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PARAMETRIC REVIEW OF POST-YIELD BUCKLING

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16. Abstract

This work consists of a parametric study to better understand post-yield stability of ship side structure subjected to ice loads. The primary objective of this phase of the work was to check the Equivalent Standards with regard to the adequacy of the design equations for the stability of frames in the post-yield condition.

The finite element method (FEM) was used to determine the response of bay stiffened panel models designed initially using the Equivalent Standards, and then for modified designs deviating from the Equivalent Standards. A nonlinear FE analysis of each panel was performed up to or exceeding the calculated ice load. The material model used was elastic-plastic following von Mises failure criteria. Ship response to ice loads was represented by combining bi-axial in-plane compressive loads due to the ship global response with the ice loads normal to the panel.

The results of the parametric study indicate that there is no clear relationship between the Equivalent Standards stability parameters (span and web ratio) and the FE determined buckling load levels (i.e. the stability) in the post-yield condition. The parametric study showed that the parameters that have the most significant effect on post-yield stability for angle and tee section main frames are: in-plane stresses, material yield stress, and tilt angle.

The parametric study indicates that the Equivalent Standards stability criteria are ineffective in controlling post-yield stability. The equations are based on span and web ratios, which control stability prior to yielding. However, in the post-yield condition they have little or no effect on stability.

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16. Résumé

Ce rapport donne les résultats d'une étude paramétrique sur le comportement en stabilité dans le domaine plastique du bordé d'un brise-glace. La présente phase avait pour objet principal de vérifier l'adéquation des Normes équivalentes et des critères de calcul qu'elles stipulent concernant la stabilité des structures navales.

On s'est servi de la méthode des éléments finis pour analyser la réponse vibratoire de panneaux maquettes étudiés selon les critères des Normes équivalentes et d'autres qui s'en écartent. Une analyse non linéaire selon la méthode des éléments finis a été faite de ces panneaux soumis à des charges égalant ou dépassant une charge glacielle de valeur donnée. Le matériau utilisé était censé avoir un comportement élastique-plastique conforme au critère de ruine de von Mises. La réponse du navire aux sollicitations était représentée par deux composantes. La première formée des charges de compression biaxiale dans le plan du panneau et qui représentent la réponse d'ensemble des structures aux sollicitations et la seconde composante représentant la réponse aux efforts locaux agissant perpendiculairement au plan du panneau.

L'analyse par la méthode des éléments finis montre clairement qu'aucun rapport ne peut être établi entre les critères de stabilité institués par les Normes équivalentes (rapport de l'écartement des serres à l'épaisseur et élancement des porques), d'une part, et les valeurs de stabilité déterminées par la méthode des éléments finis dans le domaine post-élastique, d'autre part. Les paramètres observés comme déterminants du comportement des cornières et des tés en stabilité post-élastique ont été les contraintes dans le plan du panneau, la limite élastique et l'angle d'inclinaison.

La recherche montre que les critères de stabilité des Normes équivalentes ne sont pas déterminants du comportement en stabilité post-élastique. Pour le moment, les critères des Normes équivalentes sont fondés sur le rapport écartement/épaisseur des serres et sur l'élancement des porques, facteurs que l'on sait maintenant n'avoir presque pas d'effet sur la stabilité post-élastique.

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EXECUTIVE SUMMARY

This work presents the results of a parametric study on the post-yield buckling response of ship side structure to ice load. The objectives of the previous phases of work were to gain an understanding into the mechanics of post-yield buckling of primary structure and to establish confidence in nonlinear finite element analysis (FEA) for this purpose. The primary objective of this phase of the work was to check the Equivalent Standards with regard to the adequacy of the design equations for stability of primary (and secondary) structure.

A literature review was first performed to determine the most recent work (post 1990) carried out with respect to the post-yield buckling of ship structures. As in the Phase 1 literature search, most papers regarding ship structural instability have little information specifically on post-yield buckling.

The initial work involved determining the structural variables that are expected to have a significant effect on the plastic stability of icebreaking structures. These variables were then analysed to determine the potential significance of each through identification of the practical bounds used in service, and whether the parameter is expected to have an influence on plastic stability within these bounds.

From the identification and prioritization of these parameters, structural panels were designed for detailed FEA. The FEA component of this study consisted of a parametric study to determine the response of panels designed using the structural variables identified as being significant to post-yield buckling. The program ADINA was used for the numerical analyses, and the program HyperMesh was used to present the results. The boundaries of the ADINA FE models are defined with one web frame spacing forward and aft beyond the bay of interest, and one stringer spacing above and below the bay. This results in a 3x3 grid of panel bays to accurately predict the response of the main frames in the centre bay. The nonlinear FE analysis of each panel was performed using the Load Displacement Control (LDC) method in the ADINA nonlinear analyses software. The steel is assumed to behave as elastic plastic using von Mises failure criteria.

The applied loads consist of two components. The first component is bi-axial in-plane compressive pressure loads on the boundaries of the panels. The in-plane loads represent the effect of the overall response of the ship to an iceload of total magnitude, F_{max}. The second component is the local application of the ice load, F_{max}, normal to the panel as determined from the formulae provided in the Equivalent Standards.

From the results of the FEA, it is evident that there is no clear relationship between the Equivalent Standards stability parameters (span and web ratio) and the FE determined buckling load levels (i.e. the stability) in the post-yield condition. It was found that angle section main frames experienced a decrease in stability with increasing span ratio. However, with tee section main frames, the stability was almost unaffected by changes in the span ratio. As is consistent with previous studies, flat bar main frames remain stable for all of the analysed configurations.

The results of the FEA showed a clear relationship between in-plane load and stability for angle and tee main frames in that an increase in the magnitude of in-plane loads resulted in a decrease in the buckling load. The Equivalent Standards stability equations have no explicit allowance for in-plane loads. It is recommended that the equations be modified to provide such an allowance.

Distortion and/or residual stresses at allowable and/or anticipated levels were found to have little effect on stability.

For tees and angle section main frames, stability was found to decrease with increasing the tilt angle (the angle between the main frame web and a normal to the shell plating). For tee sections, it was found that the decrease in buckling load level resulting from tilting the frame was out of proportion to the corresponding decrease in the stability criteria.

It was found that increasing the material yield strength decreases the amount of yielding present at a given ice load. This has a major impact on the predicted buckling load levels and stability was found to increase with increasing material yield strength. There is no allowance in the Equivalent Standards to account for this effect in the stability rules.

From the overall assessment of the FEA results, it has been found that the parameters which have the most significant effect on post-yield stability for angle and tee section main frames are: in-plane stresses, material yield stress, and the tilt angle of the main frames. The Equivalent Standards contain an allowance for tilt angle but no allowances for in-plane stresses or material yield strength.

The stability criteria provided by the Equivalent Standards in the absence of these parameters, was found to be ineffective in controlling post-yield stability. However, since all of the panels (angles and tees main frames) buckled at loads either close to or exceeding F_{max}, one would speculate that, in general, the application of the Equivalent Standards rules results in panel designs which have sufficient stability. However, panel designs which had very low stability criteria still provided sufficient stability. This would cause us to question whether or not imposing these restrictions may cause an unnecessary financial burden on ship manufacturers.

The conclusion reached in the study is that the present stability equations in the Equivalent Standards must be modified to include the effects of in-plane stresses and yield strength on stability. These equations are presently based on span and web ratios, which have little or no effect on stability in the post-yield condition.

SOMMAIRE

Ce rapport donne les résultats d'une étude paramétrique sur le comportement en stabilité dans le domaine plastique du bordé d'un brise-glace. Les travaux des phases antérieures de ce projet de recherche avaient consisté à approfondir le mécanisme du flambement après plastification des structures principales et de déterminer les limites de confiance de l'analyse non linéaire par la méthode des éléments finis. La présente phase avait pour objet principal de vérifier l'adéquation des Normes équivalentes et des critères de calcul qu'elles stipulent concernant la stabilité des structures principales et secondaires.

Les chercheurs ont commencé par une recherche sur les ouvrages les plus récents (après 1990) traitant du flambement après plastification des structures de navires. Comme il avait été constaté pour la phase I, la plupart des ouvrages traitant de cette question ne contenaient que peu d'information sur cette forme de flambement.

Les chercheurs ont commencé par cerner les variables qu'ils estimaient déterminantes de la stabilité dans le domaine plastique des structures de brise-glace. Ils ont ensuite analysé ces variables afin d'en déterminer l'importance relative dans les limites d'applications précises, et notamment leur influence sur la stabilité dans le domaine plastique à l'intérieur de ces limites.

Après identification et hiérarchisation de ces paramètres, des panneaux ont été construits pour servir à l'analyse détaillée par la méthode des éléments finis, c'est-à-dire évaluer la réponse vibratoire de ces panneaux conçus à la lumière des paramètres que l'étude précédente avait identifiés comme déterminants du flambement après plastification. Pour les analyses numériques, on s'est servi du programme ADINA, et du programme HyperMesh pour la présentation des résultats. Les panneaux construits aux fins de l'analyse par la méthode des éléments finis avaient été définis comme étant délimités par l'espace d'une porque à l'avant et à l'arrière du panneau visé, et par l'espace d'une serre au-dessus et au-dessous du même panneau, formant un maillage de 3 x 3 panneaux qui permet de modéliser avec précision les réactions caractérisant le panneau central. Pour les analyses non linéaires par le programme ADINA, on s'est servi de la méthode des déplacements, l'acier étant censé avoir un comportement élastique-plastique conforme au critère de ruine de von Mises.

Les charges agissantes sont formées de deux composantes. La première formée des charges de compression biaxiale dans le plan du panneau et qui représentent la réponse d'ensemble des structures à des sollicitations totalisant une force égale à F_{max}. La seconde composante représente la réponse aux efforts locaux exercés par F_{max} , agissant perpendiculairement au plan du panneau et calculés d'après les critères des Normes équivalentes.

L'analyse par la méthode des éléments finis montre clairement qu'aucun rapport ne peut être établi entre les critères de stabilité institués par les Normes équivalentes (rapport de l'écartement des serres à l'épaisseur et élancement des porques), d'une part, et les valeurs de stabilité déterminées par la méthode des éléments finis dans le domaine post-élastique, d'autre part. Il a été observé que les membrures à demi-semelle (cornières) perdent de leur stabilité à mesure qu'augmente le rapport écartement/épaisseur des serres, alors que pour les membrures à semelle complète (tés), la stabilité demeure pratiquement inchangée malgré les variations dans

l'écartement des serres. Comme l'ont montré les phases antérieures, les membrures ayant la forme de plaques planes demeurent stables quelles que soient les configurations analysées.

La méthode par la méthode des éléments finis établit, dans le cas des cornières et des tés, un rapport net entre les charges dans le plan du panneau et la stabilité, du fait qu'une augmentation des charges dans ce plan se traduit par une diminution des efforts de flambement. Les Normes équivalentes ne prévoient explicitement aucune marge pour tenir compte des charges dans le plan du panneau. Il est recommandé que ces Normes équivalentes soient modifiées en conséquence.

Ni les déformations, ni les contraintes résiduelles correspondant aux efforts (de flambement) admissibles ou prévus n'ont, au vu des expérimentations, une influence quelconque sur la stabilité.

Il a été observé que la stabilité des cornières et des tés diminue à mesure que leur angle par rapport à l'axe perpendiculaire au bordé augmente. Dans le cas des tés, il n'existe aucune commune mesure entre la réduction de l'effort de flambement due à l'inclinaison des membrures et la diminution de la stabilité qui en résulte.

Il est apparu aussi qu'avec un matériau de limite élastique plus élevée, la déformation résultant d'une charge glacielle donnée diminue, modifiant considérablement les calculs des efforts de flambement. Un accroissement de la limite élastique s'accompagne donc d'un accroissement de la stabilité. Il n'y a rien, non plus, dans les Normes équivalentes qui tienne compte de ce phénomène.

Les paramètres observés comme déterminants du comportement des cornières et des tés en stabilité post-élastique ont été les contraintes dans le plan du panneau, la limite élastique et l'angle d'inclinaison. Les Normes équivalentes ne tiennent compte que de ce dernier facteur.

La recherche montre que les critères des Normes équivalentes ne sont pas déterminants du comportement en stabilité post-élastique. Mais, puisque les cornières et les tés ont flambé à des charges proches du F_{max} ou supérieures à celles-ci, on en conclut que, règle générale, les panneaux conformes au critère de stabilité des Normes équivalentes possèdent une stabilité suffisante, et que les panneaux s'éloignant de ce critère possèdent eux aussi une stabilité suffisante. Il s'ensuit que l'on peut se poser la question de la pertinence de ce critère et se demander si son application n'impose aux armateurs des charges financières inutiles.

Il est recommandé que, d'après les résultats de cette recherche, les critères de stabilité des Normes équivalentes soient modifiés pour tenir compte des contraintes dans le plan du panneau et de la limite élastique des matériaux et de leur effet sur la stabilité post-élastique. Pour le moment, ces critères sont fondés sur le rapport écartement/épaisseur des serres et sur l'élancement des porques, facteurs que l'on sait maintenant n'avoir presque pas d'effet sur la stabilité post-élastique.

TABLE OF CONTENTS

TABLE OF CONTENTS (Cont'd)

LIST OF TABLES

LIST OF FIGURES

LIST OF FIGURES (Cont'd)

LIST OF FIGURES (Cont'd)

GLOSSARY OF ABBREVIATIONS, ACRONYMS, AND SYMBOLS

AFPM	Actual Frame Section Modulus
ASPPR	Arctic Shipping Pollution Prevention Regulations
ATS	Automatic Time Step
CAC	Canadian Arctic Class
CCG	Canadian Coast Guard
CISTI	Canadian Institute for Scientific and Technical Information
DPT	Ratio of Frame Spacing to Horizontal Length of Ice Print
DREA	Defense Research Establishment Atlantic
Equivalent Standards	Equivalent Standards for the Construction of Arctic Class Ships
F_{max}	Maximum Applied Ice Load According to ASPPR
FE	Finite Element
FEA	Finite Element Analysis
Hp	Horizontal Length of Ice Print
HW/TW	Web Ratio
ISSC	International Ship Structures Committee
K. Tonnes	Kilo Tonnes
LDC	Load Displacement Control
L_{DL}	Length of the Design Ice Load
LU/TW	Span Ratio for Flat Bars
LU/WF	Span Ratio
MPa	Mega Pascals (1 000 000 N)
N	Newtons
NTIS	National Technical Information Service
N/V	Frame Tilt Factor/Flexural Stress Factor
P_a	Average Pressure on Ice Print Outside Localized Higher Pressure Area
P_{av}	Average Pressure on Ice Print Inside Localized Higher Pressure Area
S	Frames Spacing
S_{min}	Minimum Stress
S_{max}	Maximum Stress
S_{yy}	Stress in the y Direction
V_p	Vertical Extent of the Design Ice Load
Δ	Displacement in Thousands of Tonnes
δ_x	Displacement in Global x Direction
δ_y	Displacement in Global y Direction
δ_z	Displacement in Global z Direction
σ_x	Rotation about Global x Direction
σ_y	Rotation about Global y Direction
σ_z	Rotation about Global z Direction

1. INTRODUCTION AND OBJECTIVES

1.1 Introduction

In 1972 the Arctic Shipping Pollution Prevention Regulations (ASPPR) were first published. During the late 1970s and early to mid 1980s there was a substantial construction boom for ships involved in Arctic icebreaking. During this time period the MV Arctic, Kigoriak, Robert Lemeur, Terry Fox, Ikaluk, and several Canadian Coast Guard icebreakers were constructed. Experience gained from the operation of these vessels showed that the ASPPR structural requirements were inadequate, particularly with respect to stability criteria.

In 1985 the Commissioner of the Canadian Coast Guard formed a subcommittee composed of government and industry representatives to review the existing regulations and to propose revisions. Using a combination of experience gained from operating in the Arctic together with a substantial research effort, a Proposal for the Revision of the Arctic Shipping Pollution Prevention Regulations was published by the subcommittee in 1989 [1].

Further refinement of these results brought about the publishing of the Equivalent Standards for the Construction of Arctic Class Ships (TP12260) [2] in 1995, hereafter, referred to as the "Equivalent Standards". The Equivalent Standards recognize that ships operating in the Arctic are subjected to massive and extremely variable ice forces. They also recognize that ship design will be economically feasible only if ships operating in the Arctic are permitted to undergo permanent plastic deformation under these extreme ice loads.

From a static strength point of view, a ductile steel plate loaded by an external pressure has been found to have a considerable amount of plastic strength subsequent to the onset of local yielding. This is a result of the transfer of loading from the initial linear bending capacity through large displacements into an in-plane membrane component. However, it has been found through inspection of ice damaged vessels [1] that failure does not typically occur in the hull plating, which has undergone substantial yielding, but in the supporting structure. This failure is almost always an instability which demonstrates itself in the form of tripping or buckling.

In order for a ship to achieve its full plastic design strength, the stiffening members must maintain their stability as far as practical as yielding progresses through the structure under extreme ice loads. The Equivalent Standards recognize this requirement and contain provisions which consider buckling in the presence of plasticity (i.e. post-yield buckling).

The work carried out in this study is the third phase of an effort being undertaken to validate and/or refine the Equivalent Standards pertaining to plastic stability and the post-yield strength of icebreaking ship structures.

Phase I of the previous work was conducted by Martec and is described in Reference [3]. Phase II was performed by MIL/Carleton and is described in Reference [4]. The first two phases established the effectiveness and reliability of the Finite Element Method (FEM) to study the post-yield response of typical icebreaker structures subjected to ice loads and to accurately predict the buckling response, even in regions of fully developed plasticity. They have provided an indication of the relative stability of different main frame sections and an understanding of the mechanics of the post-yield buckling process for several different types of main frame sections. They have helped us to understand how yielding affects structural stability and how that yielding can be either beneficial or detrimental to the stability, depending upon the type and spacing of the main frame sections employed.

This third phase of the research effort is directed at determining trends and establishing safe limits for parameters which are significant in controlling the design for stability. It was performed by using a combination of analytical and Finite Element (FE) modelling techniques to determine parameters to be modelled and to provide sufficient data to enable these trends and safe limits to be established.

Local ship stiffened panels were designed which conform to both the strength and stability criteria of the Equivalent Standards. If the Equivalent Standards stability criteria are effective in the design of panels in the post-yield response structural range, these panels should be capable of withstanding the loads associated with icebreaking activities. Using detailed FE analysis and the ASPPR defined ice loads, ship stiffened panels were analyzed to determine the effects of different design parameters on stability. Specifically, the study focuses on the performance of individual elements of the structure, exemplified by the main frames.

The intent of the study is to vary each stability parameter independently while maintaining a design strength capability. The results of the parametric study were studied to establish a relationship between each stability parameter and buckling load levels.

This report provides the results of this study.

1.2 Objectives

The objectives of this study were to determine trends and to establish safe limits for parameters which are significant in controlling the design of icebreaking vessels for stability in the post-yield condition. This had to be performed at design load levels which allow a significant amount of yielding. This was accomplished by performing a parametric study using the stability parameters established within the Equivalent Standards.

The project sought to accomplish the following:

- To verify/validate the present Equivalent Standards stability criteria in a post-yield environment;
- To identify any deficiencies in the Equivalent Standards stability criteria and identify other parameters which are seen to effect stability

2. DESCRIPTION OF POST-YIELD BUCKLING

Buckling is a phenomenon that results from a loss of lateral stiffness of a structure. Typically, this is thought of in linear terms as the point where the loss of lateral stiffness occurs as a result of linear compressive stresses from an applied load. The compressive stresses generate a negative lateral stiffness component called geometric stiffness which decreases the lateral stiffness. At the buckling load, the magnitude (negative) of the geometric stiffness equals the magnitude (positive) of the original lateral stiffness and the structure buckles. This is the linear critical buckling load which is found in common problems such as Euler column buckling. The displacement and strain response for this type of buckling is shown in the "linear buckling" curves of Figure 2.1. In this case, large displacement and material yielding occur during the buckling process, resulting in permanent deformations. Prior to buckling the structure behaves linearly (i.e. small displacements, no material yielding). Note that the curves show the expected response for an elastic perfectly plastic material.

Figure 2.1 also depicts the other types of buckling. "Linear elastic" buckling results in large displacement during buckling but the structure returns to its original shape after the load is removed with no associated inelastic strain effects (plasticity). A typical return path is indicated in the figure, however, this path depends largely upon the structure. The important point is that when the load is removed, the structure returns to its original state (prior to buckling) with no permanent deformations.

The most general buckling is "nonlinear plastic" or "post-yield buckling". As shown in the post-yield displacement and strain curves of Figure 2.1, both large displacement and yielding occur before the structure has buckled. Permanent deformations result.

Post-yield buckling cannot be thought of in linear terms. At the buckling load, the structural stiffness has completely changed from its original stiffness. While the structure does not have sufficient lateral stiffness to prevent buckling, this may not have occurred completely as a result of stiffness degradation due to compressive forces. The structure may have completely yielded in tension and buckled due to some combination of internal loads.

This is the phenomenon that is of interest in this study. As noted in Section 1, it has been found through inspection of ice damaged vessels that the most likely type of structural failure is buckling. This is even though substantial yielding has occurred in the hull plating. Therefore, large displacements (membrane stresses become apparent in plating at deflections between t/2 and t) and plasticity are occurring prior to buckling. This study is being carried out to understand the post-yield buckling response of ship structures and how effective the Equivalent Standards stability criteria are at accounting for this phenomenon.

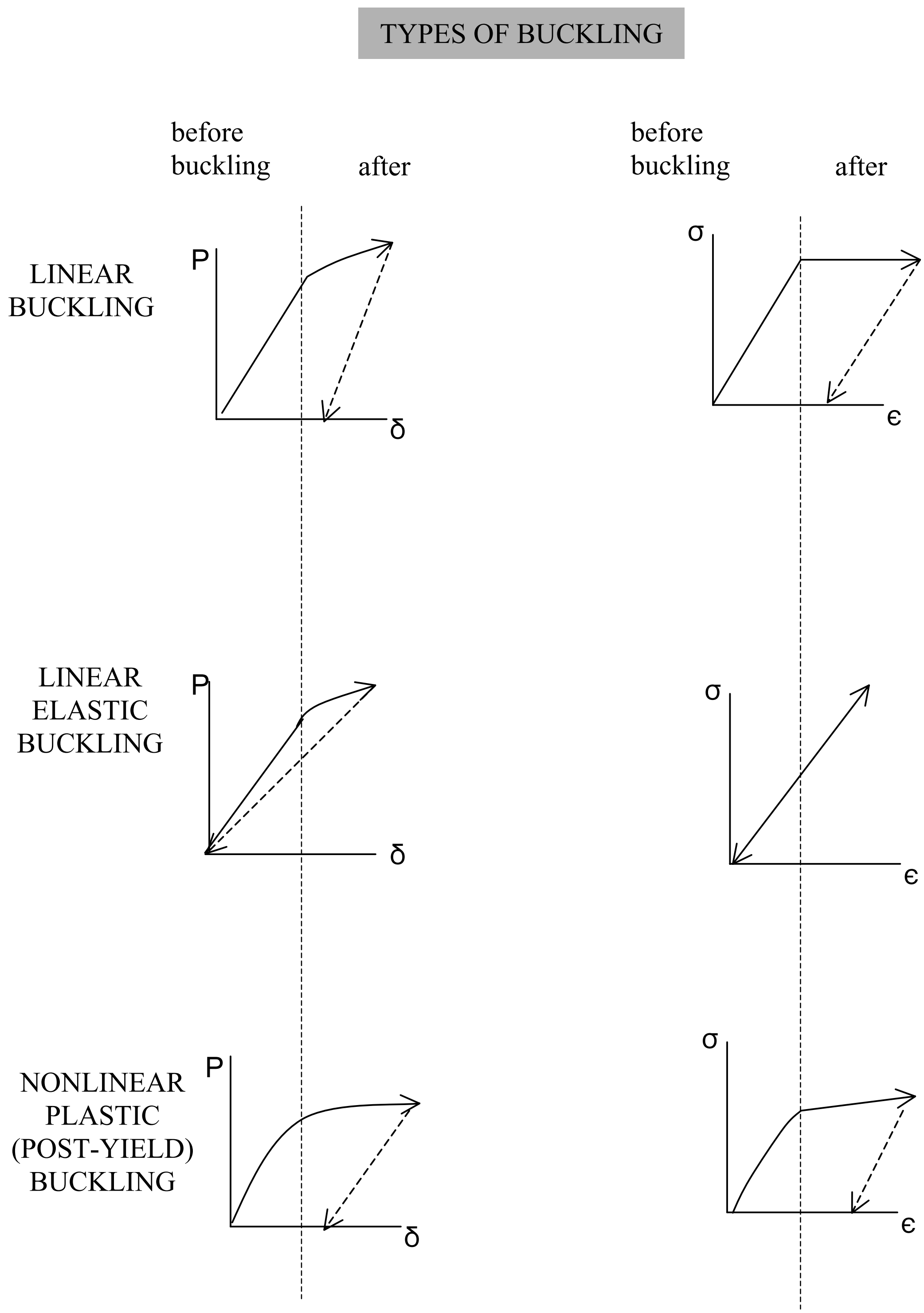


Figure 2.1. Displacement and Strain Response for Different Types of Buckling

3. COMPUTER CONFIGURATION

3.1 Software

The finite element program ADINA [5] was used to perform all of the analyses carried out in this study. The colour plots showing colour contour plots of the FE model results were generated using the pre-postprocessing program, HyperMesh [6]. The plots showing the load displacement curves and shear force difference curves were generated using Microsoft Excel [7].

3.2 Hardware

The finite element analyses carried out in this study were executed using a Hewlett Packard HP9000-800 workstation with 128 Mb RAM, 10 Gb fixed disk, running HP-UX9.04.

4. BENCHMARK ANALYSIS

As part of the initial requirements of this study, a benchmark analysis was performed. This analysis was required to confirm that the Finite Element (FE) code (and the computer platform) to be used in this contract could accurately predict (within the estimated time) the response of the 1993 physical panel test carried out by MIL Systems Engineering Ltd. at Carleton University [4]. Martec Limited used the finite element code, ADINA, for the work in this phase.

As part of a previous contract [8], Martec Limited had performed an FE analysis of a model of the physical panel using ADINA. This work had shown very good agreement with the test results. However, for this study, a newer version of the code is being used on a different platform. Therefore, re-analysis of the benchmark problem was considered necessary. Since the previous ADINA results on the analysis of the physical panels were considered accurate, the benchmark problem results were compared with the previous ADINA analysis as an indication of accuracy.

The results of the benchmark analysis demonstrated that the newest ADINA release running on the new computer configuration produced the same results as in Reference [8] and therefore can reliably predict the post-yield response of the test panel. Execution times were as expected and conform to the predictions of the proposal [9]. A separate report was developed that presents the results of this benchmark analysis. This report is found in Reference [10] and a copy is provided as Appendix A.

5. LITERATURE STUDY

A literature study was carried out to determine if any new work has been performed in the field of post-yield buckling of structures since the Phase I study conducted by Martec Limited [3]. The previous study included a literature search with material collected up to 1991. The literature search for the current study will include relevant papers post 1990. A copy of relevant abstracts (References [11] to [18] uncovered in the literature search is contained in Appendix B.

Particular emphasis was placed upon post-yield structural stability of transversely loaded plated structures, plastic design of such, and buckling and tripping as it relates to stiffened panels. Any items which were covered in the reports of the previous two work phases are not reported. The best source of information was found in a review of literature for the International Ship Structures (ISSC) Committee III.1 on Ductile Collapse by Mr. R.S. Dow in June 1995. This was one of the documents received from Dr. Neil Pegg at DREA.

This search was conducted at the following libraries/institutions:

- Canadian Institute for Scientific and Technical Information (CISTI)
- Transport Canada - Library and Information Centre
- Transport Canada - Coast Guard Library - Fleet Systems
- Industry, Science and Technology Library
- NTIS
- Defence Research Establishment Atlantic (DREA)

Except for the studies carried out in the different phases of this particular work, there is little other research explicitly performed to determine the effect of plasticity of the buckling characteristics of stiffened panels. There has been some work carried out in this area by Dr. T. Hu at Defence Research Establishment Atlantic. A discussion of the relevance of this work plus other relevant work is provided below in the following summary of the library search. Particular areas of study are highlighted with a discussion of the work performed in that area.

Imperfections

Hu [19] analysed axially loaded tee stiffened plates (using the LDC method in ADINA) into the post collapse region of the structure. He used either the first linear buckling mode or the static deformed geometry to define geometric imperfections. Residual stresses were modelled through thermal analysis methods. He concluded that the combination of the geometric imperfections and residual stresses have a significant effect on the buckling strength of stiffened plates. This was based upon the evaluation of effective stress curves.

In another paper, Hu [20] analysed laterally loaded tee stiffened panels and found that the magnitude of the imperfection did not affect the load displacement response of the panels.

In the Phase II study [3], residual stresses were measured in the panel following the fabrication process. The stress distribution at the intersection of the main frame and outer skin is shown in Figure 5.1. This compares well with what is considered to be typical residual welding stresses as determined by Smith [21] as shown in Figure 5.2.

Boundary Conditions and Loads

Bedair and Sherbourne [22] studied the effect of boundary conditions on the elastic buckling response of stiffened plates in uniaxial compression. It was found that the elastic buckling loads increase substantially when panel boundary rotational constraints were increased. This may be significant for structure directly outside the local ice-damage area. In terms of the effect of the applied loads, it was found that in-plane load effects on elastic buckling loads were considered very significant. This is also true for plastic buckling, as found in the Phase 1 study [3].

High Strength Steel

Several papers were published in Japan regarding the use of high strength steel in ship construction. Yao [23], [24] has studied the post-buckling behaviour of plates after elastic buckling occurs due to a reduction in plate thickness. What is noteworthy here is that the elastic buckling has been a consequence of using the higher strength steels to increase the strength of the structure. Future studies in the elastic buckling properties of ship structures designed using high strength steels may be worthwhile.

Analysis Methods

The finite element method is by far the most utilized numerical procedure for nonlinear structural analysis of stiffened panels to determine a response representative of the real structure. Through the different phases of work carried out by Martec Limited and other Canadian companies [3,4,8,10] it has been shown that the FE method can reliably predict the nonlinear post-yield buckling response of ship structures subjected to in-plane and lateral loads.

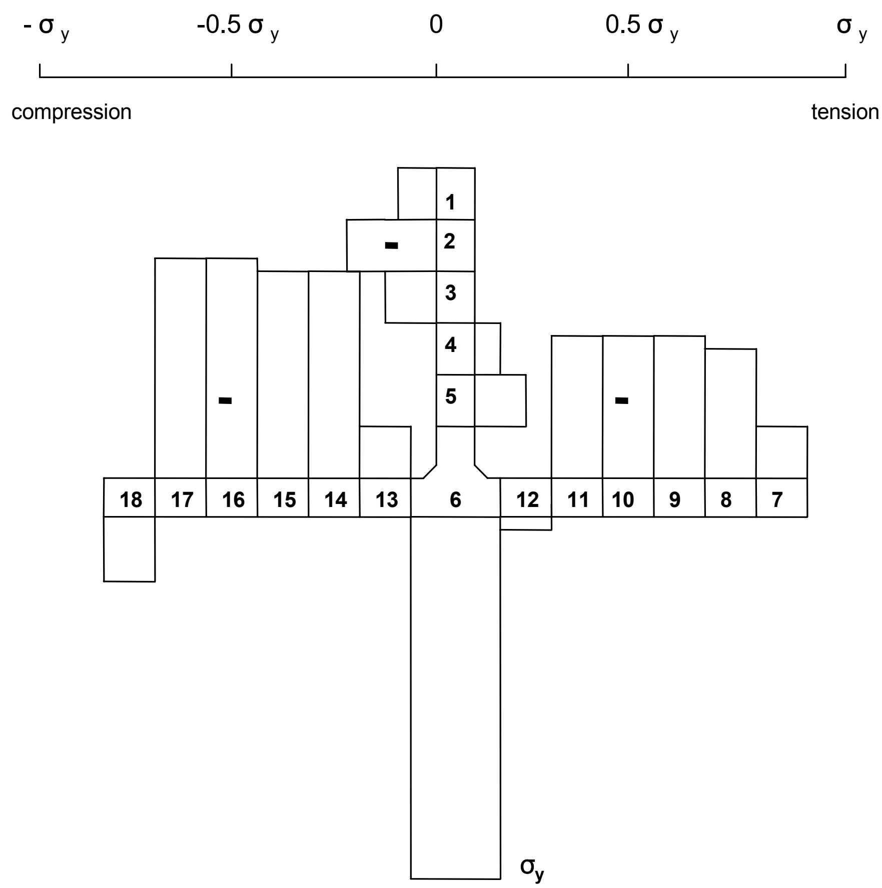


Figure 5.1. Residual Stress Pattern at the Frame/Skin Intersection from the Phase II Test

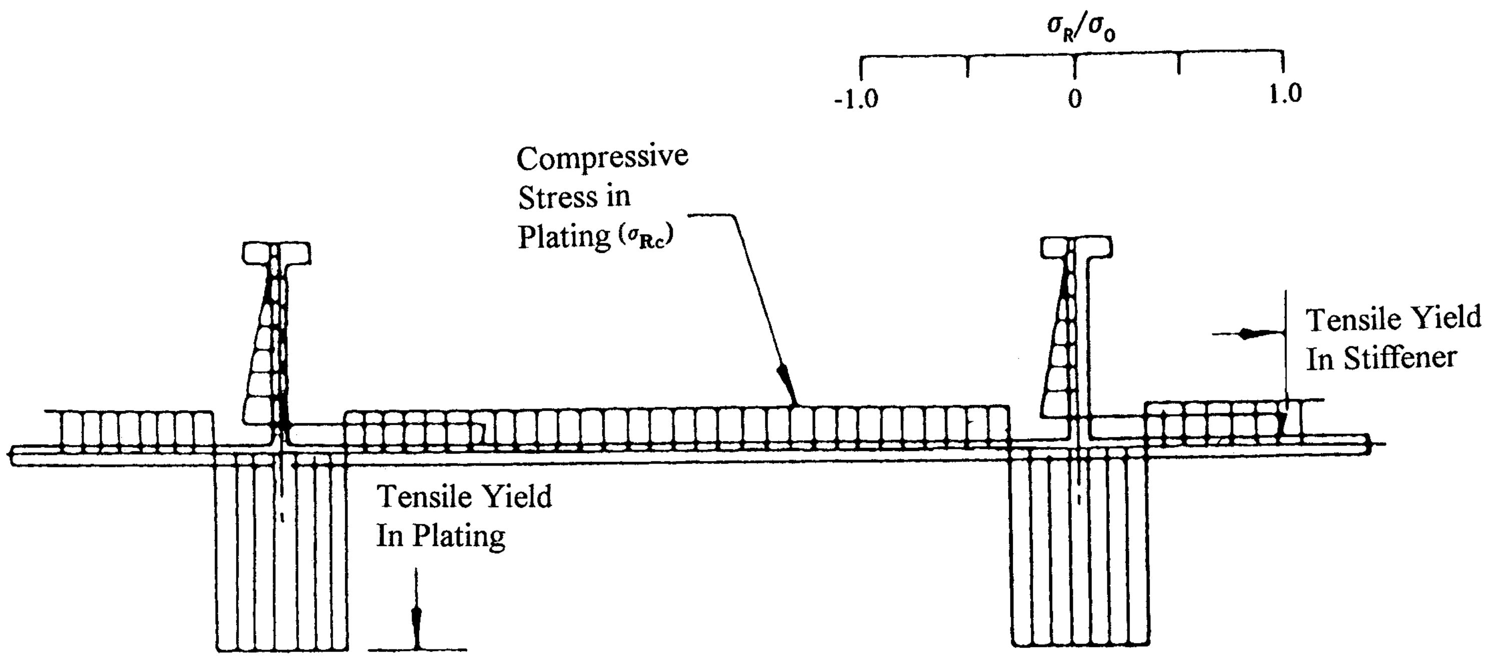


Figure 5.2. Typical Residual Welding Stresses (Reproduced from Smith [21])

6. IDENTIFICATION OF PANEL DESIGNS FOR THE PARAMETRIC STUDY

For any parametric study, it is essential to identify the type and magnitude of variables/ parameters that have an effect on the phenomena being studied. In this particular parametric study, the variables are structural parameters and influences that are expected to have a significant effect on the plastic stability of icebreaking structures in a post-yield condition.

In this section of the report, these variables are identified and analysed to determine their potential significance. This is done through identification of the practical limits of the parameters which would be encountered in service, and whether that parameter can influence plastic stability within these bounds.

Using this methodology, the most significant parameters are utilized to generate panels designs for the parametric study. Each panel is then analysed using nonlinear FEA to determine the buckling load level after significant yielding has occurred in the panel. A study of the results of the FEA then gives an indication of the effect of these parameters on the post-yield stability of ship structures.

6.1 Identification and Definition of Key Variables

The key variables which control structural stability are broken into two categories: stability parameters and stability influences. Within these categories, each parameter/influence was studied to define the limits of values that should be used in the study. This provides bounds for the investigation, which in turn enables more effective planning of the numerical analyses.

6.1.1 Stability Parameters

Stability parameters are those parameters related to the principal structural components (and their dimensions) which are found to have a direct influence on the structural stability. Table 6.1 summarizes the parameters and establishes reasonable limits on the values which would be expected to be encountered in normal design practice.

In filling out this table, the intent is to identify the max/min ranges which might be encountered in normal ship design practices for each of these parameters. Because these ships are required to meet the requirements of the Equivalent Standard to operate in the Canadian Arctic, the parameters must also conform to the Equivalent Standard strength and stability criteria. By doing this, typical maximum and minimum limits for each Canadian Arctic Class (CAC) ship can be estimated. These limits are then be used to establish reasonable ranges for the parametric study.

Table 6.1 Stability and Design Parameter Limits

	CAC1		CAC2		CAC3		CAC4	
Parameter	Max	Min	Max	Min	Max	Min	Max	Min
1. Displacement (K. Tonnes)	40	15	24	12	18	9	12	6
Power/Disp. Ratio $\frac{MW}{K.Tonnes}$	3	1	2	1	2	1	2	1
3. Steel Yield Strength (MPa)	410	230	410	230	410	230	410	230
4. Area Factor*	2.0	0.3	2.0	0.3	2.0	0.3	2.0	0.3
5 Plate Thickness Ratio* FrameSpacing/Plate Thickness	16.67	14.28	19.23	15.38	22.73	16.6 7	27.78	18.18
6. Span Ratio* - Flat Bar - Angle - Tee					 102.36 22.22 31.14	 56.4 8 12.3 2 12.6 2	 125.97	 68.24
7. Web Ratio* - Flat bar - Tee					 15.6 26.04	 9.09 12.2	 15.41	 10.01
8. Tilt Angle (degrees)*					30	0		
Orientation Angle (degrees)*					10	0		
Plate Thickness (mm)*	60	28	52	26	44	24	36	22
Frame Space (mm)*	1 000	400	1 000	400	1 000	400	1 000	400

*These parameters are described in detail in the Equivalent Standard

A Microsoft Excel spreadsheet program was developed to computerize the Equivalent Standard formulas pertaining to the strength and stability criteria. A copy of this spreadsheet is included as Table 6.2.

The approach to filling in Table 6.1 was to start with a specific CAC class of ship, select typical displacement and power for a ship of that class and to vary dimensions within reasonable ranges, while maintaining the ship design within the limits of the Equivalent Standard strength and stability criteria. Because of the high redundancy in the overall design parameters, the combinations of different designs which can be made to satisfy the criteria are virtually endless. However, by trying to use only reasonable designs, the limits for the parameters were derived. The spreadsheets associated with the calculation of the adherence to the strength and stability criteria are contained in Appendix C.

Table 6.2 Spreadsheet For Computerized Equivalent Standard Formulae
Flat Bar, MidbodC, A3, Disp = 9 000 Tonnes

DISPLACEMENT (KTonnes)	9.00	9.00	9.00	9.00	9.00	9.00	9.00	9.00
POWER (MW)	18.00	18.00	18.00	18.00	18.00	18.00	18.00	18.00
MATERIAL YIELD STRENGTH (MPa)	355.00	355.00	355.00	355.00	355.00	355.00	355.00	355.00
CAC (1,2,3, or 4)	3	3	3	3	3	3	3	3
HULL AREA (Bow or Midbody)	Midbody	Midbody	Midbody	Midbody	Midbody	Midbody	Midbody	Midbody
Arctic Class Factor	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6
Area Factor	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5
Fmax	48.973	48.973	48.973	48.973	48.973	48.973	48.973	48.973
Vp	1.166	1.166	1.166	1.166	1.166	1.166	1.166	1.166
Hp	9.331	9.331	9.331	9.331	9.331	9.331	9.331	9.331
FRAME SPACING	400	500	600	650	700	800	900	950
FRAME SPAN	2 600	2 600	2 600	2 600	2 600	2 600	2 600	2 600
SHELL PLATE DESIGN:								
Corrosion Allowance	0	0	0	0	0	0	0	0
U	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5
Omega (Degrees)	90	90	90	90	90	90	90	90
Frame Orientation Factor	1	1	1	1	1	1	1	1
Plate Design Pressure - Dpp (MPa)	4.689	3.751	3.126	2.885	2.679	2.344	2.084	1.974
Minimum Dpp (MPa)	5.750	4.600	3.833	3.538	3.286	2.875	2.556	2.421
Dpp used for Plate Thickness (MPa)	5.750	4.600	3.833	3.538	3.286	2.875	2.556	2.421
Minimum Shell Plate Thickness (mm)	22.04	24.65	27.00	28.10	29.16	31.17	33.07	33.97
Shell Plate Thickness (mm)	25.4	25.4	28.58	28.58	30.163	31.75	33.34	34.925
TRANSVERSE FRAME DESIGN:								
Type	Flat Bar	Flat Bar	Flat Bar	Flat Bar	Flat Bar	Flat Bar	Flat Bar	Flat Bar
Dimensions:								
Web Depth (mm)	288	280	282	317	290	298	303	310
Web Thickness (mm)	25.40	25.40	28.58	28.58	30.16	31.75	33.34	33.34
Flange Width (mm)								
Flange Thickness (mm)								
Phi (degrees)	90	90	90	90	90	90	90	90
REQUIRED VALUES:								
DPT	0.043	0.054	0.064	0.070	0.075	0.086	0.096	0.102
PAV	9.549	9.171	8.847	8.701	8.565	8.319	8.102	8.003
Span LB (meters)	2.600	2.600	2.600	2.600	2.600	2.600	2.600	2.600
Spacing Between Frames S (meters)	0.4	0.5	0.6	0.65	0.7	0.8	0.9	0.95
Vp / Span	0.449	0.449	0.449	0.449	0.449	0.449	0.449	0.449
Factor A (Sch 1 Table 3)	0.806	0.806	0.806	0.806	0.806	0.806	0.806	0.806
Value H	17 320	17 320	17 320	17 320	17 320	17 320	17 320	17 320
Value B	2.976	2.976	2.976	2.976	2.976	2.976	2.976	2.976
Req. Trans. Frame Shear Area (cm2)	43.62	52.36	60.61	64.58	68.47	76.00	83.27	86.82
Req. Trans. Frame Plas. Modu. (cm3)	863.80	1 036.97	1 200.34	1 278.99	1 355.89	1 505.09	1 649.04	1 719.32
MINIMUM VALUES:								
Hp min	6.00	6.00	6.00	6.00	6.00	6.00	6.00	6.00
Vp min	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75
Vp min / Span	0.288	0.288	0.288	0.288	0.288	0.288	0.288	0.288
Factor A (Sch 1 Table 3)	0.873	0.873	0.873	0.873	0.873	0.873	0.873	0.873
DPT	0.067	0.083	0.100	0.108	0.117	0.133	0.150	0.158
PAV	8.781	8.372	8.036	7.890	7.755	7.515	7.307	7.213
Factor C	1	1	1	1	1	1	1	1
Factor B	2.034	2.034	2.034	2.034	2.034	2.034	2.034	2.034
Min. Trans. Frame Shear Area (cm2)	46.58	55.51	63.94	68.00	71.98	79.72	87.21	90.87
Min. Trans. Frame Plas. Modu. (cm3)	904.71	1 078.18	1 241.91	1 320.88	1 398.20	1 548.56	1 693.97	1 765.02

Table 6.2 (Cont'd)
Flat Bar, MidbodC, A3, Disp = 9 000 Tonnes

ACTUAL VALUES:								
Factor M	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Actual Shear Area (cm2)	73.15	71.12	80.60	90.60	87.47	94.62	101.02	103.35
Actual Plastic Modulus (cm3)	1 146.29	1 086.00	1 251.57	1 565.45	1 400.28	1 559.96	1 698.86	1 782.47
STRENGTH CRITERIA:								
Actual/Required (Shear Area)	1.571	1.281	1.261	1.332	1.215	1.187	1.158	1.137
Actual/Required (Plastic Modulus)	1.267	1.007	1.008	1.185	1.001	1.007	1.003	1.010
LOCAL BUCKLING CRITERIA:								
Flange Width > 5 X Web Thick (8.(1))	NA	NA	NA	NA	NA	NA	NA	NA
Local Web Buckling (8.(2))								
- Tee or Angle – Requirements	NA	NA	NA	NA	NA	NA	NA	NA
- Tee or Angle - HW / TW	NA	NA	NA	NA	NA	NA	NA	NA
- Ratio (Required / Actual)	NA	NA	NA	NA	NA	NA	NA	NA
- Are Web Stiffeners Required?	NA	NA	NA	NA	NA	NA	NA	NA
Local Web Buckling (8.(3))								
- Flat Bar – Requirements	14.967	14.967	14.967	14.967	14.967	14.967	14.967	14.967
- Flat Bar – HW / TW	11.339	11.024	9.867	11.092	9.614	9.386	9.088	9.298
- Ratio (Required / Actual)	1.320	1.358	1.517	1.349	1.557	1.595	1.647	1.610
- Are Web Stiffeners Required?	NO	NO	NO	NO	NO	NO	NO	NO
Flange Outstand (8.94))								
- Requirements	NA	NA	NA	NA	NA	NA	NA	NA
- Tee or Angle - FOS / TF	NA	NA	NA	NA	NA	NA	NA	NA
- Ratio (Required / Actual)	NA	NA	NA	NA	NA	NA	NA	NA
TRIPPING CRITERIA:								
FPM	863.80	1 036.97	1 200.34	1 278.99	1 355.89	1 505.09	1 649.04	1 719.32
AFPM	1 146.29	1 086.00	1 251.57	1 565.45	1 400.28	1 559.96	1 698.86	1 782.47
Delta (degrees)	90.00	90.00	90.00	90.00	90.00	90.00	90.00	90.00
N	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
V	16.356	18.411	18.452	17.031	18.540	18.507	18.563	18.505
Tee:								
(i)	NA	NA	NA	NA	NA	NA	NA	NA
(ii)	NA	NA	NA	NA	NA	NA	NA	NA
(iii)	NA	NA	NA	NA	NA	NA	NA	NA
Angle	NA	NA	NA	NA	NA	NA	NA	NA
Flat Bar:								
(i)	1.0014	0.6714	1.0975	1.0024	1.3289	1.8432	5.3694	2.2785
(ii)	0.9059	0.8278	0.9227	0.8894	0.9425	0.9672	0.9958	0.9764
(iii)	0.4241	0.3767	0.4230	0.4583	0.4443	0.4685	0.4905	0.4920
Web Thks < Shell Thks (Sch.1 Pg.16)	YES	YES	YES	YES	YES	YES	YES	YES
SHEAR AREA SATISFIED	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE
PLASTIC MODULUS SATISFIED	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE
TRIPPING SATISFIED	TRUE	FALSE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE
Plate Ratio	15.75	19.69	20.99	22.74	23.21	25.20	26.99	27.20
Spane Ratio	102.36	102.36	90.97	90.97	86.20	81.89	77.98	77.98
Web Ratio	11.34	11.02	9.87	11.09	9.61	9.39	9.09	9.30

The procedure was started by putting limits on the parameters for CAC3 class ships. This was a very time-consuming process and took in the order of 40 spreadsheets, each evaluating a different design to complete. Limits were then put on flat bar designs for CAC4 class ships following the same methodology. To include the CAC1 and CAC2 classes, the range of possibilities is very large and the time required to complete this exercise is probably more than the results of the procedure warrant. Since the FE test matrix (see Section 8.4.1) is to be done initially for a selected CAC class of ship, and all of the present Canadian ships fall into CAC3 and CAC4, the established limits for CAC3 and CAC4 designs were felt to be sufficient.

6.1.2 Stability Influences

Many influences have been identified which affect the post-yield stability of stiffened panels in ship structures. Some discussion on each of these is contained in the following paragraphs. The influences are presented below in a prioritized list in which the first is expected to have the most influence on plastic stability and the last is expected to have the least effect.

In-Plane Stresses

From the Phase I work, it is apparent that in-plane stresses have an important impact on the post-yield stability of stiffened panels. Compressive in-plane stresses parallel to the main frames (i.e. in the vertical direction) cause a degradation of stability and have an important influence on the buckling load levels.

Panel in-plane stresses can result from three main actions of the ship in response to a laterally applied ice load:

1. <u>Hull/girder action to a laterally applied ice load</u> results in longitudinal bending stresses being set up as the ship responds. This results in a longitudinal compressive load in the hull on the side of the ship which is subjected to the ice load and a longitudinal tensile load in other side of the ship. This is shown in Figure 6.1.

   The compressive longitudinal stresses result in a tensile stress parallel to the main frames through a Poisson's ratio effect. This has a beneficial effect on the stability. However, because the stress is only a result of Poisson's ratio (and hence is relatively small) and is actually beneficial, it should be ignored.

   As found in the Phase I work, the magnitude of the longitudinal compressive stress is expected to be about 69 MPa. This would result in a tensile stress in the main frames of approximately .3*69=21 MPa.

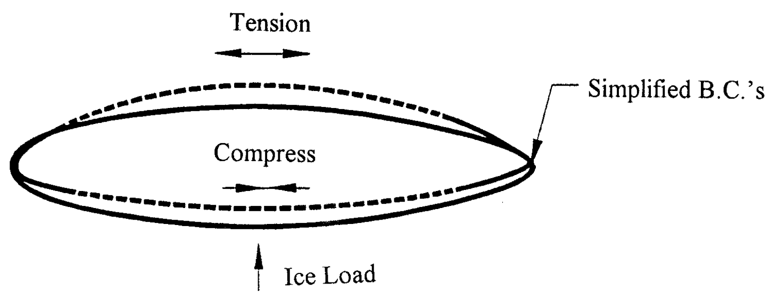


Figure 6.1 In-Plane Stresses due to Global Hull/Girder Action to Lateral Re-Load

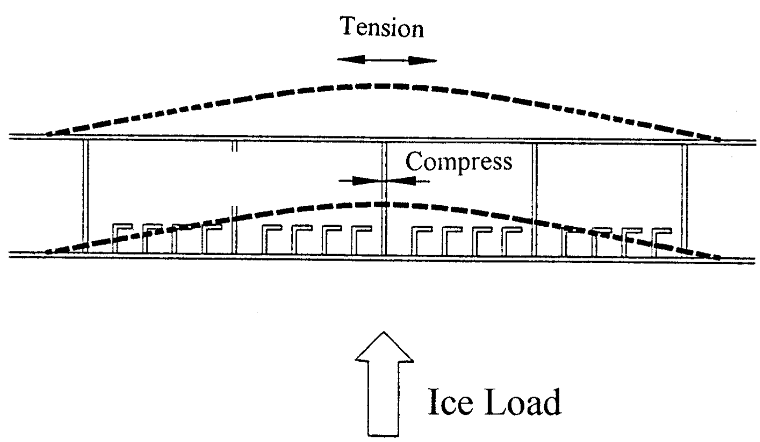


Figure 6.2 In-Plane Bi-Axial Stress due to Local-Global Response for Double Hull Structures

2. Local-global response for double hull structures in response to a laterally applied ice load results in a biaxial compressive stress in the outer hull and a longitudinal compressive stress in the main frames as shown in Figure 6.2. This longitudinal in-plane stress was found in the previous phase of the work to cause a significant degradation of instability in the main frames. The magnitude of this in-plane compressive stress is expected to range in value from a small value up to about 90 MPa.

   This influence is important and must be accounted for in the Equivalent Standard stability criteria through an allowance for in-plane stresses.

3. Local-global response for single hull structures in response to a laterally applied ice load results in a vertical compressive stress component in the hull of the ship as the hull plating forms the outer flange of the deep web frames. This phenomenon is illustrated schematically in Figure 6.3. These stresses are expected to be significant and should result in compressive in-plane stresses for the main frames which are adjacent to the deep webs which are of the same order of magnitude as for the double hull type of ship.

Boundary Conditions

The local/global assumed boundary conditions will have a substantial effect on the results. The main effect that they have is to control the in-plane stresses which will exist in the webs (and hence the outer plating). Because the in-plane stresses described in the previous paragraph are largely a function of the unsupported length of the deep webs, the boundary conditions on the deep webs have a controlling influence. Examples of this are, if the deep webs are assumed to be pinned where they intersect a major deck (as would be the case for the upper deck) or fixed where they cross a major deck. This is illustrated in Figure 6.4. The boundary conditions will range from fixed to pinned. For the purposes of this study, the maximum in-plane stress will occur when the boundary conditions are pinned. Under this boundary condition, the in-plane stresses will range up to about 100 MPa for a typical CAC3 or CAC4 vessel.

There is a second boundary condition effect which is somewhat more numerical in nature. This pertains to the boundary conditions which are applied to the FE model of the 3x3 panel which is used to model an idealized portion of the hull. This modeling effect is presented in Section 8.3 where the boundary conditions which must be applied analytically to the panel to produce accurate results are determined.

Construction Tolerances

Construction tolerances have the effect of causing plate distortions. The estimates for the values of distortion are described statistically in a paper uncovered in the literature search entitled, “Statistics of Ship Plating Distortions”, by M. Kmiecik [25]. This paper contains measured results of post-welding distortions of ship hull shell plating.

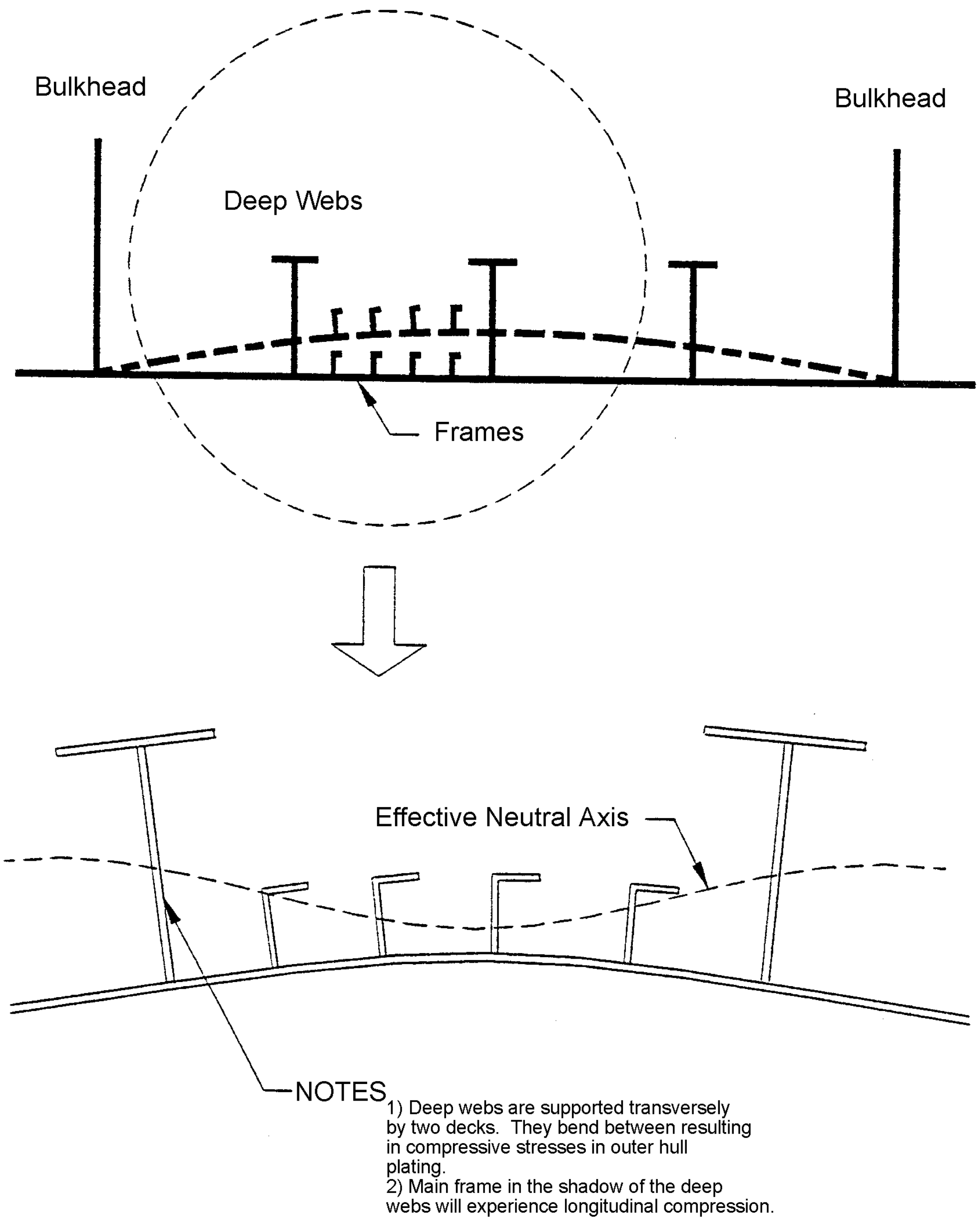


Figure 6.3 In-Plane bi-Axial Stress due to Local-Global Response for Single Hull Structures

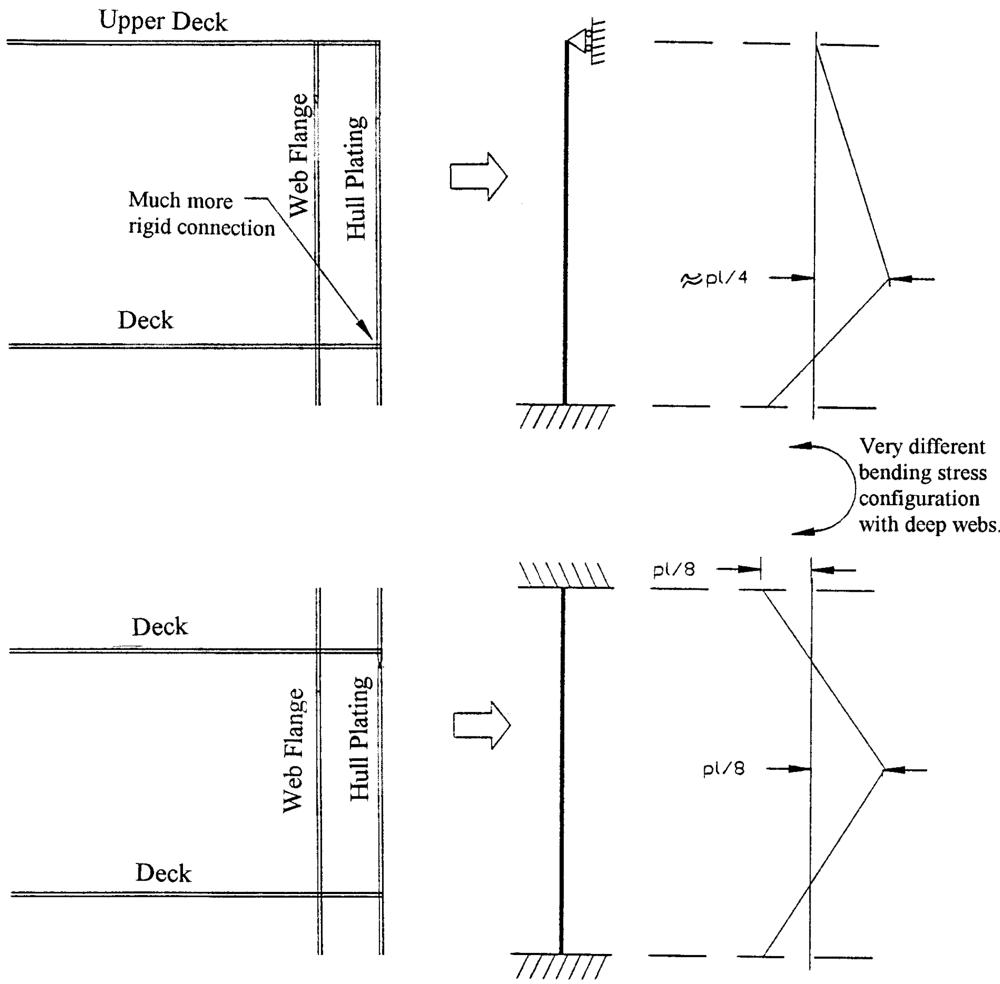


Figure 6.4 Effect of Boundary Conditions on Outer Hull Panel Models

In total, 1 988 plates from different types of ships were examined during the study. The measuring system for this research required at least 25 measurement points on a plate surface along one straight line of measuring path. For one plate, three to five such paths were usually employed.

The paper suggests that the total distortion of a plate is a result of the sum of two components, one which defines the twisting of the plate and the other which defines the bending displacement.

The results of the statistical analysis in this paper provided a ratio of maximum deflection to plate thickness for different aspect ratios (a/b) and slenderness ratios (b/t), where a, b, and t define the long dimension, short dimension and thickness of the plates, respectively.

For the panels used in our present study, the maximum senderness ratio which is expected to occur has been identified in the stability parameter study as about 30. Using this ratio, the maximum distortion as a percentage of plate thickness can be determined from the statistical study outlined in Kmiecik's paper. The paper indicates that the maximum expected distortion expected for our panels is about 18 percent of plate thickness.

