43745	-10.002162	-12.617889	-15.387735	-18.003999	0.512835
BOLT N	0.35404	-0.96195	-2.07548	-3.34112	-4.65658	-5.87187	-7.23874	-8.50491	0.65879
BOLT O	0.409122	1.483335	2.4552675	3.528945	4.5522855	5.7293145	6.8544	7.825797	0.3582495
BOLT P	0.30316	5.41713	10.02336	14.68047	19.33811	24.0461	28.95655	33.05716	-1.11406
Strain Gauge No.	STRAIN (μStrain)								
33									
34									
35									
36	-0.48	4.29	7.16	11.93	18.62	27.22	50.14	92.17	58.74
37	2.86	-54.44	-112.7	-174.8	-240.7	-313.3	-395.44	-632.32	-177.66
38									
39									
40	8.12	2.38	-1.43	-6.21	-10.99	-15.76	-20.54	-25.31	10.03
41	5.74	-21.96	-45.84	-69.73	-93.61	-116.53	-139.45	-161.42	7.64
42	6.69	6.69	6.69	5.73	4.78	4.78	2.87	0	8.6
43	6.21	10.03	13.85	16.71	19.58	22.44	24.36	31.04	12.89
44									
45	-1.44	22.44	43.46	66.38	88.35	111.28	134.2	155.21	-3.34
46	5.73	58.26	106.98	153.77	202.49	252.16	305.65	352.45	0.95
47	6.69	105.07	192.94	282.72	374.42	464.21	561.63	640.91	-14.33
48	7.16	141.84	260.28	379.68	489.52	597.46	715.9	1146.68	463.74
49	5.26	160.95	299.45	437.94	579.31	716.85	864.9	994.81	-12.89
50									
51	-2.86	-13.37	-23.88	-34.38	-43.93	-53.49	-63.04	-69.73	4.78
52									
53	-2.39	-55.88	-104.59	-155.22	-207.76	-258.38	-313.77	-368.22	-2.39
54	6.22	6.22	7.16	6.22	6.22	6.22	6.22	5.26	7.16
55	6.21	7.16	8.12	8.12	9.07	9.07	10.03	12.89	11.94
56	6.69	3.82	0.96	-2.86	-6.69	-10.51	-13.37	-20.06	3.82
57	5.26	1.44	-2.38	-7.16	-10.99	-14.8	-17.67	-16.71	11.94
58	4.78	0	-3.82	-8.59	-12.41	-16.23	-21.01	-25.78	5.74
59	3.34	-89.31	-175.28	-259.33	-345.3	-431.26	-522.96	-615.61	-6.22
60	4.29	-112.23	-216.35	-319.51	-423.62	-524.86	-630.89	-714.95	32.95
61	2.86	-133.72	-255.98	-377.29	-501.46	-622.77	-752.67	-884.48	-8.59
62	4.78	3.82	2.86	2.86	1.92	0.96	0.96	0	5.73
63	3.82	2.86	2.86	2.86	2.86	2.86	2.86	6.69	11.46
64	-0.48	-11.94	-21.49	-27.22	-37.73	-53.97	-63.52	-73.07	0.48
65									
66	-1.43	-0.47	0.48	2.39	3.35	5.26	8.12	10.99	-0.47
67	6.69	10.51	12.41	15.29	17.19	20.06	22.92	26.74	11.46

DIAL GAUGE No.	DISPLACEMENT (mm)								
1	0	4.1	7.69	11.12	14.64	18.07	21.84	26.02	1.11
2	0	4.93	9.3	13.77	18.24	22.62	27.37	32.59	0.99
3	0	3.23	6.15	9.15	12.17	15.07	18.3	21.74	0.55
4I	0	-0.06	-0.1	-0.14	-0.19	-0.24	-0.29	-0.34	0
5P	0	-0.04	-0.08	-0.12	-0.16	-0.2	-0.24	-0.28	0
6a	0	5.01	9.53	13.9	18.38	22.82	27.64	32.96	1.2

TEST NO 9:CLAMP POSITION 37.5/62.5, IPB (STUDBOLTS VERTICAL), STUDBOLT PRELOAD 180kN.

MOMENT (kNm)	0	20	40	60	80	100	120	140	0
Studbolt	Studbolt Load Variation (kN)								
BOLT A	-0.0304	-0.0304	0.0607	0.1214	0.2733	0.3340	0.4554	0.6072	-0.3340
BOLT B	0.0000	-0.0605	-0.0605	0.0000	0.0907	0.2117	0.4537	0.9376	-0.1512
BOLT C	-0.0610	-0.1219	-0.1219	0.0305	0.2743	0.6095	1.1581	2.1639	-0.0914
BOLT D	0.0000	0.0000	0.1217	0.3650	0.6996	1.1255	1.7339	2.5248	-0.0304
BOLT E	0.0000	-0.0302	0.0302	0.1511	0.3627	0.5743	0.8463	1.2695	0.0000
BOLT F	0.0000	0.0303	0.1517	0.3034	0.4248	0.6068	0.7586	0.9103	0.0000
BOLT G	0.0000	-0.0918	-0.1224	-0.1530	-0.1836	-0.2448	-0.3365	-0.4283	-0.0612
BOLT H	-0.0307	0.0307	0.1228	0.1535	0.2455	0.2762	0.3069	0.3376	-0.0614
BOLT I	-0.0610	0.0000	0.0000	0.0610	0.1220	0.1830	0.3355	0.4880	-0.3050
BOLT J	0.0000	0.0303	0.1513	0.3632	0.6053	0.8172	1.1803	1.8159	-0.1211
BOLT K	-0.0307	-0.0307	0.1229	0.3688	0.6761	1.0756	1.7825	2.8581	-0.0922
BOLT L	-0.0613	0.0000	0.2454	0.5521	0.9202	1.4723	2.1471	3.1287	-0.0613
BOLT M	-0.0610	-0.0610	0.0000	0.0915	0.2135	0.3660	0.6099	0.9149	-0.0305
BOLT N	-0.0616	-0.0616	-0.0308	0.0000	0.0000	0.0616	0.0924	0.1849	-0.0616
BOLT O	-0.0615	-0.1229	-0.1537	-0.1844	-0.2151	-0.3073	-0.3381	-0.3995	-0.0922
BOLT P	-0.0311	0.0000	0.0311	0.0311	0.0622	0.0933	0.0622	0.0622	-0.0933
Strain Gauge No.	STRAIN (μstrain)								
33	-0.96	-1.91	-3.82	-6.69	-8.60	-11.46	-15.28	-16.24	-0.96
34	-0.96	-0.96	-2.87	-4.78	-9.55	-15.28	-22.92	-33.43	-0.96
35	0.00	-4.78	-10.51	-16.24	-21.97	-29.61	-36.30	-44.89	0.00
36	0.00	0.00	30076.29	30063.87	0.00	30036.17	0.00	0.00	0.00
37									
38	0.00	-0.96	-1.91	-2.87	-3.82	-5.73	-6.69	-6.69	-0.96
39	-0.96	-4.78	-10.51	-16.24	-21.97	-29.61	-37.25	-44.89	0.96
40	0.00	-21.01	-38.21	-48.71	-55.40	-61.13	-30764.01	-30764.01	-1.91
41	0.00	-100.29	-192.94	-282.73	-297.06	-239.75	-208.23	-219.69	-2.87
42	-1.91	-13.37	-196.76	-134.68	54.44	-358.19	-353.41	-355.32	29.61
43	-0.96	-4.78	-68.77	-48.71	17.19	-46.80	-5.73	43.94	11.46
44	0.00	0.00	-0.96	-2.87	-4.78	-7.64	-11.46	-17.19	0.00
45	0.00	-2.87	-6.69	-11.46	-17.19	-22.92	-30.57	-38.21	0.00
46	-0.96	-0.96	-0.96	-1.91	-2.87	-4.78	-7.64	-9.55	-0.96
47	-0.96	0.96	1.91	2.87	3.82	3.82	3.82	3.82	-0.96
48	-0.96	0.96	1.91	2.87	2.87	3.82	3.82	4.78	-0.96
49	0.00	-1.91	-4.78	-6.69	-9.55	-12.42	-15.28	-18.15	0.00
50	-0.96	-9.55	-19.10	-28.65	-39.16	-51.58	-65.91	-83.10	-0.96
51	0.00	-9.55	-19.10	-28.66	-39.16	-49.67	-61.13	-73.55	0.00
52	0.00	0.00	0.00	31165.18	0.00	0.00	0.00	0.00	0.00
53	-0.96	1.91	4.78	6.69	9.55	13.37	16.24	20.06	-0.96
54	0.00	12.42	21.97	32.48	40.12	48.71	58.27	-30764.01	0.00
55	0.00	49.67	105.07	161.42	214.91	270.31	323.80	374.43	-10.51
56									
57	-0.96	135.63	260.76	381.11	499.55	617.99	736.43	849.14	0.00
58	0.00	-0.96	-0.96	-0.96	0.00	0.00	-0.96	-0.96	0.00
59	-0.96	-0.96	-1.91	-3.82	-5.73	-7.64	-9.55	-10.51	-0.96
60	0.00	-4.78	-9.55	-15.28	-21.01	-26.74	-32.48	-37.25	0.00
61	0.00	-6.69	-13.37	-21.01	-29.61	-36.30	-44.89	-51.58	0.00
62	0.00	-8.60	-16.24	-22.92	-30.57	-38.21	-47.76	-57.31	0.96
63	-0.96	-62.09	-118.44	-172.89	-228.28	-31905.44	-31905.44	-31905.44	-0.96
64	0.00	-109.84	-212.05	-312.34	-408.81	-509.10	-608.44	-704.91	-0.96
65	0.00	-124.17	-235.93	-347.68	-454.66	-565.46	-675.30	-781.33	-0.96
66	0.00	-3.82	-8.60	-18.15	-33.43	-54.44	-83.10	-111.75	-3.82
67	0.00	20.06	38.21	57.31	74.50	91.70	108.89	126.08	0.96

DIAL GAUGE No.	DISPLACEMENT (mm)								
1	0.00	2.77	4.95	6.89	8.72	10.53	12.31	14.03	0.05
2	0.00	4.37	8.00	11.41	14.67	17.97	21.23	24.44	0.05
3	0.00	2.76	4.84	6.76	8.63	10.48	12.26	14.04	0.02
4I	0.00	0.00	0.00	0.00	0.00	0.01	0.01	0.02	0.00
5P	0.00	0.00	0.01	0.01	0.02	0.02	0.02	0.03	0.00
6	0.00	4.19	7.63	10.89	13.98	17.11	20.21	23.20	0.07

TEST NO 10: CLAMP POSITION 37.5/62.5, OPB (STUDBOLTS HORIZONTAL),
STUDBOLT PRELOAD 180kN.

MOMENT (kNm)	0	20	40	60	80	100	120	140	0
Studbolt	Studbolt Load Variation (kN)								
BOLT A	-0.0304	-4.3113	-8.2279	-12.0534	-15.7575	-19.1883	-22.4977	-25.8071	-0.43
BOLT B	-0.0303	-0.2420	-0.3932	-0.5444	-0.5444	-0.4537	-0.3025	-0.0605	-0.18
BOLT C	-0.0610	2.8344	5.6688	8.5947	11.6729	14.9645	18.4694	22.2181	0.15
BOLT D	-0.0608	3.0419	5.9926	9.0041	12.1981	15.6355	19.1338	22.8753	0.30
BOLT E	-0.0302	2.0252	3.9597	5.9849	8.0101	10.1562	12.4232	14.7507	0.24
BOLT F	-0.0303	0.8496	1.6688	2.5184	3.3376	4.1872	5.0368	6.0077	0.06
BOLT G	0.0306	-1.0402	-1.9581	-2.8759	-3.8243	-4.8034	-5.7518	-6.6390	-0.15
BOLT H	0.0000	-3.6218	-6.9059	-10.0980	-13.2594	-16.3287	-19.3980	-22.5901	-0.12
BOLT I	-0.0305	4.4526	8.7831	13.1442	17.6578	22.3238	27.0814	32.0524	-0.12
BOLT J	-0.0908	0.3632	0.8777	1.4527	2.0278	2.7844	3.6318	4.6003	-0.06
BOLT K	-0.0307	-2.5201	-4.7636	-6.7919	-8.6358	-10.2032	-11.5554	-12.8155	-0.15
BOLT L	-0.0307	-2.6993	-5.1224	-7.2696	-9.3554	-11.2264	-12.9135	-14.5085	-0.21
BOLT M	0.0000	-2.0433	-3.9951	-5.7944	-7.5633	-9.2406	-10.8874	-12.5343	-0.12
BOLT N	0.0000	-1.0476	-2.0028	-2.8347	-3.7590	-4.5602	-5.3921	-6.2548	0.03
BOLT O	-0.0615	0.7683	1.5981	2.3357	3.1654	4.0567	4.9787	5.9006	0.12
BOLT P	-0.0622	3.4224	6.6893	9.9561	13.1297	16.4587	19.7878	23.0547	-0.03
Strain Gauge No.	STRAIN (μstrain)								
33	-0.96	17.19	34.39	49.67	67.82	86.92	106.02	127.04	2.87
34	-0.96	61.13	119.40	174.80	232.11	288.46	347.68	410.72	2.87
35	-0.96	105.07	198.67	290.37	380.16	466.12	549.22	628.50	-6.69
36	0.00	0.00	0.00	0.00	30480.33	0.00	0.00	0.00	0.00
37									0.00
38	0.00	-19.10	-35.34	-50.62	-64.00	-73.55	-82.14	-90.74	-4.78
39	-0.96	-92.65	-176.71	-255.03	-334.31	-409.77	-482.36	-546.36	7.64
40	0.00	-0.96	-1.91	-3.82	-3.82	-4.78	-5.73	-5.73	0.00
41	0.96	-289.42	-213.00	-247.39	-447.97	-594.11	-598.89	-491.91	-301.83
42	-9.55	-251.21	-173.84	-207.27	-570.23	-574.06	-577.88	-581.70	-184.35
43									
44	-0.96	3.82	8.60	12.42	16.24	20.06	23.88	27.70	0.96
45	0.00	20.06	40.12	58.27	76.41	95.52	112.71	129.90	0.95
46	-0.96	43.94	88.83	131.81	174.80	217.78	260.76	303.74	3.82
47	-0.96	86.92	168.11	248.34	328.58	407.86	490.00	573.10	5.73
48	0.00	110.80	216.82	315.21	415.50	514.83	613.22	713.51	0.00
49	0.00	135.63	259.81	382.07	501.46	621.81	740.25	862.52	1.91
50	0.00	4.78	8.60	12.42	16.24	20.06	22.92	24.83	5.73
51	-0.96	-13.37	-25.79	-37.25	-51.58	-64.95	-79.28	-94.56	0.96
52									
53	0.00	-45.85	-87.88	-128.95	-171.93	-213.96	-256.94	-299.92	0.96
54	-1.91	-1.91	-2.87	-2.87	-2.87	-1.91	0.00	1.91	-0.96
55	-0.96	-5.73	-9.55	-14.33	-18.15	-22.92	-17.19	-21.97	-0.96
56	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
57	0.00	-14.33	-27.70	-41.07	-55.40	-67.82	-81.19	-93.61	0.00
58	0.00	-6.69	-12.42	-18.15	-22.92	-26.74	-30.57	-33.43	0.96
59	0.00	-87.88	-172.89	-254.07	-335.26	-416.45	-496.69	-579.79	-2.87
60	-0.96	-106.02	-203.45	-298.97	-392.57	-484.27	-577.88	-670.53	-2.87
61	-0.96	-126.08	-245.48	-360.10	-473.76	-586.47	-697.27	-810.94	0.00
62	-0.96	0.00	0.96	0.96	1.91	3.82	4.78	4.78	0.96
63	-31956.1	2.87	4.78	6.69	9.55	12.42	13.37	15.28	-31956.1
64	-0.96	1.91	2.87	3.82	4.78	6.69	6.69	7.64	0.96
65	0.00	0.96	0.96	0.96	0.96	1.91	2.87	2.87	0.96
66	-0.96	2.87	3.82	8.60	14.33	21.01	28.65	36.30	0.00
67	0.00	0.96	1.91	2.87	3.82	5.73	6.69	7.64	0.00

DIAL GAUGE No.	DISPLACEMENT (mm)								
1	0.00	2.58	4.46	6.17	7.82	9.45	11.08	12.76	0.02
2	0.00	4.20	7.39	10.45	13.48	16.49	19.48	22.56	0.02
3	0.00	2.86	4.75	6.50	8.23	10.00	11.66	13.41	0.00
4I	0.00	-0.07	-0.15	-0.23	-0.31	-0.39	-0.47	-0.56	-0.11
5P	0.00	-0.06	-0.12	-0.18	-0.24	-0.30	-0.35	-0.41	-0.09
6	0.00	3.69	6.70	9.57	12.38	15.14	17.90	20.74	0.04

TEST NO 11: CLAMP POSITION 37.5/62.5, IPB (STUDBOLTS VERTICAL), STUDBOLT PRELOAD 180kN.

MOMENT (kNm)	0	20	40	60	80	100	120	140	0
Studbolt	Studbolt Load Variation (kN)								
BOLT A	0.0000	-0.0304	-0.1214	-0.1214	-0.1214	-0.1214	0.0304	0.2125	-0.1214
BOLT B	-0.0302	-0.0605	-0.1512	-0.2420	-0.3025	-0.3327	-0.3327	-0.3025	-0.1210
BOLT C	0.0000	-0.0610	-0.1524	-0.1829	-0.2438	-0.3048	-0.3048	-0.3657	-0.1524
BOLT D	0.0000	-0.0608	-0.0913	-0.1521	-0.1521	-0.1521	-0.1825	-0.2129	-0.0913
BOLT E	0.0000	-0.0302	-0.0302	-0.0605	-0.1209	-0.0907	-0.0907	-0.0907	0.0000
BOLT F	0.0000	-0.0303	-0.0607	-0.1214	-0.1820	-0.1820	-0.1820	-0.1820	-0.0607
BOLT G	-0.0306	-0.0306	-0.1224	-0.1530	-0.1836	-0.2142	-0.1836	-0.2142	-0.0918
BOLT H	0.0000	-0.0614	-0.1535	-0.1228	-0.0921	-0.0614	0.0614	0.1842	-0.0614
BOLT I	0.0000	-0.0610	-0.0610	0.0305	0.1525	0.3355	0.5794	0.8844	-0.3050
BOLT J	0.0000	-0.0908	-0.1211	-0.1211	-0.0908	-0.0908	-0.0303	0.0303	-0.2421
BOLT K	0.0000	-0.0615	-0.0922	-0.1229	-0.1537	-0.1844	-0.1537	-0.1537	-0.1844
BOLT L	0.0000	-0.0307	-0.0613	-0.0613	-0.0920	-0.1227	-0.1227	-0.1534	-0.1227
BOLT M	0.0000	-0.0305	-0.0305	-0.0915	-0.0915	-0.1220	-0.1525	-0.1525	-0.0915
BOLT N	-0.0308	-0.0924	-0.0924	-0.1541	-0.1541	-0.1849	-0.2157	-0.2465	-0.1232
BOLT O	-0.0307	-0.0922	-0.1537	-0.1844	-0.2151	-0.1844	-0.1537	-0.1537	-0.1844
BOLT P	0.0000	-0.0933	-0.0933	-0.0622	-0.0311	0.0622	0.2489	0.4356	-0.3111
Strain Gauge No.	STRAIN (μstrain)								
33	0.96	-15.28	2.87	9.55	12.42	7.64	-11.46	-8.60	-2.87
34	2.87	-42.03	-10.51	0.00	6.69	3.82	-12.42	7.64	6.69
35	-0.96	-7.64	-9.55	-8.60	-7.64	-12.42	-19.10	-21.01	-8.60
36	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
37	0.00	0.00	0.00	0.00	28388.51	0.00	0.00	28343.62	0.00
38	-0.96	-42.03	-43.94	-68.77	11.46	-65.91	-97.43	9.55	-24.83
39									
40									
41									
42	-9.55	-251.21	31.52	63.04	95.52	23.88	-208.23	42.03	-230.19
43	-2.87	-155.69	-36.30	10.51	67.82	87.88	82.14	-24121.78	-84.06
44	0.00	358.19	219.69	75.46	-20.06	-92.65	-89.79	-98.38	-86.92
45	-3.82	7.64	1.91	-7.64	-14.33	-20.06	-16.24	-25.79	3.82
46	7.64	-31.52	22.92	44.89	62.09	59.22	31.52	59.22	52.53
47	0.00	0.96	3.82	6.69	9.55	14.33	20.06	24.83	0.00
48	0.96	-6.69	5.73	18.15	39.16	55.40	65.91	85.01	0.96
49	0.00	0.00	2.87	4.78	5.73	5.73	7.64	8.60	0.00
50	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
51	1.91	-38.21	-4.78	13.37	9.55	3.82	-27.70	-11.46	-9.55
52	0.00	24734.04	24251.68	0.00	0.00	0.00	0.00	0.00	24672.91
53	0.00	1.91	4.78	8.60	10.51	14.33	17.19	21.01	0.00
54									
55	0.00	0.00	0.00	0.00	0.00	0.00	25822.93	25734.10	0.00
56	3.82	-47.76	72.59	72.59	24.83	38.21	79.28	141.37	-14.33
57	0.00	130.86	254.07	368.69	488.09	608.44	724.97	844.37	0.96
58	0.96	0.96	0.96	0.00	0.00	-0.96	0.00	2.87	-1.91
59	0.00	-0.96	-0.96	-3.82	-9.55	-18.15	-27.70	-44.89	-5.73
60	0.00	-5.73	-11.46	-17.19	-21.97	-27.70	-32.48	-32.48	3.82
61	0.00	-10.51	-22.92	-34.39	-46.80	-60.18	-70.68	-82.14	-0.96
62	34.38	1284.70	880.66	320.94	-76.41	-433.64	-471.85	-416.45	-307.56
63									
64	4.78	-55.40	-92.65	-137.54	-184.35	-273.18	-420.27	-613.22	-39.16
65	0.00	-115.58	-228.28	-331.44	-439.38	-548.27	-651.42	-760.31	0.00
66	1.91	4.78	12.42	15.28	15.28	14.33	12.42	11.46	-9.55
67	4.78	-11.46	14.33	21.97	27.70	23.88	-2.87	0.96	-15.28

DIAL GAUGE No.	DISPLACEMENT (mm)								
1	0.00	2.21	3.99	5.45	6.87	8.27	9.61	11.00	0.02
2	0.00	3.20	5.89	8.30	10.74	13.22	15.58	18.06	0.07
3	0.00	2.30	4.03	5.55	7.06	8.61	10.05	11.58	0.02
4I	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
5P	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	-0.01
6	0.00	3.10	5.71	8.01	10.33	12.68	14.92	17.26	0.07

TEST NO 12: CLAMP POSITION 37.5/62.5, OPB (STUDBOLTS HORIZONTAL), STUDBOLT PRELOAD 180kN.

MOMENT (kNm)	0	20	40	60	80	100	120	140	0
Studbolt	Studbolt Load Variation (kN)								
BOLT A	0.0000	-0.1518	-0.2733	-0.3340	-0.2733	-0.0607	0.1518	0.3643	0.5769
BOLT B	0.0000	-0.2117	-0.3327	-0.5747	-0.7864	-1.0284	-1.3611	-1.6333	0.2117
BOLT C	0.0000	-0.3657	-0.7010	-1.1277	-1.5544	-1.9201	-2.3468	-2.7734	0.0000
BOLT D	0.0000	-0.4867	-1.0343	-1.5514	-2.0685	-2.6161	-3.1028	-3.6807	0.0000
BOLT E	0.0000	-0.5441	-1.0579	-1.5416	-2.0554	-2.5995	-3.1134	-3.6272	0.0000
BOLT F	0.0000	-0.4248	-0.7585	-1.1833	-1.6081	-2.0026	-2.4274	-2.8521	0.0000
BOLT G	0.0000	-0.1836	-0.3059	-0.4895	-0.7037	-0.9178	-1.1320	-1.3462	0.1836
BOLT H	0.0307	-0.1535	-0.3069	-0.3069	-0.1535	0.0921	0.3683	0.6446	0.4911
BOLT I	0.0000	0.3050	0.4880	0.4880	0.3355	0.0915	-0.1525	-0.5185	0.0000
BOLT J	0.0000	0.0605	0.2119	0.3329	0.4540	0.4540	0.5145	0.4237	0.0605
BOLT K	0.0000	0.1229	0.3073	0.5225	0.7069	0.9220	1.0756	1.2293	0.0615
BOLT L	0.0000	0.1227	0.2761	0.4294	0.5828	0.7975	0.9509	1.1042	0.0307
BOLT M	0.0610	0.2135	0.3965	0.5794	0.7929	1.0064	1.1894	1.3724	0.0000
BOLT N	0.0308	0.1541	0.2773	0.3697	0.5238	0.6779	0.8319	0.9244	0.0308
BOLT O	0.0615	0.0922	0.1537	0.2459	0.3073	0.3688	0.3688	0.2766	0.0615
BOLT P	0.0311	0.3422	0.5911	0.6223	0.4978	0.2178	-0.1244	-0.5911	0.1245
Strain Gauge No.	STRAIN (µstrain)								
33	3.82	25.79	50.62	64.00	78.32	90.74	110.80	107.93	31.52
34	5.73	46.80	98.38	144.23	213.96	247.39	375.38	466.12	61.13
35	-0.96	71.64	156.65	267.45	386.84	502.42	612.26	714.46	3.82
36									
37	0.00	0.00	0.00	0.00	0.00	28257.65	28273.89	28248.10	0.00
38	5.73	0.95	0.95	-171.93	-684.85	109.84	148.05	131.81	-401.17
39									
40	-34.39	-64.95	133.72	-3.82	-1268.46	13.37	1134.74	1616.14	-578.83
41	0.00	0.00	0.00	0.00	62595.90	0.00	0.00	0.00	0.00
42	9.55	67.82	359.14	322.85	-737.39	337.17	134.68	75.46	-72.59
43	14.33	52.53	127.99	127.99	-106.02	169.06	72.59	-148.05	51.58
44	-47.76	-200.59	-206.32	-346.73	-706.82	-537.76	-648.56	-874.93	-824.31
45	-1.91	10.51	26.74	32.48	0.96	64.95	20.06	-53.49	-73.55
46	14.33	61.13	125.13	155.69	128.95	220.64	192.94	128.95	130.86
47	-0.96	28.66	64.95	109.84	162.38	225.42	288.46	354.37	4.78
48	2.87	84.05	183.39	292.28	404.04	506.24	610.35	715.42	40.12
49	0.00	131.81	261.72	383.98	503.37	624.68	745.99	865.38	1.91
50									
51	4.78	9.55	21.97	21.97	20.06	13.37	4.78	4.78	38.21
52									
53	0.00	-41.07	-81.19	-119.40	-158.56	-199.63	-241.66	-282.73	1.91
54									
55	11.46	26.74	67.82	77.37	85.01	80.23	78.32	85.97	63.04
56	21.01	78.32	164.29	191.99	205.36	199.63	202.49	227.33	192.94
57	0.00	-16.24	-32.48	-44.89	-57.31	-69.73	-83.10	-94.56	-0.96
58									
59	-0.96	-38.21	-87.88	-139.45	-192.94	-251.21	-309.47	-362.96	99.34
60	-0.96	-77.37	-161.42	-246.43	-331.44	-419.32	-508.15	-594.11	6.69
61	0.00	-113.66	-230.20	-340.04	-451.79	-567.37	-682.94	-796.61	0.96
62	-217.78	-759.36	-563.55	-1007.70	-1308.58	-1646.71	-1917.97	-2057.43	-2345.89
63	4.78	24.83	55.40	66.86	78.32	79.28	83.10	89.79	61.13
64	1.91	2.87	5.73	7.64	16.24	22.92	26.74	22.92	24.83
65	0.00	-1.91	-4.78	-4.78	-4.78	-3.82	-2.87	-2.87	0.00
66	0.96	3.82	6.69	7.64	7.64	7.64	8.60	9.55	9.55
67	0.96	5.73	29.61	40.12	45.85	44.89	51.58	72.59	44.89

DIAL GAUGE No.	DISPLACEMENT (mm)								
1	0.00	2.07	3.76	5.30	6.78	8.27	9.71	11.13	0.00
2	0.00	2.87	5.62	8.18	10.70	13.25	15.81	18.33	0.13
3	0.00	2.45	4.19	5.75	7.29	8.84	10.40	11.93	0.07
4I	0.00	0.00	0.00	0.00	0.00	0.00	0.00	-0.01	0.00
5P	0.00	0.00	0.01	0.01	0.01	0.01	0.00	0.00	0.00
6	0.00	3.04	5.66	8.12	10.52	12.98	15.41	17.80	0.07

TEST NO 13: CLAMP POSITION 37.5/62.5, IPB (STUDBOLTS VERTICAL), STUDBOLT PRELOAD 180kN.

MOMENT (kNm)	0	20	40	60	80	100	120	140	0
Studbolt	Studbolt Load Variation (kN)								
BOLT A	0.0000	-0.0517	-0.1552	-0.2069	-0.4138	-0.4138	-0.4655	-0.5689	-0.3621
BOLT B	0.0000	-0.1000	-0.1500	-0.2500	-0.4000	-0.5500	-0.7000	-0.8000	-0.2000
BOLT C	0.0506	-0.1011	-0.1011	-0.1517	-0.3034	-0.4046	-0.5058	-0.6575	-0.2529
BOLT D	0.1006	0.0000	0.0000	-0.0503	-0.1509	-0.2514	-0.3017	-0.3520	-0.1509
BOLT E	0.0000	-0.0505	-0.0505	-0.1009	-0.2018	-0.2522	-0.3026	-0.3531	-0.1513
BOLT F	0.0513	-0.1026	-0.1539	-0.2052	-0.3078	-0.3078	-0.4103	-0.4616	-0.1539
BOLT G	-0.0506	-0.0506	-0.2023	-0.3035	-0.3540	-0.4552	-0.5058	-0.6069	-0.2023
BOLT H	0.0513	0.0000	-0.1027	-0.1540	-0.2567	-0.2567	-0.2567	-0.2567	-0.3080
BOLT I	0.0501	0.0000	-0.0501	0.0000	0.0000	0.1503	0.3006	0.4008	-0.1503
BOLT J	0.0000	0.0000	-0.1014	-0.1522	-0.1522	-0.1522	-0.1014	-0.2029	-0.1522
BOLT K	0.0511	0.1021	0.0000	0.0000	-0.0510	-0.1021	-0.1021	-0.2553	-0.1021
BOLT L	0.0000	-0.1048	-0.1048	-0.1048	-0.1572	-0.2096	-0.2096	-0.2620	-0.1572
BOLT M	0.0512	-0.1025	0.0000	-0.1025	-0.1025	-0.1537	-0.1537	-0.2562	-0.1537
BOLT N	0.0000	-0.0505	-0.1516	-0.2021	-0.2526	-0.3537	-0.3537	-0.5053	-0.2021
BOLT O	0.0508	-0.1016	-0.2033	-0.2033	-0.3049	-0.3049	-0.3049	-0.4065	-0.2033
BOLT P	0.0502	0.0000	-0.0502	-0.0502	0.0502	0.0502	0.1507	0.1507	-0.4019
Strain Gauge No.	STRAIN (μstrain)								
33	11.462	15.283	30.565	41.072	49.669	54.445	65.906	70.682	58.265
34	22.924	25.790	77.369	130.858	189.123	356.277	451.794	461.346	93.606
35	11.462	15.283	21.969	22.924	22.924	21.014	22.924	24.834	42.027
36	-10.507	-54.445	-64.952	-156.647	-285.595	-309.474	-369.650	-468.032	-375.380
37	74.503	117.486	169.065	196.764	221.599	234.016	259.805	274.133	286.550
38	19.103	27.700	42.982	45.848	41.072	18.148	12.417	2.865	16.237
39									
40									
41	-23267.861	-27316.814	-27316.814	-27316.814	-27316.814	-27316.814	-27316.814	-27316.814	-27316.814
42	1196.822	2153.898	2683.063	2718.404	2695.479	2803.414	2576.084	2799.592	3809.205
43	178.617	282.729	414.542	458.480	541.579	382.067	410.722	407.856	338.129
44	-64.951	-171.930	-298.012	-434.601	-561.638	-641.872	-722.106	-792.788	-840.547
45	-46.803	-51.579	-120.351	-170.020	-208.226	-198.675	-221.599	-234.016	-272.223
46	-1.910	17.193	50.624	49.669	44.893	28.655	42.027	48.714	69.727
47	0.955	0.955	2.866	5.731	8.597	10.507	13.372	16.238	1.910
48	10.507	17.193	25.790	31.521	40.117	50.624	62.086	74.503	17.193
49	0.000	0.955	1.910	5.731	7.641	8.597	10.507	11.462	1.910
50									
51									
52	189.121	33.430	153.783	-42.029	-42.029	-42.029	81.188	-42.029	-42.029
53	0.955	3.821	4.776	8.596	12.417	16.238	19.103	22.924	1.910
54									
55	169.065	297.057	410.721	445.108	408.812	170.019	119.396	28.655	-5.732
56									
57	0.000	131.813	254.074	371.560	490.956	606.531	725.927	844.367	2.865
58	6.686	0.955	3.821	-1.910	-6.686	-20.058	-23.879	-29.610	-30.565
59	16.238	60.175	70.682	78.323	83.099	76.413	85.965	115.575	174.795
60	4.776	7.641	4.776	-1.910	-19.103	-55.400	-75.458	-99.337	-56.355
61	0.000	-11.462	-20.058	-31.521	-43.938	-57.310	-69.727	-83.099	1.910
62	702.046	-213.958	-747.896	-1782.342	-2800.551	-2752.793	-2402.246	-2141.484	-2387.918
63	0.000	0.000	0.000	0.000	0.000	0.000	19591.426	21639.305	0.000
64	98.382	108.889	125.127	96.471	48.713	-49.669	-102.203	-176.706	335.263
65	0.000	-115.575	-224.464	-333.353	-440.332	-543.490	-650.469	-755.537	0.000
66	4.776	15.283	8.597	-5.731	-28.655	-32.476	-42.982	-53.489	-49.669
67	87.875	155.692	204.406	220.643	239.747	233.060	271.267	312.339	294.191

DIAL GAUGE No.	DISPLACEMENT (mm)								
1	0.00	2.09	3.83	5.29	6.69	8.03	9.36	10.68	0.20
2	0.00	3.18	5.79	8.20	10.58	12.92	15.25	17.60	0.10
3	0.00	2.31	4.01	5.55	7.04	8.52	9.98	11.49	0.07
4I	0.00	0.00	0.00	0.01	0.01	0.01	0.01	0.01	0.00
5P	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01
6	0.00	3.07	5.63	7.97	10.26	12.49	14.68	16.04	0.06

TEST NO 14: CLAMP POSITION 37.5/62.5, OPB (STUDBOLTS HORIZONTAL), STUDBOLT PRELOAD 180kN.