As far as permissible imperfections, the Canadian Coast Guard Welding Specification [26] provides limitations on deformations. Lateral stiffener deformations cannot exceed 5°. The maximum plating deformation varies for different ship locations, with the limit being 9.0 mm at several locations. In the shell plating the maximum parallel body plating deformation is 6.0 mm, and the maximum fore and aft body plating deformation is 7.0 mm. Any imperfections that are used in this study will be based on the CCG values.

Residual Welding Stresses

Residual welding stresses in ship plating are typically the result of welding stringers or frames to the hull. The distribution of stresses, as shown by Smith [21], is characterized by a region of high plate tensile stresses near stiffener attachments balanced by lower compressive stresses in the rest of the plate. A region of stiffener tensile stresses also exists near the web/hull interface. This rapidly changes to a zone of compressive stress that reduces linearly towards the stiffener flange. The distribution of these stresses is shown schematically in Figure 5.2 (reproduced from Smith).

These values compare well with the residual stresses measured in the test panel in the Phase II work [4]. The "method of sections" was used to measure the residual stress pattern near the main frame web intersection with the shell plating. The residual stress pattern for a typical main frame is shown in Figure 5.1. The residual stress pattern and magnitudes compare well with those from Smith.

The limits of these stresses can be established as follows:

- Stiffeners: S_{min} = 0.0
  S_{max} = yield at connection of web to shell plating

- Shell Plating S_{min} = 0.05 yield stress at connection of web to shell plating
  S_{max} = yield stress in shell where stiffener is attached (tension).

Material Properties

The most important material property of steel with regards to plastic stability is the yield strength. In ship construction mild steel yield strengths are typically in the range of 230 MPa. In more recent years, higher strength steels are more commonly used in fabrication yards. The highest strength of these steels was found to be in the range of 410 Mpa.

Details of the material properties used for the study are contained in Section 8.1.2.

7. PRELIMINARY FEA STUDIES

Prior to performing the full nonlinear FEA using FE models of the designed panels, several preliminary studies were required. This is a procedure that is necessary for any large FE analysis and, in particular, for nonlinear analysis. It is necessary to undertake preliminary analysis in order to become familiar with the detailed requirements of the specific problem. These preliminary studies always save analysis time and effort.

The first study was performed to verify/validate the overall analysis methodology and to determine the extent of the structure to include in the FE model. With the large number of FE analyses to be performed, the size of the model has a significant impact on the individual model analysis times which effects the overall elapsed time in the project and the effort required to interpret the results. The first study was carried out to determine whether the model was optimized with respect to model size and accuracy of results.

The second study was performed to define a method to accurately determine the point of initiation of an instability within the structure. In the past phases of work on the effects of yielding on the stability of ship structures, identification of the point of instability has usually been "subjectively" chosen from load-displacement curves. While subjective in nature, these choices have been made with engineering judgement and expertise in this type of structural analysis. The instability initiation point has typically been the location where a dramatic change in slope occurs which is indicative of a large loss of lateral stiffness. While this procedure generally identifies the point of stability initiation, it is somewhat qualitative and a more rigorous and quantitative approach is required for review of the Equivalent Standard stability rules.

7.1 Extent of the FE Model

In order to save time in the analysis of the large number of runs identified for the parametric study, it is necessary to ensure that the model which is utilized is optimized from a size point of view. In the Phase I [3] study, a 3x3 bay model was used for most of the work. It was concluded at the end of the study that similar results could be extracted from a smaller model if proper boundary conditions were applied. That is, equivalent results could be achieved using a 1x3 bay model which employs proper boundary conditions in the form of displacements and in-plane forces. However, it was not known what level of effort would be required to achieve the required degree of equivalency such that a 1x3 bay model could be utilized to give the same accuracy as a 3x3 model in this phase of the study. To determine this, the 3x3 bay model shown in Figure 7.1a was developed. A 1x3 bay model was also developed and is shown in Figure 7.1b.

Several analyses were conducted on these models with equivalent boundary conditions and loads. The results of a linear analysis of the 3x3 and 1x3 bay models with both in-plane and lateral ice loads are shown in Figures 7.2a and 7.2b, respectively. From these plots it can be seen that the stress magnitudes in the main frames are significantly different.

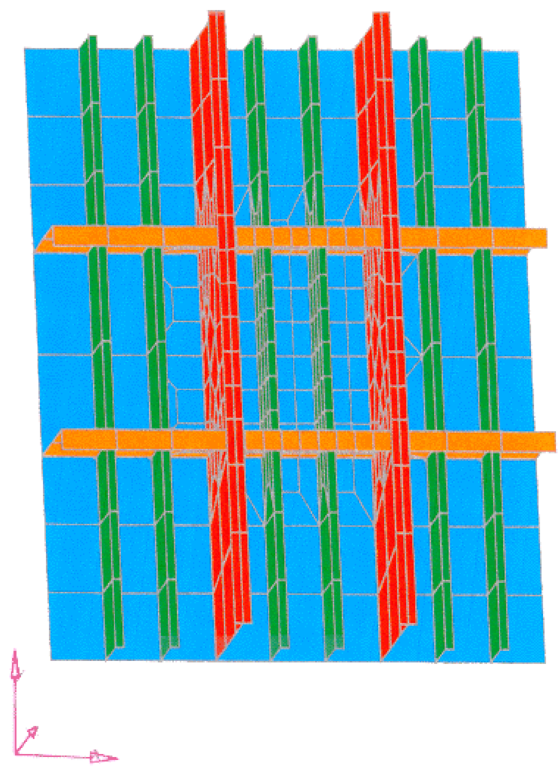


Figure 7.1(a) 3x3 Bay Model

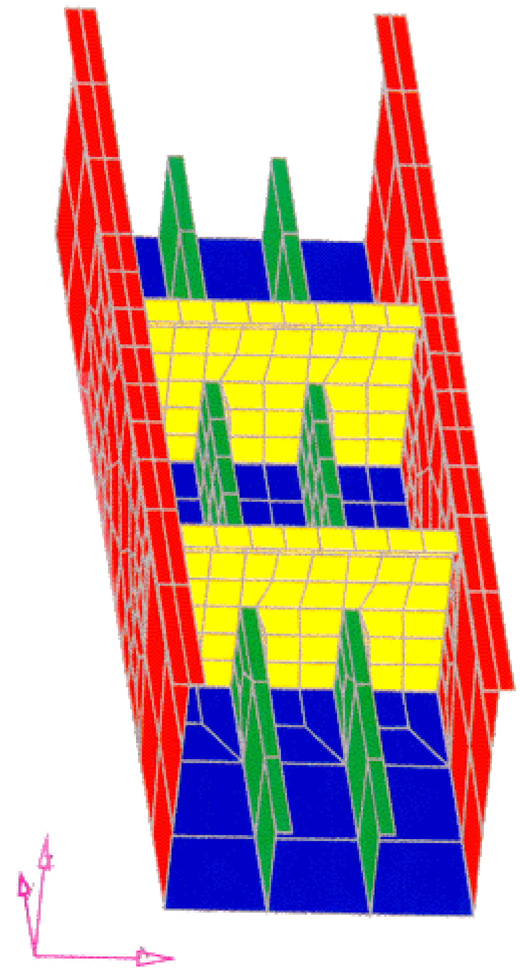


Figure 7.1(b) 1x3 Bay Model

The reason for the differences is related to the difference in stiffness between the two models. In the 1x3 bay model, the stiffness of the two outside bays is removed while the majority of the applied ice load is still applied to the center bay. This dramatic reduction in stiffness without any significant change in boundary conditions or loads results in a significantly different response between the two models. If prescribed displacements are used as boundary conditions on the 1x3 bay model, then the removed stiffness is accounted for in the prescribed displacements. This approach would work well in predicting linear response. However, this would impose linear displacements at the edges of the center bay panel, where a highly nonlinear response occurs. Therefore, prescribed displacements cannot be used on the boundaries of the 1x3 bay model. Consequently, the conclusion for the boundary condition study is that the 1x3 model cannot be used to accurately predict the nonlinear response for the panel analyzed. The 3x3 model was used for this study. A fairly crude mesh will be used to model the outer bays and a detailed mesh was used in the center bay. In this manner, boundary conditions are accurately imposed on the center bay without too much of a penalty in model size.

7.2 Identification of Instabilities

Two methods were investigated for the identification of the buckling load level of main frames in the ship stiffened panel structures subjected to a normal pressure load from the application of ice forces. In the Phase I [3] study, a heavy reliance was put on the utilization of load-displacement curves. In this study, a more quantitative approach is desired.

7.2.1 Linear Eigenvalue Method

One proposed procedure for the identification of instabilities is to perform linear eigenvalue buckling analyses at different load levels within a nonlinear analysis. Linearized (eigenvalue) buckling provides a quantitative prediction of the buckling load. However, when this is performed using the original stiffness matrix for the unloaded structure, the effects of yielding and large displacement are not accounted for. As the structure is loaded, both yielding and large displacements are expected to occur. Either of these phenomena affects the stiffness matrix and hence changes the load level at which buckling occurs. Therefore, when a linearized buckling analysis is performed under load, the predicted buckling load changes to account for the change in stiffness.

An investigation was undertaken to determine if a linearized buckling analysis, performed under loading such that it includes the effects of yielding and large displacements, can be used to accurately determine the buckling load. However, this investigation showed that convergence to an accurate prediction was found to be a time-consuming and expensive venture. As a consequence, this procedure is not expected to provide better predictions than those obtained by analysis of load-displacement curves. Details of the investigation are contained in Appendix D.

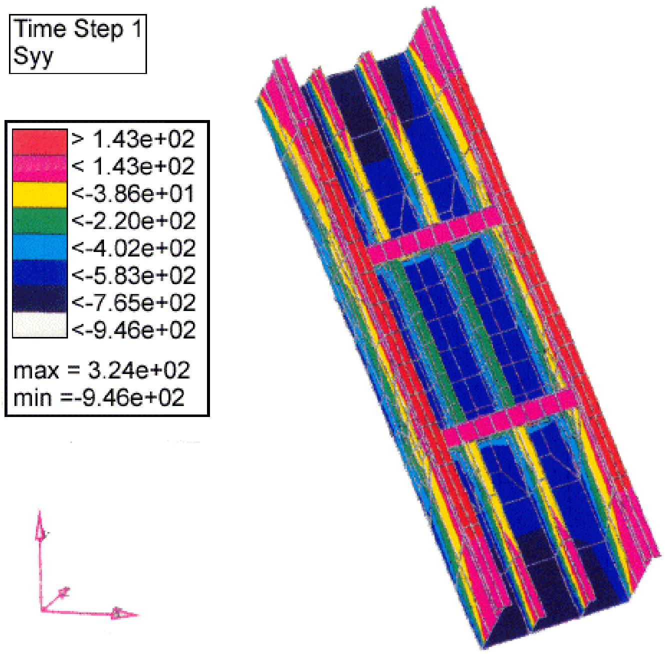


Figure 7.2(a) Stress Results of 3x3 Bay Model

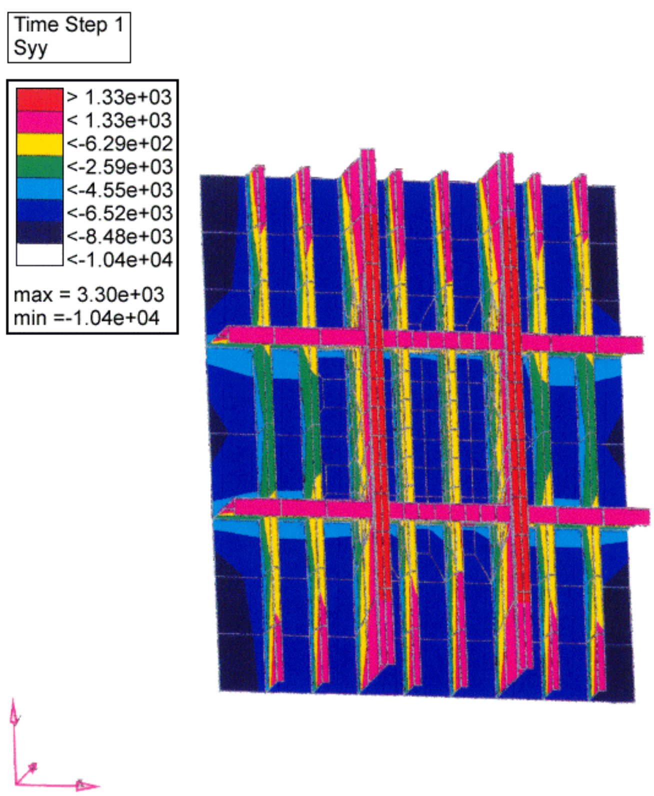


Figure 7.2(b) Stress Results of 1x3 Bay Model

Another alternative was then investigated. This method is to base the initiation of buckling on the shear force carried by the structural member. When a member buckles, it sheds load that it previously was capable of carrying which results in a decrease in shear force. A description of this methodology and the study to determine the viability of using it is presented in the following section.

7.2.2 Shear Force Difference Method

The basis for the shear force difference method is that the shear force existing in a member is proportional to the bending load that is carried by that member. If the shear force carried by a member decreases, then it is because the bending load carried by that member decreased. A decrease in load carrying capacity under increasing load can be associated with the onset of an instability. As the structure becomes unstable it cannot continue to carry the load which it supported prior to buckling. These loads are shed to the surrounding structure.

In order to develop a methodology to exploit this phenomenon to predict the onset of buckling, a representative test problem was defined. This problem is illustrated in Figure 7.3. The structure is basically a simple beam with pinned boundary conditions and two equally spaced vertical concentrated forces of magnitude 1 000 000 N. The corresponding shear force diagram for this load case is shown where two equal magnitude (opposite in sign) regions of shear exist. The values of shear equal the reactions at the supports which equals the individual force magnitudes.

The detailed FE model of this structure is also shown in Figure 7.3. The structural configuration consists of an angle main frame attached to a small section of shell plating. The location of the applied loads, P, are shown on the model along with two dashed lines which are drawn vertically through the model at the two sections where the shear force carried by the section is determined. At any cross section of the frame, the integration of the vertical shear stress over the area of the section will determine the total shear force carried at that section. Using an FE model, this integration can be performed by summing the product of the shear stress and the element area for all elements in a given cross section. This will give the total shear force carried by that section.

To validate the integration procedure algorithm, a nonlinear analysis was performed (using the LDC method in ADINA) on the model in Figure 7.3. The resulting vertical shear stress distribution in the beam model is shown in Figure 7.4. These results are at time step 1 with an applied load vector factor of 1.41. The total applied load is 1.14*2 000 000=2 280 000N. The integration of FE predicted shear stresses over the two sections (as indicated by the dashed lines in Figure 7.3) produces a total shear carried by the section of 2 282 950 N. These values are considered to be the same with only a difference of 0.13 percent. Therefore, the procedure to determine the shear force carried by a section of structure from the shear stress distribution using an FE mesh has been validated.

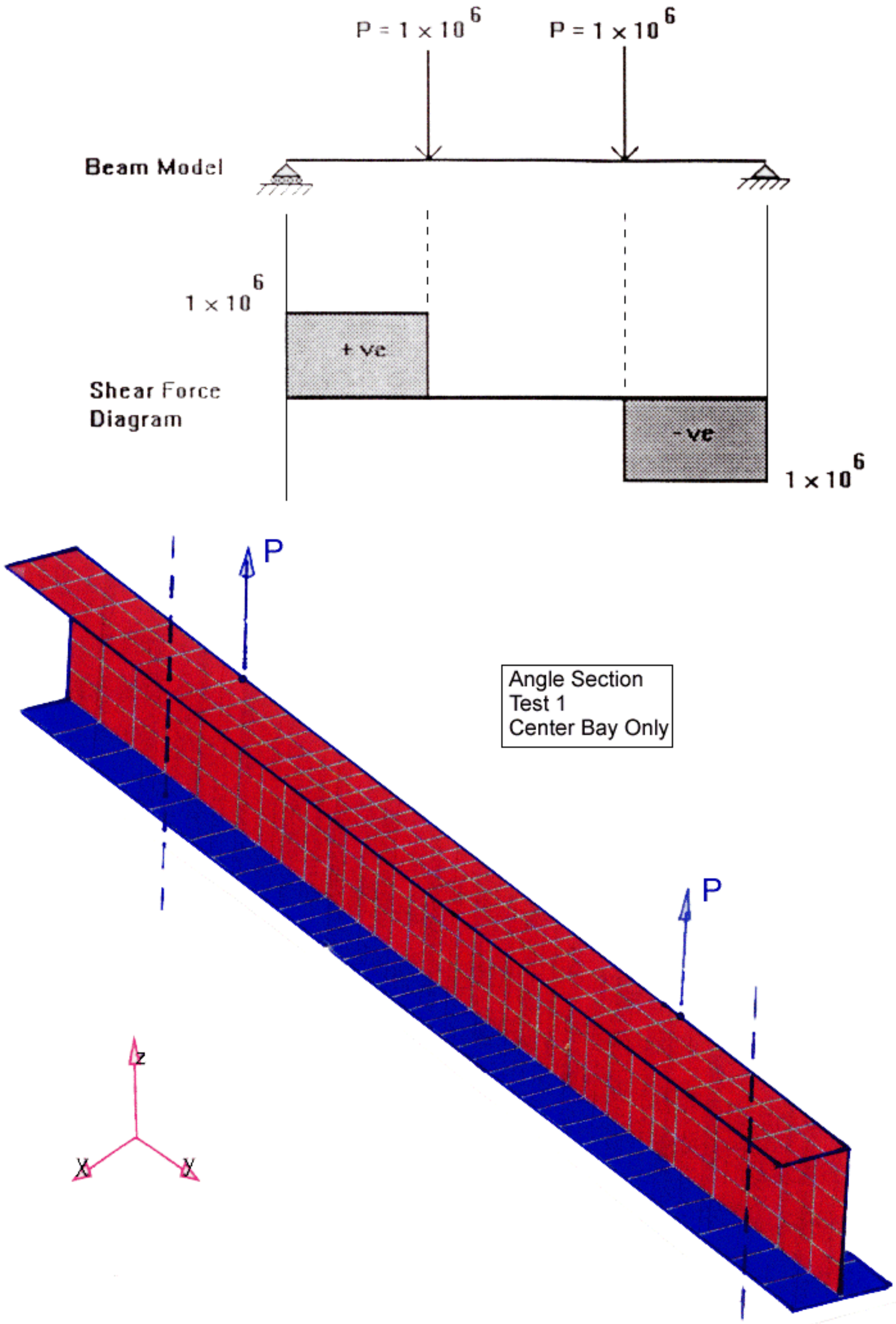


Figure 7.3 Test Problem for Validation/Development of Shear Force Methodology

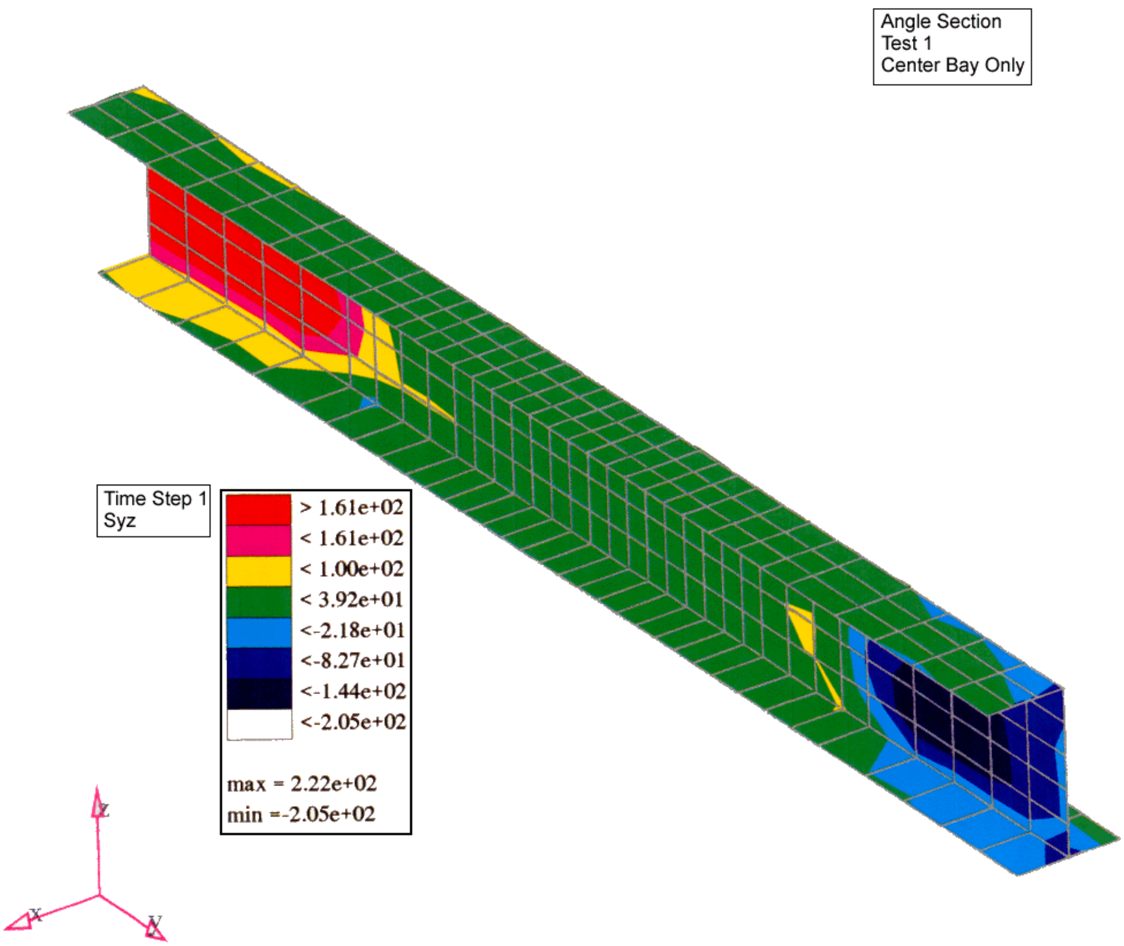


Figure 7.4 Vertical Shear Stress Distribution for Beam Sample Problem

To use this method to determine the point of initiation of an instability for a stiffened panel type structure requires a structural configuration where load shedding is possible. This allows the applied load to continue to increase while the load carried by the stiffener decreases as it is shed to the surrounding structure (as a result of buckling). In the beam structure of Figure 7.3, as long as the load is incrementally increased, the shear force across the beam will increase. For this problem, in order for the shear force to drop at a particular section, the total applied load must drop.

However, for redundant structures such as the 3x3 bay ship panels, the load carried by the particular section of interest (in this case, a main frame) must drop in order for the shear force across the section to drop. This, however, does not mean that the overall applied load to the structure has to drop. If stresses exceed yield or if buckling occurs, the stiffness of the structure changes. This can result in load shedding to other sections of the structure. In the 3x3 bay model, the load is shed from the buckled main frame to structure surrounding the main frame. Therefore, the total applied load can continue to increase while the load carried by a buckled main frame drops.

This methodology is illustrated through a nonlinear analysis of one of the 3x3 bay models used in the parametric study. The model has angle main frames and the loads and boundary conditions are the same as those used in the parametric FE analyses (see Section 8). This model is shown in Figure 7.5. The darkened lines show the main frame cross-sections where the shear force integration is performed.

The results of this analysis are shown in the curves of Figure 7.6. The top curve shows a load displacement curve of the lateral displacement at a node on the midspan of the middle main frame in the center bay at the intersection of the web and flange of the main frame. At a load of approximately 1.2 F_{max}, large lateral displacement occurs. This response is indicative of buckling and is typical of the presentation of results that have been used to identify buckling in the Phase I [3] study.

The bottom curve shows the shear force carried by the section of the middle main frame between the darkened lines of Figure 7.5. The total shear force is determined by integrating the FE predicted shear stress over the cross section areas at these sections. This curve shows that at a load level of approximately 1.08 F_{max}, the shear force at this section of the main frames drops while the total applied load continues to increase. This drop in shear force is a result of a drop in load carrying capability of the main frame. This drop in load carrying capability occurs at a load level below the initiation of instability (1.2 F_{max}) as identified by the load displacement plot in the top curve. This drop in shear force is the first indication of buckling. (It should be noted that the selection of the point of initiation of instability is visually selected from the curve. The error estimate for this methodology is predicted at ± 0.05 F_{max}).

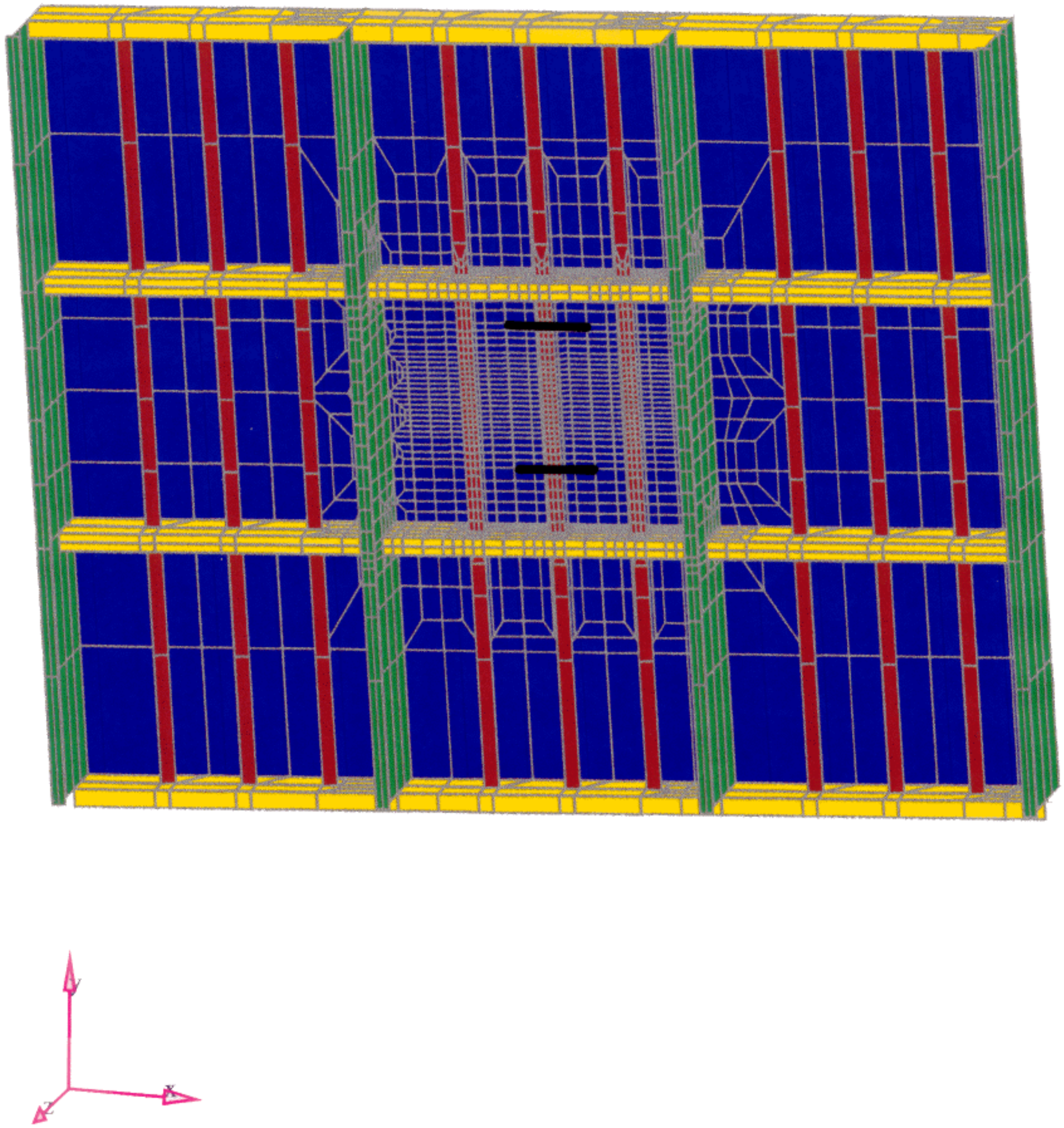
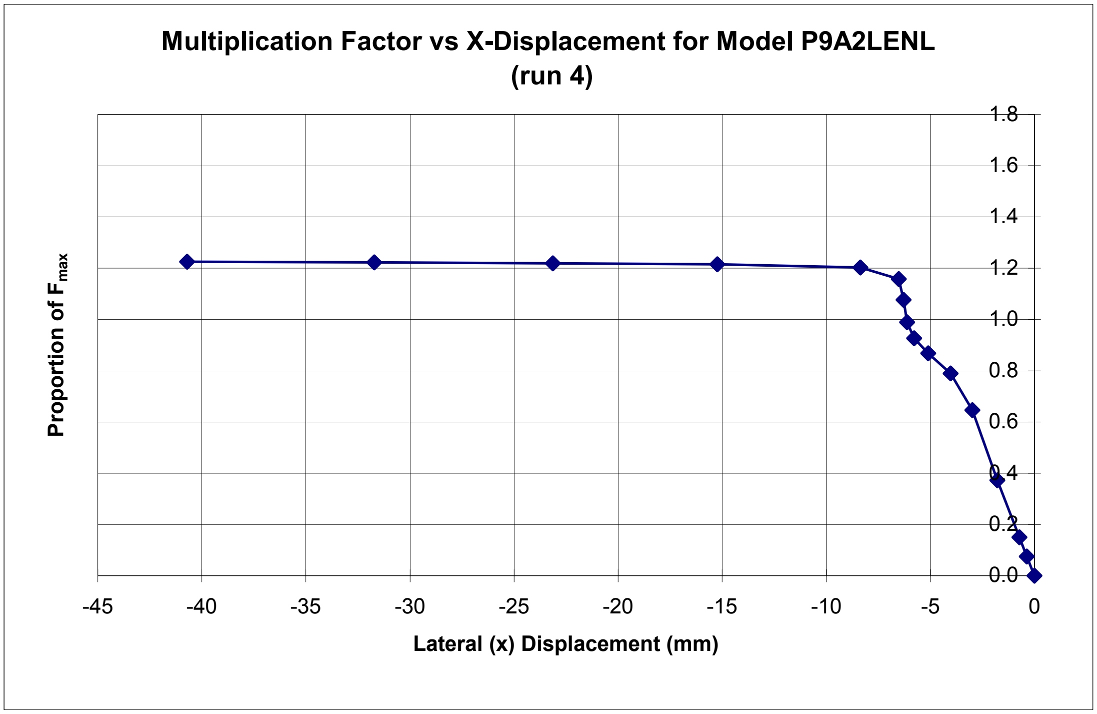


Figure 7.5. FE Model of 3x3 Bay Panel



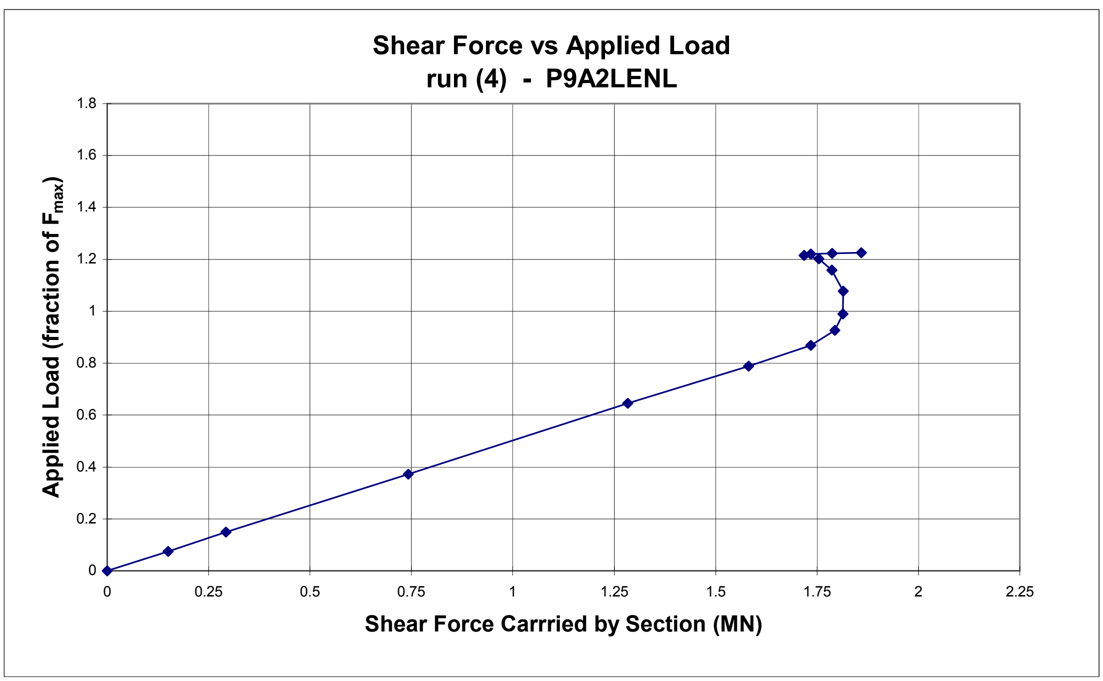


Figure 7.6 Response Curves for Sample Problem

As the load increases, the apparent shear force carried by the section starts to increase again. This is attributed to global plasticity occurring across the entire panel. The algorithm was developed to determine shear force based upon bending stresses developed from a bending response. In this case the onset of plasticity changes the response from one of bending to membrane, where a membrane stress component develops in the main frames. This additional strain component is captured in the algorithm and added to the shear force carried by the section. This results in an increase in the apparent shear force carried by the section. This response is not significant since the algorithm was developed to identify the initial point of buckling which is associated with a reduction in load carrying capacity of the main frame. Up to the point of initiation of instability, this methodology is considered valid and is used in the remainder of this study to identify the initiation of instability of main frames. More detail on the interpretation of the shear force difference curve is provided in Section 8.4.3.

8. PARAMETRIC STUDY

In Section 6, the structural parameters and influences that are felt to have the most significant effect on plastic stability were determined. In this section, these parameters and influences are prioritized in order of significance, and used to design panels for detailed FE analysis.

The methodology for the analysis procedure involved starting with the most significant parameter and performing a series of analyses using FEA to determine the effect that this parameter has on stability of the main frames. This was accomplished by varying the one parameter while attempting to keep all other parameters constant. Upon completion of the study for this parameter, the original prioritized sequence was reviewed and changes made, if required, based upon the knowledge gained.

8.1 Description of FE Models and Analysis Methodology

This section of the report provides details of the development of the FE model, the material properties used in the analyses, and the ADINA analysis methodology employed for the nonlinear analyses.

8.1.1 FE Model Description

The element type that is to be used for the detailed 3x3 bay FE models must be able to predict in-plane membrane and shear strains, and out-of-plane bending strains. The most versatile element available in the ADINA library for this is the shell element, with the 4 and 8 noded elements being the most promising choices.

In the Phase I [3] study, the 8-noded ADINA shell element was selected for the analysis. The selection was based upon previous experience with this element. The 3x3x2 integration order provides many integration points which allows the program to pick up the onset of plasticity, and the 8-noded shell element can predict a linear stress variation very well. In the Phase II analysis [4], the 4 node ANSYS shell element was used for the analysis. This element performed very well in matching the response of the physical panel test at Carleton University. In an independent check [8], it was found that the 4 node ADINA shell element also performed well in matching these results. Based upon the performance of the ADINA 4 node shell in matching actual experimental test results, this element type was selected for the FE work in the current phase of the study. The ADINA recommended 2x2x2 integration order was used. The element is also capable of multiple types of plastic material property definition.

The structural configurations that are used for the development of the 3x3 bay FE models are shown in Figures 8.1, 8.2, and 8.3 for the angle, tee, and flat bar main frame sections, respectively. The dimensions shown for the shell, deep webs, stringers, and main frame spacing are maintained throughout the parametric study. The main frame dimensions are altered for the purposes of changing the different stability parameters/influences in the parametric study. The main frame dimensions in these figures are typical of the designs.

8.1.2 Material Properties

The material used in this study was steel. While the yield strength was one of the influence parameters selected to be studied, a base set of material properties for mild steel was selected for the initial analyses. The constitutive (stress/strain) relationship used was bilinear elastic plastic with a yield strength of 355 MPa, a modulus of elasticity of 207 000 MPa, and a strain hardening modulus of 5 175 MPa.

The strain hardening modulus was taken from the previous ADINA nonlinear analyses of 3x3 bay models [3] and is considered to be accurate for the purposes of this study. The kinematic relationship used was large displacement, small strain. In the nonlinear FE analyses to be performed in this study, the maximum applied load levels did not significantly exceed the calculated ice load, F_{max}. Therefore large strains were not significant.

8.1.3 Analysis Methodology

The method of solution used for the ADINA nonlinear analyses is a procedure called the Load Displacement Control (LDC) method. When conventional applied force methods are used in the solution of large displacement nonlinear analyses, the solution often fails at regions of high nonlinearity (for example at the bifurcation point of buckled structures) due to non-positive definite (or very small) stiffness terms in the stiffness matrix. This results in an inability of the algorithm to converge to a unique solution.

The LDC method eliminates this problem by using displacements to control the solution. A load vector must be provided; however, the algorithm is started at the first load step with an initial nodal displacement in the desired direction instead of a force. The program automatically determines the load factor (a constant multiplied by the load vector) necessary to displace the structure by the initial displacement while maintaining equilibrium. The program then automatically determines the next incremental displacement and continues to the next load step. This procedure continues until either the maximum specified displacement is reached, the maximum number of time steps is reached, or the nonlinearity is extreme enough to prevent convergence within four iterations (maximum) of repeatedly reducing the incremental load. The advantage of using displacement control occurs when a large displacement results from a very small force (due to small stiffness). Without LDC, if a force is applied, the program has difficulty converging to a displacement.

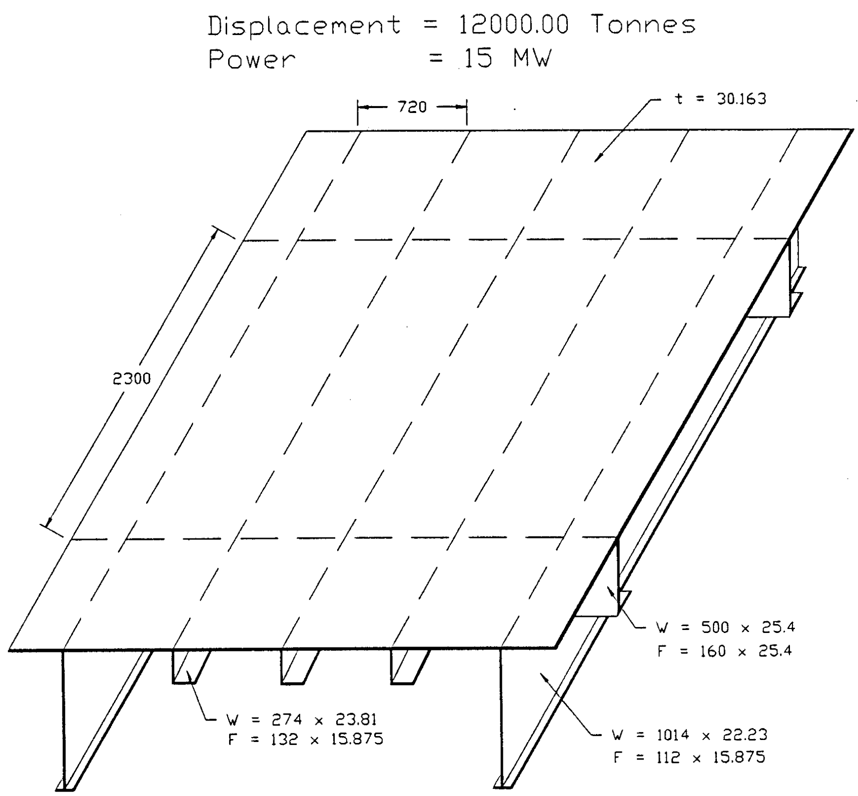


Figure 8.1 3x3 Bay Model (Showing the Centre Bay) with Angle Main Frames

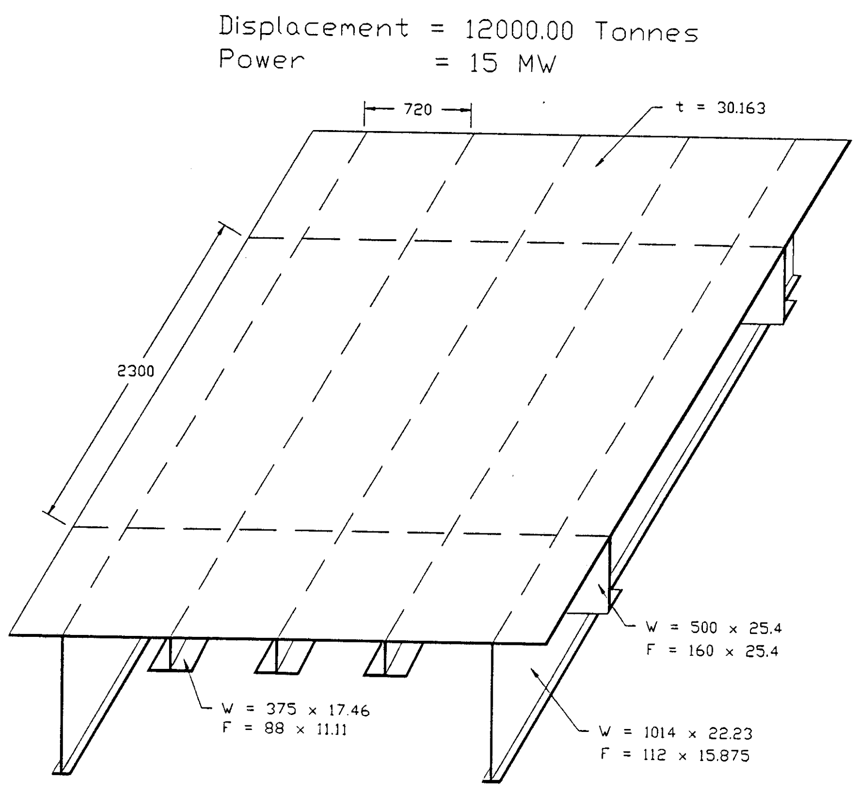


Figure 8.2 3x3 Bay Model (Showing the Centre Bay) with Tee Main Frames

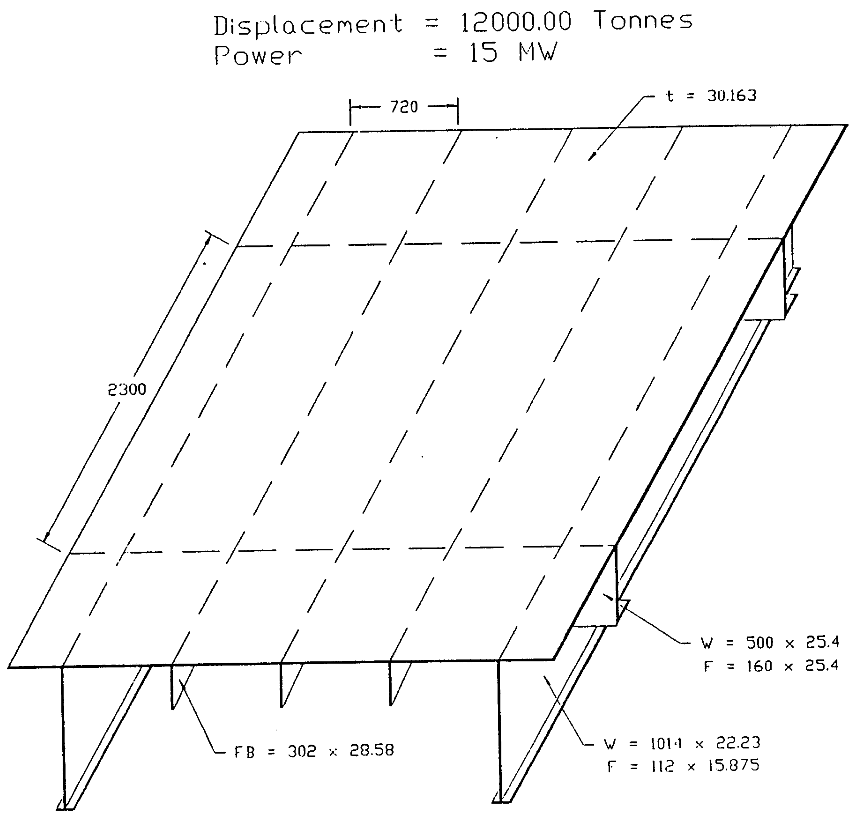


Figure 8.3 3x3 Bay Model (Showing the Centre Bay) with Flat Bar Main Frames

The LDC method is a nonlinear solution algorithm where all of the loads and initial conditions are incrementally increased at each load step. With the total load including both the biaxial in-plane loads and ice loads (applied normal to the plate), at a load factor of 0.5, the in-plane loads and the ice load, F_{max} , are both 50 percent of their value at F_{max}. While this may seem likely to generate potentially invalid responses (because the loads are not exactly applied as they should be), in actuality the source of error will be small for the region of the response of interest and will be exact at the ice load magnitude equal to F_{max} (i.e. at full ice load). Since the load application has no error at F_{max}, where all of the load components are totally applied, and the response at or near F_{max} is of most importance, the correct response will be predicted. The error expected in the response is minimal due to the domination of the correctly applied ice load.

8.2 Loads

The loads applied to the FE model are a combination of the applied ice load and a bi-axial in-plane stress on the boundaries of the panels.

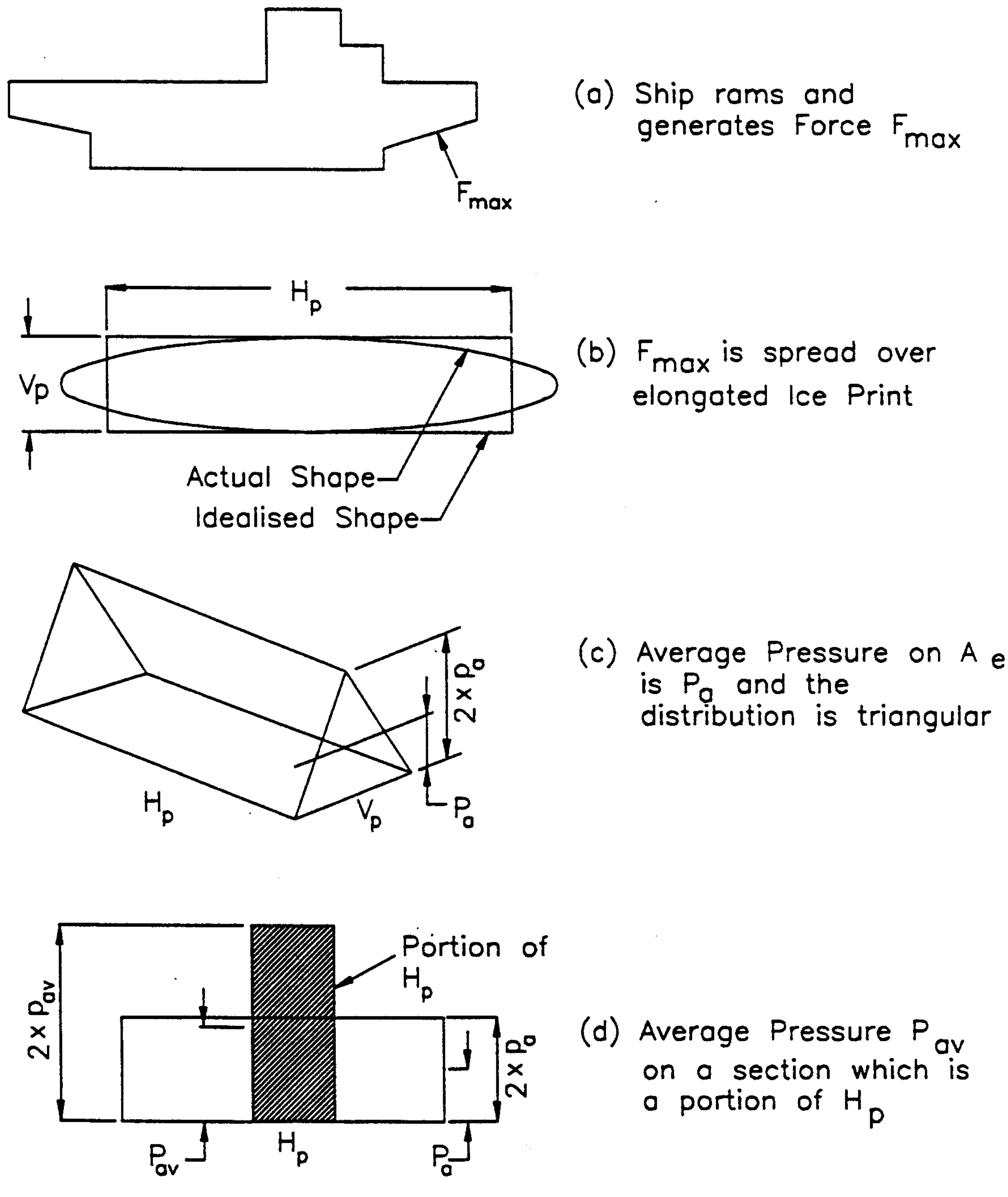
As found in the Phase I [3] study, in-plane compressive stresses result from the overall ship response. They are transmitted through the structure outside of the local panel area and greatly affect the post-yield buckling response. Therefore, including the effects of the global ship response is extremely important.

In the Phase I study, the in-plane compressive stresses from the global ship response were included through prescribed displacements from an independent analysis of the ship using MAESTRO [27]. However, it was determined that these in-plane stresses are fairly constant, and they may be determined through simpler methods. They can also be applied to the model as in-plane forces. That is the procedure used in the current study.

The magnitude of the in-plane stresses at a load level equal to F_{max} in the Phase I study was determined to be approximately 69 MPa compression in both the vertical and longitudinal directions. These values were considered to be accurate since they were determined from an overall analysis of a ship structure using a combination of still water bending moment, hydrostatic pressure load, and the ice loads.

The iceload is characterized by a force distributed over an area designated in the Equivalent Standards as the iceprint with dimensions L_{DL} x V_p (length x height). The shape of the idealized iceprint used in this study is as provided in ASPPR. Figure 8.4 (reproduced from Figure 4.9 of the ASPPR [1]) shows the actual and idealized shape of the iceprint. This idealized model assumes that a triangular pressure distribution is acting on this ice print.

As shown in Figure 8.4, the peak of this triangle has a magnitude of 2 x P_a (where P_a is the average pressure). The pressure also has variation along the length of the ice print. This results in a higher pressure on a smaller section of the ice print. This higher pressure is defined as P_{av} and is shown in Figure 8.4. The value of P_{av} is calculated from Section 15 of



Note: H_p is equivalent to L_{DL} as defined in the Equivalent Standards.

Figure 8.4 Ice Load Model (from Figure 4.9 of the ASPPR) the Equivalent Standards regulations [2]. This value is a function of DPT which is defined in Section 18 of the Equivalent Standards as:

$$DPT = \frac{S}{L_{DL}}$$

where S = frame spacing
L_{DL} = horizontal length of the ice print

It should be noted that P_{av}, is for a load acting at the bow area of a CAC1 ship. To achieve the proper values for different CACs and areas of the ship, these values are multiplied by appropriate class and area factors.

The size of the iceprint and ice load is based upon the force developed due to ship ice interaction while the vessel is engaged in icebreaking activity. To derive expressions for the ice-load parameters, several models and full scale tests were conducted during the development of ASPPR. The effects of several parameters (i.e. speed, displacement, bow geometry, and power) on this value were examined. It was found that displacement and power of the ship had a direct influence on the ice forces. The knowledge gained was subsequently used in the development of the iceload model for the Equivalent Standards. The resulting formula for the iceprint to be used in framing design is as follows:

$$L_{DL} = 2.80\sqrt{\Delta^{0.7} + \Delta^{0.48} \cdot P^{0.33}}$$

where $V_p = L_{DL}/8$
Δ = the displacement in thousands of tonnes
P = the shaft power in megawatts

The value of F_{max} is the total applied ice load on the ship.

For structural models with the dimensions as shown in Figures 8.1 to 8.3, the magnitude of the ice load for the midbody region is as shown in Figure 8.5. Since the overall panel geometry does not change in this study, this ice load is used for all analyses. It should be noted that for load values not equal to F_{max}, the same proportional distribution of pressures is maintained. The peak pressure load is applied over the most central main frame in the center bay.

8.3 Boundary Conditions

The boundary conditions to be used on all of the 3x3 bay FE models must be representative of the actual boundary conditions. In the Phase I study, the boundary conditions used were prescribed displacements that were determined from a MAESTRO analysis of the complete

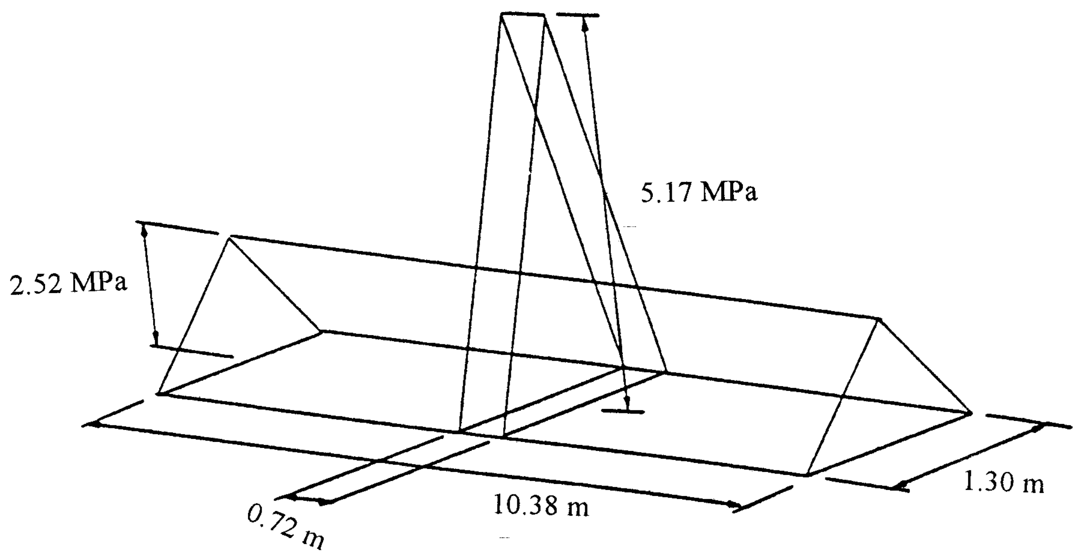


Figure 8.5 Ice Load for Midbody Region Calculated using the Equivalent Standards and ASPPR.

ship. The benefit of modelling the boundary conditions using prescribed displacements is that the response of the overall ship is accurately included in the local 3x3 bay panel response. The disadvantage of using prescribed displacements is that a model of the complete ship has to be generated and analyzed to get the proper boundary conditions. It was realized that this is not practical for a parametric study where the potential exists to analyze many different ship configurations. It was also concluded from the Phase I study that the prescribed displacements are not necessary if the overall ship response is included through in-plane stresses. Therefore, in the current study, a set of boundary conditions is defined using a combination of loads and constrained degrees of freedom that accurately reflect the global ship response.

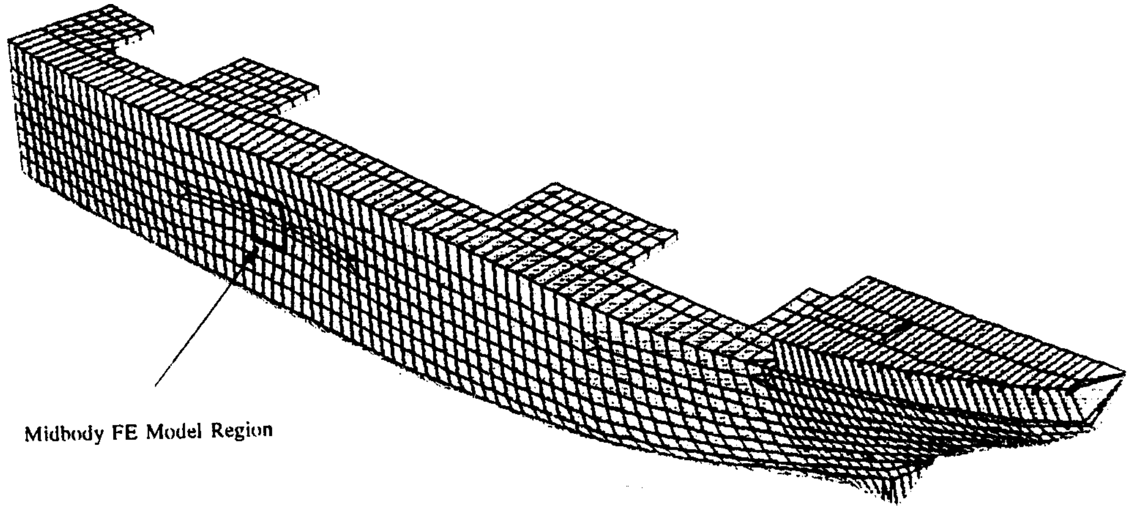
[Note: While the panel boundary conditions are made as realistic as possible, the Phase I study found that results for members in the centre bay were not very sensitive to the panel boundary conditions, other than in-plane stresses. The eight surrounding bays are included (with a relatively coarse mesh) to provide realistic boundary conditions for the centre bay.]

The rationale behind selection of the boundary conditions used for all 3x3 bay analyses in this study is detailed in Figure 8.6. Figure 8.6 (a) shows a sketch of the expected overall ship response to an ice load along with an outline of the local 3x3 bay region. The overall ship model is from the MAESTRO analysis of the M.V. Arctic taken from the Phase I study, where centre line symmetry was applied to the model. This means that the reaction force (not shown) is opposite to the ice force and generates no full global bending. Thus, the induced in-plane stresses indicated are a minimum and could be designated as local/global.

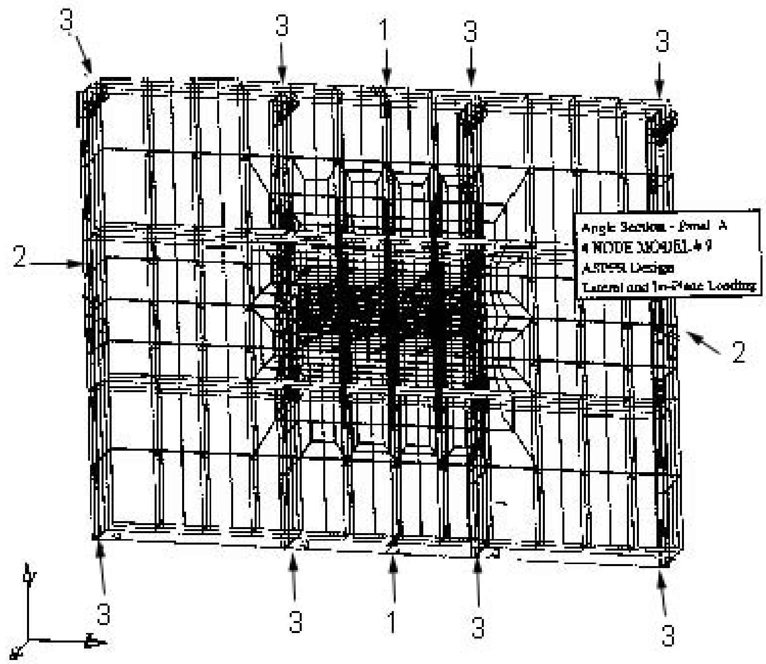
It can be seen in Figure 8.6 that the upper and lower boundaries of the 3x3 bay panel region displace laterally at approximately the same magnitude. This is due to the fact that the ice load is applied (inward) not only to the local 3x3 bay region but to the entire ice print area along the side of the ship. One component of the global response in the area of the 3x3 bay model is a rigid body deflection of the panel in the lateral direction as the hull displaces inwards due to the application of the massive ice load.

It is also shown that the upper and lower boundaries rotate about a longitudinal axis. While this degree of rotation depends upon the proximity of decks above and below the 3x3 bay region, the highest stresses result when the boundaries are free to rotate. In this case, higher compressive stresses are generated on the hull plating (which acts as the outer flange of the deep webs) and higher tensile stresses are generated on the inner flanges of the deep webs (or inner skin for double hull vessels).

The forward and aft vertical edges displace concavely inward, restrained only by the structure above and below the 3x3 bay panel. There is only a small contribution from the longitudinal stringers because the structure forward and aft of the panel is displaced laterally from the ice load. Consequently, it does not put any significant lateral constraint on 3x3 bay stiffened panel.



(a) Overall Structure Response to Ice Load



Designation	Degree of Freedom Constrained
1	δ_x
2	δ_y
3	δ_z
4	σ_x
5	σ_y
6	σ_z

(b) 3x3 Bay FE Model Boundary Conditions

Figure 8.6 Selection of Boundary Conditions for Study

Based on the response of the overall ship, as shown in Figure 8.6 (a), the boundary conditions used on all 3x3 bay models in this study are shown in Figure 8.6 (b). The numbers shown on the plot indicate the global degrees of freedom which are constrained. From this it can be see that the upper and lower panel boundaries are fixed against lateral (z) displacement. Lateral constraint is required for numerical purposes and it also ensures the top and bottom boundaries displace at the same magnitudes. Since the overall model analysis showed rotation at the top and bottom boundaries, no rotational constraints are imposed at these locations.

As discussed above, the forward and aft vertical boundaries are considered not to be constrained inwardly by the adjacent structure and displace freely. Therefore, no lateral (z) displacement boundary conditions are applied at the for/aft boundaries.

Additional boundary conditions are imposed to satisfy numerical conditions and to permit application of the bi-axial in-plane pressure loads (for those runs where in-plane loads are required). Nodes at the center of the top and bottom edges are constrained in the longitudinal (x) direction only. This provides suppression of rigid body motion while permitting the structure to compress in the longitudinal direction under influence of the in-plane pressure loads. Similarly, nodes at the center of the forward and aft edges are constrained in the upward (y) direction such that free body motion is suppressed and the structure is still permitted to compress under the vertically applied in-plane pressure loads.

8.4 Analysis

This section provides the details of the nonlinear FE analyses carried out in this study. In section 8.4.1, a test matrix is generated as Table 8.1 that shows the FE models that were developed based upon different stability parameters to be studied. The results of the analyses are also provided in this table in terms of the determined buckling load of each panel. Section 8.4.2 provides details of a linear analysis of the panel, and Section 8.4.3 provides details of a representative nonlinear analysis. It was found that the same general response occurred in all nonlinear panel analyses, therefore a section was provided to present the typical response of a representative panel. In addition to this, Microsoft Excel files (that present load-displacement and shear force difference curves) are included in the attached CD-ROM for most of the analyses in the test matrix (see Appendix E).

8.4.1 Test Matrix

The test matrix is shown in Table 8.1. It has been compiled using the results of the nonlinear analyses performed in the parametric study. Three different panel main frame types are shown (tees, angles and flat bars) with varying design parameters under the different stability influences as identified by each “Influence Item”. For the tee and flat bar sections, there are two stability criteria which are considered in the study. For the angles there is one stability criterion. These criteria are identified by the name of the parameter itself and the report section number in the Equivalent Standards. For example, LU/WF is the span ratio stability criterion for tee main frames as found in Section 24.1 of the Equivalent Standards [2]. A copy of the stability equations is provided in Appendix C, where a portion of Section 24 of the Equivalent Standards has been reproduced. This section of the Standard presents the tripping rules for the frame instability along with a plot of tripping and buckling failure modes copied from ASPPR.

Five rows and three columns are shown with each stability criterion for each Influence Item. The first row shows the value of the stability parameter with the value decreasing from left to right. This should correspond to an increase in stability of the panels from left to right as shown in row 2 by the stability criterion. A value of 1.0 for the stability criterion indicates that the panel is designed (with the Equivalent Standard specified limit value) to fail from an instability at F_{max}. A stability criterion value higher than 1.0 means the panel should be more stable (based on the Equivalent Standards design); a value less than 1.0 means the panel should be unstable at a load less than F_{max}.

The third row in each Influence Item shows the corresponding strength criterion for each panel. As with the stability criterion, a value of 1.0 means the panel is designed not to fail in bending or shear below F_{max}. It was difficult to achieve a value of 1.0 for all of the panels designs since the only change made in the panels was the main frames dimensions. Changing these dimensions not only modified the strength criterion, but also the stability criteria. It was decided to select a stability criterion beneficial to the parametric study and to determine a strength criterion as close to unity as possible.

The fourth row in each Influence Item shows the proportion of the F_{max} load at which the panel buckles as predicted by the FE analysis. The determination of this load level is based upon the shear force difference method described in Section 7.2.2 as applied to the results of each of the nonlinear analyses in the table.

The last row in each Influence Item shows a run identifier. This is the designated number for a particular nonlinear FE analysis. These run identifiers are used to identify the analyses in Table 8.1 throughout the presentation of results in this section.

Table 8.2 shows the design tables for several selected sample runs. The balance of the designs is included in Appendix C.

8.4.2 Linear Analysis Results

Prior to performing the nonlinear analyses of the 3x3 bay model for the parametric study, a linear analysis was performed to check the element performance and the element density in regions of interest. An angle main frame model was chosen for this analysis with the same loads and boundary conditions as used for the nonlinear analyses. Several iterations of model refinement were performed with the final element density to be used for the nonlinear

Table 8.1 Matrix Of Analyses To Create Database For Quantitative Assessment of the Equivalent Standards

Influence Item	Description	Stability Parameter	Tees I(i) LU/WF (24.1)*			Tees II(i) HW/TW (24.1)*	Angles III(i) LU/WF (24.2)*			Flat Bars** IV(i) LU/TW (24.3)*			Flat Bars** V(i) HW/TW (24.3)*		
1	No In-plane Load	Value of Stab. Parameter	511	37.7	26.1		34.8	26.1	17.4	99.9	83.0	80.5	17.9	13.0	8.9
		Stability criteria	0.50	0.70	1.06		0.50	0.61	0.77	0.49	0.84	1.00	0.53	0.69	1.01
		Strength criteria	1.14	1.19	1.28		1.21	1.28	1.21	1.03	1.03	1.07	1.15	1.04	1.02
		Buckling Load Level	1.05	1.05	1.08		1.06	1.07	1.23	1.17	1.09	1.07	1.06	1.30	1.11
		Run Identifier	(8)	(7)	(6)		(3)	(2)	(1)	(18)	(19)	(13)	(15)	(14)	(25)
2	Biaxial In-plane Load = 69 MPa	Value of Stab. Parameter			26.1		34.8	26.1	17.4	99.9		80.5			8.9
		Stability criteria			1.06		0.50	0.61	0.77	0.49		0.84			1.01
		Strength criteria			1.28		1.21	1.28	1.21	1.03		1.03			1.02
		Buckling Load Level			0.98		0.96	1.00	1.08			0.99			0.98
		Run Identifier			(9)		(49)	(50)	(4)	(22)		(16)			(26)
3	Baxial In-plane Load = 34 MPa	Value of Stab. Parameter			26.1				17.4			80.5			8.9
		Stability criteria			1.06				0.77			1.00			1.01
		Strength criteria			1.28				1.21			1.07			1.02
		Buckling Load Level			1.02				1.15			1.03			1.02
		Run Identifier			(10)				(5)			(17)			(27)
4	Biaxial In-plane Load = 103 MPa	Value of Stab. Parameter	511	37.7	26.1				17.4			80.5			8.9
		Stability criteria	0.50	0.70	1.06				0.77			1.00			1.01
		Strength criteria	1.14	1.19	1.28				1.21			1.07			1.02
		Buckling Load Level	0.92	0.90	0.91				0.96			0.92			0.91
		Run Identifier	(30)	(29)	(31)				(32)			(33)			(28)
5	In-plane Load = 69 MPa (Vertical Direction Only)	Value of Stab. Parameter							17.4						
		Stability criteria							0.77						
		Strength Criteria							1.21						
		Buckling Load Level							1.17						
		Run Identifier							(21)						
6	Modify Boundary Conditions	Value of Stab. Parameter			26.1										
		Stability criteria			1.06										
		Strength criteria			1.28										
		Buckling Load Level													
		Run Identifier			(20)(23)(24)										

**Note that for flat bars "Buckling Load Level" does not indicate buckling. It indicates the point at which the flat bar does not carry additional incremental load.

Table 8.1 (cont'd)

Influence Item	Description	Stability Parameter	Tees I(i) LU/WF (24.1)*	Tees II(i) HW/TW (24.1)*	Angles III(i) LU/WF (24.2)*	Flat Bars IV(i) LU/TW (24.3)*	Flat Bars V(i) HW/TW (24.3)*
7	Tilt Angle = 30° with 69 MPA	Value of Stab. Parameter	26.1		17.4		8.9
		Stability criteria	100		0.67		100
		Strength criteria	119		105		101
		Buckling Load Level	0.92		0.92		0.94
		Run Identifier	(40)		(34)		(42)
8	Tilt Angle = 15° with 69 MPa	Value of Stab. Parameter	26.1		17.4		8.9
		Stability criteria	106		100		100
		Strength criteria	129		121		102
		Buckling Load Level	0.94		0.98		0.94
		Run Identifier	(41)		(35)		(43)
9	Distortion = 100% of CCG Acceptable with 69 MPa in-plane load	Value of Stab. Parameter			17.4		
		Stability criteria			0.77		
		Strength criteria			121		
		Buckling Load Level			105		
		Run Identifier			(36)		
10	Distortion = 200% of CCG Acceptable with 69 MPa in-plane load	Value of Stab. Parameter			17.4		
		Stability criteria			0.77		
		Strength criteria			121		
		Buckling Load Level			110		
		Run Identifier			(37)		
11	Residual Stress = σ yield with 69 MPa in-plane load	Value of Stab. Parameter			17.4		
		Stability criteria			0.77		
		Strength criteria			121		
		Buckling Load Level			109		
		Run Identifier			(38)		
12	σ yield = 500 MPa with no in-plane load	Value of Stab. Parameter			34.8 26.1 17.4		
		Stability criteria			0.50 0.61 0.77		
		Strength criteria			171 181 171		
		Buckling Load Level			142 145 132		
		Run Identifier			(45) (44) (48)		
13	σ yield = 500 MPa with 69 MPa in-plane load	Value of Stab. Parameter			34.8 26.1 17.4		
		Stability criteria			0.50 0.61 0.77		
		Strength criteria			171 181 171		
		Buckling Load Level			130 130 130		
		Run Identifier			(46) (47) (39)		

*These are the report section numbers in the Equivalent Standards[2]

Table 8.2 Examples Of Panel Design Tables

Angle, Midbody, CAC3, Disp = 9 000 t

DISPLACEMENT (KTonnes)	9.00	9.00	9.00	9.00	9.00	9.00	9.00	9.00
POWER (MW)	18.00	18.00	18.00	18.00	18.00	18.00	18.00	18.00
MATERIAL YIELD STRENGTH (MPa)	355.00	355.00	355.00	355.00	355.00	355.00	355.00	355.00
CAC (1,2,3, or 4)	3	3	3	3	3	3	3	3
HULL AREA (Bow or Midbody)	Midbody	Midbody	Midbody	Midbody	Midbody	Midbody	Midbody	Midbody
Arctic Class Factor	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6
Area Factor	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5
Fmax	48.973	48.973	48.973	48.973	48.973	48.973	48.973	48.973
Vp	1.166	1.166	1.166	1.166	1.166	1.166	1.166	1.166
Hp	9.331	9.331	9.331	9.331	9.331	9.331	9.331	9.331
FRAME SPACING	400	500	600	650	700	800	900	1000
FRAME SPAN	2600	2600	2600	2600	2600	2600	2600	2600
SHELL PLATE DESIGN:								
Corrosion Allowance	0	0	0	0	0	0	0	0
U	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5
Omega (Degrees)	90	90	90	90	90	90	90	90
Frame Orientation Factor	1	1	1	1	1	1	1	1
Plate Design Pressur - Dpp (MPa)	4.689	3.751	3.126	2.885	2.679	2.344	2.084	1.875
MInimum Dpp (MPa)	5.750	4.600	3.833	3.538	3.286	2.875	2.556	2.300
Dpp used for Plate Thickness (MPa)	5.750	4.600	3.833	3.538	3.286	2.875	2.556	2.300
Minimum Shell Plate Thickness (mm)	22.04	24.65	27.00	28.10	29.16	31.17	33.07	34.85
Shell Plate Thickness (mm)	22.23	25.4	28.58	28.58	30.163	31.75	33.34	34.93
TRANSVERSE FRAME DESIGN:								
Type	Angle	Angle	Angle	Angle	Angle	Angle	Angle	Angle
Dimensions:								
Web Depth (mm)	300	300	300	300	312	320	350	350
Web Thickness (mm)	20.00	20.00	20.00	20.00	20.00	22.00	22.00	24.00
Flange Width (mm)	117	125	132	136	138	140	142	150
Flange Thickness (mm)	20	20	20	20	20	20	20	20
Phi (degrees)	90	90	90	90	90	90	90	90
REQUIRED VALUES:								
DPT	0.043	0.054	0.064	0.070	0.075	0.086	0.096	0.107
PAV	9.549	9.171	8.847	8.701	8.565	8.319	8.102	7.909
Span LB (meters)	2.600	2.600	2.600	2.600	2.600	2.600	2.600	2.600
Spacing Between Frames S (meters)	0.4	0.5	0.6	0.65	0.7	0.8	0.9	1
Vp / Span	0.449	0.449	0.449	0.449	0.449	0.449	0.449	0.449
Factor A (Sch 1 Table 3)	0.806	0.806	0.806	0.806	0.806	0.806	0.806	0.806
Value H	15000	15000	15000	15000	15000	15000	15000	15000
Value B	2.976	2.976	2.976	2.976	2.976	2.976	2.976	2.976
Req. Trans. Frame Shear Area (cm2)	37.77	45.35	52.49	55.93	59.29	65.82	72.11	78.22
Req. Trans. Frame Plas. Modu. (cm3)	863.80	1036.97	1200.34	1278.99	1355.89	1505.09	1649.04	1788.59
MINIMUM VALUES:								
Hp min	6.00	6.00	6.00	6.00	6.00	6.00	6.00	6.00
Vp min	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75
Vp min / Span	0.288	0.288	0.288	0.288	0.288	0.288	0.288	0.288
Factor A (Sch 1 Table 3)	0.873	0.873	0.873	0.873	0.873	0.873	0.873	0.873
DPT	0.067	0.083	0.100	0.108	0.117	0.133	0.150	0.167
PAV	8.781	8.372	8.036	7.890	7.755	7.515	7.307	7.124
Factor C	1	1	1	1	1	1	1	1
Factor B	2.034	2.034	2.034	2.034	2.034	2.034	2.034	2.034
Min. Trans. Frame Shear Area (cm2)	40.34	48.07	55.37	58.89	62.34	69.05	75.53	81.82
Min. Trans. Frame Plas. Modu. (cm3)	904.71	1078.18	1241.91	1320.88	1398.20	1548.56	1693.97	1835.05
ACTUAL VALUES:								
Factor M	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Actual Shear Area (cm2)	60.00	60.00	60.00	60.00	62.40	70.40	77.00	84.00
Actual Plastic Modulus (cm3)	1718.10	1782.95	1841.87	1867.81	1997.89	2206.61	2545.60	2749.10

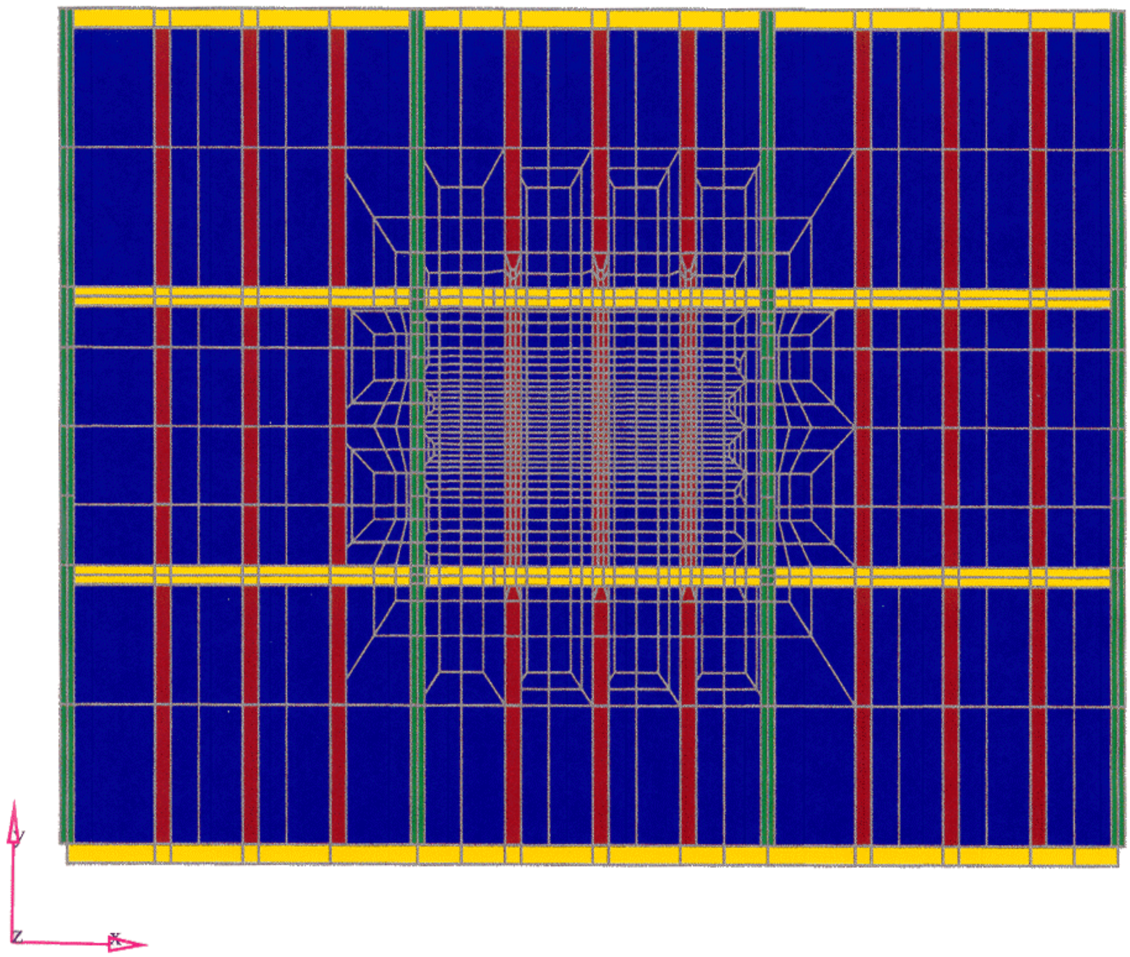


Figure 8.7 Final Element Density

analysis shown in Figure 8.7. It should be noted that the linear analysis can only be used as a preliminary indication of mesh refinement. Yielding and large displacements can alter the mesh selection significantly. However, much can still be learned from the linear results and it is highly recommended [28] that the analysis be performed prior to any nonlinear analysis.

8.4.3 Representative Nonlinear Analysis Results

The nonlinear analysis performed in this section serves to check the solution procedure and also to demonstrate the typical response of all of the nonlinear analyses in the parametric study. It was found that while the magnitude of buckling load changed from one analysis to the next, the overall response of each panel in Table 8.1 was basically the same. This response is explained in detail below. (Note: Similar response curves for most of the analyses are supplied in the files on the attached CD-ROM in Appendix E.)

The FE model primarily used to explain the representative nonlinear response of the 3x3 bay panels is from Run (4) in Table 8.1. This is an angle main frame model with an the Equivalent Standards stability criterion of 0.77 and with the Equivalent Standards strength criterion of 1.21. The loads and boundary conditions applied to the model are as described in Sections 8.2 and 8.3, respectively.

The nonlinear analysis was performed up to a load level of 1.23 F_{max} which required 15 load steps to complete. The region of most interest in this model is the center bay central main frame. This main frame carries the highest peak component of the lateral ice load. The shear force difference curve for this main frame is shown in Figure 8.8(a). (Note: All shear force difference curves are generated based upon the shear forces in the web section only of the main frames.) The global x and z displacements of a node at the intersection of the flange and web at the midspan of the center main frame are shown in the load displacement curves of Figures 8.8(b) and (c), respectively.

From Figure 8.8(a), the shear force difference can be seen to increase with applied load in a linear manner to an applied load of approximately .9 F_{max}. After 1.08 F_{max}, the shear force difference decreases in the frame and the frame sheds approximately 10 percent of the load that it was carrying even while the applied load increases by an additional 30 percent. The shedding of this load is associated with the initiation of buckling in the frame which occurs at 1.08 F_{max}. However, as the load is increased further, the shear force difference appears to once again increase. Typically for other runs, as the load is further increased, the calculated shear force difference increases rapidly with applied load. Interpretation of this phenomenon was confusing as it points to the fact that the frame is still capable of carrying further bending load even though we have already concluded that it has experienced buckling.

The explanation for this is that the calculation of the shear force difference carried by the frame is made under a set of assumptions. As shown in Figure 8.9(a), the shear force difference calculations are determined based upon a frame that is experiencing bending due to an applied lateral load. Under this scenario, the shear stresses in the frame are a direct

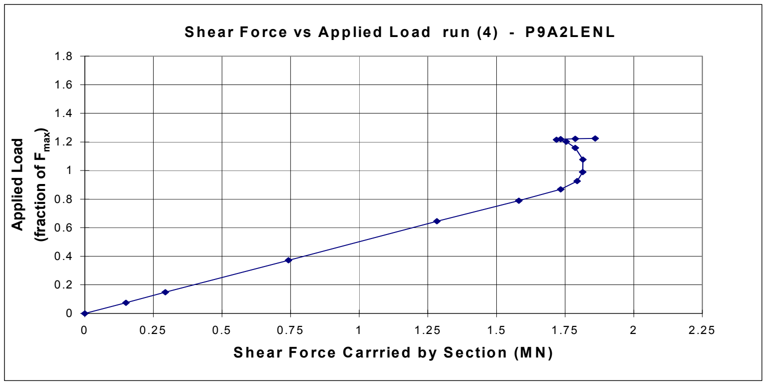


Figure 8.8(a) Shear Force Curve

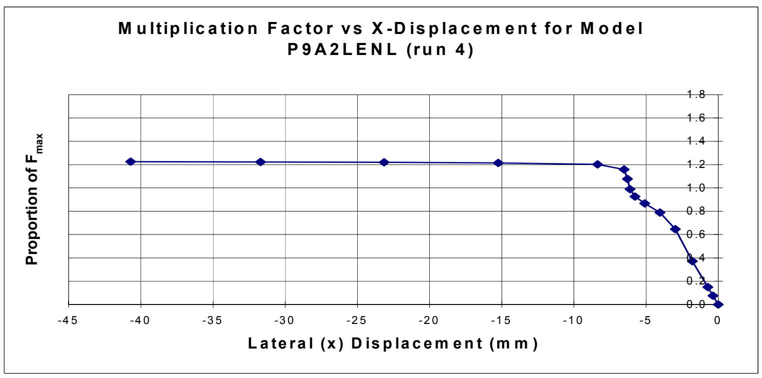


Figure 8.8(b) X-Displacement vs Load Curve

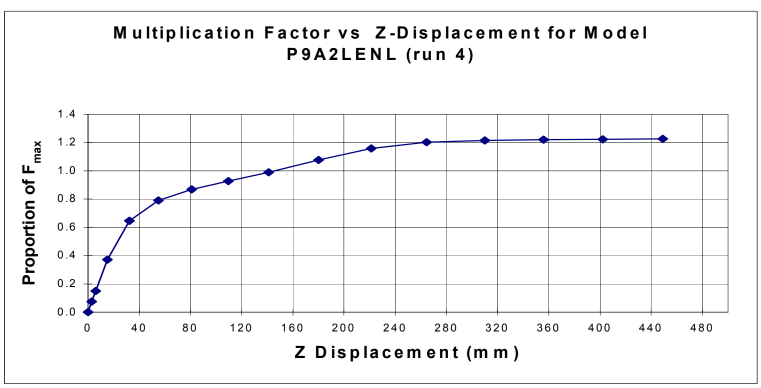
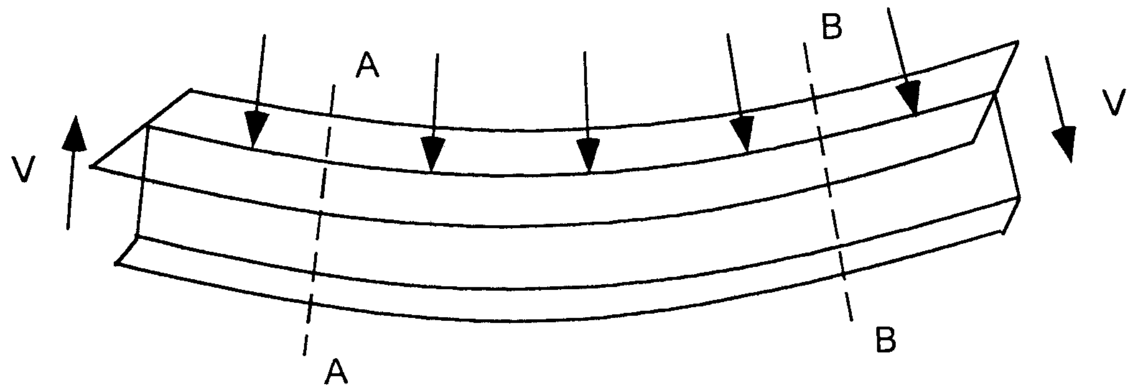


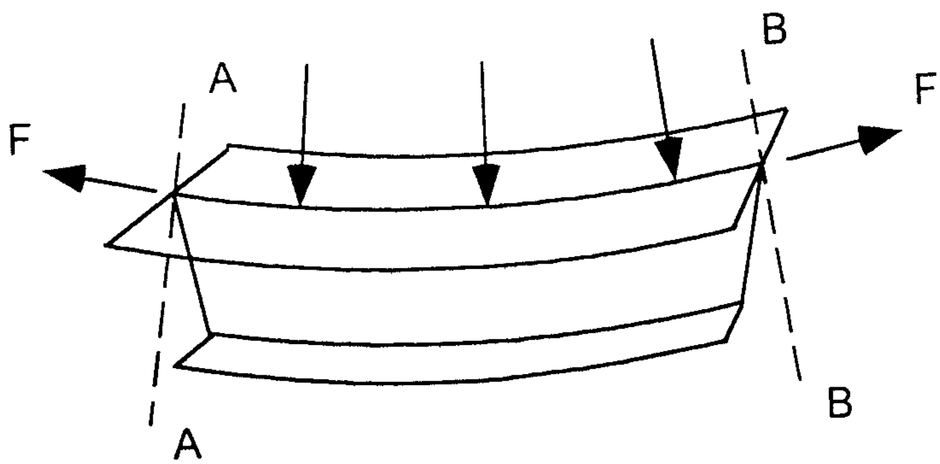
Figure 8.8(c) Z-Displacement vs Load Curve

Figure 8.8 Load Displacement Curves for Nonlinear Representative Analysis



- shear development is linear
- based upon bending
- plane sections remains plane

(a) Shear Force Development during Linear Stage



- membrane action provides in-plane forces
- plane sections do <u>not</u> remain plane
- generates another shear component

(b) Shear Force Development After Plasticity and Large Displacement

Figure 8.9 Development of Shear Forces in the Main Frame
during Nonlinear Analysis

result of bending stresses set up in the frame to carry the applied load. The shear stresses are proportional to the applied load and if the shear stresses are integrated over the area at two plane cross sections (A-A and B-B, which remain effectively plane in the elastic range) of the frame, then the difference in the total shear force carried at the two sections is equal to the load applied between the two sections. This is a well established procedure to experimentally determine applied load that can also be used analytically in this study.

However, the non-linear response in this study associated with the large applied loads causes a change in the load and stress distribution from the initially assumed configuration. Figure 8.9(b) illustrates this response where under increased load, large displacements and membrane action occur in the panel. An in-plane load is generated which creates a new shear stress component in the main frame. This shear stress component increases as the membrane action further develops.

To determine the extent of membrane action occurring in the FE model, a plot of lateral (z) out-of-plane displacement at two locations (one location on the skin (node 115) and one at the outer corner fibres of the center bay (node 80)) is shown in Figure 8.10(a). As the load is increased it can be seen that the difference between the two curves increases. This is an indication that the center node is displacing significantly more than the corner node. Figure 8.10(b) shows a plot of the difference in lateral displacement between the two points shown in Figure 8.10(a). The curve can be seen to show four distinct regions:

Points 1-4 A region showing linear bending stiffness of the frames. In this region, all of the applied load is carried through bending stiffness in the frames.

Points 5-7 Yielding occurs in the outer fibres of the main frames in the center bay causing a decrease in bending stiffness.

Points 8-11 An increase in stiffness occurs as the applied load is carried through membrane stresses in the plating

Points 12-15 An overall loss of stiffness due to buckling of the deep web frame.

[Note: In Figure 8.10, the results from Run 50 are presented rather than from Run 4, as shown in Figure 8.8. This is a result of the extreme data storage requirements of the analyses in this project. After Run 4 was executed and processed, the data files were erased to generate sufficient storage space for new analyses. Therefore, to generate the curves in Figure 8.10 the data that was available had to be utilized. This arbitrary use of data does not compromise the validity of the results. All nonlinear analyses gave very similar results, therefore, any one analysis could be used for presentation of the results.]

The integration of shear stress (as described above) to determine the applied load is only a valid assumption while the load is carried through bending stresses in the frames. This

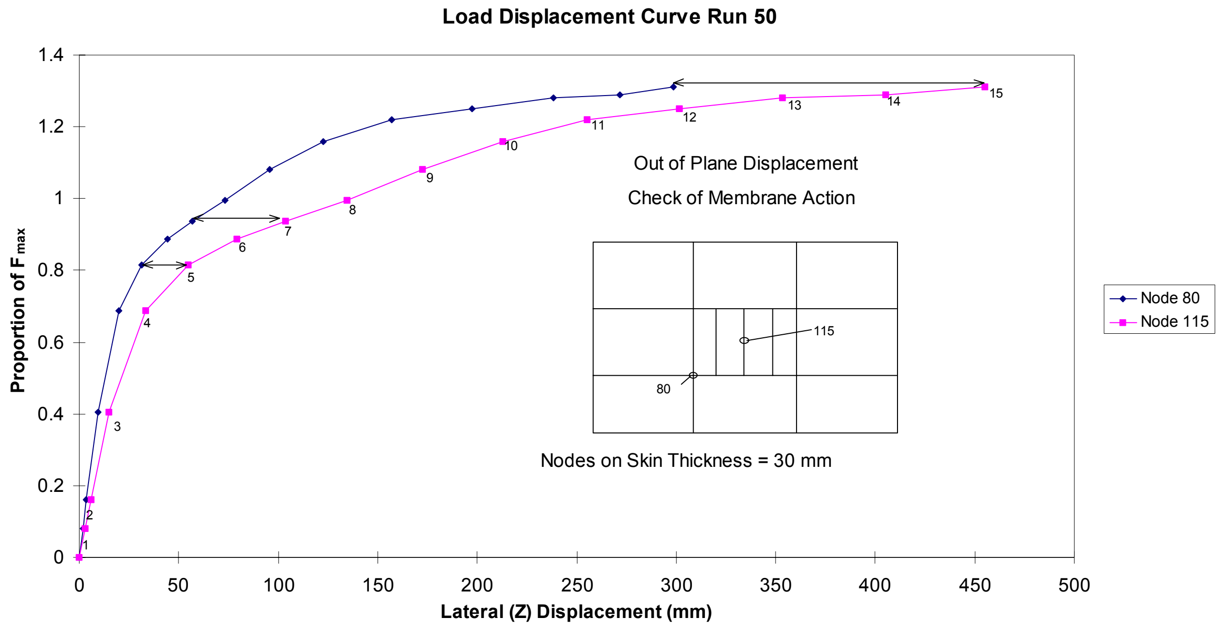


Figure 8.10(a)Lateral Displacement (Z) at the Center and Edge of the Center Bay Panel

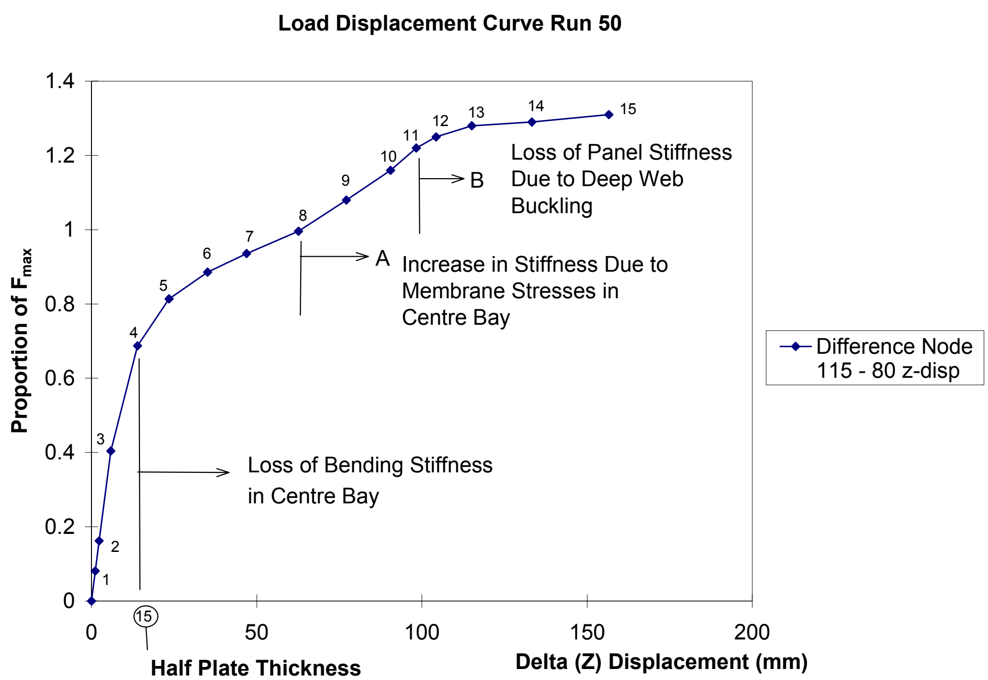


Figure 8.10(b) Difference between Lateral Displacement at the Center and Edge of the Center Bay Panel

means that up to about point 8 in the curve shown in Figure 8.10(b), the calculation is expected to be valid. After this point, new shear stresses develop in the frames which are a result of a load being carried through a membrane action both in the plate and also partially in the frames. The shear stress becomes an indication of this load rather than the level of bending stress.

Therefore, in Figure 8.8(a) the increase in shear force observed subsequent to load step 12 is believed to be a result of a change in the load condition. The frame is still believed to have started to buckle at load step 8, where it first sheds some applied load.

One exception to the typical shear force difference curve response was found during this study. This occurred during the analyses of the flat bar main frame models. A typical shear force difference curve for the flat bar main frame FE models is shown in Figure 8.11. While the general shape of this curve is similar to those for the angle and tee section main frames, one significant difference is evident. At the point where angle and tee section shear force difference curves typically shows a loss of load carrying capability associated with buckling (load step 8), the flat bar shear force difference curve is vertical. This shows that while the section does not carry any additional load, it does not shed any load. Therefore, the flat bar main frame sections do not buckle. This finding is consistent with previous findings regarding the response of flat bar main frame panels.

The response of the whole panel (from Run 4) can be explained in more detail from the z displacement load curve of Figure 8.8(c). This curve is reproduced in Figure 8.12 with an explanation as to the changes in stiffness that occur to produce the predicted response. Up to a load level of 0.39 F_{max}, the response is linear. Above this point, the out of plane deflection of the panel starts to increase nonlinearly. This is due to the development of plasticity in the panel and a change in stiffness due to the large displacements. Between 0.39 and 0.8 F_{max}, the displacements are becoming significant enough to start to transform the panel response from bending to membrane. From 0.8 F_{max} to 1.2 F_{max}, the panel response is now largely membrane. At a load level of 1.2 F_{max}, a hinge forms in two of the deep webs and very large lateral displacements occur.

A sequence of the panel displacement during this applied loading is shown in Figure 8.13. This figure shows displacement contours of the panel at different load steps from Run 4. From load step 5 (.79 F_{max}) to load step 10 (1.15 F_{max}), the out-of-plane displacements are symmetric. At load levels above this, the displacement becomes asymmetric as shown by the displacement contours at load steps 11,12 and 15. This asymmetry is the result of the plastic hinge forming in the deep webs on one side of the panel.

The explanation for the asymmetric response occurring in the FEA of the panel is that the model is not totally symmetric. While the loads and boundary conditions are symmetric, and the geometry of most panels in the parametric study is symmetric (except for the angle main frames), the FE mesh is not. The density is slightly higher on the side of the panel where the plastic hinge forms. This higher element density results in an FE model with slightly lower

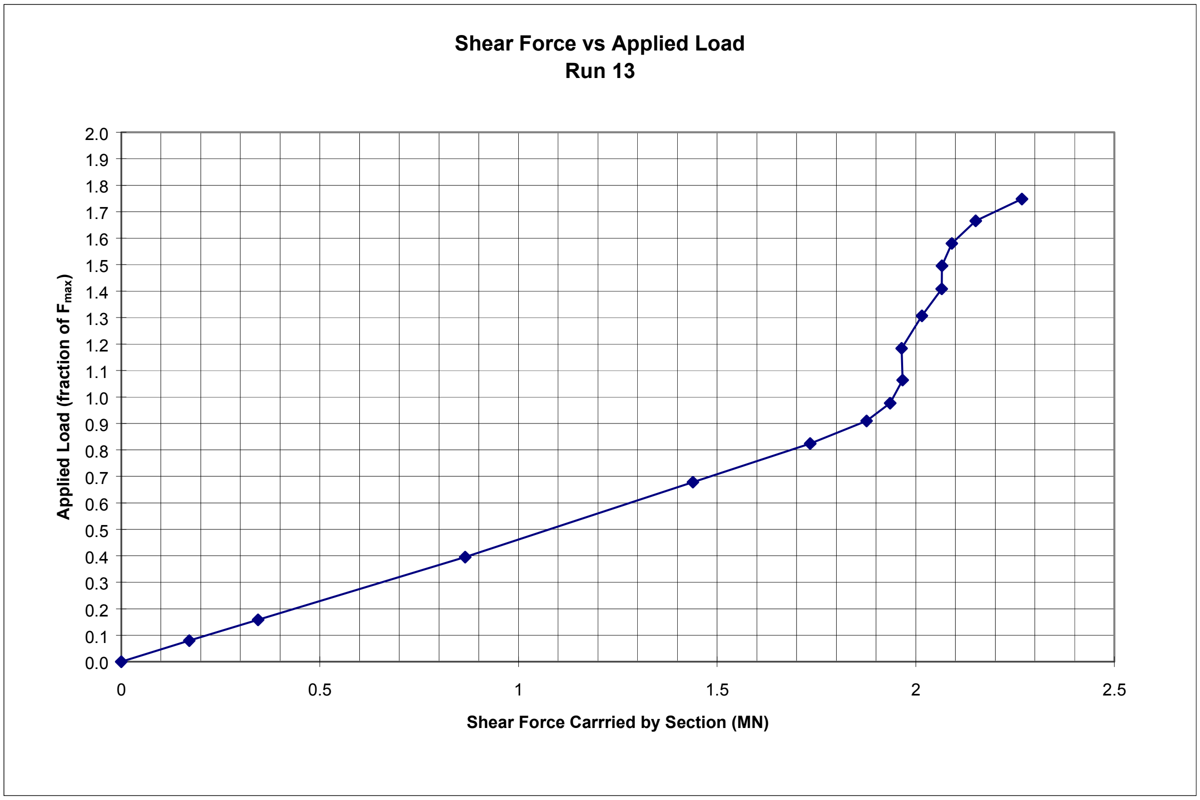


Figure 8.11 Typical Shear Force Curve for the Flat Bar Main Frame FE Model

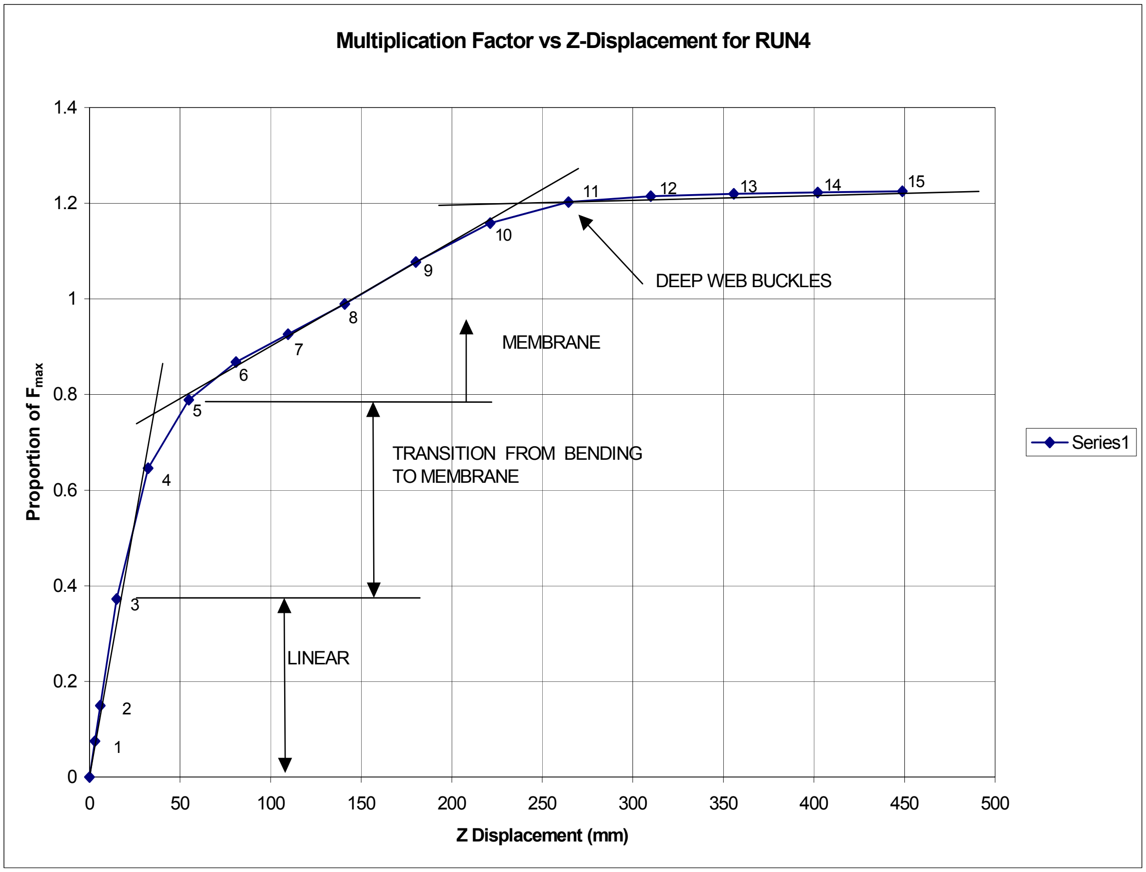


Figure 8.12 Load Displacement Curve Showing Regions of Change in Stiffness

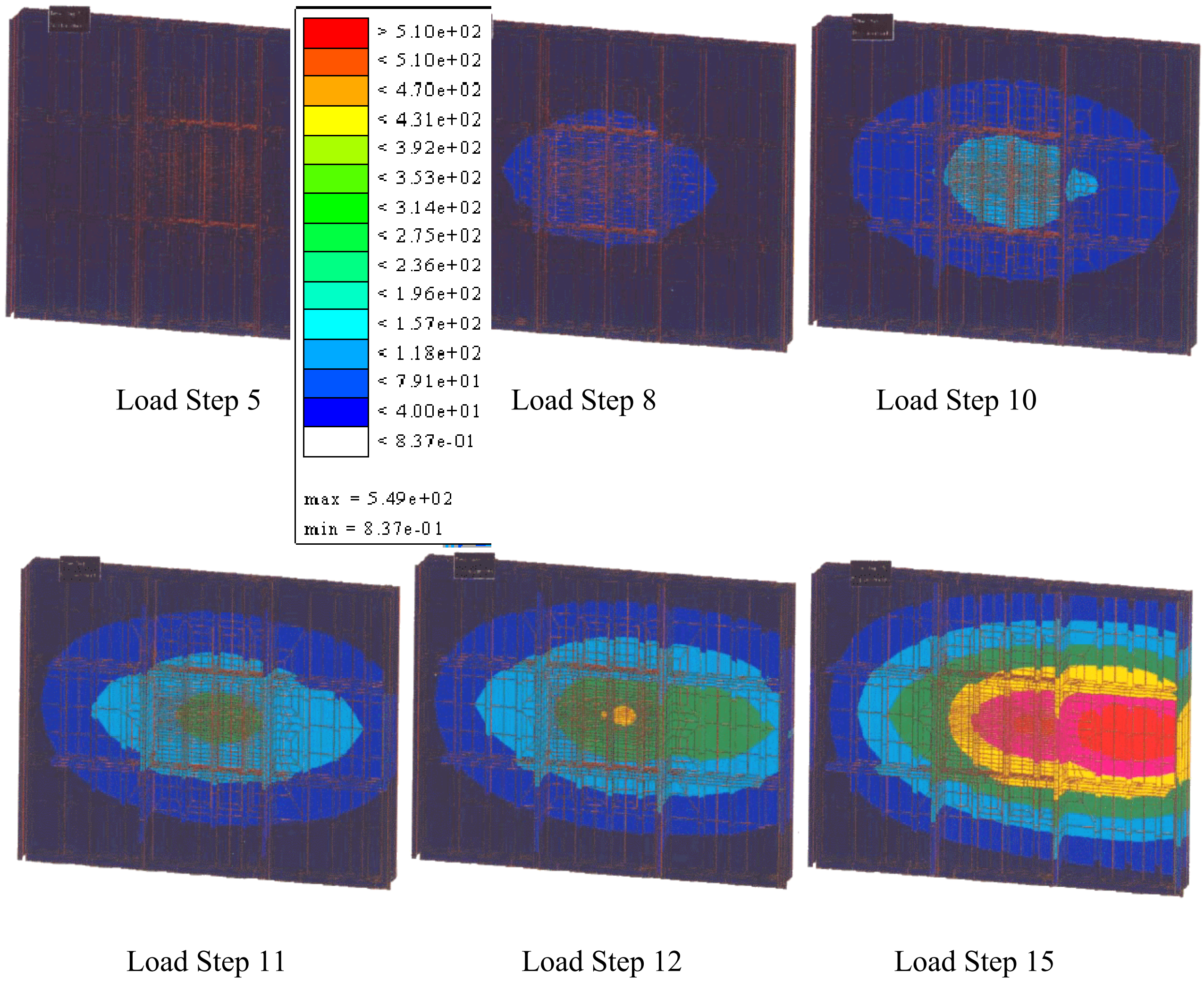


Figure 8.13 Typical Progression of Displacement in the Nonlinear Analysis of the 3x3 Bay Models

stiffness in these regions. This produces sufficient asymmetry to cause one side of the structure to yield before the other. Even though this response is a result of the numerical procedure employed in the FEA, there will most likely be sufficient asymmetry in the actual ship structural load conditions to produce a response similar to this.

What is important to note here is that the deep web plastic hinge forms at a load level greater than 1.2 F_{max}. With the main frame of interest already showing initiation of buckling at a load level of 1.08 F_{max}, the response of the deep web is not significant to the study.

A typical plot of progression of stress with increasing load is shown in Figure 8.14 from the Run 4 results. The stress component shown is the vertical S_{yy}, component which shows bending stresses in the linear range of the response that progress into membrane stress for the nonlinear range.

Figure 8.15 (from Run 4) shows a typical plot of the progression of yield (based upon the vonMises criterion) at various load steps. Yielding initiates at approximately 0.4 F_{max} (step 3) at outer fibres of the center bay central main frames at midspan. At 0.8 F_{max} (step 5), a large portion of the center section of the two deep webs and the center bay main frames have yielded. As the load increases, the plasticity progresses across the panel attempting to form a hinge. At the maximum load (1.31 F_{max} - step 15) the hinge has formed on the right side of the panel, but not completely on the left side as is evident by the unyielded section of the main frame. This asymmetry in the plasticity development is consistent with the displacement asymmetry as shown in Figure 8.13. At the highest load level, the plasticity has also fully developed vertically in the two central deep webs.

8.4.4 Effect of Stability Criteria Without In-plane Loads

It is likely that a ship stiffened panel subject to ice breaking forces will experience some in-plane loads that are caused by the global response of the ship. However, in the interests of better understanding the Equivalent Standards stability criteria and the effect that it has on main frame stability, the effect of stability criterion was evaluated without in-plane loads. The criterion was varied systematically from the design value (i.e. 1.0) and then decreased by approximately 30 percent (i.e. to 0.70) and to approximately 50 percent (i.e. to 0.50) without the application of in-plane loads. This was performed for each of flat bar, angle and tee main frame sections. For flat bar sections, this was performed for each of the span ratio and web ratio models. This constituted twelve of the fifty runs that were performed in the study. In the following sections, the FEA predicted responses for these twelve runs will be described and the overall effect of stability criteria on the main frame stability evaluated.

Angle Main Frame Sections

Figure 8.16 shows the 3x3 bay panel model which employs angle sections for the main frames. This was analyzed for three different main frame sections employing different

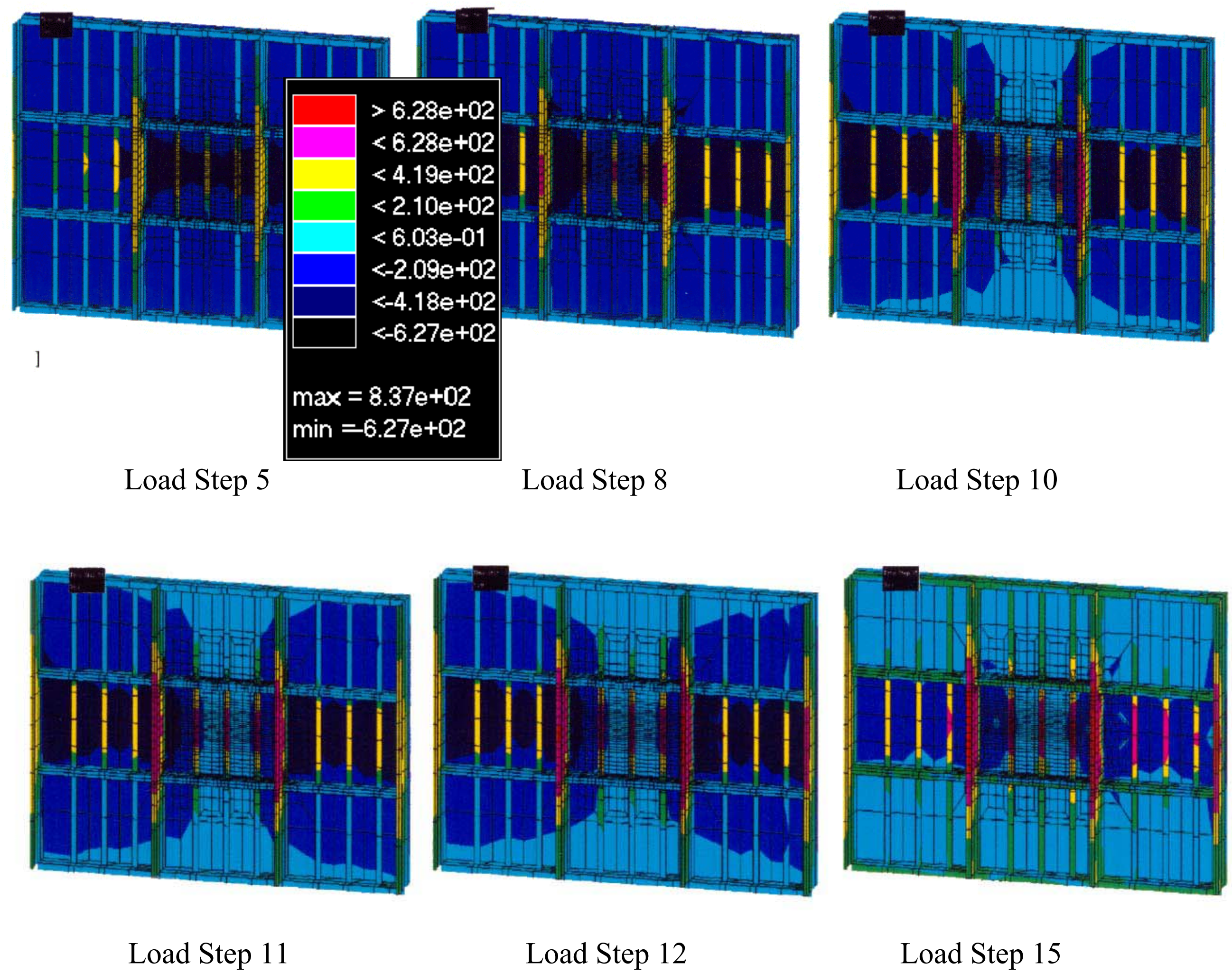


Figure 8.14 Typical Progression of S_{yy} Stresses in the Nonlinear Analysis of the 3x3 Bay Models

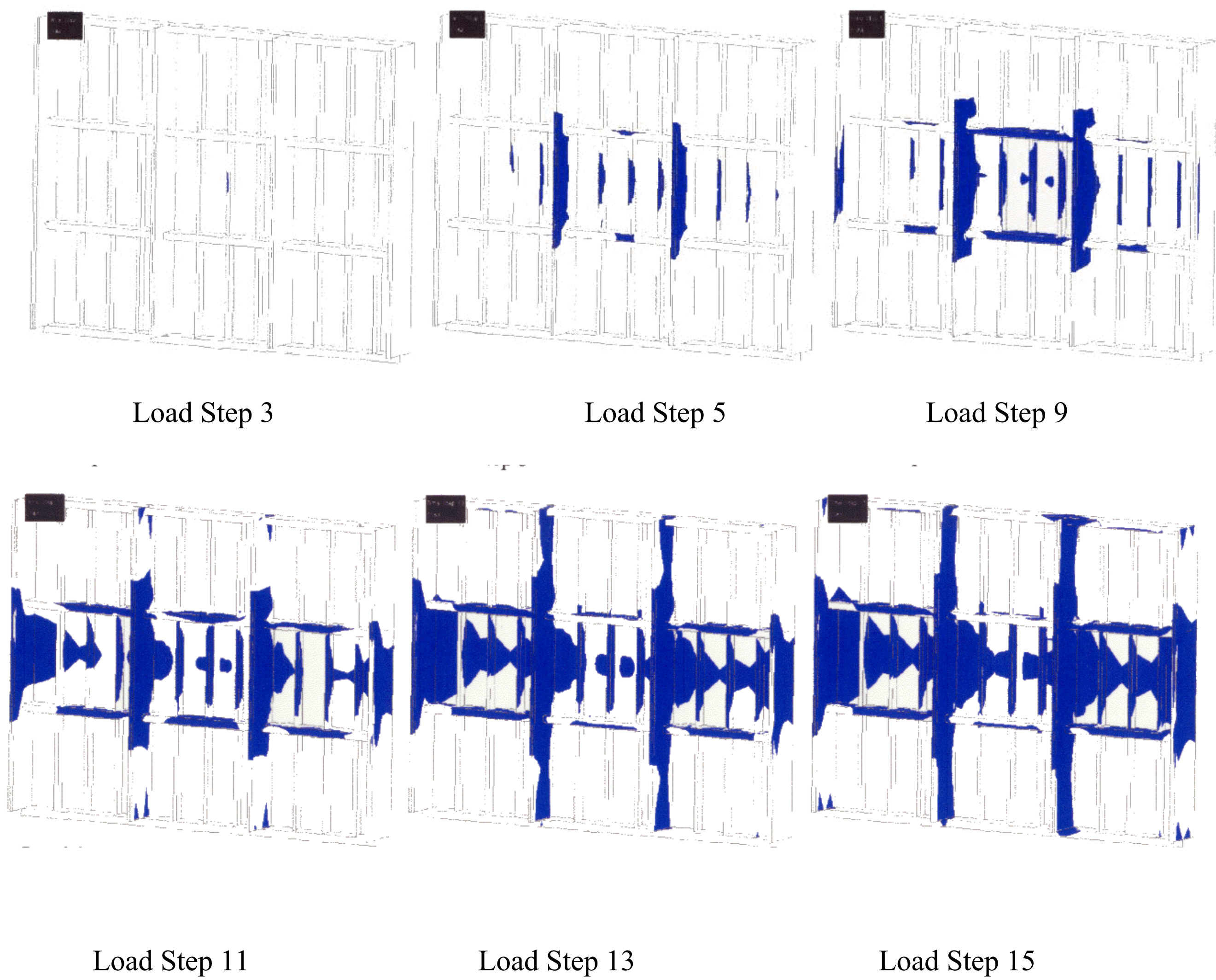


Figure 8.15 Typical Progression of Yielding in the Nonlinear Analysis of the 3x3 Bay Models

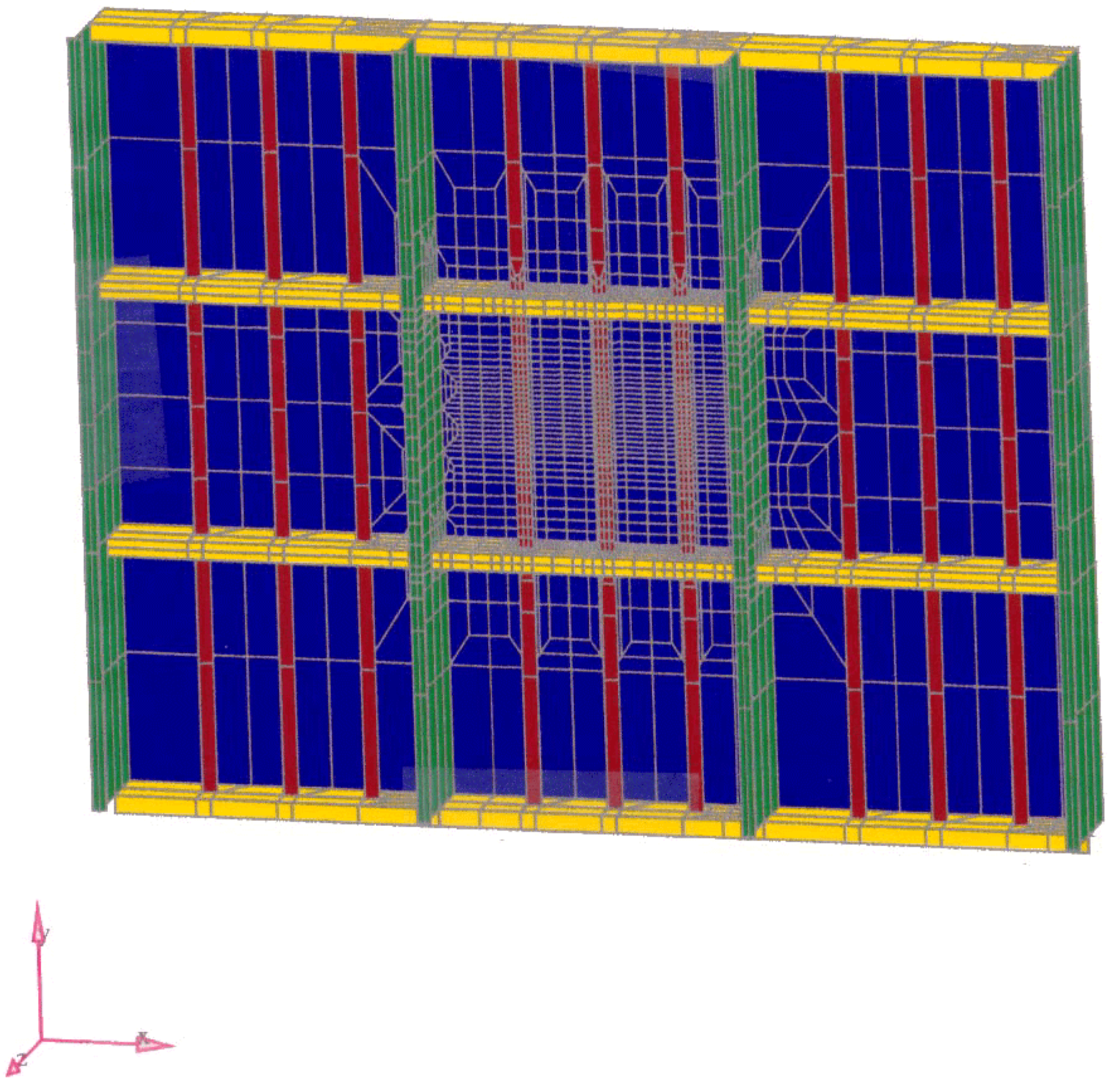


Figure 8.16 Plot of 3x3 Bay Panel Model Employing Angle Section Main Frame

stability criteria. All other scantlings which make up the panel were identical. Run (1) was designed such that the stability criterion was 0.77, or 77 percent of that required by the Equivalent Standards. This panel was slightly overdesigned for strength (1.21) and showed initiation of buckling, as indicated by a decrease in load carried by the main frame, at a load level of 1.23 F_{max}. When the stability criterion was further decreased to 0.61 (Run (2)) and then to 0.50 (Run (3)), the buckling load level decreased to 1.07 and 1.06, respectively. The strength criterion remained relatively unchanged at 1.28 and 1.21, respectively. The buckling load is plotted versus the stability criterion (span ratio) in Figure 8.17.

As can be seen from the figure and from the numbers presented in the above paragraph and in Table 8.1, main frame angle sections show a very weak and inconsistent dependence of buckling load level on the Equivalent Standards stability criterion. A 20 percent decrease in the stability criteria (i.e. from 0.77 to 0.61) resulted in a decrease in the buckling load of 13 percent. However, a further 18 percent decrease in the Equivalent Standards stability criterion (i.e. from 0.61 to 0.50) resulted in a decrease of less than 1 percent in buckling load.

Tee Main Frame Sections

Figure 8.18 shows the 3x3 bay panel model which employs tee sections for the main frames. As with the angle models, this was analyzed for three different main frame sections. All other scantlings which make up the panel were identical. Run (6) was designed such that the stability criterion was 1.06 of that required by the Equivalent Standards. This panel was slightly overdesigned for strength (1.28) and showed initiation of buckling at a load level of 1.08 F_{max}. When the stability criterion was then decreased to .70 (Run (7)) and then to .50 (Run (8)), the buckling load level decreased to 1.05 F_{max} for both. The strength criteria remained relatively unchanged at 1.19 and 1.14, respectively. The buckling load is plotted versus the stability criteria (span ratio) in Figure 8.19.

As can be seen from the figure and from the numbers presented in the above paragraph and in Table 8.1, the tee section shows virtually no dependence of buckling load level on the stability criterion. Even for a very large decrease in the stability criteria (i.e. from 1.06 to 0.50), the buckling load decreased by less than 3 percent.