MOMENT (kNm)	0	20	40	60	80	100	120	140	0
Studbolt	Studbolt Load Variation (kN)								
BOLT A	0.0000	0.2586	0.5172	0.6724	0.7241	0.7241	0.6724	0.5172	-0.4138
BOLT B	-0.0500	0.1000	0.2000	0.3000	0.4500	0.6500	0.9001	1.2001	0.0000
BOLT C	-0.0506	0.3540	0.8092	1.3150	1.7196	2.2254	2.7311	3.1863	0.0506
BOLT D	-0.0503	0.6035	1.3075	1.9110	2.6151	3.2688	3.9729	4.6769	0.1006
BOLT E	0.1009	0.8574	1.4121	2.1182	2.7738	3.4294	4.0851	4.6902	0.2018
BOLT F	-0.0513	0.3590	0.8207	1.2310	1.6926	2.2056	2.6159	3.0775	0.0000
BOLT G	-0.0506	0.0000	0.1012	0.2023	0.3540	0.5563	0.7081	0.8598	-0.1012
BOLT H	-0.0513	0.2054	0.4621	0.5134	0.4621	0.3080	0.0000	-0.3594	-0.5134
BOLT I	-0.0501	-0.4509	-0.7515	-0.9018	-0.9018	-0.8016	-0.7014	-0.6012	-0.0501
BOLT J	-0.0507	-0.1522	-0.3550	-0.5579	-0.7608	-0.9637	-1.1158	-1.2173	-0.0507
BOLT K	0.0000	-0.3574	-0.8169	-1.1742	-1.5827	-2.0421	-2.5016	-2.9101	0.0000
BOLT L	-0.0524	-0.3667	-0.7335	-1.1526	-1.5193	-1.9385	-2.2528	-2.6719	0.0524
BOLT M	0.0000	-0.5124	-0.9737	-1.4861	-1.9473	-2.4085	-2.8697	-3.3822	0.0000
BOLT N	-0.0505	-0.3537	-0.6063	-0.9095	-1.2632	-1.5159	-1.8190	-2.1727	0.0000
BOLT O	0.0000	-0.1524	-0.3557	-0.5590	-0.7114	-0.8639	-1.0163	-1.1687	0.0000
BOLT P	-0.0502	-0.5024	-0.8039	-0.9546	-1.0048	-0.9044	-0.7034	-0.6029	-0.2512
ain Gauge No.	STRAIN (μstrain)								
33	0.96	17.19	27.70	38.21	49.67	64.95	81.19	92.65	16.24
34	0.00	12.42	37.25	64.95	78.32	103.16	165.24	203.45	13.37
35	1.91	77.37	149.96	228.28	322.85	433.65	563.55	679.12	9.55
36	6.69	-64.95	-248.34	-315.21	-295.15	-359.14	-308.52	-353.41	-342.90
37	10.51	13.37	2.87	-17.19	-31.52	-46.80	-60.18	-72.59	67.82
38	3.82	6.69	5.73	7.64	17.19	32.48	49.67	58.27	21.97
39									
40									
41									
42	165.24	427.91	506.24	506.24	586.47	562.59	602.71	576.92	525.34
43	26.74	98.38	128.95	128.95	148.05	137.54	127.04	165.24	112.71
44	2.87	-84.05	-168.11	-290.37	-334.31	-414.54	-446.06	-504.33	-661.93
45	-19.10	-72.59	-99.34	-126.08	-141.36	-154.74	-167.15	-174.80	-304.70
46	2.87	29.61	40.12	44.89	56.35	68.77	85.97	100.29	-28.66
47	-1.91	28.65	60.18	98.38	145.19	196.76	252.16	307.56	2.87
48	0.00	73.55	149.96	239.75	345.77	453.70	549.22	655.24	0.96
49	0.96	136.59	264.58	388.75	512.92	637.10	759.36	883.53	2.87
50									
51									
52	0.00	0.00	0.00	0.00	5961.20	4415.73	3154.92	0.00	0.00
53	0.00	-42.03	-82.14	-122.26	-164.29	-205.36	-246.43	-288.46	1.91
54									
55	26.74	75.46	86.92	85.01	92.65	89.79	102.20	97.43	109.84
56									
57	0.00	-21.97	-41.07	-60.18	-78.32	-95.52	-111.75	-127.99	0.00
58	2.87	1.91	-1.91	-7.64	-11.46	-16.24	-28559.49	-28559.49	-28559.49
59	25.79	-112.71	-114.62	-233.06	-297.06	-298.01	-281.77	-236.88	-11.46
60	0.95	-72.59	-144.23	-226.37	-306.61	-390.66	-472.81	-552.09	10.51
61	-0.96	-119.40	-231.15	-343.86	-456.57	-572.14	-684.85	-802.34	1.91
62	187.21	133.72	-52.54	-483.32	-559.73	-792.79	-751.72	-928.42	-1341.06
63	21.97	71.64	93.61	91.70	103.16	106.98	117.49	118.44	127.99
64	16.24	51.58	63.04	62.09	66.86	64.00	73.55	71.64	93.61
65	0.00	0.00	0.96	2.87	6.69	11.46	15.28	19.10	0.00
66	0.96	-4.78	-10.51	-18.15	-20.06	-22.92	-23.88	-26.74	-32.48
67	15.28	38.21	39.16	28.66	33.43	42.03	56.35	63.04	78.32

DIAL GAUGE No.	DISPLACEMENT (mm)								
1	0.00	2.05	3.59	5.03	6.40	7.76	9.12	10.49	0.02
2	0.00	3.22	5.80	8.26	10.71	13.18	15.64	18.13	0.06
3	0.00	2.49	4.20	5.83	7.43	9.05	10.63	12.25	0.07
4I	0.00	0.00	0.00	0.01	0.01	0.01	0.01	0.01	-0.01
5P	0.00	0.00	0.00	0.01	0.01	0.01	0.01	0.01	-0.01
6	0.00	3.14	5.65	8.04	10.40	12.77	15.13	17.52	0.07

TEST NO 15: CLAMP POSITION 37.5/62.5, IPB (STUDBOLTS VERTICAL)
STUDBOLT PRELOAD 180 kN.

MOMENT (kNm)	0	20	40	60	80	100	120	140	0
Studbolt	Studbolt Load Variation (kN)								
BOLT A									
BOLT B	0.100	0.100	0.000	-0.100	-0.050	0.000	0.100	0.400	0.500
BOLT C	0.051	-0.051	-0.101	-0.202	-0.202	-0.253	-0.253	-0.152	0.101
BOLT D	0.101	0.151	0.050	0.000	0.000	0.000	0.000	0.000	0.101
BOLT E	-0.050	0.000	0.000	-0.101	-0.101	-0.101	0.000	0.000	0.000
BOLT F	0.000	0.103	0.051	-0.051	-0.051	-0.103	-0.103	0.000	0.051
BOLT G	0.000	0.000	-0.101	-0.253	-0.303	-0.303	-0.354	-0.354	0.051
BOLT H	0.103	0.103	0.103	0.000	0.051	0.154	0.154	0.308	0.205
BOLT I	0.050	0.050	0.050	0.150	0.200	0.401	0.601	0.902	-0.601
BOLT J	0.101	0.101	0.051	0.000	-0.051	-0.101	-0.254	-0.203	-0.254
BOLT K	0.102	0.051	0.051	0.000	-0.153	-0.204	-0.306	-0.357	-0.102
BOLT L	0.157	0.105	0.105	0.105	0.000	-0.052	-0.157	-0.210	0.105
BOLT M	0.154	0.051	0.000	0.000	-0.102	-0.102	-0.205	-0.205	0.051
BOLT N	0.101	0.051	0.000	-0.051	-0.202	-0.202	-0.303	-0.404	0.000
BOLT O	0.102	0.000	-0.051	-0.152	-0.152	-0.203	-0.254	-0.254	-0.051
BOLT P	0.100	0.050	0.100	0.100	0.100	0.151	0.251	0.402	-0.100
Strain Gauge No.	STRAIN (μstrain)								
33	7.641	14.327	20.058	27.699	34.385	37.251	38.207	25.789	48.713
34	25.789	63.996	89.786	154.737	241.657	293.236	337.174	370.605	141.365
35	5.731	9.552	9.552	13.372	16.238	17.193	16.238	16.238	21.014
36	-70.682	-139.454	-203.450	-240.702	-274.133	-242.613	-298.012	-340.039	-322.846
37	47.758	95.517	105.069	113.665	123.217	132.768	142.320	144.230	178.616
38	13.373	23.879	26.745	28.655	29.610	28.655	25.790	23.879	35.342
39									
40									
41	-57143.805	-62595.898	-17104.172	0.000	0.000	0.000	0.000	-49886.449	-21706.164
42	0.000	0.000	7150.377	4.775	0.000	7623.186	3509.285	0.000	0.000
43	88.831	200.585	199.629	284.639	270.312	378.246	-24511.490	-24511.490	-24511.490
44	-187.212	-469.941	-604.620	-793.744	-945.615	-1028.714	-1171.034	-1318.130	-1254.133
45	-11.462	-51.579	-68.772	-92.651	-116.530	-130.858	-155.692	-174.796	-207.271
46	24.834	54.445	62.086	70.682	79.279	88.830	95.517	101.248	91.696
47	0.955	2.865	5.731	8.596	12.417	15.283	18.148	21.969	0.955
48	11.462	26.745	35.341	44.893	55.400	69.727	85.965	102.203	34.386
49	0.000	3.821	8.597	15.283	19.103	23.879	27.700	31.521	0.000
50									
51									
52									
53	0.955	1.910	2.865	5.731	8.596	10.507	12.417	14.328	0.000
54									
55	120.351	237.837	274.133	307.564	338.129	364.874	390.664	407.857	364.874
56	127.039	274.135	303.746	-13547.129	-614.170	-13547.129	-13547.129	-13547.129	181.482
57	0.955	134.678	256.940	375.380	493.821	611.307	728.792	847.233	1.910
58	3.821	4.776	2.866	0.000	-2.865	-5.731	-8.596	-10.507	-5.731
59	3.821	3.821	-0.955	-10.507	-23.879	-43.938	-82.144	-116.530	3.821
60	1.910	-4.776	-13.372	-21.969	-30.565	-39.162	-48.713	-59.220	6.686
61	0.955	-13.372	-25.789	-41.072	-58.265	-75.458	-92.651	-108.889	0.955
62	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	18434.719
63	127.992	224.464	241.657	262.670	286.549	291.326	298.967	293.236	457.525
64	62.086	66.862	40.117	2.866	-45.848	-106.979	-185.302	-274.133	270.312
65	0.955	-116.530	-226.375	-335.264	-440.332	-545.400	-652.379	-757.447	-0.955
66	9.552	19.103	25.789	30.565	33.431	38.207	41.072	43.938	29.610
67	149.006	283.684	322.846	360.098	407.856	457.524	527.252	572.145	474.718

DIAL GAUGE No.	DISPLACEMENT (mm)								
1	0.000	2.150	3.820	5.340	6.750	8.100	9.430	10.760	0.040
2	0.000	3.220	5.810	8.290	10.700	13.080	15.470	17.870	0.100
3	0.000	2.350	4.010	5.550	7.110	8.600	10.100	11.610	0.050
4I	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-0.010	-0.010
5P	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-0.010
6	0.000	3.090	5.640	8.050	10.370	12.640	14.920	17.200	0.060

TEST NO 16: CLAMP POSITION 37.5/62.5, OPB (STUDBOLTS HORIZONTAL)
STUDBOLT PRELOAD 180 kN.

MOMENT (kNm)	0	20	40	60	80	100	120	140	0
Studbolt	Studbolt Load Variation (kN)								
BOLT A									
BOLT B	0.0000	0.2500	0.6000	0.8501	1.1501	1.5001	1.8001	2.1501	0.1000
BOLT C	0.0506	0.6069	1.0621	1.5678	2.0230	2.5794	3.0851	3.6415	0.1517
BOLT D	0.0503	0.7040	1.3578	2.0619	2.7156	3.4700	4.0734	4.7272	0.1006
BOLT E	0.2017	0.8573	1.5129	2.1686	2.8746	3.5303	4.1859	4.8415	0.2521
BOLT F	0.0513	0.4616	0.9233	1.3849	1.8465	2.3082	2.7185	3.2314	0.1026
BOLT G	0.1011	0.2023	0.2529	0.3540	0.4552	0.7586	0.8598	1.1633	0.1517
BOLT H	0.0513	0.4107	0.6161	0.7701	0.7188	0.5647	0.3080	0.1027	-0.1027
BOLT I	0.0000	-0.3507	-0.5010	-0.4008	-0.1503	0.5010	1.1022	1.9538	0.2004
BOLT J	0.0000	-0.1014	-0.2536	-0.4058	-0.5579	-0.7101	-0.7608	-0.7101	-0.0507
BOLT K	0.0000	-0.3063	-0.6127	-0.9700	-1.3785	-1.7358	-2.1953	-2.6548	0.0000
BOLT L	0.0524	-0.3143	-0.6287	-0.9430	-1.3098	-1.6765	-2.1480	-2.5148	0.1048
BOLT M	0.0000	-0.4612	-0.8712	-1.3324	-1.8448	-2.2548	-2.8185	-3.3309	0.1025
BOLT N	-0.0505	-0.2526	-0.5558	-0.8590	-1.1622	-1.4148	-1.8190	-2.1222	0.0000
BOLT O	0.0000	-0.1016	-0.3049	-0.4573	-0.6606	-0.7622	-0.9147	-1.0163	0.0508
BOLT P	-0.0502	-0.3517	-0.6531	-0.8039	-0.8039	-0.6531	-0.4522	-0.2512	0.0502
Strain Gauge No.	STRAIN (μstrain)								
33	-0.96	12.42	24.83	39.16	60.18	95.52	122.26	138.50	21.01
34	-11.46	99.34	211.09	294.19	380.16	417.41	447.97	476.63	106.02
35	-6.69	76.41	127.99	196.76	343.86	439.38	512.92	668.62	55.40
36	37.25	-42.03	-17.19	-122.26	-123.22	-176.71	-27568.98	-5.73	-9.55
37	-5.73	-16.24	-31.52	-56.35	-80.23	-106.98	-135.63	-171.93	68.77
38	-3.82	-5.73	-5.73	-6.69	0.00	15.28	27.70	46.80	33.43
39									
40									
41	-62595.90	-62595.90	-62595.90	-62595.90	-62595.90	-62595.90	-62595.90	-62595.90	-62595.90
42	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
43	-16.24	38.21	78.32	80.23	87.88	74.50	63.04	84.05	170.97
44	127.04	-63.04	-97.43	-212.05	-272.22	-324.76	-387.80	-459.43	-385.89
45	23.88	-24.83	-52.53	-61.13	-67.82	-63.04	-66.86	-65.91	-186.26
46	-5.73	23.88	52.53	70.68	86.92	116.53	135.63	158.56	42.98
47	0.00	28.66	60.18	97.43	142.32	194.85	245.48	298.97	6.69
48	-2.87	76.41	160.47	252.16	351.50	463.26	560.68	657.15	31.52
49	0.96	134.68	259.81	381.11	499.55	619.90	739.30	862.52	1.91
50									
51									
52	0.00	0.00	0.00	0.00	0.00	2930.45	0.00	0.00	0.00
53	0.96	-39.16	-78.32	-117.49	-156.65	-196.76	-236.88	-277.00	1.91
54									
55	-10.51	37.25	79.28	82.14	92.65	97.43	101.25	105.07	168.11
56									
57	0.96	-16.24	-32.48	-47.76	-61.13	-73.55	-87.88	-100.29	0.96
58	-1.91	-7.64	-12.42	-19.10	-23.88	-27.70	-32.48	-36.30	-10.51
59	3.82	-74.50	-130.86	-243.57	-239.75	-340.04	-404.99	-450.84	-83.10
60	0.00	-70.68	-127.99	-207.27	-287.51	-370.60	-454.66	-535.85	12.42
61	0.96	-115.58	-225.42	-335.26	-443.20	-555.91	-668.62	-783.24	0.96
62	-47.76	-234.02	-135.63	-568.32	-14519.49	-14609.28	-14609.28	-14609.28	-822.40
63	-18.15	43.94	95.52	104.11	122.26	129.90	139.46	149.01	193.90
64	-7.64	13.37	29.61	36.30	43.94	48.71	52.53	57.31	84.05
65	0.00	-2.87	-4.78	-5.73	-4.78	-3.82	-2.87	-2.87	0.00
66	-1.91	1.91	5.73	7.64	9.55	11.46	12.42	14.33	12.42
67	-16.24	33.43	67.82	69.73	85.96	98.38	106.98	113.66	185.30

DIAL GAUGE No.	DISPLACEMENT (mm)								
1	0.00	1.96	3.55	5.03	6.40	7.74	9.04	10.37	0.01
2	0.00	3.02	5.55	8.00	10.37	12.80	15.20	17.67	0.05
3	0.00	2.29	3.98	5.58	7.12	8.68	10.21	11.77	0.00
4I	0.00	0.00	0.01	0.01	0.01	0.00	0.00	-0.01	0.00
5P	0.00	0.00	0.00	0.01	0.01	0.01	0.01	0.01	-0.01
6	0.00	2.96	5.47	7.87	10.19	12.51	14.77	17.09	0.03

TEST NO 17: CLAMP POSITION 37.5/62.5, OPB (STUDBOLTS HORIZONTAL)
STUDBOLT PRELOAD 180 kN.

MOMENT (kNm)	0	20	40	60	80	100	120	140	0
Studbolt	Studbolt Load Variation (kN)								
BOLT A									
BOLT B	-0.1000	-0.2000	-0.3000	-0.4000	-0.4000	-0.3000	-0.1500	0.1000	0.5500
BOLT C	-0.1011	-0.5563	-1.0621	-1.4161	-1.8207	-2.0736	-2.2759	-2.5794	0.0000
BOLT D	-0.0503	-0.8046	-1.4584	-2.0619	-2.6653	-3.1179	-3.7214	-4.1740	0.0503
BOLT E	-0.1513	-0.8069	-1.5129	-2.1181	-2.6729	-3.1772	-3.6815	-4.3372	0.0000
BOLT F	-0.1026	-0.6155	-0.9746	-1.3849	-1.6927	-1.8978	-2.2056	-2.5133	0.0000
BOLT G	-0.1012	-0.1012	-0.2023	-0.1517	-0.2023	-0.1517	-0.1517	-0.1517	0.0506
BOLT H	-0.1027	-0.4621	-0.7188	-0.7701	-0.8214	-0.5134	-0.0513	0.3080	0.3594
BOLT I	-0.1002	0.5010	1.0521	1.4028	1.7034	1.8536	2.0039	2.0039	-1.0521
BOLT J	-0.0507	0.2029	0.5579	1.0144	1.3187	1.7752	2.0795	2.4345	-0.5072
BOLT K	-0.1532	0.3063	0.8168	1.3274	1.8379	2.4506	3.0122	3.5227	-0.1532
BOLT L	-0.1048	0.3667	0.8906	1.2574	1.6765	2.3052	2.7243	3.2482	-0.1048
BOLT M	-0.1025	0.5124	1.1274	1.6398	2.1523	2.8185	3.3309	3.8946	-0.0512
BOLT N	-0.1516	0.3032	0.6569	1.1116	1.4653	1.8696	2.2738	2.6780	-0.0505
BOLT O	-0.0508	0.1016	0.3557	0.7114	0.9655	1.3720	1.6261	1.9310	-0.1524
BOLT P	-0.1005	0.2512	0.5527	0.6029	0.6531	0.7034	0.6029	0.5527	-0.5024
Strain Gauge No.	STRAIN (μstrain)								
33	0.00	-14.33	-22.92	-34.39	-39.16	-49.67	-52.53	-62.09	31.52
34	0.96	-22.92	-54.44	-106.02	-143.27	-202.50	-241.66	-268.40	43.94
35	-3.82	-84.05	-168.11	-264.58	-126.08	-405.95	-302.79	-435.56	17.19
36									
37	0.00	28525.10	28575.72	28639.72	28662.64	28654.05	28323.56	0.00	28525.10
38	0.00	21.97	27.70	18.15	15.28	-1.91	-8.60	-25.79	-6.69
39									
40									
41									
42									
43	15.28	74.50	160.47	134.68	168.11	161.42	177.66	166.20	227.33
44	-11.46	-92.65	-203.45	-346.73	-391.62	-373.47	-422.18	-506.24	-411.68
45	-11.46	-62.09	-106.98	-114.62	-153.78	-140.41	-179.57	-208.23	-131.81
46	1.91	-3.82	-15.28	-45.85	-58.27	-88.83	-103.16	-125.13	21.01
47	-0.96	-32.48	-64.95	-104.11	-142.32	-186.26	-231.15	-277.95	-7.64
48	0.00	-75.46	-149.01	-251.21	-332.40	-434.60	-534.89	-665.75	30.57
49	-0.96	-133.72	-252.16	-369.65	-476.63	-587.43	-698.23	-815.71	-1.91
50									
51									
52									
53	-1.91	38.21	72.59	106.98	138.50	172.89	203.45	235.93	0.96
54									
55	11.46	54.45	81.19	49.67	76.41	47.76	73.55	81.19	118.44
56	-9.55	19.10	24.83	-41.07	-31.52	-99.34	-78.33	-57.31	36.29
57	0.00	0.96	0.96	0.00	-0.96	0.00	0.96	1.91	0.96
58	-0.96	3.82	6.69	4.78	7.64	7.64	10.51	13.37	-6.69
59	0.96	41.07	64.00	208.23	159.51	250.25	263.63	308.52	-4.78
60	3.82	118.44	187.21	295.15	372.52	519.61	602.71	756.49	132.77
61	-0.96	144.23	268.40	389.71	502.42	619.90	733.57	853.92	0.96
62	56.36	211.09	191.04	-468.03	-271.27	-564.50	-449.88	-559.73	-8.60
63	14.33	60.18	89.79	62.09	86.92	51.58	73.55	80.23	149.96
64	4.78	17.19	25.79	22.92	29.61	21.97	26.74	29.61	71.64
65	0.00	-12.42	-23.88	-34.39	-47.76	-60.18	-72.59	-85.01	1.91
66	0.00	2.87	3.82	2.87	3.82	1.91	2.87	2.87	8.60
67	14.33	64.95	101.25	85.97	120.35	101.25	130.86	147.10	131.81

DIAL GAUGE No.	DISPLACEMENT (mm)								
1	0.00	2.46	4.27	5.90	7.28	8.67	10.02	11.36	0.05
2	0.00	3.57	6.35	9.00	11.35	13.78	16.21	18.63	0.08
3	0.00	2.68	4.53	6.21	7.68	9.19	10.69	12.18	0.03
4I	0.00	0.00	0.01	0.01	0.00	0.00	-0.01	-0.03	-0.01
5P	0.00	0.00	0.01	0.01	0.01	0.01	0.01	0.01	0.00
6	0.00	3.51	6.27	8.86	11.11	13.43	15.81	18.08	0.06

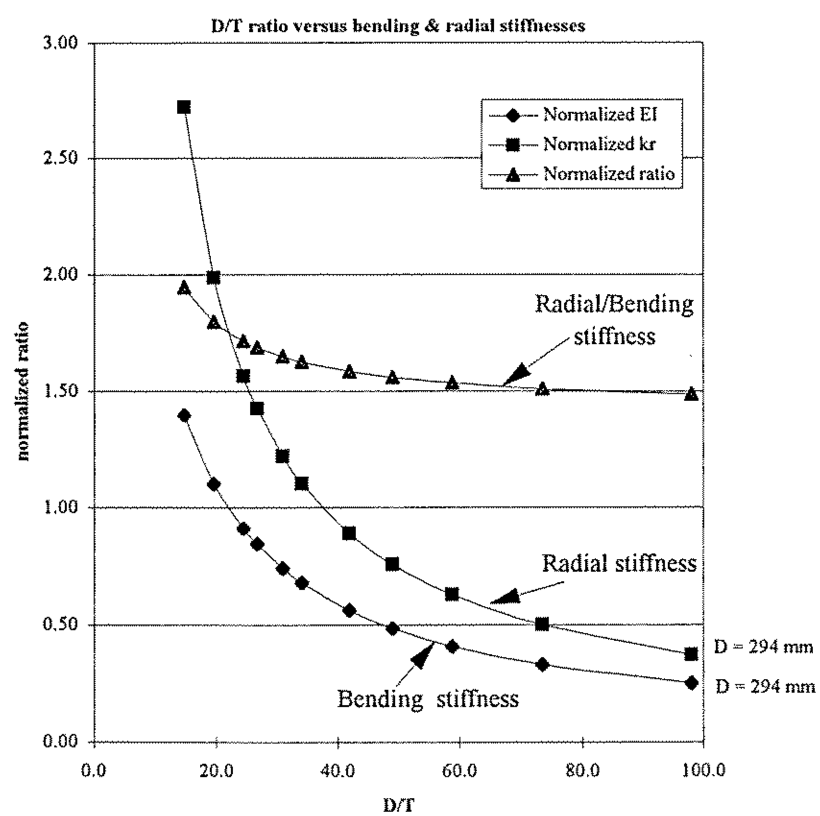
APPENDIX B

CALCULATION OF FE MODEL GEOMETRIES

Study of the relationship between tubular D/T ratios and bending and radial siffnesses

Youngs Modulus, E=210,000N/mm^2
Poissons ratio = 0.3
Tubular outside diameter, base case D=324mm
Tubular outside diameter, modified case D=294mm

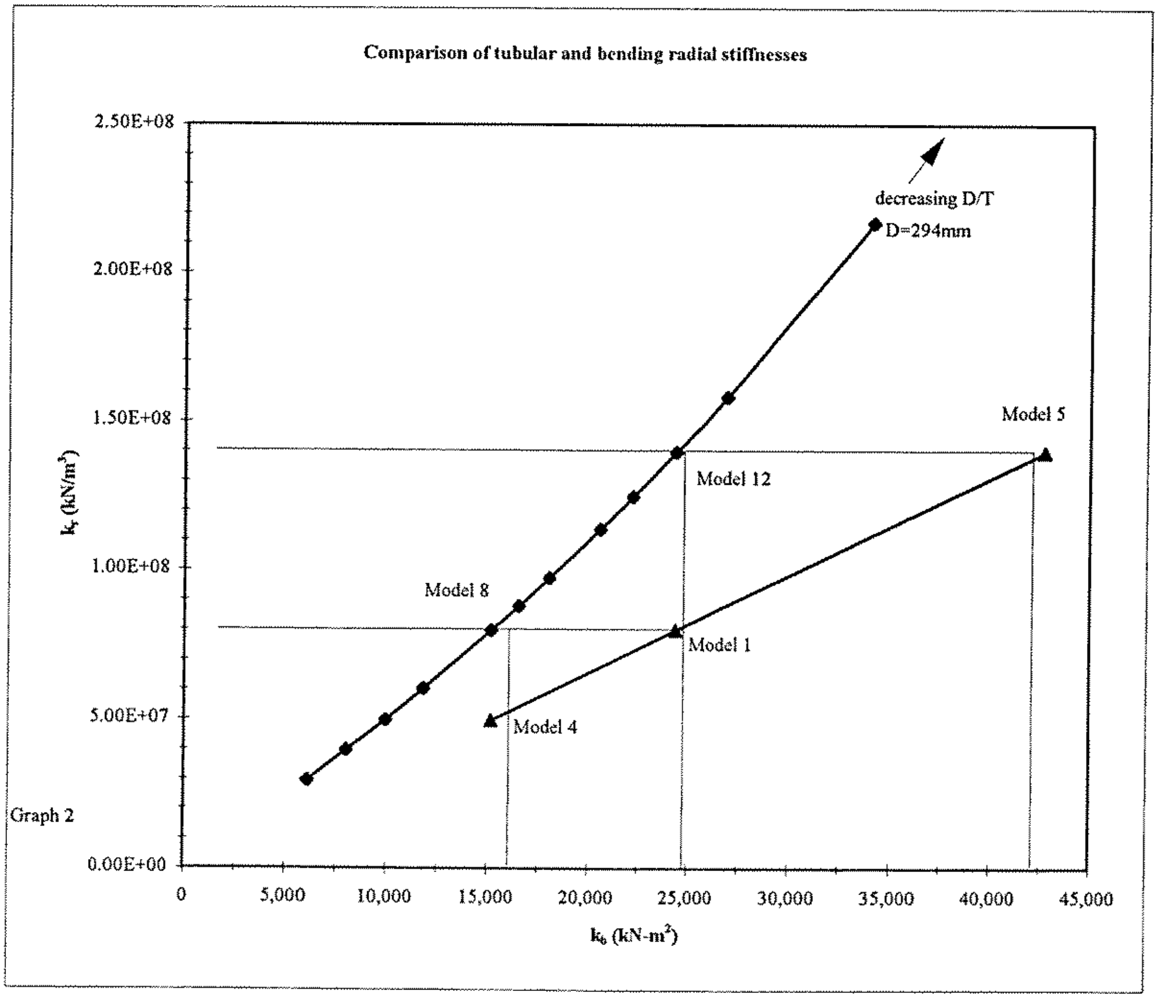
Tubular diameter D (mm)	Wall thickness T (mm)	Ratio D/T	2nd momnt of area I (cm^4)	Bending stiffness EI (kN-m)	Normalized EI	Radial stiffness k (kN-m)	Normalized k_r	Ratio Stiffness radial/bend	Normalized ratio
294	20.0	14.7	16242	34109	1.40	216893983	2.72	6359	1.95
294	15.0	19.6	12830	26942	1.10	158257466	1.99	5874	1.80
294	12.0	24.5	10587	22233	0.91	124534651	1.56	5601	1.71
294	11.0	26.7	9805	20591	0.84	113531222	1.42	5514	1.69
294	9.5	30.9	8600	18061	0.74	97245368	1.22	5384	1.65
294	8.6	34.1	7875	16537	0.68	87812296	1.10	5310	1.63
294	7.0	42.0	6502	13655	0.56	70678688	0.89	5176	1.58
294	6.0	49.0	5631	11825	0.48	60250794	0.76	5095	1.56
294	5.0	58.8	4741	9956	0.41	49934946	0.63	5016	1.54
294	4.0	73.5	3832	8047	0.33	39730109	0.50	4937	1.51
294	3.0	98.0	2903	6097	0.25	29635242	0.37	4861	1.49



Comparison of tubular bending and radial stiffnesses

Youngs Modulus, E=210,000N/mm^2 (UNO)
Poissons ratio = 0.3
Tubular outside diameter, base case D=324mm

Tubular diameter D (mm)	Wall thickness T (mm)	Ratio D/T	2nd momnt of area I (cm^4)	Bending stiffness k_b (kN-m^2)	Radial stiffness k_r (kN-m^3)	Comments
324	9.5	34.1	11,616	24,393	79,685,113	Model 1 - base case
324	9.5	34.1	11,616	15,187	49,613,469	Model 4 - E = 130,750N/mm^2
324	9.5	34.1	11,616	42,727	139,576,824	Model 5 - E = 367,837N/mm^2
294	20.0	14.7	16,242	34,109	216,893,983	
294	15.0	19.6	12,830	26,942	158,257,466	
294	13.35	22.0	11,615	24,391	139,576,808	Model 12 - (k_r as Model 5)
294	12.0	24.5	10,587	22,233	124,534,651	
294	11.0	26.7	9,805	20,591	113,531,222	
294	9.5	30.9	8,600	18,061	97,245,368	
294	8.6	34.1	7,875	16,537	87,812,296	
294	7.855	37.4	7,233	15,188	79,684,169	Model 8 (k_r as Model 1)
294	6.0	49.0	5,631	11,825	60,250,794	
294	5.0	58.8	4,741	9,956	49,934,946	
294	4.0	73.5	3,832	8,047	39,730,109	
294	3.0	98.0	2,903	6,097	29,635,242	



APPENDIX C

FE RESULTS

	Finite Element Model No.											
Studbolt	1	2	3	4	5	6	7	8	9	10	11	12
	Studbolt Bolt Load Variation (kN)											
BOLT A	-0.074	-0.066	-0.076	-0.055	-0.082	-0.081	-0.087	-0.128	-0.121	-0.215	-0.084	-0.155
BOLT B	-0.281	-0.209	-0.350	-0.299	-0.246	-0.291	-0.295	-0.150	-0.161	-0.575	-0.315	-0.083
BOLT C	-0.357	-0.266	-0.444	-0.400	-0.301	-0.369	-0.374	-0.139	-0.167	-0.891	-0.398	-0.029
BOLT D	-0.358	-0.272	-0.438	-0.407	-0.300	-0.368	-0.372	-0.113	-0.152	-1.016	-0.397	-0.004
BOLT E	-0.358	-0.272	-0.438	-0.407	-0.300	-0.368	-0.372	-0.113	-0.152	-1.015	-0.397	-0.005
BOLT F	-0.356	-0.266	-0.443	-0.400	-0.300	-0.368	-0.373	-0.138	-0.167	-0.889	-0.398	-0.029
BOLT G	-0.281	-0.209	-0.350	-0.299	-0.245	-0.291	-0.295	-0.149	-0.161	-0.573	-0.313	-0.083
BOLT H	-0.075	-0.067	-0.077	-0.056	-0.082	-0.082	-0.088	-0.128	-0.121	-0.215	-0.085	-0.154

Table C.1: Studbolt load variation for in-plane bending (M = 140 kNm)

	Finite Element Model No.											
Studbolt	1	2	3	4	5	6	7	8	9	10	11	12
	Studbolt Bolt Load Variation (kN)											
BOLT A	-1.013	-0.811	-1.180	-1.159	-0.958	-1.014	-1.286	-1.546	-1.346	-4.470	-0.934	-0.951
BOLT B	-0.979	-0.862	-0.984	-0.476	-1.467	-1.576	-1.762	-1.320	-1.272	-1.570	-1.496	-1.689
BOLT C	-2.852	-2.134	-3.482	-2.530	-3.077	-3.314	-3.689	-3.152	-2.698	-4.120	-3.302	-3.303
BOLT D	-3.924	-2.876	-4.890	-3.760	-3.956	-4.267	-4.754	-4.209	-3.561	-5.830	-4.277	-4.150
BOLT E	-3.907	-2.862	-4.872	-3.747	-3.920	-4.269	-4.757	-4.207	-3.558	-5.850	-4.266	-4.147
BOLT F	-2.780	-2.085	-3.386	-2.488	-2.941	-3.309	-3.684	-3.142	-2.689	-4.130	-3.288	-3.289
BOLT G	-0.859	-0.781	-0.824	-0.403	-1.267	-1.560	-1.746	-1.304	-1.262	-1.540	-1.476	-1.672
BOLT H	-0.927	-0.752	-1.065	-1.108	-0.812	-1.000	-1.273	-1.532	-1.337	-4.440	-0.898	-0.936
BOLT I	1.263	1.024	1.457	1.354	1.236	1.177	1.462	1.800	1.587	4.960	1.102	1.259
BOLT J	1.634	1.367	1.780	1.145	2.073	2.158	2.352	1.618	1.593	2.810	2.125	1.854
BOLT K	3.662	2.765	4.461	3.417	3.785	4.051	4.437	3.429	3.030	6.080	4.099	3.360
BOLT L	4.718	3.510	5.831	4.655	4.632	5.001	5.498	4.434	3.863	8.080	5.071	4.159
BOLT M	4.702	3.498	5.813	4.643	4.598	5.005	5.500	4.433	3.861	8.090	5.060	4.156
BOLT N	3.582	2.711	4.356	3.370	3.640	4.044	4.429	3.417	3.021	6.090	4.083	3.346
BOLT O	1.510	1.284	1.616	1.068	1.869	2.140	2.334	1.602	1.583	2.790	2.101	1.837
BOLT P	1.173	0.962	1.338	1.297	1.088	1.160	1.445	1.786	1.577	4.930	1.065	1.244

Table C.2: Studbolt load variation for out-of-plane bending (M = 140 kNm)

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CONTROLLED DOCUMENT

STRENGTHENING, MODIFICATION AND REPAIR OF OFFSHORE INSTALLATIONS

PART VI - DIVERLESS IMPLEMENTATION STUDIES

DOC REF C11100R236 Rev 1 NOVEMBER 1995

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STRENGTHENING, MODIFICATION AND REPAIR OF OFFSHORE INSTALLATIONS

PART VI - DIVERLESS IMPLEMENTATION STUDIES

STRENGTHENING, MODIFICATION AND REPAIR OF OFFSHORE INSTALLATIONS

PART VI - DIVERLESS IMPLEMENTATION STUDIES

CONTENTS

VI 1 INTRODUCTION

The principal global activity in respect of this Joint Industry Project (JIP) has related to exhaustive appraisals of all strengthening and repair techniques, leading to the preparation of detailed design and applications guidance. In addition to this, a series of feasibility studies have been conducted to examine the deployment of selected systems using diverless technology. These feasibility studies have comprised the following:

(a) Feasibility of deploying heavy steelwork using currently available ROVs and buoyancy chambers.

(b) Examination of various strengthening/repair systems and scenarios, and selection of a relevant combined scenario for further study.

(c) Feasibility of diverless implementation of the selected scenario from (b).

In addition to the above studies, Mobil North Sea Limited have kindly shared details with this JIP of a project concerning the world's first subsea diverless implementation of six stressed repair clamps. MSL Engineering acted as Consulting Engineer for this project. The clamps were successfully deployed in Mobil's Beryl Bravo platform in 1993. Mobil have also kindly given permission for the projects details to be catalogued in documentation issued as part of this JIP.

This Part VI presents the findings from all the JIP activities on diverless implementation of strengthening/repair systems, including details of the Mobil project. The document is structured along the following lines:

- Section VI 2 - Deployment of heavy steelwork
- Section VI 3 - Scenarios
- Section VI 4 - Feasibility study for selected scenario
- Section VI 5 - Mobil's Beryl Bravo repairs
- Section VI 6 - Closure.

VI 2 DEPLOYMENT OF HEAVY STEELWORK USING ROVs AND BUOYANCY CHAMBERS

A feasibility evaluation in this respect has been conducted and is reported in detail in Appendix A. Appendix A was discussed at the Project Steering Committee (PSC) meeting held on 30 June 1993.

VI 3 SCENARIOS AND SELECTION

A detailed appraisal of damage scenarios has been conducted and is reported in Appendix B. This was discussed in detail at the PSC meeting on 29 September 1993, and it was agreed that a damage scenario representing a dented member (from a dropped object) should be considered, and a repair/strengthening scenario which encompasses the need for a new brace member (attached to the jacket structure by means of clamps) should be investigated in the feasibility exercise for diverless implementation.

VI 4 FEASIBILITY INVESTIGATIONS FOR SELECTED SCENARIO

VI 4.1 GENERAL AND FEASIBILITY CONCEPT

The premise for the repair/strengthening feasibility study is based on a shallow water application which can be adapted for deepwater sites. Throughout this document, reference is made to alternative and/or additional requirements of a deepwater application. A number of aspects concerning the feasibility concept can be noted:

- The scenario of a dropped object, resulting in a dent in an external bracing member, has been investigated. The dent reduces the ability of the brace to sustain load. By installing an addmember, the capacity is restored.

- Elastomer-lined clamps are used as end-connection devices of the addmember. This type of clamp can be used on the damaged brace adjacent to the dent location where ovality is within fabrication tolerances. A repair/strengthening scenario which requires the clamp to cover the dent/damage will also require the member ovality and/or straightness to be determined, using an ROV survey. The survey may be carried out using photogrammetry in conjunction with scale yard sticks and callipers to measure distances and ovality respectively.

- If it is permissible to attach the addmember away from the dent/damage location then the repair scheme avoids the requirements for a detailed underwater survey. However, an eye-ball survey is required to determine approximate locations of obstructions (e.g. anodes, doubler plates).

- The clamps are elastomer-lined to conform to member surfaces and to prevent crevice corrosion which may otherwise occur between two mating steel surfaces.

- The addmember has pinned connections at each end to allow rotation of the clamps in the plane of the structure in which the add-member is located. This is done to accommodate variation in the linear distance between the nominal attachment points.