Flat Bar Main Frame Sections using Span Ratio as the Stability Criterion

Figure 8.20 shows the 3x3 bay panel model which employs flat bar sections for the main frames. As with the angle and tee models, this was analyzed for three different main frame sections. All other scantlings which make up the panel were identical. Run (13) was designed such that the stability criteria was 1.00 (i.e. conforms to that required by the Equivalent Standards). This panel was just slightly over-designed for strength (1.07). The results show that at a load level of 1.07 F_{max} the plot of shear force versus applied load shows a dramatic change. While the angle and tee section main frames show a drop in shear force,

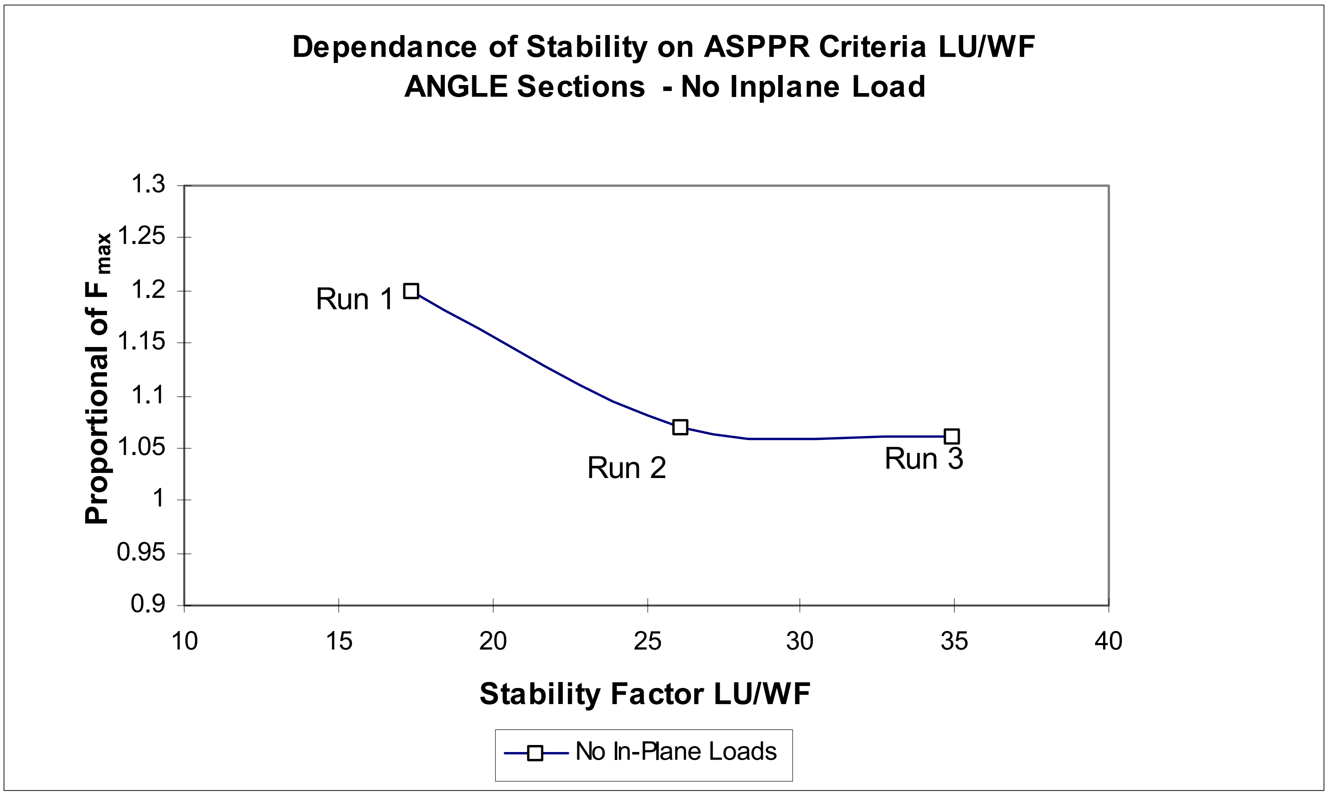


Figure 8.17 Buckling Load vs Stability Criteria for Angle Main Frames
No In-Plane Loads (Runs 1,2,3)

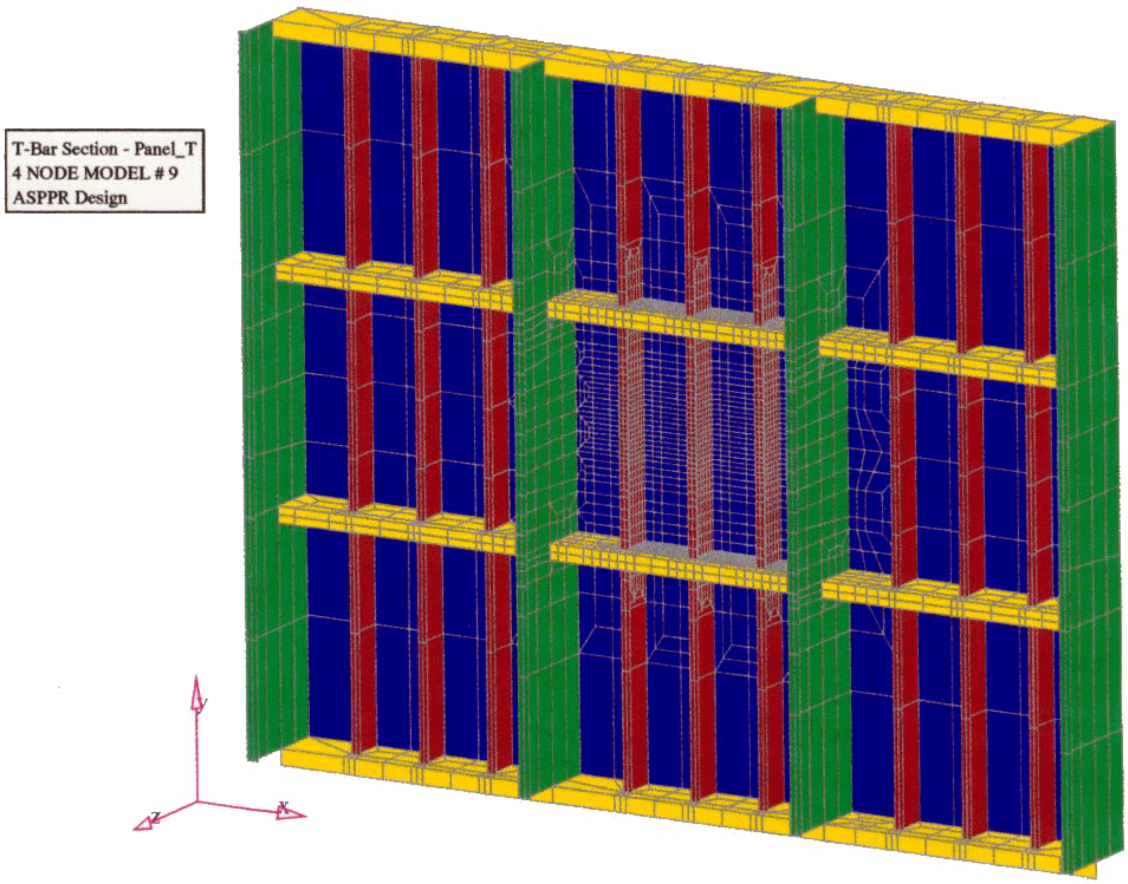


Figure 8.18 Plot of 3x3 Bay Panel Model Employing Tee Main Frame Sections

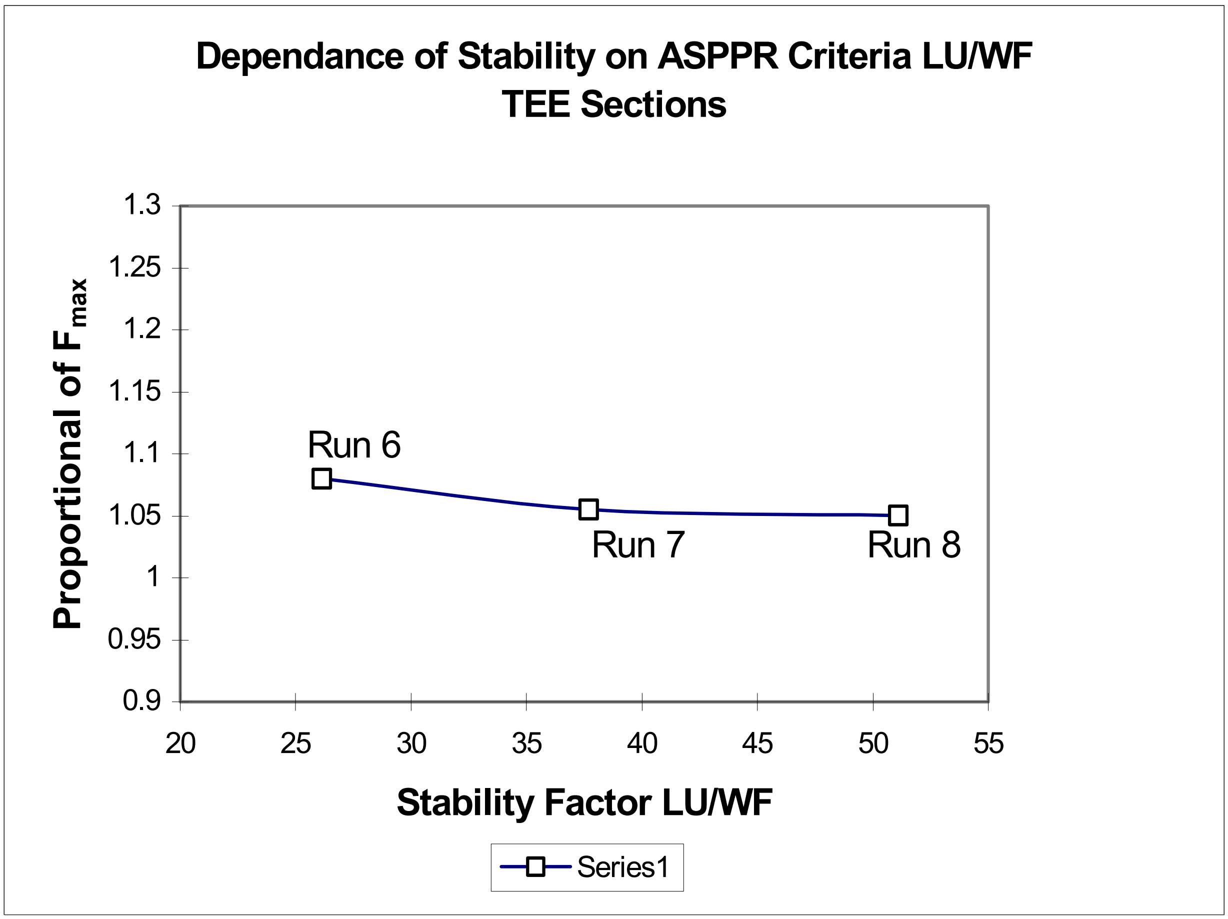


Figure 8.19 Buckling Load vs Stability Criteria for Tee Main Frames
No In-Plane Loads (Runs 6,7,8)

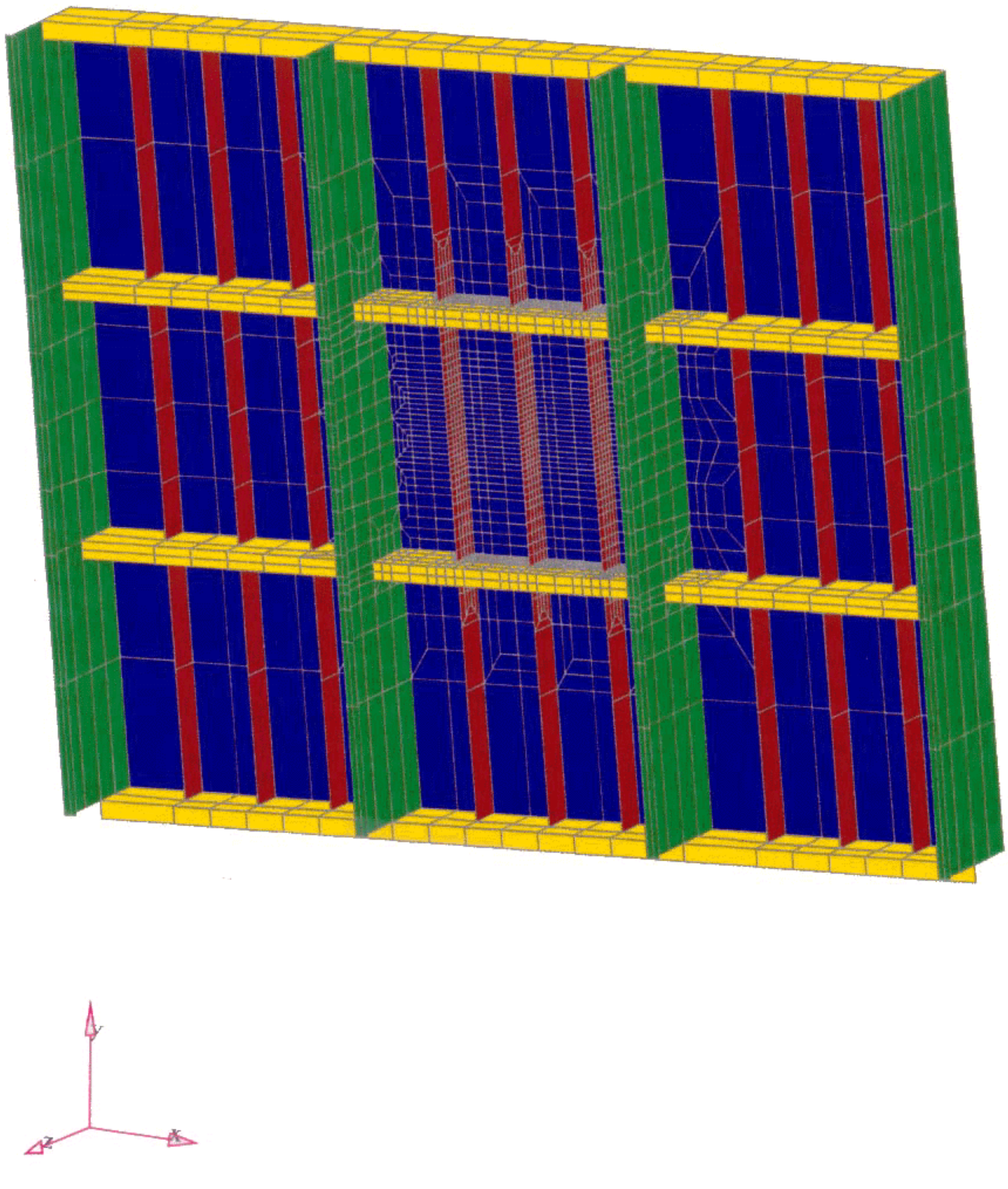


Figure 8.20 3x3 Bay Model Employing Flat Bar Main Frames

the flat bar section main frames do not show a drop in the shear force carried by the section. The flat bars do, however, lose their load carrying capability as shown by the vertical section of the shear force curve. When the stability criteria was then decreased to .84 (Run (19)) and then to .49 (Run (18)), the load level at which no further incremental load is carried (i.e. loading carrying capacity) increased to 1.09 F_{max} and 1.17 F_{max}, respectively.

The strength criteria remained virtually unchanged at 1.03 for both runs. The load carrying capacity is plotted versus the stability criteria (span ratio) in Figure 8.21.

As can be seen from the figure and from the numbers presented in the above paragraph and in Table 8.1, the flat bar sections shows virtually no dependence of load carrying capacity level on the stability criterion (based upon span ratio). For a very large decrease in the stability criterion (i.e. from 1.00 to 0.49), the load carrying capacity was seen to increase by 10 percent. The reason for the increase is unexplained.

The conclusion is reached that the flat bars modified according to span ratio show no indication of buckling and no dependence of load capacity level on the Equivalent Standards stability criterion.

Flat Bar Main Frame Sections using Web Ratio as the Stability Criterion

Figure 8.22 shows the 3x3 bay panel model which employs flat bar sections for the main frames for the web ratio analysis. This was analyzed for three different main frame sections (with different stability criterion) while keeping all other scantlings in the panel identical. Run (25) was designed such that the stability criterion was 1.01 of that required by the Equivalent Standards. This panel was well designed for strength (1.02) and showed the limit of load carrying capacity at a load level of 1.11 F_{max}. When the stability criterion was then decreased to .69 (Run (14)) and then to .53 (Run (15)), the load capacity level increased to 1.30 F_{max} and then decreased to 1.06 F_{max}, respectively. The strength criterion increased from 1.02 to 1.04 to 1.15 for runs (25), (14) and (15), respectively. The load carrying capacity is plotted versus the stability criterion (span ratio) in Figure 8.23.

As can be seen from the figure and from the numbers presented in the above paragraph and in Table 8.1, the flat bar sections show virtually no consistent dependence of load capacity level on the Equivalent Standards stability criterion. For a very large 30 percent decrease in the stability criteria (i.e. from 1.01 to 0.69), the load capacity was seen to increase by 18 percent. The reason for the increase is unexplained. When the buckling criteria was further decreased to .53 (Run (15)), the load carrying capacity decreased to 1.06 F_{max}.

The conclusion is reached that the flat bars modified to give different web ratios show no indication of buckling and dependence of load capacity on the Equivalent Standards stability criteria.

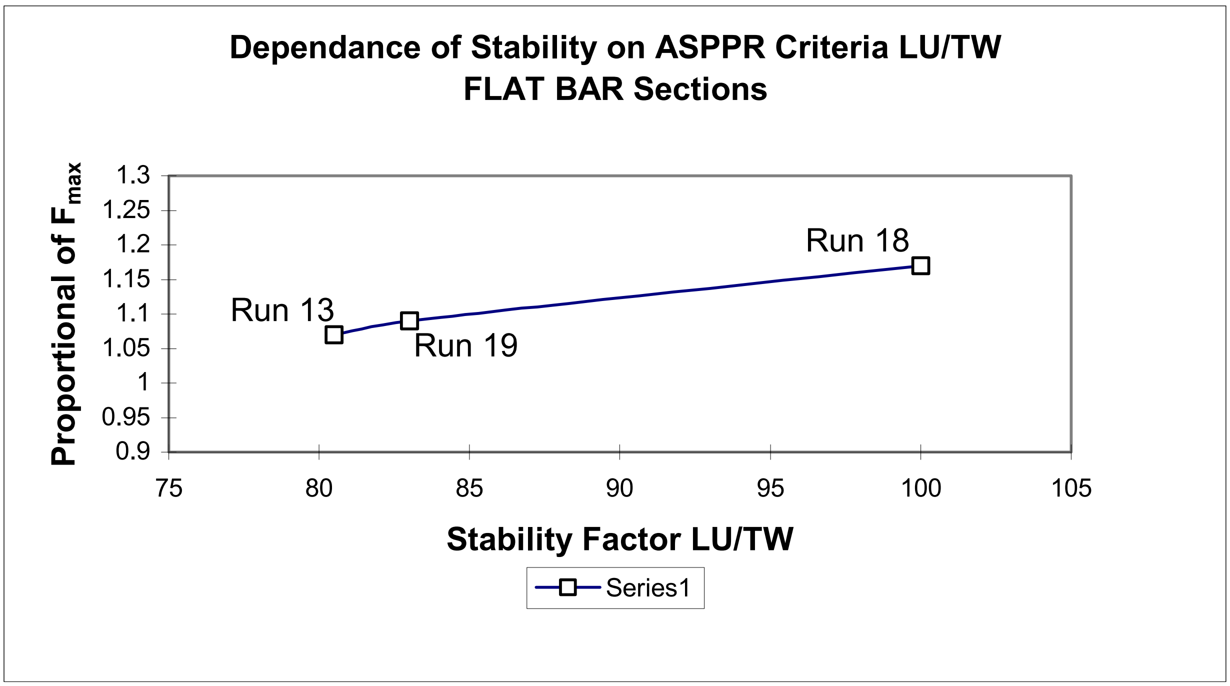


Figure 8.21 Buckling Load vs Stability Criteria for Flat Bar Main Frames No In-Plane Loads – Span Ratio (Runs 13, 19, 18)

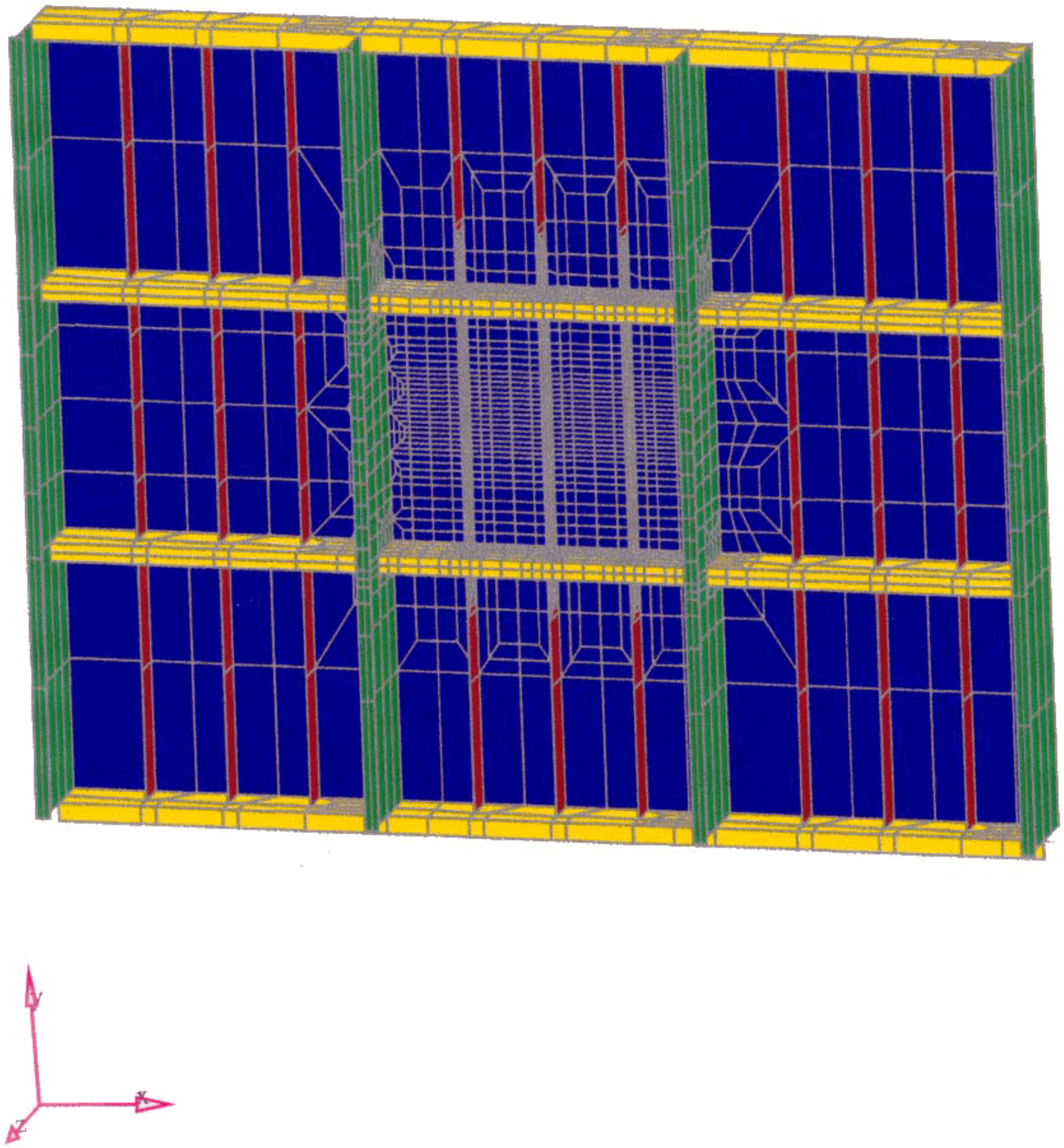


Figure 8.22 3x3 Bay Model Employing Flat Bar Main Frames Designed using Web Ratio

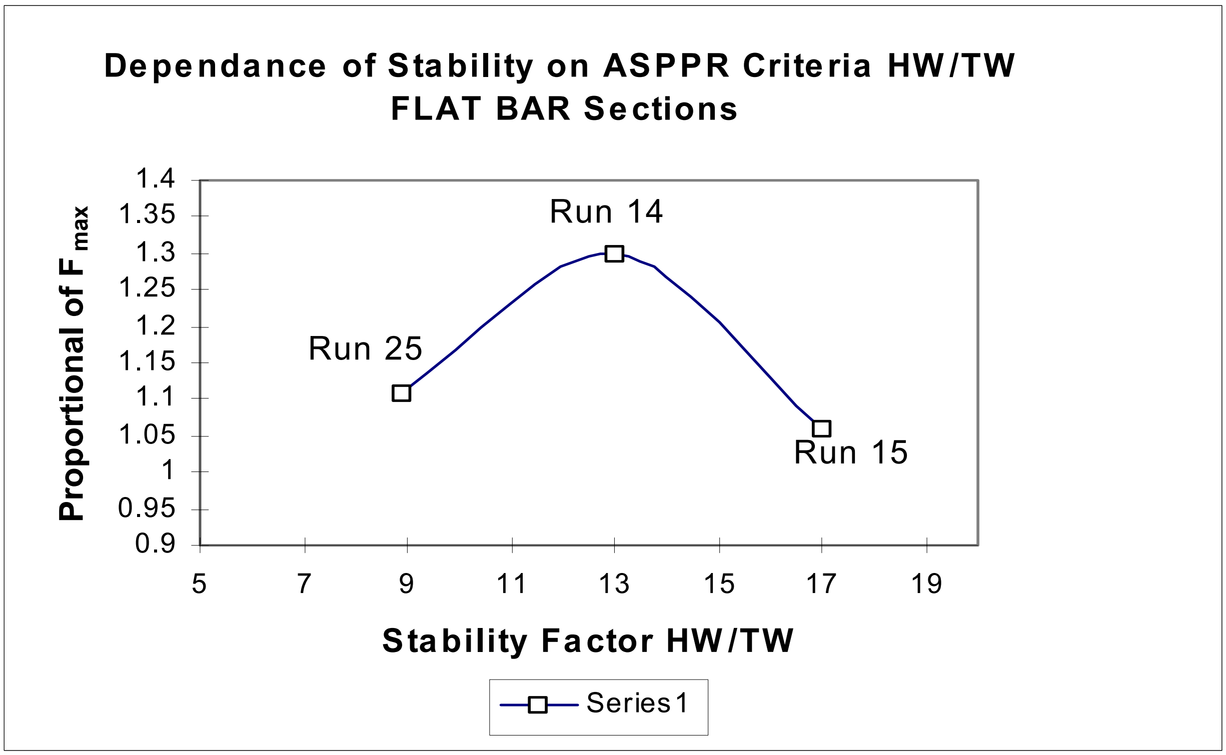


Figure 8.23 Buckling Load vs Stability Criteria for Flat Bar Main Frames
No In-Plane Loads – Web Ratio (Runs 25, 14, 15)

8.4.5 Effect of Stability Criteria in the Presence of In-plane Loads

The study that was described in the previous section 8.4.4, investigated the effects of stability criteria on main frame stability without the existence of applied in-plane stresses. The results showed that there was no clear dependence of stability on the Equivalent Standards stability criteria. The question remains of whether or not a panel with applied in-plane loads shows a different dependence upon the stability criteria than one without in-plane loads.

As described in Section 6.1.2, the global action of the ship, in response to applied ice loads at midship, will impart in-plane compressive stresses on the 3x3 bay stiffened panel. To investigate the effect of this, it was decided to repeat Runs (1-3) which were modified to include an applied in-plane biaxial stress of 69 MPa. The same procedure was followed as in section 8.4.4 and the same models were re-run with the applied in-plane stress. These runs were numbered Run (4), Run (50) and Run (49). The panel was found to show initiation of buckling at load levels of 1.08, 1.0 and 0.96 for corresponding stability criterias of .77, .61 and .50, respectively. The results are plotted in Figure 8.24 along with the results for Runs (1-3) without the in-plane stress.

From Figure 8.24, one can see that the in-plane stress component had the effect of decreasing the stability and that the curve shifted down (i.e. the buckling load decreased) by about 10 percent. The slope of the curve again indicates a decrease in stability with decreasing stability criteria. However, a decrease of 35 percent in stability criteria, decreased the buckling load by only about 11 percent. The change was found to be more stable with the in-plane load and the slope of the curve can be seen to be relatively constant over the three points investigated.

The results showed that for stability in the presence of in-plane loads, the stability criterion has an effect on the observed stability but that it is a relatively small effect. Therefore, it can be concluded that the stability is relatively insensitive to the changes in stability criteria for main frames whose design is controlled by span ratio.

Time did not warrant repeating Runs (6, 7, 8), Runs (25, 14, 15), and Runs (13, 18, 19) with an in-plane stress to determine whether or not tees and flat bars would also show little dependence of stability upon the Equivalent Standards stability criterion in the presence of in-plane stresses. However, it is concluded from the study using angle main frame sections that whatever phenomena is present without in-plane stresses is likely to hold with in-plane stresses, with a shift in the Buckling Load vs. Stability Criteria curve. Therefore, it is concluded that tees and flat bar sections will not show a clear relationship between stability and stability criteria in the presence of in-plane stresses, since section 8.4.4 showed no clear relationship without in-plane stresses.

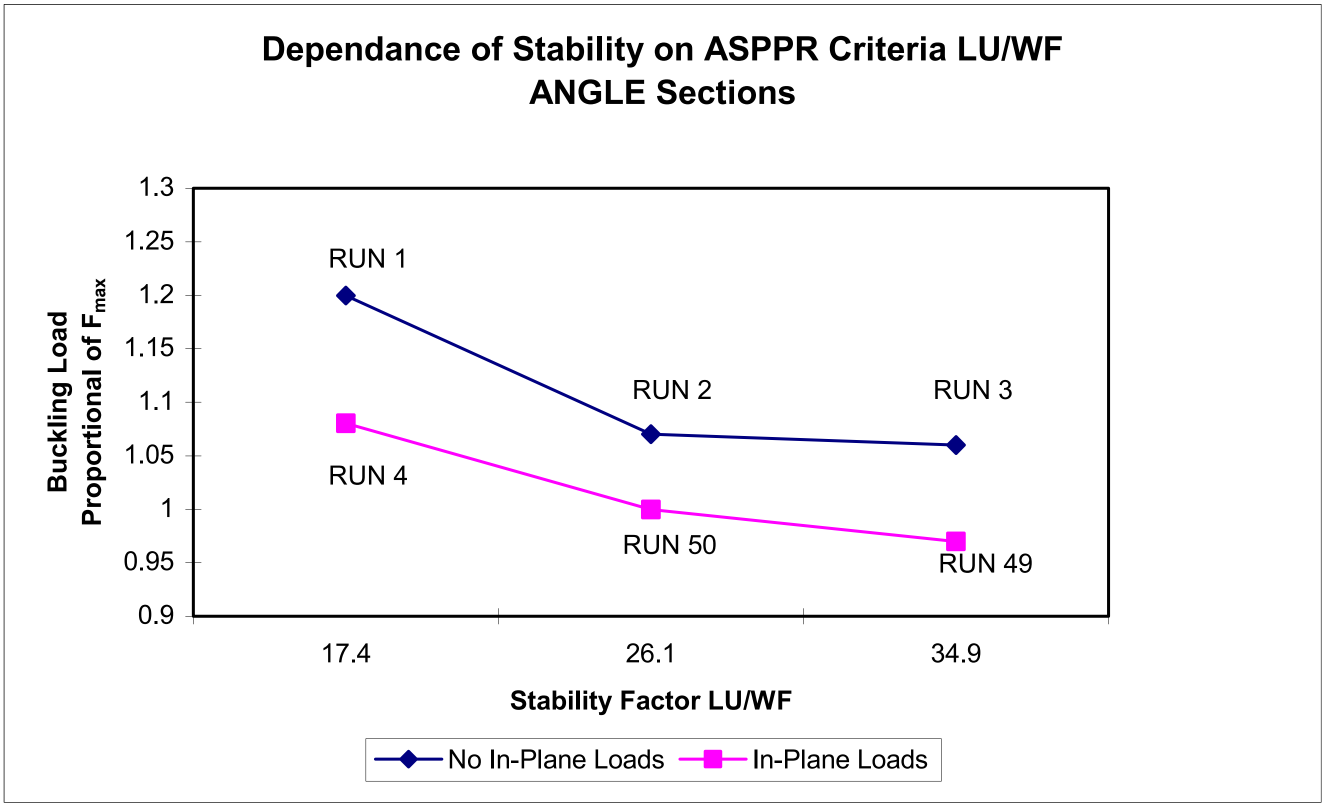


Figure 8.24 Buckling Load vs Stability Criteria for Angle Main Frames
In-Plane vs. No In-Plane Loads (Runs 1-3, Runs 4,50,49)

8.4.6 Effect of In-plane Loads on Plastic Stability

The effect of global in-plane stresses is considered to be the most significant influence (see Section 6.1.2) on main frame stability and in particular, on stability subsequent to yielding. Four FE models were developed for this aspect of the study. One model was developed for tee main frames, one for angle main frames and one each for flat bar main frames according to the two the Equivalent Standards stability criteria models. Each of these models was then analysed using various magnitudes of in-plane loads. Details of the results are presented below.

Tee Sections

Run (6) shown in Table 8.1 is a tee section main frame panel with a stability criteria of 1.06. According to the Equivalent Standards, this main frame should remain stable at load levels up to F_{max}. Also shown in Table 8.1 are runs (10), (9) and (31) which are the same panel with different levels of bi-axial in-plane load, run (6) being with no in-plane load, and runs (10), (9), and (31) including a 34, 69, and 103 MPa in-plane load, respectively.

The nonlinear FE results of each of these runs were processed to determine buckling load level, main frame lateral displacement and normal panel displacement as a function of applied load. The buckling load was determined using the shear force difference method. Figure 8.25(a) shows a plot of shear force difference carried by the main frame section versus applied load as a fraction of F_{max} for each of the runs. The point of initial instability is selected as the point where the shear force difference carried by the section decreases with applied load. It can be seen from this curve that as the in-plane load increases the buckling load level decreases. These levels are shown in Table 8.1. Figures 8.25(b) and 8.25(c) show load displacement curves for the frame lateral and out-of-plane displacements, respectively. Similar to the shear force difference curve, it can be seen that as the in-plane load increases the point of initiation of large lateral displacements decreases. These curves all show a distinct relationship between the magnitude of the in-plane load and the buckling load where higher in-plane loads produce lower buckling loads.

A summary plot of buckling load vs. stability criteria is contained in Figure 8.26 with the results of angle and flat bar sections as described in the following paragraphs.

Angle Sections

Run (1) shown in Table 8.1 is an angle section main frame panel with a stability criteria of 0.77. Based upon the Equivalent Standards criteria, with a stability criterion less than 1.0, this panel should become unstable at load levels well below F_{max}. Also shown in Table 8.1 are runs (5), (4) and (32) which are the same panel with different levels of in-plane load, run (1) being with no in-plane load, and runs (5), (4), and (32) including a 34, 69, and 103 MPa biaxial in-plane loads, respectively.

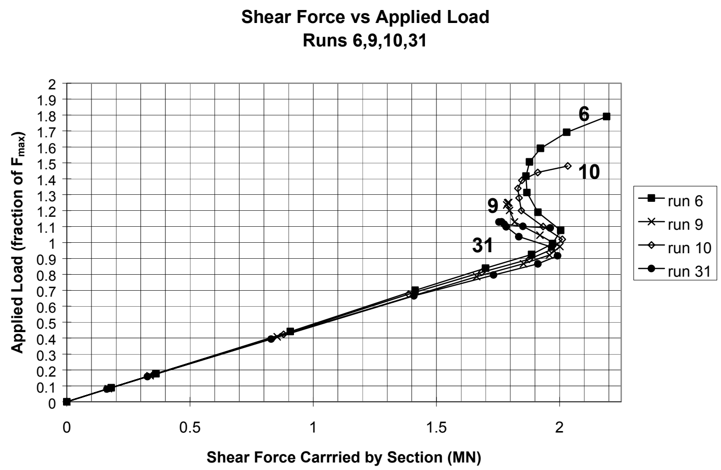


Figure 8.25(a) Shear Force vs Applied Load

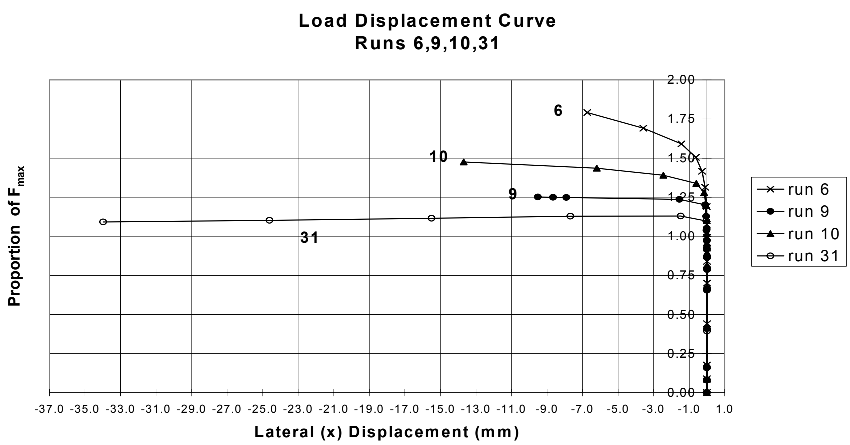


Figure 8.25(b) Load vs Lateral Displacement Curves

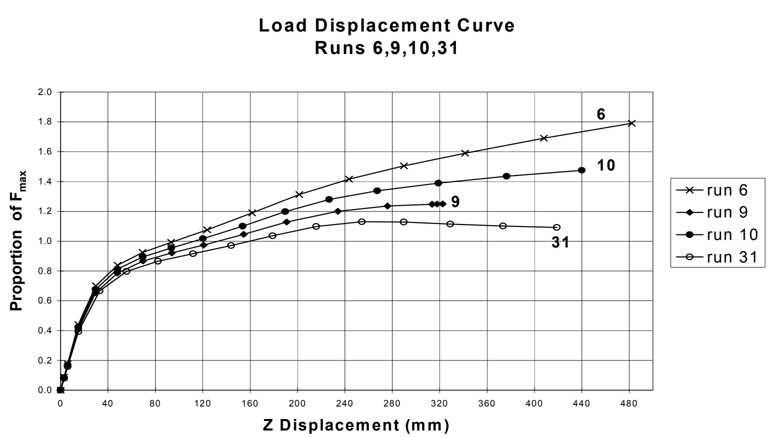


Figure 8.25(c) Load vs Out-of-Plane Displacement Curves

Figure 8.25 Effect of In-Plane Load on Stability for Tee Sections

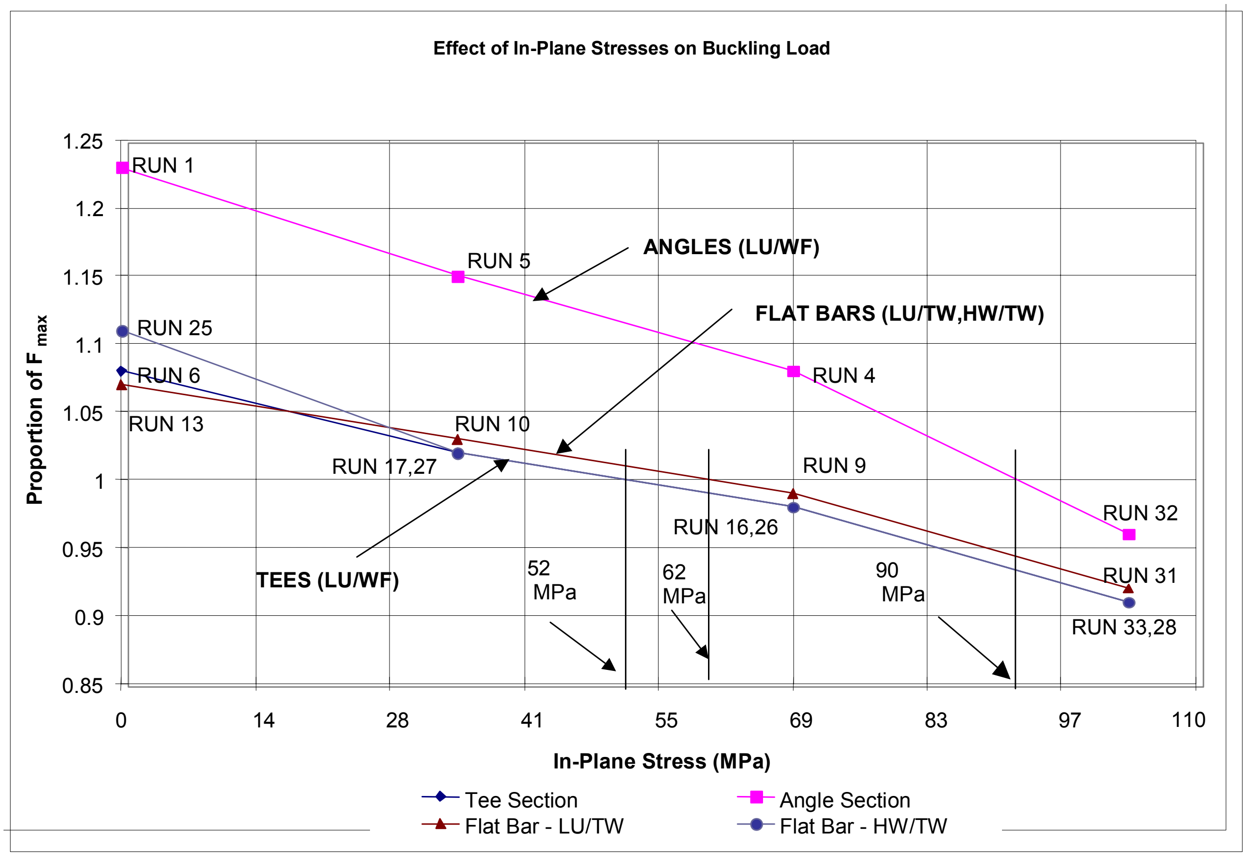


Figure 8.26 Buckling Load vs Stability Criteria Showing Effects of In-Plane Loads for Angle, Tee and Flat Bar Main Frames

These analyses produce the same type of results as from the tee model analyses. A distinct relationship exists where higher in-plane loads produce lower buckling loads in angle main frame panels.

Flat Bar Sections

Run (13) shown in Table 8.1 is a flat bar section main frame panel with a stability criteria of 1.00 based upon the span ratio, LU/TW. Similar to the tee section, this panel has been designed according to the Equivalent Standards such that the main frames should remain stable at load levels up to F_{max}. Also shown in Table 8.1 are runs (17), (16) and (33) which are the same panel with different levels of in-plane load, run (13) being with no in-plane load, and runs (17), (16), and (33) including a 34, 69, and 103 MPa in-plane load respectively. The web ratio, HW/TW was maintained at a value of 8.9 for all of these analyses.

As with the angle and tee main frame sections, these results show that higher in-plane loads produce a change in the shear force difference curves indicating that loss of incremental load carrying capability occurs at lower load levels in flat bar main frame panels.

Runs (25), (27),(26) and (28) employ the same panel again but with the stability design for the flat bar main frames designed using the web ratio, HW/TW, stability formula. This results in a change in the dimensions of the main frame flat bars. Run (25) employs no in-plane load, and runs (27), (26), and (28) include a 34, 69, and 103 MPa in-plane load respectively. The span ratio, LU/TW, was maintained at 80.5 for all of these analyses.

The incremental load carrying capability is virtually identical to that for the span ratio designed panel except for a small deviation for no in-plane load. This is due to the fact that buckling does not occur and the curves are indicating the level of extensive yielding.

Summary

The results of these series of analyses are displayed in Figure 8.26 where the buckling load (as determined from the shear force difference curves) is plotted versus the magnitude of applied load for each tee, angle and flat FE model analysis. (Note that for the flat bars which do not buckle, this indicates the point where no incremental load can be carried by the frame.) Although the in-plane stress is shown in the plot as a given value, the in-plane stress is applied to the model proportionately with applied lateral load and the value shown in the plot is the magnitude of the in-plane stress at F_{max} .

From the figure it can be seen that for all main frame sections, the buckling load decreases with increasing in-plane stress in what appears to be a fairly linear relationship. Vertical lines are drawn from the in-plane stress value to where each curve crosses the 1.0 F_{max} load. For angle sections, it can be seen that the Equivalent Standards underpredicts the buckling load below an in-plane stress of about 90 MPa (i.e. is conservative) and overpredicts the buckling load for in-plane stresses in excess of 90 MPa. (i.e. is nonconservative). For tee main frame sections, this value is approximately 52 MPa.

8.4.7 Effect of Tilt Angle on Plastic Stability

The effect of tilt angle on plastic stability was initially assumed to be insignificant. To check this assumption, three FE models were developed: one model for tee, one for angle, and one for flat bar main frames. The Equivalent Standards stability criterion was maintained for each model and the FEA was carried out using two different values of tilt angle. The models developed for these analyses also included the 69 MPa bi-axial in-plane loads. This was considered to be a more realistic loading condition.

The stiffened panel model employing angle section main frames designed in run (4) was modified to include a tilt angle in the main frames of 15° and 30° in runs (35) and (34), respectively. The tee section main frames designed in run (9) were as modified for 15 and 30 degree tilt angles in runs (41) and (40), respectively. And, the flat bar section panel in run (26) was modified to include tilt angles of 15° and 30° in runs (43) and (42) respectively.

For tees and angles, stability (and for flat bars, the point of loss of incremental strength) was found to decrease with increasing the tilt angle (of the web to a normal to the shell plating). This is shown in Figure 8.27. The stability equations in the Equivalent Standards make some allowance for this for tilt angles in excess of 15°. For tee sections, a decrease in buckling load level of 17 percent was achieved by tilting the frame by 30°. This is out of proportion to the corresponding decrease in the stability criterion of only 6 percent.

8.4.8 Effect of Distortion on Stability

In ship construction, it is normal to have distortion in main frames. Distortion is defined as the geometric imperfections imparted into the otherwise perfect dimensions by construction practices. This can be caused by welding or by construction tolerances. Distortion has been identified in section 6.1.2 as one of the stability influences which are to be investigated in the current study.

As described in section 6.1.2, the Canadian Coast Guard has developed construction guidelines which define the amount of acceptable distortion for main frames in terms of the "out-of-straightness" of the frame and allows lateral stiffener deformation not greater than 5 degrees.

In determining the effect of distortion on stability, three runs were conducted which varied the distortion from 0 percent to 100 percent to 200 percent of the CCG established acceptable distortion. The distortion was assumed to be a multiple of the fundamental linear

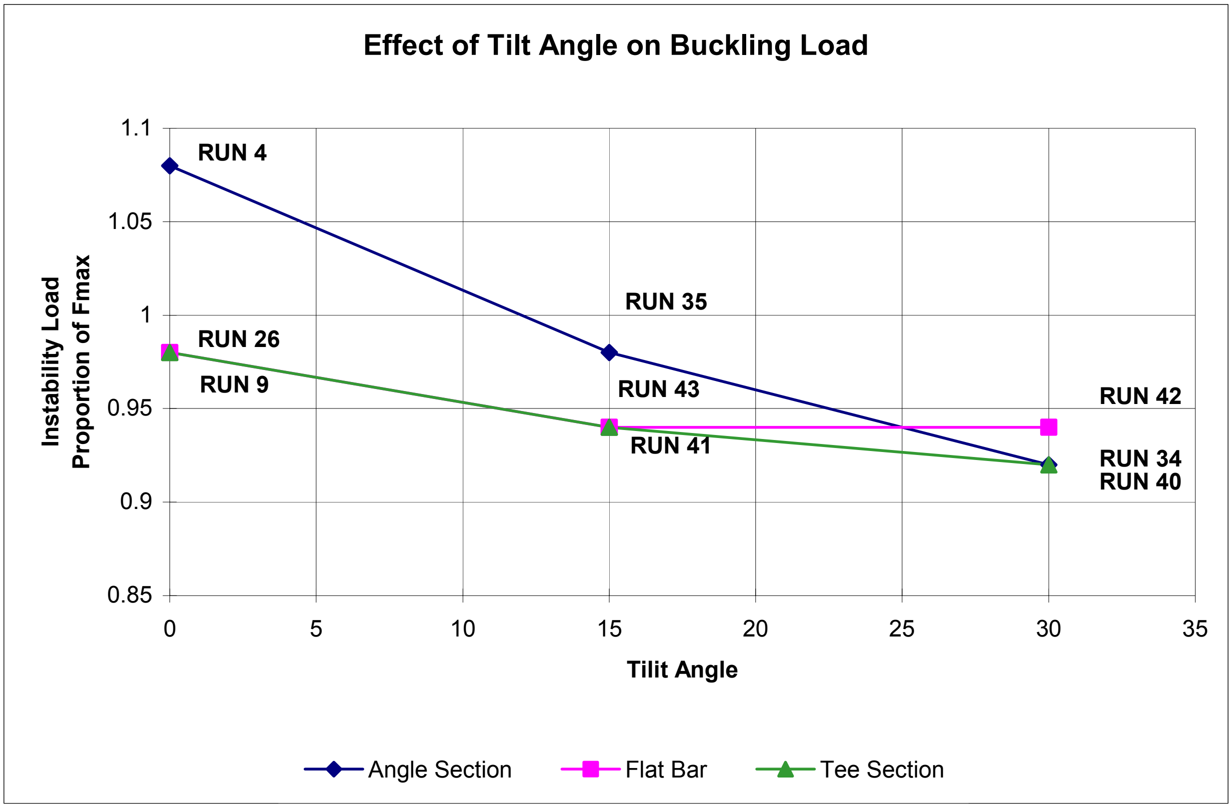


Figure 8.27 Tilt Angle vs Buckling Load

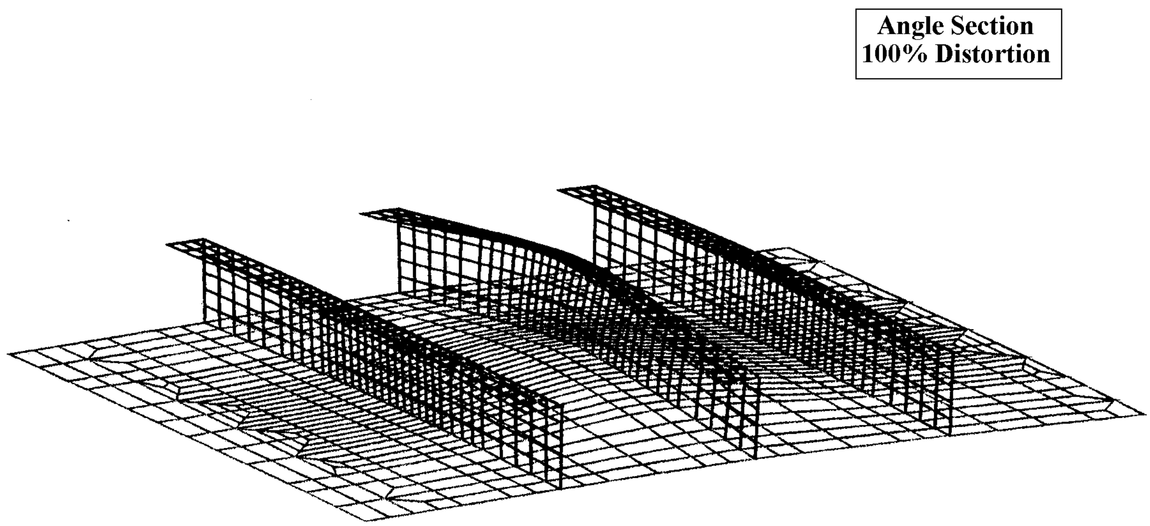


Figure 8.28 Fundamental Buckling Load of Angle Main Frame

buckling mode shape for the frame which is shown in Figure 8.28. The frame used for the analysis was the angle section main frame defined in Run (4) which includes an in-plane stress of 10 000 psi. The Run (4) model was then modified by imposing a geometric distortion to the main frame as depicted by the mode shape in Figure 8.28. Run (36) included a 100 percent distortion and Run (37) included a 200 percent distortion.

Since the same model was used for each of the runs and the only changes between models was the applied distortion, the design criteria (which does not include any effect of distortion) was identical for all three runs. The panels were designed according to the Equivalent Standards with a stability criteria of 0.77 and a strength criteria of 1.21. One would expect these panels to buckle at a load slightly less than F_{max}. The results of the three runs are summarized in the plot in Figure 8.29 which shows that the buckling load is unaffected by the distortion. The buckling load levels are 1.08, 1.05 and 1.10 for 0 percent, 100 percent and 200 percent distortion, respectively.

The conclusion drawn from these runs is that main frame stability is unaffected by distortion of up to twice the CCG acceptable distortion level for the geometry investigated.

8.4.9 Effect of Residual Stresses on Stability

To determine if residual stresses affect main frame stability subsequent to yielding, the run (4) model was modified for Run (38) to include residual stresses. Based upon the typical residual stress pattern found in welded ship structures as outlined in Section 6, initial strains were included in the FE model to represent residual stresses. The residual stress pattern for Run (38) is as shown in Figure 8.30. This pattern shows that a yield strain of .0017 mm/mm was applied at the intersection of the main frame and outer shell. This strain decreases linearly to zero at a distance of 90 mm in the shell plating and at a distance of 68.5 mm in the main frame web.

The shear force difference curve and load-displacement curves for Run (38) are shown in Figure 8.31. When these curves are compared with the corresponding curves for Run (4), there is virtually no difference in the results. A plot of the buckling load (as determined from the shear force difference curves) versus the residual stress used is shown in Figure 8.32 for Runs (4) and (38). This curve shows that residual stress seems to have very little effect on plastic stability.

It should be noted that there is some uncertainty in making a general conclusion on the overall effect of residual stresses based upon a very local application using FE analysis. The residual stress pattern used in this study was only applied to the most central main frame. If any significant change in response had occurred, a more extensive residual stress pattern would have been modelled. However, this did not occur and the conclusion is that residual stress does not seem to have any effect on stability, therefore the remaining resources were put into studying other stability parameters.

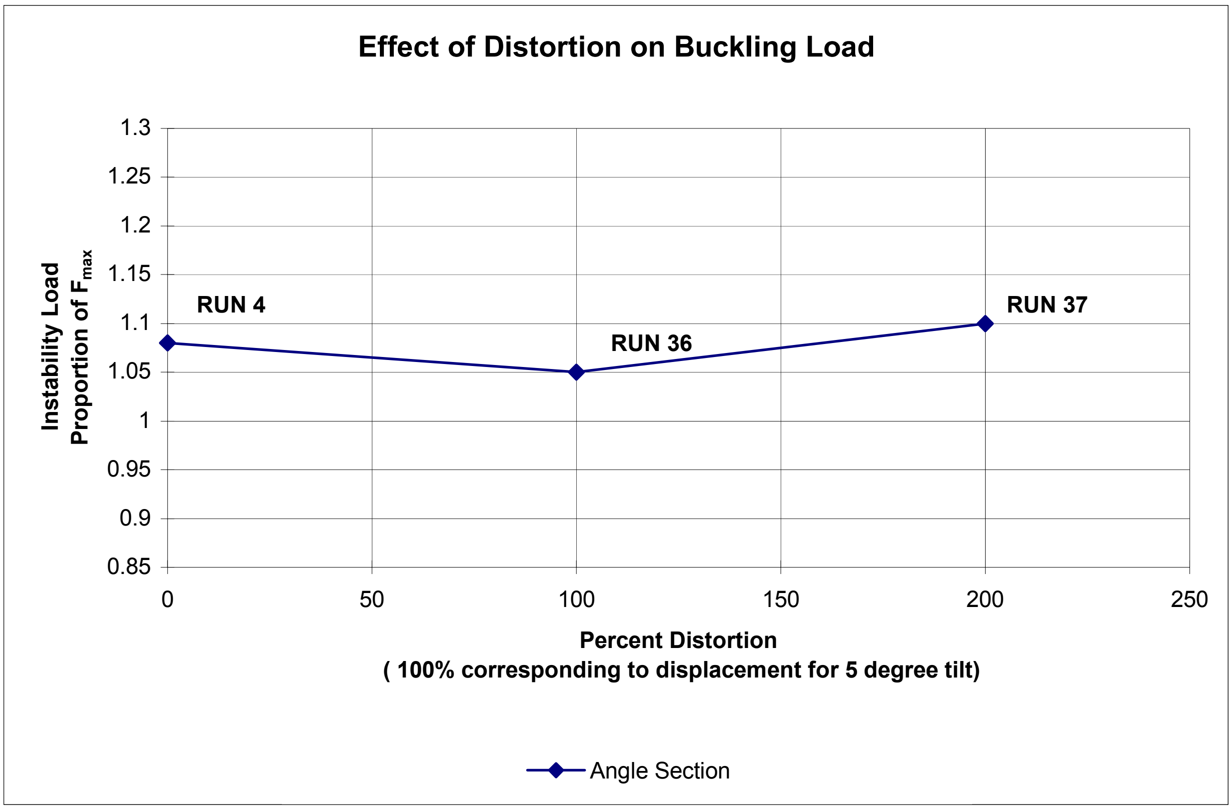


Figure 8.29 Buckling Load vs Distortion

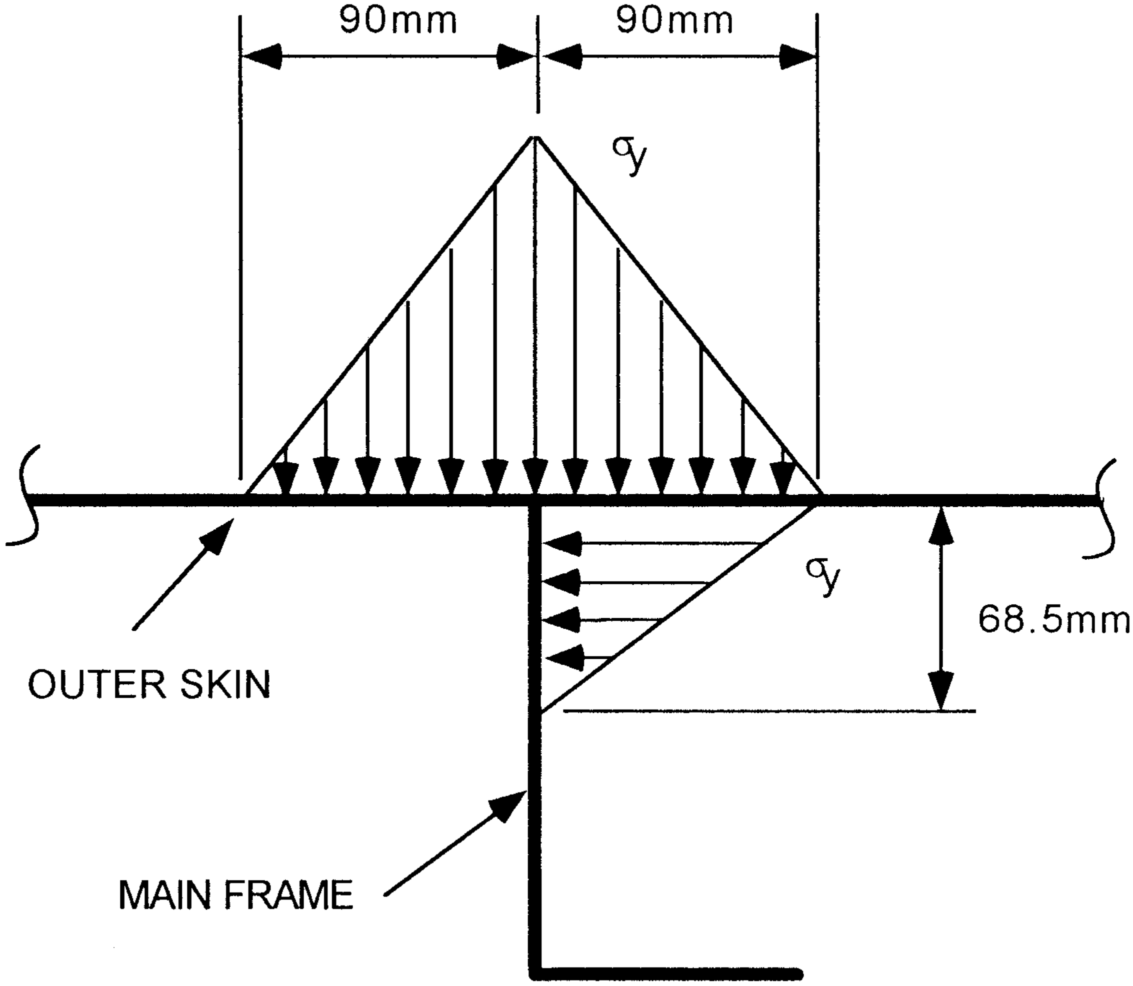
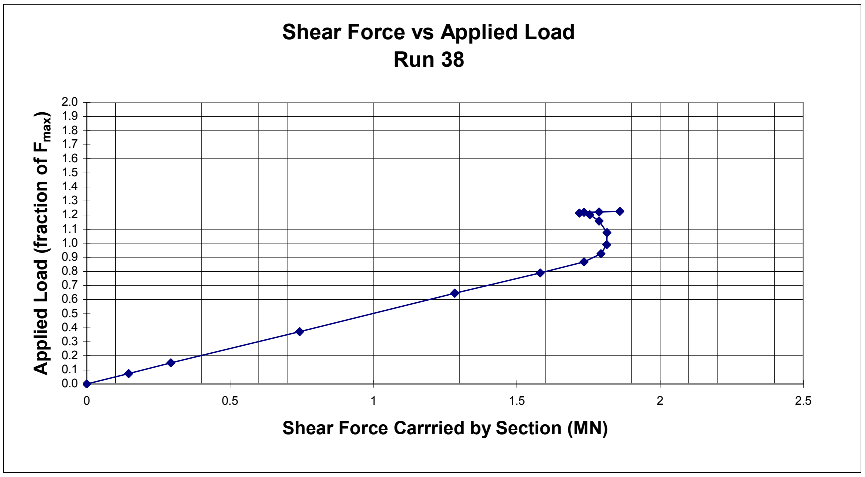
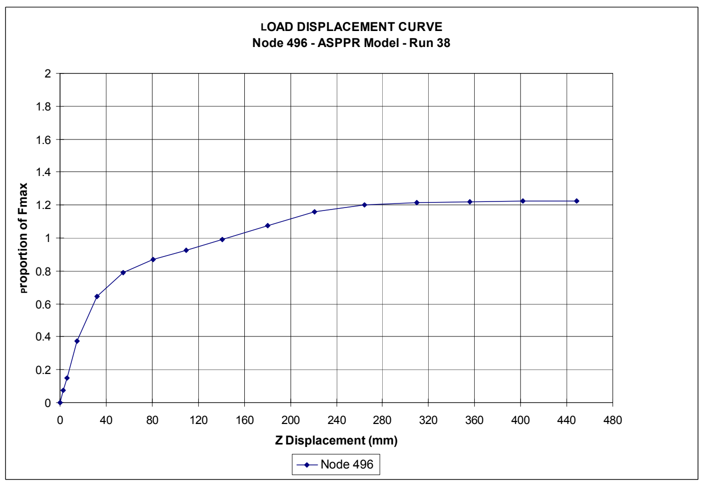


Figure 8.30 Sketch of Residual Stress Pattern





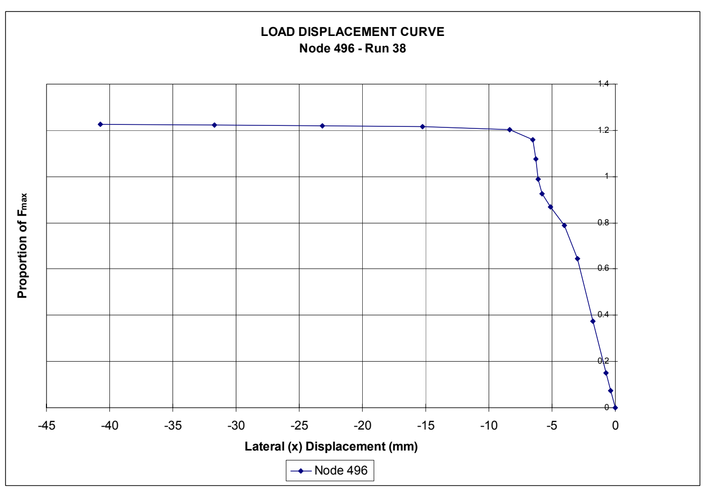


Figure 8.31 Shear Force vs Applied Load and Load Displacement for Residual Stress Analysis

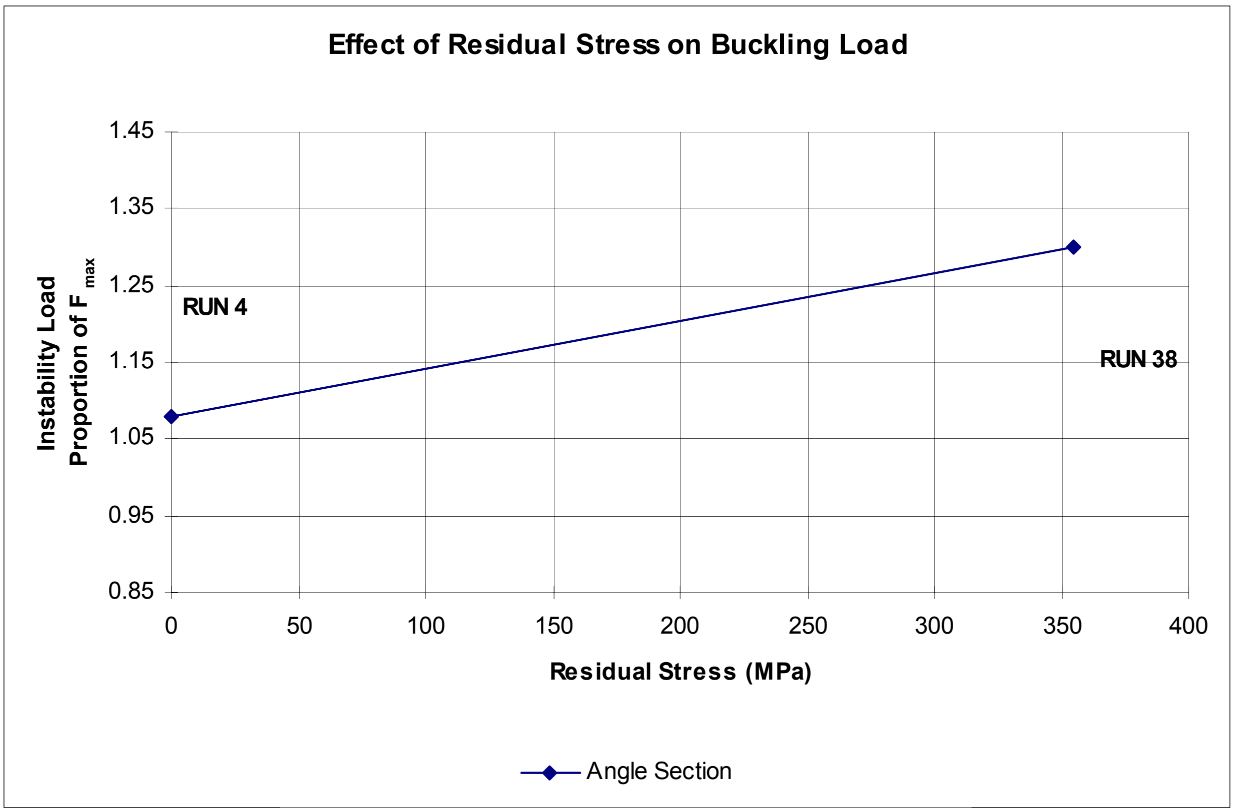


Figure 8.32 Buckling Load vs Residual Stress

8.4.10 Effect of Yield Stress on Stability

One of the influence parameters identified in section 6.1.2 as having the potential to influence the post-yield stability of stiffened panels subjected to ice loading is material yield stress. The previous investigations undertaken and the current investigation have shown that post-yield buckling load levels are affected by yielding and hence by yield stress. In the linear range, eigenvalue analysis of the stiffened panel defined as Run (4) indicated that no buckling occurred until well above the value of F_{max}. In fact, the buckling modes which dominated the response were not in the area of interest (i.e. the main frames) but in other areas of the structure.

When the nonlinear analysis Run (4) was conducted, which included a representative yield stress (355 MPa) for the material, the main frames buckled at a load level of 1.08 F_{max}. The material yield stress was then increased to 500 MPa Run (39). This resulted in an increase in the buckling load level to 1.30 F_{max}. This is shown on the plot of buckling load versus yield strength in Figure 8.33.

The FE models used for these analyses were the angle sections which include an in-plane stress of 10 000 psi.

Since the same geometric model was used for each of the runs with the only change between the two models being the yield stress, the stability criterion (which does not include any effect of material yield stress) was identical for the two models. The Run (4) panels were designed according to the Equivalent Standards with a stability criterion of 0.77 and a strength criterion of 1.21. However, when the material yield stress was changed from 355 MPa to 500 MPa (a 41 percent increase), the strength criterion increased from 1.21 to 1.71 (also a 41 percent increase) for Run (39). The predicted buckling load level increased by about 20 percent corresponding to this change.

Following these analyses, the angle main frames with a lower stability criteria were reanalyzed with a high strength steel to determine the effect of stability criteria on stability with an increase in material yield strength.

The first set of analyses were with models without in-plane loads. The Run (2) model, with a stability criteria of 0.61, was modified for a material with a yield strength of 500 MPa. This increase resulted in an increase in the strength criteria to 1.81. The analysis of this model, Run (44), resulted in a buckling load level of 1.45 F_{max}. The Run (3) model, with a stability criterion of 0.50 was then modified to incorporate the high strength material. This increased the strength criteria to 1.71 and resulted in a buckling load of 1.42 F_{max} for Run (45).

The second set of analyses were for models with in-plane loads. The Run (50) and Run (49) models were modified for the higher strength materials to produce the models for Runs (47)

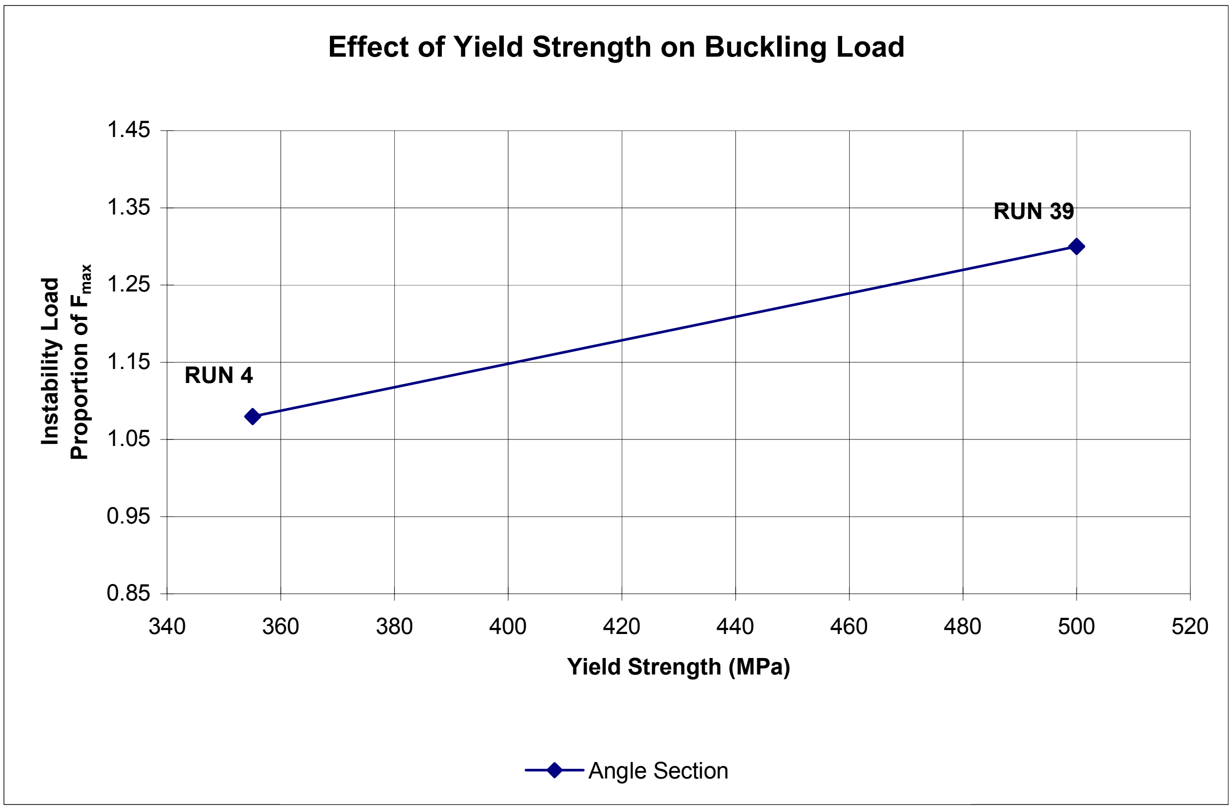


Figure 8.33 Buckling Load vs Yield Strength

and (46). The strength criteria increased to 1.81 for Run (47) and 1.71 for Run (46) with a predicted buckling load of 1.30 F_{max} for both models.

Both of these sets of analyses, with high strength steel, show the same trend as with the models with a yield strength of 355 MPa where the stability parameter was reduced (i.e. Runs (0, 2, and 3) and Runs (4, 50, and 49)). There is no clear relationship between stability and the stability parameter for angle main frames, regardless of material yield strength.

The overall conclusion drawn from the runs in this section is that main frame stability is heavily affected by the magnitude of the material yield stress. Material yielding has been found to cause a degradation of stability for angle main frames and causes buckling at lower load levels than what is predicted using linear predictions (i.e. without material yielding). The Equivalent Standards stability criterion does not include any effects of yielding and hence cannot account for the effects of different material yield strengths on stability. This is demonstrated in Appendix F where the corresponding Equivalent Standard equations are presented for angle section main frames. It is recommended that an allowance be incorporated into the Equivalent Standards to account for the effects of employing different material yield strengths on main frame stability.

Overall Summary

Figure 8.34 shows a plot of buckling load versus stability criteria for all of the runs conducted in the parametric study pertaining to stability criteria.

From the figure, it is evident that there is no clear relationship between the Equivalent Standards stability parameters (span and web ratio) and the FE determined buckling load levels (i.e. the stability).

These results lead us to believe that the stability parameters which work quite effectively to control stability below yield levels, are not the controlling parameters at load levels which cause a substantial amount of yielding. That is, the stability parameters are ineffective in controlling post-yield stability.

In-plane plate stresses were found to have a major effect on the stability predicted from the FE analyses. The results of the parametric study showed a clear relationship between in-plane load and stability for angles and tees and between in-plane load and load carrying capability for flat bars. However, the Equivalent Standards stability equations have no explicit allowance for in-plane loads. It is recommended that the equations be modified to provide such an allowance.

The parametric study shows that distortion and/or residual stresses at allowable and/or anticipated levels were found to have little effect on stability.

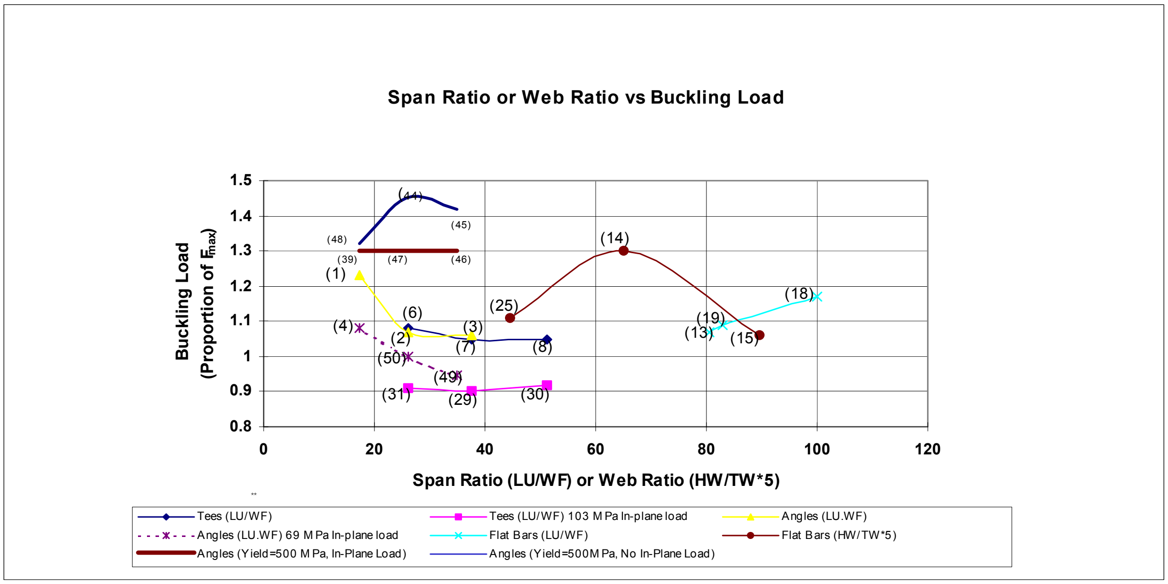


Figure 8.34 Summary of Plot Buckling vs Stability Criteria

Varying tilt angle was found to affect stability. For tees and angles, stability decreased with increasing the tilt angle (of the main frame web to a normal to the shell plating). The stability equations in the Equivalent Standards make some allowance for this for tilt angles in excess of 15°. For tee sections, a decrease in buckling load level of 17 percent was achieved by tilting the frame by 30°. This is out of proportion to the corresponding decrease in the stability criteria of only 6 percent.

Variation in yield strength was found to have a major impact on stability. Increasing the material yield strength decreases the amount of yielding present at a given ice load. It was found that this has a major impact on the predicted buckling load levels and stability was found to increase with increasing material yield strength. There is no allowance in the Equivalent Standards to account for this effect in the stability rules.

9. DESIGN CRITERIA

It has been found from the results of this study that two stability influences significantly affect the plastic stability of icebreaking ship structures subjected to ice loads. These influences are in-plane stresses and material yield strength which are not presently accounted for in the Equivalent Standards stability design criteria.

The present stability rules in Equivalent Standards are based on the definition of two geometric factors: span ratio (LU/WF), and web ratio (HW/TW). Fleet Technology published a report [29] in 1988 that provides details on the determination of the equations that define the stability criteria in Equivalent Standards. This report indicates that the current equations are based upon elastic models with no inclusion for the effects of in-plane stresses from the overall ship response. It does include a correction for lateral load effects. The authors recognized the need to incorporate the effect of plasticity and a more detailed understanding of the effects of lateral loads into the rules, and as such, recommended further study in these areas.

In the Martec proposal [9], it was proposed to modify the existing stability rules with new terms that include the effects of factors and influences (i.e. in–plane loads, etc.) that were found to have a significant effect on plastic stability. Unfortunately, the work carried out in this study has shown that there is no clear relationship between the Equivalent Standards stability criteria (based on span/web ratios) and the FE determined buckling loads. While the existing Equivalent Standards rules have been found to effectively work for elastic stability design, it may be too difficult to modify these rules for plastic stability design. This is because any additional terms in the equations (as they now exist) would only be modifying the span and web ratios, and it has been determined that post-yield stability is unaffected by changes in these ratios. It may be possible to modify the existing equations such that the plastic stability does not depend entirely upon the span or web ratio. However, it is likely to be more effective to define a complete new set of rules for plastic stability design and leave the existing rules in place for elastic considerations.

It was found that the post-yield buckling load level is very much influenced by the load level at which yielding occurs in the structure. In a structure, yielding produces effective changes in geometry that alter the stiffness of that structure. It is known that the stability of a structure is controlled by its stiffness. Therefore, yielding can have a direct influence on the stability characteristics. The Equivalent Standards stability criteria does not account for this influence.

It is also known that material yielding (and the associated geometric changes) directly affects the strength of a structure. Since the material yield strength effects the calculation of the strength criteria in the Equivalent Standards, it is feasible to use the strength criteria to control stability under the influence of yielding.

The analyses carried out in this study confirmed that the buckling load level is dependent upon the Equivalent Standards strength criteria as calculated for each of the designed panels. The strength criterion defines the material yield point that, in turn, affects the load level at which yielding occurs. Based on this, it may be possible to use the existing Equivalent Standards strength criteria to modify the stability criteria to control plastic stability. This would require definition of a methodology to include the effects of plasticity in the calculation of the stability criterion for main frame sections.

As discovered in the Phase I work, and verified in the Phase II test, flat bar main frame sections do not become unstable. This is because the stiffness changes resulting from yielding have a beneficial effect on the main frame stability. Under the applied lateral load, the yielding in the main frame flat bar progresses from the free edge to the intersection of the frame with the hull plating. This yielding produces an effective change in geometry that decreases the web ratio (HW/TW) due to yielding in the outer frame fibers rendering this material ineffective in carrying incremental load. In effect, the section becomes "more stubby" under increased load which increases the stability of the flat bar main frame in the post-yield condition. Therefore, in flat bar main frame sections, the effective geometric changes resulting from plasticity have a beneficial effect on their post-yield buckling characteristics.

In tee and angle section main frames, the application of the applied load, the asymmetry of the structure, and imperfections in the structure produce an overall forward or aft rotation in the web/flange assembly. This rotation produces material yielding in the flange that is not symmetric forward and aft of the main frame web. This results in sections that are more susceptible to instability under the influence of plasticity. Therefore, in angle and tee main frame sections, the geometric changes resulting from plasticity have a detrimental effect on their post-yield buckling characteristics.

These findings are important and have to be included in any considerations of changes to the stability criteria in the Equivalent Standards for the purposes of controlling plastic stability. Completion of this work is beyond the scope of the present project and further study is recommended.

10. CONCLUSIONS AND RECOMMENDATIONS

The present stability criteria provided in the Equivalent Standards was found to be ineffective in controlling post-yield stability. This was determined by designing panels according to the Equivalent Standards, using FE analysis to subject the panels to an ice load of magnitude F_{max}, and determining the post-yield buckling load. Geometries were varied to modify the stability criteria to include overdesigned and underdesigned panels (with respect to a panel designed exactly at the limit of the Equivalent Standards stability rules). The results of the study showed that, in general, as long as the section properties (moment of inertia and area) were maintained, virtually no change occurred in the buckling load level between panel designs with varying stability criteria.

All of the initial (Equivalent Standards conforming) panel designs were found to be stable up to a load level that exceeded F_{max}. This may have led to the conclusion that the Equivalent Standards stability criteria effectively controls the stability subsequent to yielding and results in panel designs which are found, on the most part, to have sufficient stability. However, panel designs which had the stability criteria decreased showed virtually no change in stability, and panel designs for which the stability criteria increased also showed no difference in stability. Therefore, even though each of the designed panels remained stable at F_{max}, their stability was not related to adherence to the Equivalent Standards stability criterion.

It has been found that the parameters that have the greatest effect on post-yield stability are: in-plane stresses, material yield stress, and tilt angle.

The results of the parametric study showed a clear relationship between in-plane load and the stability of angles and tees. However, the Equivalent Standards stability equations have no explicit allowance for in-plane loads.

It was also found that increasing the material yield strength decreased the amount of yielding present at a given ice load. This has a major impact on the predicted buckling load levels since stability was found to increase with increasing material yield strength. There is no allowance in the Equivalent Standards to account for this effect in the stability rules.

With respect to main frame tilt angles for tees and angles, stability decreased with increasing the tilt angle (of the web to a normal to the shell plating). The stability equations in the Equivalent Standards make some allowance for this for tilt angles in excess of 15°. For tee sections, a decrease in buckling load level of 17 percent was achieved by tilting the frame by 30°. This is out of proportion to the corresponding decrease in the stability criteria of only 6 percent.

Based upon these findings, the stability equations in the Equivalent Standards should be modified to account for the effect that in-plane stresses and yield strength have on stability (as per Section 8). While the current equations may effectively control elastic stability they cannot be used to control post-yield stability. As identified in Section 9, the present Equivalent Standards stability equations are defined entirely by two geometric factors: web ratio and span ratio. Since it has been found that post-yield stability is not influenced directly by either web or span ratios, it is concluded that these equations cannot be effectively modified to account for in-plane stresses and material yield strength.

Post-yield stability does, however, seem to be controlled by the strength of the structure. Therefore, it is felt that the present strength criteria in the Equivalent Standards could be used to design stiffened panel structures for plastic stability. Sufficient data was not collected during this study to define new guidelines or rules. It is recommended that further study be carried out to confirm that this is possible and to determine the Equivalent Standards design criteria for plastic stability. It is anticipated that the design criteria would include a component incorporating the existing Equivalent Standards strength criteria.

Rupture of the outer hull plating will most likely be the ultimate failure mode for icebreaking ships subjected to high lateral pressure loads. However, it is not expected that the ice loads seen by the CAC class of ships will be large enough to produce rupture at or near F_{max}. With a very significant membrane strength component developed from the onset of large displacements and plasticity, the overall strength of the structure is not questioned. Even at high load levels of transverse load (F_{max}), plate membrane stresses are still relatively low. There is a great deal of reserve strength remaining in the plating even at these load levels. This reserve strength, in itself, does not reduce the need to control the damage produced by ice loads. Tripped/buckled main frames will have a significantly reduced moment of inertia, and thus a significantly reduced stiffness. This reduced stiffness now makes the section more vulnerable to elastic buckling under loading conditions other than the direct ice load. This load may produce elastic buckling in plastically deformed main frames.

Based upon this, it is recommended that further study be performed to limit the allowable plastic deformation of primary structure such that subsequent to the application of the ice load, the structure does not become susceptible to elastic buckling failure under normal loading conditions.

In conclusion it is felt that the results presented in this report on the post-yield response of flat bar, angle, and tee main frame sections are accurate. However, the testing performed to date to validate the response predicted by the FE analysis has primarily concentrated on flat bar main frames which were confirmed to remain stable subsequent to yielding, even up to load levels which caused plate rupture. An experimental program is currently ongoing for the “Juniper” panel where tests have been performed on laterally loaded stiffened panels employing angle and tee section main frames. These results are not yet published for comparison to the analytical results. It is recommended that the Juniper panel experimental results be used together with analytical (FE) results to validate the accuracy of the shear force difference method in predicting the onset of buckling in the panel.

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APPENDIX A

BENCHMARK FE ANALYSIS

A.1. INTRODUCTION

The initial requirement of the present work being carried out by Martec Limited to perform a parametric review of the post yield strength of icebreaker structures is to perform a benchmark analysis using the proposed numerical procedure. This analysis was required to confirm that the Finite Element (FE) code ADINA could be used on an HP800 workstation to accurately predict the response of the 1993 physical panel (subsequently referred to as the "MIL panel") test carried out by MIL Systems Engineering Ltd./Carleton University [1].

Martec Limited has previously performed an FE analysis of a model of the MIL panel using the FE program, ADINA [2]. This work [3] showed very good agreement with the test results. Martec proposed to repeat this analysis since a new version of ADINA executing on a different computer platform would be used. Since ADINA is used for both analyses and the original analysis showed excellent agreement with the test results, the new benchmark problem results will be compared with the original ADINA analysis for accuracy.

A.2. OBJECTIVES

The objectives of the benchmark analysis are to verify that the FE code ADINA can accurately predict the post-yield response of the test panel, and to check the execution time and disk storage requirements on the HP9000 workstation for this problem.

A.3. ANALYSIS CODE AND PLATFORM

The FE analysis was performed using the program ADINA [4] Version 6.1. The platform used for the analysis was an HP9000 running HPUX UNIX Version 9.04. A 4 Gigabyte SCSI II hard drive was used for storage and analysis requirements.

A.4. FE ANALYSIS RESULTS

The ADINA FE model used in the nonlinear analysis of the physical panel is shown in Figure A.1 along with the boundary conditions designation codes. The "A" on all boundary nodes defines a boundary condition where all six degrees-of-freedom are fixed. The applied pressure load for the analysis is shown in Figure A.2. The initial load vector for the analysis consisted of a constant pressure of 10 MPa at all jack locations. This is equivalent to a total applied force of 3000 KN.

A.4.1 Nonlinear Analysis Results

The nonlinear analysis of the physical panel was carried out using the load displacement control (LDC) solution method of ADINA. In this procedure, the load vector and an initial nodal displacement (in the direction of the deflection of the structure at that node) is used to start the solution procedure. The initial displacement was in the +z direction and located at the midspan node (1076) of the most central main frame where the frame intersects with the plate. The magnitude of the initial displacement was equal to 1/10 of the magnitude of the maximum linear displacement at node 1076. The LDC method has its own equilibrium iteration method that uses an energy tolerance convergence criteria. This procedure is equivalent to full Newton equilibrium iterations. Stiffness reformation is carried out for each equilibrium iteration.

In the first load step, equilibrium is established if a solution is successfully obtained using the fraction of the load vector that produces the initial nodal displacement. Since this is a nonlinear analysis, this can only be achieved if the equilibrium iteration criteria is met. In this particular panel analysis, the initial displacement of 1/10 of the linear results (from reference [2]) produced a solution. Otherwise, a smaller initial displacement would have been required.

For the second and subsequent load steps, the LDC method automatically determines the next load/displacement increment, and tries to establish equilibrium at each load step. The LDC solution stops if either the maximum number of load steps specified in the input file has been reached, the maximum specified displacement at the node where the initial displacement was applied is reached, or if the solution fails to converge. If after the first iteration the solution does not converge, the LDC method automatically reduces the load/displacement increment for the second iteration, and so on.

As specified in Section 1, to verify the accuracy of this analysis the ADINA results will be compared with the original ADINA analysis of the test panel [3]. This comparison is performed with curves plotting displacement versus the applied load at all vertical and lateral LVDT locations. The LVDT locations on the panel are shown in Figure A.3.

The load-displacement curves of the original ADINA nonlinear analysis are shown in Figures A.4 and A.5. The load-displacement curves for the new ADINA analysis are shown in Figures A.6 and A.7. Agreement between the two FE analyses is virtually exact up to the maximum load (8590 KN). A cursory look at the two sets of curves shows a small difference in the presented results between loads of 1900 and 3500 KN. The reason for this difference is that no intermediate results are plotted between 1900 and 3500 KN in the original analysis. Therefore, the results of the original ADINA analysis appear linear between 1900 and 3500 KN. The more correct response is as shown in the new curves.

A.4.2 FEA Execution Times

The Martec proposal predicted the number of nonlinear FE analyses that could be undertaken in this study based upon an estimated execution time of 1.5 hours per time step. One of the objectives of the benchmark problem was to verify this estimate using the proposed software and computer platform. The execution times for various linear and nonlinear analyses running ADINA on the HP800 workstation are shown in Table A.1.

TABLE A.1: FEA Execution Times

	Linear		Nonlinear	
Description/Configuration	CPU	Clock	CPU	Clock
Single HP800 Estimated Times		1.5 Hrs*		45 Hrs*
Single HP800 (4 GIG SCSI II Drive)	0.5 Hr	.75 Hr	34 Hrs	51 Hrs
2 Concurrent HP800 (4 GIG SCSI II)			45 Hrs*	70 Hrs*

*Estimated Times

These analyses are based upon a 45,000 degree-of-freedom model executed up to an approximate load magnitude of F_{max}. For the times shown above, approximately 20 time steps were performed with 70 complete stiffness reformation steps.

As shown in this table, the actual execution times (0.75 hr.) for linear analyses (which corresponds to one stiffness reformation in one time step) is smaller than the predicted value (1.5 hr.). The actual times (51 hrs.) for the nonlinear analysis is very close to the predicted value of 45 hours. The reason for a higher "actual" than "predicted" nonlinear analysis time, when the "actual" linear analysis time is smaller than the "predicted" time, is that more stiffness reformations were carried out than predicted. This is attributed to numerous instabilities (main frame tripping outside the centre bay) in the panel. This may be considered the "worse case" condition for the number of stiffness reformation steps in the nonlinear analyses to be carried out in this study.

Based upon these numbers, execution times are within the range of those predicted in the proposal and, therefore, will not be a controlling factor in the number of nonlinear FE analyses that can be performed in this study.

A.5. CONCLUSIONS

The results of the benchmark analysis demonstrate that the FE program, ADINA, can reliably predict the post-yield response of the test panel, and that the analysis times are consistent with those presented in the Martec proposal.

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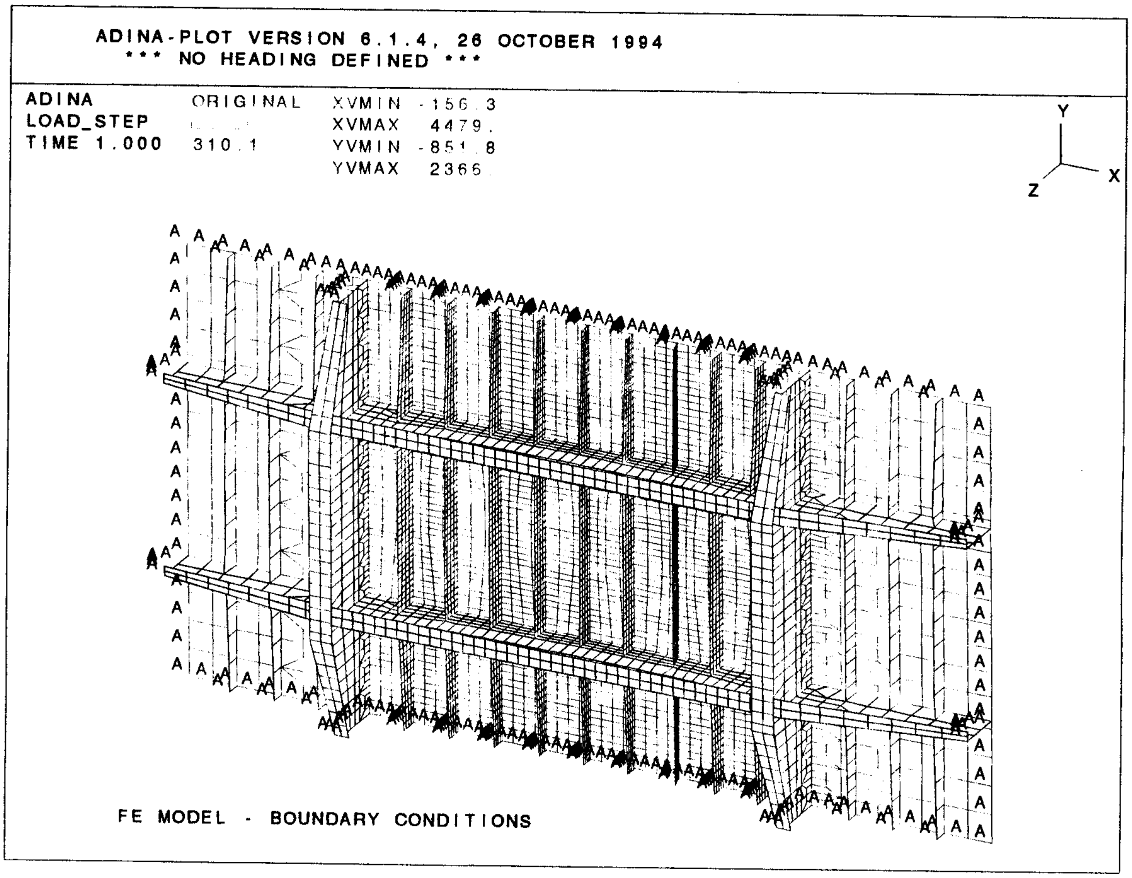


FIGURE A.1 ADINA FE Model of the Physical Panel

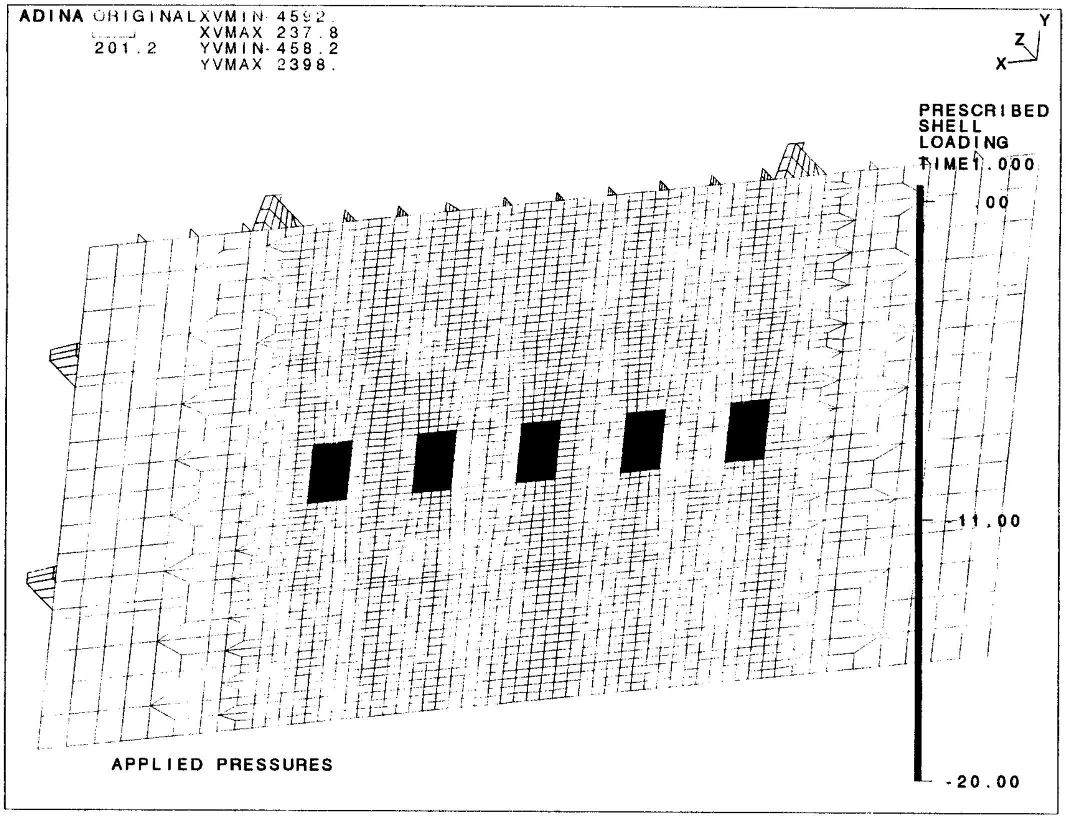


FIGURE A.2 ADINA FE Model Showing the Applied Pressure Load

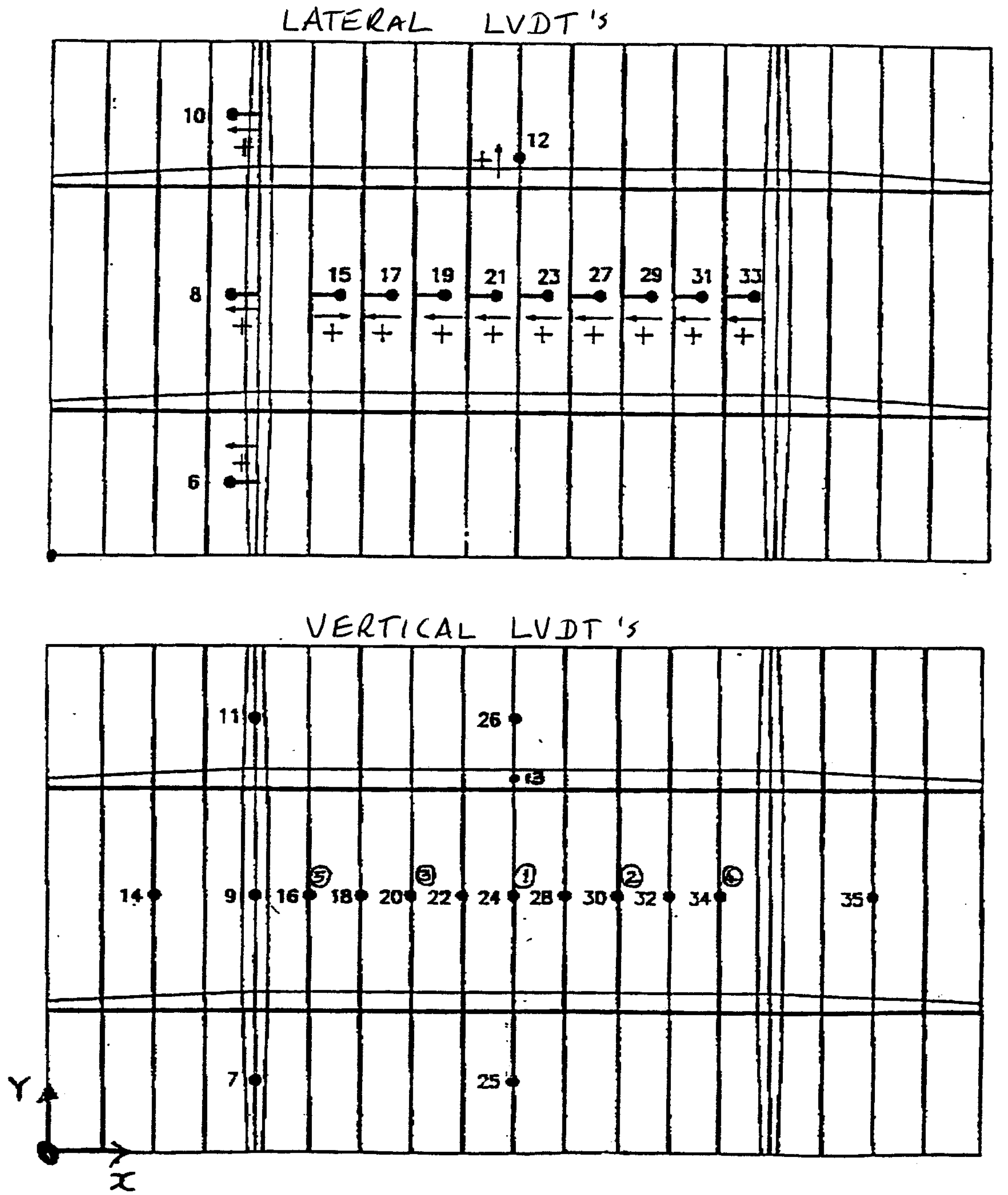


FIGURE A.3 LVDT Locations on the Test Panel

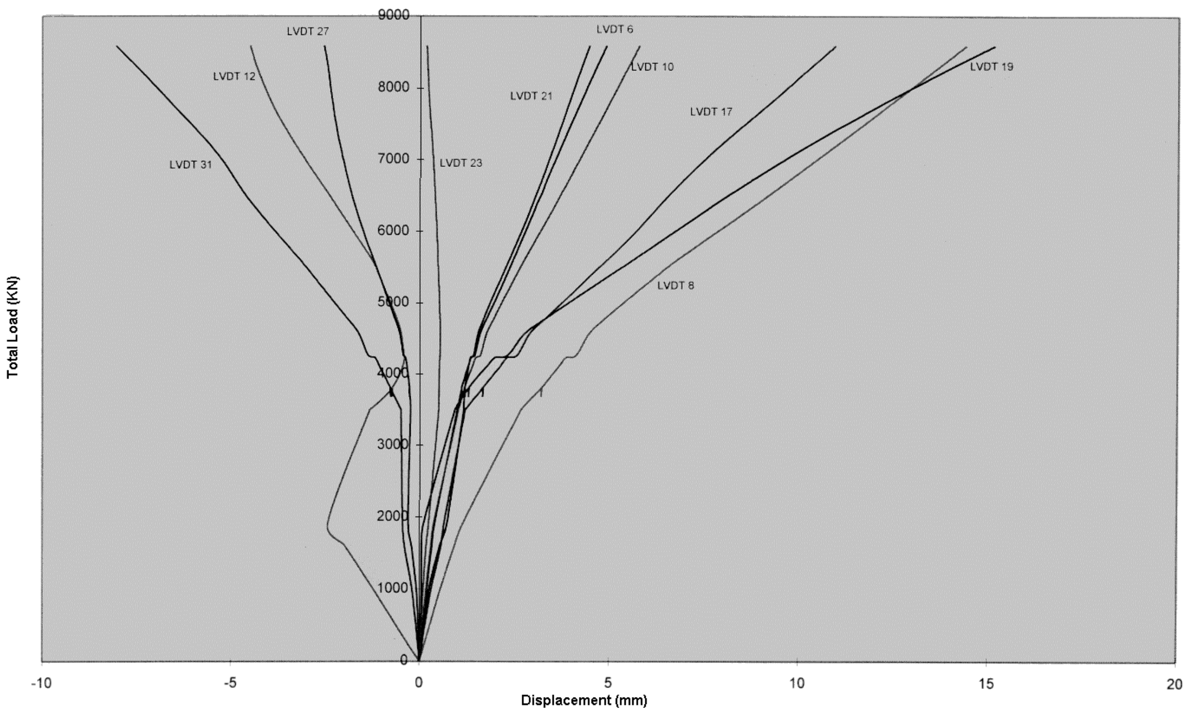


FIGURE A.4 Lateral LVDT Results from ADINA Analysis of the MIL Panel

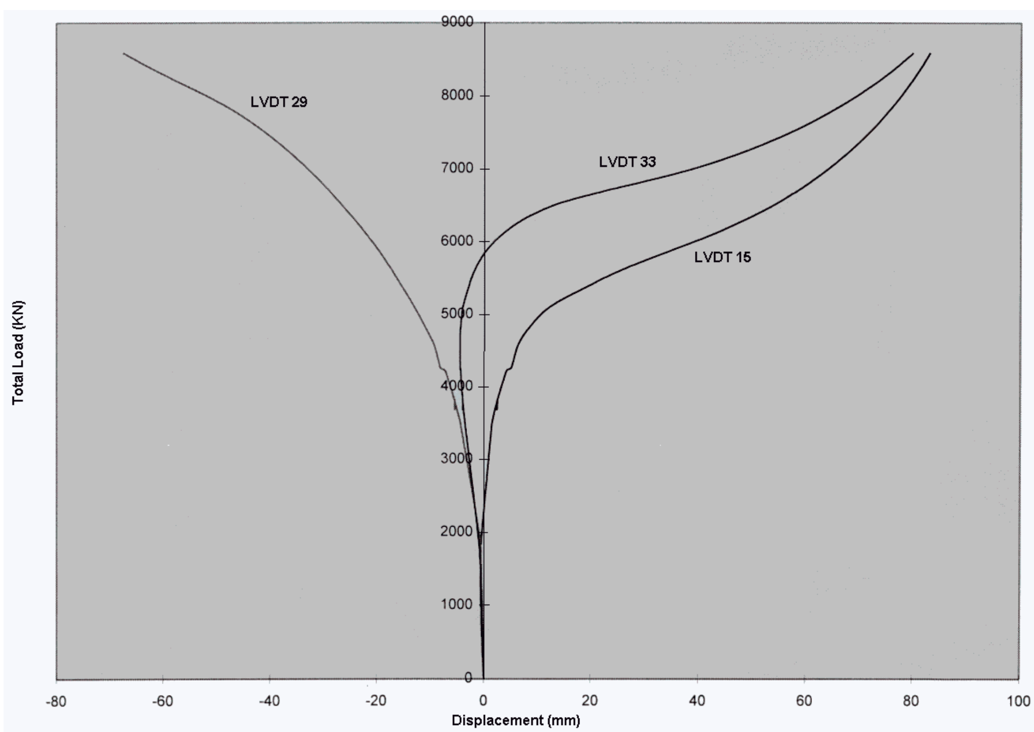


FIGURE A.4 (Cont'd) Lateral LVDT Results from ADINA Analysis of the MIL Panel

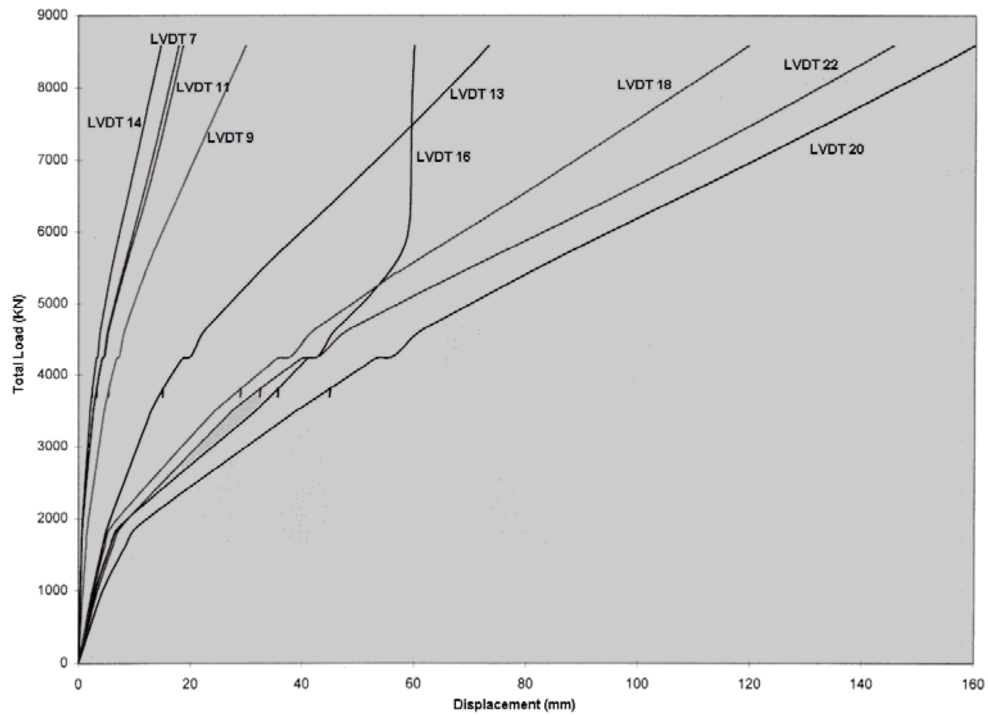


FIGURE A.5 Vertical LVDT Results from ADINA Analysis of the MIL Panel

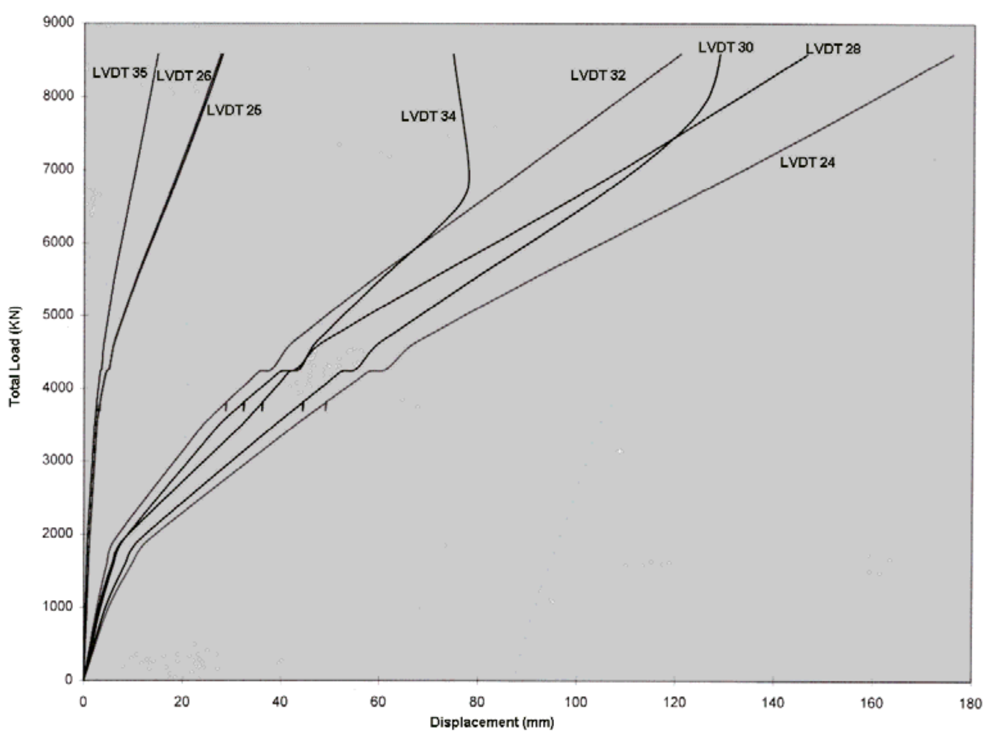


FIGURE A.5(Cont'd) Vertical LVDT Results from ADINA Analysis of the MIL Panel

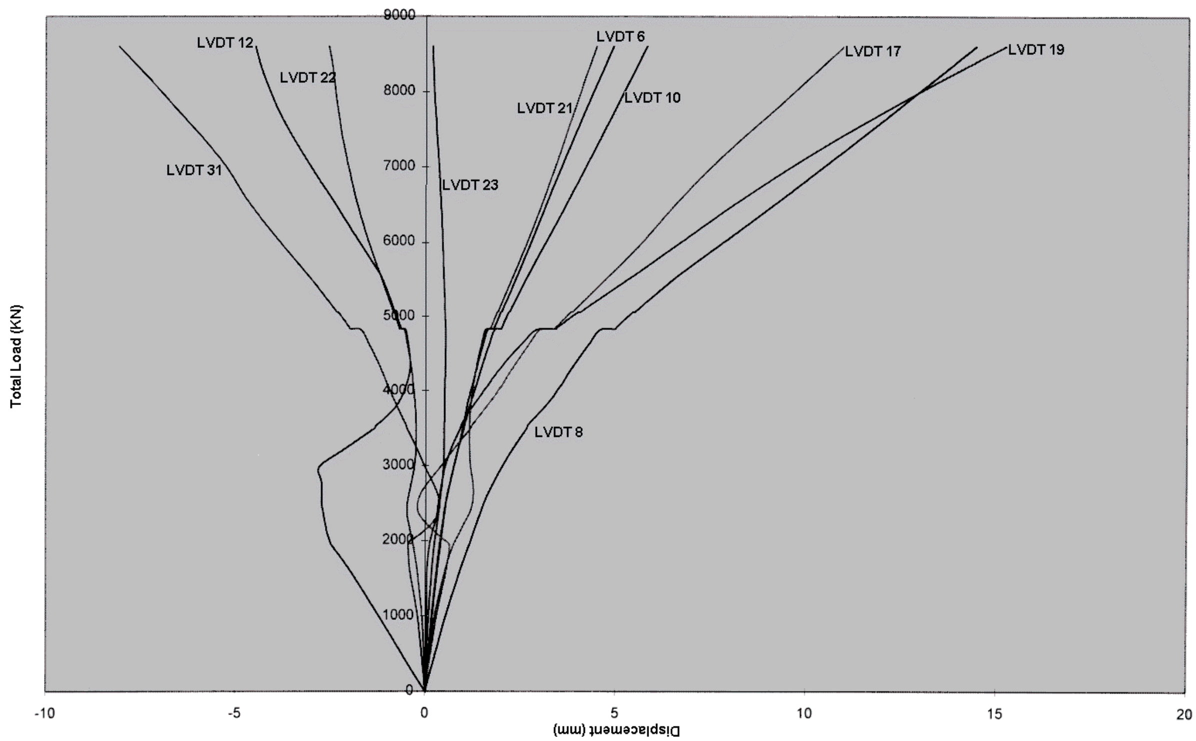


FIGURE A.6 New Lateral LVDT Results from ADINA Analysis of the MIL Panel

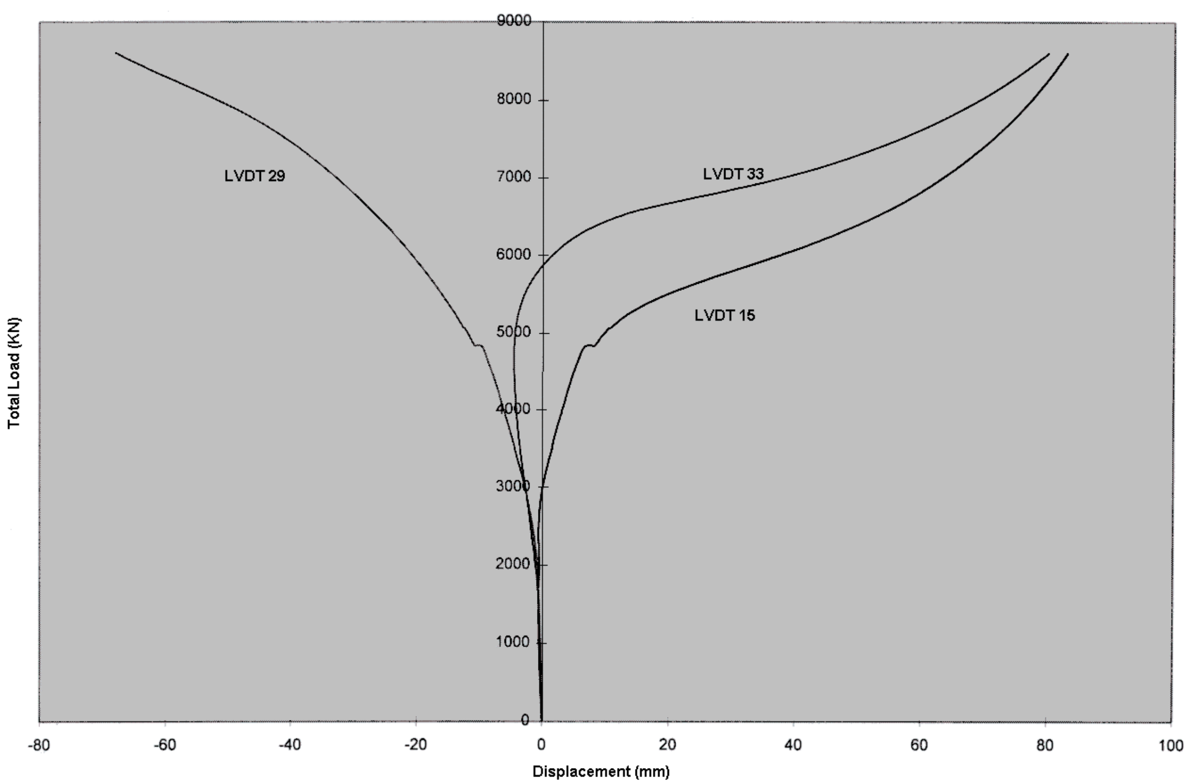


FIGURE A.6(Cont'd) New Lateral LVDT Results from ADINA Analysis of the MIL Panel

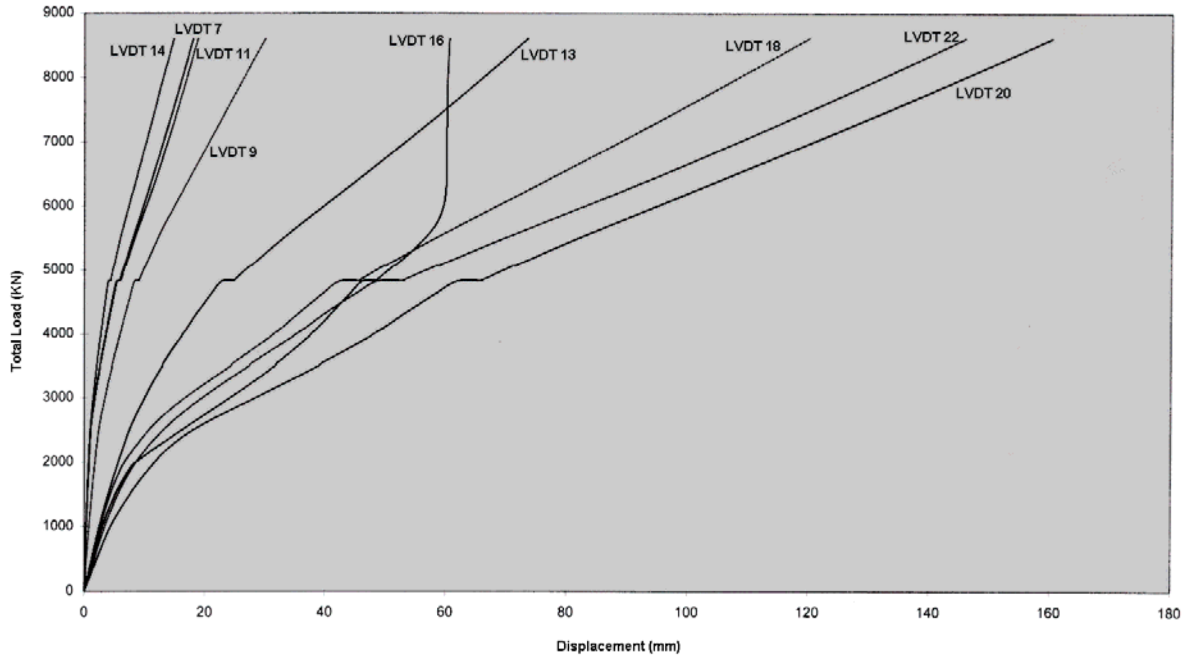


FIGURE A.7 New Vertical LVDT Results from ADINA Analysis of the MIL Panel

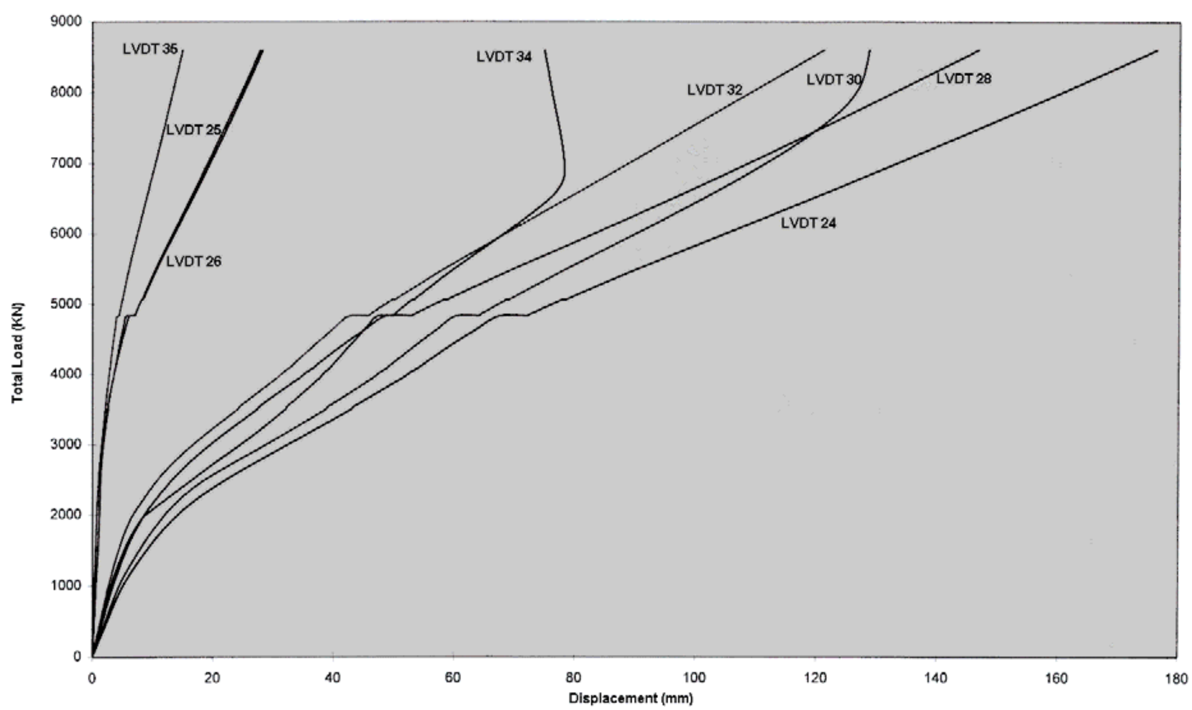


FIGURE A.7(Cont'd) New Vertical LVDT Results from ADINA Analysis of the MIL Panel

APPENDIX B

LITERATURE SEARCH ABSTRACTS

Paik, J.H. Ham and J.H. Ko, "A New Plate Buckling Design Formula (2nd Report) ¾ On the Plasticity Correction", Journal of the Society of Naval Architects of Japan, Volume 172, pp. 417-425, 1992.

In the previous paper, a new buckling design formula for simply supported plate panels subjected to combined in-plane and lateral loads was suggested. The effects of welding residual stress was also included. For the plasticity correction, the well-known Johnson-Ostenfeld formula was used.

In the present study, more advanced formula of the plasticity correction is proposed taking account of edge condition effects. The applicability of the proposed formula is demonstrated by comparing with the present and the conventional result.

Kmiecik, T. Jastrzebski and J. Kuznair, "Statistics of Ship Plating Distortions", Marine Structures, Volume 8, pp. 119-132, 1995.