- The dropped object scenario considered herein has been assumed to occur to the external bracing on the jacket. It may be possible for the deck crane to reach above the repair area. If this is not possible, either a crawler crane or 'A' frame can be used as a means of lowering the addmember assembly from the deck. Another option is to use a supply vessel or ship crane. A third possibility is to construct a working platform beneath the topsides.

- The design of the repair system relies on conventional technology and currently available ROVs.

VI 4.2 GEOMETRICAL CONSIDERATIONS

Appendix C contains two idealised addmember assemblies (Schemes 1 and 2), including details of a clamp closure technique and studbolt engagement methods. The main difference between the schemes is the location of the clamp hinge, and this is related to the installation method.

Details contained in Appendix C show the structural hinge as a major component of the clamp assembly. The location of this hinge, which varies between installation Scheme 1 and Scheme 2, may, after detailed design, differ between top and bottom clamp assemblies, ie. a combination of Scheme 1 and Scheme 2 arrangements may be finally chosen.

The structural hinge performs two important functions which are as follows:

1. It aids clamp installation by negating a requirement for separate lowering of clamp halves and by halving the number of studbolts.

2. Conventional clamps can only transfer frictional load through the clamp half to which the addmember is attached. The structural hinge enables frictional load to be transferred through both clamp halves, regardless of which clamp half the addmember is attached to. The structural hinge can be designed to resist these additional loads. The structural hinge will consist of a bar running through tolerance fit holes in the extended stiffeners. The total applied load through the hinge resulting from studbolt tensioning will be similar to the total studbolt load apart from some frictional loss. These frictional effects need to be considered when assessing the frictional capacity of the clamp.

The clamp geometry detailed in Appendix C is fairly typical of a continuous flanged clamp assembly. During detailed design, this geometry may alter to enable efficient use of steelwork for load transfer and ease of installation.

The addmember is provided with a structural hinge at each end. This allows, in effect, a length adjustment by permitting translational displacement of the clamps along the members. The addmember buoyancy may be set at the most beneficial value.

VI 4.3 INSTALLATION CONSIDERATIONS

Appendix D contains installation sequences for two proposed shallow water schemes. The main differences between the schemes are as follows:

- Slinging arrangement
- Clamp positioning order for top and bottom clamps.

The installation sequence schematics for Scheme 1 and Scheme 2, contained in Appendix D, details a frame-by-frame sequence for the addmember assembly to its final resting position. The methods and procedures required to achieve this are also contained in Appendix D. All operations are designed to be fully reversible, in case any difficulties are encountered.

The installation for a deepwater site is not dissimilar from that presented in Appendix D. The salient differences between shallow and deepwater site applications are discussed in Appendix D.

VI 4.4 SCHEDULING CONSIDERATIONS

A schedule for detailed design, fabrication and implementation has been estimated and is presented in Figure 4.4.1. The following assumptions have been made in the preparation of the schedule:

- The engineering, procurement and fabrication, pre-installation and installation phases are carried out with a limited overlap between these four main activities.

- The engineering team is assumed to comprise 2 Engineers and 1 Draughtperson.

- The fabrication is carried out by one fabricator allowing four weeks for the clamps and an additional week for the addmember and ancillaries. In addition, it is estimated that a further twelve weeks will be required to procure steelwork and order items such as bolts, pins, neoprene, hydraulics and tensioning tools.

- An onshore fit-up trial is assumed for familiarisation purposes with the clamp/ addmember operations.

- The complete installation is assumed to take approximately 2 weeks offshore.

The schedule in Figure 1 indicates the following timetable for the four major activities:

• Engineering and Design	8 weeks
• Procurement and Fabrication	16 weeks
• Pre-installation work	16 weeks excluding initial inspection
• Offshore Installation	2 weeks including mob/demob.

The total estimated timescale is 24 weeks. A number of options exist to reduce this time frame:

(i) Increase Engineering and Design resource.

(ii) Increase work shifts.

(iii) Consider more overlap of the major activities, particularly with early procurement of special components.

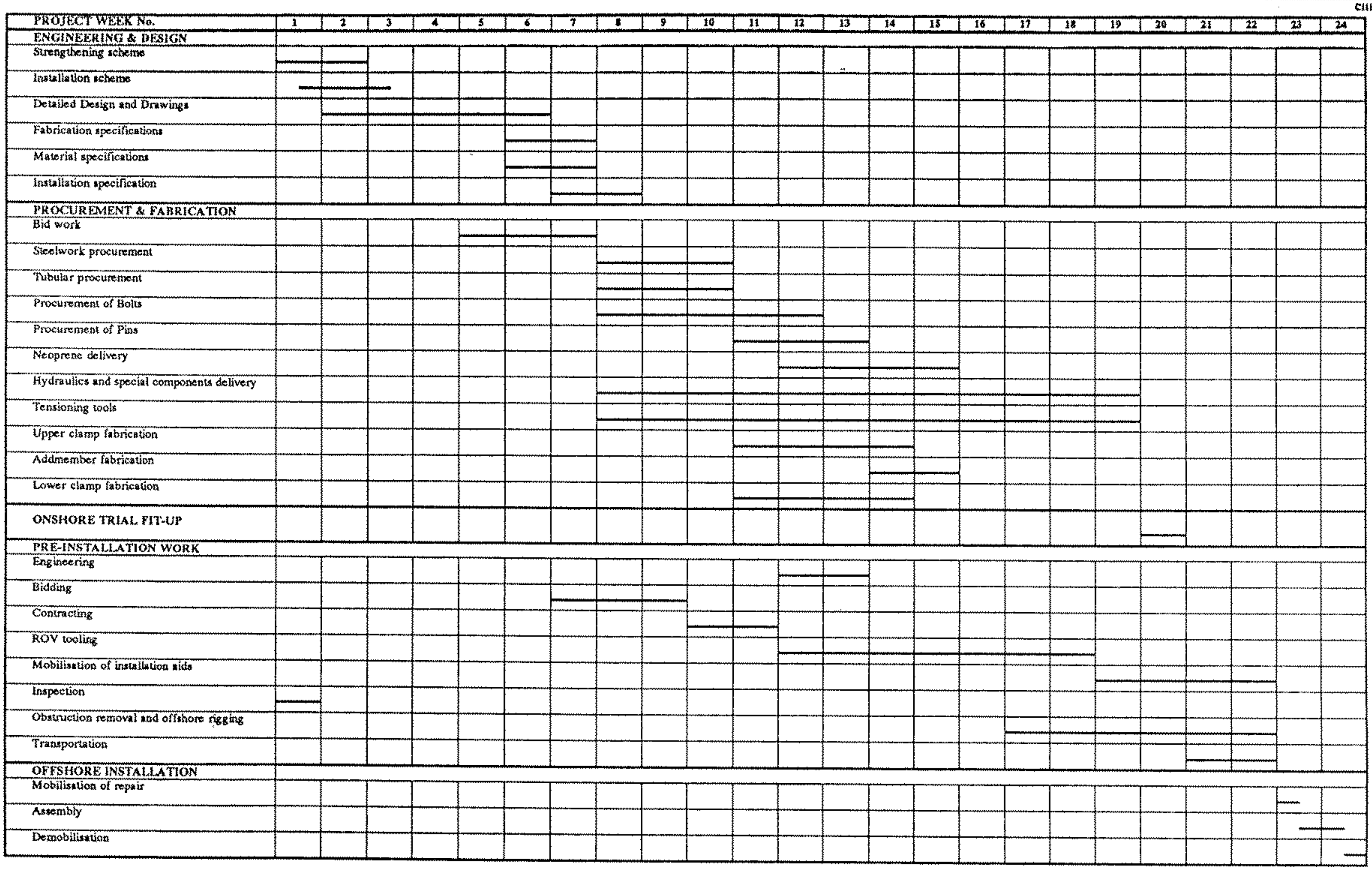

Figure 4.4.1: Estimated Schedule for Add-Member Scenario

VI 5 MOBIL BERYL BRAVO REPAIR

Appendix E presents details of the repair of six caissons on Mobil's Beryl Bravo platform using stressed clamps which were deployed successfully using diverless technology.

VI 6 CLOSURE

The investigations reported herein demonstrate that diverless implementation of add-member systems for strengthening, modification or repair purposes is entirely feasible and cost-effective. The adoption of diverless technology leads to substantial benefits to safety. Mobil's Beryl Bravo repairs have demonstrated these aspects in a live repair project.

APPENDIX A

DEPLOYMENT OF HEAVY STEELWORK

MSL

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Issued to Participants	0	June 1993	NJS		

JIP - STRENGTHENING, MODIFICATION AND REPAIR TECHNIQUES

WORK PACKAGES SMR 16: FEASIBILITY STUDY ON DIVERLESS SMR & SMR 17: DIVERLESS IMPLEMENTATION

CONCEPTUAL STUDY ON ROV REQUIREMENTS FOR HANDLING REPAIR COMPONENTS

DOC REF C11100R228 Rev 0 JUNE 1993

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CONTENTS

1. INTRODUCTION

This document presents the findings of a feasibility study into the possibility of using ROVs in handling large repair components for the underwater structural strengthening/modification/repair (SMR) of offshore steel installations. The study has been conducted by MSL Engineering as part of the joint industry project on SMR techniques.

The operational requirements of ROVs to handle repair components are established in the following sections for scenarios involving the installation of clamps weighing up to 10 tonnes. However, the results could be applicable for other repair components, eg. replacement members particularly where internal buoyancy can be provided. Geometrical sizing of the clamps and other associated components is addressed in Section 2.

Sea water depths up to 1000m are considered in the study. In Section 3, certain current flows are assumed in establishing the hydrodynamic forces acting on the repair components. Wave forces, as the ROV and repair components pass through the splash zone region, are not considered herein. Should these be large, there are established techniques, such as use of a stinger, to overcome any potential problem in the passage through the critical surface zone.

2. SIZING OF REPAIR COMPONENTS

2.1 Description of Components

The repair components are, of course, scenario dependent but in this study it is assumed that a substantial component, ie. a clamp, is to be installed. Experience gained with diver-installed clamps suggests that the weights of clamps normally lies in the range of 2 to 10 tonnes. They may be configured as tube-to-tube connections, T-, DT- or K-clamps. Although these weights can be deployed by being slung from the bottom of suitable ROVs, with the weight taken by the umbilical, this would lead to operational difficulties with access and structure safety. For flexibility in operations, the ability to use an ROV to manoeuvre the clamp without reliance on umbilical tension is required. Present-day ROVs do not have the lift capability for the above weight range, either due to internal buoyancy or thrust. Thus there is a requirement for buoyancy (external to the ROV) to offset the weight of the clamp.

The drag forces acting on the buoyancy unit, whether this be a pressure vessel or syntactic foam block, may be acceptable to the structure. This may be influenced by the depth of the repair. However, it is more likely that the drag forces will prove to be too high and therefore it will be necessary to remove the buoyancy unit once the clamp has been deployed and placed in its final position. The net buoyancy of the unit during clamp deployment is necessarily approximately equal in magnitude to the submerged weight of the clamp (assuming ROV is neutrally buoyant). Since on separation from the clamp (or ROV) the unit will rapidly rise to the surface possibly causing damage, the unit must be designed with variable buoyancy such that neutral buoyancy of the whole system can be maintained at all times.

There are several aspects to consider for systems giving variable buoyancy.

- Buoyancy can be provided by a fluid of lower density than seawater, which when replaced by seawater, leads to a variable buoyancy system. The fluid may be a gas or an oil. In the case of the latter, large volumes are required since the differential density with seawater is small. Since this would lead to large containment vessels and hence large drag forces, oil is not considered further herein.

- The gas containment vessel could be of conventional steel or aluminium pressure vessel construction, or spherical chambers of glass or plastic.

  Glass or plastic (acrylic) vessels open up the possibility of shattering them (using, perhaps, an explosive charge) once the clamp is in position, thereby simplifying ROV recovery. This might be a practical option for one off clamp installations but reusable vessels would prove more favourable for scenarios requiring the installation of two or more clamps.

For the purposes of the present study, steel pressure vessels are selected on the basis that their extra weight and size will maximise drag forces and will therefore place an upper bound on ROV requirements.

- The steel pressure vessel could be spherical, cylindrical or any other shape. The shape will affect the thickness of the vessel (spherical vessels being more efficient than cylindrical) and the drag forces acting on it. Spherical vessels are more difficult, and hence more costly, to fabricate than cylindrical ones. However, they do have the advantage of reduced drag and are easier to control. (Cylindrical vessels could possibly be subject to 'weather vaning'.) In this study both spherical and cylindrical vessels are considered. Various aspect ratios (length/diameter ratios) are explored for the cylindrical vessels which are also assumed to have hemi-spherical end caps.

- The weight of the pressure vessel (and the amount of ensuing self-buoyancy required) could be reduced by considering stiffened forms of construction or by initially pressurising the gas to the ambient pressure at the site of the repair. These techniques would increase the cost of the vessel and lead to increased engineering complexity. As neither technique is really required for this study, they have not been further considered.

- The buoyancy unit is to be approximately neutrally buoyant when it is separated from the clamp. Two possibilities can be considered. Firstly, it is possible to arrange for not all of the gas in the vessel to be displaced by seawater. This could be achieved by a vent-pipe protruding a pre-determined distance into the vessel, or by initially filling the vessel at the pressure such that when the flooding valve is opened at the depth of the repair site, the gas becomes compressed to the required volume when the valve is then closed.

  Secondly, a simpler and more appropriate solution may be provided by the use of syntactic foam which provides the correct amount of residual buoyancy after the pressure vessel has been completely flooded. It is also a more robust solution as it is less dependent on flooding valves being completely closed as required in some of the options discussed above. The syntactic foam can be cast into any shape. For this study, the foam is assumed to blanket the straight part of cylindrical pressure vessels (not the end caps) or the whole of spherical pressure vessels.

In summary, the repair components considered in this study consist of:

•	A clamp:	Ranging from 2 to 10 tonnes and of various configurations (shapes).
•	A pressure vessel:	Unstiffened steel construction and either spherical or cylindrical with hemispherical end caps.
•	Syntactic foam:	Wrapped around pressure vessel.

2.2 Sizing of Components

2.2.1 General

The general objective in sizing is to identify the external geometries of the repair components as it is these which will determine hydrodynamic drag forces and thus ROV operational requirements.

It has already been stated that clamp weights generally fall within the range of 2 to 10 tonnes. It was decided therefore to select clamps of 2, 5 and 10 tonnes to progress the study. It is then possible to size the clamps (in terms of geometry) and the other repair components from these weights.

The various steps are outlined in Figure 1 and are discussed below.

2.2.2 Clamps

Clamp geometry was selected using a number of simplified rules for the above mentioned clamp weights. Experience in clamp design has shown that between 3 and 5 tonnes, the clamp configuration is more likely to be for a T-joint. Below 3 tonnes, the clamp is likely to take the form of a split sleeve. The clamp configuration for weights between 5 and 7 tonnes may be either a T, DT/X or K joint. Above 7 tonnes, the clamp configuration is more likely to take the form of a joint with a minimum of two braces.

The first rule in sizing the three clamps, is that for each member coming into the joint, the clamp length is at least equal to the member diameter. This is to ensure adequate load transfer from the member into the clamp. The majority of clamps designed in recent times have utilised saddle thicknesses rarely greater than 30mm. Therefore, for the second rule, clamps upto 5 tonnes are assumed to utilise saddle thicknesses of 25mm and above 5 tonnes it is 30mm. Now that the saddle thickness has been determined, the top plate and side plates can be assumed to have the same thickness.

For the 2 tonne split sleeve, the saddle was assumed to be 1m diameter and 25mm thick with a total length of 3m, see Figure 2. The 5 tonne split sleeve is thicker-walled and is illustrated in Figure 3.

The 5 tonne T-clamp is assumed to be a stressed grouted clamp. This alone determines dimensions not already estimated using the rules. A chord diameter of 700mm and brace diameter of 400mm were assumed for the 5 tonne T-clamp. Allowing for the grout annulus (50mm), saddle thickness (25mm), studbolts centre line and to the side plates (80mm) and top-plate over-hang (50mm), the chord clamp width is estimated to be 1100mm. As a general rule for any size clamp, the clamp width is approximately equal to the member diameter plus 400mm. This is repeated for the brace clamp. The T-clamp is shown in Figure 4.

The chord diameter for the 10 tonne K clamp was estimated to be 1100mm and the two brace diameters is 700mm. Sizing of the clamp widths is the same process as that carried out for the T-clamp. The clamp is shown in Figure 5.

2.2.3 Pressure vessel and syntactic foam

The internal volume of the pressure vessel is simply obtained as a certain volume of seawater, the weight of which is equal to the submerged weight of the clamp. The submerged clamp weight partly depends on the extent of any enclosed voids within the clamp and partly on seawater. For simplicity, and conservatively, the submerged weight of the clamp is taken as equal to the dry weight (ie. 2, 5 or 10 tonnes) for this study.

Having established the internal volume and making assumptions about the shape of the vessel (spherical or cylindrical of particular aspect ratio), the thickness of vessel to resist ambient pressure can be calculated in accordance with a design code. BS5500[1] was selected for this purpose. Thickness requirements were investigated at 10, 50, 200, 400, and 1000m sea depth.

The volume of syntactic foam is obtained from a consideration of the differential density of the foam and seawater, and the requirement that the (submerged) weight of the steel in the pressure vessel is equal to the buoyancy given by the foam. The foam is assumed to be wrapped around the straight portion of cylindrical pressure vessels or to encircle spherical pressure vessels, see Figures 6 and 7 respectively.

Calculations were conducted using the MATHCAD package and sample calculations are presented in the Appendix. A summary of the results for the 2, 5 and 10 tonne clamps are presented in Tables 1 to 3 respectively. Refer to Figures 6 and 7 for geometric notation. The last two columns indicates the stress in the pressure vessel as a proportion of the yield stress. Finite values in the second column, headed m(=L/D), indicates the aspect ratio of cylindrical pressure vessels; where zero is given a spherical pressure vessel is implied.

It can be seen from the results that the thickness of cylindrical pressure vessels is up to 3 times that of spherical vessels, though this ratio decreases with increasing water depth. It may also be observed that the external dimensions of the pressure vessel are not greatly sensitive to water depth (this is because the steel thickness is only a small fraction of the (fixed) internal diameter). However, the thickness of foam, being directly related to the weight and hence the wall thickness of the vessel, is very depth dependent.

3. HYDRODYNAMIC FORCES

3.1 Calculation Philosophy

Offshore structures exist in a fluid environment where hydrodynamic forces are an important consideration. Waves are not considered in this application; the technology is available to get the ROV and the repair components through the critical surface zone (wave zone).

The term "current" may be defined as "a flow of water past a fixed location" and is usually described with a current speed and direction; measurements are usually analysed in terms of the tidal current and residual currents. Residual currents are irregular but at most locations the largest residual to be considered in the design is the storm surge current. In this investigation, the storm surge current is not considered since the clamp deployment operation is assumed to take place during fine weather.

Three representative velocities are considered, 1.5, 1.0, 0.5m/sec for the water depths chosen for the purpose of this study. For currents in deep waters, the choice of high velocities is fully justified on the basis of the UK Health and Safety Executive Guidance Notes[2] where measurements of hourly-mean currents in the range of 1.2-1.4 m/sec resulting from internal waves are reported. In the same document, a comment is made that peak currents may be even higher since the measuring systems used a 1-hour averaging time used to obtain the above measurements. During the summer time, the internal waves experienced during the operation may be more severe than for other seasons because of the temperature difference between the sea surface and the sea bed for deep waters.

For the present application, the hydrodynamic loading considered is the pressure drag loading caused by vortices generated in the flow as it passes the repair components. Disregarding the range of very small Reynolds numbers, which are not a governing case in this study, hydrodynamic force (drag) may be estimated from:

$$F_d = 0.5 * C_d * \rho * A * v^2$$

where:

F_d = drag force in N,
C_d = drag coefficient,
ρ = mass density of the fluid in kg/m^3,
A = suitable area of the repair component; the projected plan-form area or the frontal area in m^2,
v = flow speed in m/sec.

The drag force is proportional to incident velocity squared and also increases with the size of the body involved. The drag coefficient is derived from graphs as a function of the shape and angle of attack of the body under consideration. It is independent of the medium considered since the flow pattern in the vicinity of a body is identical in air, water, or any other liquid [Hoerner, 1965].

For the case of the cylindrical pressure vessel and split sleeve clamp, the drag coefficient for the analysis is derived from Figure 8 for flow perpendicular to the axis and from Figure 9 for flow along the axis. Figure 8 is based on the cylinder diameter and the results are presented as a function of the Reynolds Number (R_e). In Figure 9 the results are presented as a function of the ratio L/D (L = length of the cylinder; D = diameter of the cylinder).

For the case of the spherical pressure vessel, the drag coefficient adopted in the analysis is equal to 0.23; this is a conservative estimation based on Figure 10 for Reynolds numbers in the expected range of $7*10^5$ and 10^7 to be experienced by the vessel in the present application.

For the case of T and K clamps, the drag coefficient adopted is equal to 1.20 and is a conservative estimation of the drag coefficient of an equivalent cube at Reynolds Numbers between 10^4 and 10^6 (Figure 11).

3.2 Results

Calculations were again carried out using MATHCAD, and sample calculations are included in the Appendix. The results of the calculations are tabulated in Table 4 and illustrated in Figures 12 to 18. Figures 16 to 18 are a representation of the 5 tonne clamp data first shown in Figures 13 and 14. In the Table and Figures, the results for the cylindrical pressure vessel relate to an aspect ratio (L/D) of 1.

A comparison between the data for spherical and cylindrical pressure vessels (eg. as in Figures 12 to 15) reveals that the drag forces on the spherical vessel are significantly lower than those on the cylindrical vessel in currents of 1.5m/s, slightly lower in 1.0m/s current, and approximately similar in currents of 0.5m/s. In the case of the 5 tonne clamp for which two configurations were examined, ie. the split sleeve and the T-clamp, it is found that the split sleeve has less drag than the T-clamp, see Figures 16 to 18. Again, this observation is more pronounced for faster currents. For all types of clamp configurations and weights, drag forces tend to increase with depth. This is mainly a consequence of the increase projected area due to syntactic foam.

4. CLOSURE

The results presented in Table 4 show that for the 2 tonne clamp, a maximum of 3.5 kN drag force is required to be met by the ROV; for the 5 tonne clamp 6.5 kN; and for the 10 tonne clamp 17.5 kN. The drag forces on the repair components for the 2 or 5 tonne clamps for the full range of velocities considered can be handled by available 'work horse' ROVs; assuming 8 kN max horizontal thrust capacity for an ROV. In the case of the repair components for the 10 tonne clamp, the drag force can be handled by the ROV at 0.5 and 1.0 m/sec velocities. However, at 1.5 m/sec velocity, additional capacity is required by the ROV.

It may thus be inferred that present-day ROVs could feasibly be used to deploy and manoeuvre small and moderately-sized clamps with associated repair components in currents up to at least 1.5m/s, and somewhat larger clamps in lower currents. Stability issues require checking before these conclusions can be endorsed.

REFERENCES

1. BS 5500:1985 'Specification for Unfired fusion welded pressure vessels'. British Standards Institution.

2. UK Department of Energy. 'Offshore Installations: Guidance on design construction and certification'. Fourth Edition, 1990.

3. Hoerner SF. 'Fluid - Dynamic Drag'. 1965.

4. Barltrop NDP and Adams AJ. 'Dynamics of Fixed Marine Structures'. The Marine Technology Directorate Ltd 1991.

Depth (m)	m(L/D)	D (mm)	D1 (mm)	D_f (mm)	e (mm)	e_f (mm)	e/D	p/p_y	p_c/p_{yss}
10	0	1550	1553	1614	1.5	30.5	0.001		0.08
	1	1355	1364	1548	4.5	92	0.0033	0.048	
	2	1075	1084	1262	4.5	89	0.0042	0.035	
	3	939.5	948.5	1121.5	4.5	86.5	0.0048	0.028	
	4	853.5	862.5	1034.5	4.5	86	0.0053	0.022	
50	0	1550	1557	1692	3.5	67.5	0.002		0.177
	1	1355	1372	1702	8.5	165	0.0063	0.126	
	2	1075	1092	1408	8.5	158	0.0079	0.085	
	3	939.5	956.5	1262.5	8.5	153	0.0090	0.09	
	4	853.5	870.5	1172.5	8.5	151	0.0100	0.063	
200	0	1550	1565	1830	7.5	132.5	0.005		0.312
	1	1355	1385	1927	15	271	0.0111	0.294	
	2	1075	1105	1620	15	257.5	0.0140	0.23	
	3	939.5	969.5	1465.5	15	248	0.0160	0.168	
	4	853.5	883.5	1370.5	15	243.5	0.0176	0.181	
400	0	1550	1575	1979	12.5	202	0.008		0.371
	1	1355	1392	2039	18.5	323.5	0.0137	0.416	
	2	1075	1112	1724	18.5	306	0.0172	0.341	
	3	939.5	976.5	1565.5	18.5	294.5	0.0197	0.244	
	4	853.5	890.5	1466.5	18.5	288	0.0217	0.272	
1000	0	1550	1608	2367	29	379.5	0.019		0.38
	1	1355	1425	2506	35	540.5	0.0258	0.567	
	2	1075	1145	2155	35	505	0.0326	0.522	
	3	939.5	1009.5	1973.5	35	482	0.0373	0.598	
	4	853.5	923.5	1862.5	35	469.5	0.0410	0.435	

TABLE 1 SIZING OF REPAIR COMPONENTS - 2 (TONNE) CLAMP INCLUDED

Depth (m)	m(L/D)	D (mm)	D1 (mm)	D_f (mm)	e (mm)	e_f (mm)	e/D	p/p_y	p_c/p_{yss}
10	0	2100	2104	2185	2	40.5	0.001		0.08
	1	1838.5	1850.5	2091.5	6	120.5	0.003	0.047	
	2	1459	1471	1705	6	117	0.004	0.033	
	3	1274.5	1286.5	2607.5	6	660.5	0.005	0.027	
	4	1158	1170	1400	6	115	0.005	0.029	
50	0	2100	2110	2301	5	95.5	0.002		0.185
	1	1838.5	1860.5	2281.5	11	210.5	0.006	0.117	
	2	1459	1481	1886	11	202.5	0.008	0.079	
	3	1274.5	1296.5	1695.5	11	199.5	0.009	0.082	
	4	1158	1180	1574	11	197	0.009	0.058	
200	0	2100	2120	2473	10	176.5	0.005		0.309
	1	1838.5	1878.5	2590.5	20	356	0.011	0.286	
	2	1459	1499	2178	20	339.5	0.014	0.223	
	3	1274.5	1314.5	1978.5	20	332	0.016	0.223	
	4	1158	1198	1851	20	326.5	0.017	0.175	
400	0	2100	2132	2655	16	261.5	0.008		0.371
	1	1838.5	1888.5	2748.5	25	430	0.014	0.413	
	2	1459	1509	2325	25	408	0.017	0.338	
	3	1274.5	1324.5	2120.5	25	398	0.020	0.243	
	4	1158	1208	1989	25	390.5	0.022	0.27	
1000	0	2100	2180	3220	40	520	0.019		0.38
	1	1838.5	1930.5	3338.5	46	704	0.025	0.561	
	2	1459	1551	2872	46	660.5	0.032	0.514	
	3	1274.5	1366.5	2644.5	46	639	0.036	0.528	
	4	1158	1250	2498	46	624	0.040	0.419	

TABLE 2 SIZING OF REPAIR COMPONENTS - 5 (TONNE) CLAMP INCLUDED

Depth (m)	m(L/D)	D (mm)	D1 (mm)	D_f (mm)	e (mm)	e_f (mm)	e/D	p/p_y	p_c/p_{yss}
10	0	2650	2655	2756	2.5	50.5	0.001		0.079
	1	2316	2331	2628	7.5	148.5	0.003	0.046	
	2	1838.5	1853.5	2149.5	7.5	148	0.004	0.033	
	3	1606	1621	1910	7.5	144.5	0.005	0.027	
	4	1459	1474	1761	7.5	143.5	0.005	0.029	
50	0	2650	2662	2892	6	115	0.002		0.178
	1	2316	2344	2872	14	264	0.006	0.119	
	2	1838.5	1866.5	2388.5	14	261	0.008	0.08	
	3	1606	1634	2140	14	253	0.009	0.084	
	4	1459	1487	1988	14	250.5	0.010	0.059	
200	0	2650	2675	3117	12.5	221	0.005		0.307
	1	2316	2366	3247	25	440.5	0.011	0.282	
	2	1838.5	1888.5	2748.5	25	430	0.014	0.219	
	3	1606	1656	2485	25	414.5	0.016	0.16	
	4	1459	1509	2325	25	408	0.017	0.17	
400	0	2650	2690	3344	20	327	0.008		0.369
	1	2316	2380	3465	32	542.5	0.014	0.424	
	2	1838.5	1902.5	2957.5	32	527.5	0.017	0.349	
	3	1606	1670	2683	32	506.5	0.020	0.25	
	4	1459	1523	2519	32	498	0.022	0.279	
1000	0	2650	2748	4034	49	643	0.018		0.38
	1	2316	2432	4188	58	878	0.025	0.562	
	2	1838.5	1954.5	3638.5	58	842	0.032	0.514	
	3	1606	1722	3330	58	804	0.036	0.528	
	4	1459	1575	3147	58	786	0.040	0.421	

TABLE 3 SIZING OF REPAIR COMPONENTS - 10 (TONNE) CLAMP INCLUDED

Clamp		Weight (tn)	2			5						10		
		Type	Split sleeve			Split sleeve			T			K		
Current Velocity (m/sec)			1.5	1.0	0.5	1.5	1.0	0.5	1.5	1.0	0.5	1.5	1.0	0.5
		Pressure Vessel												
Sea depth (m)	10	Cyl.	2.21	0.85	0.47	4.4	1.67	0.41	6.19	2.59	0.61	14.57	6.19	1.45
		Sph.	1.79	0.7	0.41	3.36	1.36	0.37	5.15	2.29	0.57	12.5	5.56	1.39
	50	Cyl.	2.37	0.89	0.44	4.78	1.74	0.42	6.57	2.66	0.62	14.81	6.32	1.47
		Sph.	1.84	0.73	0.41	3.46	1.41	0.39	5.25	2.34	0.58	12.66	5.63	1.41
	200	Cyl.	2.52	0.96	0.45	5.33	1.87	0.45	7.12	2.89	0.64	15.69	6.7	1.57
		Sph.	1.94	0.77	0.42	3.64	1.49	0.4	5.43	2.41	0.6	12.94	5.75	1.44
	400	Cyl.	2.74	0.99	0.48	5.51	2.03	0.46	7.3	2.96	0.66	16	6.81	1.6
		Sph.	2.06	0.82	0.44	3.83	1.57	0.43	5.62	2.5	0.62	13.25	5.89	1.47
	1000	Cyl.	3.2	1.21	0.49	6.16	2.49	0.55	7.97	3.42	0.74	17.4	7.49	1.69
		Sph.	2.41	0.98	0.48	4.52	1.88	0.5	6.31	2.81	0.7	14.31	6.36	1.59