The paper contains the results of many years of measurements of post-welding distortions of ship hull shell plating. Altogether 1998 plates of general cargo ships, multi-purpose tugs, bulk carriers, chemical carriers, tankers, research vessels as well as passenger/cargo ferries were examined. The dead-weight of the ships investigated was within the range of 1300 and 33350 tons. Results of statistical and regression analyses of the measurements are presented together with a brief description of the measurement techniques.

Ueda, S.M.H. Rashed, J.K. Paik, "Buckling and Ultimate Strength Interaction in Plates and Stiffened Panels Under Combined Inplane Biaxial and Shearing Forces", Marine Structures, Volume 8, pp. 1-36, 1995.

The main portion of a ship's structure is usually composed of stiffened plates. Between girders and floors, stiffeners are furnished to plates usually in the longitudinal direction. Under various loads applied to a ship, such as those due to cargo, buoyancy and waves, these stiffened plates are subjected to combined inplane and lateral loads. Imperfection due to fabrication exist mainly in the form of initial deflection and residual stresses. The behaviour of perfectly flat plates is, however, an important reference in design.

In this paper, buckling, ultimate and fully plastic strength interaction relationships for rectangular perfectly flat plates and uniaxially stiffened plates subjected to inplane biaxial and shearing forces are derived and expressed in explicit forms based on the results of theoretical investigations of the nonlinear behaviour of plates and stiffened plates.

The accuracy of these interaction relationships is confirmed through comparison with the results of other analysis methods.

With the aid of these interaction relationships, buckling load, ultimate strength and/or fully plastic strength of such perfectly flat plates and uniaxially stiffened plates subjected to inplane loads may be predicted by hand calculation.

Hu, "Numerical Study of Tripping Behaviour of Stiffened Plates", Second Canadian Marine Dynamics Conference, August 9-11, 1993.

Tripping of the stiffeners is considered a premature failure mode because of reduction in the usable strength of the stiffened plate. There are several sets of criteria in various ship design standards to exclude the tripping from possible failure modes. These criteria, either defining the maximum allowable distance between lateral supports in terms of span-to-slange width ratio or requiring the calculated torsional buckling stress to be less than a certain stress level, give inconsistent margins of safety. This paper reviews tripping criteria in various standards. Linear buckling analysis and nonlinear load-displacement analysis by using a finite element method were performed to investigate the efficiency of these criteria. The flat stiffened plates with different geometries, loadings and boundary conditions were analyzed. It is concluded that the current criteria may be inadequate for flat plates reinforced with standard Tee stiffeners.

Hu, "A Finite Element Assessment of the Buckling Strength Equations of Stiffened Plates", Ships Structures Symposium 1993, November 16-17, 1993.

The collapse of inplane loaded stiffeners in ship structures cause simultaneous buckling of adjacent plates. DMEM10 (Structural Design of Surface Warship, Canadian Forces) and NES110 (Naval Engineering Standard, UK MOD) evaluate the ultimate strength of a stiffened plate in a way that the ultimate load carrying capacity is obtained by iterating between the ultimate plate compressive strength curve and the column strength curve. Currently, the ultimate compressive plate strength is obtained based on Faulkner's effective width equation, while the combined stiffener and plate strength is evaluated by Bleich's parabola. The original derivation of the parabolic curve only takes the material inelasticity into account without considering imperfections. Smith et al. have derived sets of column strength curves for small, average and large imperfections based on finite element results. These results are presented in a data sheet format in SSCP23 (Design of Surface Ship Structures, UK MOD). A comparison between the ultimate strength of the conventional procedure and the design curves in SSCP23 shows substantial discrepancies. Finite element analyses, including the effects of imperfections and residual stresses, are employed to study these discrepancies. In order to provide alternatives in design procedures, some related provisions in civil structural and offshore construction standards are also examined.

Bedair and A. Sherbourne, "Unified Approach to Local Stability of Plate/Stiffener Assemblies", Journal of Engineering mechanics, Volume 121, No. 2, pp. 214-229, February 1995.

A semi-analytical approach for the computation of the local buckling of stiffened plates under and combination of applied biaxial compression, inplane bending and shear stress is presented. The plate is treated as partially restrained against rotation and inplane translation. In the first stage, the plate is treated as infinitely long and the buckling mode is idealised by straight lines with arbitrary parameters, The energy method is then used to formulate the buckling coefficient, K, in terms of general functions that describe the longitudinal and transverse displacement profiles. In the second stage, sequential quadratic programming (SQP) is used to find the critical combinations of the parameters in the idealised buckling mode that minimise the coefficient, K. Modification factors are then suggested to compute the buckling stress for plates of finite length. Using the derived formulations, a closed-form expression for the K factor is determined by choosing approximate displacement functions. Validation, accuracy and comparison of the derived K factor is shown for the limiting conditions of simply supported and clamped edges. Finally, results are presented for the more general case of a plate partially restrained against rotations. The transition from rotationally free to rotationally clamped boundaries is shown by modifying the stiffener torsional rigidities. The destabilising effect boundaries is shown by modifying the stiffener torsional rigidities. The destabilising effect of the lateral restraint on the buckling stress is also shown.

Lee, K.T. Chung and Y.T. Yang, "Geometrically Nonlinear Analysis of Eccentrically Stiffened Plates", selected papers of the Society of Naval Architects of Korea, Vol. 1, No. 1, pp. 91-100, 1993.

A displacement based finite element method is presented for the geometrical nonlinear analysis of eccentrically stiffened plates. A nonlinear degenerated shell element and a nonlinear degenerated eccentric isoparametric beam (isobeam) element are formulated on the basis of total Agrangian and updated Lagrangian descriptions. In the formulation of the isobeam element, some additional local degrees-of-freedom are implemented to describe the stiffener's local plate buckling modes. Therefore this element can be effectively employed to model the eccentric stiffener with fewer DOFs than the case of a degenerated shell element.

Some detailed buckling and nonlinear analyses of an eccentrically stiffened plate are performed to estimate the critical buckling loads and the post-buckling behaviour including the local plate buckling of the stiffeners discretized with the degenerated shell elements and the isobeam elements. The critical buckling load are found to be higher than the analytical plate buckling load but lower than Euler buckling load of the corresponding column, i.e., buckling strength requirements of the classification societies for the stiffened plates.

Jang and S.I. Seo, "A Simplified Approach to the Analysis of the Ultimate Compressive Strength of Welded Stiffened Plates", Trans. the Society of Naval Architects of Korea, Vol. 30, No. 2, pp. 141-154, 1993.

In this paper, a method to calculate the ultimate compressive strength of welded one-sided stiffened plates simply supported along all edges is proposed. At first initial imperfections such as distortions and residual stresses due to welding are predicted by using simplified methods. Then, the collapse modes of the stiffened plate are assumed and collapse loads for each mode are calculated. Among these loads, the lowest value is selected as the ultimate strength of the plate. Collapse modes are assumed as follows:

Overall buckling of the stiffened plate → Overall collapse due to stiffener bending;

Local buckling of the plate part → Local collapse of the plate part ® Overall collapse due to stiffener yielding;

Local buckling of the plate part → Overall collapse due to stiffener bending; and

Local buckling of the plate part → Local collapse of the plate part ® Overall collapse due to stiffener tripping.

The elastic large deflection analysis based on the Raleigh-Ritz method is carried out and plastic analysis assuming hinge lines is also carried out. Collapse load is defined as the cross-point of the two analysis curves. This method enables the ultimate strength to be calculated with small computing time and a good accuracy. Using the present method, characteristics of the stiffener including torsional rigidity, bending and tripping can also be clarified.

Ham and U.N. Kim, "Buckling Characteristics of Ship Bottom Plate ¾ On the Stiffener Restraint Effects", Trans. the Society of Naval Architects of Korea, Vol. 31, No. 4, pp. 130-138, 1994.

Bottom plates of empty hold are subjected to not only water pressure but also bi-axial inplane loads, especially in the alternate full loading condition of bulk carrier. This kind of plate behaviour is very difficult to be explained and to be estimated using common buckling design guide in the initial design stage of hull structure, therefore, some more concrete studies for this plate structure was performed based on the currently developed buckling estimation formula.

In this buckling formula, torsional stiffness effects of edge stiffener are included additionally and effects of elastic buckling strength of plate panel are treated as characteristic value problem. Also considering boundary stiffener effects and inplane and lateral loading, evaluation of bottom plate scantling using this formula, calculated results using various classification regulation of buckling strength and results of first report approach are compared *each other and useful guides using developed formula for bottom plate scantling design are discussed.*

Bedair and A.N. Sherbourne, "Plate-Stiffener Assemblies in Uniform Compression. Part I: Buckling", Journal of Engineering Mechanics, 10. 119(10), pp. 1937-55, 1993.

This study focuses on the influence of the rotational and in-plane translation restraints on the buckling analysis of stiffened plate elements. While the stiffeners were assumed to possess finite rotation capacity, three in-plane boundary conditions were distinguished. The details of the study are described, and the findings are presented. Numerical results found a rapid increase in the buckling load for relatively small amounts of rotational restraint. The study highlighted the importance of the in-plane restraint that the stiffeners offer to the attached plate whereby the buckling load may decrease by 30 percent.

Stanway, "Behaviur of a Web Plate in Shear with an Intermediate Stiffener", Proceedings of the Institution of Civil Engineers: Structures and Buildings, 08.99(3), pp. 327-344, 1993.

The behaviour of a simply supported plate with a single stiffener, subject to in-plane shear forces, is studied using elastic critical buckling analyses and non-linear elasto-plastic finite-element analyses. Particular attention is paid to factors relevant to design of the stiffener. Variations in stiffener rigidity, panel side ratio, panel slenderness, and imperfection shapes are considered. A primary conclusion for the panel slendernesses considered is that the stiffener behaviour depends primarily on moments caused by transverse shear forces in the panels on each side of the stiffener, and that the axial force in the stiffener has a relatively small effect. This contrasts with current design methods which are based primarily on axial forces resulting from tension field action. Stiffener requirements based on the analyses are compared with the requirements of current design codes, and rather large differences are found. Large differences between codes are also found. A subsequent paper will use the results presented in developing and validating a phenomenologically based model for stiffener design.

Danielson, "Analytical Tripping Loads for Stiffened Plates", Technical Report No. NPS-MA-94-006 ¾ Naval Postgraduate School, Dept. of Mathematics, Monterey, CA, December 1993.

The subject of this paper is the buckling behavior of a rectangular plate, with parallel thin-walled stiffeners attached to one side, subjected to a combination of axial compression, lateral pressure, and bending moment. The plate is modeled by the Von Karman plate equations and the stiffeners by a nonlinear beam theory recently derived. An analytical solution is obtained for the buckling load corresponding to a torsional tripping mode of the stiffeners. The effects of various boundary conditions, imperfections, and residual stress are included

APPENDIX C

PANEL DESIGN SPREADSHEETS AND STABILITY EQUATIONS BASED ON PROPOSED REVISIONS TO ASPPR, DECEMBER 1989 TP9981, REF [1]

DISPLACEMENT (KTonnes)	18.00	18.00	18.00	18.00	18.00	18.00	18.00	18.00
POWER (MW)	18.00	18.00	18.00	18.00	18.00	18.00	18.00	18.00
MATERIAL YIELD STRENGTH (MPa)	355.00	355.00	355.00	355.00	355.00	355.00	355.00	355.00
CAC (1,2,3, or 4)	3	3	3	3	3	3	3	3
HULL AREA (Bow or Midbody)	Bow	Bow	Bow	Bow	Bow	Bow	Bow	Bow
Arctic Class Factor	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6
Area Factor	1	1	1	1	1	1	1	1
Fmax	83.349	83.349	83.349	83.349	83.349	83.349	83.349	83.349
Vp	1.522	1.522	1.522	1.522	1.522	1.522	1.522	1.522
Hp	12.173	12.173	12.173	12.173	12.173	12.173	12.173	12.173
FRAME SPACING	400	500	600	650	700	800	900	1000
FRAME SPAN	2600	2600	2600	2600	2600	2600	2600	2600
SHELL PLATE DESIGN:								
Corrosion Allowance	0	0	0	0	0	0	0	0
U	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8
Omega (Degrees)	90	90	90	90	90	90	90	90
Frame Orientation Factor	1	1	1	1	1	1	1	1
Plate Design Pressur - Dpp (MPa)	9.864	7.891	6.576	6.070	5.637	4.932	4.384	3.946
MInimum Dpp (MPa)	5.750	4.600	3.833	3.538	3.286	2.875	2.556	2.300
Dpp used for Plate Thickness (MPa)	9.864	7.891	6.576	6.070	5.637	4.932	4.384	3.946
Minimum Shell Plate Thickness (mm)	28.87	32.28	35.36	36.80	38.19	40.83	43.31	45.65
Shell Plate Thickness (mm)	30.163	33.34	36.513	38.1	39.687	41.275	44.45	46.038
TRANSVERSE FRAME DESIGN:								
Type	Angle	Angle	Angle	Angle	Angle	Angle	Angle	Angle
Dimensions:								
Web Depth (mm)	350	370	400	400	420	450	460	500
Web Thickness (mm)	27.50	31.50	33.80	36.00	37.00	38.00	40.60	42.00
Flange Width (mm)	144	158.5	170	181	186	191	204	211
Flange Thickness (mm)	20	20	20	20	20	20	20	20
Phi (degrees)	90	90	90	90	90	90	90	90
REQUIRED VALUES:								
DPT	0.033	0.041	0.049	0.053	0.058	0.066	0.074	0.082
PAV	9.964	9.619	9.316	9.177	9.047	8.807	8.592	8.398
Span LB (meters)	2.600	2.600	2.600	2.600	2.600	2.600	2.600	2.600
Spacing Between Frames S (meters)	0.4	0.5	0.6	0.65	0.7	0.8	0.9	1
Vp / Span	0.585	0.585	0.585	0.585	0.585	0.585	0.585	0.585
Factor A (Sch 1 Table 3)	0.751	0.751	0.751	0.751	0.751	0.751	0.751	0.751
Value H	15000	15000	15000	15000	15000	15000	15000	15000
Value B	3.674	3.674	3.674	3.674	3.674	3.674	3.674	3.674
Req. Trans. Frame Shear Area (cm2)	95.89	115.71	134.48	143.52	152.36	169.51	186.05	202.06
Req. Trans. Frame Plas. Modu. (cm3)	2225.63	2685.77	3121.29	3331.19	3536.45	3934.59	4318.37	4689.91
MINIMUM VALUES:								
Hp min	6.00	6.00	6.00	6.00	6.00	6.00	6.00	6.00
Vp min	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75
Vp min / Span	0.288	0.288	0.288	0.288	0.288	0.288	0.288	0.288
Factor A (Sch 1 Table 3)	0.873	0.873	0.873	0.873	0.873	0.873	0.873	0.873
DPT	0.067	0.083	0.100	0.108	0.117	0.133	0.150	0.167
PAV	8.781	8.372	8.036	7.890	7.755	7.515	7.307	7.124
Factor C	1	1	1	1	1	1	1	1
Factor B	2.034	2.034	2.034	2.034	2.034	2.034	2.034	2.034
Min. Trans. Frame Shear Area (cm2)	80.68	96.14	110.75	117.79	124.68	138.09	151.06	163.64
Min. Trans. Frame Plas. Modu. (cm3)	1809.42	2156.36	2483.81	2641.75	2796.40	3097.13	3387.93	3670.09
ACTUAL VALUES:								
Factor M	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Actual Shear Area (cm2)	96.25	116.55	135.20	144.00	155.40	171.00	186.76	210.00
Actual Plastic Modulus (cm3)	2909.77	3607.91	4406.90	4707.48	5245.19	6036.44	6718.83	7982.74

STRENGTH CRITERIA:								
Actual/Required (Shear Area)	1.004	1.007	1.005	1.003	1.020	1.009	1.004	1.039
Actual/Required (Plastic Modulus)	1.307	1.343	1.412	1.413	1.483	1.534	1.556	1.702
LOCAL BUCKLING CRITERIA:								
Flange Width > 5 X Web Thick (8.(1))	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE
Local Web Buckling (8.(2))								
- Tee or Angle - Requirements	53.074	53.074	53.074	53.074	53.074	53.074	53.074	53.074
- Tee or Angle - HW / TW	12.727	11.746	11.834	11.111	11.351	11.842	11.330	11.905
- Ratio (Required / Actual)	4.170	4.519	4.485	4.777	4.676	4.482	4.684	4.458
- Are Web Stiffeners Required?	NO	NO	NO	NO	NO	NO	NO	NO
Local Web Buckling (8.(3))								
- Flat Bar - Requirements	NA	NA	NA	NA	NA	NA	NA	NA
- Flat Bar - HW / TW	NA	NA	NA	NA	NA	NA	NA	NA
- Ratio (Required / Actual)	NA	NA	NA	NA	NA	NA	NA	NA
- Are Web Stiffeners Required?	NA	NA	NA	NA	NA	NA	NA	NA
Flange Outstand (8.(4))								
- Requirements	8.2265	8.2265	8.2265	8.2265	8.2265	8.2265	8.2265	8.2265
- Tee or Angle - FOS / TF	5.825	6.35	6.81	7.25	7.45	7.65	8.17	8.45
- Ratio (Required / Actual)	1.4123	1.2955	1.2080	1.1347	1.1042	1.0754	1.0069	0.9736
TRIPPING CRITERIA:								
FPM	2225.63	2685.77	3121.29	3331.19	3536.45	3934.59	4318.37	4689.91
AFPM	2909.77	3607.91	4406.90	4707.48	5245.19	6036.44	6718.83	7982.74
Delta (degrees)	86.42	86.42	86.42	86.42	86.42	86.42	86.42	86.42
N	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
V	16.478	16.256	15.857	15.850	15.471	15.212	15.105	14.442
Tee:								
(i)	NA	NA	NA	NA	NA	NA	NA	NA
(ii)	NA	NA	NA	NA	NA	NA	NA	NA
(iii)	NA	NA	NA	NA	NA	NA	NA	NA
Angle	1.0083	1.1250	1.2370	1.3177	1.3872	1.4488	1.5583	1.6858
Flat Bar:								
(i)	NA	NA	NA	NA	NA	NA	NA	NA
(ii)	NA	NA	NA	NA	NA	NA	NA	NA
(iii)	NA	NA	NA	NA	NA	NA	NA	NA
Web Thks < Shell Thks (Sch.1 Pg.16)	YES	YES	YES	YES	YES	YES	YES	YES
SHEAR AREA SATISFIED	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE
PLASTIC MODULUS SATISFIED	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE
TRIPPING SATISFIED	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE
Plate Ratio	13.26	15.00	16.43	17.06	17.64	19.38	20.25	21.72
Span Ratio	18.06	16.40	15.29	14.36	13.98	13.61	12.75	12.32
Web Ratio	12.73	11.75	11.83	11.11	11.35	11.84	11.33	11.90

DISPLACEMENT (KTonnes)	18.00	18.00	18.00	18.00	18.00	18.00	18.00	18.00
POWER (MW)	18.00	18.00	18.00	18.00	18.00	18.00	18.00	18.00
MATERIAL YIELD STRENGTH (MPa)	355.00	355.00	355.00	355.00	355.00	355.00	355.00	355.00
CAC (1,2,3, or 4)	3	3	3	3	3	3	3	3
HULL AREA (Bow or Midbody)	Midbody	Midbody	Midbody	Midbody	Midbody	Midbody	Midbody	Midbody
Arctic Class Factor	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6
Area Factor	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5
Fmax	83.349	83.349	83.349	83.349	83.349	83.349	83.349	83.349
Vp	1.522	1.522	1.522	1.522	1.522	1.522	1.522	1.522
Hp	12.173	12.173	12.173	12.173	12.173	12.173	12.173	12.173
FRAME SPACING	400	500	600	650	700	800	900	1000
FRAME SPAN	2600	2600	2600	2600	2600	2600	2600	2600
SHELL PLATE DESIGN:								
Corrosion Allowance	0	0	0	0	0	0	0	0
U	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5
Omega (Degrees)	90	90	90	90	90	90	90	90
Frame Orientation Factor	1	1	1	1	1	1	1	1
Plate Design Pressur - Dpp (MPa)	4.932	3.946	3.288	3.035	2.818	2.466	2.192	1.973
MInimum Dpp (MPa)	5.750	4.600	3.833	3.538	3.286	2.875	2.556	2.300
Dpp used for Plate Thickness (MPa)	5.750	4.600	3.833	3.538	3.286	2.875	2.556	2.300
Minimum Shell Plate Thickness (mm)	22.04	24.65	27.00	28.10	29.16	31.17	33.07	34.85
Shell Plate Thickness (mm)	22.23	25.4	28.58	28.58	30.163	31.75	33.34	34.93
TRANSVERSE FRAME DESIGN:								
Type	Angle	Angle	Angle	Angle	Angle	Angle	Angle	Angle
Dimensions:								
Web Depth (mm)	300	300	300	313	300	300	312	330
Web Thickness (mm)	20.00	20.00	22.50	23.00	26.00	29.00	30.00	30.00
Flange Width (mm)	129	140	143	147	147	150	155	160
Flange Thickness (mm)	20	20	20	20	20	20	20	20
Phi (degrees)	90	90	90	90	90	90	90	90
REQUIRED VALUES:								
DPT	0.033	0.041	0.049	0.053	0.058	0.066	0.074	0.082
PAV	9.964	9.619	9.316	9.177	9.047	8.807	8.592	8.398
Span LB (meters)	2.600	2.600	2.600	2.600	2.600	2.600	2.600	2.600
Spacing Between Frames S (meters)	0.4	0.5	0.6	0.65	0.7	0.8	0.9	1
Vp / Span	0.585	0.585	0.585	0.585	0.585	0.585	0.585	0.585
Factor A (Sch 1 Table 3)	0.751	0.751	0.751	0.751	0.751	0.751	0.751	0.751
Value H	15000	15000	15000	15000	15000	15000	15000	15000
Value B	3.674	3.674	3.674	3.674	3.674	3.674	3.674	3.674
Req. Trans. Frame Shear Area (cm2)	47.94	57.86	67.24	71.76	76.18	84.76	93.02	101.03
Req. Trans. Frame Plas. Modu. (cm3)	1112.82	1342.89	1560.65	1665.59	1768.22	1967.29	2159.18	2344.95
MINIMUM VALUES:								
Hp min	6.00	6.00	6.00	6.00	6.00	6.00	6.00	6.00
Vp min	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75
Vp min / Span	0.288	0.288	0.288	0.288	0.288	0.288	0.288	0.288
Factor A (Sch 1 Table 3)	0.873	0.873	0.873	0.873	0.873	0.873	0.873	0.873
DPT	0.067	0.083	0.100	0.108	0.117	0.133	0.150	0.167
PAV	8.781	8.372	8.036	7.890	7.755	7.515	7.307	7.124
Factor C	1	1	1	1	1	1	1	1
Factor B	2.034	2.034	2.034	2.034	2.034	2.034	2.034	2.034
Min. Trans. Frame Shear Area (cm2)	40.34	48.07	55.37	58.89	62.34	69.05	75.53	81.82
Min. Trans. Frame Plas. Modu. (cm3)	904.71	1078.18	1241.91	1320.88	1398.20	1548.56	1693.97	1835.05
ACTUAL VALUES:								
Factor M	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Actual Shear Area (cm2)	60.00	60.00	67.50	71.99	78.00	87.00	93.60	99.00
Actual Plastic Modulus (cm3)	1795.17	1879.76	2036.43	2221.15	2243.38	2420.74	2666.07	2950.29

STRENGTH CRITERIA:								
Actual/Required (Shear Area)	1.251	1.037	1.004	1.003	1.024	1.026	1.006	0.980
Actual/Required (Plastic Modulus)	1.613	1.400	1.305	1.334	1.269	1.230	1.235	1.258
LOCAL BUCKLING CRITERIA:								
Flange Width > 5 X Web Thick (8.(1))	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE
Local Web Buckling (8.(2))								
- Tee or Angle - Requirements	53.074	53.074	53.074	53.074	53.074	53.074	53.074	53.074
- Tee or Angle - HW / TW	15.000	15.000	13.333	13.609	11.538	10.345	10.400	11.000
- Ratio (Required / Actual)	3.538	3.538	3.981	3.900	4.600	5.131	5.103	4.825
- Are Web Stiffeners Required?	NO	NO	NO	NO	NO	NO	NO	NO
Local Web Buckling (8.(3))								
- Flat Bar - Requirements	NA	NA	NA	NA	NA	NA	NA	NA
- Flat Bar - HW / TW	NA	NA	NA	NA	NA	NA	NA	NA
- Ratio (Required / Actual)	NA	NA	NA	NA	NA	NA	NA	NA
- Are Web Stiffeners Required?	NA	NA	NA	NA	NA	NA	NA	NA
Flange Outstand (8.(4))								
- Requirements	8.2265	8.2265	8.2265	8.2265	8.2265	8.2265	8.2265	8.2265
- Tee or Angle - FOS / TF	5.45	6	6.025	6.2	6.05	6.05	6.25	6.5
- Ratio (Required / Actual)	1.5095	1.3711	1.3654	1.3269	1.3598	1.3598	1.3162	1.2656
TRIPPING CRITERIA:								
FPM	1112.82	1342.89	1560.65	1665.59	1768.22	1967.29	2159.18	2344.95
AFPM	1795.17	1879.76	2036.43	2221.15	2243.38	2420.74	2666.07	2950.29
Delta (degrees)	86.42	86.42	86.42	86.42	86.42	86.42	86.42	86.42
N	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
V	14.835	15.925	16.494	16.316	16.728	16.985	16.956	16.798
Tee:								
(i)	NA	NA	NA	NA	NA	NA	NA	NA
(ii)	NA	NA	NA	NA	NA	NA	NA	NA
(iii)	NA	NA	NA	NA	NA	NA	NA	NA
Angle	1.0034	1.0144	1.0004	1.0396	1.0140	1.0190	1.0548	1.0991
Flat Bar:								
(i)	NA	NA	NA	NA	NA	NA	NA	NA
(ii)	NA	NA	NA	NA	NA	NA	NA	NA
(iii)	NA	NA	NA	NA	NA	NA	NA	NA
Web Thks < Shell Thks (Sch.1 Pg.16)	YES	YES	YES	YES	YES	YES	YES	YES
SHEAR AREA SATISFIED	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	FALSE
PLASTIC MODULUS SATISFIED	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE
TRIPPING SATISFIED	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE
Plate Ratio	17.99	19.69	20.99	22.74	23.21	25.20	26.99	28.63
Span Ratio	20.16	18.57	18.18	17.69	17.69	17.33	16.77	16.25
Web Ratio	15.00	15.00	13.33	13.61	11.54	10.34	10.40	11.00

DISPLACEMENT (KTonnes)	9.00	9.00	9.00	9.00	9.00	9.00	9.00	9.00
POWER (MW)	18.00	18.00	18.00	18.00	18.00	18.00	18.00	18.00
MATERIAL YIELD STRENGTH (MPa)	355.00	355.00	355.00	355.00	355.00	355.00	355.00	355.00
CAC (1,2,3, or 4)	3	3	3	3	3	3	3	3
HULL AREA (Bow or Midbody)	Bow	Bow	Bow	Bow	Bow	Bow	Bow	Bow
Arctic Class Factor	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6
Area Factor	1	1	1	1	1	1	1	1
Fmax	48.973	48.973	48.973	48.973	48.973	48.973	48.973	48.973
Vp	1.166	1.166	1.166	1.166	1.166	1.166	1.166	1.166
Hp	9.331	9.331	9.331	9.331	9.331	9.331	9.331	9.331
FRAME SPACING	400	500	600	650	700	800	900	1000
FRAME SPAN	2600	2600	2600	2600	2600	2600	2600	2600
SHELL PLATE DESIGN:								
Corrosion Allowance	0	0	0	0	0	0	0	0
U	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8
Omega (Degrees)	90	90	90	90	90	90	90	90
Frame Orientation Factor	1	1	1	1	1	1	1	1
Plate Design Pressur - Dpp (MPa)	9.377	7.502	6.251	5.771	5.358	4.689	4.168	3.751
MInimum Dpp (MPa)	5.750	4.600	3.833	3.538	3.286	2.875	2.556	2.300
Dpp used for Plate Thickness (MPa)	9.377	7.502	6.251	5.771	5.358	4.689	4.168	3.751
Minimum Shell Plate Thickness (mm)	28.15	31.47	34.48	35.88	37.24	39.81	42.23	44.51
Shell Plate Thickness (mm)	28.58	31.75	34.925	36.513	38.1	41.275	42.86	46.04
TRANSVERSE FRAME DESIGN:								
Type	Angle	Angle	Angle	Angle	Angle	Angle	Angle	Angle
Dimensions:								
Web Depth (mm)	320	350	360	370	380	390	400	410
Web Thickness (mm)	26.00	28.00	30.80	32.00	33.00	36.00	38.00	40.00
Flange Width (mm)	140	145	155	161	166	181	191	201
Flange Thickness (mm)	20	20	20	20	20	20	20	20
Phi (degrees)	90	90	90	90	90	90	90	90
REQUIRED VALUES:								
DPT	0.043	0.054	0.064	0.070	0.075	0.086	0.096	0.107
PAV	9.549	9.171	8.847	8.701	8.565	8.319	8.102	7.909
Span LB (meters)	2.600	2.600	2.600	2.600	2.600	2.600	2.600	2.600
Spacing Between Frames S (meters)	0.4	0.5	0.6	0.65	0.7	0.8	0.9	1
Vp / Span	0.449	0.449	0.449	0.449	0.449	0.449	0.449	0.449
Factor A (Sch 1 Table 3)	0.806	0.806	0.806	0.806	0.806	0.806	0.806	0.806
Value H	15000	15000	15000	15000	15000	15000	15000	15000
Value B	2.976	2.976	2.976	2.976	2.976	2.976	2.976	2.976
Req. Trans. Frame Shear Area (cm2)	75.55	90.70	104.98	111.86	118.59	131.64	144.23	156.43
Req. Trans. Frame Plas. Modu. (cm3)	1727.60	2073.95	2400.69	2557.98	2711.78	3010.17	3298.08	3577.18
MINIMUM VALUES:								
Hp min	6.00	6.00	6.00	6.00	6.00	6.00	6.00	6.00
Vp min	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75
Vp min / Span	0.288	0.288	0.288	0.288	0.288	0.288	0.288	0.288
Factor A (Sch 1 Table 3)	0.873	0.873	0.873	0.873	0.873	0.873	0.873	0.873
DPT	0.067	0.083	0.100	0.108	0.117	0.133	0.150	0.167
PAV	8.781	8.372	8.036	7.890	7.755	7.515	7.307	7.124
Factor C	1	1	1	1	1	1	1	1
Factor B	2.034	2.034	2.034	2.034	2.034	2.034	2.034	2.034
Min. Trans. Frame Shear Area (cm2)	80.68	96.14	110.75	117.79	124.68	138.09	151.06	163.64
Min. Trans. Frame Plas. Modu. (cm3)	1809.42	2156.36	2483.81	2641.75	2796.40	3097.13	3387.93	3670.09
ACTUAL VALUES:								
Factor M	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Actual Shear Area (cm2)	83.20	98.00	110.88	118.40	125.40	140.40	152.00	164.00
Actual Plastic Modulus (cm3)	2414.10	2960.61	3390.60	3688.94	3979.53	4550.26	5013.80	5520.47

STRENGTH CRITERIA:								
Actual/Required (Shear Area)	1.031	1.019	1.001	1.005	1.006	1.017	1.006	1.002
Actual/Required (Plastic Modulus)	1.334	1.373	1.365	1.396	1.423	1.469	1.480	1.504
LOCAL BUCKLING CRITERIA:								
Flange Width > 5 X Web Thick (8.(1))	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE
Local Web Buckling (8.(2))								
- Tee or Angle - Requirements	53.074	53.074	53.074	53.074	53.074	53.074	53.074	53.074
- Tee or Angle - HW / TW	12.308	12.500	11.688	11.563	11.515	10.833	10.526	10.250
- Ratio (Required / Actual)	4.312	4.246	4.541	4.590	4.609	4.899	5.042	5.178
- Are Web Stiffeners Required?	NO	NO	NO	NO	NO	NO	NO	NO
Local Web Buckling (8.(3))								
- Flat Bar - Requirements	NA	NA	NA	NA	NA	NA	NA	NA
- Flat Bar - HW / TW	NA	NA	NA	NA	NA	NA	NA	NA
- Ratio (Required / Actual)	NA	NA	NA	NA	NA	NA	NA	NA
- Are Web Stiffeners Required?	NA	NA	NA	NA	NA	NA	NA	NA
Flange Outstand (8.(4))								
- Requirements	8.2265	8.2265	8.2265	8.2265	8.2265	8.2265	8.2265	8.2265
- Tee or Angle - FOS / TF	5.7	5.85	6.21	6.45	6.65	7.25	7.65	8.05
- Ratio (Required / Actual)	1.4433	1.4062	1.3247	1.2754	1.2371	1.1347	1.0754	1.0219
TRIPPING CRITERIA:								
FPM	1727.60	2073.95	2400.69	2557.98	2711.78	3010.17	3298.08	3577.18
AFPM	2414.10	2960.61	3390.60	3688.94	3979.53	4550.26	5013.80	5520.47
Delta (degrees)	86.42	86.42	86.42	86.42	86.42	86.42	86.42	86.42
N	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
V	15.939	15.770	15.854	15.690	15.553	15.325	15.281	15.167
Tee:								
(i)	NA	NA	NA	NA	NA	NA	NA	NA
(ii)	NA	NA	NA	NA	NA	NA	NA	NA
(iii)	NA	NA	NA	NA	NA	NA	NA	NA
Angle	1.0135	1.0609	1.1281	1.1840	1.2315	1.3628	1.4422	1.5291
Flat Bar:								
(i)	NA	NA	NA	NA	NA	NA	NA	NA
(ii)	NA	NA	NA	NA	NA	NA	NA	NA
(iii)	NA	NA	NA	NA	NA	NA	NA	NA
Web Thks < Shell Thks (Sch.1 Pg.16)	YES	YES	YES	YES	YES	YES	YES	YES
SHEAR AREA SATISFIED	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE
PLASTIC MODULUS SATISFIED	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE
TRIPPING SATISFIED	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE
Plate Ratio	14.00	15.75	17.18	17.80	18.37	19.38	21.00	21.72
Span Ratio	18.57	17.93	16.77	16.15	15.66	14.36	13.61	12.94
Web Ratio	12.31	12.50	11.69	11.56	11.52	10.83	10.53	10.25

DISPLACEMENT (KTonnes)	18.00	18.00	18.00	18.00	18.00	18.00	18.00	18.00
POWER (MW)	18.00	18.00	18.00	18.00	18.00	18.00	18.00	18.00
MATERIAL YIELD STRENGTH (MPa)	355.00	355.00	355.00	355.00	355.00	355.00	355.00	355.00
CAC (1,2,3, or 4)	3	3	3	3	3	3	3	3
HULL AREA (Bow or Midbody)	Bow	Bow	Bow	Bow	Bow	Bow	Bow	Bow
Arctic Class Factor	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6
Area Factor	1	1	1	1	1	1	1	1
Fmax	83.349	83.349	83.349	83.349	83.349	83.349	83.349	83.349
Vp	1.522	1.522	1.522	1.522	1.522	1.522	1.522	1.522
Hp	12.173	12.173	12.173	12.173	12.173	12.173	12.173	12.173
FRAME SPACING	400	500	600	650	700	800	900	1000
FRAME SPAN	2600	2600	2600	2600	2600	2600	2600	2600
SHELL PLATE DESIGN:								
Corrosion Allowance	0	0	0	0	0	0	0	0
U	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8
Omega (Degrees)	90	90	90	90	90	90	90	90
Frame Orientation Factor	1	1	1	1	1	1	1	1
Plate Design Pressur - Dpp (MPa)	9.864	7.891	6.576	6.070	5.637	4.932	4.384	3.946
MInimum Dpp (MPa)	5.750	4.600	3.833	3.538	3.286	2.875	2.556	2.300
Dpp used for Plate Thickness (MPa)	9.864	7.891	6.576	6.070	5.637	4.932	4.384	3.946
Minimum Shell Plate Thickness (mm)	28.87	32.28	35.36	36.80	38.19	40.83	43.31	45.65
Shell Plate Thickness (mm)	30.163	33.34	36.513	38.1	39.687	41.275	44.45	46.038
TRANSVERSE FRAME DESIGN:								
Type	Angle	Angle	Angle	Angle	Angle	Angle	Angle	Angle
Dimensions:								
Web Depth (mm)	350	370	400	400	420	450	460	500
Web Thickness (mm)	27.50	31.50	33.80	36.00	37.00	38.00	40.60	42.00
Flange Width (mm)	144	158.5	170	181	186	191	204	211
Flange Thickness (mm)	20	20	20	20	20	20	20	20
Phi (degrees)	90	90	90	90	90	90	90	90
REQUIRED VALUES:								
DPT	0.033	0.041	0.049	0.053	0.058	0.066	0.074	0.082
PAV	9.964	9.619	9.316	9.177	9.047	8.807	8.592	8.398
Span LB (meters)	2.600	2.600	2.600	2.600	2.600	2.600	2.600	2.600
Spacing Between Frames S (meters)	0.4	0.5	0.6	0.65	0.7	0.8	0.9	1
Vp / Span	0.585	0.585	0.585	0.585	0.585	0.585	0.585	0.585
Factor A (Sch 1 Table 3)	0.751	0.751	0.751	0.751	0.751	0.751	0.751	0.751
Value H	15000	15000	15000	15000	15000	15000	15000	15000
Value B	3.674	3.674	3.674	3.674	3.674	3.674	3.674	3.674
Req. Trans. Frame Shear Area (cm2)	95.89	115.71	134.48	143.52	152.36	169.51	186.05	202.06
Req. Trans. Frame Plas. Modu. (cm3)	2225.63	2685.77	3121.29	3331.19	3536.45	3934.59	4318.37	4689.91
MINIMUM VALUES:								
Hp min	6.00	6.00	6.00	6.00	6.00	6.00	6.00	6.00
Vp min	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75
Vp min / Span	0.288	0.288	0.288	0.288	0.288	0.288	0.288	0.288
Factor A (Sch 1 Table 3)	0.873	0.873	0.873	0.873	0.873	0.873	0.873	0.873
DPT	0.067	0.083	0.100	0.108	0.117	0.133	0.150	0.167
PAV	8.781	8.372	8.036	7.890	7.755	7.515	7.307	7.124
Factor C	1	1	1	1	1	1	1	1
Factor B	2.034	2.034	2.034	2.034	2.034	2.034	2.034	2.034
Min. Trans. Frame Shear Area (cm2)	80.68	96.14	110.75	117.79	124.68	138.09	151.06	163.64
Min. Trans. Frame Plas. Modu. (cm3)	1809.42	2156.36	2483.81	2641.75	2796.40	3097.13	3387.93	3670.09
ACTUAL VALUES:								
Factor M	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Actual Shear Area (cm2)	96.25	116.55	135.20	144.00	155.40	171.00	186.76	210.00
Actual Plastic Modulus (cm3)	2909.77	3607.91	4406.90	4707.48	5245.19	6036.44	6718.83	7982.74

STRENGTH CRITERIA:								
Actual/Required (Shear Area)	1.004	1.007	1.005	1.003	1.020	1.009	1.004	1.039
Actual/Required (Plastic Modulus)	1.307	1.343	1.412	1.413	1.483	1.534	1.556	1.702
LOCAL BUCKLING CRITERIA:								
Flange Width > 5 X Web Thick (8.(1))	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE
Local Web Buckling (8.(2))								
- Tee or Angle - Requirements	53.074	53.074	53.074	53.074	53.074	53.074	53.074	53.074
- Tee or Angle - HW / TW	12.727	11.746	11.834	11.111	11.351	11.842	11.330	11.905
- Ratio (Required / Actual)	4.170	4.519	4.485	4.777	4.676	4.482	4.684	4.458
- Are Web Stiffeners Required?	NO	NO	NO	NO	NO	NO	NO	NO
Local Web Buckling (8.(3))								
- Flat Bar - Requirements	NA	NA	NA	NA	NA	NA	NA	NA
- Flat Bar - HW / TW	NA	NA	NA	NA	NA	NA	NA	NA
- Ratio (Required / Actual)	NA	NA	NA	NA	NA	NA	NA	NA
- Are Web Stiffeners Required?	NA	NA	NA	NA	NA	NA	NA	NA
Flange Outstand (8.(4))								
- Requirements	8.2265	8.2265	8.2265	8.2265	8.2265	8.2265	8.2265	8.2265
- Tee or Angle - FOS / TF	5.825	6.35	6.81	7.25	7.45	7.65	8.17	8.45
- Ratio (Required / Actual)	1.4123	1.2955	1.2080	1.1347	1.1042	1.0754	1.0069	0.9736
TRIPPING CRITERIA:								
FPM	2225.63	2685.77	3121.29	3331.19	3536.45	3934.59	4318.37	4689.91
AFPM	2909.77	3607.91	4406.90	4707.48	5245.19	6036.44	6718.83	7982.74
Delta (degrees)	86.42	86.42	86.42	86.42	86.42	86.42	86.42	86.42
N	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
V	16.478	16.256	15.857	15.850	15.471	15.212	15.105	14.442
Tee:								
(i)	NA	NA	NA	NA	NA	NA	NA	NA
(ii)	NA	NA	NA	NA	NA	NA	NA	NA
(iii)	NA	NA	NA	NA	NA	NA	NA	NA
Angle	1.0083	1.1250	1.2370	1.3177	1.3872	1.4488	1.5583	1.6858
Flat Bar:								
(i)	NA	NA	NA	NA	NA	NA	NA	NA
(ii)	NA	NA	NA	NA	NA	NA	NA	NA
(iii)	NA	NA	NA	NA	NA	NA	NA	NA
Web Thks < Shell Thks (Sch.1 Pg.16)	YES	YES	YES	YES	YES	YES	YES	YES
SHEAR AREA SATISFIED	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE
PLASTIC MODULUS SATISFIED	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE
TRIPPING SATISFIED	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE
Plate Ratio	13.26	15.00	16.43	17.06	17.64	19.38	20.25	21.72
Span Ratio	18.06	16.40	15.29	14.36	13.98	13.61	12.75	12.32
Web Ratio	12.73	11.75	11.83	11.11	11.35	11.84	11.33	11.90

DISPLACEMENT (KTonnes)	9.00	9.00	9.00	9.00	9.00	9.00	9.00	9.00
POWER (MW)	18.00	18.00	18.00	18.00	18.00	18.00	18.00	18.00
MATERIAL YIELD STRENGTH (MPa)	355.00	355.00	355.00	355.00	355.00	355.00	355.00	355.00
CAC (1,2,3, or 4)	3	3	3	3	3	3	3	3
HULL AREA (Bow or Midbody)	Midbody	Midbody	Midbody	Midbody	Midbody	Midbody	Midbody	Midbody
Arctic Class Factor	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6
Area Factor	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5
Fmax	48.973	48.973	48.973	48.973	48.973	48.973	48.973	48.973
Vp	1.166	1.166	1.166	1.166	1.166	1.166	1.166	1.166
Hp	9.331	9.331	9.331	9.331	9.331	9.331	9.331	9.331
FRAME SPACING	400	500	600	650	700	800	900	1000
FRAME SPAN	2600	2600	2600	2600	2600	2600	2600	2600
SHELL PLATE DESIGN:								
Corrosion Allowance	0	0	0	0	0	0	0	0
U	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5
Omega (Degrees)	90	90	90	90	90	90	90	90
Frame Orientation Factor	1	1	1	1	1	1	1	1
Plate Design Pressur - Dpp (MPa)	4.689	3.751	3.126	2.885	2.679	2.344	2.084	1.875
MInimum Dpp (MPa)	5.750	4.600	3.833	3.538	3.286	2.875	2.556	2.300
Dpp used for Plate Thickness (MPa)	5.750	4.600	3.833	3.538	3.286	2.875	2.556	2.300
Minimum Shell Plate Thickness (mm)	22.04	24.65	27.00	28.10	29.16	31.17	33.07	34.85
Shell Plate Thickness (mm)	22.23	25.4	28.58	28.58	30.163	31.75	33.34	34.925
	1.23	1.09	1.00	1.01	1.01	1.02	1.02	1.02
TRANSVERSE FRAME DESIGN:	1.368	1.254	1.185	1.030	1.036	1.094	1.095	1.099
Type	Tee	Tee	Tee	Tee	Tee	Tee	Tee	Tee
Dimensions:								
Web Depth (mm)	300	300	300	300	300	320	320	320
Web Thickness (mm)	16.50	17.50	18.50	19.80	21.00	22.00	24.00	26.00
Flange Width (mm)	83.5	88.5	93.5	100	106	111	121	131
Flange Thickness (mm)	16.5	17.5	18.5	12	12	12	12	12
Phi (degrees)	90	90	90	90	90	90	90	90
REQUIRED VALUES:								
DPT	0.043	0.054	0.064	0.070	0.075	0.086	0.096	0.107
PAV	9.549	9.171	8.847	8.701	8.565	8.319	8.102	7.909
Span LB (meters)	2.600	2.600	2.600	2.600	2.600	2.600	2.600	2.600
Spacing Between Frames S (meters)	0.4	0.5	0.6	0.65	0.7	0.8	0.9	1
Vp / Span	0.449	0.449	0.449	0.449	0.449	0.449	0.449	0.449
Factor A (Sch 1 Table 3)	0.806	0.806	0.806	0.806	0.806	0.806	0.806	0.806
Value H	15000	15000	15000	15000	15000	15000	15000	15000
Value B	2.976	2.976	2.976	2.976	2.976	2.976	2.976	2.976
Req. Trans. Frame Shear Area (cm2)	37.77	45.35	52.49	55.93	59.29	65.82	72.11	78.22
Req. Trans. Frame Plas. Modu. (cm3)	863.80	1036.97	1200.34	1278.99	1355.89	1505.09	1649.04	1788.59
MINIMUM VALUES:								
Hp min	6.00	6.00	6.00	6.00	6.00	6.00	6.00	6.00
Vp min	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75
Vp min / Span	0.288	0.288	0.288	0.288	0.288	0.288	0.288	0.288
Factor A (Sch 1 Table 3)	0.873	0.873	0.873	0.873	0.873	0.873	0.873	0.873
DPT	0.067	0.083	0.100	0.108	0.117	0.133	0.150	0.167
PAV	8.781	8.372	8.036	7.890	7.755	7.515	7.307	7.124
Factor C	1	1	1	1	1	1	1	1
Factor B	2.034	2.034	2.034	2.034	2.034	2.034	2.034	2.034
Min. Trans. Frame Shear Area (cm2)	40.34	48.07	55.37	58.89	62.34	69.05	75.53	81.82
Min. Trans. Frame Plas. Modu. (cm3)	904.71	1078.18	1241.91	1320.88	1398.20	1548.56	1693.97	1835.05
ACTUAL VALUES:								
Factor M	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Actual Shear Area (cm2)	49.50	52.50	55.50	59.40	63.00	70.40	76.80	83.20
Actual Plastic Modulus (cm3)	1237.52	1352.02	1471.45	1360.23	1448.43	1693.54	1854.38	2016.41

STRENGTH CRITERIA:								
Actual/Required (Shear Area)	1.227	1.092	1.002	1.009	1.011	1.020	1.017	1.017
Actual/Required (Plastic Modulus)	1.368	1.254	1.185	1.030	1.036	1.094	1.095	1.099
LOCAL BUCKLING CRITERIA:								
Flange Width > 5 X Web Thick (8.(1))	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE
Local Web Buckling (8.(2))								
- Tee or Angle - Requirements	53.074	53.074	53.074	53.074	53.074	53.074	53.074	53.074
- Tee or Angle - HW / TW	18.182	17.143	16.216	15.152	14.286	14.545	13.333	12.308
- Ratio (Required / Actual)	2.919	3.096	3.273	3.503	3.715	3.649	3.981	4.312
- Are Web Stiffeners Required?	NO	NO	NO	NO	NO	NO	NO	NO
Local Web Buckling (8.(3))								
- Flat Bar - Requirements	NA	NA	NA	NA	NA	NA	NA	NA
- Flat Bar - HW / TW	NA	NA	NA	NA	NA	NA	NA	NA
- Ratio (Required / Actual)	NA	NA	NA	NA	NA	NA	NA	NA
- Are Web Stiffeners Required?	NA	NA	NA	NA	NA	NA	NA	NA
Flange Outstand (8.(4))								
- Requirements	8.2265	8.2265	8.2265	8.2265	8.2265	8.2265	8.2265	8.2265
- Tee or Angle - FOS / TF	2.030303	2.0285714	2.027027	3.3416667	3.5416667	3.7083333	4.0416667	4.375
- Ratio (Required / Actual)	4.0519	4.0553	4.0584	2.4618	2.3228	2.2184	2.0354	1.8804
TRIPPING CRITERIA:								
FPM	863.80	1036.97	1200.34	1278.99	1355.89	1505.09	1649.04	1788.59
AFPM	1237.52	1352.02	1471.45	1360.23	1448.43	1693.54	1854.38	2016.41
Delta (degrees)	90.00	90.00	90.00	90.00	90.00	90.00	90.00	90.00
N	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
V	15.741	16.501	17.017	18.270	18.230	17.762	17.768	17.745
Tee:								
(i)	1.0455306	1.0650107	1.1091847	1.1025844	1.2263128	1.3267125	1.5731767	1.8966608
(ii)	0.541565	0.5479516	0.5616794	0.5599301	0.5951851	0.5999387	0.6542785	0.7097001
(iii)	0.8058727	0.8148182	0.8347217	0.8315377	0.8833889	0.9494	1.034615	1.1215401
Angle	NA	NA	NA	NA	NA	NA	NA	NA
Flat Bar:								
(i)	NA	NA	NA	NA	NA	NA	NA	NA
(ii)	NA	NA	NA	NA	NA	NA	NA	NA
(iii)	NA	NA	NA	NA	NA	NA	NA	NA
Web Thks < Shell Thks (Sch.1 Pg.16)	YES	YES	YES	YES	YES	YES	YES	YES
Web Thks > (Sch.1 Pg.16)	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE
SHEAR AREA SATISFIED	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE
PLASTIC MODULUS SATISFIED	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE
TRIPPING SATISFIED	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE
Plate Ratio	17.99	19.69	20.99	22.74	23.21	25.20	26.99	28.63
Span Ratio	31.14	29.38	27.81	26.00	24.53	23.42	21.49	19.85
Web Ratio	18.18	17.14	16.22	15.15	14.29	14.55	13.33	12.31

DISPLACEMENT (KTonnes)	18.00	18.00	18.00	18.00	18.00	18.00	18.00	18.00
POWER (MW)	18.00	18.00	18.00	18.00	18.00	18.00	18.00	18.00
MATERIAL YIELD STRENGTH (MPa)	355.00	355.00	355.00	355.00	355.00	355.00	355.00	355.00
CAC (1,2,3, or 4)	3	3	3	3	3	3	3	3
HULL AREA (Bow or Midbody)	Midbody	Midbody	Midbody	Midbody	Midbody	Midbody	Midbody	Midbody
Arctic Class Factor	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6
Area Factor	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5
Fmax	83.349	83.349	83.349	83.349	83.349	83.349	83.349	83.349
Vp	1.522	1.522	1.522	1.522	1.522	1.522	1.522	1.522
Hp	12.173	12.173	12.173	12.173	12.173	12.173	12.173	12.173
FRAME SPACING	400	500	600	650	700	800	900	1000
FRAME SPAN	2600	2600	2600	2600	2600	2600	2600	2600
SHELL PLATE DESIGN:								
Corrosion Allowance	0	0	0	0	0	0	0	0
U	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5
Omega (Degrees)	90	90	90	90	90	90	90	90
Frame Orientation Factor	1	1	1	1	1	1	1	1
Plate Design Pressur - Dpp (MPa)	4.932	3.946	3.288	3.035	2.818	2.466	2.192	1.973
MInimum Dpp (MPa)	5.750	4.600	3.833	3.538	3.286	2.875	2.556	2.300
Dpp used for Plate Thickness (MPa)	5.750	4.600	3.833	3.538	3.286	2.875	2.556	2.300
Minimum Shell Plate Thickness (mm)	22.04	24.65	27.00	28.10	29.16	31.17	33.07	34.85
Shell Plate Thickness (mm)	22.23	25.4	28.58	28.58	30.163	31.75	33.34	34.925
TRANSVERSE FRAME DESIGN:								
Type	Tee	Tee	Tee	Tee	Tee	Tee	Tee	Tee
Dimensions:								
Web Depth (mm)	300	320	330	330	340	355	360	380
Web Thickness (mm)	16.50	18.50	21.00	22.00	23.00	24.00	26.00	27.00
Flange Width (mm)	95	100	100	111	116	121	131	136
Flange Thickness (mm)	16	16	16	12	12	12	12	12
Phi (degrees)	90	90	90	90	90	90	90	90
REQUIRED VALUES:								
DPT	0.033	0.041	0.049	0.053	0.058	0.066	0.074	0.082
PAV	9.964	9.619	9.316	9.177	9.047	8.807	8.592	8.398
Span LB (meters)	2.600	2.600	2.600	2.600	2.600	2.600	2.600	2.600
Spacing Between Frames S (meters)	0.4	0.5	0.6	0.65	0.7	0.8	0.9	1
Vp / Span	0.585	0.585	0.585	0.585	0.585	0.585	0.585	0.585
Factor A (Sch 1 Table 3)	0.751	0.751	0.751	0.751	0.751	0.751	0.751	0.751
Value H	15000	15000	15000	15000	15000	15000	15000	15000
Value B	3.674	3.674	3.674	3.674	3.674	3.674	3.674	3.674
Req. Trans. Frame Shear Area (cm2)	47.94	57.86	67.24	71.76	76.18	84.76	93.02	101.03
Req. Trans. Frame Plas. Modu. (cm3)	1112.82	1342.89	1560.65	1665.59	1768.22	1967.29	2159.18	2344.95
MINIMUM VALUES:								
Hp min	6.00	6.00	6.00	6.00	6.00	6.00	6.00	6.00
Vp min	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75
Vp min / Span	0.288	0.288	0.288	0.288	0.288	0.288	0.288	0.288
Factor A (Sch 1 Table 3)	0.873	0.873	0.873	0.873	0.873	0.873	0.873	0.873
DPT	0.067	0.083	0.100	0.108	0.117	0.133	0.150	0.167
PAV	8.781	8.372	8.036	7.890	7.755	7.515	7.307	7.124
Factor C	1	1	1	1	1	1	1	1
Factor B	2.034	2.034	2.034	2.034	2.034	2.034	2.034	2.034
Min. Trans. Frame Shear Area (cm2)	40.34	48.07	55.37	58.89	62.34	69.05	75.53	81.82
Min. Trans. Frame Plas. Modu. (cm3)	904.71	1078.18	1241.91	1320.88	1398.20	1548.56	1693.97	1835.05
ACTUAL VALUES:								
Factor M	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Actual Shear Area (cm2)	49.50	59.20	69.30	72.60	78.20	85.20	93.60	102.60
Actual Plastic Modulus (cm3)	1282.57	1567.50	1806.14	1768.23	1949.96	2194.78	2442.39	2787.02

STRENGTH CRITERIA:								
Actual/Required (Shear Area)	1.032	1.023	1.031	1.012	1.027	1.005	1.006	1.016
Actual/Required (Plastic Modulus)	1.153	1.167	1.157	1.062	1.103	1.116	1.131	1.189
LOCAL BUCKLING CRITERIA:								
Flange Width > 5 X Web Thick (8.(1))	TRUE	TRUE	FALSE	TRUE	TRUE	TRUE	TRUE	TRUE
Local Web Buckling (8.(2))								
- Tee or Angle - Requirements	53.074	53.074	53.074	53.074	53.074	53.074	53.074	53.074
- Tee or Angle - HW / TW	18.182	17.297	15.714	15.000	14.783	14.792	13.846	14.074
- Ratio (Required / Actual)	2.919	3.068	3.377	3.538	3.590	3.588	3.833	3.771
- Are Web Stiffeners Required?	NO	NO	NO	NO	NO	NO	NO	NO
Local Web Buckling (8.(3))								
- Flat Bar - Requirements	NA	NA	NA	NA	NA	NA	NA	NA
- Flat Bar - HW / TW	NA	NA	NA	NA	NA	NA	NA	NA
- Ratio (Required / Actual)	NA	NA	NA	NA	NA	NA	NA	NA
- Are Web Stiffeners Required?	NA	NA	NA	NA	NA	NA	NA	NA
Flange Outstand (8.(4))								
- Requirements	8.2265	8.2265	8.2265	8.2265	8.2265	8.2265	8.2265	8.2265
- Tee or Angle - FOS / TF	2.4531	2.5469	2.4688	3.7083	3.875	4.0417	4.375	4.5417
- Ratio (Required / Actual)	3.3535	3.2301	3.3323	2.2184	2.1230	2.0354	1.8804	1.8113
TRIPPING CRITERIA:								
FPM	1112.82	1342.89	1560.65	1665.59	1768.22	1967.29	2159.18	2344.95
AFPM	1282.57	1567.50	1806.14	1768.23	1949.96	2194.78	2442.39	2787.02
Delta (degrees)	90.00	90.00	90.00	90.00	90.00	90.00	90.00	90.00
N	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
V	17.550	17.439	17.514	18.286	17.942	17.838	17.715	17.283
Tee:								
(i)	1.0063	1.0963	1.1548	1.2306	1.3437	1.4155	1.6465	1.7671
(ii)	0.4857	0.5138	0.5632	0.5651	0.5844	0.5874	0.6319	0.6372
(iii)	0.8224	0.8712	0.8674	0.9222	0.9822	1.0305	1.1234	1.1955
Angle	NA	NA	NA	NA	NA	NA	NA	NA
Flat Bar:								
(i)	NA	NA	NA	NA	NA	NA	NA	NA
(ii)	NA	NA	NA	NA	NA	NA	NA	NA
(iii)	NA	NA	NA	NA	NA	NA	NA	NA
Web Thks < Shell Thks (Sch.1 Pg.16)	YES	YES	YES	YES	YES	YES	YES	YES
Web Thks > (Sch.1 Pg.16)	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE
SHEAR AREA SATISFIED	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE
PLASTIC MODULUS SATISFIED	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE
TRIPPING SATISFIED	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE
Plate Ratio	17.99	19.69	20.99	22.74	23.21	25.20	26.99	28.63
Span Ratio	27.37	26.00	26.00	23.42	22.41	21.49	19.85	19.12
Web Ratio	18.18	17.30	15.71	15.00	14.78	14.79	13.85	14.07

DISPLACEMENT (KTonnes)	9.00	9.00	9.00	9.00	9.00	9.00	9.00	9.00
POWER (MW)	18.00	18.00	18.00	18.00	18.00	18.00	18.00	18.00
MATERIAL YIELD STRENGTH (MPa)	355.00	355.00	355.00	355.00	355.00	355.00	355.00	355.00
CAC (1,2,3, or 4)	3	3	3	3	3	3	3	3
HULL AREA (Bow or Midbody)	Bow	Bow	Bow	Bow	Bow	Bow	Bow	Bow
Arctic Class Factor	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6
Area Factor	1	1	1	1	1	1	1	1
Fmax	48.973	48.973	48.973	48.973	48.973	48.973	48.973	48.973
Vp	1.166	1.166	1.166	1.166	1.166	1.166	1.166	1.166
Hp	9.331	9.331	9.331	9.331	9.331	9.331	9.331	9.331
FRAME SPACING	400	500	600	650	700	800	900	1000
FRAME SPAN	2600	2600	2600	2600	2600	2600	2600	2600
SHELL PLATE DESIGN:								
Corrosion Allowance	0	0	0	0	0	0	0	0
U	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8
Omega (Degrees)	90	90	90	90	90	90	90	90
Frame Orientation Factor	1	1	1	1	1	1	1	1
Plate Design Pressur - Dpp (MPa)	9.377	7.502	6.251	5.771	5.358	4.689	4.168	3.751
MInimum Dpp (MPa)	5.750	4.600	3.833	3.538	3.286	2.875	2.556	2.300
Dpp used for Plate Thickness (MPa)	9.377	7.502	6.251	5.771	5.358	4.689	4.168	3.751
Minimum Shell Plate Thickness (mm)	28.15	31.47	34.48	35.88	37.24	39.81	42.23	44.51
Shell Plate Thickness (mm)	28.58	31.75	34.925	28.58	30.163	31.75	33.34	34.925
TRANSVERSE FRAME DESIGN:								
Type	Tee	Tee	Tee	Tee	Tee	Tee	Tee	Tee
Dimensions:								
Web Depth (mm)	430	450	460	470	500	500	500	520
Web Thickness (mm)	19.00	21.50	24.50	25.50	26.00	27.80	30.50	32.00
Flange Width (mm)	96	108.5	123.5	128.5	131	140	153.5	161
Flange Thickness (mm)	14	14	14	14	14	14	14	14
Phi (degrees)	90	90	90	90	90	90	90	90
REQUIRED VALUES:								
DPT	0.043	0.054	0.064	0.070	0.075	0.086	0.096	0.107
PAV	9.549	9.171	8.847	8.701	8.565	8.319	8.102	7.909
Span LB (meters)	2.600	2.600	2.600	2.600	2.600	2.600	2.600	2.600
Spacing Between Frames S (meters)	0.4	0.5	0.6	0.65	0.7	0.8	0.9	1
Vp / Span	0.449	0.449	0.449	0.449	0.449	0.449	0.449	0.449
Factor A (Sch 1 Table 3)	0.806	0.806	0.806	0.806	0.806	0.806	0.806	0.806
Value H	15000	15000	15000	15000	15000	15000	15000	15000
Value B	2.976	2.976	2.976	2.976	2.976	2.976	2.976	2.976
Req. Trans. Frame Shear Area (cm2)	75.55	90.70	104.98	111.86	118.59	131.64	144.23	156.43
Req. Trans. Frame Plas. Modu. (cm3)	1727.60	2073.95	2400.69	2557.98	2711.78	3010.17	3298.08	3577.18
MINIMUM VALUES:								
Hp min	6.00	6.00	6.00	6.00	6.00	6.00	6.00	6.00
Vp min	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75
Vp min / Span	0.288	0.288	0.288	0.288	0.288	0.288	0.288	0.288
Factor A (Sch 1 Table 3)	0.873	0.873	0.873	0.873	0.873	0.873	0.873	0.873
DPT	0.067	0.083	0.100	0.108	0.117	0.133	0.150	0.167
PAV	8.781	8.372	8.036	7.890	7.755	7.515	7.307	7.124
Factor C	1	1	1	1	1	1	1	1
Factor B	2.034	2.034	2.034	2.034	2.034	2.034	2.034	2.034
Min. Trans. Frame Shear Area (cm2)	80.68	96.14	110.75	117.79	124.68	138.09	151.06	163.64
Min. Trans. Frame Plas. Modu. (cm3)	1809.42	2156.36	2483.81	2641.75	2796.40	3097.13	3387.93	3670.09
ACTUAL VALUES:								
Factor M	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Actual Shear Area (cm2)	81.70	96.75	112.70	119.85	130.00	139.00	152.50	166.40
Actual Plastic Modulus (cm3)	2479.83	3048.76	3626.54	3871.57	4403.56	4720.50	5192.08	5844.19