TABLE 4 DRAG FORCES (kN) ON REPAIR COMPONENTS

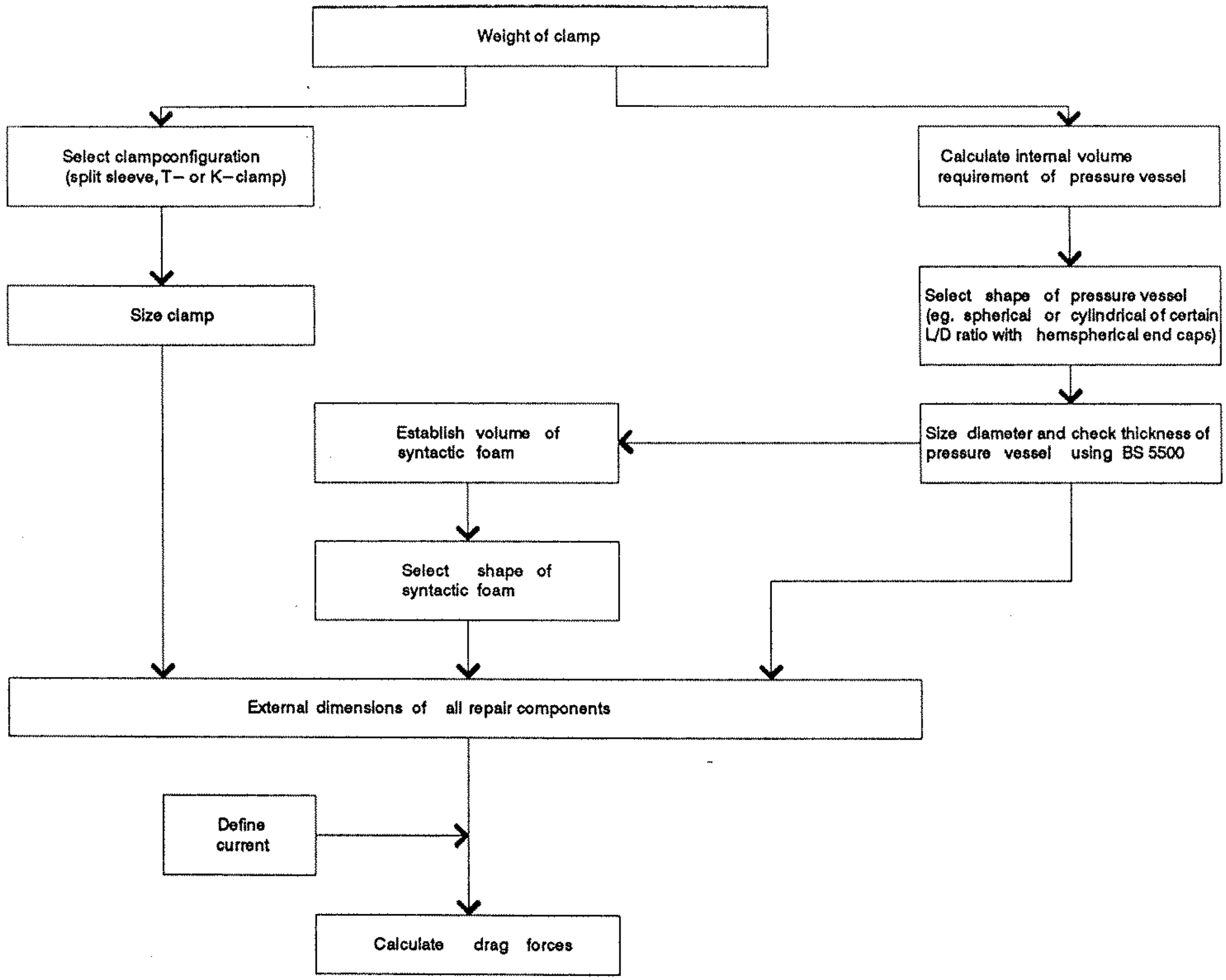

FIGURE 1: FLOWCHART OF CALCULATIONS

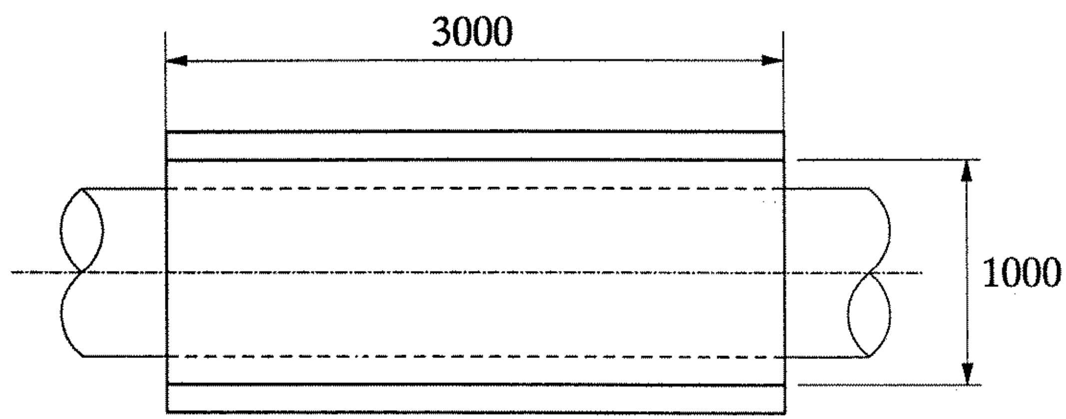

FIGURE 2: 2 TONNES SPLIT SLEEVE - OVERALL DIMENSIONS

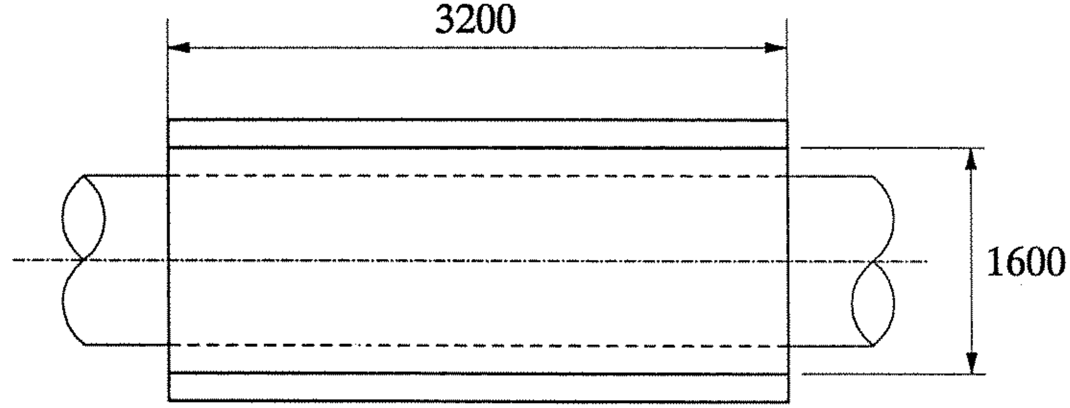

FIGURE 3: 5 TONNES SPLIT SLEEVE - OVERALL DIMENSIONS

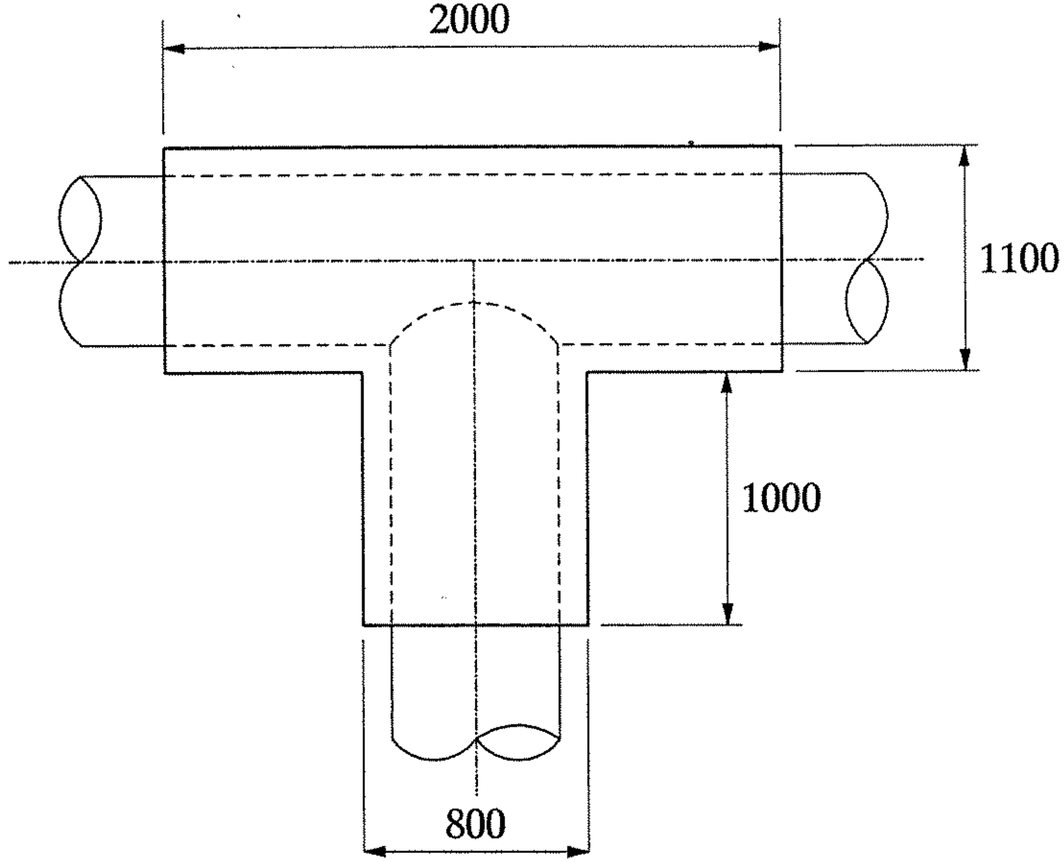

FIGURE 4: 5 TONNES T-CLAMP - OVERALL DIMENSIONS

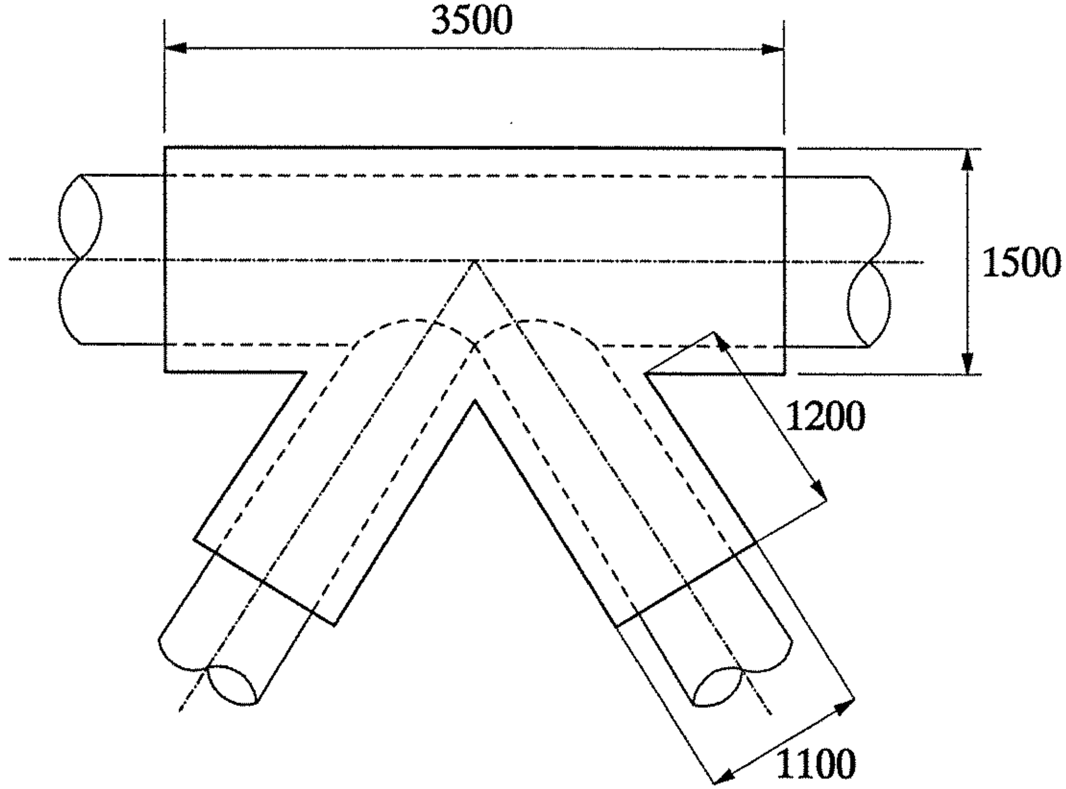

FIGURE 5: 10 TONNES K-CLAMP - OVERALL DIMENSIONS

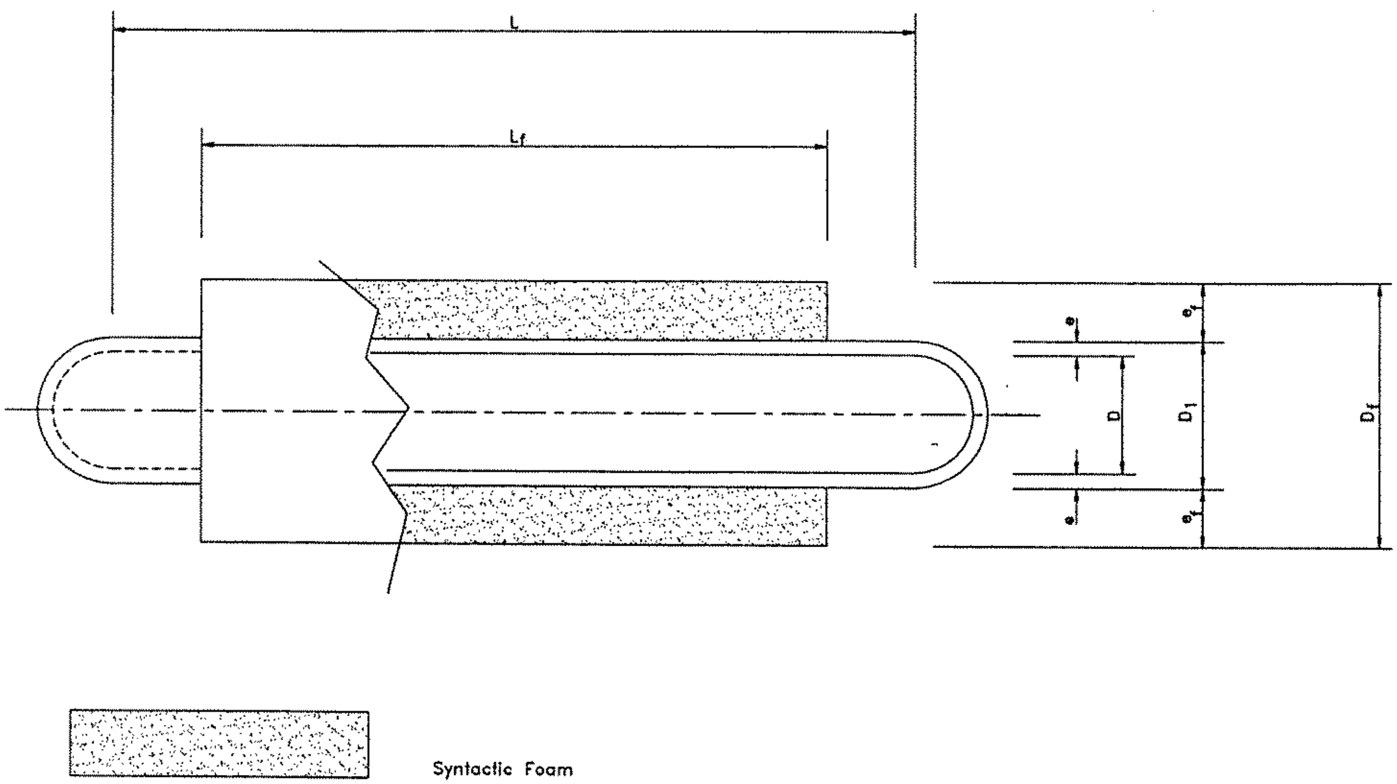

FIGURE 6: NOTATION FOR CYLINDRICAL PRESSURE VESSEL AND FOAM

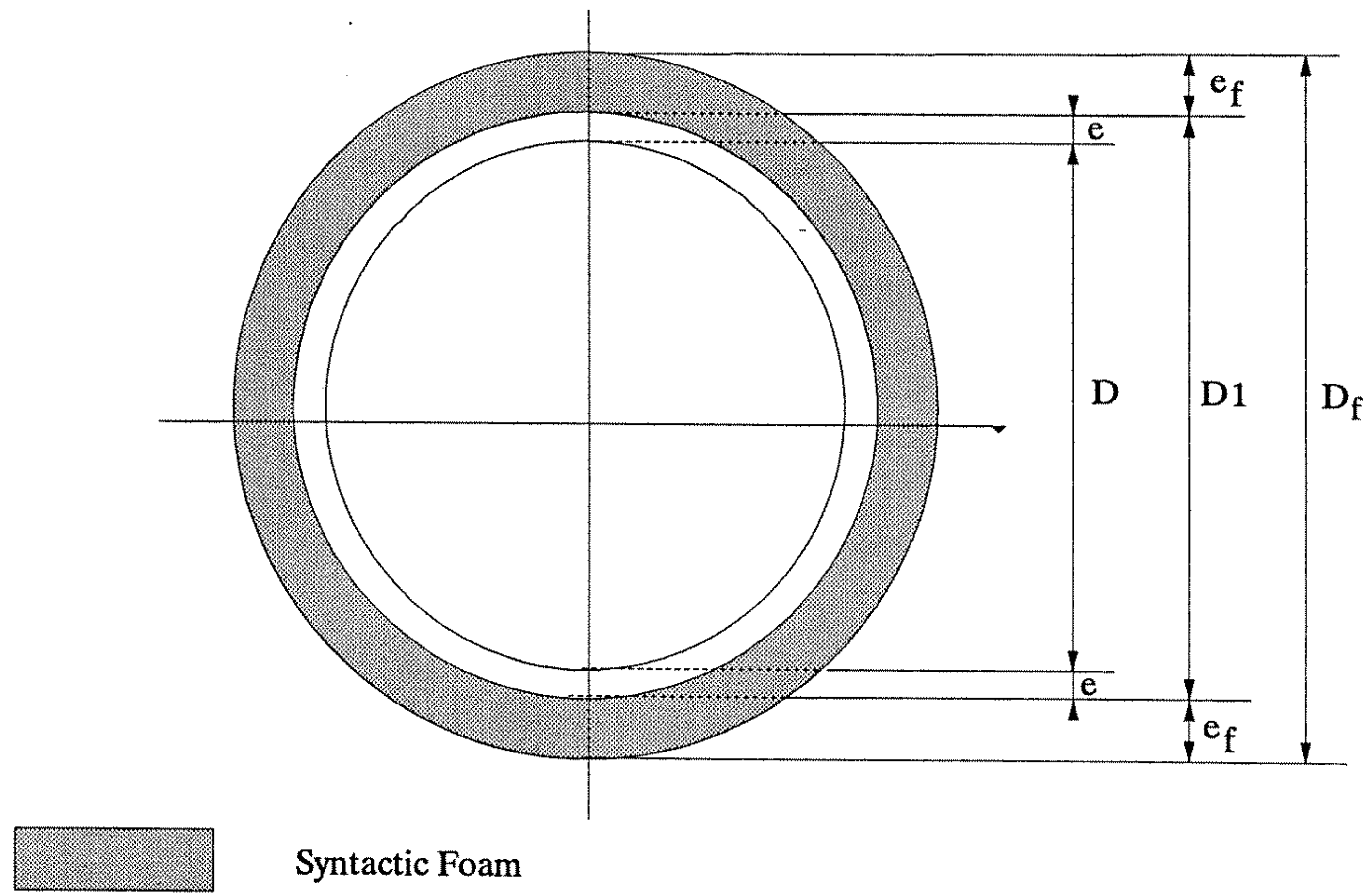

FIGURE 7: NOTATION FOR SPHERICAL PRESSURE VESSEL AND FOAM

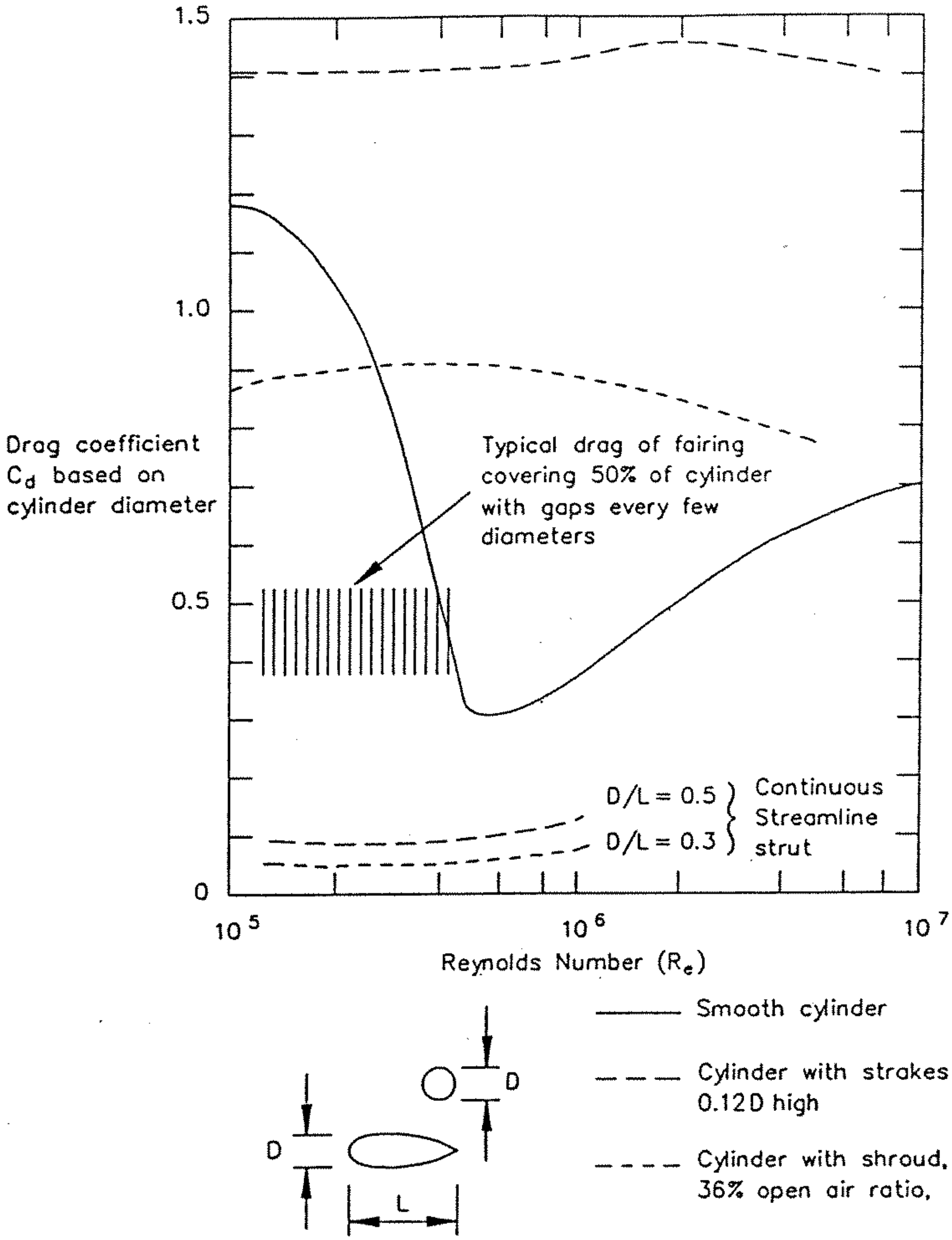

FIGURE 8: DRAG COEFFICIENT FOR A CIRCULAR CYLINDER OF INFINITE LENGTH TO DIAMETER RATIO [BARLTROP, 1991]

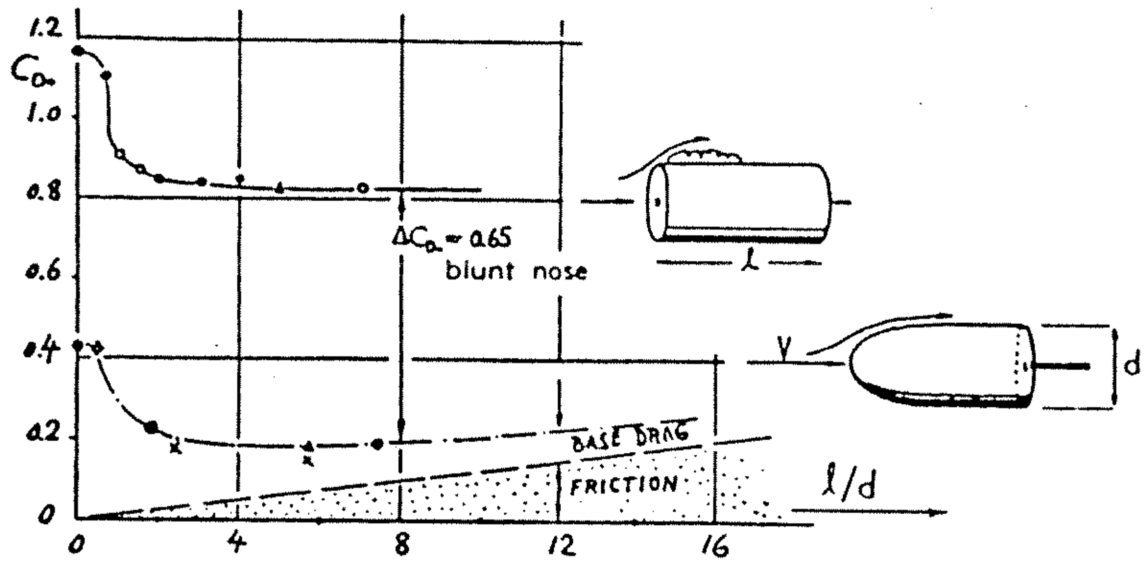

FIGURE 9: DRAG COEFFICIENTS FOR CYLINDRICAL BODIES IN AXIAL FLOW, WITH BLUNT SHAPE (IN THE UPPER PART) AND WITH ROUNDED OR STREAMLINED HEAD FORMS (LOWER PART) - AS A FUNCTION OF THE FINENESS RATIO L/D [HOERNER, 1965]

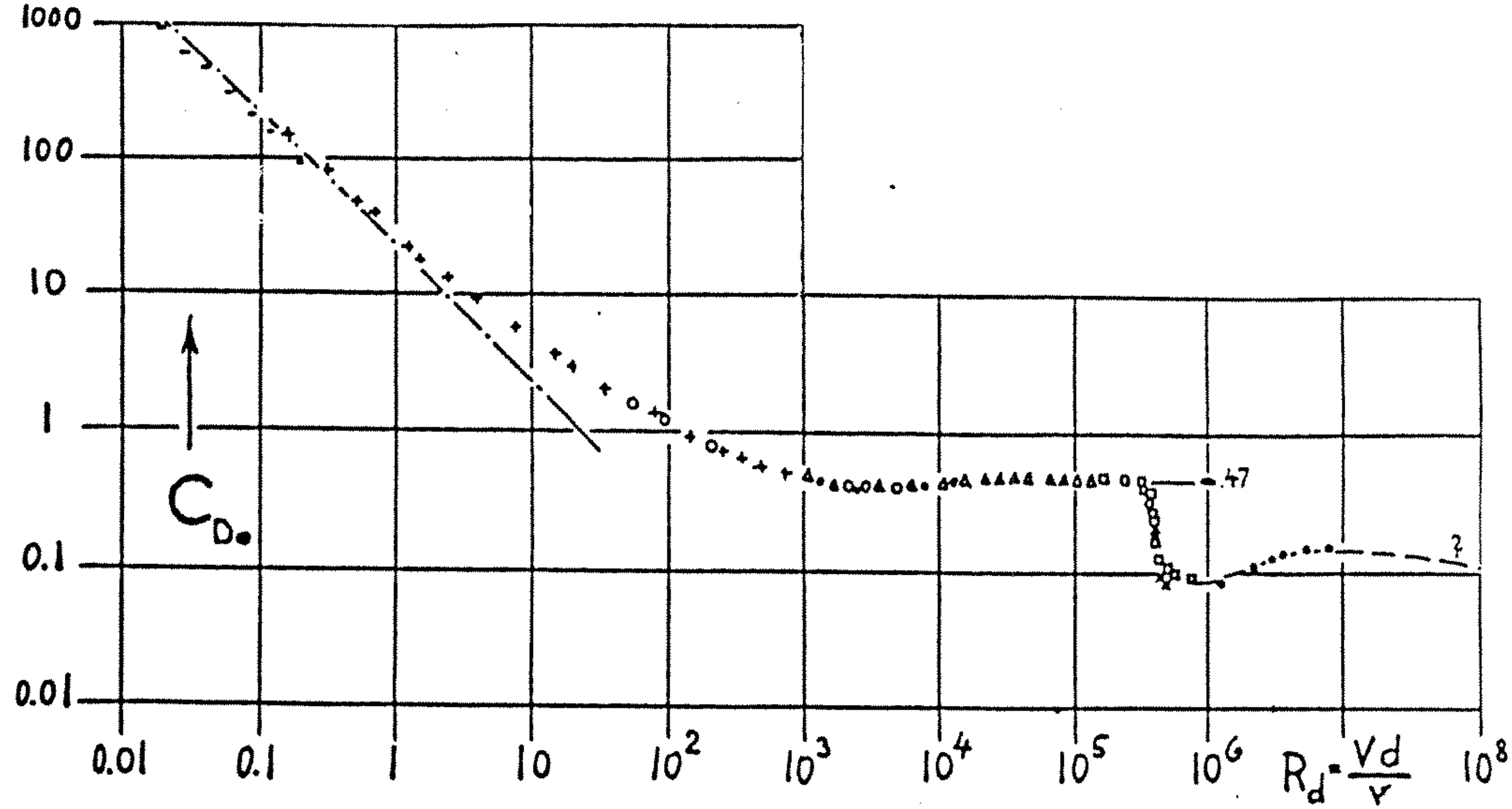

FIGURE 10: EXPERIMENTAL DRAG COEFFICIENTS OF THE SPHERE AS A FUNCTION OF REYNOLD'S NUMBER [HOERNER, 1965]

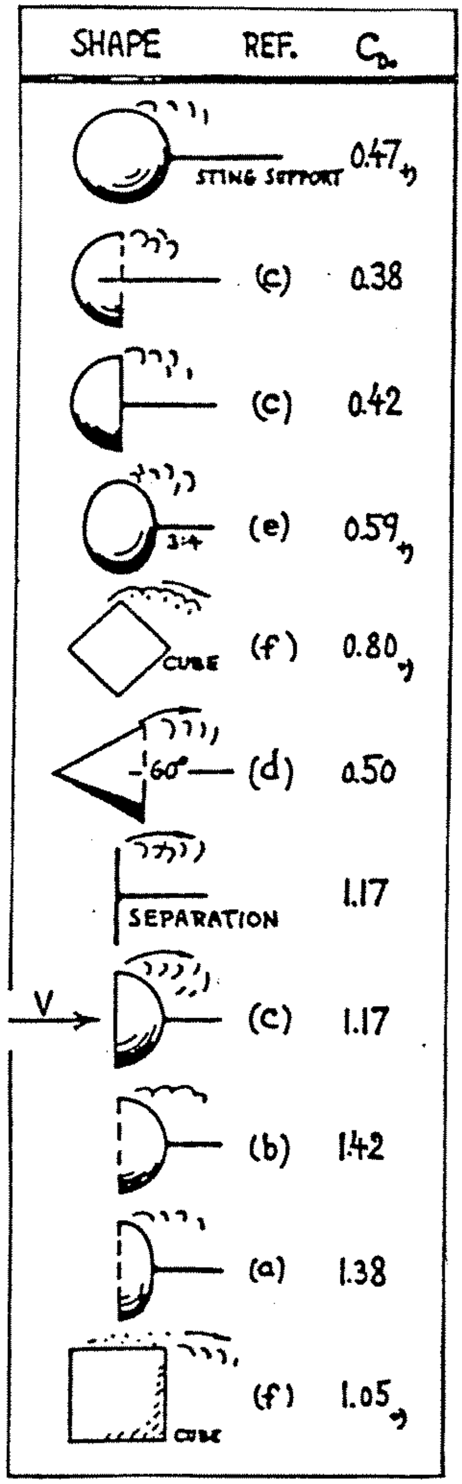

FIGURE 11: DRAG COEFFICIENTS OF VARIOUS 3-DIMENSIONAL BODIES AT REYNOLD'S NUMBERS BETWEEN 10^4 AND 10^6 [HOERNER, 1965]

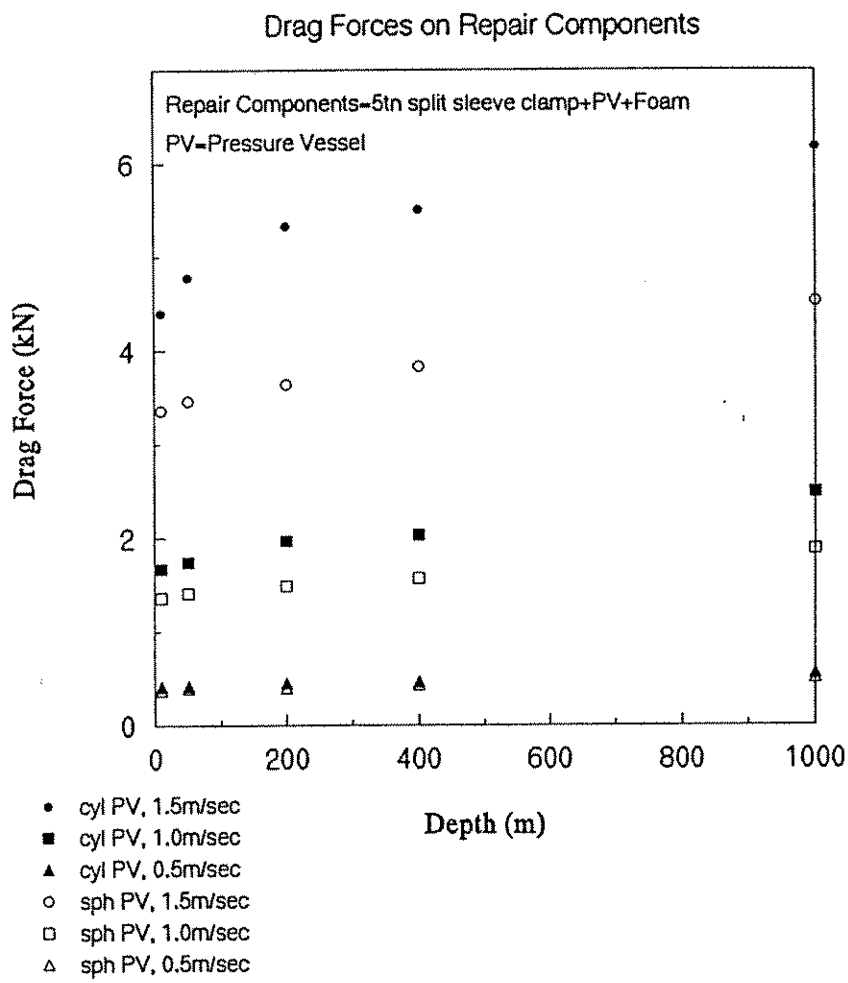

FIGURE 12: THE DRAG FORCES ON THE REPAIR COMPONENTS AS A FUNCTION OF DEPTH FOR DIFFERENT SHAPES OF PRESSURE VESSEL AND AT VELOCITIES OF 1.5, 1.0, 0.5M/SEC.

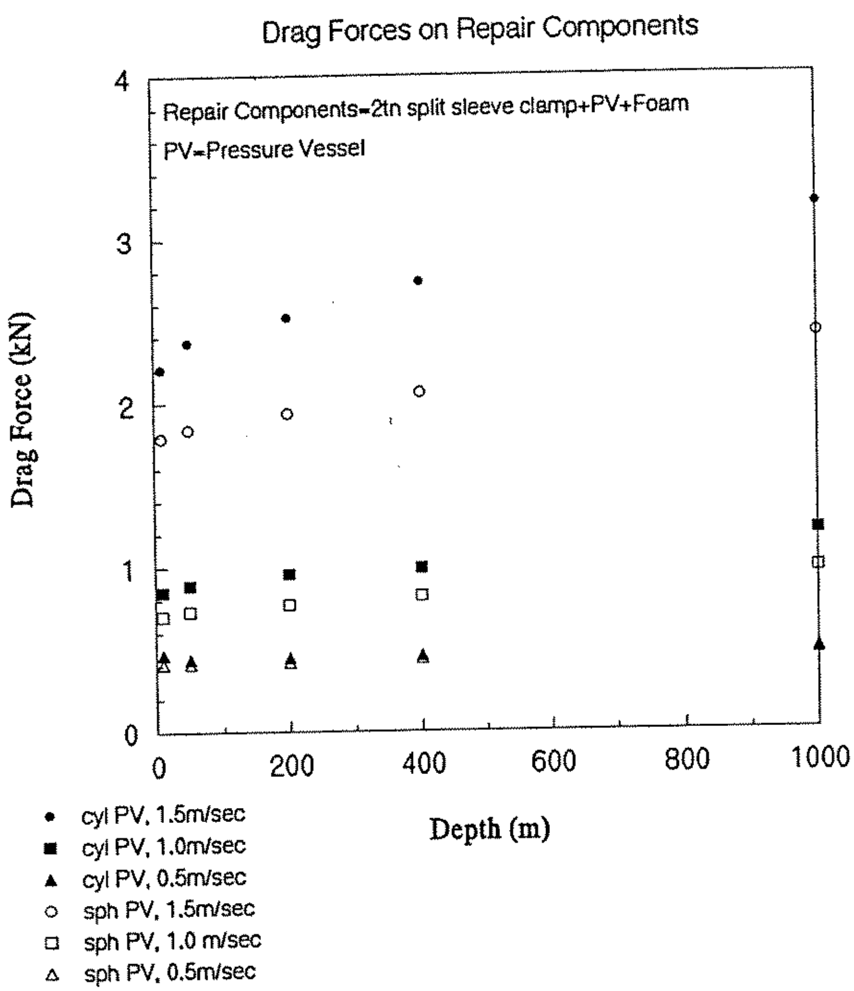

FIGURE 13: THE DRAG FORCES ON THE REPAIR COMPONENTS AS A FUNCTION OF DEPTH FOR DIFFERENT SHAPES OF PRESSURE VESSEL AND AT VELOCITIES OF 1.5, 1.0, 0.5M/SEC.

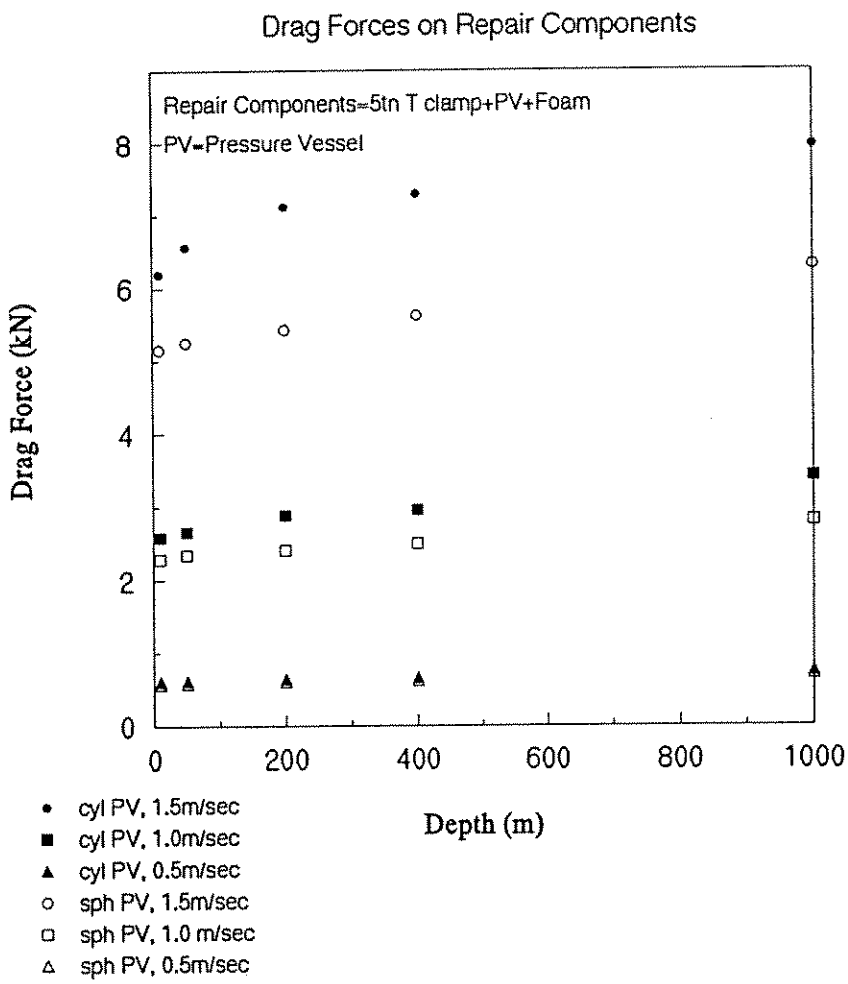

FIGURE 14: THE DRAG FORCES ON THE REPAIR COMPONENTS AS A FUNCTION OF DEPTH FOR DIFFERENT SHAPES OF PRESSURE VESSEL AND AT VELOCITIES OF 1.5, 1.0, 0.5M/SEC.