STRENGTH CRITERIA:								
Actual/Required (Shear Area)	1.013	1.006	1.018	1.018	1.043	1.007	1.010	1.017
Actual/Required (Plastic Modulus)	1.371	1.414	1.460	1.466	1.575	1.524	1.533	1.592
LOCAL BUCKLING CRITERIA:								
Flange Width > 5 X Web Thick (8.(1))	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE
Local Web Buckling (8.(2))								
- Tee or Angle - Requirements	53.074	53.074	53.074	53.074	53.074	53.074	53.074	53.074
- Tee or Angle - HW / TW	22.632	20.930	18.776	18.431	19.231	17.986	16.393	16.250
- Ratio (Required / Actual)	2.345	2.536	2.827	2.880	2.760	2.951	3.238	3.266
- Are Web Stiffeners Required?	NO	NO	NO	NO	NO	NO	NO	NO
Local Web Buckling (8.(3))								
- Flat Bar - Requirements	NA	NA	NA	NA	NA	NA	NA	NA
- Flat Bar - HW / TW	NA	NA	NA	NA	NA	NA	NA	NA
- Ratio (Required / Actual)	NA	NA	NA	NA	NA	NA	NA	NA
- Are Web Stiffeners Required?	NA	NA	NA	NA	NA	NA	NA	NA
Flange Outstand (8.(4))								
- Requirements	8.2265	8.2265	8.2265	8.2265	8.2265	8.2265	8.2265	8.2265
- Tee or Angle - FOS / TF	2.75	3.1071429	3.5357143	3.6785714	3.75	4.0071429	4.3928571	4.6071429
- Ratio (Required / Actual)	2.9915	2.6476	2.3267	2.2363	2.1937	2.0530	1.8727	1.7856
TRIPPING CRITERIA:								
FPM	1727.60	2073.95	2400.69	2557.98	2711.78	3010.17	3298.08	3577.18
AFPM	2479.83	3048.76	3626.54	3871.57	4403.56	4720.50	5192.08	5844.19
Delta (degrees)	90.00	90.00	90.00	90.00	90.00	90.00	90.00	90.00
N	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
V	15.726	15.540	15.330	15.315	14.786	15.046	15.017	14.741
Tee:								
(i)	1.0858546	1.2867937	1.5823933	1.6683597	1.753531	1.9047939	2.2678607	2.4924803
(ii)	0.4355043	0.4765478	0.538523	0.549104	0.5451243	0.5727834	0.6296329	0.6470769
(iii)	0.9274074	1.0607239	1.2239255	1.2746984	1.3460325	1.41363	1.5529519	1.6593087
Angle	NA	NA	NA	NA	NA	NA	NA	NA
Flat Bar:								
(i)	NA	NA	NA	NA	NA	NA	NA	NA
(ii)	NA	NA	NA	NA	NA	NA	NA	NA
(iii)	NA	NA	NA	NA	NA	NA	NA	NA
Web Thks < Shell Thks (Sch.1 Pg.16)	YES	YES	YES	YES	YES	YES	YES	YES
Web Thks > (Sch.1 Pg.16)	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE
SHEAR AREA SATISFIED	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE
PLASTIC MODULUS SATISFIED	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE
TRIPPING SATISFIED	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE
Plate Ratio	14.00	15.75	17.18	22.74	23.21	25.20	26.99	28.63
Span Ratio	27.08	23.96	21.05	20.23	19.85	18.57	16.94	16.15
Web Ratio	22.63	20.93	18.78	18.43	19.23	17.99	16.39	16.25

DISPLACEMENT (KTonnes)	18.00	18.00	18.00	18.00	18.00	18.00	18.00	18.00
POWER (MW)	18.00	18.00	18.00	18.00	18.00	18.00	18.00	18.00
MATERIAL YIELD STRENGTH (MPa)	355.00	355.00	355.00	355.00	355.00	355.00	355.00	355.00
CAC (1,2,3, or 4)	3	3	3	3	3	3	3	3
HULL AREA (Bow or Midbody)	Bow	Bow	Bow	Bow	Bow	Bow	Bow	Bow
Arctic Class Factor	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6
Area Factor	1	1	1	1	1	1	1	1
Fmax	83.349	83.349	83.349	83.349	83.349	83.349	83.349	83.349
Vp	1.522	1.522	1.522	1.522	1.522	1.522	1.522	1.522
Hp	12.173	12.173	12.173	12.173	12.173	12.173	12.173	12.173
FRAME SPACING	400	500	600	650	700	800	900	1000
FRAME SPAN	2600	2600	2600	2600	2600	2600	2600	2600
SHELL PLATE DESIGN:								
Corrosion Allowance	0	0	0	0	0	0	0	0
U	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8
Omega (Degrees)	90	90	90	90	90	90	90	90
Frame Orientation Factor	1	1	1	1	1	1	1	1
Plate Design Pressur - Dpp (MPa)	9.864	7.891	6.576	6.070	5.637	4.932	4.384	3.946
MInimum Dpp (MPa)	5.750	4.600	3.833	3.538	3.286	2.875	2.556	2.300
Dpp used for Plate Thickness (MPa)	9.864	7.891	6.576	6.070	5.637	4.932	4.384	3.946
Minimum Shell Plate Thickness (mm)	28.87	32.28	35.36	36.80	38.19	40.83	43.31	45.65
Shell Plate Thickness (mm)	30.163	33.34	36.513	38.1	39.687	41.275	44.45	46.04
TRANSVERSE FRAME DESIGN:								
Type	Tee	Tee	Tee	Tee	Tee	Tee	Tee	Tee
Dimensions:								
Web Depth (mm)	500	500	500	500	500	500	500	500
Web Thickness (mm)	19.20	24.00	27.00	29.00	31.00	34.00	38.00	41.00
Flange Width (mm)	97	121	136	146	156	171	191	206
Flange Thickness (mm)	15	15	15	15	15	15	15	15
Phi (degrees)	90	90	90	90	90	90	90	90
REQUIRED VALUES:								
DPT	0.033	0.041	0.049	0.053	0.058	0.066	0.074	0.082
PAV	9.964	9.619	9.316	9.177	9.047	8.807	8.592	8.398
Span LB (meters)	2.600	2.600	2.600	2.600	2.600	2.600	2.600	2.600
Spacing Between Frames S (meters)	0.4	0.5	0.6	0.65	0.7	0.8	0.9	1
Vp / Span	0.585	0.585	0.585	0.585	0.585	0.585	0.585	0.585
Factor A (Sch 1 Table 3)	0.751	0.751	0.751	0.751	0.751	0.751	0.751	0.751
Value H	15000	15000	15000	15000	15000	15000	15000	15000
Value B	3.674	3.674	3.674	3.674	3.674	3.674	3.674	3.674
Req. Trans. Frame Shear Area (cm2)	95.89	115.71	134.48	143.52	152.36	169.51	186.05	202.06
Req. Trans. Frame Plas. Modu. (cm3)	2225.63	2685.77	3121.29	3331.19	3536.45	3934.59	4318.37	4689.91
MINIMUM VALUES:								
Hp min	6.00	6.00	6.00	6.00	6.00	6.00	6.00	6.00
Vp min	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75
Vp min / Span	0.288	0.288	0.288	0.288	0.288	0.288	0.288	0.288
Factor A (Sch 1 Table 3)	0.873	0.873	0.873	0.873	0.873	0.873	0.873	0.873
DPT	0.067	0.083	0.100	0.108	0.117	0.133	0.150	0.167
PAV	8.781	8.372	8.036	7.890	7.755	7.515	7.307	7.124
Factor C	1	1	1	1	1	1	1	1
Factor B	2.034	2.034	2.034	2.034	2.034	2.034	2.034	2.034
Min. Trans. Frame Shear Area (cm2)	80.68	96.14	110.75	117.79	124.68	138.09	151.06	163.64
Min. Trans. Frame Plas. Modu. (cm3)	1809.42	2156.36	2483.81	2641.75	2796.40	3097.13	3387.93	3670.09
ACTUAL VALUES:								
Factor M	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Actual Shear Area (cm2)	96.00	120.00	135.00	145.00	155.00	170.00	190.00	205.00
Actual Plastic Modulus (cm3)	3305.14	4151.41	4694.01	5054.37	5416.56	5955.51	6689.94	7236.22

STRENGTH CRITERIA:								
Actual/Required (Shear Area)	1.001	1.037	1.004	1.010	1.017	1.003	1.021	1.015
Actual/Required (Plastic Modulus)	1.485	1.546	1.504	1.517	1.532	1.514	1.549	1.543
LOCAL BUCKLING CRITERIA:								
Flange Width > 5 X Web Thick (8.(1))	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE
Local Web Buckling (8.(2))								
- Tee or Angle - Requirements	53.074	53.074	53.074	53.074	53.074	53.074	53.074	53.074
- Tee or Angle - HW / TW	26.042	20.833	18.519	17.241	16.129	14.706	13.158	12.195
- Ratio (Required / Actual)	2.038	2.548	2.866	3.078	3.291	3.609	4.034	4.352
- Are Web Stiffeners Required?	NO	NO	NO	NO	NO	NO	NO	NO
Local Web Buckling (8.(3))								
- Flat Bar - Requirements	NA	NA	NA	NA	NA	NA	NA	NA
- Flat Bar - HW / TW	NA	NA	NA	NA	NA	NA	NA	NA
- Ratio (Required / Actual)	NA	NA	NA	NA	NA	NA	NA	NA
- Are Web Stiffeners Required?	NA	NA	NA	NA	NA	NA	NA	NA
Flange Outstand (8.(4))								
- Requirements	8.2265	8.2265	8.2265	8.2265	8.2265	8.2265	8.2265	8.2265
- Tee or Angle - FOS / TF	2.5933333	3.2333333	3.6333333	3.9	4.1666667	4.5666667	5.1	5.5
- Ratio (Required / Actual)	3.1722	2.5443	2.2642	2.1094	1.9744	1.8014	1.6130	1.4957
TRIPPING CRITERIA:								
FPM	2225.63	2685.77	3121.29	3331.19	3536.45	3934.59	4318.37	4689.91
AFPM	3305.14	4151.41	4694.01	5054.37	5416.56	5955.51	6689.94	7236.22
Delta (degrees)	90.00	90.00	90.00	90.00	90.00	90.00	90.00	90.00
N	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
V	15.461	15.155	15.364	15.296	15.224	15.315	15.138	15.168
Tee:								
(i)	1.0749642	1.4917049	1.7511928	1.9926411	2.2790773	2.7451734	3.8513836	5.1170148
(ii)	0.3849609	0.4909325	0.5447735	0.5877323	0.6312297	0.6882343	0.7781839	0.8379246
(iii)	0.9531235	1.212992	1.3447861	1.4500952	1.5567267	1.6963504	1.9168762	2.0632428
Angle	NA	NA	NA	NA	NA	NA	NA	NA
Flat Bar:								
(i)	NA	NA	NA	NA	NA	NA	NA	NA
(ii)	NA	NA	NA	NA	NA	NA	NA	NA
(iii)	NA	NA	NA	NA	NA	NA	NA	NA
Web Thks < Shell Thks (Sch.1 Pg.16)	YES	YES	YES	YES	YES	YES	YES	YES
Web Thks > (Sch.1 Pg.16)	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE
SHEAR AREA SATISFIED	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE
PLASTIC MODULUS SATISFIED	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE
TRIPPING SATISFIED	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE
Plate Ratio	13.26	15.00	16.43	17.06	17.64	19.38	20.25	21.72
Span Ratio	26.80	21.49	19.12	17.81	16.67	15.20	13.61	12.62
Web Ratio	26.04	20.83	18.52	17.24	16.13	14.71	13.16	12.20

DISPLACEMENT (KTonnes)	9.00	9.00	9.00	9.00	9.00	9.00	9.00	9.00
POWER (MW)	18.00	18.00	18.00	18.00	18.00	18.00	18.00	18.00
MATERIAL YIELD STRENGTH (MPa)	355.00	355.00	355.00	355.00	355.00	355.00	355.00	355.00
CAC (1,2,3, or 4)	3	3	3	3	3	3	3	3
HULL AREA (Bow or Midbody)	Midbody	Midbody	Midbody	Midbody	Midbody	Midbody	Midbody	Midbody
Arctic Class Factor	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6
Area Factor	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5
Fmax	48.973	48.973	48.973	48.973	48.973	48.973	48.973	48.973
Vp	1.166	1.166	1.166	1.166	1.166	1.166	1.166	1.166
Hp	9.331	9.331	9.331	9.331	9.331	9.331	9.331	9.331
FRAME SPACING	400	500	600	650	700	800	900	950
FRAME SPAN	2600	2600	2600	2600	2600	2600	2600	2600
SHELL PLATE DESIGN:								
Corrosion Allowance	0	0	0	0	0	0	0	0
U	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5
Omega (Degrees)	90	90	90	90	90	90	90	90
Frame Orientation Factor	1	1	1	1	1	1	1	1
Plate Design Pressure - Dpp (MPa)	4.689	3.751	3.126	2.885	2.679	2.344	2.084	1.974
MInimum Dpp (MPa)	5.750	4.600	3.833	3.538	3.286	2.875	2.556	2.421
Dpp used for Plate Thickness (MPa)	5.750	4.600	3.833	3.538	3.286	2.875	2.556	2.421
Minimum Shell Plate Thickness (mm)	22.04	24.65	27.00	28.10	29.16	31.17	33.07	33.97
Shell Plate Thickness (mm)	25.4	25.4	28.58	28.58	30.163	31.75	33.34	34.925
TRANSVERSE FRAME DESIGN:								
Type	Flat Bar	Flat Bar	Flat Bar	Flat Bar	Flat Bar	Flat Bar	Flat Bar	Flat Bar
Dimensions:								
Web Depth (mm)	288	280	282	317	290	298	303	310
Web Thickness (mm)	25.40	25.40	28.58	28.58	30.16	31.75	33.34	33.34
Flange Width (mm)								
Flange Thickness (mm)								
Phi (degrees)	90	90	90	90	90	90	90	90
REQUIRED VALUES:								
DPT	0.043	0.054	0.064	0.070	0.075	0.086	0.096	0.102
PAV	9.549	9.171	8.847	8.701	8.565	8.319	8.102	8.003
Span LB (meters)	2.600	2.600	2.600	2.600	2.600	2.600	2.600	2.600
Spacing Between Frames S (meters)	0.4	0.5	0.6	0.65	0.7	0.8	0.9	0.95
Vp / Span	0.449	0.449	0.449	0.449	0.449	0.449	0.449	0.449
Factor A (Sch 1 Table 3)	0.806	0.806	0.806	0.806	0.806	0.806	0.806	0.806
Value H	17320	17320	17320	17320	17320	17320	17320	17320
Value B	2.976	2.976	2.976	2.976	2.976	2.976	2.976	2.976
Req. Trans. Frame Shear Area (cm2)	43.62	52.36	60.61	64.58	68.47	76.00	83.27	86.82
Req. Trans. Frame Plas. Modu. (cm3)	863.80	1036.97	1200.34	1278.99	1355.89	1505.09	1649.04	1719.32
MINIMUM VALUES:								
Hp min	6.00	6.00	6.00	6.00	6.00	6.00	6.00	6.00
Vp min	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75
Vp min / Span	0.288	0.288	0.288	0.288	0.288	0.288	0.288	0.288
Factor A (Sch 1 Table 3)	0.873	0.873	0.873	0.873	0.873	0.873	0.873	0.873
DPT	0.067	0.083	0.100	0.108	0.117	0.133	0.150	0.158
PAV	8.781	8.372	8.036	7.890	7.755	7.515	7.307	7.213
Factor C	1	1	1	1	1	1	1	1
Factor B	2.034	2.034	2.034	2.034	2.034	2.034	2.034	2.034
Min. Trans. Frame Shear Area (cm2)	46.58	55.51	63.94	68.00	71.98	79.72	87.21	90.87
Min. Trans. Frame Plas. Modu. (cm3)	904.71	1078.18	1241.91	1320.88	1398.20	1548.56	1693.97	1765.02
ACTUAL VALUES:								
Factor M	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Actual Shear Area (cm2)	73.15	71.12	80.60	90.60	87.47	94.62	101.02	103.35
Actual Plastic Modulus (cm3)	1146.29	1086.00	1251.57	1565.45	1400.28	1559.96	1698.86	1782.47

STRENGTH CRITERIA:								
Actual/Required (Shear Area)	1.571	1.281	1.261	1.332	1.215	1.187	1.158	1.137
Actual/Required (Plastic Modulus)	1.267	1.007	1.008	1.185	1.001	1.007	1.003	1.010
LOCAL BUCKLING CRITERIA:								
Flange Width > 5 X Web Thick (8.(1))	NA	NA	NA	NA	NA	NA	NA	NA
Local Web Buckling (8.(2))								
- Tee or Angle - Requirements	NA	NA	NA	NA	NA	NA	NA	NA
- Tee or Angle - HW / TW	NA	NA	NA	NA	NA	NA	NA	NA
- Ratio (Required / Actual)	NA	NA	NA	NA	NA	NA	NA	NA
- Are Web Stiffeners Required?	NA	NA	NA	NA	NA	NA	NA	NA
Local Web Buckling (8.(3))								
- Flat Bar - Requirements	14.967	14.967	14.967	14.967	14.967	14.967	14.967	14.967
- Flat Bar - HW / TW	11.339	11.024	9.867	11.092	9.614	9.386	9.088	9.298
- Ratio (Required / Actual)	1.320	1.358	1.517	1.349	1.557	1.595	1.647	1.610
- Are Web Stiffeners Required?	NO	NO	NO	NO	NO	NO	NO	NO
Flange Outstand (8.94))								
- Requirements	NA	NA	NA	NA	NA	NA	NA	NA
- Tee or Angle - FOS / TF	NA	NA	NA	NA	NA	NA	NA	NA
- Ratio (Required / Actual)	NA	NA	NA	NA	NA	NA	NA	NA
TRIPPING CRITERIA:								
FPM	863.80	1036.97	1200.34	1278.99	1355.89	1505.09	1649.04	1719.32
AFPM	1146.29	1086.00	1251.57	1565.45	1400.28	1559.96	1698.86	1782.47
Delta (degrees)	90.00	90.00	90.00	90.00	90.00	90.00	90.00	90.00
N	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
V	16.356	18.411	18.452	17.031	18.540	18.507	18.563	18.505
Tee:								
(i)	NA	NA	NA	NA	NA	NA	NA	NA
(ii)	NA	NA	NA	NA	NA	NA	NA	NA
(iii)	NA	NA	NA	NA	NA	NA	NA	NA
Angle	NA	NA	NA	NA	NA	NA	NA	NA
Flat Bar:								
(i)	1.0014	0.6714	1.0975	1.0024	1.3289	1.8432	5.3694	2.2785
(ii)	0.9059	0.8278	0.9227	0.8894	0.9425	0.9672	0.9958	0.9764
(iii)	0.4241	0.3767	0.4230	0.4583	0.4443	0.4685	0.4905	0.4920
Web Thks < Shell Thks (Sch.1 Pg.16)	YES	YES	YES	YES	YES	YES	YES	YES
SHEAR AREA SATISFIED	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE
PLASTIC MODULUS SATISFIED	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE
TRIPPING SATISFIED	TRUE	FALSE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE
Plate Ratio	15.75	19.69	20.99	22.74	23.21	25.20	26.99	27.20
Spane Ratio	102.36	102.36	90.97	90.97	86.20	81.89	77.98	77.98
Web Ratio	11.34	11.02	9.87	11.09	9.61	9.39	9.09	9.30

DISPLACEMENT (KTonnes)	18.00	18.00	18.00	18.00	18.00	18.00	18.00	18.00
POWER (MW)	18.00	18.00	18.00	18.00	18.00	18.00	18.00	18.00
MATERIAL YIELD STRENGTH (MPa)	355.00	355.00	355.00	355.00	355.00	355.00	355.00	355.00
CAC (1,2,3, or 4)	3	3	3	3	3	3	3	3
HULL AREA (Bow or Midbody)	Midbody	Midbody	Midbody	Midbody	Midbody	Midbody	Midbody	Midbody
Arctic Class Factor	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6
Area Factor	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5
Fmax	83.349	83.349	83.349	83.349	83.349	83.349	83.349	83.349
Vp	1.522	1.522	1.522	1.522	1.522	1.522	1.522	1.522
Hp	12.173	12.173	12.173	12.173	12.173	12.173	12.173	12.173
FRAME SPACING	400	500	600	650	700	800	900	950
FRAME SPAN	2600	2600	2600	2600	2600	2600	2600	2600
SHELL PLATE DESIGN:								
Corrosion Allowance	0	0	0	0	0	0	0	0
U	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5
Omega (Degrees)	90	90	90	90	90	90	90	90
Frame Orientation Factor	1	1	1	1	1	1	1	1
Plate Design Pressure - Dpp (MPa)	4.932	3.946	3.288	3.035	2.818	2.466	2.192	2.077
MInimum Dpp (MPa)	5.750	4.600	3.833	3.538	3.286	2.875	2.556	2.421
Dpp used for Plate Thickness (MPa)	5.750	4.600	3.833	3.538	3.286	2.875	2.556	2.421
Minimum Shell Plate Thickness (mm)	22.04	24.65	27.00	28.10	29.16	31.17	33.07	33.97
Shell Plate Thickness (mm)	28.58	28.58	30.163	30.163	31.75	31.75	33.34	34.925
TRANSVERSE FRAME DESIGN:								
Type	Flat Bar	Flat Bar	Flat Bar	Flat Bar	Flat Bar	Flat Bar	Flat Bar	Flat Bar
Dimensions:								
Web Depth (mm)	266	359	346	399	319	406	360	350
Web Thickness (mm)	28.58	28.58	30.16	30.16	31.75	31.75	33.34	34.93
Flange Width (mm)								
Flange Thickness (mm)								
Phi (degrees)	90	90	90	90	90	90	90	90
REQUIRED VALUES:								
DPT	0.033	0.041	0.049	0.053	0.058	0.066	0.074	0.078
PAV	9.964	9.619	9.316	9.177	9.047	8.807	8.592	8.493
Span LB (meters)	2.600	2.600	2.600	2.600	2.600	2.600	2.600	2.600
Spacing Between Frames S (meters)	0.4	0.5	0.6	0.65	0.7	0.8	0.9	0.95
Vp / Span	0.585	0.585	0.585	0.585	0.585	0.585	0.585	0.585
Factor A (Sch 1 Table 3)	0.751	0.751	0.751	0.751	0.751	0.751	0.751	0.751
Value H	17320	17320	17320	17320	17320	17320	17320	17320
Value B	3.674	3.674	3.674	3.674	3.674	3.674	3.674	3.674
Req. Trans. Frame Shear Area (cm2)	55.36	66.80	77.64	82.86	87.96	97.87	107.41	112.07
Req. Trans. Frame Plas. Modu. (cm3)	1112.82	1342.89	1560.65	1665.59	1768.22	1967.29	2159.18	2252.78
MINIMUM VALUES:								
Hp min	6.00	6.00	6.00	6.00	6.00	6.00	6.00	6.00
Vp min	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75
Vp min / Span	0.288	0.288	0.288	0.288	0.288	0.288	0.288	0.288
Factor A (Sch 1 Table 3)	0.873	0.873	0.873	0.873	0.873	0.873	0.873	0.873
DPT	0.067	0.083	0.100	0.108	0.117	0.133	0.150	0.158
PAV	8.781	8.372	8.036	7.890	7.755	7.515	7.307	7.213
Factor C	1	1	1	1	1	1	1	1
Factor B	2.034	2.034	2.034	2.034	2.034	2.034	2.034	2.034
Min. Trans. Frame Shear Area (cm2)	46.58	55.51	63.94	68.00	71.98	79.72	87.21	90.87
Min. Trans. Frame Plas. Modu. (cm3)	904.71	1078.18	1241.91	1320.88	1398.20	1548.56	1693.97	1765.02
ACTUAL VALUES:								
Factor M	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Actual Shear Area (cm2)	76.02	102.60	104.36	120.35	101.28	128.91	120.02	122.24
Actual Plastic Modulus (cm3)	1119.74	1988.33	1962.89	2582.50	1776.24	2821.41	2360.51	2352.61

STRENGTH CRITERIA:								
Actual/Required (Shear Area)	1.373	1.536	1.344	1.452	1.151	1.317	1.117	1.091
Actual/Required (Plastic Modulus)	1.006	1.481	1.258	1.550	1.005	1.434	1.093	1.044
LOCAL BUCKLING CRITERIA:								
Flange Width > 5 X Web Thick (8.(1))	NA	NA	NA	NA	NA	NA	NA	NA
Local Web Buckling (8.(2))								
- Tee or Angle - Requirements	NA	NA	NA	NA	NA	NA	NA	NA
- Tee or Angle - HW / TW	NA	NA	NA	NA	NA	NA	NA	NA
- Ratio (Required / Actual)	NA	NA	NA	NA	NA	NA	NA	NA
- Are Web Stiffeners Required?	NA	NA	NA	NA	NA	NA	NA	NA
Local Web Buckling (8.(3))								
- Flat Bar - Requirements	14.967	14.967	14.967	14.967	14.967	14.967	14.967	14.967
- Flat Bar - HW / TW	9.307	12.561	11.471	13.228	10.047	12.787	10.798	10.021
- Ratio (Required / Actual)	1.608	1.192	1.305	1.131	1.490	1.170	1.386	1.493
- Are Web Stiffeners Required?	NO	NO	NO	NO	NO	NO	NO	NO
Flange Outstand (8.94))								
- Requirements	NA	NA	NA	NA	NA	NA	NA	NA
- Tee or Angle - FOS / TF	NA	NA	NA	NA	NA	NA	NA	NA
- Ratio (Required / Actual)	NA	NA	NA	NA	NA	NA	NA	NA
TRIPPING CRITERIA:								
FPM	1112.82	1342.89	1560.65	1665.59	1768.22	1967.29	2159.18	2252.78
AFPM	1119.74	1988.33	1962.89	2582.50	1776.24	2821.41	2360.51	2352.61
Delta (degrees)	90.00	90.00	90.00	90.00	90.00	90.00	90.00	90.00
N	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
V	18.783	15.484	16.800	15.131	18.799	15.733	18.020	18.437
Tee:								
(i)	NA	NA	NA	NA	NA	NA	NA	NA
(ii)	NA	NA	NA	NA	NA	NA	NA	NA
(iii)	NA	NA	NA	NA	NA	NA	NA	NA
Angle	NA	NA	NA	NA	NA	NA	NA	NA
Flat Bar:								
(i)	1.5024	1.0002	1.0006	1.0013	1.0092	1.0016	1.0014	1.2427
(ii)	0.9610	0.8637	0.8717	0.8393	0.8895	0.8350	0.8634	0.9092
(iii)	0.4155	0.5040	0.4903	0.5444	0.4612	0.5511	0.5052	0.5173
Web Thks < Shell Thks (Sch.1 Pg.16)	YES	YES	YES	YES	YES	YES	YES	YES
SHEAR AREA SATISFIED	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE
PLASTIC MODULUS SATISFIED	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE
TRIPPING SATISFIED	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE
Plate Ratio	14.00	17.49	19.89	21.55	22.05	25.20	26.99	27.20
Spane Ratio	90.97	90.97	86.20	86.20	81.89	81.89	77.98	74.45
Web Ratio	9.31	12.56	11.47	13.23	10.05	12.79	10.80	10.02

DISPLACEMENT (KTonnes)	9.00	9.00	9.00	9.00	9.00	9.00	9.00	9.00
POWER (MW)	18.00	18.00	18.00	18.00	18.00	18.00	18.00	18.00
MATERIAL YIELD STRENGTH (MPa)	355.00	355.00	355.00	355.00	355.00	355.00	355.00	355.00
CAC (1,2,3, or 4)	3	3	3	3	3	3	3	3
HULL AREA (Bow or Midbody)	Bow	Bow	Bow	Bow	Bow	Bow	Bow	Bow
Arctic Class Factor	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6
Area Factor	1	1	1	1	1	1	1	1
Fmax	48.973	48.973	48.973	48.973	48.973	48.973	48.973	48.973
Vp	1.166	1.166	1.166	1.166	1.166	1.166	1.166	1.166
Hp	9.331	9.331	9.331	9.331	9.331	9.331	9.331	9.331
FRAME SPACING	400	500	600	650	700	800	900	950
FRAME SPAN	2600	2600	2600	2600	2600	2600	2600	2600
SHELL PLATE DESIGN:								
Corrosion Allowance	0	0	0	0	0	0	0	0
U	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8
Omega (Degrees)	90	90	90	90	90	90	90	90
Frame Orientation Factor	1	1	1	1	1	1	1	1
Plate Design Pressure - Dpp (MPa)	9.377	7.502	6.251	5.771	5.358	4.689	4.168	3.948
MInimum Dpp (MPa)	5.750	4.600	3.833	3.538	3.286	2.875	2.556	2.421
Dpp used for Plate Thickness (MPa)	9.377	7.502	6.251	5.771	5.358	4.689	4.168	3.948
Minimum Shell Plate Thickness (mm)	28.15	31.47	34.48	35.88	37.24	39.81	42.23	43.38
Shell Plate Thickness (mm)	31.75	31.75	34.925	36.513	38.1	41.275	42.863	44.45
TRANSVERSE FRAME DESIGN:								
Type	Flat Bar	Flat Bar	Flat Bar	Flat Bar	Flat Bar	Flat Bar	Flat Bar	Flat Bar
Dimensions:								
Web Depth (mm)	323	445	370	380	380	400	440	460
Web Thickness (mm)	31.75	31.75	34.93	36.51	38.10	39.69	39.69	39.69
Flange Width (mm)								
Flange Thickness (mm)								
Phi (degrees)	90	90	90	90	90	90	90	90
REQUIRED VALUES:								
DPT	0.043	0.054	0.064	0.070	0.075	0.086	0.096	0.102
PAV	9.549	9.171	8.847	8.701	8.565	8.319	8.102	8.003
Span LB (meters)	2.600	2.600	2.600	2.600	2.600	2.600	2.600	2.600
Spacing Between Frames S (meters)	0.4	0.5	0.6	0.65	0.7	0.8	0.9	0.95
Vp / Span	0.449	0.449	0.449	0.449	0.449	0.449	0.449	0.449
Factor A (Sch 1 Table 3)	0.806	0.806	0.806	0.806	0.806	0.806	0.806	0.806
Value H	17320	17320	17320	17320	17320	17320	17320	17320
Value B	2.976	2.976	2.976	2.976	2.976	2.976	2.976	2.976
Req. Trans. Frame Shear Area (cm2)	87.23	104.72	121.22	129.16	136.93	152.00	166.54	173.63
Req. Trans. Frame Plas. Modu. (cm3)	1727.60	2073.95	2400.69	2557.98	2711.78	3010.17	3298.08	3438.65
MINIMUM VALUES:								
Hp min	6.00	6.00	6.00	6.00	6.00	6.00	6.00	6.00
Vp min	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75
Vp min / Span	0.288	0.288	0.288	0.288	0.288	0.288	0.288	0.288
Factor A (Sch 1 Table 3)	0.873	0.873	0.873	0.873	0.873	0.873	0.873	0.873
DPT	0.067	0.083	0.100	0.108	0.117	0.133	0.150	0.158
PAV	8.781	8.372	8.036	7.890	7.755	7.515	7.307	7.213
Factor C	1	1	1	1	1	1	1	1
Factor B	2.034	2.034	2.034	2.034	2.034	2.034	2.034	2.034
Min. Trans. Frame Shear Area (cm2)	93.15	111.02	127.87	136.00	143.97	159.45	174.42	181.74
Min. Trans. Frame Plas. Modu. (cm3)	1809.42	2156.36	2483.81	2641.75	2796.40	3097.13	3387.93	3530.04
ACTUAL VALUES:								
Factor M	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Actual Shear Area (cm2)	102.55	141.29	129.22	138.75	144.78	158.75	174.62	182.56
Actual Plastic Modulus (cm3)	1819.02	3367.94	2616.27	2889.55	3026.63	3502.58	4215.94	4604.62

STRENGTH CRITERIA:								
Actual/Required (Shear Area)	1.101	1.273	1.011	1.020	1.006	0.996	1.001	1.005
Actual/Required (Plastic Modulus)	1.005	1.562	1.053	1.094	1.082	1.131	1.244	1.304
LOCAL BUCKLING CRITERIA:								
Flange Width > 5 X Web Thick (8.(1))	NA	NA	NA	NA	NA	NA	NA	NA
Local Web Buckling (8.(2))								
- Tee or Angle - Requirements	NA	NA	NA	NA	NA	NA	NA	NA
- Tee or Angle - HW / TW	NA	NA	NA	NA	NA	NA	NA	NA
- Ratio (Required / Actual)	NA	NA	NA	NA	NA	NA	NA	NA
- Are Web Stiffeners Required?	NA	NA	NA	NA	NA	NA	NA	NA
Local Web Buckling (8.(3))								
- Flat Bar - Requirements	14.967	14.967	14.967	14.967	14.967	14.967	14.967	14.967
- Flat Bar - HW / TW	10.173	14.016	10.594	10.407	9.974	10.079	11.087	11.591
- Ratio (Required / Actual)	1.471	1.068	1.413	1.438	1.501	1.485	1.350	1.291
- Are Web Stiffeners Required?	NO	NO	NO	NO	NO	NO	NO	NO
Flange Outstand (8.94))								
- Requirements	NA	NA	NA	NA	NA	NA	NA	NA
- Tee or Angle - FOS / TF	NA	NA	NA	NA	NA	NA	NA	NA
- Ratio (Required / Actual)	NA	NA	NA	NA	NA	NA	NA	NA
TRIPPING CRITERIA:								
FPM	1727.60	2073.95	2400.69	2557.98	2711.78	3010.17	3298.08	3438.65
AFPM	1819.02	3367.94	2616.27	2889.55	3026.63	3502.58	4215.94	4604.62
Delta (degrees)	90.00	90.00	90.00	90.00	90.00	90.00	90.00	90.00
N	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
V	18.362	14.785	18.048	17.728	17.835	17.467	16.665	16.282
Tee:								
(i)	NA	NA	NA	NA	NA	NA	NA	NA
(ii)	NA	NA	NA	NA	NA	NA	NA	NA
(iii)	NA	NA	NA	NA	NA	NA	NA	NA
Angle	NA	NA	NA	NA	NA	NA	NA	NA
Flat Bar:								
(i)	1.0800	1.0016	1.1066	1.3609	1.7754	2.0760	1.5628	1.4611
(ii)	0.8994	0.8107	0.8786	0.9106	0.9445	0.9543	0.9093	0.8902
(iii)	0.4722	0.5864	0.5284	0.5625	0.5834	0.6205	0.6503	0.6656
Web Thks < Shell Thks (Sch.1 Pg.16)	YES	YES	YES	YES	YES	YES	YES	YES
SHEAR AREA SATISFIED	TRUE	TRUE	TRUE	TRUE	TRUE	FALSE	TRUE	TRUE
PLASTIC MODULUS SATISFIED	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE
TRIPPING SATISFIED	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE
Plate Ratio	12.60	15.75	17.18	17.80	18.37	19.38	21.00	21.37
Spane Ratio	81.89	81.89	74.45	71.21	68.24	65.51	65.51	65.51
Web Ratio	10.17	14.02	10.59	10.41	9.97	10.08	11.09	11.59

DISPLACEMENT (KTonnes)	18.00	18.00	18.00	18.00	18.00	18.00	18.00	18.00
POWER (MW)	18.00	18.00	18.00	18.00	18.00	18.00	18.00	18.00
MATERIAL YIELD STRENGTH (MPa)	355.00	355.00	355.00	355.00	355.00	355.00	355.00	355.00
CAC (1,2,3, or 4)	3	3	3	3	3	3	3	3
HULL AREA (Bow or Midbody)	Bow	Bow	Bow	Bow	Bow	Bow	Bow	Bow
Arctic Class Factor	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6
Area Factor	1	1	1	1	1	1	1	1
Fmax	83.349	83.349	83.349	83.349	83.349	83.349	83.349	83.349
Vp	1.522	1.522	1.522	1.522	1.522	1.522	1.522	1.522
Hp	12.173	12.173	12.173	12.173	12.173	12.173	12.173	12.173
FRAME SPACING	400	500	600	650	700	800	900	1000
FRAME SPAN	2600	2600	2600	2600	2600	2600	2600	2600
SHELL PLATE DESIGN:								
Corrosion Allowance	0	0	0	0	0	0	0	0
U	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8
Omega (Degrees)	90	90	90	90	90	90	90	90
Frame Orientation Factor	1	1	1	1	1	1	1	1
Plate Design Pressure - Dpp (MPa)	9.864	7.891	6.576	6.070	5.637	4.932	4.384	3.946
MInimum Dpp (MPa)	5.750	4.600	3.833	3.538	3.286	2.875	2.556	2.300
Dpp used for Plate Thickness (MPa)	9.864	7.891	6.576	6.070	5.637	4.932	4.384	3.946
Minimum Shell Plate Thickness (mm)	28.87	32.28	35.36	36.80	38.19	40.83	43.31	45.65
Shell Plate Thickness (mm)	33.34	33.34	36.513	38.1	39.687	41.275	44.45	46.038
TRANSVERSE FRAME DESIGN:								
Type	Flat Bar	Flat Bar	Flat Bar	Flat Bar	Flat Bar	Flat Bar	Flat Bar	Flat Bar
Dimensions:								
Web Depth (mm)	385	520	430	435	444	475	485	505
Web Thickness (mm)	33.34	33.34	36.51	38.10	39.69	41.28	44.45	46.04
Flange Width (mm)								
Flange Thickness (mm)								
Phi (degrees)	90	90	90	90	90	90	90	90
REQUIRED VALUES:								
DPT	0.033	0.041	0.049	0.053	0.058	0.066	0.074	0.082
PAV	9.964	9.619	9.316	9.177	9.047	8.807	8.592	8.398
Span LB (meters)	2.600	2.600	2.600	2.600	2.600	2.600	2.600	2.600
Spacing Between Frames S (meters)	0.4	0.5	0.6	0.65	0.7	0.8	0.9	1
Vp / Span	0.585	0.585	0.585	0.585	0.585	0.585	0.585	0.585
Factor A (Sch 1 Table 3)	0.751	0.751	0.751	0.751	0.751	0.751	0.751	0.751
Value H	17320	17320	17320	17320	17320	17320	17320	17320
Value B	3.674	3.674	3.674	3.674	3.674	3.674	3.674	3.674
Req. Trans. Frame Shear Area (cm2)	110.72	133.61	155.27	165.72	175.93	195.73	214.82	233.31
Req. Trans. Frame Plas. Modu. (cm3)	2225.63	2685.77	3121.29	3331.19	3536.45	3934.59	4318.37	4689.91
MINIMUM VALUES:								
Hp min	6.00	6.00	6.00	6.00	6.00	6.00	6.00	6.00
Vp min	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75
Vp min / Span	0.288	0.288	0.288	0.288	0.288	0.288	0.288	0.288
Factor A (Sch 1 Table 3)	0.873	0.873	0.873	0.873	0.873	0.873	0.873	0.873
DPT	0.067	0.083	0.100	0.108	0.117	0.133	0.150	0.167
PAV	8.781	8.372	8.036	7.890	7.755	7.515	7.307	7.124
Factor C	1	1	1	1	1	1	1	1
Factor B	2.034	2.034	2.034	2.034	2.034	2.034	2.034	2.034
Min. Trans. Frame Shear Area (cm2)	93.15	111.02	127.87	136.00	143.97	159.45	174.42	188.95
Min. Trans. Frame Plas. Modu. (cm3)	1809.42	2156.36	2483.81	2641.75	2796.40	3097.13	3387.93	3670.09
ACTUAL VALUES:								
Factor M	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Actual Shear Area (cm2)	128.36	173.37	157.01	165.74	176.21	196.06	215.58	232.49
Actual Plastic Modulus (cm3)	2684.89	4796.57	3662.26	3920.46	4261.53	5060.95	5707.01	6405.59

STRENGTH CRITERIA:								
Actual/Required (Shear Area)	1.159	1.298	1.011	1.000	1.002	1.002	1.004	0.997
Actual/Required (Plastic Modulus)	1.206	1.786	1.173	1.177	1.205	1.286	1.322	1.366
LOCAL BUCKLING CRITERIA:								
Flange Width > 5 X Web Thick (8.(1))	NA	NA	NA	NA	NA	NA	NA	NA
Local Web Buckling (8.(2))								
- Tee or Angle - Requirements	NA	NA	NA	NA	NA	NA	NA	NA
- Tee or Angle - HW / TW	NA	NA	NA	NA	NA	NA	NA	NA
- Ratio (Required / Actual)	NA	NA	NA	NA	NA	NA	NA	NA
- Are Web Stiffeners Required?	NA	NA	NA	NA	NA	NA	NA	NA
Local Web Buckling (8.(3))								
- Flat Bar - Requirements	14.967	14.967	14.967	14.967	14.967	14.967	14.967	14.967
- Flat Bar - HW / TW	11.548	15.597	11.777	11.417	11.188	11.508	10.911	10.969
- Ratio (Required / Actual)	1.296	0.960	1.271	1.311	1.338	1.301	1.372	1.364
- Are Web Stiffeners Required?	NO	YES	NO	NO	NO	NO	NO	NO
Flange Outstand (8.94))								
- Requirements	NA	NA	NA	NA	NA	NA	NA	NA
- Tee or Angle - FOS / TF	NA	NA	NA	NA	NA	NA	NA	NA
- Ratio (Required / Actual)	NA	NA	NA	NA	NA	NA	NA	NA
TRIPPING CRITERIA:								
FPM	2225.63	2685.77	3121.29	3331.19	3536.45	3934.59	4318.37	4689.91
AFPM	2684.89	4796.57	3662.26	3920.46	4261.53	5060.95	5707.01	6405.59
Delta (degrees)	90.00	90.00	90.00	90.00	90.00	90.00	90.00	90.00
N	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
V	17.155	14.099	17.394	17.368	17.164	16.613	16.390	16.122
Tee:								
(i)	NA	NA	NA	NA	NA	NA	NA	NA
(ii)	NA	NA	NA	NA	NA	NA	NA	NA
(iii)	NA	NA	NA	NA	NA	NA	NA	NA
Angle	NA	NA	NA	NA	NA	NA	NA	NA
Flat Bar:								
(i)	1.0016	1.0008	1.0018	1.1277	1.3038	1.4214	2.1610	2.4970
(ii)	0.8481	0.7640	0.8201	0.8472	0.8749	0.8787	0.9394	0.9500
(iii)	0.5307	0.6458	0.5732	0.5991	0.6314	0.6785	0.7406	0.7798
Web Thks < Shell Thks (Sch.1 Pg.16)	YES	YES	YES	YES	YES	YES	YES	YES
SHEAR AREA SATISFIED	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	FALSE
PLASTIC MODULUS SATISFIED	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE
TRIPPING SATISFIED	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE
Plate Ratio	12.00	15.00	16.43	17.06	17.64	19.38	20.25	21.72
Spane Ratio	77.98	77.98	71.21	68.24	65.51	62.99	58.49	56.48
Web Ratio	11.55	15.60	11.78	11.42	11.19	11.51	10.91	10.97

DISPLACEMENT (KTonnes)	6.00	6.00	6.00	6.00	6.00	6.00	6.00	6.00
POWER (MW)	12.00	12.00	12.00	12.00	12.00	12.00	12.00	12.00
MATERIAL YIELD STRENGTH (MPa)	355.00	355.00	355.00	355.00	355.00	355.00	355.00	355.00
CAC (1,2,3, or 4)	4	4	4	4	4	4	4	4
HULL AREA (Bow or Midbody)	Midbody	Midbody	Midbody	Midbody	Midbody	Midbody	Midbody	Midbody
Arctic Class Factor	0.4	0.4	0.4	0.4	0.4	0.4	0.4	0.4
Area Factor	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5
Fmax	33.078	33.078	33.078	33.078	33.078	33.078	33.078	33.078
Vp	0.959	0.959	0.959	0.959	0.959	0.959	0.959	0.959
Hp	7.669	7.669	7.669	7.669	7.669	7.669	7.669	7.669
FRAME SPACING	400	500	600	650	700	800	900	1000
FRAME SPAN	2600	2600	2600	2600	2600	2600	2600	2600
SHELL PLATE DESIGN:								
Corrosion Allowance	0	0	0	0	0	0	0	0
U	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5
Omega (Degrees)	90	90	90	90	90	90	90	90
Frame Orientation Factor	1	1	1	1	1	1	1	1
Plate Design Pressure - Dpp (MPa)	3.051	2.441	2.034	1.877	1.743	1.525	1.356	1.220
MInimum Dpp (MPa)	4.500	3.600	3.000	2.769	2.571	2.250	2.000	1.800
Dpp used for Plate Thickness (MPa)	4.500	3.600	3.000	2.769	2.571	2.250	2.000	1.800
Minimum Shell Plate Thickness (mm)	19.50	21.80	23.88	24.86	25.80	27.58	29.25	30.83
Shell Plate Thickness (mm)	20.64	22.23	25.4	28.58	28.58	28.58	30.163	31.75
TRANSVERSE FRAME DESIGN:								
Type	Flat Bar	Flat Bar	Flat Bar	Flat Bar	Flat Bar	Flat Bar	Flat Bar	Flat Bar
Dimensions:								
Web Depth (mm)	318	273	300	330	289	305	300	311
Web Thickness (mm)	20.64	22.23	22.23	25.40	25.40	25.40	28.58	28.58
Flange Width (mm)								
Flange Thickness (mm)								
Phi (degrees)	90	90	90	90	90	90	90	90
REQUIRED VALUES:								
DPT	0.052	0.065	0.078	0.085	0.091	0.104	0.117	0.130
PAV	9.218	8.822	8.488	8.341	8.204	7.959	7.744	7.555
Span LB (meters)	2.600	2.600	2.600	2.600	2.600	2.600	2.600	2.600
Spacing Between Frames S (meters)	0.4	0.5	0.6	0.65	0.7	0.8	0.9	1
Vp / Span	0.369	0.369	0.369	0.369	0.369	0.369	0.369	0.369
Factor A (Sch 1 Table 3)	0.839	0.839	0.839	0.839	0.839	0.839	0.839	0.839
Value H	17320	17320	17320	17320	17320	17320	17320	17320
Value B	2.522	2.522	2.522	2.522	2.522	2.522	2.522	2.522
Req. Trans. Frame Shear Area (cm2)	24.02	28.74	33.18	35.32	37.42	41.48	45.41	49.22
Req. Trans. Frame Plas. Modu. (cm3)	471.16	563.62	650.78	692.76	733.82	813.57	890.61	965.38
MINIMUM VALUES:								
Hp min	6.00	6.00	6.00	6.00	6.00	6.00	6.00	6.00
Vp min	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75
Vp min / Span	0.288	0.288	0.288	0.288	0.288	0.288	0.288	0.288
Factor A (Sch 1 Table 3)	0.873	0.873	0.873	0.873	0.873	0.873	0.873	0.873
DPT	0.067	0.083	0.100	0.108	0.117	0.133	0.150	0.167
PAV	8.781	8.372	8.036	7.890	7.755	7.515	7.307	7.124
Factor C	0.83	0.83	0.83	0.83	0.83	0.83	0.83	0.83
Factor B	2.034	2.034	2.034	2.034	2.034	2.034	2.034	2.034
Min. Trans. Frame Shear Area (cm2)	38.66	46.07	53.07	56.44	59.75	66.17	72.38	78.41
Min. Trans. Frame Plas. Modu. (cm3)	750.91	894.89	1030.78	1096.33	1160.51	1285.31	1405.99	1523.09
ACTUAL VALUES:								
Factor M	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Actual Shear Area (cm2)	65.64	60.69	66.69	83.82	73.41	77.47	85.74	88.88
Actual Plastic Modulus (cm3)	1111.34	895.84	1085.05	1502.81	1165.61	1292.12	1415.41	1523.25

STRENGTH CRITERIA:								
Actual/Required (Shear Area)	1.698	1.317	1.257	1.485	1.229	1.171	1.185	1.134
Actual/Required (Plastic Modulus)	1.480	1.001	1.053	1.371	1.004	1.005	1.007	1.000
LOCAL BUCKLING CRITERIA:								
Flange Width > 5 X Web Thick (8.(1))	NA	NA	NA	NA	NA	NA	NA	NA
Local Web Buckling (8.(2))								
- Tee or Angle - Requirements	NA	NA	NA	NA	NA	NA	NA	NA
- Tee or Angle - HW / TW	NA	NA	NA	NA	NA	NA	NA	NA
- Ratio (Required / Actual)	NA	NA	NA	NA	NA	NA	NA	NA
- Are Web Stiffeners Required?	NA	NA	NA	NA	NA	NA	NA	NA
Local Web Buckling (8.(3))								
- Flat Bar - Requirements	14.967	14.967	14.967	14.967	14.967	14.967	14.967	14.967
- Flat Bar - HW / TW	15.407	12.281	13.495	12.992	11.378	12.008	10.497	10.882
- Ratio (Required / Actual)	0.971	1.219	1.109	1.152	1.315	1.246	1.426	1.375
- Are Web Stiffeners Required?	YES	NO	NO	NO	NO	NO	NO	NO
Flange Outstand (8.94))								
- Requirements	NA	NA	NA	NA	NA	NA	NA	NA
- Tee or Angle - FOS / TF	NA	NA	NA	NA	NA	NA	NA	NA
- Ratio (Required / Actual)	NA	NA	NA	NA	NA	NA	NA	NA
TRIPPING CRITERIA:								
FPM	471.16	563.62	650.78	692.76	733.82	813.57	890.61	965.38
AFPM	1111.34	895.84	1085.05	1502.81	1165.61	1292.12	1415.41	1523.25
Delta (degrees)	90.00	90.00	90.00	90.00	90.00	90.00	90.00	90.00
N	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
V	12.268	14.945	14.592	12.792	14.950	14.951	14.946	15.000
Tee:								
(i)	NA	NA	NA	NA	NA	NA	NA	NA
(ii)	NA	NA	NA	NA	NA	NA	NA	NA
(iii)	NA	NA	NA	NA	NA	NA	NA	NA
Angle	NA	NA	NA	NA	NA	NA	NA	NA
Flat Bar:								
(i)	1.0026	1.0089	0.7975	#NUM!	2.9641	1.3160	#NUM!	#NUM!
(ii)	0.8888	0.9154	0.8531	1.0108	0.9877	0.9358	1.0709	1.0293
(iii)	0.4594	0.4062	0.4160	0.5422	0.4640	0.4639	0.5222	0.5203
Web Thks < Shell Thks (Sch.1 Pg.16)	YES	YES	YES	YES	YES	YES	YES	YES
SHEAR AREA SATISFIED	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE
PLASTIC MODULUS SATISFIED	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE
TRIPPING SATISFIED	TRUE	TRUE	FALSE	#NUM!	TRUE	TRUE	#NUM!	#NUM!
Plate Ratio	19.38	22.49	23.62	22.74	24.49	27.99	29.84	31.50
Spane Ratio	125.97	116.96	116.96	102.36	102.36	102.36	90.97	90.97
Web Ratio	15.41	12.28	13.50	12.99	11.38	12.01	10.50	10.88

DISPLACEMENT (KTonnes)	12.00	12.00	12.00	12.00	12.00	12.00	12.00	12.00
POWER (MW)	12.00	12.00	12.00	12.00	12.00	12.00	12.00	12.00
MATERIAL YIELD STRENGTH (MPa)	355.00	355.00	355.00	355.00	355.00	355.00	355.00	355.00
CAC (1,2,3, or 4)	4	4	4	4	4	4	4	4
HULL AREA (Bow or Midbody)	Midbody	Midbody	Midbody	Midbody	Midbody	Midbody	Midbody	Midbody
Arctic Class Factor	0.4	0.4	0.4	0.4	0.4	0.4	0.4	0.4
Area Factor	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5
Fmax	56.391	56.391	56.391	56.391	56.391	56.391	56.391	56.391
Vp	1.252	1.252	1.252	1.252	1.252	1.252	1.252	1.252
Hp	10.013	10.013	10.013	10.013	10.013	10.013	10.013	10.013
FRAME SPACING	400	500	600	650	700	800	900	1000
FRAME SPAN	2600	2600	2600	2600	2600	2600	2600	2600
SHELL PLATE DESIGN:								
Corrosion Allowance	0	0	0	0	0	0	0	0
U	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5
Omega (Degrees)	90	90	90	90	90	90	90	90
Frame Orientation Factor	1	1	1	1	1	1	1	1
Plate Design Pressure - Dpp (MPa)	3.161	2.529	2.107	1.945	1.806	1.581	1.405	1.264
MInimum Dpp (MPa)	4.500	3.600	3.000	2.769	2.571	2.250	2.000	1.800
Dpp used for Plate Thickness (MPa)	4.500	3.600	3.000	2.769	2.571	2.250	2.000	1.800
Minimum Shell Plate Thickness (mm)	19.50	21.80	23.88	24.86	25.80	27.58	29.25	30.83
Shell Plate Thickness (mm)	25.4	25.4	25.4	25.4	28.58	28.58	30.163	31.75
TRANSVERSE FRAME DESIGN:								
Type	Flat Bar	Flat Bar	Flat Bar	Flat Bar	Flat Bar	Flat Bar	Flat Bar	Flat Bar
Dimensions:								
Web Depth (mm)	231	253	283	343	271	286	299	311
Web Thickness (mm)	25.40	25.40	25.40	25.40	28.58	28.58	28.58	28.58
Flange Width (mm)								
Flange Thickness (mm)								
Phi (degrees)	90	90	90	90	90	90	90	90
REQUIRED VALUES:								
DPT	0.040	0.050	0.060	0.065	0.070	0.080	0.090	0.100
PAV	9.664	9.293	8.973	8.829	8.695	8.450	8.232	8.038
Span LB (meters)	2.600	2.600	2.600	2.600	2.600	2.600	2.600	2.600
Spacing Between Frames S (meters)	0.4	0.5	0.6	0.65	0.7	0.8	0.9	1
Vp / Span	0.481	0.481	0.481	0.481	0.481	0.481	0.481	0.481
Factor A (Sch 1 Table 3)	0.792	0.792	0.792	0.792	0.792	0.792	0.792	0.792
Value H	17320	17320	17320	17320	17320	17320	17320	17320
Value B	3.152	3.152	3.152	3.152	3.152	3.152	3.152	3.152
Req. Trans. Frame Shear Area (cm2)	31.05	37.33	43.25	46.10	48.89	54.30	59.52	64.57
Req. Trans. Frame Plas. Modu. (cm3)	617.29	742.05	859.81	916.52	971.95	1079.48	1183.20	1283.70
MINIMUM VALUES:								
Hp min	6.00	6.00	6.00	6.00	6.00	6.00	6.00	6.00
Vp min	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75
Vp min / Span	0.288	0.288	0.288	0.288	0.288	0.288	0.288	0.288
Factor A (Sch 1 Table 3)	0.873	0.873	0.873	0.873	0.873	0.873	0.873	0.873
DPT	0.067	0.083	0.100	0.108	0.117	0.133	0.150	0.167
PAV	8.781	8.372	8.036	7.890	7.755	7.515	7.307	7.124
Factor C	0.83	0.83	0.83	0.83	0.83	0.83	0.83	0.83
Factor B	2.034	2.034	2.034	2.034	2.034	2.034	2.034	2.034
Min. Trans. Frame Shear Area (cm2)	38.66	46.07	53.07	56.44	59.75	66.17	72.38	78.41
Min. Trans. Frame Plas. Modu. (cm3)	750.91	894.89	1030.78	1096.33	1160.51	1285.31	1405.99	1523.09
ACTUAL VALUES:								
Factor M	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Actual Shear Area (cm2)	58.67	64.26	71.88	87.12	77.45	81.74	85.45	88.88
Actual Plastic Modulus (cm3)	752.20	894.53	1108.42	1604.79	1160.15	1285.67	1406.42	1523.25

STRENGTH CRITERIA:								
Actual/Required (Shear Area)	1.518	1.395	1.355	1.544	1.296	1.235	1.181	1.134
Actual/Required (Plastic Modulus)	1.002	1.000	1.075	1.464	1.000	1.000	1.000	1.000
LOCAL BUCKLING CRITERIA:								
Flange Width > 5 X Web Thick (8.(1))	NA	NA	NA	NA	NA	NA	NA	NA
Local Web Buckling (8.(2))								
- Tee or Angle - Requirements	NA	NA	NA	NA	NA	NA	NA	NA
- Tee or Angle - HW / TW	NA	NA	NA	NA	NA	NA	NA	NA
- Ratio (Required / Actual)	NA	NA	NA	NA	NA	NA	NA	NA
- Are Web Stiffeners Required?	NA	NA	NA	NA	NA	NA	NA	NA
Local Web Buckling (8.(3))								
- Flat Bar - Requirements	14.967	14.967	14.967	14.967	14.967	14.967	14.967	14.967
- Flat Bar - HW / TW	9.094	9.961	11.142	13.504	9.482	10.007	10.462	10.882
- Ratio (Required / Actual)	1.646	1.503	1.343	1.108	1.578	1.496	1.431	1.375
- Are Web Stiffeners Required?	NO	NO	NO	NO	NO	NO	NO	NO
Flange Outstand (8.94))								
- Requirements	NA	NA	NA	NA	NA	NA	NA	NA
- Tee or Angle - FOS / TF	NA	NA	NA	NA	NA	NA	NA	NA
- Ratio (Required / Actual)	NA	NA	NA	NA	NA	NA	NA	NA
TRIPPING CRITERIA:								
FPM	617.29	742.05	859.81	916.52	971.95	1079.48	1183.20	1283.70
AFPM	752.20	894.53	1108.42	1604.79	1160.15	1285.67	1406.42	1523.25
Delta (degrees)	90.00	90.00	90.00	90.00	90.00	90.00	90.00	90.00
N	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
V	17.068	17.161	16.595	14.239	17.246	17.265	17.282	17.297
Tee:								
(i)	NA	NA	NA	NA	NA	NA	NA	NA
(ii)	NA	NA	NA	NA	NA	NA	NA	NA
(iii)	NA	NA	NA	NA	NA	NA	NA	NA
Angle	NA	NA	NA	NA	NA	NA	NA	NA
Flat Bar:								
(i)	#NUM!	2.1921	1.0010	1.0014	#NUM!	1.9377	1.2220	1.0008
(ii)	1.0823	0.9829	0.9086	0.8737	1.0274	0.9724	0.9292	0.8926
(iii)	0.4064	0.4042	0.4180	0.4871	0.4526	0.4521	0.4516	0.4512
Web Thks < Shell Thks (Sch.1 Pg.16)	YES	YES	YES	YES	YES	YES	YES	YES
SHEAR AREA SATISFIED	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE
PLASTIC MODULUS SATISFIED	TRUE	FALSE	TRUE	TRUE	FALSE	TRUE	TRUE	TRUE
TRIPPING SATISFIED	#NUM!	TRUE	TRUE	TRUE	#NUM!	TRUE	TRUE	TRUE
Plate Ratio	15.75	19.69	23.62	25.59	24.49	27.99	29.84	31.50
Spane Ratio	102.36	102.36	102.36	102.36	90.97	90.97	90.97	90.97
Web Ratio	9.09	9.96	11.14	13.50	9.48	10.01	10.46	10.88