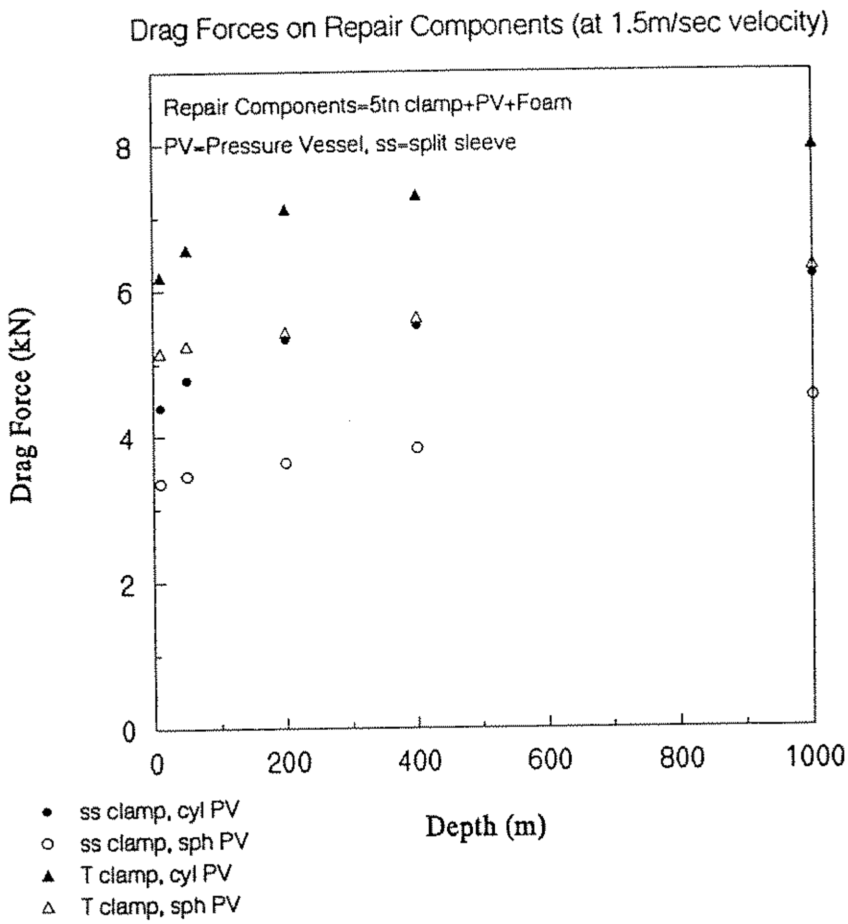

FIGURE 15: THE DRAG FORCES ON THE REPAIR COMPONENTS AS A FUNCTION OF DEPTH FOR DIFFERENT SHAPES OF CLAMP AND PRESSURE VESSEL AT 1.5 M/SEC VELOCITY

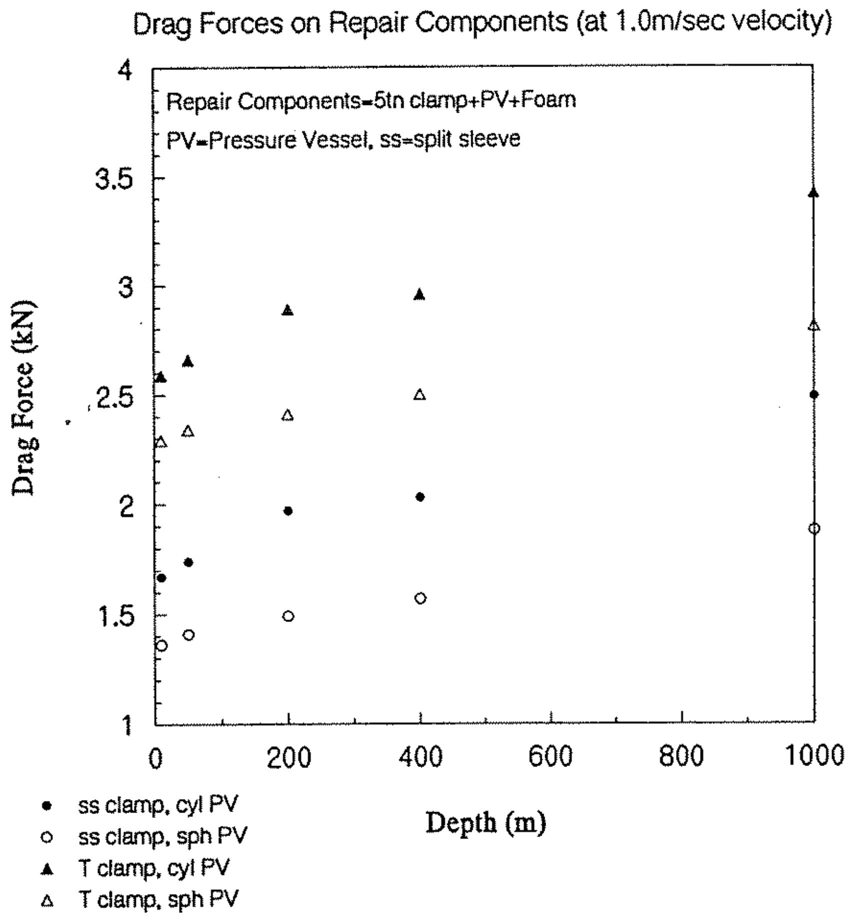

FIGURE 16: THE DRAG FORCES ON THE REPAIR COMPONENTS AS A FUNCTION OF DEPTH FOR DIFFERENT SHAPES OF CLAMP AND PRESSURE VESSEL AT 1.0M/SEC VELOCITY

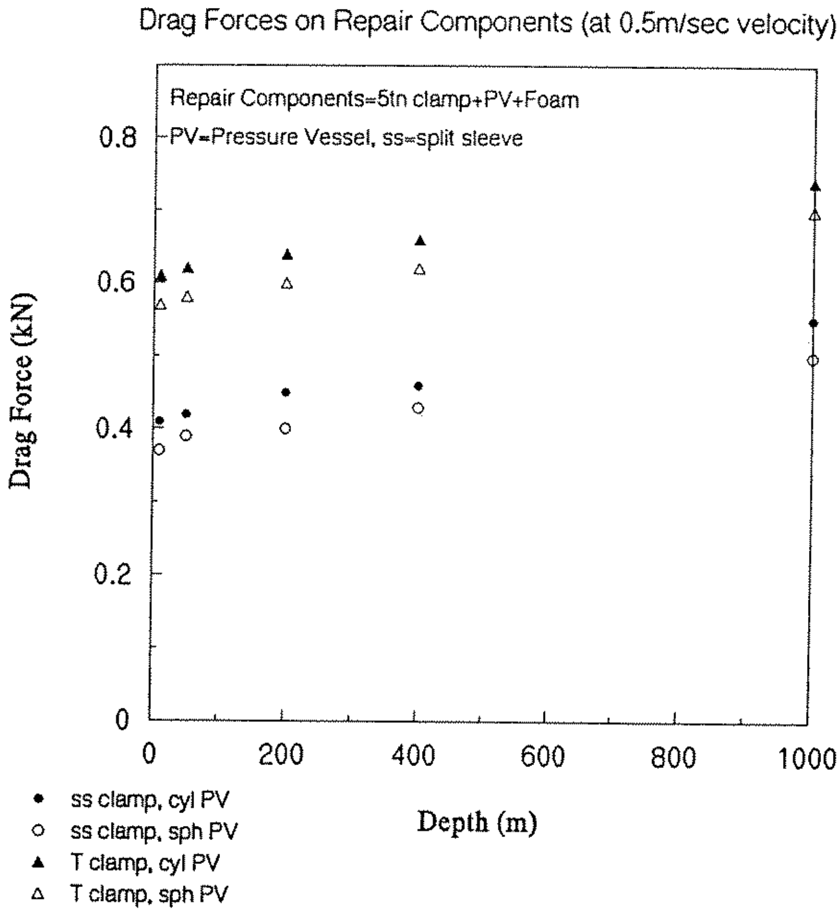

FIGURE 17: THE DRAG FORCES ON THE REPAIR COMPONENTS AS A FUNCTION OF DEPTH FOR DIFFERENT SHAPES OF CLAMP AND PRESSURE VESSEL AT 0.5M/SEC VELOCITY

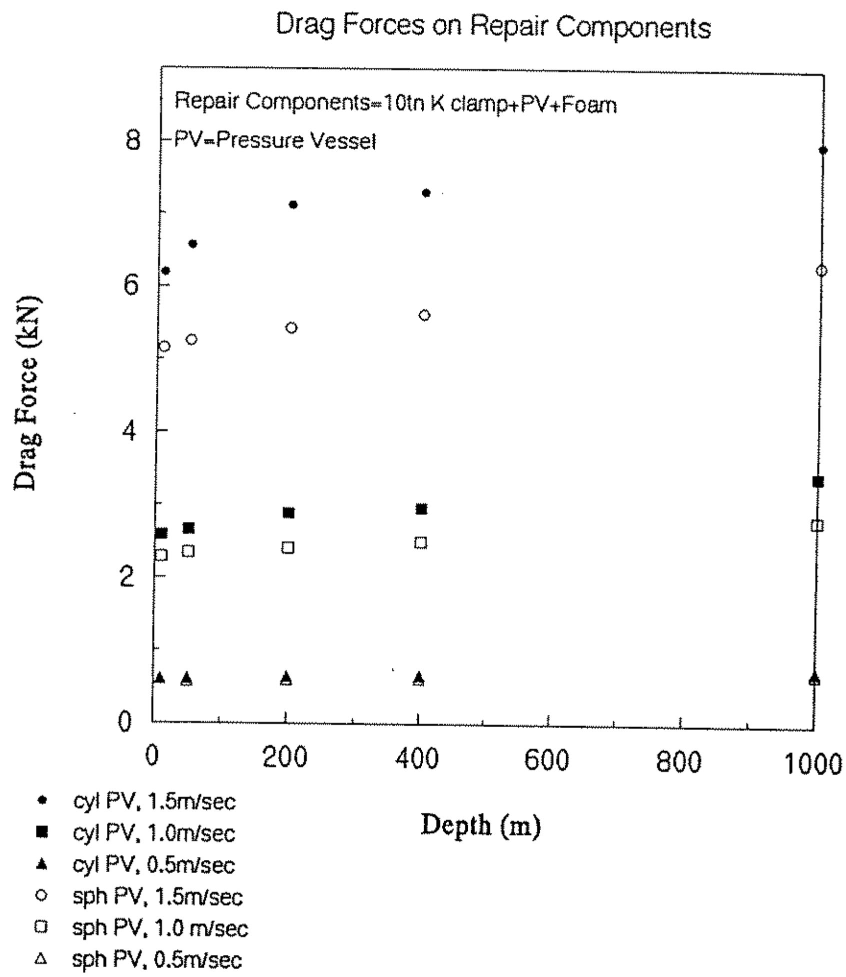

FIGURE 18: THE DRAG FORCES ON THE REPAIR COMPONENTS AS A FUNCTION OF DEPTH FOR DIFFERENT SHAPES OF PRESSURE VESSEL AND AT VELOCITIES OF 1.5, 1.0, 0.5M/SEC

APPENDIX

SAMPLE CALCULATIONS

Sizing of Repair Components

Neutral Bouyancy going down

Uplift = Up(foam)+Up(tank)-W(clamp)-W(tank)-W(foam)

where: Up(foam)=foam buoyancy

Up(tank)=buoyancy of the tank (excluding weight buoyancy)

W(clamp)=clamp weight (wet)

W(tank)=tank weight (wet)

W(foam)=foam weight (wet)

For Uplift=0, Up(foam)+Up(tank)=W(clamp)+W(tank)+W(foam) (1)

Neutral Buoyancy going up

Uplift = Up(foam)-W(tank)-W(foam)

For Uplift=0, Up(foam)=W(tank)+W(foam) (2)

From (1) & (2), Up(tank)=W(clamp) (3)

Cylindrical Pressure Vessel (tank)

From (3), 1.025*V(tank)=W(clamp)

1.025*{3.14*(D-2e)^2*L/4}=W(clamp) (3a)

where: D=tank diameter,

e=tank thickness,

L=tank length

Assume (D-2e)=D, L=mD and $Wc := 5$ t

From (3a), $1.025 \cdot \left(\pi \cdot \frac{D^2}{4}\right) \cdot m \cdot D = Wc$ $D := \left(\frac{Wc \cdot 4}{\pi \cdot m \cdot 1.025}\right)^{\frac{1}{3}}$

Spherical Pressure Vessel

From (3a), $1.025 \cdot \left[\frac{4}{3} \cdot \pi \cdot \left(\frac{D}{2}\right)^3\right] = Wc$, $D := \left(\frac{6 \cdot Wc}{\pi \cdot 1.025}\right)^{\frac{1}{3}}$, $D = 2.1042$ m

Design of Cylindrical Pressure Vessel

Diameter of Pressure Vessel

$m := 1$

$D := \left(\frac{4 \cdot Wc}{1.025 \cdot m \cdot \pi}\right)^{\frac{1}{3}}$ $\quad D = 1.8382 \;\; m$

Length of Pressure Vessel

$L := m \cdot D$ $\quad L = 1.8382 \;\; m$

Thickness of Pressure Vessel

$d := 200\ m$ $\quad p := d \cdot 1.025 \cdot 10^3 \cdot 9.81$ $\quad p = 2.0111 \cdot 10^6$ N/mm2

$fy := 325$ N/mm2 $\quad f := \frac{fy}{1.25}$ N/mm2

$e := \frac{p \cdot D}{2 \cdot f - p}$ $\quad e = 1.8377$ mm $\quad e := 0.020$ m

py: pressure at which the mean circumferential stress in a cylindrical shell midway between stiffeners reaches the yield point of the material

$s := 1.40$ (ferritic alloy steel)

$R := \frac{D}{2}$ $\quad py := \frac{s \cdot f \cdot e}{R}$ $\quad py = 7.9209$ N/mm2

$E := 210$ GPa

pm: elastic instability pressure for collapse of a cylindrical shell

$\frac{e}{2 \cdot R} = 0.0109$

$\frac{L}{2 \cdot R} = 1$ $\quad n := 5$

$Z := \frac{\pi \cdot R}{L}$

$\mu := 0.30$

$$\varepsilon := \frac{1}{n^2 - 1 + \frac{Z^2}{2}} \cdot \left[\frac{1}{\left(\frac{n^2}{Z^2} + 1\right)^2} + \frac{e^2}{12 \cdot R^2 \cdot (1 - \mu^2)} \cdot \left(n^2 - 1 + Z^2\right)^2\right] \qquad \varepsilon = 0.0015$$

$pm := \frac{E \cdot e \cdot \varepsilon \cdot 10^3}{R}$ $\quad pm = 6.9626$ N/mm2

$\frac{pm}{py} = 0.879$ $\quad con := 0.286$ $\quad p := con \cdot py$ $\quad p = 2.2654$ N/mm2

$d := \frac{p \cdot 10^6}{1.025 \cdot 10^3 \cdot 9.81}$ $\quad d = 225.2937$ m

MST.

Sizing of foam - Cylindrical Pressure Vessel

From (2), V(steel)*(7.835-1.025) + V(foam)*(0.7-1.025) = 0

V(steel)*6.81 - V(foam)*0.325 = 0

V(foam) = V(steel)*6.81/0.325 (4)

Assume the cylindrical tank to be wrapped with foam

For $d = 225.2937$ m $D = 1.8382$ m $e := 0.020$ m $L = 1.8382$ m

$D1 := D + 2 \cdot e$ $D1 = 1.8782$ m

$$Vsteel := \pi \cdot \frac{D1^2 - D^2}{4} \cdot L \qquad Vsteel = 0.2146 \text{ m3}$$

From (4), $$Vfoam := \frac{Vsteel \cdot 6.81}{0.325} \qquad Vfoam = 4.4969 \text{ m3}$$

Assume $Lf := 1.8$ m

V(foam)=3.14*(Df^2 - D1^2)*Lf/4

Df = D1+ 2*ef

$$ef := \frac{\sqrt{\frac{4 \cdot Vfoam}{\pi \cdot Lf} + D1^2} - D1}{2} \qquad ef = 0.3559 \text{ m}$$

$$Df := D1 + 2 \cdot ef \qquad Df = 2.5901 \text{ m}$$

Foam Specification

Outer diameter $Df = 2.5901 \cdot m$

Thickness $ef = 0.3559 \cdot m$

Length $Lf = 1.8 \cdot m$

Design of Spherical Pressure Vessel

Diameter of Pressure Vessel

$D = 2.10$ m

Thickness of Pressure Vessel

d 10 m p $d \cdot 1.025$ t/m2

$fy = 325$ N/mm2 $f = \frac{fy}{1.25}$ (N/mm2)

$e = 0.010$ m

pyss: pressure at which the membrane stress in a spherical shell
reaches the yield point of the material

$s = 1.40$ (ferritic alloy steel)

$R = \frac{D}{2}$ $pyss := \frac{2 \cdot s \cdot f \cdot e}{R}$ $pyss = 6.933$ N/mm2

$E = 210$ GPa

pe: elastic instability pressure for collapse of a spherical shell

$pe = \frac{1.21 \cdot E \cdot 10^3 \cdot e^2}{R^2}$ $pe = 23.048$ N/mm2

$\frac{pe}{pyss} = 3.324$ $con = 0.309$ $pe := con \cdot pyss$ $pe = 2.142$ N/mm2

$d = \frac{pe \cdot 10^6}{1.025 \cdot 10^3 \cdot 9.81}$ $d = 213.063$ m

Sizing of foam - Spherical Pressure Vessel

From (2), V(steel)*(7.835-1.025) + V(foam)*(0.7-1.025) = 0

V(steel)*6.81 - V(foam)*0.325 = 0

V(foam) = V(steel)*6.81/0.325 (5)

Assume the cylindrical tank to be wrapped with foam

For $d := 213.063\ m$ $D := 2.10\ m$ $e := 0.010\ m$

$D1 := D + 2 \cdot e$ $D1 = 2.12\ m$

$Vsteel := \frac{4}{3} \cdot \pi \cdot \left(\frac{D1^3 - D^3}{8} \right)$ $Vsteel = 0.1399$ m3

From (5), $Vfoam := \frac{Vsteel \cdot 6.81}{0.325}$ $Vfoam = 2.9308$ m3

V(foam)=4/3*3.14*((Df - D1)/2)^3

Df = D1+ 2*ef

$ef := \frac{\left(\frac{6 \cdot Vfoam}{\pi} + D1^3 \right)^{\frac{1}{3}} - D1}{2}$ $ef = 0.1765\ m$

$Df := D1 + 2 \cdot ef$ $Df = 2.4731\ m$

Foam Specification

Outer diameter $Df = 2.4731 \cdot m$

Thickness $ef = 0.1765 \cdot m$

Hydrodynamic Forces

Split - sleeve 1600 ϕ x 40 thick x 3.2 m

$D := 1.6$ m $\quad e := 0.04$ m $\quad L := 3.2$ m

Reynolds number

$v1 := 1.5$ m/sec $\quad Re1 := 5.5 \cdot 10^5 \cdot v1 \cdot D \quad Re1 = 1.32 \cdot 10^6$

$v2 := 1$ m/sec $\quad Re2 := 5.5 \cdot 10^5 \cdot v2 \cdot D \quad Re2 = 8.8 \cdot 10^5$

$v3 := 0.5$ m/sec $\quad Re3 := 5.5 \cdot 10^5 \cdot v3 \cdot D \quad Re3 = 4.4 \cdot 10^5$

Drag calculation

Fd = 0.5*Cd*p*A*v2

where: Cd = drag coefficient

p = density of salty water $\quad \rho := 1.025$ t/m2

A = member dimension

v = velocity of flow resolved normal to the member

Flow perpendicular to axis

$A := D \cdot L \quad A = 5.12$ m2

Drag coefficient

Hoerner '65, cylinder
(flow perpendicular to axis)

For $v1 := 1.5$ m/sec $\quad Cd1 := 0.40$

$v2 := 1$ m/sec $\quad Cd2 := 0.35$

$v3 := 0.5$ m/sec $\quad Cd3 := 0.40$

For $v1 := 1.5$ m/sec $\quad Fd1ss := 0.5 \cdot Cd1 \cdot \rho \cdot A \cdot v1^2 \quad Fd1ss = 2.3616$ kN

$v2 := 1$ m/sec $\quad Fd2ss := 0.5 \cdot Cd2 \cdot \rho \cdot A \cdot v2^2 \quad Fd2ss = 0.9184$ kN

$v3 := 0.5$ m/sec $\quad Fd3ss := 0.5 \cdot Cd3 \cdot \rho \cdot A \cdot v3^2 \quad Fd3ss = 0.2624$ kN

Flow along the axis

$A := \pi \cdot \frac{D^2}{4} \quad A = 2.0106$ m2

Drag coefficient

Hoerner '65, cylinder
(flow along axis)

For $\frac{L}{D} = 2 \quad Cd := 0.22$

For $v1 := 1.5$ m/sec $\quad Fd1 := 0.5 \cdot Cd \cdot \rho \cdot A \cdot v1^2 \quad Fd1 = 0.5101$ kN

$v2 := 1$ m/sec $\quad Fd2 := 0.5 \cdot Cd \cdot \rho \cdot A \cdot v2^2 \quad Fd2 = 0.2267$ kN

$v3 := 0.5$ m/sec $\quad Fd3 := 0.5 \cdot Cd \cdot \rho \cdot A \cdot v3^2 \quad Fd3 = 0.0567$ kN

Hydrodynamic Forces

T-clamp Stressed Grouted

Reynolds number (sea water at zero degrees Celsius)

$a := 2$ m $b := 1.1$ m $c := .8$ m $d := 1$ m

$l := \sqrt{a \cdot b - c \cdot d}$ $l = 1.7321$ m

$v1 := 1.5$	m/sec	$Re1 := 5.5 \cdot 10^5 \cdot v1 \cdot l$	$Re1 = 1.4289 \cdot 10^6$
$v2 := 1$	m/sec	$Re2 := 5.5 \cdot 10^5 \cdot v2 \cdot l$	$Re2 = 9.5263 \cdot 10^5$
$v3 := 0.5$	m/sec	$Re3 := 5.5 \cdot 10^5 \cdot v3 \cdot l$	$Re3 = 4.7631 \cdot 10^5$

Drag calculation

Fd = 0.5*Cd*p*A*v2

where: Cd = drag coefficient $Cd := 1.20$ Hoerner '65 2-D flow, square

p = density of salty water $\rho := 1.025$ t/m2

A = member dimension $A := a \cdot b + c \cdot d$

v = velocity of flow resolved normal to the member

For	$v1 := 1.5$	m/sec	$Fd1t := 0.5 \cdot Cd \cdot \rho \cdot A \cdot v1^2$	$Fd1t = 4.1512$	kN
	$v2 := 1$	m/sec	$Fd2t := 0.5 \cdot Cd \cdot \rho \cdot A \cdot v2^2$	$Fd2t = 1.845$	kN
	$v3 := 0.5$	m/sec	$Fd3t := 0.5 \cdot Cd \cdot \rho \cdot A \cdot v3^2$	$Fd3t = 0.4612$	kN

Spherical Pressure Vessel + Foam

$Df := 2.4731$ m

Reynolds number

$v1 := 1.5$ m/sec $\quad Re1 := 5.5 \cdot 10^5 \cdot v1 \cdot Df \quad Re1 = 2.0403 \cdot 10^6$

$v2 := 1$ m/sec $\quad Re2 := 5.5 \cdot 10^5 \cdot v2 \cdot Df \quad Re2 = 1.3602 \cdot 10^6$

$v3 := 0.5$ m/sec $\quad Re3 := 5.5 \cdot 10^5 \cdot v3 \cdot Df \quad Re3 = 6.801 \cdot 10^5$

Drag calculation

Fd = 0.5*Cd*p*A*v2

where: Cd = drag coefficient

p = density of salty water $\quad \rho := 1.025$ t/m2

A = member dimension $\quad A := \pi \cdot \frac{Df^2}{4} \quad A = 4.8037$ m2

v = velocity of flow resolved normal to the member

Drag coefficient

Hoerner '65, sphere
(3-D flow)

$Cd := 0.23$

For $v1 := 1.5$ m/sec $\quad Fd1sph := 0.5 \cdot Cd \cdot \rho \cdot A \cdot v1^2 \quad Fd1sph = 1.274$ kN

$v2 := 1$ m/sec $\quad Fd2sph := 0.5 \cdot Cd \cdot \rho \cdot A \cdot v2^2 \quad Fd2sph = 0.5662$ kN

$v3 := 0.5$ m/sec $\quad Fd3sph := 0.5 \cdot Cd \cdot \rho \cdot A \cdot v3^2 \quad Fd3sph = 0.1416$ kN

Cylindrical Pressure Vessel +Foam

$Df := 2.5901$ m $\qquad L := 1.8382$ m

Reynolds number

$v1 := 1.5$	m/sec	$Re1 := 5.5\cdot10^5\cdot v1\cdot Df$	$Re1 = 2.1368\cdot10^6$
$v2 := 1$	m/sec	$Re2 := 5.5\cdot10^5\cdot v2\cdot Df$	$Re2 = 1.4246\cdot10^6$
$v3 := 0.5$	m/sec	$Re3 := 5.5\cdot10^5\cdot v3\cdot Df$	$Re3 = 7.1228\cdot10^5$

Drag calculation

Fd = 0.5*Cd*p*A*v2

where: Cd = drag coefficient

p = density of salty water $\qquad \rho := 1.025$ t/m2

A = member dimension

v = velocity of flow resolved normal to the member

Flow perpendicular to axis

$A := Df\cdot L \qquad A = 4.7611$ m2

Drag coefficient

Hoerner '65, cylinder
(flow perpendicular to axis)

For	$v1 := 1.5$	m/sec	$Cd1 := 0.54$
	$v2 := 1$	m/sec	$Cd2 := 0.43$
	$v3 := 0.5$	m/sec	$Cd3 := 0.30$

For	$v1 := 1.5$	m/sec	$Fd1c := 0.5\cdot Cd1\cdot\rho\cdot A\cdot v1^2$	$Fd1c = 2.9647$	kN
	$v2 := 1$	m/sec	$Fd2c := 0.5\cdot Cd2\cdot\rho\cdot A\cdot v2^2$	$Fd2c = 1.0492$	kN
	$v3 := 0.5$	m/sec	$Fd3c := 0.5\cdot Cd3\cdot\rho\cdot A\cdot v3^2$	$Fd3c = 0.183$	kN

Flow along the axis

$A := \pi\cdot\frac{Df^2}{4} \qquad A = 5.2689$ m2

Drag coefficient

Hoerner '65, cylinder
(flow along axis)

For $\frac{L}{Df} = 0.7097 \qquad Cd := 0.32$

For	$v1 := 1.5$	m/sec	$Fd1 := 0.5\cdot Cd\cdot\rho\cdot A\cdot v1^2$	$Fd1 = 1.9442$	kN
	$v2 := 1$	m/sec	$Fd2 := 0.5\cdot Cd\cdot\rho\cdot A\cdot v2^2$	$Fd2 = 0.8641$	kN
	$v3 := 0.5$	m/sec	$Fd3 := 0.5\cdot Cd\cdot\rho\cdot A\cdot v3^2$	$Fd3 = 0.216$	kN

MST

Thrust Capacity required by ROVs

$d := 200\ m$

T-clamp + Cylindrical Pressure Vessel

$Fd1t + Fd1c = 7.1159$	kN	$v1 = 1.5$	m/sec
$Fd2t + Fd2c = 2.8942$	kN	$v2 = 1$	m/sec
$Fd3t + Fd3c = 0.6443$	kN	$v3 = 0.5$	m/sec

Split sleeve + Cylindrical Pressure Vessel

$Fd1ss + Fd1c = 5.3263$	kN	$v1 = 1.5$	m/sec
$Fd2ss + Fd2c = 1.9676$	kN	$v2 = 1$	m/sec
$Fd3ss + Fd3c = 0.4454$	kN	$v3 = 0.5$	m/sec

T-clamp + Spherical Pressure Vessel

$Fd1t + Fd1sph = 5.4253$	kN	$v1 = 1.5$	m/sec
$Fd2t + Fd2sph = 2.4112$	kN	$v2 = 1$	m/sec
$Fd3t + Fd3sph = 0.6028$	kN	$v3 = 0.5$	m/sec

Split-sleeve + Spherical Pressure Vessel

$Fd1ss + Fd1sph = 3.6356$	kN	$v1 = 1.5$	m/sec
$Fd2ss + Fd2sph = 1.4846$	kN	$v2 = 1$	m/sec
$Fd3ss + Fd3sph = 0.404$	kN	$v3 = 0.5$	m/sec

MSL

APPENDIX B

SCENARIOS

MSL

Purpose of Issue	Rev	Date of Issue	Author	Agreed	Approved
Issued to Participants	0	September 1993	Various		

JIP - STRENGTHENING, MODIFICATION AND REPAIR TECHNIQUES

WORK PACKAGES SMR 15: DEEPWATER PREMISE

TECHNICAL REPORT NO. 14
DEEPWATER REPAIR PREMISE

DOC REF C11100R232 Rev 0 SEPTEMBER 1993

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NUMBER	DETAILS OF REVISION
0	Issued to Participants, September 1993

JIP - STRENGTHENING, MODIFICATION AND REPAIR TECHNIQUES FOR SHALLOW WATER AND DEEPWATER OFFSHORE PLATFORMS

TECHNICAL REPORT NO. 14
DEEPWATER REPAIR PREMISE

CONTENTS

1. INTRODUCTION

1.1 General

One of the main aims of the current phase of the Joint Industry Project on repairs is to investigate the feasibility of and to develop appropriate diverless implementation techniques for conducting underwater strengthening/repair (S/R) operations. In order to develop suitable strategies, the circumstances of the requirements for the underwater operations need to be specified. It is the intention of this report to define these and specifically:

- the geometry of the offshore structure
- a possible event leading to a requirement for S/R
- an outline of the techniques to be used in the S/R scheme.

All three items above are described in the following sections to a level sufficient to give a clear indication of the scenario to be addressed. The project steering committee (PSC) are invited to comment on the proposed premise.

1.2 Scope of Work

The diverless implementation techniques are to be based on the repair of a face frame element of a deepwater jacket structure at a depth of 1000 feet (305 metres). To cater for the desirability that the developed techniques offer the widest flexibility in addressing potential problems, the PSC advised that the repair scheme should consist of incorporating a new member (an addmember) into the structure by way of two clamps. Recognising the difficulties that may be associated with developing new technology, the PSC further advised that the clamps should connect with existing tubular members, rather than joint nodes. The PSC also requested that internal grout-filling of a horizontal member be considered; either as part of or separate to the addmember/clamp scheme.

2. SELECTION OF PLATFORM

It is important to choose a platform design which is representative of the type of installation which may require repairing at depth a depth of 1000 feet. Key factors are the size and spacing of structural elements and the associated local structural topology. It would also be advantageous if the sizes of elements selected were similar to those which are in use in platforms in shallower waters.

The structure would need to stand in at least 1000 feet of water, and be of steel tubular construction, relying on driven piles to resist overturning moments.

There is a small number of deepwater platforms installed in the coastal waters of the USA. These structures have typically been installed as barge-launched jackets, requiring two or three jacket pieces to be connected underwater. The structures typically use a large number of relatively small diameter element sizes to form the structural frame. Piles may often be installed concentrically in all the jacket legs, or be regularly distributed around the structural base perimeter.

In the North Sea, deepwater structures take on a different form, with concrete gravity base structures, tension leg platforms and steel self-floating towers proposed and used for the deepest water applications. The deepest steel piled North Sea structure stands in just over 600 feet of water - the BP Magnus tower. North Sea structures do tend to use a different structural form compared to those in use in American waters. The European preference is for corner cluster pile groups with the jacket or tower frame consisting of a small number of relatively large diameter elements. North Sea platforms tend to have high deck loads (25,000 tonnes plus is not uncommon) needing large diameter leg sections which then require heavily stiffened leg nodes underwater.

It follows that an accommodation should be made for the regional variation in deepwater structural designs. The accommodation should take account of the variation between the two major structural forms in current use for deepwater platforms, and yet address the possibility that operators may be interested in the diverless repair of elements at a depth less than 1000 ft.

Deepwater platforms may have structural members having external ring stiffeners to resist the hydrostatic pressure, the stiffeners being typically spaced at about four times the member diameter. Generally, the incorporation of a clamp on such a member would necessarily involve the removal of one or more of the ring stiffeners. To prevent member collapse, the member would require flooding before ring removal. However, since the removal of ring stiffeners is not part of the scope of work as discussed, ring stiffened members are not addressed further in this document.

Figure 1 shows the outline of the proposed study structure. The damaged structural element is at a depth of 1000 feet.

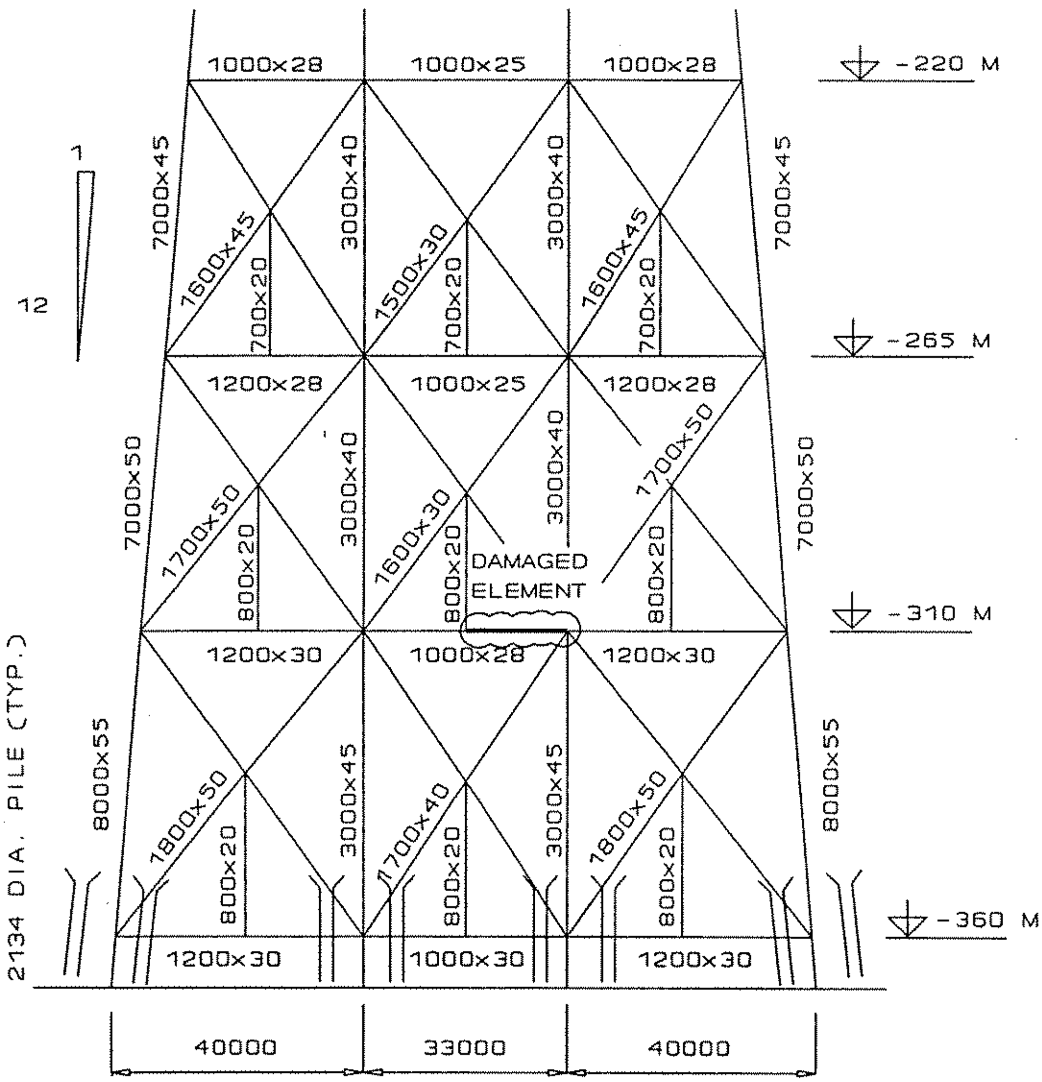

Figure 1 : The Jacket Layout - Lower Section

3. DAMAGE

3.1 Scenario

The step of choosing a damage scenario is required because the complete design study requires an anchor point for cause and effect before considering the available repair options. For example, if the S/R scheme is to consider the use of a steel clamp to fix the damaged element, then it will be necessary to look at the circumstances of the damage in order to calculate the ovality of the structural element to be repaired, as this data will be required to design the repair clamp. This and other points of detail required for design can be developed by describing a self-consistent scheme.

The scope of work demands that the damage be to a primary structural element on one of the external structural frames. The structural element will be at a depth of approximately 1000 feet.

The following scenarios may be considered viable:

1. An object is dropped from the topsides or from a supply boat in the vicinity of the platform, causing denting and possibly bending of the element, both actions being applied in a predominantly vertical direction.

2. The structure suffers a collision from a submarine, causing denting and bending by a force applied towards the centre of the structure.

3. The element has been damaged by hydrostatic forces, triggered by tensile storm loads (interactive tensile-hydrostatic collapse).

4. The element has been damaged by hydrostatic forces triggered by fabrication errors or by corrosion.

5. The element has been damaged by the build-up of drilling cuttings - either by impact during installation over a pre-drilled template or by a slow accumulation of weight on an element.

6. The element has suffered a fatigue crack in a butt weld in the member mid-span.

Of the six suggested scenarios, it is believed that scenario (1) is most likely to happen and to result in damage requiring repair. Items are regularly dropped from offshore platforms and when dropped, they can cause serious damage to the structural assembly below. The following mitigation is proposed for the other scenarios:

Scenario 2 — Submarines tend to operate at relatively shallow water depths and even though there have been reports of damage by submarine collision [1], such an event is considered rare.

Scenario 3 This type of damage has never been reported and although theoretically possible, it requires the combination of a design error and a suitably high storm load with the correct directionality to produce tension in the brace. The brace must be subsequently inspected and found to be damaged. Note that with scenario (1) or (2), the structure would automatically be inspected because a damaging event had occurred.

Scenario 4 Hydrostatic collapse damage can be a serious problem and is most likely to occur at depths of the order of 1000 ft. The type of repair associated with such damage may be the sleeved repair of the element or the removal and replacement of the element requiring a nodal clamp. This repair is outside the scope of work for the study.

Scenario 5 The damage described in this scenario would require the removal of soil, cement or cuttings from near the mudline and as such represents a repair which is outside the scope of work for the present study.

Scenario 6 The fatigue crack would need to be found by an inspection requiring cleaning of the element. The repair may take the form of a grouted sleeve.

3.2 Assessment of Impact Damage

It is only possible to make an accurate assessment of impact damage caused by the energy of a dropped object for a particular element size - because the diameter, wall thickness and unsupported length of the element influences whether it is damaged by denting, bending or tearing. For a given element size, typical damage scenarios can be developed by considering different dropped objects. Each dropped object has a mass and velocity, which defines the kinetic energy which may be released in an impact. The structural response to impact has been calculated using the impact analysis program "IMPOS" [2] which charts the element impact history until all the impact energy is absorbed by structural actions.

There are many items of equipment on board an offshore platform. Moveable items of equipment, supplies or material handling equipment can all be dropped into the water. Initial hand calculations have shown that large bluff objects such as containers or skids are unlikely to reach high velocities in water.

The following table gives details of some objects which could possibly be dropped, and may do damage to the structure below.

Object	Mass kg	Dim X	Dim Y	Dim Z
Pipe lifting tool	655	800	800	700
30" valve	2300	930	930	1500
Pipe bundle	9500	1200	1200	10000
Anchor 5 tonnes	5300	400	300	2400
Anchor 10 tonnes	10000	600	300	3000

The terminal velocity of an object falling in water can be estimated from the mass, volume, frontal area and shape. The shape of the object affects the drag on the falling object. The terminal velocities of the objects listed above have been calculated and are indicated in the following table along with the associated kinetic energies.

Object	Mass in kg	Velocity in water m/sec	Kinetic Energy kJ
Pipe lifting tool	655	5.3	9.2
30" valve	2300	10.0	115
Pipe bundle	9500	6.7	213
Anchor 5 tonnes	5300	9.0	215
Anchor 10 tonnes	10000	9.4	442

Typical energy levels required to seriously damage tubular steel elements lie in the range 100 to 2000 kJ with the higher energy level required to damage a jacket leg section.

A detailed examination of the circumstances of the impact of dropped objects has been made. The program IMPOS [2] was used to check the type and extent of damage which would be incurred by an impact.

The analysis concentrated upon the behaviour of a 1000 dia by 28 thick structural element of length 25m, effectively fixed at its ends. A 10 tonne anchor impact gave the following predicted results:

Dent depth	85.5mm
Flattened area width	559 mm
Permanent plastic width	25 mm

The analysis showed that the element would not be able to respond elastically to the impact and that a significant dent and bend would be left in the element. IMPOS assumes that the hit is central to the brace. If the impact is in the middle half of the span, the damage is similar to that for a mid-span collision.

For impact damage closer to one end of the brace, the pattern of damage can be significantly different, with the dent becoming larger whilst the possibility of permanent bending (plastic set) diminishes. For impacts close to the node, the possibility of tearing the steel becomes more likely. Such an event was reported by Williams and Callan [3] who investigated damage on a jacket at a depth of 95m. The dented element had been damaged by a dropped tubular object, and the damage included a crack or tear some 275mm long. A 800 long by 100 mm deep dent had been formed in the 1524 by 29 thick steel member.

3.3 Residual Strength

A post-damage check was conducted on the brace to ensure that the damage would lead to a requirement for repair. The following data was calculated for the 1000 x 28 brace:

Item	Original	Damaged
Axial area	854.7 cm2	693.4 cm2
Section modulus	20211 cm3	11847 cm3
Plastic moment capacity	9650 kNm	6535 kNm
Neutral axis eccentricity	0.0 mm	134.8 mm
Axial load	10400 kN	10400 kN
Axial stress	121 N/mm2	149 N/mm2
P-Delta moment	0	1393 kNm
Additional bending stress	0	117 N/mm2
Combined stress	121 N/mm2	266 N/mm2
Proportion of yield	0.35	0.77

It is concluded that the predicted damage is consistent with the repair scheme chosen as described below. The brace would be so damaged that it would require repairing before it was subjected to an extreme event load condition.

4. REPAIR SCHEME

The above section has put forward a dropped object as being a possible event leading to a requirement for repair work. The resulting damage may consist of denting and bowing.

The desire of the PSC that grout-filling and the placement of an addmember are examined is immediately met as these techniques have direct application to mitigating such damage.

The purpose of the repair is to prevent the overstressing of the damaged element by preventing further buckling damage. The grout filling of the damaged element will prevent the development of local buckling. The additional bracing member, detailed in the vicinity of the site of damage, will reduce the possibility of element buckling failure by reducing the unsupported length of the damaged member in the weakened axis direction.

A potential repair scheme is shown in Figure 2, although equally the new member could run from the horizontal member upwards to the other diagonal member.

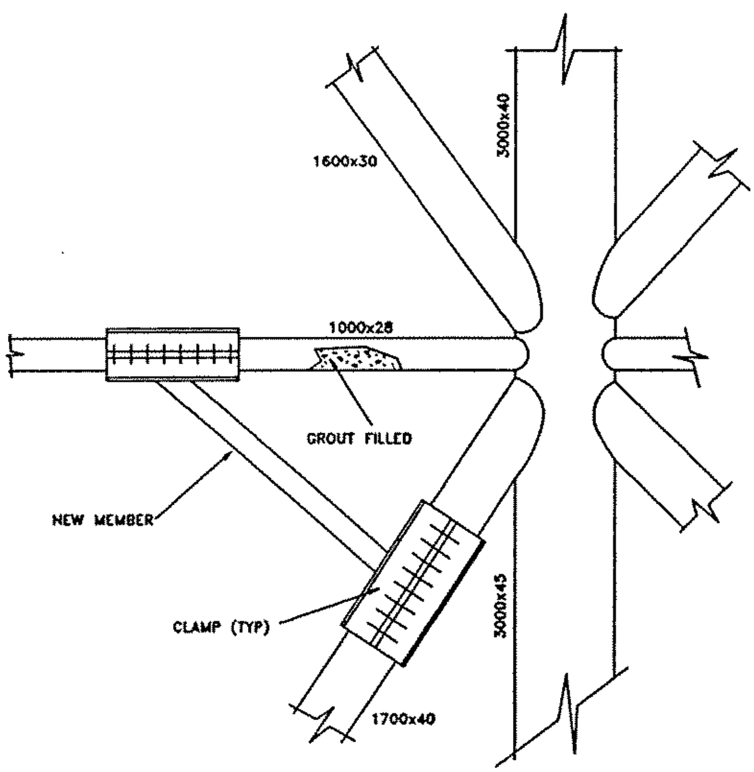

Figure 2 : Outline of Repair

5. CLOSURE

A study of platform configurations and framing has resulted in the selection of a 1000 x 28 brace member size for further study. Although this was based on a 1000 foot depth, this member size is also not untypical of more shallow depths and hence can provide a wide range of depth application.

A detailed study of possible events has identified at least one possible S/R scenario (ie. dropped object) which may be considered not to be rare. However, the selected S/R techniques should not be seen as being limited to apply to this one scenario. Indeed, the S/R techniques were initially selected as giving flexibility to cater for a number of potential scenarios.

The repair will consider the use of "1995" technology - that is - using installation methods and equipment which may either be immediately available, are presently under development, or could be made available following a relatively short development period.

REFERENCES

1. Thuestad TC and Nielsen FG. "Submarine impact with the Oseberg jacket". Ninth conference on Offshore Mechanics and Arctic Engineering, Vol 1, Part B, February 18-23, 1990.

2. IMPOS Rev 1.05 Ship impact analysis program manual. PCL, Ascot, 1992.

3. Williams, DE and Callan, MD. "Repair of a cracked and dented X-node on an offshore platform". 20th annual Offshore Technology Conference, Houston, Texas, 1989.

APPENDIX C

GEOMETRIC CONSIDERATIONS FOR ADDMEMBER SCENARIOS

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MAIN STRUCTURAL COMPONENTS

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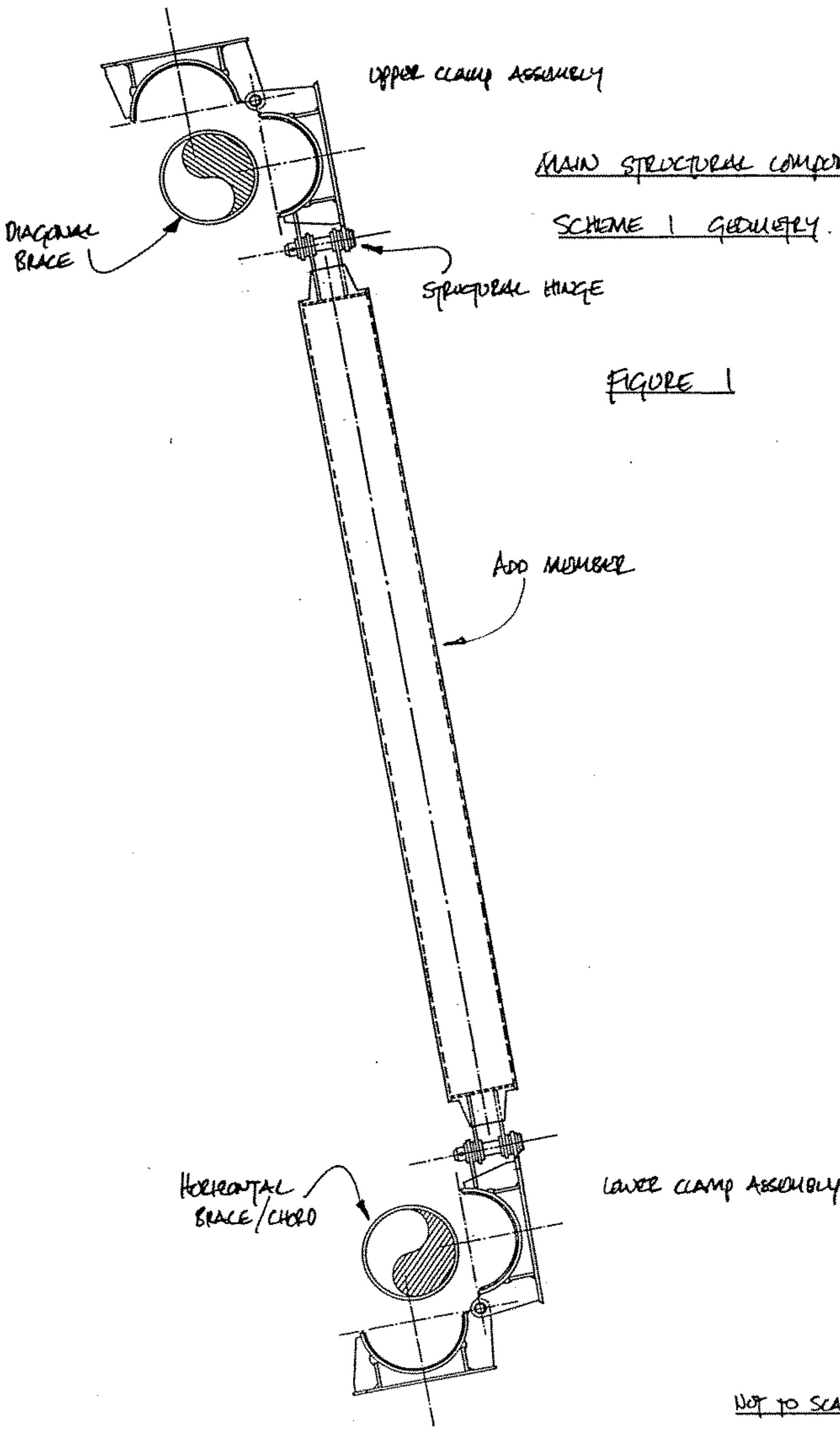

FIGURE 1

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MAIN STRUCTURAL COMPONENTS

SCHEME 2 GEOMETRY.

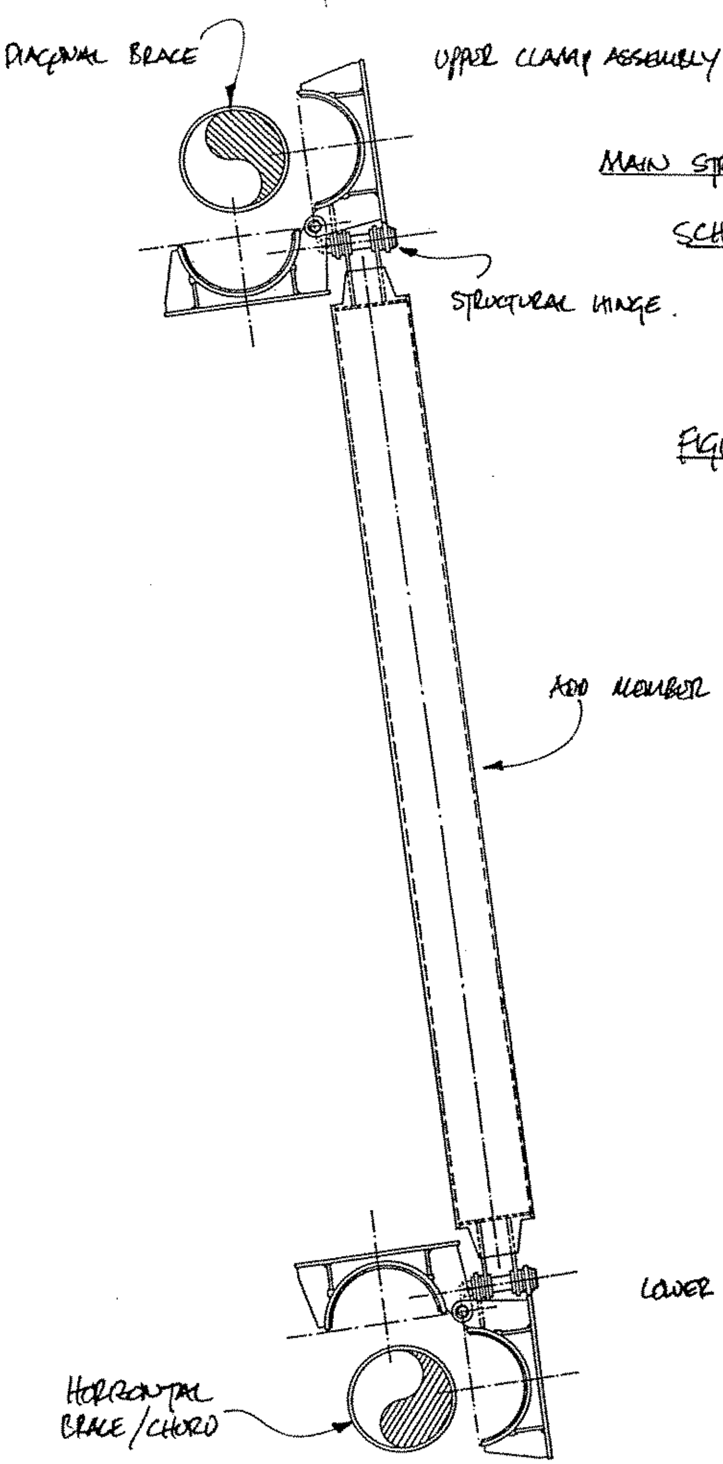

FIGURE 2.

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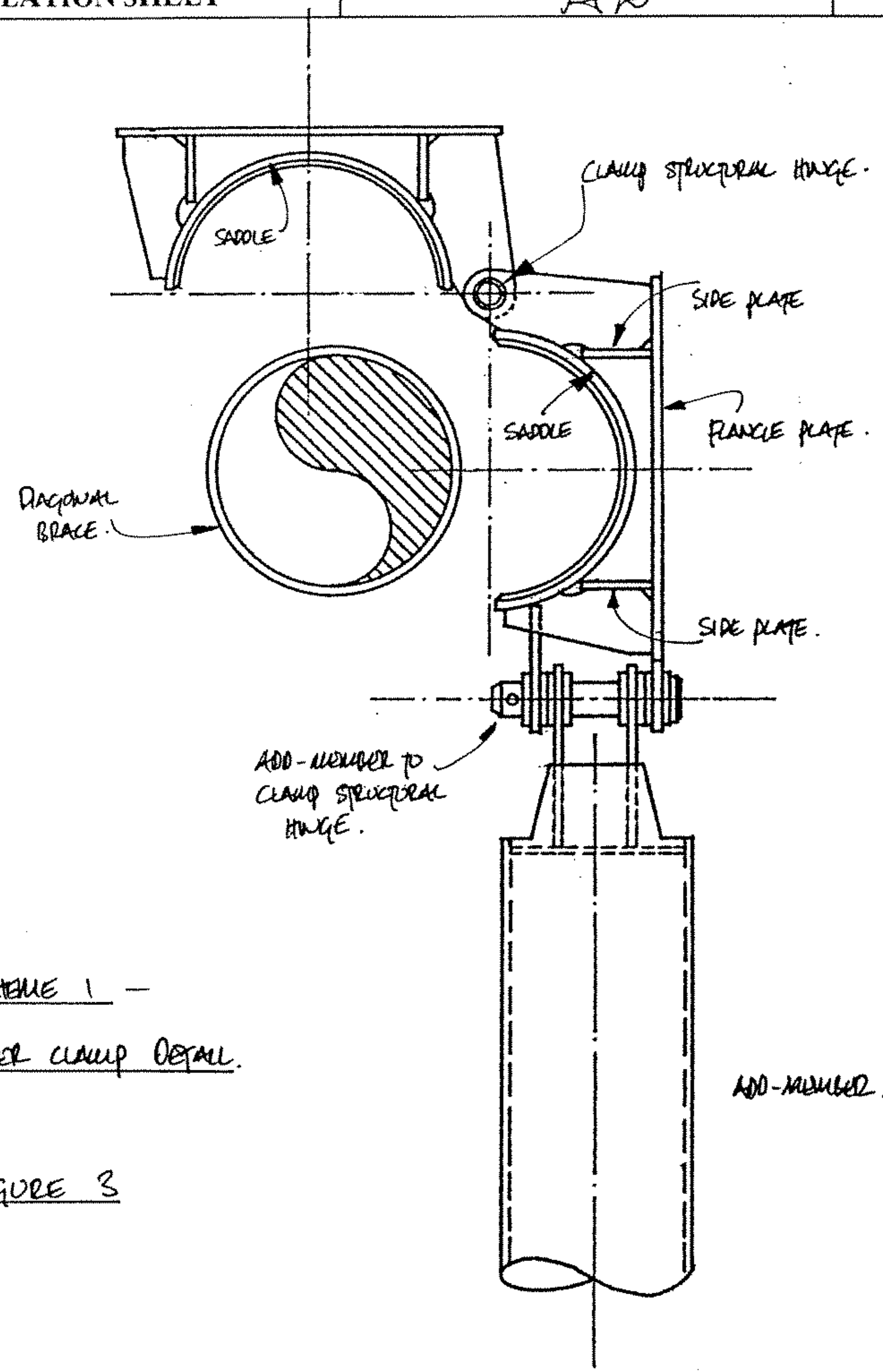

SCHEME 1 –

UPPER CLAMP DETAIL.

FIGURE 3

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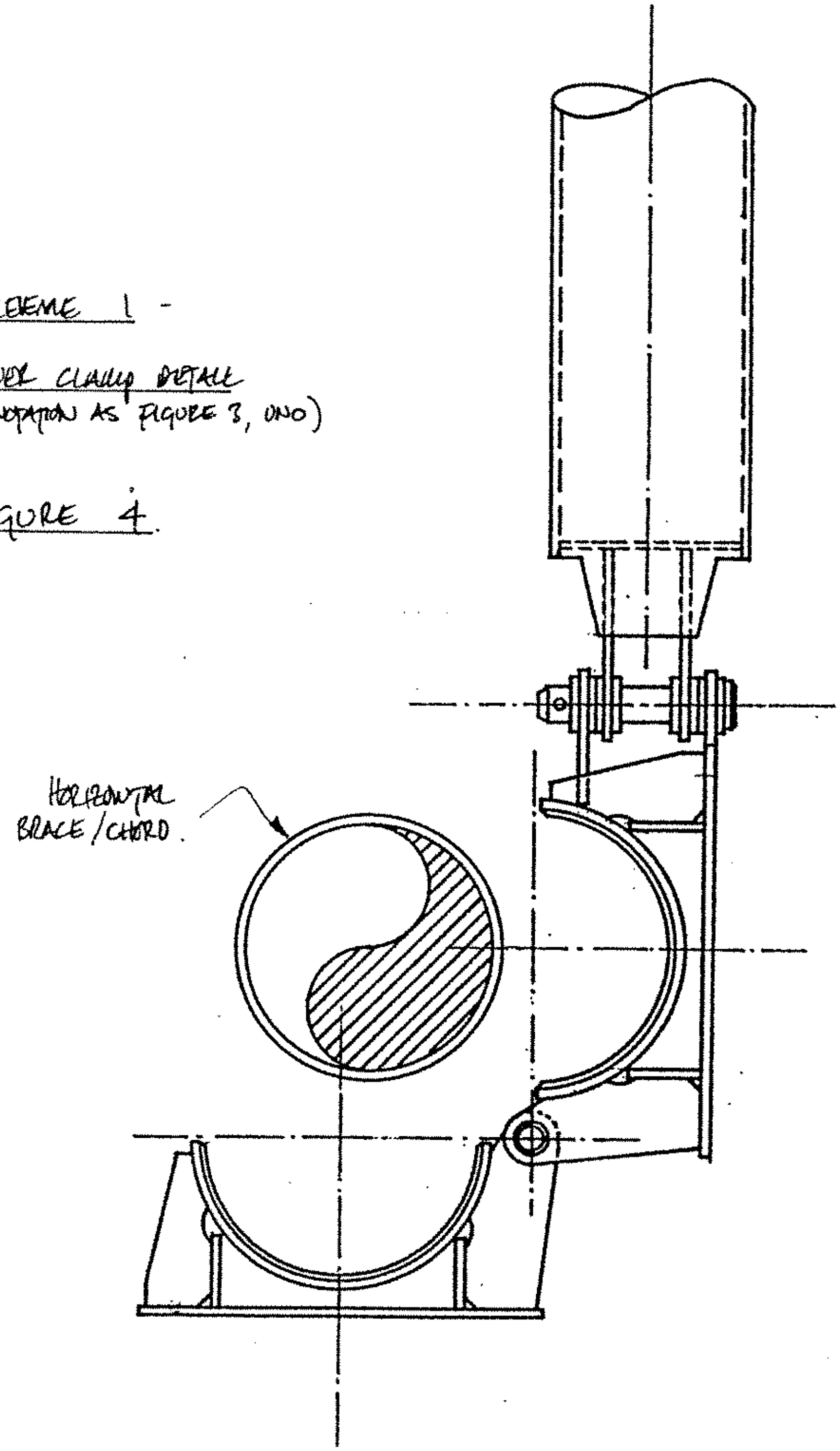
SCHEME 1 -

LOWER CLAMP DETAIL
(ANNOTATION AS FIGURE 3, UNO)

FIGURE 4.



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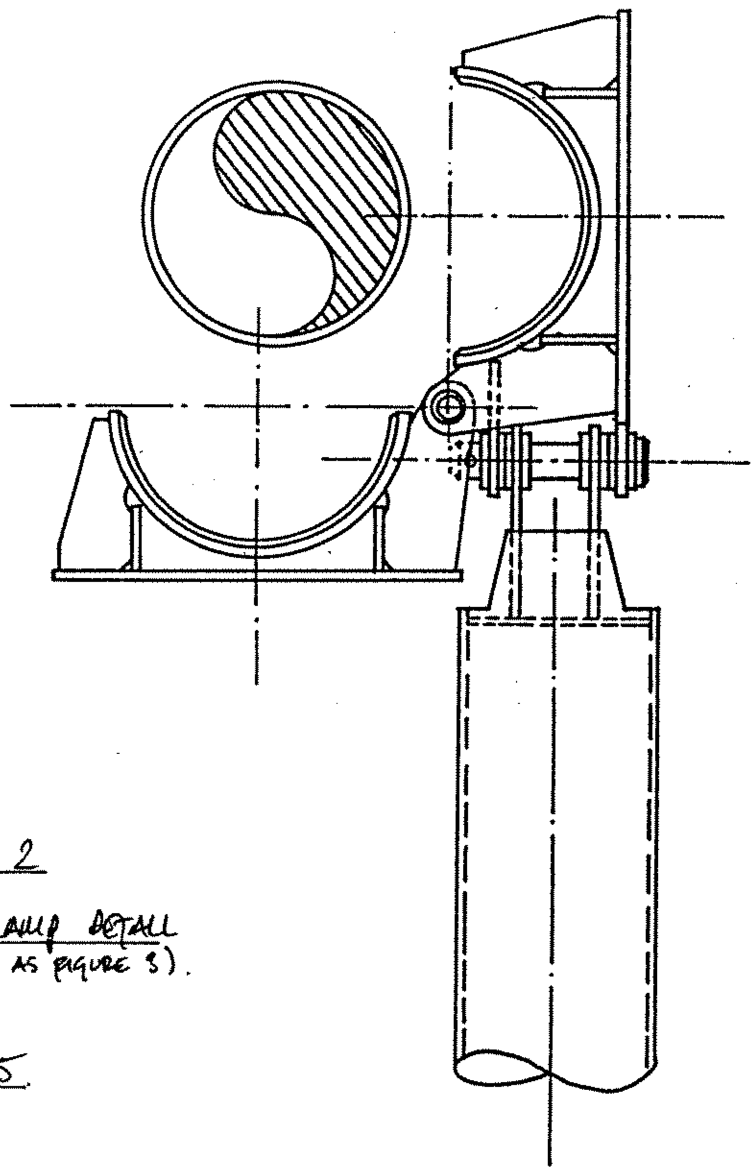

SCHEME 2

UPPER CLAMP DETAIL
(ANNOTATION AS FIGURE 3).

FIGURE 5.

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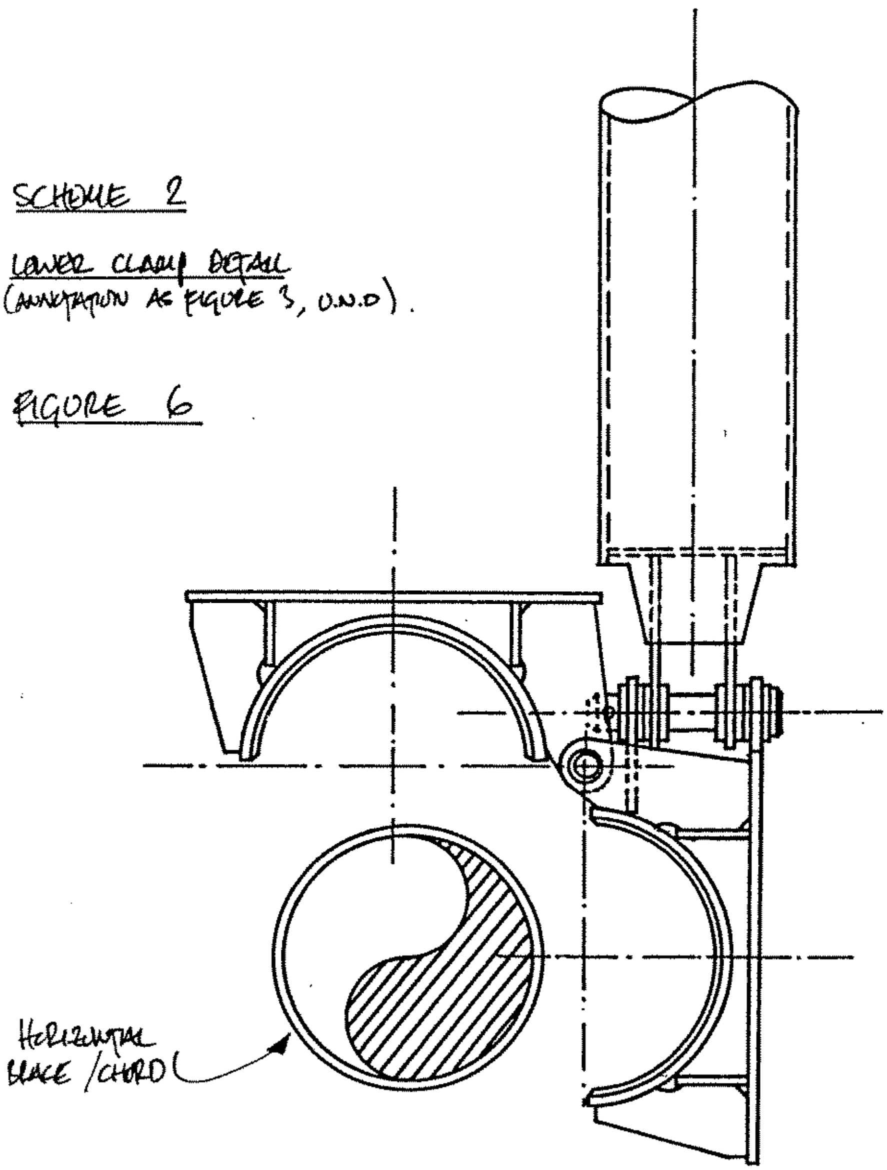
SCHEME 2

LOWER CLAMP DETAIL
(ANNOTATION AS FIGURE 3, U.N.O).

FIGURE 6



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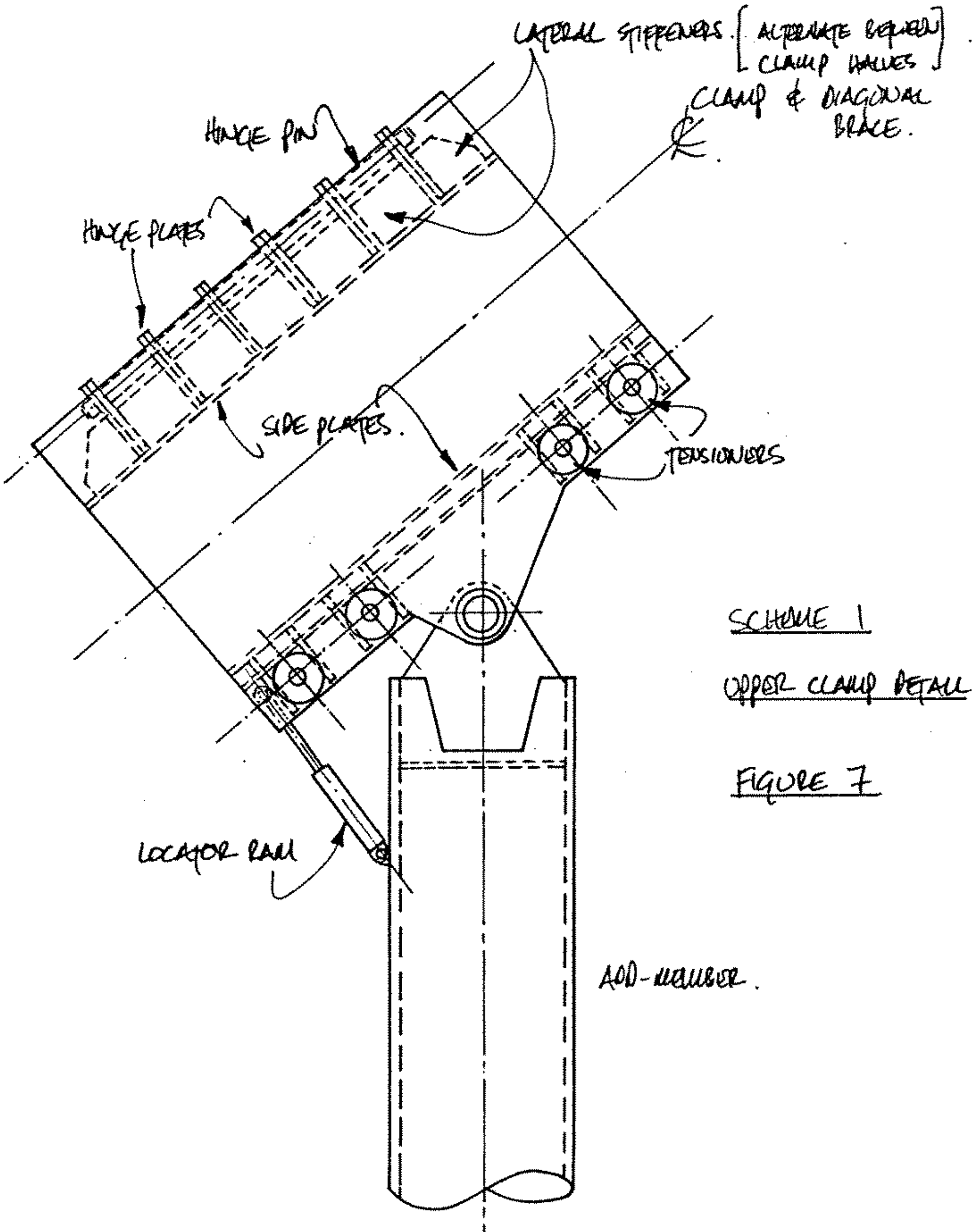

SCHEME 1

UPPER CLAMP DETAIL

FIGURE 7

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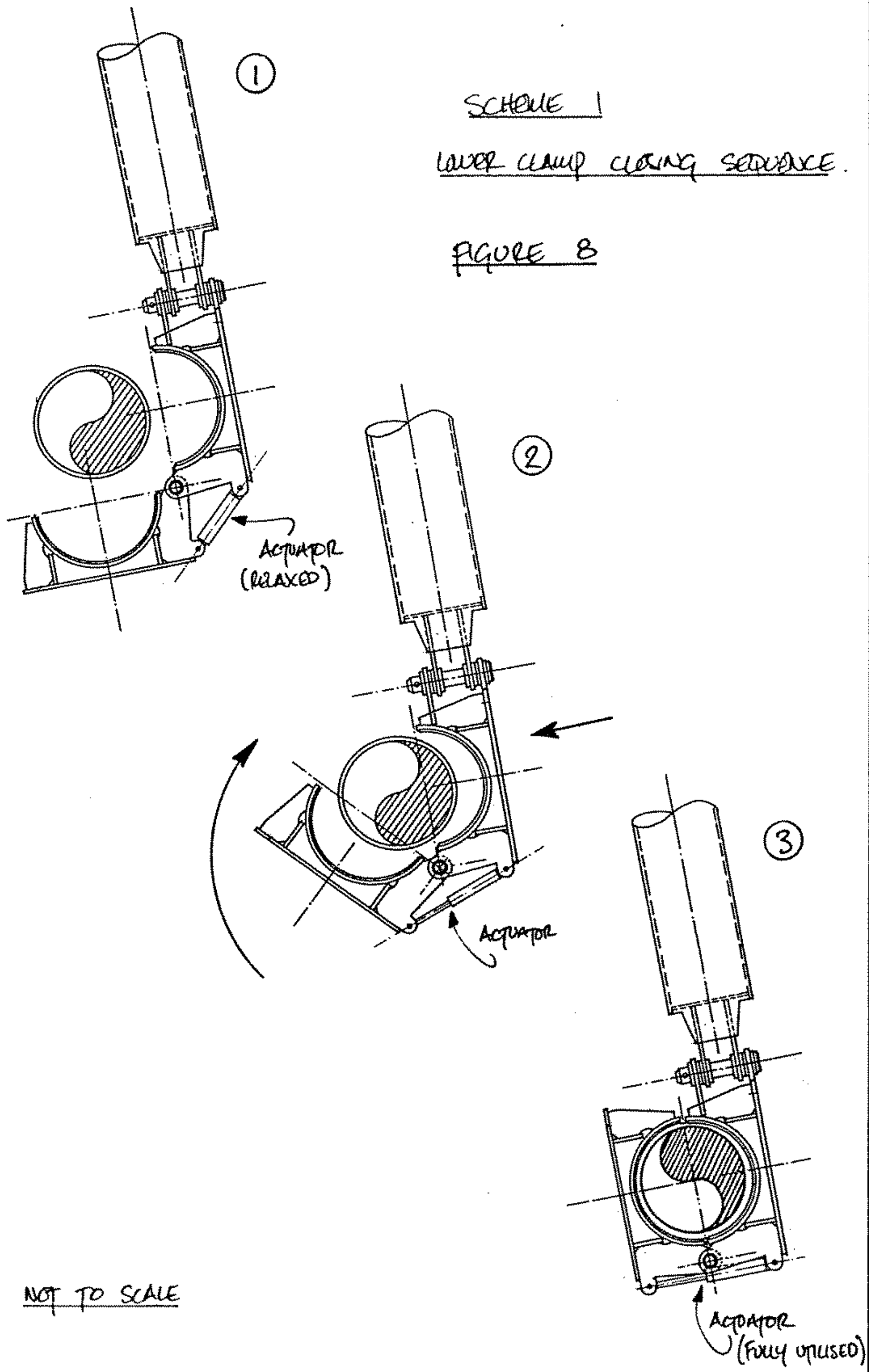
REF

SCHEME 1

LOWER CLAMP CLOSING SEQUENCE.

FIGURE 8



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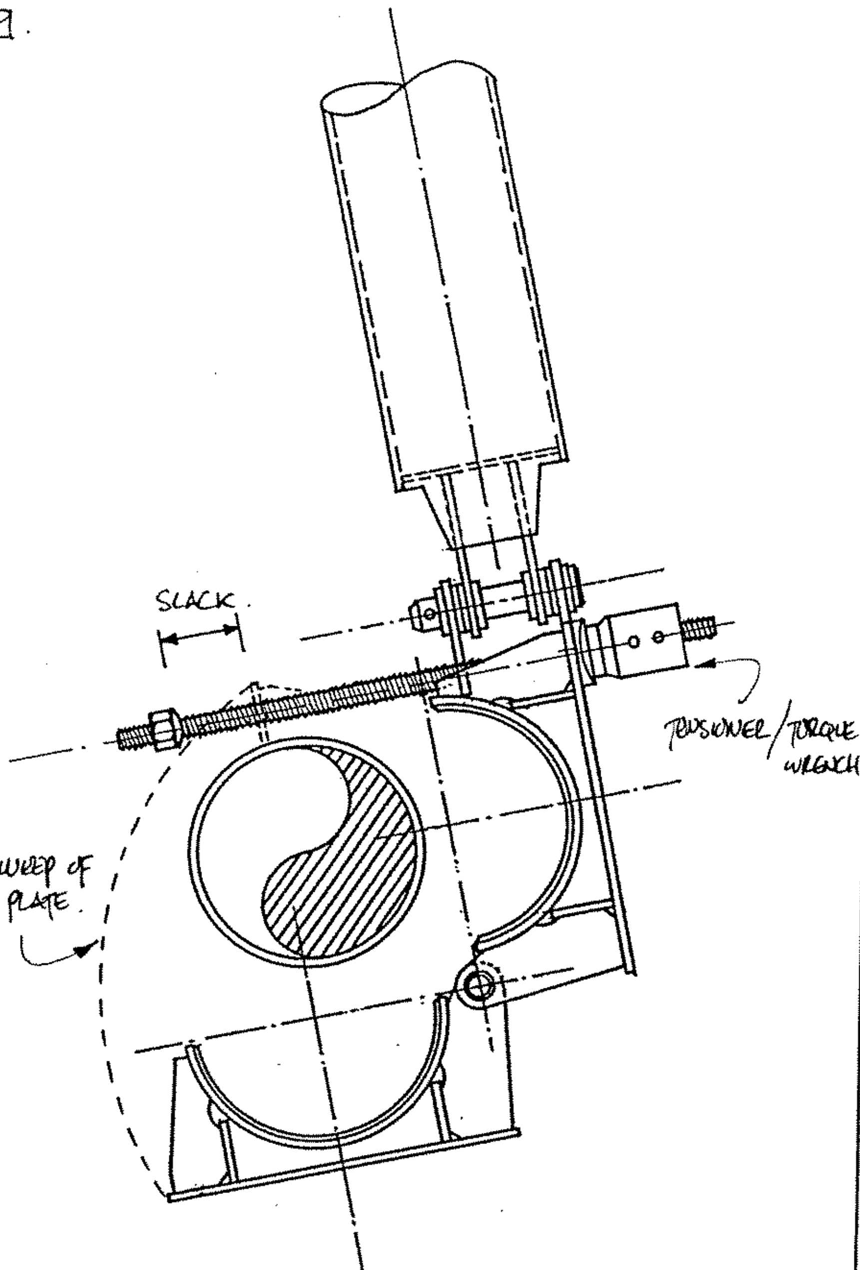
STUDBOLT ENGAGEMENT - OPTION 1.

FIGURE 9.



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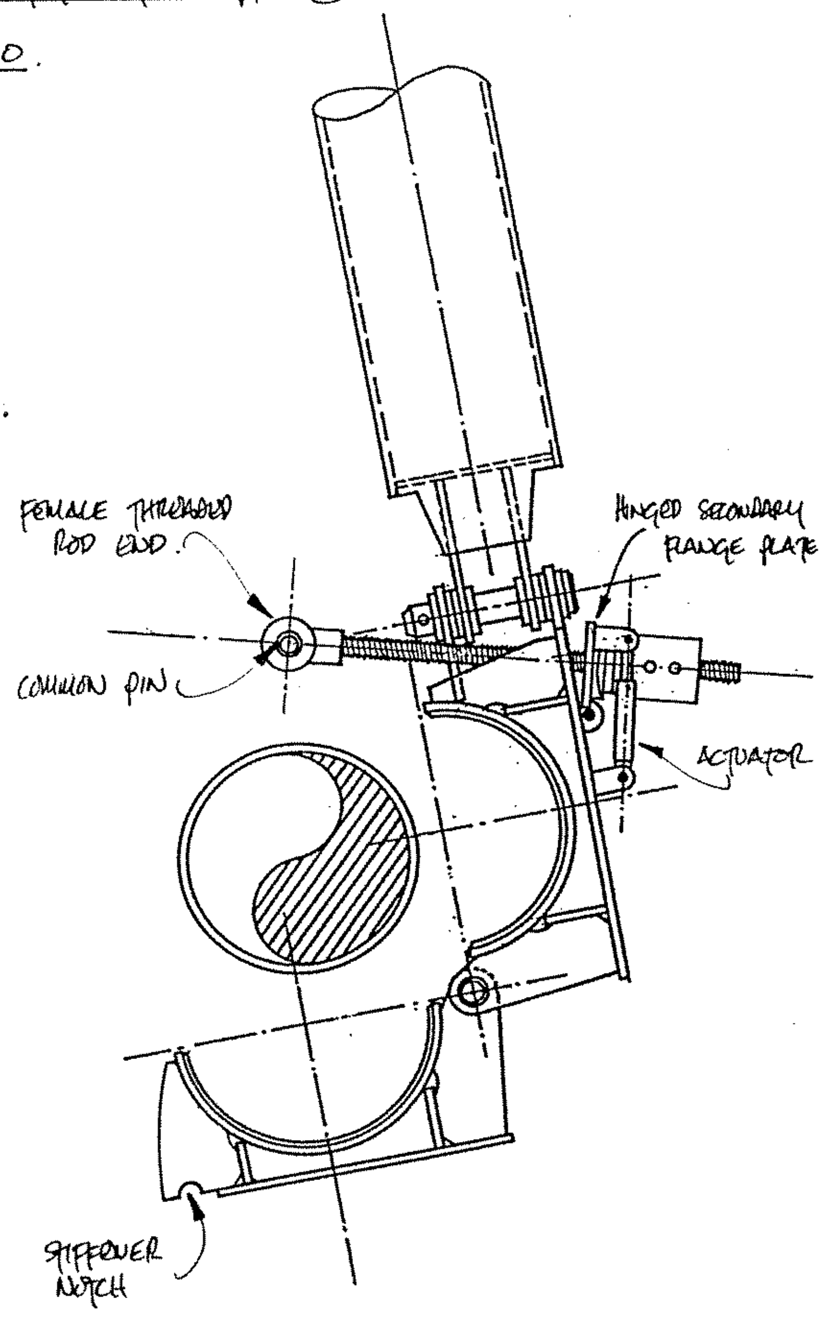
STUDBOLT ENGAGEMENT - OPTION (2)

FIGURE 10.



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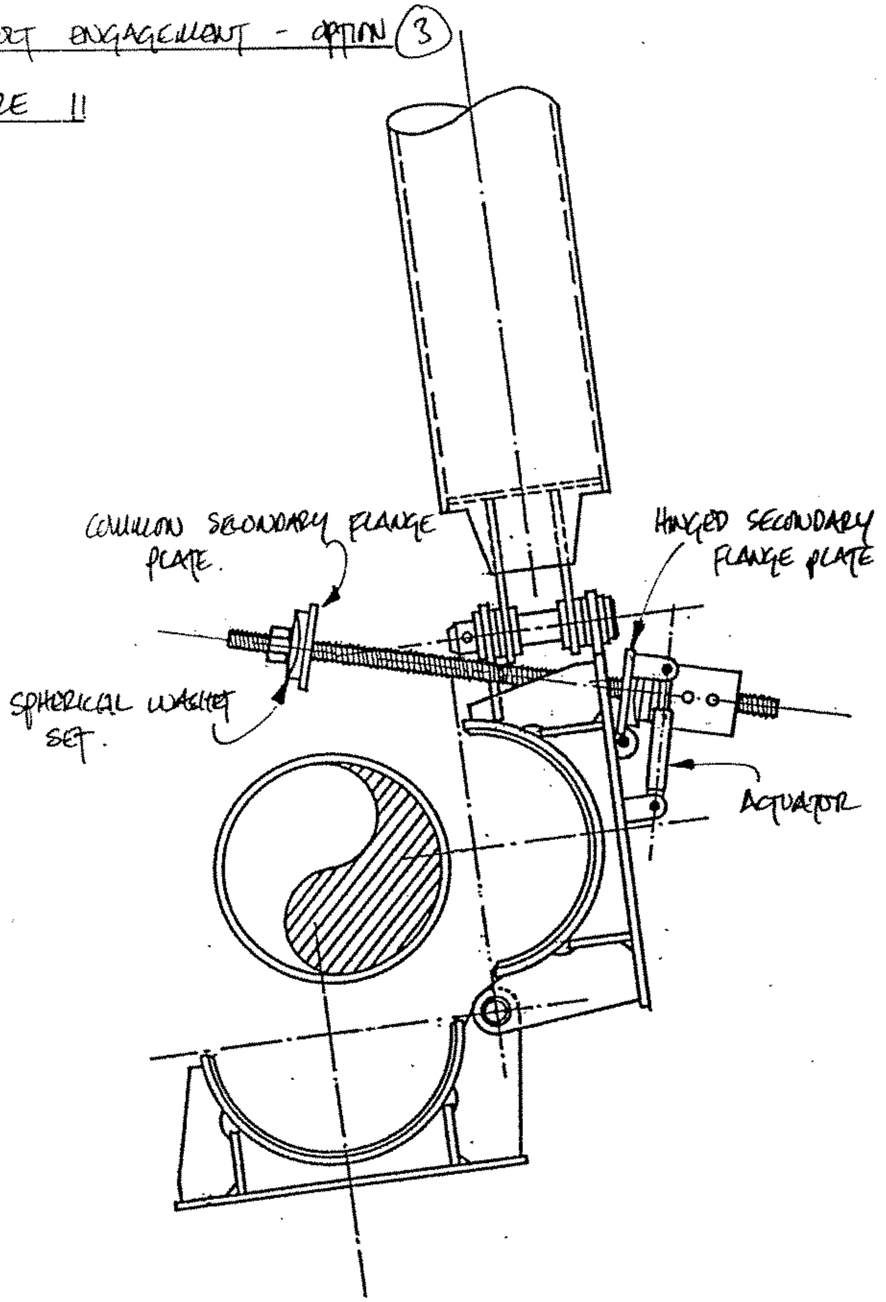
STUDBOLT ENGAGEMENT - OPTION 3

FIGURE 11



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APPENDIX D

INSTALLATION CONSIDERATIONS FOR ADDMEMBER SCENARIOS

Power Supply

There are a number of hydraulically operated functions on the addmember assembly to enable successful installation. Some of the options available are equally applicable to deepwater strengthening/repairs and are as follows:

1. The use of a conventional hydraulic power pack on topsides which supplies hydraulic power via an umbilical to a manifolding system on the addmember assembly. The umbilical will contain one feed and one return hydraulic line. The manifolding can be switched between functions either by electric switch valves or by operation by the ROV.

2. A submersible hydraulic power pack is attached to the addmember assembly which is controlled from the surface via an umbilical which solely contains electrical cabling. The power pack will be recoverable by the action of a release mechanism operated by the ROV or from the surface.

3. Hot stab the ROV's hydraulic supply to power each individual hydraulic function from a common manifold.

For deepwater strengthening/repairs, options 2 and 3 are the options available. However, due to the high volume demanding hydraulic pretensions, such as the bolt tensioning gear, the ROV hydraulic supply is not likely to be adequate unless modified.

ROV Deployment

A work class ROV can be used to carry out necessary functions. One of the functions of the ROV is to act as an underwater camera to enable clash avoidance and aid in positioning of the assembly. The ROV can be deployed from the jacket for shallow water operation. However, for deepwater operation, the ROV may have to be deployed from a boat, or supply vessel.

Pre-Installation Site Preparation

All marine growth and loose corrosion products must be cleaned from each of the clamp installation sites. This task can be carried out by the ROV, using either a high pressure water jet or rotating brushes. Once this activity is complete it may be necessary for the ROV to carry out a survey to establish member ovality and straightness. This could be carried out using a photogrammetric survey in conjunction with scale yard sticks and callipers. If such a survey is necessary, then these activities will need to be conducted prior to addmember fabrication. Immediately before installation, the ROV may need to remove any marine growth which has accumulated since the previous cleaning.

The strengthening/repair location will have an influence on the location of a lay-down area and/or launch location for the addmember assembly. The possible scenarios are as follows:

- If a deck crane is to be used to lower the addmember assembly to the repair/strengthening site, then the following points need to be considered:

    1. Operational requirements as well as appropriate certification will determine the feasibility of using a deck crane.

    2. Use of a deck crane is only applicable if direct access to the repair/strengthening site is possible.

    3. The permanent crane rigging should preferably not enter the water. Pennant wires may be used to attach the crane rigging to the addmember assembly.

    4. The above points will mean that this option will only be applicable to shallow water repair/strengthening schemes.

    5. The main advantage in using a deck crane is that it has the ability to move the addmember assembly in all three directions.

- If the overhang of the topsides is such that the jacket crane does not have direct overhead access to the strengthening/repair location, then either a laydown platform will need to be constructed below the Module Support Frame (MSF) to support a trolley arrangement or a transfer frame will need to be constructed to enable transfer of the addmember assembly from the deck crane to the launch location. The assembly can then be lowered using a winch which can be attached to a secure structural point on the MSF directly over the strengthening/repair location. The degree of overhang and the length of time which the assembly is likely to remain on the laydown area will influence the above options. This option is also applicable to strengthening/repairs on an internal location to the jacket.

- For a deepwater repair, the selection of one of the possible solutions detailed below will be influenced by installation timescales and/or platform deck layout:-

    1. Delivery of the assembly can be by either a supply vessel or barge. Both could be equipped with a crane. This could be utilised to perform a direct installation from the vessel. In this manner, the vessel may perform two functions ie. ROV deployment and assembly installation. However, vessel movements, particularly heave, may mean this option is not appropriate unless calm conditions prevail.

    2. An 'A' frame could be constructed from a suitable location such that a winch can be directly overhead of the strengthening/repair location.

This frame may be mounted from the MSF or the upper jacket structure depending on geometry. The frame could be equipped with a beam to allow lateral movement of the addmember assembly should installation require it.

An important pre-installation operation for a deepwater strengthening/repair could be the installation of guide lines. These may be necessary to prevent the addmember assembly from drifting in the current. The ROV would attach these guide lines to suitable locations to ensure that the addmember is guided into its correct position. (This is a routine operation for subsea wells.)

Installation

This Appendix contains an idealised installation which is applicable to Schemes 1 and 2 arrangements as presented in Appendix C (Figures 1 and 2 of Appendix C refers). For the purpose of this installation discussion, Scheme 1 is selected. However, Scheme 2, will be similar in application.

The studbolts and tensioners will descend with the addmember assembly as detailed in Appendix C (Figures 9, 10 and 11 of Appendix C refers). There are a number of options available for latching the studbolts into the hinged clamp half, and are discussed below. Caution during decent is necessary to prevent obstruction collision with the protruding studbolt assembly.

Descent

The descent of the addmember assembly (see installation sequences at the end of this Appendix) can be monitored by flying the ROV from the outside of the platform. As the assembly nears its location, the ROV will play a large role in positioning the assembly horizontally. For Scheme 1, the uppermost clamp can be positioned first. Final adjustments up and down the diagonal member can be achieved by using the crane in conjunction with the ROV using its thrusters. The assembly is then brought close to the damaged member and positioned using the ROV onboard cameras.

Once the uppermost clamp is in position, the hydraulic rams can be activated to partially close the hinged half of the clamp. The clamp is only partially closed, thus allowing the addmember assembly to rotate and the lower clamp to swing into place. Note that even with partial closure, friction effects may need to be overcome to allow adjustment. At this point the ROV can be used to 'eye-ball' the lower clamp into position.

The locator ram (detailed in Appendix C, Figure 7 refers) can now be activated to swing the addmember and lower clamp about the upper pivot. This will enable accurate positioning of the lower clamp as the assembly is lowered further. The pivots at the upper and lower clamp connections will compensate for variation in the linear distance between the nominal attachment points.

Once the lower assembly is in position, both the upper and lower hydraulic rams can be fully activated to close the hinged half of each clamp. The sling retains the assembly weight, and tensioning can commence.

Tensioning

There are three main issues which will influence the type of tensioner to be used. These are as follows:

1. Various methods by which the studbolts engage in the hinged half of the clamp will determine the amount of travel required to take up the slack of the excess studbolt length.

2. If tool recovery is deemed necessary then the type of tensioning tool will influence the ease with which this may be achieved. Conventional hydraulic tensioners may prove difficult to recover. However, a purpose-built tensioner or torque wrench could ease this difficulty. To aid recovery further, all tensioning equipment could be mounted on a common, detachable, plate.

3. The hydraulic supply will have limits on pressure and flow rate depending on supply method and depth at which they are to be used. This will have a direct effect on which type of tensioning tool will function adequately.

A number of options for tensioning are available. Three such options are illustrated in Appendix C (Figures 9, 10 and 11 refers) and are discussed below:-

- The first method is to have a nut with a spherical base on the end of each of the studbolts. The spherical base would engage with a slot in the main plate of the lower clamp half as the clamp is closed. This slot is profiled to provide the other part of a spherical washer set with the shaped nut. The studbolts are loosely guided by tubes welded to the underside of the main plate of the top half of the clamp. The studbolts are prevented from rotating whilst the nuts are tightened by a light steel rod passing through clearance holes in the lower ends of each of the studbolts.

- The second method is to have female threaded rod ends on the ends of each of the studbolts, connected by a common pin. The spherical bearing in the rod end provides the equivalent function to the spherical washer set and the studbolts latch to the lower clamp half by engaging the common pin in notches in the stiffeners of the lower clamp half. The common pin restrains the studbolts from spinning when the nuts are tightened. This method requires additional hydraulics on the clamp, if recovery is necessary and the ROV is unable to detach the studbolt assembly.

- The third method is to have a common plate for each studbolt end to run through. This plate acts as a secondary flange plate to transfer load through to the stiffeners. This method also requires additional hydraulics on the clamp.

Of the three options, the first requires the greatest amount of slack to be taken up by the tensioning tool. The most appropriate method can be chosen during detailed design.

The studbolts can be tensioned once in position. This can be done either by hydraulic tensioners (jacks) or by torque wrenches. Both of these types of tensioning gear typically require a hydraulic supply with a pressure in the range 5,000 to 10,000 psi. However, neither type of tensioning gear requires a high flow rate. Consequently, the hydraulic pressure may readily be supplied by lines from the surface, even for deepwater sites. Alternatively, custom-made designs of either type of tensioning gear can be used, with larger pistons which are able to function with the lower hydraulic pressure available from the ROV hydraulic system. Systems of this type can be costly.

Use of conventional hydraulic tensioners has the advantage of direct calibration between supplied hydraulic pressure and resulting studbolt tension. Unfortunately, they only provide limited stroke (about 25mm). Hence, their operation requires that the two nuts (which capture the tensioner) must be rotated in sequence during each operating stroke of the tensioned piston. Access to the lower of these two nuts is limited, and prevents direct operation by ROV. This may be overcome by one of two means. Firstly, two small hydraulic motors can be used to drive the nuts. These two motors may be supplied with hydraulic pressure for the entire operating stroke of the tensioner, thus avoiding the need for complex valving. Alternatively, split threaded collets may be used in place of the nuts. These will click over the studbolt as it is pulled through, and lock onto the studbolt as it tries to fall back when the tensioner is run off the stud by reversing the appropriate hydraulic motor. The latter method precludes recovery of the tensioners, but should be considered in detailed design because of the relatively low unit cost of the tensioners and the simplicity of the method.

Torque wrenches are more simple to operate, as tensioning is achieved by simply cycling the hydraulic supply to a single hydraulic cylinder. Torque wrenches must be calibrated for the particular application once the clamp has been fabricated. The higher price of torque wrenches relative to hydraulic tensioners means that it may be economic to recover them after installing strengthening/repair schemes. Recovery is not a problem because the torque wrench is a simple drop fit over the nut to be tightened. The use of torque wrenches is simpler and probably more cost-effective than the hydraulic jacks, although it may take longer due to the larger volume of hydraulic fluids to be moved during tightening. The choice between torque wrenches and tensioners can be made during detailed design, on the basis of simplicity of operation, timescale for operation and cost (which in turn is dependent on the number of bolts used).

An option used for tensioning in the diverless repair of six water lift caissons on Mobil's Beryl Bravo Platform were specially-built, long travel, sacrificial jacks. Once the studbolts had been tensioned, the tension was locked in the studbolts by introducing epoxy into the jack chamber separated from the oil by a shuttle. The epoxy displaced the oil and was held under pressure until cured. The shuttle was used so that epoxy need only be introduced during the last stages of tensioning. This meant that recovery, if necessary, would be easier as only hydraulic fluid would be present.

A separate torque wrench or hydraulic tensioner is required for each studbolt. All the torque wrenches or tensioners on a clamp can be manifolded together, to ensure that all of the studbolts on the clamp are tensioned simultaneously, to the same value.

Tools and Equipment Recovery

After tensioning, the ROV can be used to recover the tensioning tool from the clamp. This can be achieved by having the tensioning tool mounted on a common plate, and using the ROV to attach a recovery line to this plate, if not pre-installed.

The hydraulic lines can be detached from the clamp by means of quick detach connectors in the hydraulic lines (these items are off-the-shelf components).

Closure

The methods discussed above in respect of installation represents the collective findings from the detailed appraisals conducted in this JIP. The findings reported, coupled with the methodologies deployed in the successful diverless installation of clamps on Mobil's Beryl Bravo platform (Appendix E refers), can collectively be considered during the detailed design phase of any diverless implementation project to develop the most appropriate, reliable and safe installation procedure. The installation procedure will vary for each strengthening/repair conducted, for obvious reasons.

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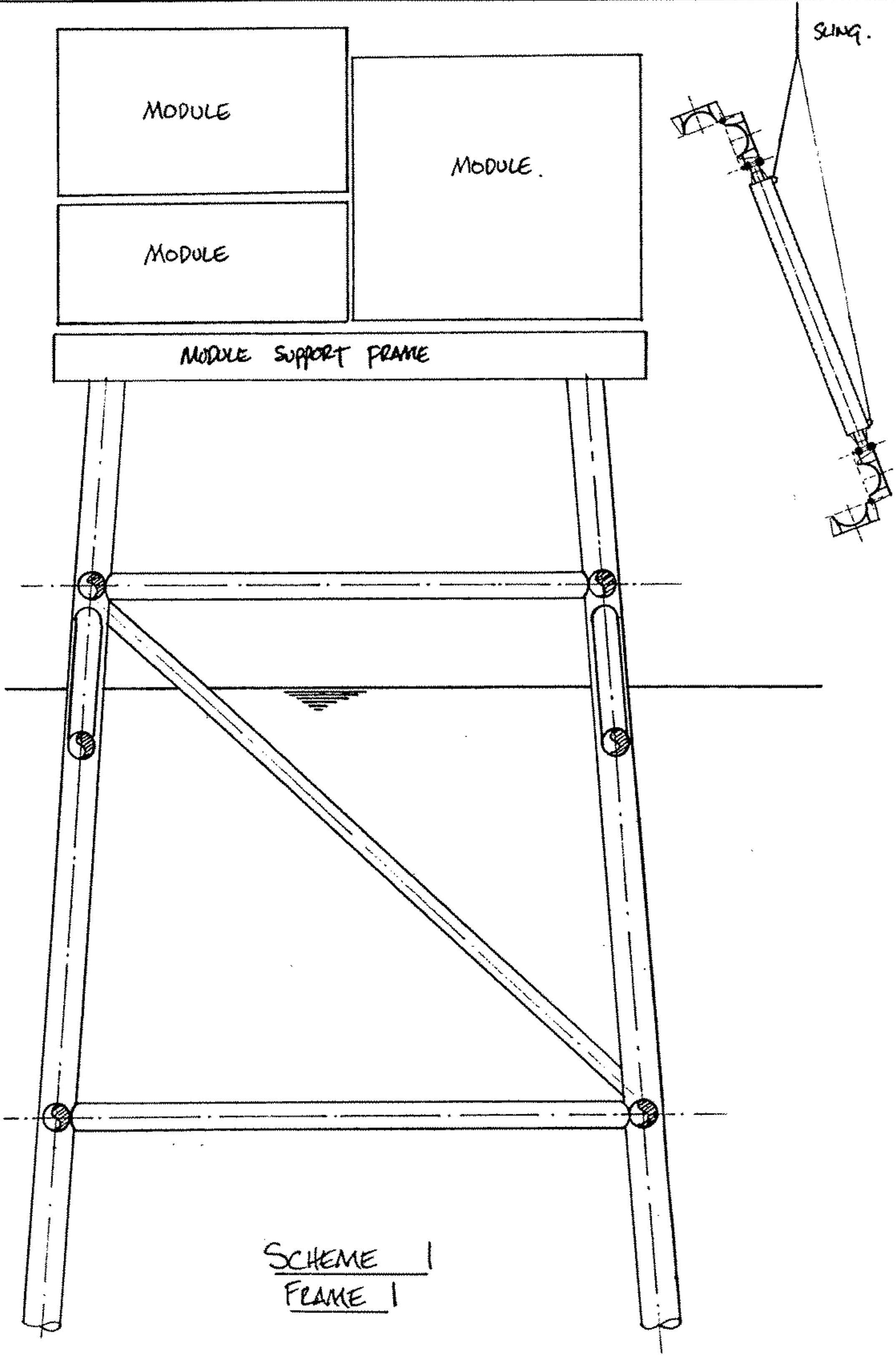

SCHEME 1
FRAME 1

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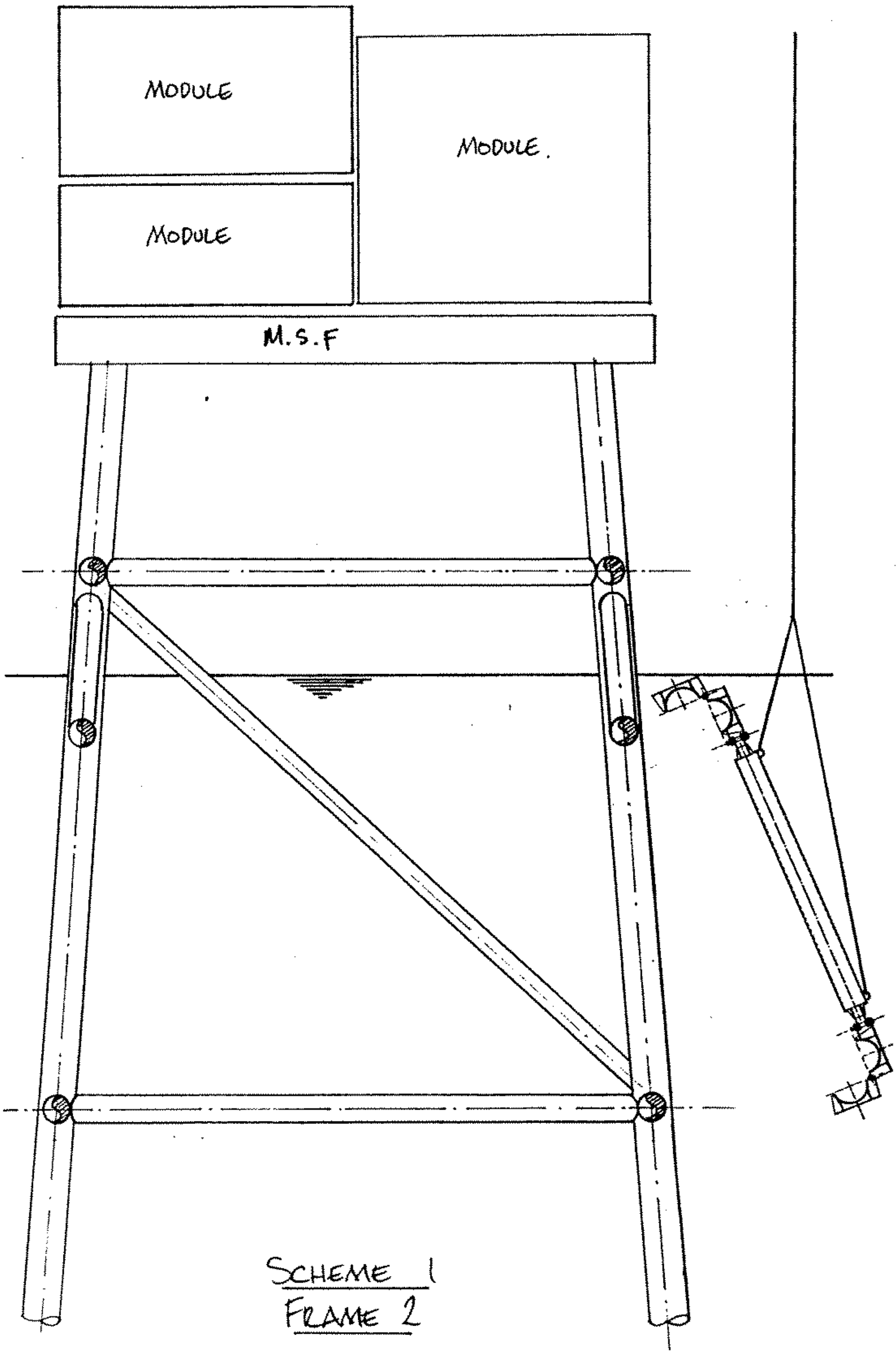

SCHEME 1
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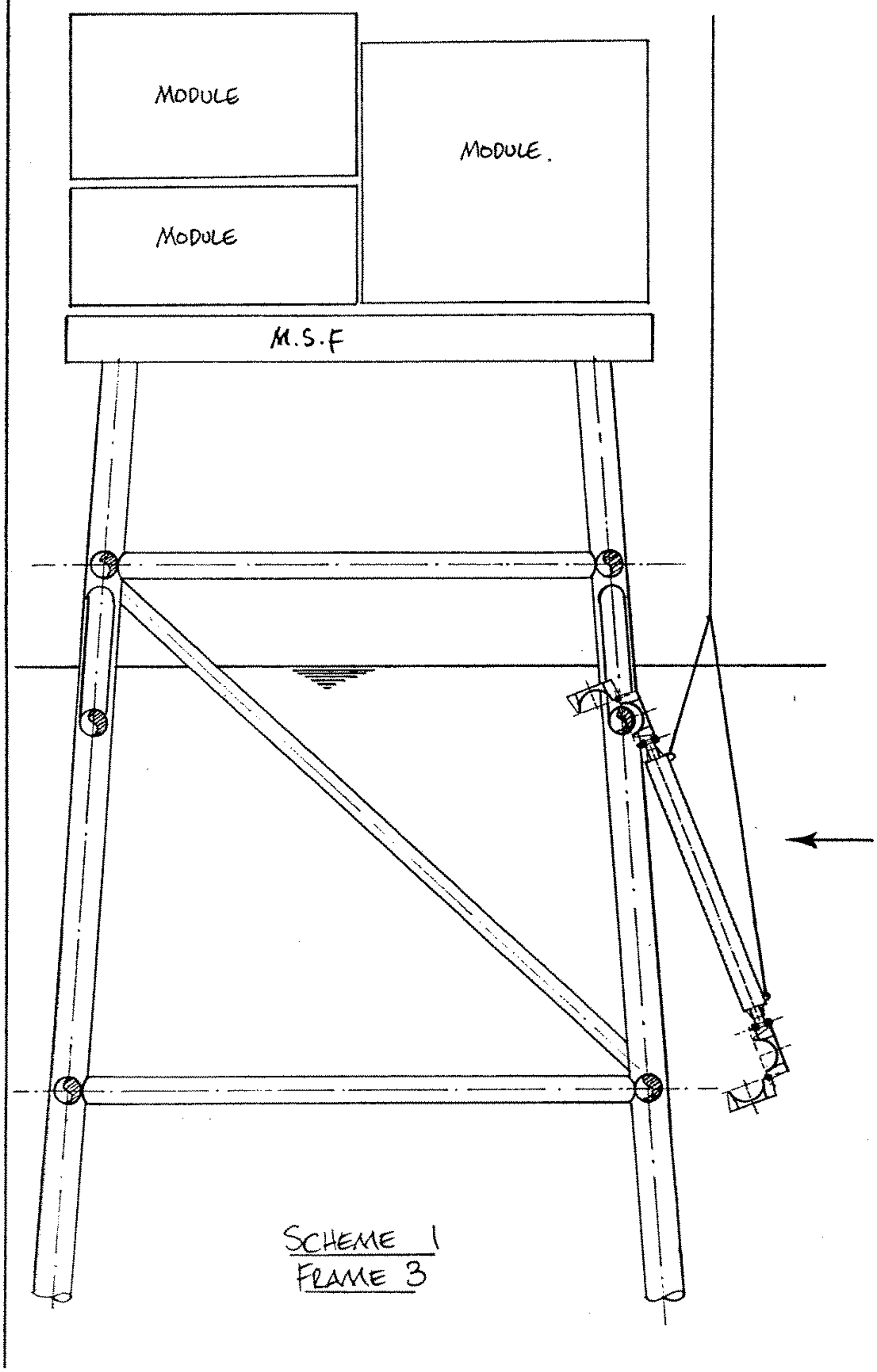

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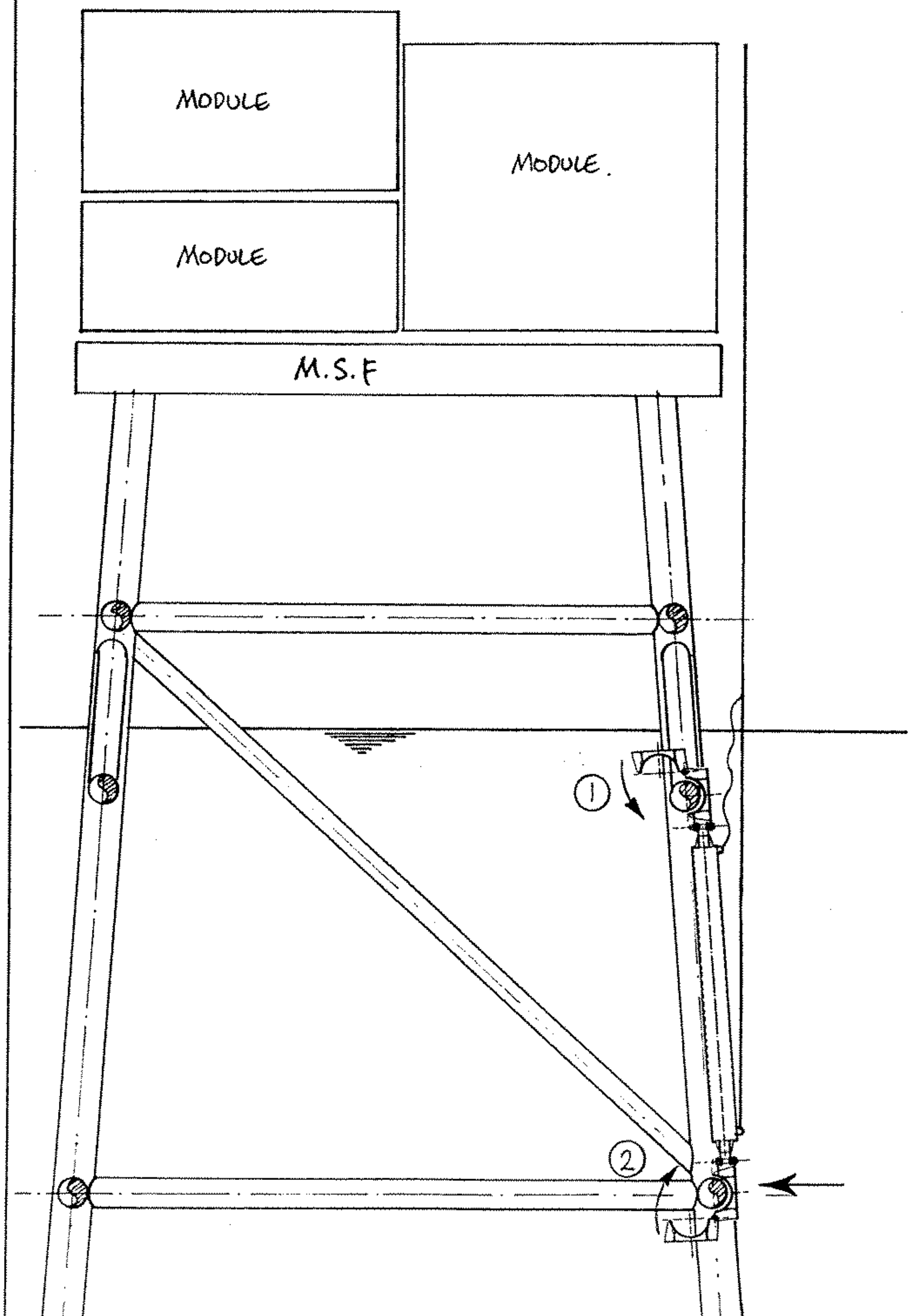

SCHEME 1
FRAME 4

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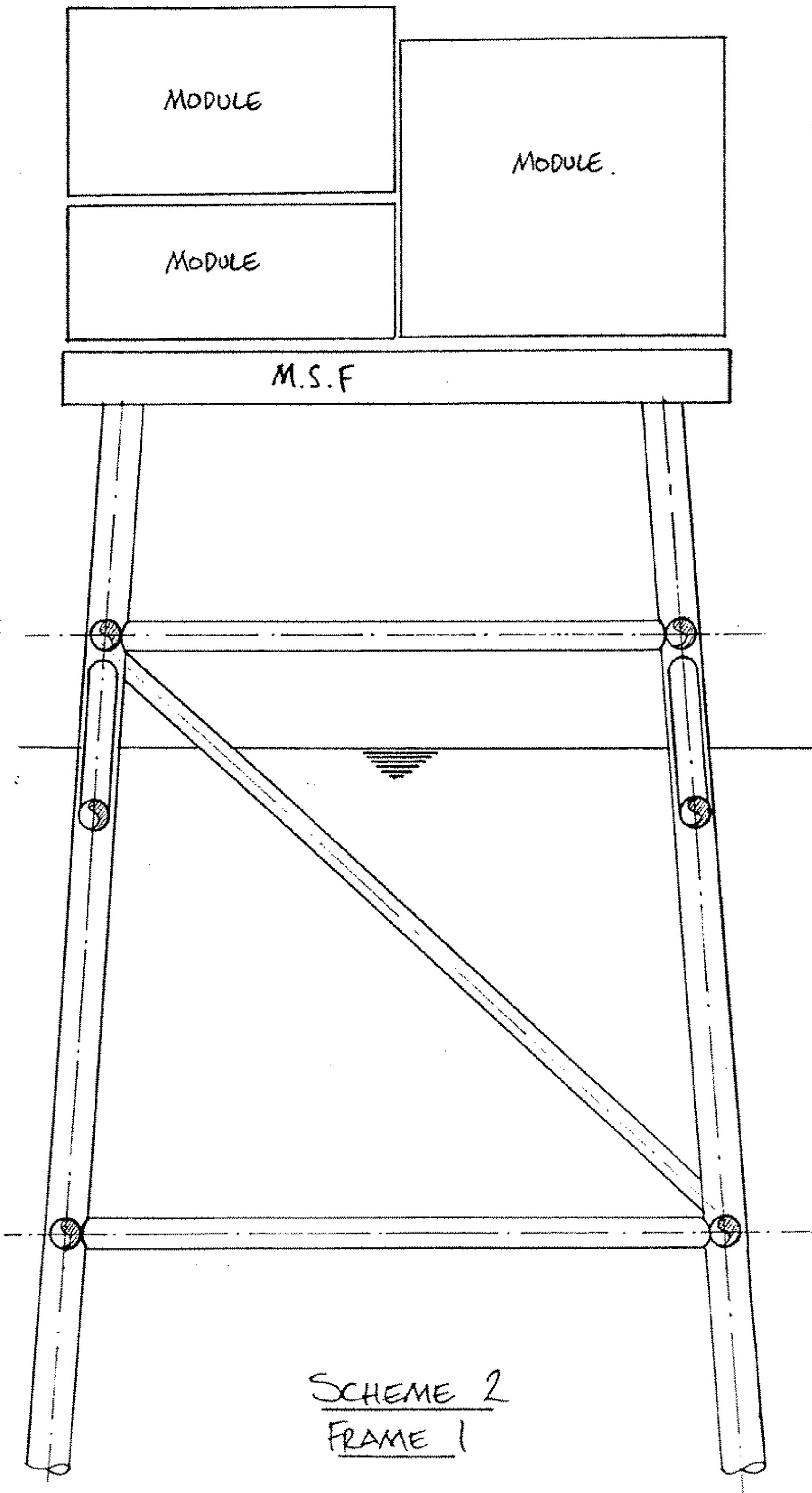

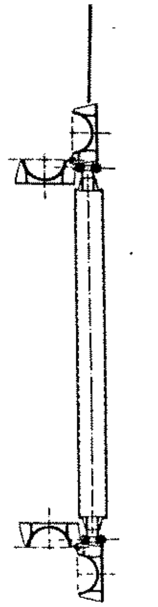

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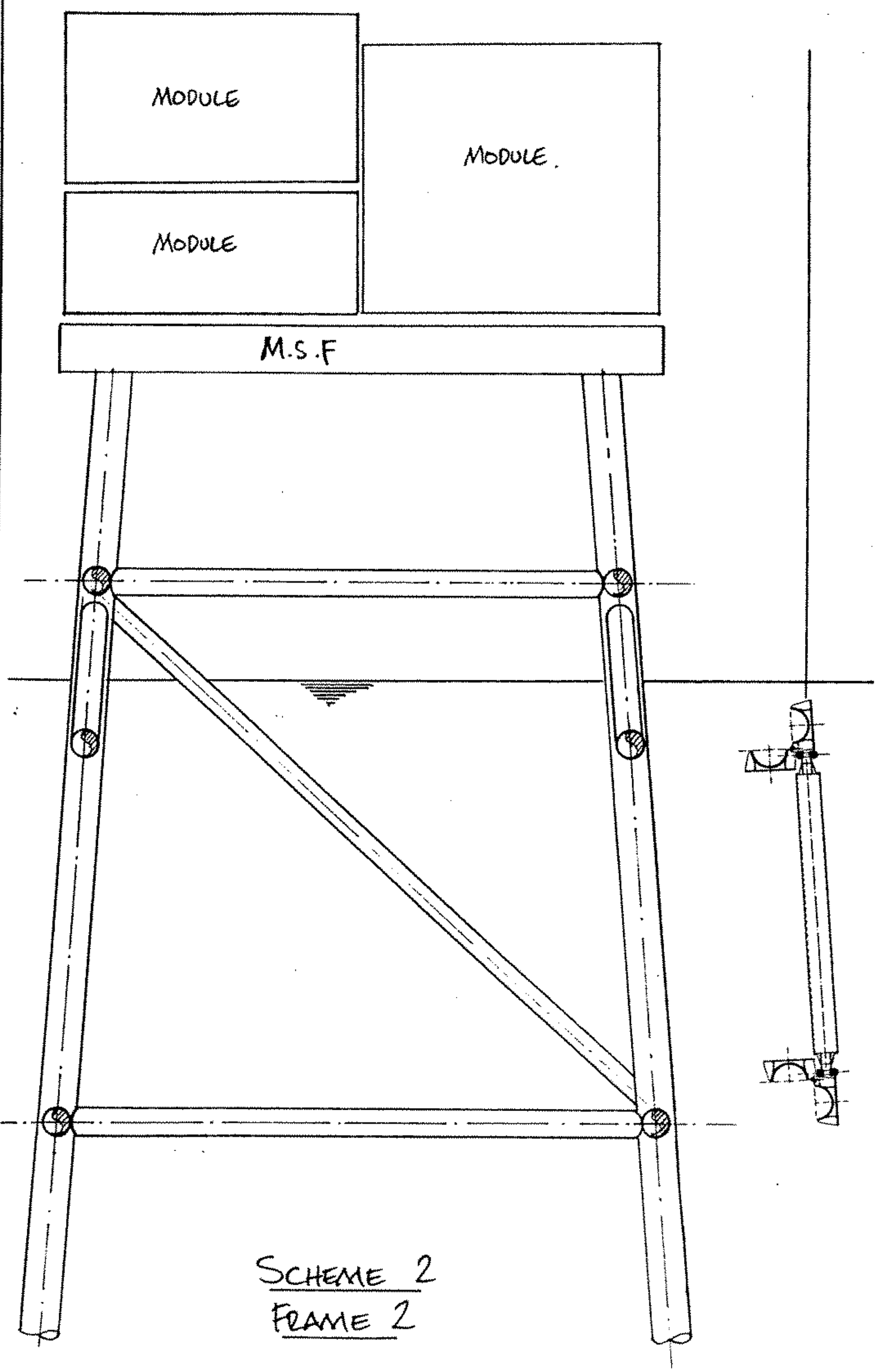

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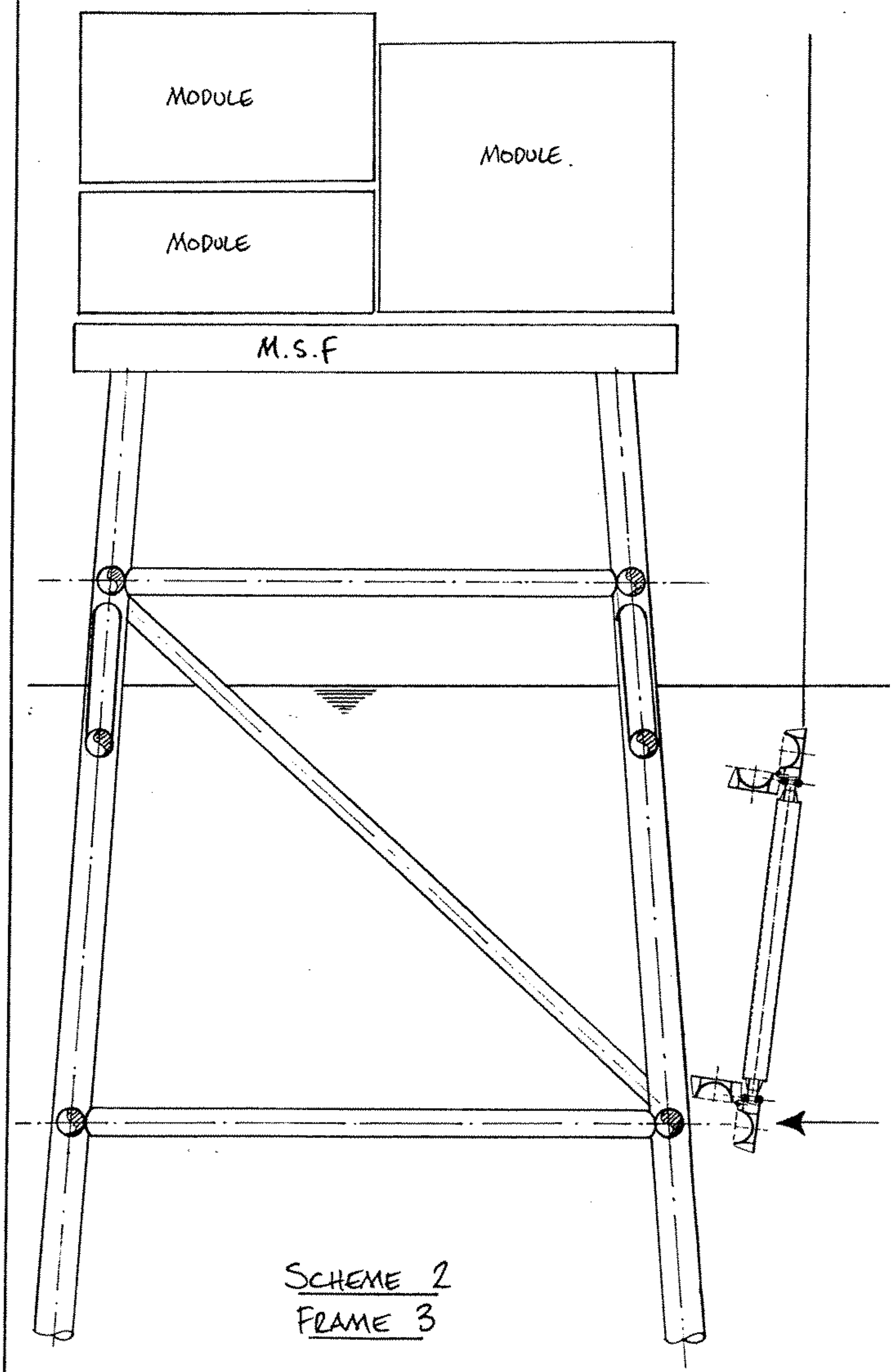

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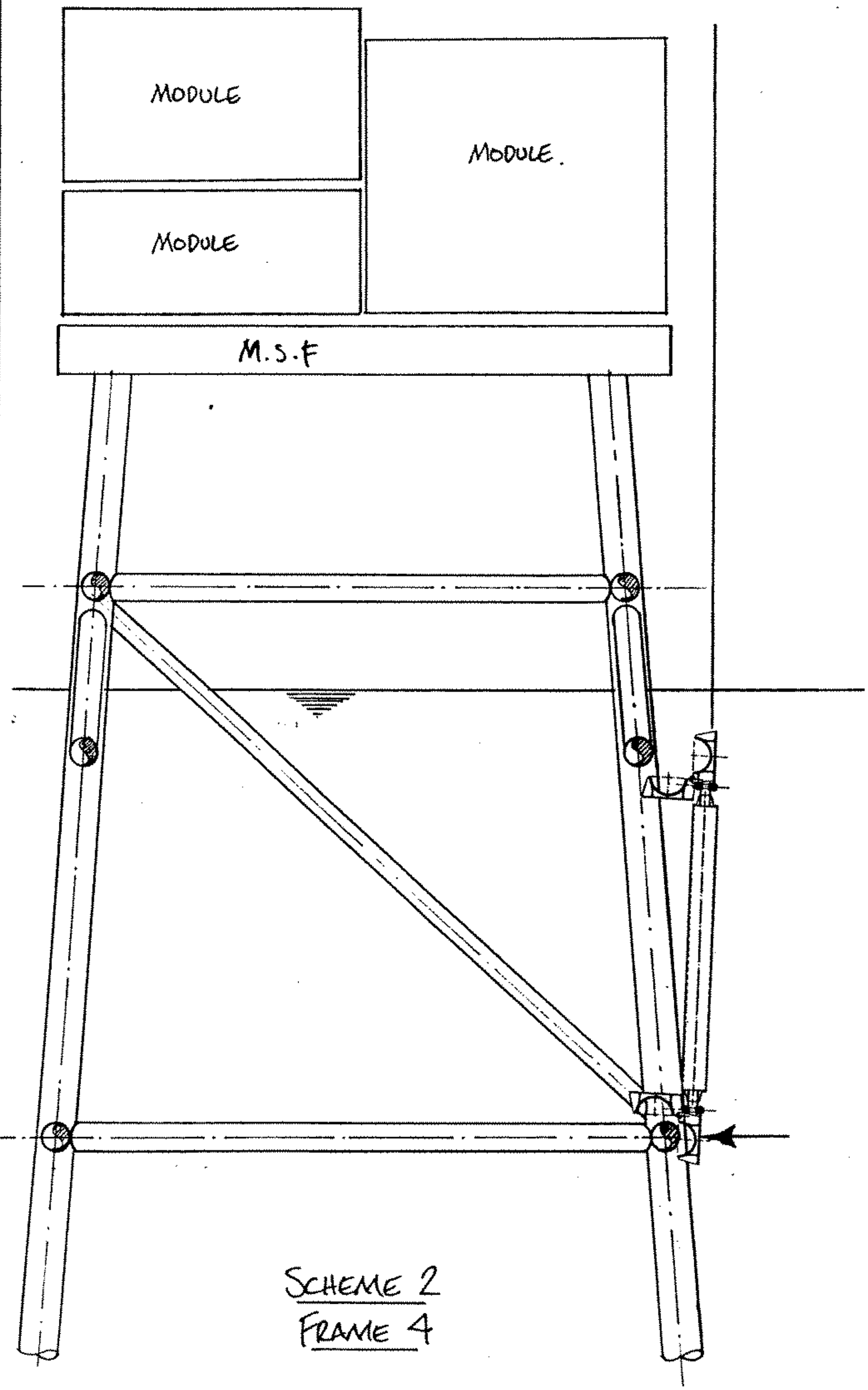

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	Checked by	Date Jan '94	

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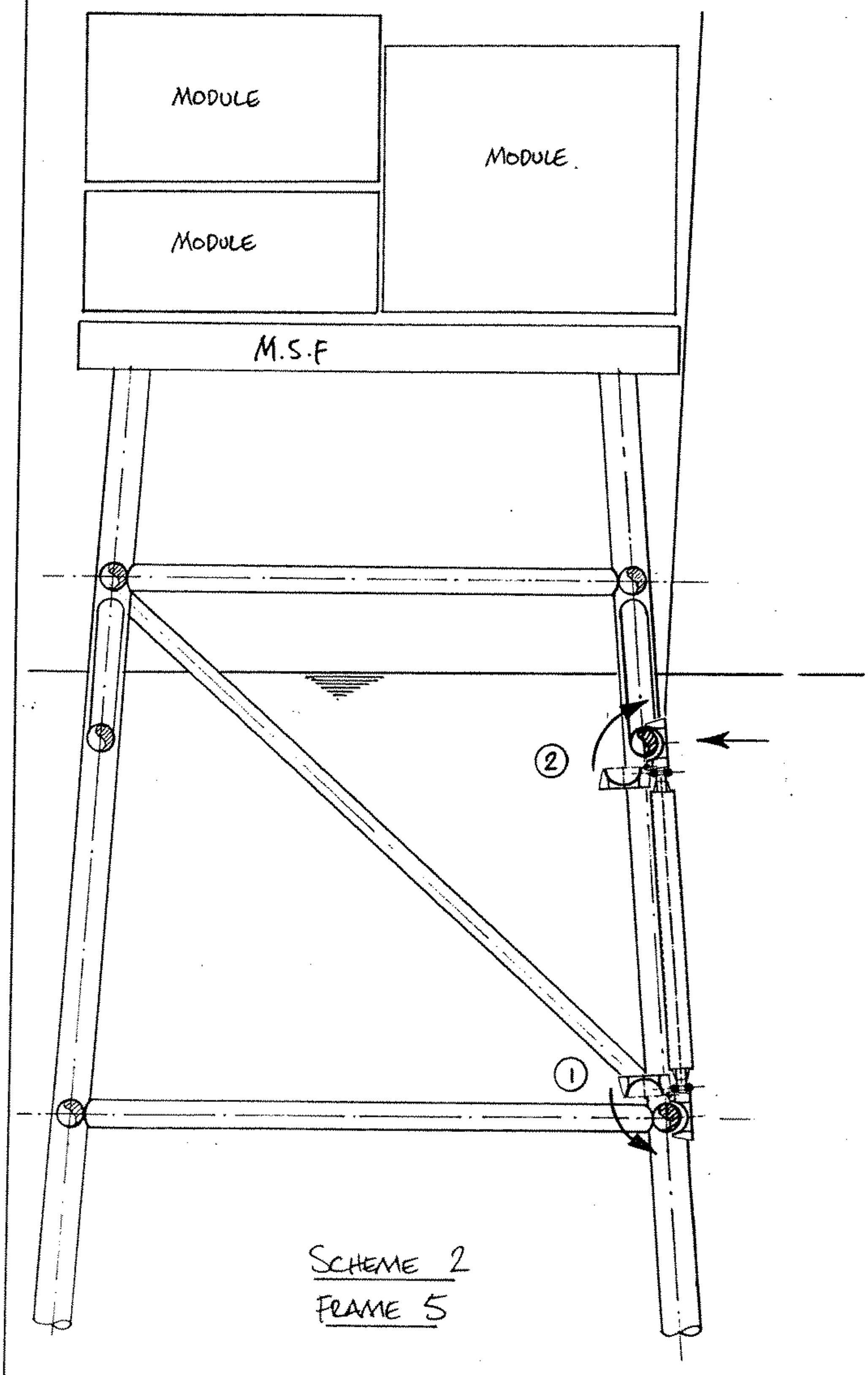

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APPENDIX E

MOBIL BERYL BRAVO DIVERLESS STRUCTURAL CLAMPS

Beryl's diverless structural clamps

The world's first diverless underwater structural repairs were successfully implemented in the North Sea last year. Installation of structural stressed clamps to repair several seawater lift caissons on Mobil's Beryl Bravo platform was remotely controlled. ***David Galbraith*** *of Mobil North Sea,* ***Minaz Lalani*** *and* ***Paul Sincock*** *of MSL Engineering and consultant* ***Tom Geddes*** *(formerly of SubSea Offshore), describe the project, including the novel features introduced to allow diverless implementation.*

During a routine annual general visual examination of Mobil's Beryl Bravo platform by rov in 1990, two dead fish were found pinned by hydrostatic pressure to the wall of a seawater lift caisson. The platform was installed in the northern North Sea in 1983 and has a 'traditional' barge launched steel jacket with a number of caissons, including 10 used for seawater lift. It is operated by Mobil North Sea on behalf of co-venturers Amerada Hess, Enterprise Oil, BG North Sea Holdings and OMV (UK).

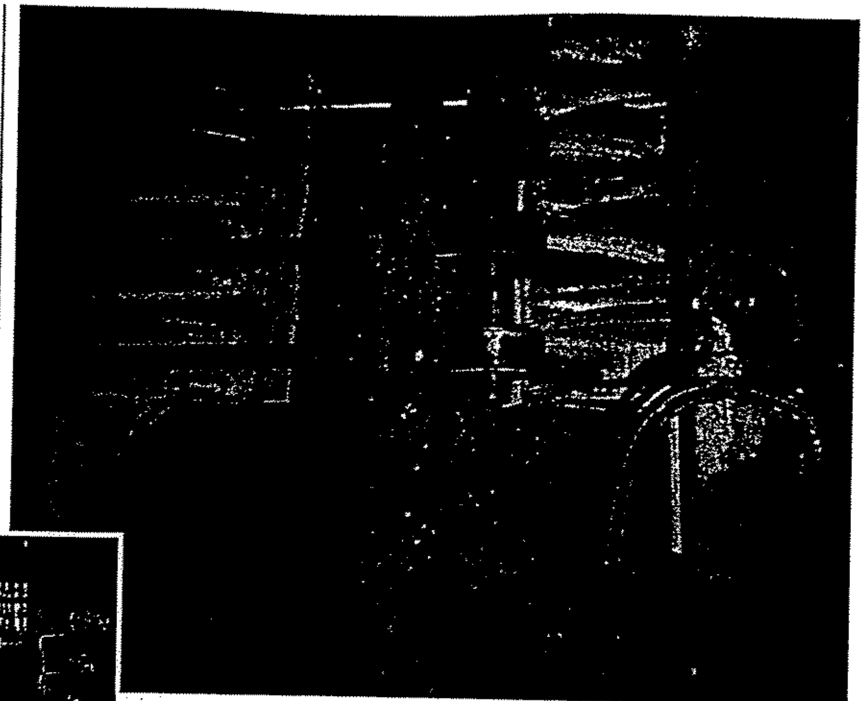
Following shutdown of the pump within the caisson, two 10mm diameter holes were observed in the caisson wall and, as the external condition and the CP potentials were good, the cause was deduced to be galvanic corrosion from the inside following upgrading of the pump strainer located inside the caisson at the level of the holes.

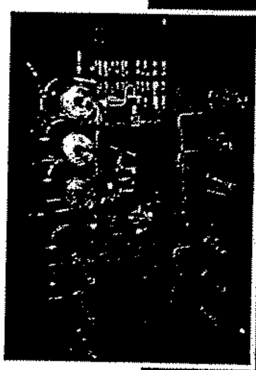
A detailed survey was conducted the following summer, and this identified severe corrosion of six seawater lift caissons, with lesser corrosion to a further four fire water caissons. The depth range of the corrosion corresponded exactly to the locations of the pump strainers and confirmed the hypothesised cause.

Fibreglass wrap plates were installed over the thinnest areas of the caissons in 1991 as an interim measure. Their purpose was to prevent the ingress of water from the holes which were located at approximately 14m water depth. The water at this depth is rich in oxygen and bacteria, and therefore poses a risk of overloading the platform's treatment facilities and contaminating the reservoir.

Initially, a conventional diver-intervention repair was designed by MSL Engineering for implementation in 1992. Subsequent analyses of the structural integrity of the caissons allowed the repairs to be postponed to 1993. The analyses also revealed that at least the two worst affected caissons would require repair before the 93/94 winter.

In 1992, Mobil appointed MSL Engineering as consulting engineer for the Beryl Bravo diverless repair project. Following further design work in 1992, Mobil was convinced that not only was a diverless repair feasible, but that it would be competitive on cost compared with conventional diver solutions. Although cost was the dominant reason for selecting a diverless repair, the significant inherent dangers of diving were also recognised. Hazards from diving operations on jackets are essentially threefold. There are unavoidable diving risks; keeping a man at a stable pressure, and having men at the underwater job site relying on umbilicals for their life support. Secondly, there are the long term effects on the diver's health to consider and, thirdly, there are the risks associated with operat-



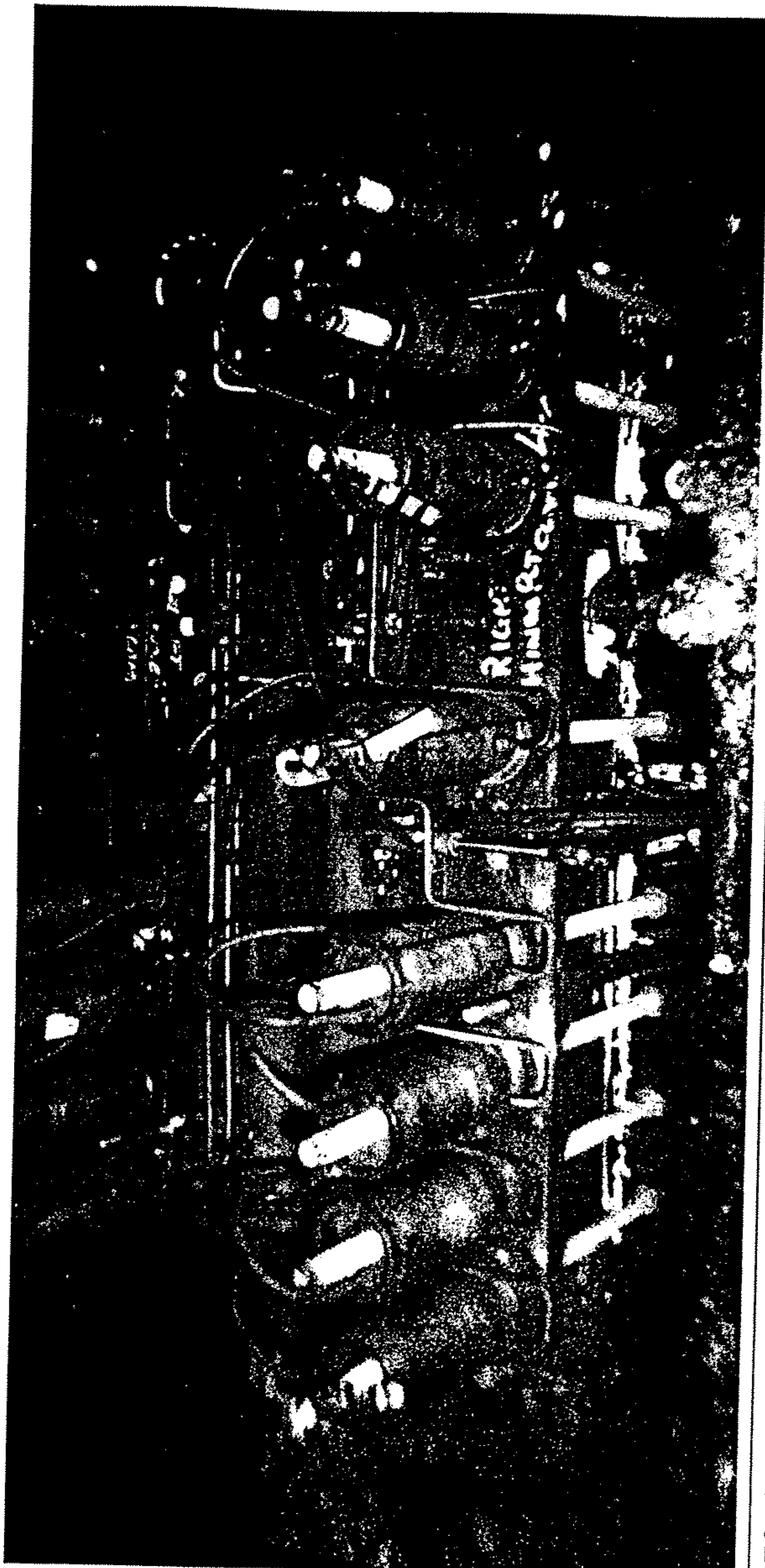


RIGHT: Diverless installed structural repair clamp used in response to severe corrosion on Mobil's Beryl B platform.

LEFT: The cause of the repair operation. The stainless steel strainer mesh for the seawater lift pump can be seen clearly through the 400mm hole. To the right is the chemical injection line.

ABOVE: Rod ends and studs passing between caisson and chemical injection line.

INSET: Clamps installed on a 508mm caisson. All the hydraulics are redundant following installation.



ing a diving support vessel immediately alongside a live platform. Platforms are generally designed to withstand an impact from a supply boat rather than the larger DSV's.

The decision to proceed with a diverless repair was consistent with the conclusions reached in a joint industry project (JIP) being undertaken by MSL Engineering on behalf of seven oil and gas operating companies and two regulatory bodies. The JIP is concerned with the preparation of a detailed design and applications manual for strengthening/repair schemes, and conceptual studies for diverless implementation of selected schemes to water depths approaching 1000m.

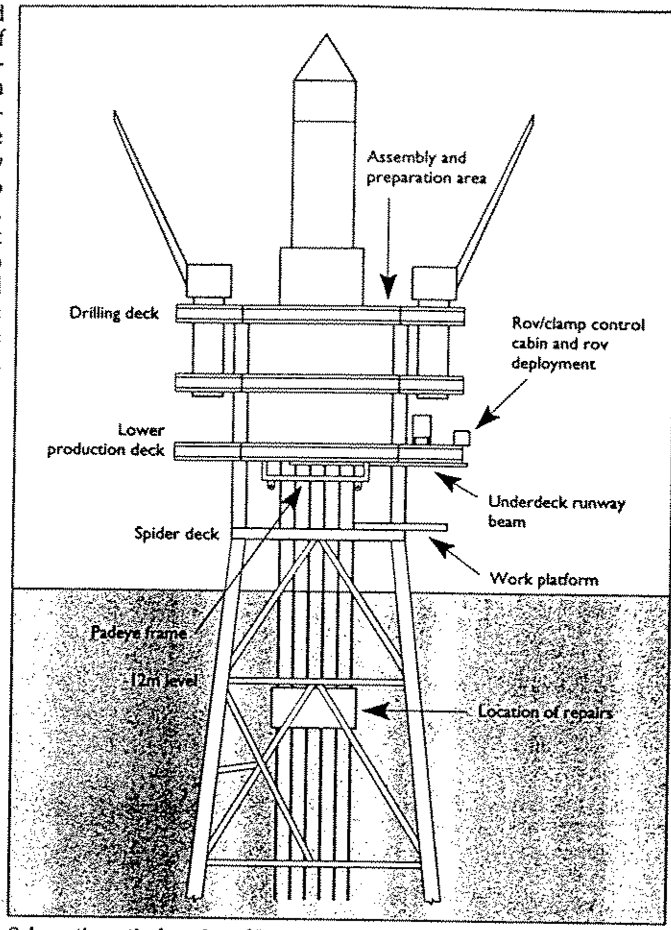
Unlike more recent platform designs, the caissons on Beryl B are fully welded to the jacket at each plan bracing elevation. This meant it was not possible to pull the caissons to the surface to effect a dry repair. Further, there was insufficient clearance inside the caissons to allow an adequate thickness of liners to be installed. The repair would have to both seal the holes in the caissons and reinstate the shear load carrying capacity of the caissons. The design case assumed that the caisson might part across the reduced wall section, thereby imparting the full load carrying burden onto the repair system.

There are several possible arrangements for effecting structural repairs. Those arrangements requiring grout were considered unsuitable due to the holes in the caissons. A smooth and sealing contact with the caisson wall was necessary to prevent any intake of water at these holes. It was concluded that an elastomer-lined stressed structural clamp would be the optimum solution.

In December 1992, Mobil awarded a long term contract for the routine underwater inspection of the Beryl area facilities and associated SAGE gas export pipeline to SubSea Offshore. The contract allowed for the inclusion of this repair, and SSOL started work on the installation aspects. The complexities of ensuring a practical, efficient installation and a satisfactory long term performance of the clamps necessitated frequent and involved communications between MSL and SSOL. The extent of the communications was typified by the fortnightly and subsequently weekly technical progress meetings where Mobil, MSL, SSOL, DnV (the certifying authority) and eventually the clamp manufacturer (OIL Engineering) were all present to discuss technical matters and solutions.

Although the six caissons requiring repair were conveniently located in a line, adjacent to an open area through the plan bracings, there were obstructions which were of major significance to the installation of the repair clamps. Each of the caissons had a 50mm chemical injection line running down the outside and supported off the caisson at 2m centres. For the four larger diameter caissons, a support for the chemical injection line was within the en velope of the clamp bodies and this dictated the orientation of the repairs. The chemical injection lines themselves are nylon lined to give chemical resistance to the highly corrosive hypochlorite solution they carry and, consequently, damage to the lines could not be tolerated. The clamp bolting arrangement therefore had to be detailed to pass between the caissons and the chemical injection line. The other major obstructions were primary diagonal members on the jacket's launch frames. There was a clearance of only 370mm between the faces of the caissons and the diagonals. Although the clamps were detailed with chamfered corners to maximise clearance, the eventual clearance between the clamps and the diagonal braces was less than 25mm.

The installation method was different for the four larger diameter caissons and the two smaller diameter ones due to the above-mentioned constraints in respect of the orientation of the chemical injection line supports. In both cases, however, a number of common features were adopted, including the use of an installation frame, lowered on wire ropes and guided by a wheeled trolley travelling down the caisson. For the larger caissons, each clamp half was mounted on hydraulically jacked guides which allowed common movement along the axis of the installation frame, but independent movement transverse to that axis. For the smaller caissons, the clamps had common movement transverse to and in line with the installation frame axis, but one half of the clamp could be rotated along one edge on a floating hinge.



Schematic vertical section of Beryl Bravo showing arrangements for the 1993 seawater lift caisson repair work.

The fundamental issue for diverless installation of clamps is the manner in which the stud bolts are threaded and tensioned. The solution adopted for the Beryl Bravo repairs was to thread 'rod ends' to the studs through which a bearing bar was fitted. This bearing bar was locked over cradles in the stiffeners of the back half of the clamps. The rod ends incorporated spherical bearings to avoid introducing damaging bending moments into the studs. The bearings are a normal component of marine diesel engines. On the front, or business face of the clamps, permanent hydraulic jacks were fitted. Normally, hydraulic tensioners would stretch the studs and the diver manually rotates the nuts to lock the stud load. For the Beryl repairs, the studs were tensioned by long-travel jacks. The tension was then locked in the studs by introducing epoxy to the jack chamber separated from the oil by a shuttle. The epoxy displaced the oil and was held under pressure until it cured, thereby 'freezing' the loads in the studs.

A series of onshore trials were conducted to finalise the installation procedures, to check the operation of the complete system and to confirm clearances. These trials involved fitting one clamp of each size onto dummy caissons which included representation of the chemical injection line and the other obstructions.

The offshore work was almost in the centre of the structure and straddled the longitudinal centreline of the platform. Consequently, substantial temporary works were required, and included a runway beam from the edge of the platform, a padeye frame, an underdeck work platform and a scaffold platform to allow access to the caisson trolley. The deck structure of Beryl Bravo is comparatively highly stressed, and this necessitated the use of the padeye support frame and governed its structural arrangement.

The topsides preparatory work was conducted by the Beryl field EPIC contractor, AMEC. As a significant amount of work was required below the deck, a total of three PHAs (primary hazard analyses) were conducted which involved all the relevant organisations and personnel. Safety features introduced beyond normal underdeck work included installation of telephones and platform status lights beside the runway beam and on the spider deck, and a watchman system controlling access to the spider deck. All personnel were required to wear buoyancy aids and fall arrestors if working above or outside normal handrail access.

Installation of the clamps was carried out in conjunction with the routine annual rov inspection of the platform. Controls for the installation, together with television monitors for the frame mounted cameras, were placed in the rov cabin so that the rov pilot and clamp controller were able to communicate easily and observe each other's monitors. Radio communication to the spider deck was established. Power packs for the clamp hydraulic functions and the epoxy spread were located adjacent to the rov cabin on the lower deck of the platform.

The installation frame and each clamp were lowered to the spider deck work platform separately, where the clamp was mounted, fully tested and adjusted if necessary. From the work platform, the frame and clamp were cross-hauled and connected to the trolley mounted on the caisson below the spider deck. Extensive use of abseilers was made for both rigging and adjustments to the assembly.

The first clamp entered the water on 1 September 1993. On 26 September, the installation frame was recovered for the last time, following the successful installation of the six clamps.

During this period, substantial time was lost due to adverse weather conditions, given the restriction imposed by the standby boats' ability to react to a man-overboard emergency. The clamp installations were carried out in weather conditions that would have prevented shallow diving operations. The greatest difficulty in high sea states was in controlling the rov used for observation.

Beryl Bravo's successful diverless repairs have demonstrated that the technology now exists to install large, structural repair or strengthening clamps subsea without diver intervention, with the attendant benefits in safety and cost. The JIP on repairs and strengthening at MSL Engineering will allow this technology to be applied in water depths approaching 1000m. The strengthening and repair of offshore installations is generally regarded as an important part of offshore structural engineering, to maintain or enhance structural integrity, and it is expected that diverless implementation techniques will be increasingly employed in the field. ■

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CONTROLLED DOCUMENT

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STRENGTHENING, MODIFICATION AND REPAIR OF OFFSHORE INSTALLATIONS

PART VII - BIBLIOGRAPHY

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STRENGTHENING, MODIFICATION AND REPAIR OF OFFSHORE INSTALLATIONS

PART VII - BIBLIOGRAPHY

STRENGTHENING, MODIFICATION AND REPAIR OF OFFSHORE INSTALLATIONS

PART VII - BIBLIOGRAPHY

CONTENTS

VII 1 INTRODUCTION

The following bibliography comprises papers, reports and other documents of relevance to the repair, strengthening and modification of steel platforms.

An assessment has been made of selected documents and keywords assigned to each. The keywords, in conjunction with the title of any particular document, allow a rapid appraisal of the likely content of the work. This is useful for those documents covering two or more subject areas and where clarification of the titles has been found necessary.

For ease of use, the bibliography is structured into subject areas mainly categorised according to repair technique. For added convenience, within each category, the references are arranged in reverse chronological order. The following categories are used:

- General papers, reports and works
- Welding technology:
  - dry welding
  - wet welding
  - other welding
- Weld improvement
- Sleeve/clamp technology:
  - unstressed grouted sleeves/clamps
  - stressed sleeves/clamps
- Grout-filled members/joints
- Grout and grouting
- Adhesives/resins
- Bolts/bolting and bolted repairs
- Miscellaneous techniques
- Local assessment of damage.

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VII 2.2 WELDING TECHNOLOGY

VII 2.2.1 <u>Dry Welding</u>

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VII 2.3 WELD IMPROVEMENT

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VII 2.4 SLEEVE/CLAMP TECHNOLOGY

VII 2.4.1 Unstressed Grouted Sleeves/Clamps

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Keywords: Grouted sleeves data, numerical analysis.

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Keywords: Grouted clamps, fatigue.

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Keywords: Grouted sleeves, review (of JIRRP data).

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Keywords: Grouted clamps, grout filled joints, review.

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Keywords: Grouted sleeves.

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Keywords: Grouted sleeves, data (limited).

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Keywords: Grouted sleeves/clamps, data, case history (includes welding of beads).

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Keywords: Grouted sleeves, data (static and fatigue).

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Keywords: Grouted sleeves/clamps, data (summary).

VII 2.4.2 Stressed Sleeves/Clamps

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