DISPLACEMENT (KTonnes)	6.00	6.00	6.00	6.00	6.00	6.00	6.00	6.00
POWER (MW)	12.00	12.00	12.00	12.00	12.00	12.00	12.00	12.00
MATERIAL YIELD STRENGTH (MPa)	355.00	355.00	355.00	355.00	355.00	355.00	355.00	355.00
CAC (1,2,3, or 4)	4	4	4	4	4	4	4	4
HULL AREA (Bow or Midbody)	Bow	Bow	Bow	Bow	Bow	Bow	Bow	Bow
Arctic Class Factor	0.4	0.4	0.4	0.4	0.4	0.4	0.4	0.4
Area Factor	1	1	1	1	1	1	1	1
Fmax	33.078	33.078	33.078	33.078	33.078	33.078	33.078	33.078
Vp	0.959	0.959	0.959	0.959	0.959	0.959	0.959	0.959
Hp	7.669	7.669	7.669	7.669	7.669	7.669	7.669	7.669
FRAME SPACING	400	500	600	650	700	800	900	1000
FRAME SPAN	2600	2600	2600	2600	2600	2600	2600	2600
SHELL PLATE DESIGN:								
Corrosion Allowance	0	0	0	0	0	0	0	0
U	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8
Omega (Degrees)	90	90	90	90	90	90	90	90
Frame Orientation Factor	1	1	1	1	1	1	1	1
Plate Design Pressure - Dpp (MPa)	6.102	4.881	4.068	3.755	3.487	3.051	2.712	2.441
MInimum Dpp (MPa)	4.500	3.600	3.000	2.769	2.571	2.250	2.000	1.800
Dpp used for Plate Thickness (MPa)	6.102	4.881	4.068	3.755	3.487	3.051	2.712	2.441
Minimum Shell Plate Thickness (mm)	22.71	25.39	27.81	28.95	30.04	32.11	34.06	35.90
Shell Plate Thickness (mm)	25.4	28.58	28.58	30.163	31.75	33.34	34.93	36.513
TRANSVERSE FRAME DESIGN:								
Type	Flat Bar	Flat Bar	Flat Bar	Flat Bar	Flat Bar	Flat Bar	Flat Bar	Flat Bar
Dimensions:								
Web Depth (mm)	366	340	372	395	377	420	435	471
Web Thickness (mm)	25.40	28.58	28.58	28.58	31.75	31.75	33.34	33.34
Flange Width (mm)								
Flange Thickness (mm)								
Phi (degrees)	90	90	90	90	90	90	90	90
REQUIRED VALUES:								
DPT	0.052	0.065	0.078	0.085	0.091	0.104	0.117	0.130
PAV	9.218	8.822	8.488	8.341	8.204	7.959	7.744	7.555
Span LB (meters)	2.600	2.600	2.600	2.600	2.600	2.600	2.600	2.600
Spacing Between Frames S (meters)	0.4	0.5	0.6	0.65	0.7	0.8	0.9	1
Vp / Span	0.369	0.369	0.369	0.369	0.369	0.369	0.369	0.369
Factor A (Sch 1 Table 3)	0.839	0.839	0.839	0.839	0.839	0.839	0.839	0.839
Value H	17320	17320	17320	17320	17320	17320	17320	17320
Value B	2.522	2.522	2.522	2.522	2.522	2.522	2.522	2.522
Req. Trans. Frame Shear Area (cm2)	48.05	57.48	66.37	70.65	74.83	82.97	90.82	98.45
Req. Trans. Frame Plas. Modu. (cm3)	942.32	1127.24	1301.56	1385.52	1467.64	1627.13	1781.22	1930.76
MINIMUM VALUES:								
Hp min	6.00	6.00	6.00	6.00	6.00	6.00	6.00	6.00
Vp min	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75
Vp min / Span	0.288	0.288	0.288	0.288	0.288	0.288	0.288	0.288
Factor A (Sch 1 Table 3)	0.873	0.873	0.873	0.873	0.873	0.873	0.873	0.873
DPT	0.067	0.083	0.100	0.108	0.117	0.133	0.150	0.167
PAV	8.781	8.372	8.036	7.890	7.755	7.515	7.307	7.124
Factor C	0.83	0.83	0.83	0.83	0.83	0.83	0.83	0.83
Factor B	2.034	2.034	2.034	2.034	2.034	2.034	2.034	2.034
Min. Trans. Frame Shear Area (cm2)	77.32	92.14	106.14	112.88	119.49	132.34	144.77	156.83
Min. Trans. Frame Plas. Modu. (cm3)	1501.82	1789.78	2061.57	2192.65	2321.02	2570.61	2811.99	3046.18
ACTUAL VALUES:								
Factor M	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Actual Shear Area (cm2)	92.96	97.17	106.32	112.89	119.70	133.35	145.03	157.03
Actual Plastic Modulus (cm3)	1819.31	1790.78	2129.44	2399.85	2446.32	3022.64	3407.67	3984.77

STRENGTH CRITERIA:								
Actual/Required (Shear Area)	1.202	1.055	1.002	1.000	1.002	1.008	1.002	1.001
Actual/Required (Plastic Modulus)	1.211	1.001	1.033	1.094	1.054	1.176	1.212	1.308
LOCAL BUCKLING CRITERIA:								
Flange Width > 5 X Web Thick (8.(1))	NA	NA	NA	NA	NA	NA	NA	NA
Local Web Buckling (8.(2))								
- Tee or Angle - Requirements	NA	NA	NA	NA	NA	NA	NA	NA
- Tee or Angle - HW / TW	NA	NA	NA	NA	NA	NA	NA	NA
- Ratio (Required / Actual)	NA	NA	NA	NA	NA	NA	NA	NA
- Are Web Stiffeners Required?	NA	NA	NA	NA	NA	NA	NA	NA
Local Web Buckling (8.(3))								
- Flat Bar - Requirements	14.967	14.967	14.967	14.967	14.967	14.967	14.967	14.967
- Flat Bar - HW / TW	14.409	11.896	13.016	13.821	11.874	13.228	13.047	14.127
- Ratio (Required / Actual)	1.039	1.258	1.150	1.083	1.260	1.131	1.147	1.059
- Are Web Stiffeners Required?	NO	NO	NO	NO	NO	NO	NO	NO
Flange Outstand (8.94))								
- Requirements	NA	NA	NA	NA	NA	NA	NA	NA
- Tee or Angle - FOS / TF	NA	NA	NA	NA	NA	NA	NA	NA
- Ratio (Required / Actual)	NA	NA	NA	NA	NA	NA	NA	NA
TRIPPING CRITERIA:								
FPM	942.32	1127.24	1301.56	1385.52	1467.64	1627.13	1781.22	1930.76
AFPM	1819.31	1790.78	2129.44	2399.85	2446.32	3022.64	3407.67	3984.77
Delta (degrees)	90.00	90.00	90.00	90.00	90.00	90.00	90.00	90.00
N	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
V	13.560	14.949	14.730	14.316	14.594	13.824	13.622	13.115
Tee:								
(i)	NA	NA	NA	NA	NA	NA	NA	NA
(ii)	NA	NA	NA	NA	NA	NA	NA	NA
(iii)	NA	NA	NA	NA	NA	NA	NA	NA
Angle	NA	NA	NA	NA	NA	NA	NA	NA
Flat Bar:								
(i)	1.0018	1.5920	1.0994	1.0320	2.4237	1.5879	2.0478	1.6461
(ii)	0.8598	0.9447	0.8762	0.8491	0.9695	0.9187	0.9452	0.9067
(iii)	0.5115	0.5221	0.5298	0.5452	0.5941	0.6272	0.6684	0.6942
Web Thks < Shell Thks (Sch.1 Pg.16)	YES	YES	YES	YES	YES	YES	YES	YES
SHEAR AREA SATISFIED	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE
PLASTIC MODULUS SATISFIED	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE
TRIPPING SATISFIED	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE
Plate Ratio	15.75	17.49	20.99	21.55	22.05	24.00	25.77	27.39
Spane Ratio	102.36	90.97	90.97	90.97	81.89	81.89	77.98	77.98
Web Ratio	14.41	11.90	13.02	13.82	11.87	13.23	13.05	14.13

DISPLACEMENT (KTonnes)	12.00	12.00	12.00	12.00	12.00	12.00	12.00	12.00
POWER (MW)	12.00	12.00	12.00	12.00	12.00	12.00	12.00	12.00
MATERIAL YIELD STRENGTH (MPa)	355.00	355.00	355.00	355.00	355.00	355.00	355.00	355.00
CAC (1,2,3, or 4)	4	4	4	4	4	4	4	4
HULL AREA (Bow or Midbody)	Bow	Bow	Bow	Bow	Bow	Bow	Bow	Bow
Arctic Class Factor	0.4	0.4	0.4	0.4	0.4	0.4	0.4	0.4
Area Factor	1	1	1	1	1	1	1	1
Fmax	56.391	56.391	56.391	56.391	56.391	56.391	56.391	56.391
Vp	1.252	1.252	1.252	1.252	1.252	1.252	1.252	1.252
Hp	10.013	10.013	10.013	10.013	10.013	10.013	10.013	10.013
FRAME SPACING	400	500	600	650	700	800	900	1000
FRAME SPAN	2600	2600	2600	2600	2600	2600	2600	2600
SHELL PLATE DESIGN:								
Corrosion Allowance	0	0	0	0	0	0	0	0
U	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8
Omega (Degrees)	90	90	90	90	90	90	90	90
Frame Orientation Factor	1	1	1	1	1	1	1	1
Plate Design Pressure - Dpp (MPa)	6.322	5.058	4.215	3.890	3.613	3.161	2.810	2.529
MInimum Dpp (MPa)	4.500	3.600	3.000	2.769	2.571	2.250	2.000	1.800
Dpp used for Plate Thickness (MPa)	6.322	5.058	4.215	3.890	3.613	3.161	2.810	2.529
Minimum Shell Plate Thickness (mm)	23.11	25.84	28.31	29.46	30.58	32.69	34.67	36.55
Shell Plate Thickness (mm)	28.58	30.163	31.75	31.75	31.75	33.34	34.93	38.1
TRANSVERSE FRAME DESIGN:								
Type	Flat Bar	Flat Bar	Flat Bar	Flat Bar	Flat Bar	Flat Bar	Flat Bar	Flat Bar
Dimensions:								
Web Depth (mm)	311	330	345	360	397	400	415	420
Web Thickness (mm)	28.58	30.16	31.75	31.75	31.75	33.34	34.93	38.10
Flange Width (mm)								
Flange Thickness (mm)								
Phi (degrees)	90	90	90	90	90	90	90	90
REQUIRED VALUES:								
DPT	0.040	0.050	0.060	0.065	0.070	0.080	0.090	0.100
PAV	9.664	9.293	8.973	8.829	8.695	8.450	8.232	8.038
Span LB (meters)	2.600	2.600	2.600	2.600	2.600	2.600	2.600	2.600
Spacing Between Frames S (meters)	0.4	0.5	0.6	0.65	0.7	0.8	0.9	1
Vp / Span	0.481	0.481	0.481	0.481	0.481	0.481	0.481	0.481
Factor A (Sch 1 Table 3)	0.792	0.792	0.792	0.792	0.792	0.792	0.792	0.792
Value H	17320	17320	17320	17320	17320	17320	17320	17320
Value B	3.152	3.152	3.152	3.152	3.152	3.152	3.152	3.152
Req. Trans. Frame Shear Area (cm2)	62.10	74.65	86.50	92.20	97.78	108.60	119.03	129.14
Req. Trans. Frame Plas. Modu. (cm3)	1234.58	1484.10	1719.63	1833.03	1943.90	2158.97	2366.40	2567.40
MINIMUM VALUES:								
Hp min	6.00	6.00	6.00	6.00	6.00	6.00	6.00	6.00
Vp min	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75
Vp min / Span	0.288	0.288	0.288	0.288	0.288	0.288	0.288	0.288
Factor A (Sch 1 Table 3)	0.873	0.873	0.873	0.873	0.873	0.873	0.873	0.873
DPT	0.067	0.083	0.100	0.108	0.117	0.133	0.150	0.167
PAV	8.781	8.372	8.036	7.890	7.755	7.515	7.307	7.124
Factor C	0.83	0.83	0.83	0.83	0.83	0.83	0.83	0.83
Factor B	2.034	2.034	2.034	2.034	2.034	2.034	2.034	2.034
Min. Trans. Frame Shear Area (cm2)	77.32	92.14	106.14	112.88	119.49	132.34	144.77	156.83
Min. Trans. Frame Plas. Modu. (cm3)	1501.82	1789.78	2061.57	2192.65	2321.02	2570.61	2811.99	3046.18
ACTUAL VALUES:								
Factor M	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Actual Shear Area (cm2)	88.88	99.54	109.54	114.30	126.05	133.36	144.96	160.02
Actual Plastic Modulus (cm3)	1509.16	1792.49	2063.41	2238.85	2702.14	2889.51	3261.08	3665.26

STRENGTH CRITERIA:								
Actual/Required (Shear Area)	1.150	1.080	1.032	1.013	1.055	1.008	1.001	1.020
Actual/Required (Plastic Modulus)	1.005	1.002	1.001	1.021	1.164	1.124	1.160	1.203
LOCAL BUCKLING CRITERIA:								
Flange Width > 5 X Web Thick (8.(1))	NA	NA	NA	NA	NA	NA	NA	NA
Local Web Buckling (8.(2))								
- Tee or Angle - Requirements	NA	NA	NA	NA	NA	NA	NA	NA
- Tee or Angle - HW / TW	NA	NA	NA	NA	NA	NA	NA	NA
- Ratio (Required / Actual)	NA	NA	NA	NA	NA	NA	NA	NA
- Are Web Stiffeners Required?	NA	NA	NA	NA	NA	NA	NA	NA
Local Web Buckling (8.(3))								
- Flat Bar - Requirements	14.967	14.967	14.967	14.967	14.967	14.967	14.967	14.967
- Flat Bar - HW / TW	10.882	10.941	10.866	11.339	12.504	11.998	11.881	11.024
- Ratio (Required / Actual)	1.375	1.368	1.377	1.320	1.197	1.247	1.260	1.358
- Are Web Stiffeners Required?	NO	NO	NO	NO	NO	NO	NO	NO
Flange Outstand (8.94))								
- Requirements	NA	NA	NA	NA	NA	NA	NA	NA
- Tee or Angle - FOS / TF	NA	NA	NA	NA	NA	NA	NA	NA
- Ratio (Required / Actual)	NA	NA	NA	NA	NA	NA	NA	NA
TRIPPING CRITERIA:								
FPM	1234.58	1484.10	1719.63	1833.03	1943.90	2158.97	2366.40	2567.40
AFPM	1509.16	1792.49	2063.41	2238.85	2702.14	2889.51	3261.08	3665.26
Delta (degrees)	90.00	90.00	90.00	90.00	90.00	90.00	90.00	90.00
N	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
V	17.041	17.144	17.200	17.049	15.981	16.286	16.050	15.769
Tee:								
(i)	NA	NA	NA	NA	NA	NA	NA	NA
(ii)	NA	NA	NA	NA	NA	NA	NA	NA
(iii)	NA	NA	NA	NA	NA	NA	NA	NA
Angle	NA	NA	NA	NA	NA	NA	NA	NA
Flat Bar:								
(i)	1.0817	1.0804	1.1503	1.0281	1.0021	1.0947	1.2562	2.5685
(ii)	0.9059	0.8957	0.8989	0.8691	0.8407	0.8598	0.8810	0.9664
(iii)	0.4580	0.4804	0.5041	0.5086	0.5425	0.5590	0.5943	0.6598
Web Thks < Shell Thks (Sch.1 Pg.16)	YES	YES	YES	YES	YES	YES	YES	YES
SHEAR AREA SATISFIED	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE
PLASTIC MODULUS SATISFIED	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE
TRIPPING SATISFIED	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE
Plate Ratio	14.00	16.58	18.90	20.47	22.05	24.00	25.77	26.25
Spane Ratio	90.97	86.20	81.89	81.89	81.89	77.98	74.43	68.24
Web Ratio	10.88	10.94	10.87	11.34	12.50	12.00	11.88	11.02

"t_w" is the thickness of the web in centimetres; and

"f_y" is the nominal yield stress of the steel in megapascals.

23.6 Subject to paragraph 23.7, on framing members that do not comply with paragraphs 23.2, 23.3, 22.4 and 23.5, as appropriate, additional stiffening must be fitted with spacing and scantlings designed to prevent elastic buckling.

23.7 Framing members that do not comply with paragraphs 23.2, 23.3, 23.4, 23.5 and 23.6, as appropriate, must be checked for elastic buckling by detailed calculation using a pressure of 110 per cent of P_{AV} where P_{AV} is the appropriate design pressure according to section 18 or 19.

23.8 Notwithstanding paragraphs 23.2, 23.3, 23.5, 23.6 and 23.7 the webs of all framing members forming the second and third levels of support of the shell plating must be fitted with stiffeners to distribute the loads transmitted at the points of intersection with the members forming the first or second levels of support respectively.

«t_w» est l'épaisseur de l'âme en centimètres; et

«f_y» est la limite d'élasticité de l'acier en mégapascals.

23.6 Sous réserve des dispositions de l'alinéa 23.7, les membrures de charpentes, qui ne se conforment pas aux alinéas 23.2, 23.3, 23.4, et 23.5 selon le cas, devront être munis de renforts additionnels de dimensions et d'écartement suffisants pour empêcher le gauchissement élastique.

23.7 Les membrures de charpentes non conformes aux dispositions des alinéas 23.2, 23.3, 23.4, 23.5, et 23.6 selon le cas, devront être vérifiées contre le gauchissement élastique à l'aide de calculs détaillés, en se servant d'une pression de 110 pour cent de PAV alors que PAV est la contrainte appropriée, selon l'article 18 ou 19.

23.8 Par dérogation aux dispositions des alinéas 23.2, 23.3, 23.4, 23.6 et 23.7 les âmes de toutes les membrures de charpente qui forment le second et le troisième niveau de support du bordé de la coque, devront être munis de renforts pour répartir les charges transmises aux intersections avec les membrures qui forment le premier ou le second niveau de support respectivement.

24. Tripping of framing

24.1 Framing members consisting of tee sections must satisfy one of the following criteria;

24. Gauchissement de la charpente

24.1 Les membrures de charpente composées de profilés en T doivent se conformer à l'un des critères suivants:

$$\frac{LU \times V}{WF} \leq \frac{395 \times N}{\left[1 - \left(\frac{155 \times N \times Tw}{Hw \times V}\right)^2\right]^{0.75}}$$

$$\frac{Hw}{Tw} \leq \frac{155 \times N}{V}$$

or

où:

$$\frac{LU}{WF} \leq \frac{395 \times N}{V}$$

24.2 Framing members consisting of angle sections

24.2 Les membrures de charpente composées de must satisfy the criterion;

profilés en L doivent se conformer au critère suivant:

$$\frac{LU}{WF} \leq \frac{300 \times N}{V}$$

24.3 Framing members consisting of flat bar sections must satisfy one of the following criteria;

24.3 Les membrures de charpente composés de sections de barres, devront répondre à l'un des critères suivants:

$$\frac{LU \times V}{Tw} \leq \frac{710 \times N}{\sqrt{1 - \left(\frac{168 \times N \times Tw}{Hw \times V}\right)^2}}$$

$$\frac{Hw}{Tw} \leq \frac{168 \times N}{V}$$

or

ou

$$\frac{LU}{Tw} \leq \frac{710 \times N}{V}$$

24.4 Framing members consisting of offset bulb bars must comply with the following flange slenderness ratio :-

24.4 Les membrures de charpente composées de varangues désaxées de barres de bulbe devront répondre aux critères de minceur de collet suivants:

$$\frac{LU}{B} \leq \frac{719}{V}$$

24.5 where:

"Z_p^{AF}" is the as fitted modulus calculated in accordance with section 22;

"B" is the width of the bulb outstand in centimetres (see figure);

24.5 où:

«Z_p^{AF}» est le module tel qu'installé, calculé selon l'article 22;

«B» est la largeur de l'excroissance du bulbe en centimètres (Voir le Schéma);

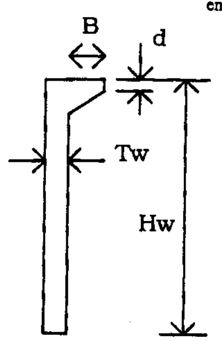
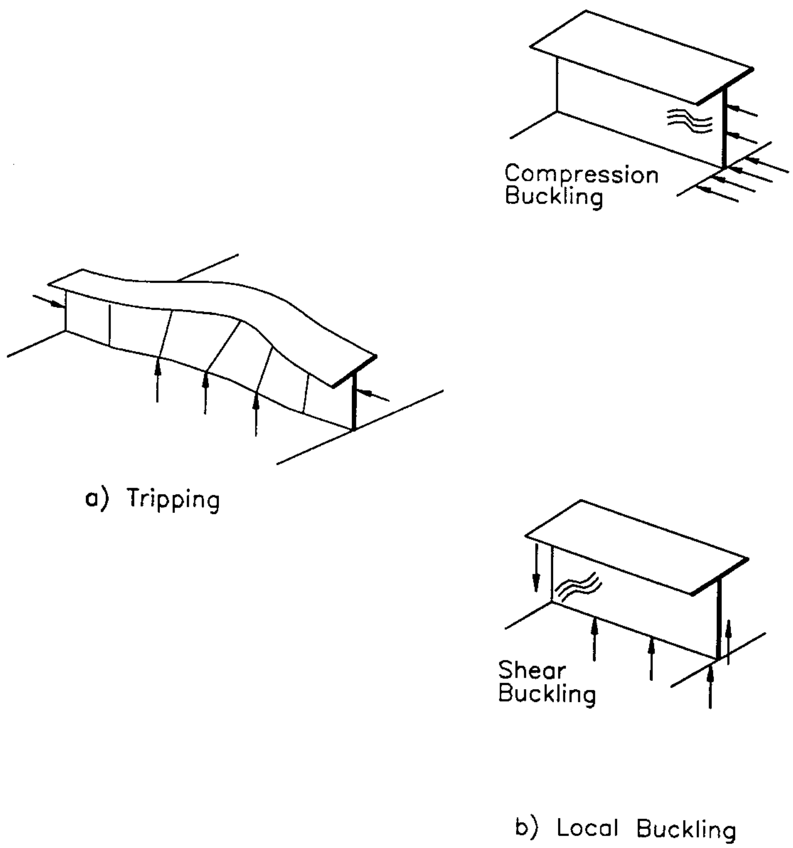


Figure 7.10

Tripping and Buckling Failure Modes



APPENDIX D

LINEAR EIGENVALUE BUCKLING STUDY

D 1.0 Eigenvalue Buckling Methodolgy for Prediction of the Onset of Buckling in Nonlinear Analyses

One proposed procedure for the identification of instabilities is to perform linear eigenvalue buckling analyses at different load levels within a nonlinear analysis. Linearized (eigenvalue) buckling provides a quantitative prediction of the buckling load. However, when this is performed using the original stiffness matrix for the unloaded structure, the effects of yielding and large displacement are not accounted for. As the structure is loaded, both yielding and large displacements are expected to occur. Either of these phenomena effect the stiffness matrix and hence change the load level at which buckling occurs. Therefore, when a linearized buckling analysis is performed under load, the predicted buckling load changes. The following section provides details of an investigation which was undertaken to determine if a linearized buckling analysis, performed under loading such that it includes the effects of yielding and large displacements, can be used to accurately determine the buckling load. A summary of the results of the investigation are repeated in Section 7.2 of this report.

The procedure to be followed is to:

1. Perform a linearized buckling analysis at time t=0.0 to determine the critical buckling load. This is the benchmark buckling load level which does not include yielding and large displacement effects.

2. Perform a nonlinear static analysis to determine nonlinear response of the structure to an applied load. This will predict the buckling response and the nonlinear buckling load level including the effects of large displacements and yielding.

3. Perform restart linearized buckling analyses at different load levels (i.e. at t= 1.0, 2.0, 3.0 etc.,) of the nonlinear buckling analysis to determine critical buckling loads based the stiffness of the structure at discrete points in the response which includes yielding and large displacements.

4. Compare the results of the different linearized buckling analyses to see if the critical buckling load converges to that predicted from the nonlinear buckling analysis.

D 1.1 Geometry (FE Model), Loads, and Boundary Conditions

The model used in this study is shown in Figure D.1. The structure consists of a section of plating with two angle main frames. A lateral pressure load of 500 psi is applied to the structure as shown in Figure D.2. This equals a total applied lateral load of 480,000 pounds. The applied boundary conditions are shown in Figure D.3. The restraints are defined to promote a well defined nonlinear buckling response.

For this geometry, it was expected that the predicted buckling load from the linearized buckling analysis at t=0.0 will be much larger than that predicted from the nonlinear buckling analyses where the stiffness is calculated for a loaded structure. This is due to the fact that in the linearized buckling analysis at t=0.0 (i.e. applied load = 0.0) the initial stiffness of the structure is used where no yielding and large displacements exist. It was expected that as displacements become large and yielding progresses the restart linearized buckling analyses performed would now predict a drop in the buckling load which should converge with the nonlinear static analysis predicted buckling load.

D1.2 Linearized Buckling Analysis at T=0.0

The first six mode shapes of the linearized eigenvalue buckling analysis are shown in Figures D.4 to D.9. The critical buckling factor (mode 1 in Figure D.4) is 1.98313. This is equal to an applied load of 1.98313*480000 = 951902 pounds. Buckling loads 2 through 4 are very close to the critical buckling load with loads of 953736, 959971, and 963101 pounds, respectively. Therefore, any of these buckling modes can be expected to occur first.

Buckling modes 5 and 6 occur at loads of 1613040 and 1640894 pounds, respectively. These loads are considerably higher than the first four buckling loads.

D1.3 Nonlinear Buckling Analysis

A nonlinear static analysis was performed using the same FE model as used in the linearized buckling analysis. Initially, the Load-Displacement-Control (LDC) method was used for this analysis. These results are shown in Figures D.10 and D.11. Figure D.10 shows the displaced shape of the two main frames. Lateral buckling of the two main frames is evident. This lateral buckling is clearly seen in the load-displacement curve of node 360 (see Figure D.10) as shown in Figure D.11. This node is at the intersection point of the web and flange at midspan of one of the main frames. The response of both main frames is very similar.

A restart linearized buckling analysis was then attempted with the results of the LDC nonlinear analysis, however, it was discovered that ADINA does not support restart linearized buckling analysis during an LDC analysis. Therefore, the Automatic-Time-Step (ATS) method was used for this investigation. Similar to the LDC method, the ATS method automatically step through the nonlinear analysis. The only potential disadvantage to using the ATS method over the LDC method is that the ATS method does not converge as well in regions of high instabilities. Therefore, the ATS analysis was not performed to the same load limit as the LDC method. It was performed up to the point of instability initiation.

The ATS nonlinear analysis was carried out using 20 load steps up to a total load of 1.46 times the total applied load (1.46 * 480000 = 700800 pounds). The displaced shape of the panel at load step 20 is shown in Figure D.12. The load-displacement curve for the lateral deflection of node 360 (see Figure D.10) is shown in Figure D.13. This curve mirrors the load-displacement curve of the LDC method up to 700800 pounds. The instability (the same as that produced from the LDC method) is evident by the change in slope of the curve which starts at a load just under 600000 pounds. This point is chosen since it represents the first sign of a detectable loss of stiffness.

A plot of the longitudinal stress (S_{yy}) in the panel is shown in Figure D.14. A yield strength of 34000 psi and a tangent stiffness of 1/40*Young's Modulus was used in the analysis. As presented at the bottom of the legend in Figure D.14, the maximum stress in the main frame is 43500 psi. This is well above the yield strength of the material.

D1.4 Restart Linearized Eigenvalue Analyses

Restart linearized buckling analyses were carried out using the results of the ATS nonlinear analysis at steps 1.0, 2.0, 3.0 and 4.0. Any attempted restart buckling analysis above this point was unsuccessful. This was because a part of the structure has yielded such that it has insufficient lateral bending stiffness to provide a sufficiently conditioned structure to be solved.

For any linearized buckling analysis, buckling occurs when the lateral bending stiffness (called geometric stiffness) of a structure is zero. To determine the geometric stiffness in a linearized buckling analysis, an initial linear static analysis is performed. In the restart analysis, this initial linear static analysis is performed starting at the restart time with an applied load equal to the portion of the load vector from the restart time to the next time step. (For example, if the restart time is 2.0, then the initial linear analysis is performed starting at t=2.0 with an applied load equal to the difference of the applied loads between times 2.0 and 3.0. The linearized eigenvalue analysis is then carried out at step 3.0.)

If, at the restart time, the structure has sufficient yielding such that the lateral bending stiffness terms are very small, any additional applied load produces negative stiffness terms. This happened for all restart analyses above step 4.0, regardless of the magnitude of the applied load.

A plot of restart time step versus predicted buckling load for steps 1.0 to 4.0 is shown in Figure D.15. At the last time step, the buckling load is 512,655 pounds. This is lower than the chosen buckling point of just under 600000 for the nonlinear analysis. It was originally thought that this curve would monotonically converge to the nonlinear predicted buckling load, however, this is not the case. The buckling load drops dramatically from that at t=0.0 to that at time t=1.0, and increases to a value close to the nonlinear predicted value. However, the curve is fairly linear from steps 2.0 to 4.0. This provides no indication of convergence.

To determine if convergence was possible, the ATS analysis was restarted at step 4.0 with smaller load increments. A restart eigenvalue analysis was then performed at the new step 5.0. This failed to produce a solution. Several iterations of reducing the load steps (above step 4.0) in the ATS analysis were carried out, however, no successful restart eigenvalue solutions could be achieved. If restart analyses above step 4.0 were possible, the restart buckling analyses may converge to the nonlinear value.

The mode shape for the restart buckling analysis at step 4.0 is shown in Figure D.16. This mode shape is not similar to the displaced shape predicted from the nonlinear analysis (Figure 10) even though the buckling loads are of similar magnitude. However, it is similar to mode 1 of the linearized buckling analysis at t=0.0 (Figure D.4). Since (as detailed above) any of the first three linear buckling modes at t=0.0 can be expected to occur first, the restart linearized buckling mode shape at t=4.0 could be expected to take the form of any one of these first three modes. In this case it was a shape similar to the first mode while the nonlinear analysis predicts a shape similar to the second model.

D1.5 Summary and Conclusions

The restart eigenvalue buckling analysis emerged as a potentially valid procedure to predict the buckling load of stiffened panels which have undergone large displacements and plasticity. However, convergence to an accurate prediction was found to be a time consuming and expensive venture which is not expected to provide better predictions than those obtained by analysis of load-displacement curves.

In the analysis described in the previous sections, complete convergence was not achieved between the eigenvalue method and nonlinear methods. This was because the eigenvalue method cannot predict buckling modes when yielding has reduced the lateral bending stiffness to near zero. Convergence can be improved by a subdivision of load steps between the last successful restart linearized buckling analysis and the next step. The ATS method was reanalysed for these smaller steps and the restart eigenvalue buckling performed at the new intermediate ATS time step. However, for this model, no restart eigenvalue analyses (at the new steps) could be successfully analysed. The yielding at step 4.0 was too extensive. With small enough increments, this process may help to provide convergence of the two predicted buckling loads. However, this is a very time consuming procedure for large models.

An alternative procedure is to base the initiation of buckling on a discernable loss of stiffness determined from inspection of the load-displacement curve. This was the approach taken in the Phase I [3] study. For buckling that displays characteristics similar to bifurcation buckling (see Figure D.11), the point of initiation of an instability is more easily identified. It is manifested by a substantial increase in the rate of change of slope of the load-displacement curve. This rate of change can be used as the indicator for all buckling since it represents a significant decrease in the lateral bending stiffness of a structure.

The critical rate of change of slope can be selected from example problems where bifurcation buckling occurs based upon a selected set of criteria (such as magnitude of lateral deflection, etc.). This rate can then be discriminantly used for other types of buckling.

Another alternative is to base the initiation of buckling on the shear force carried by the structural member. When a member buckles, it sheds load that it previously was capable of carrying which results in a decrease in shear force. A description of this methodology and the study to determine the viability of using it is presented in the section 7.2.2 of this report.

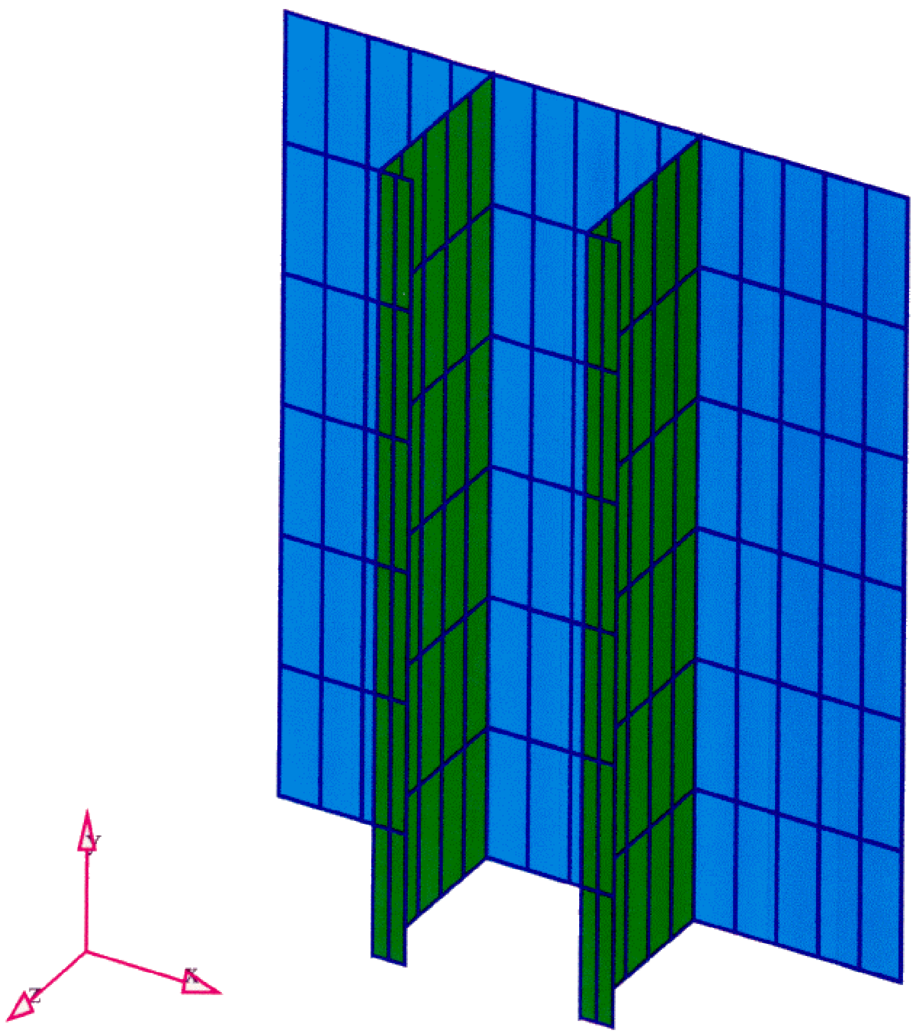


Figure D.1 FE Model used for Eigenvalue Buckling Analysis Study

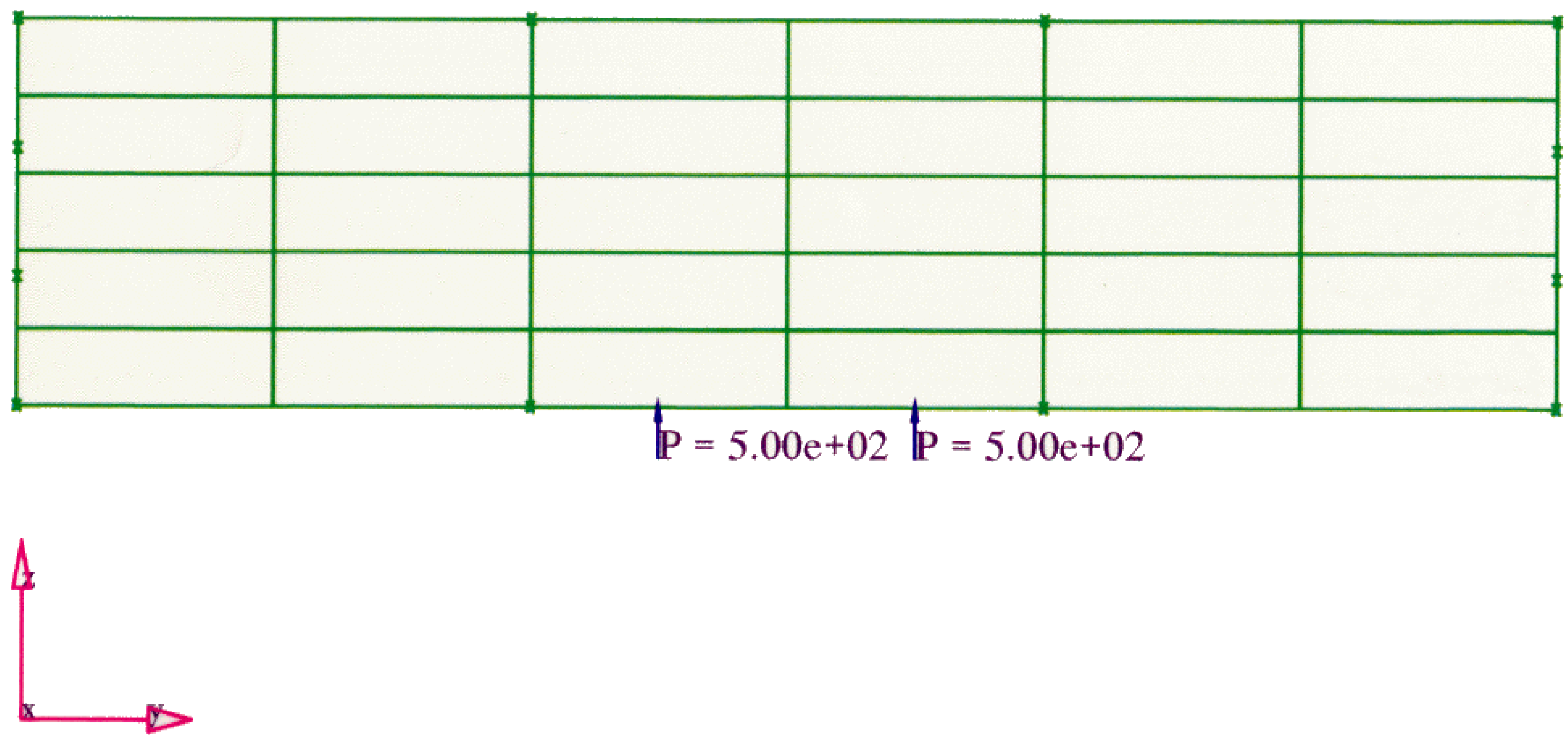


Figure D.2 Appied Load on FE Model

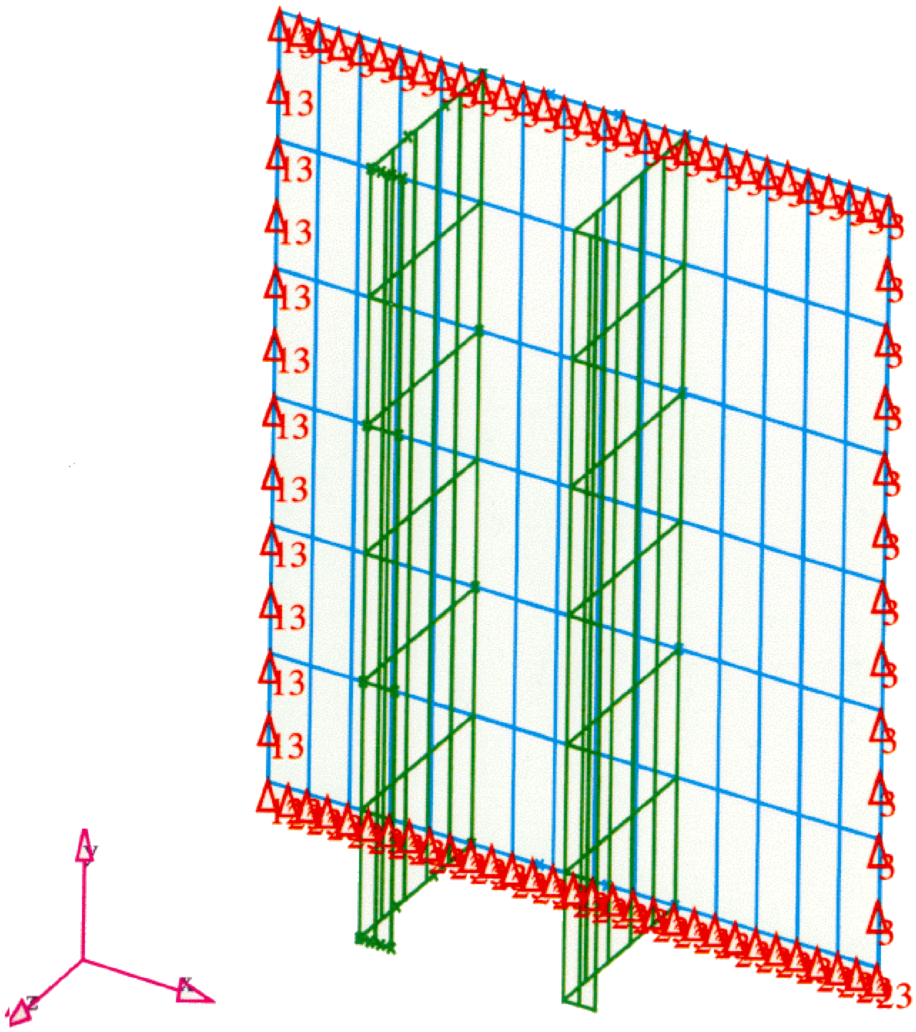


Figure D.3 Boundary Conditions on FE Model

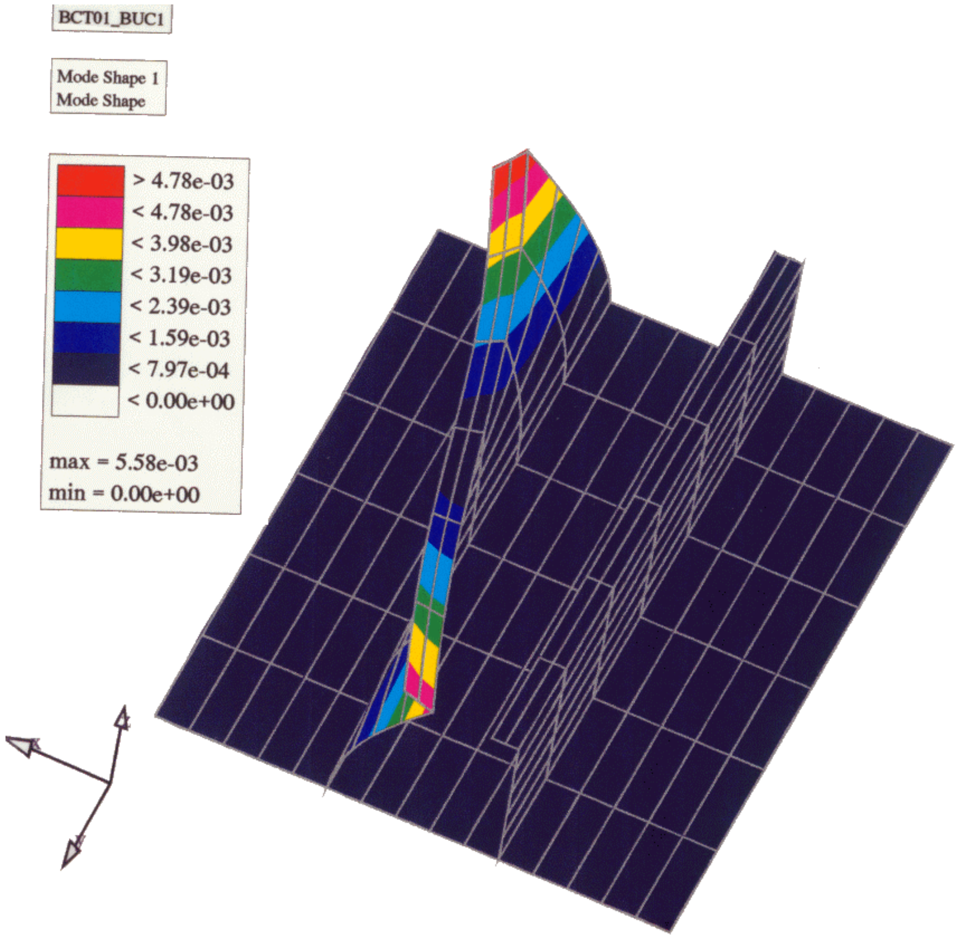


Figure D.4 Linear Buckling Mode #1 of FE Model

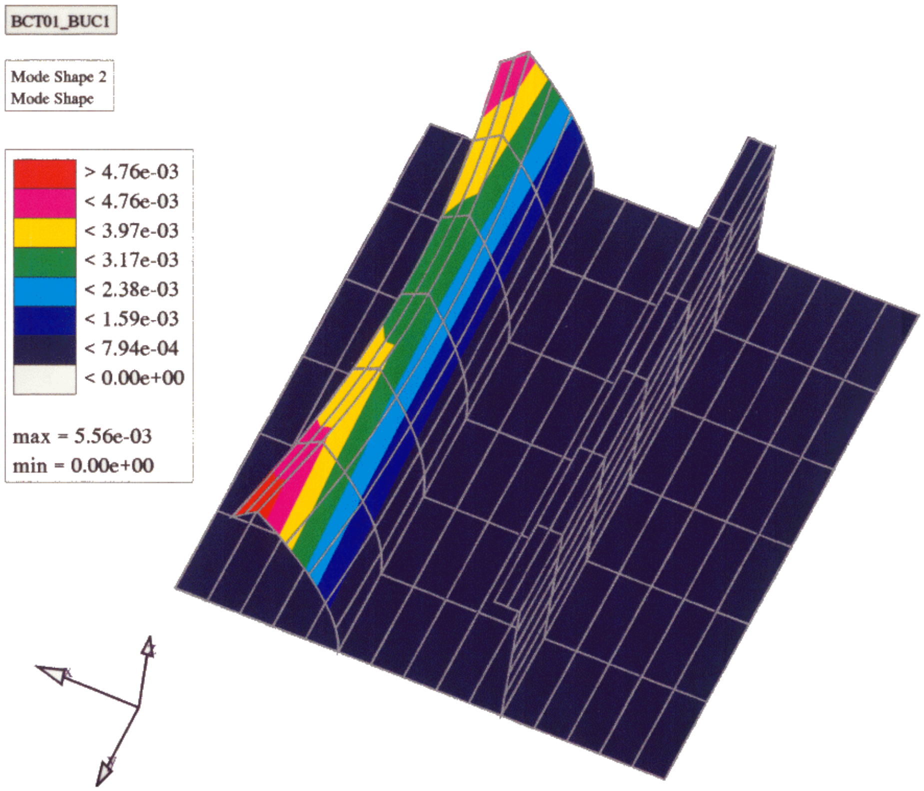


Figure D.5 Linear Buckling Mode #2 of FE Model

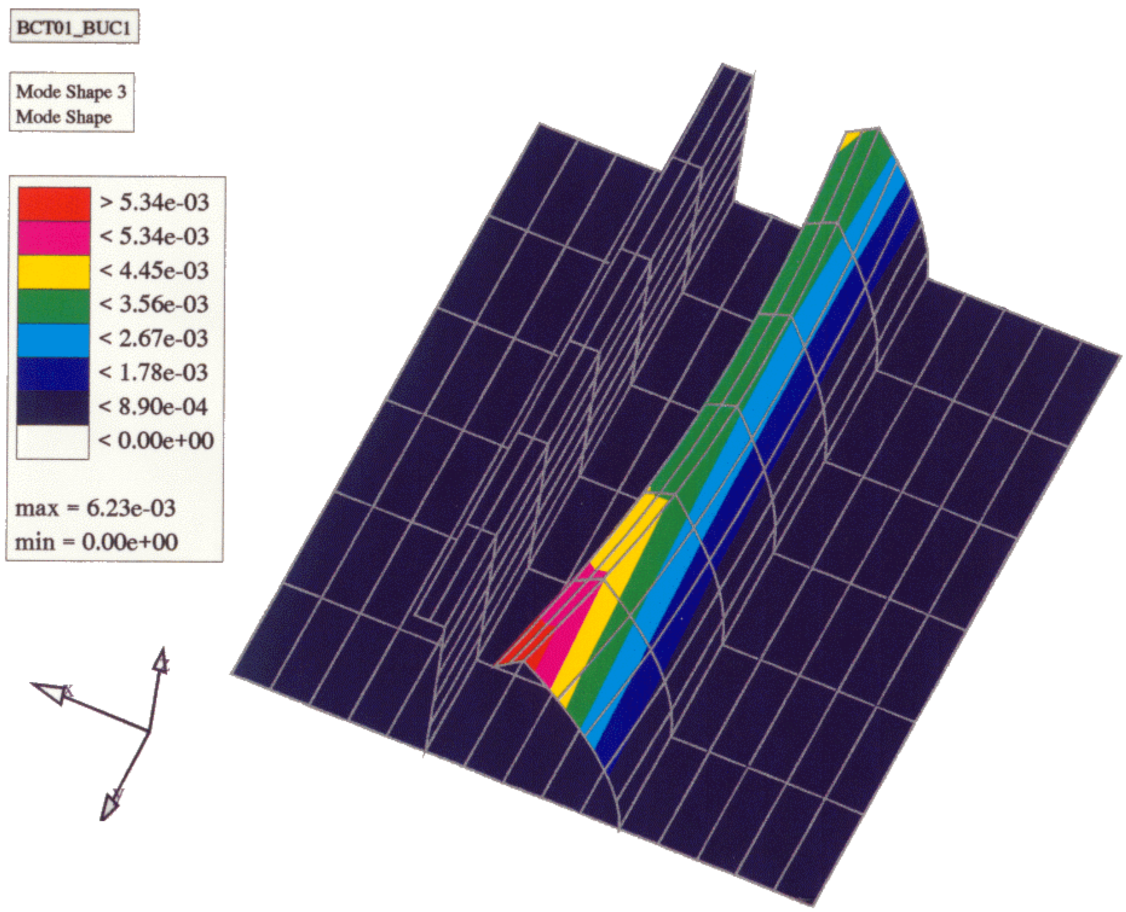


Figure D.6 Linear Buckling Mode #3 of FE Model

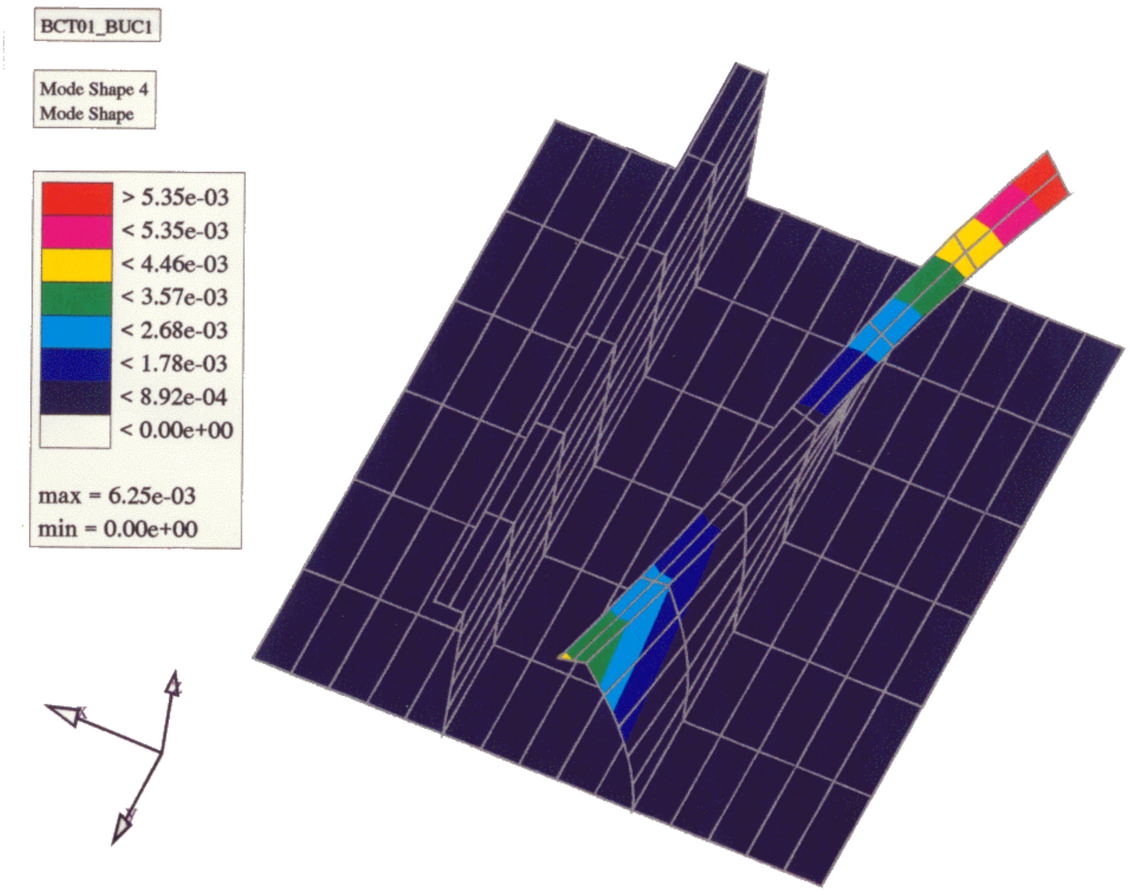


Figure D.7 Linear Buckling Mode #4 of FE Model

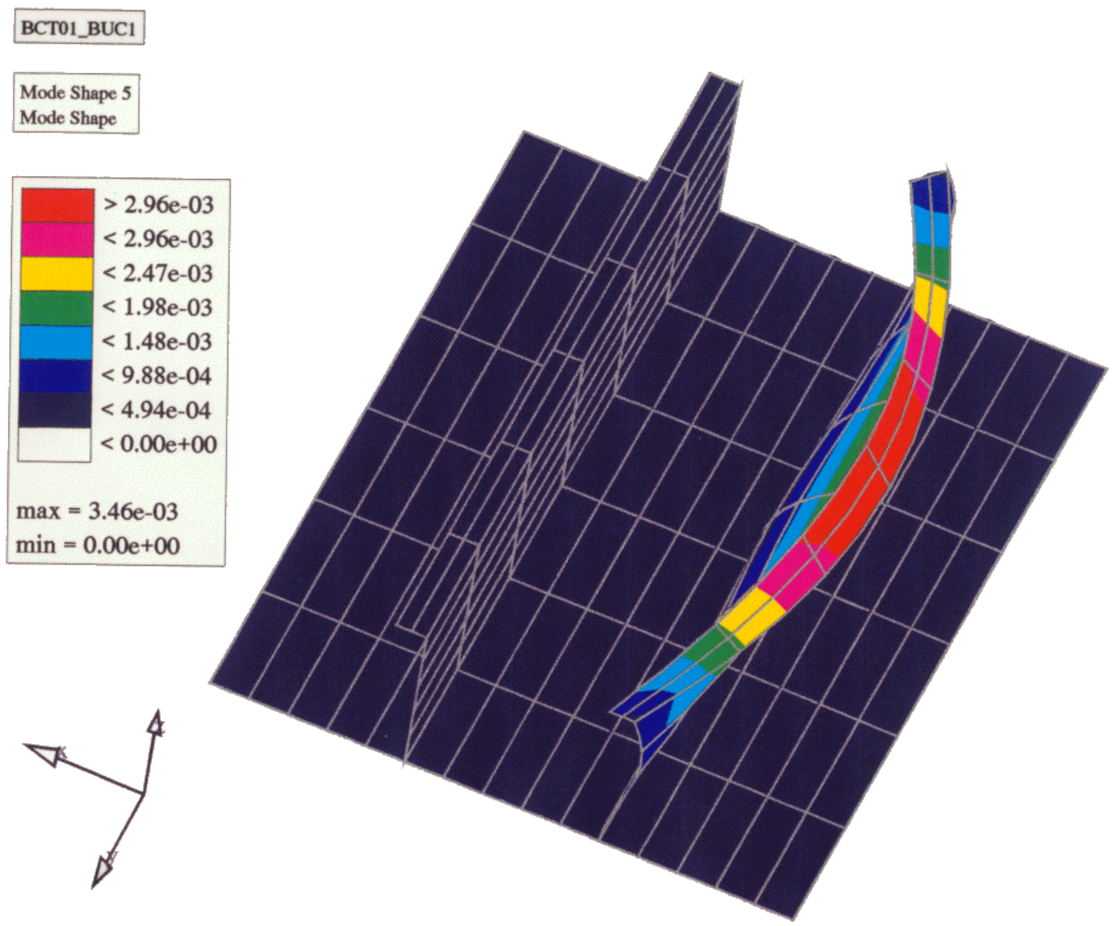


Figure D.8 Linear Buckling Mode #5 of FE Model

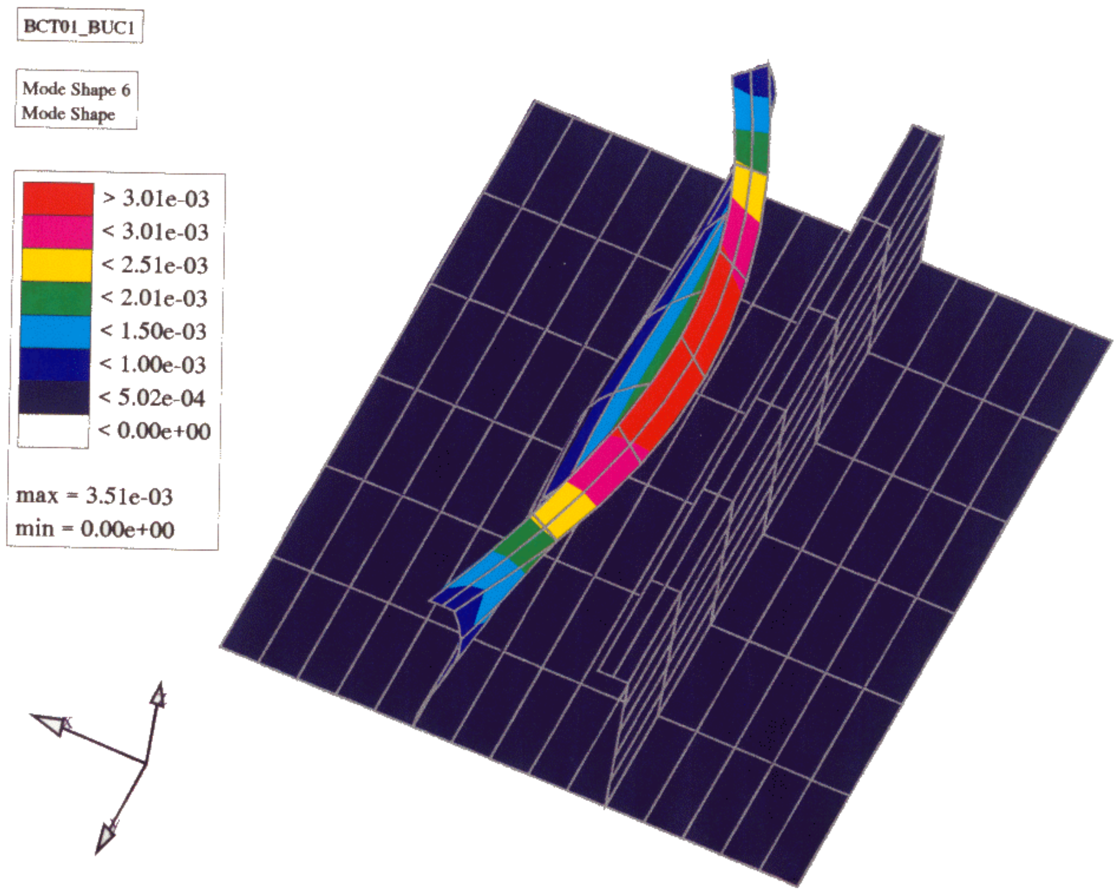


Figure D.9 Linear Buckling Mode #6 of FE Model

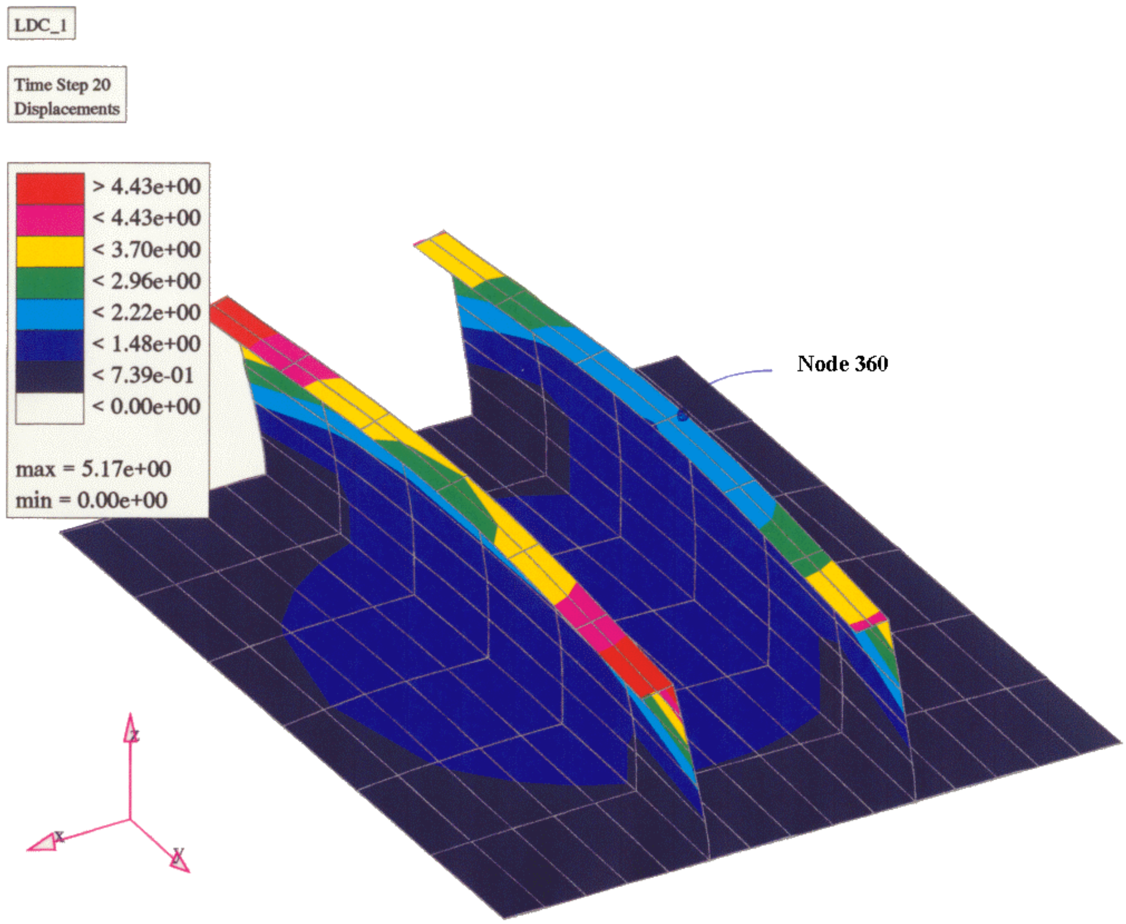


Figure D.10 Displaced Shape at Load Step 20 for Nonlinear Analysis of FE Model using the LDC Method

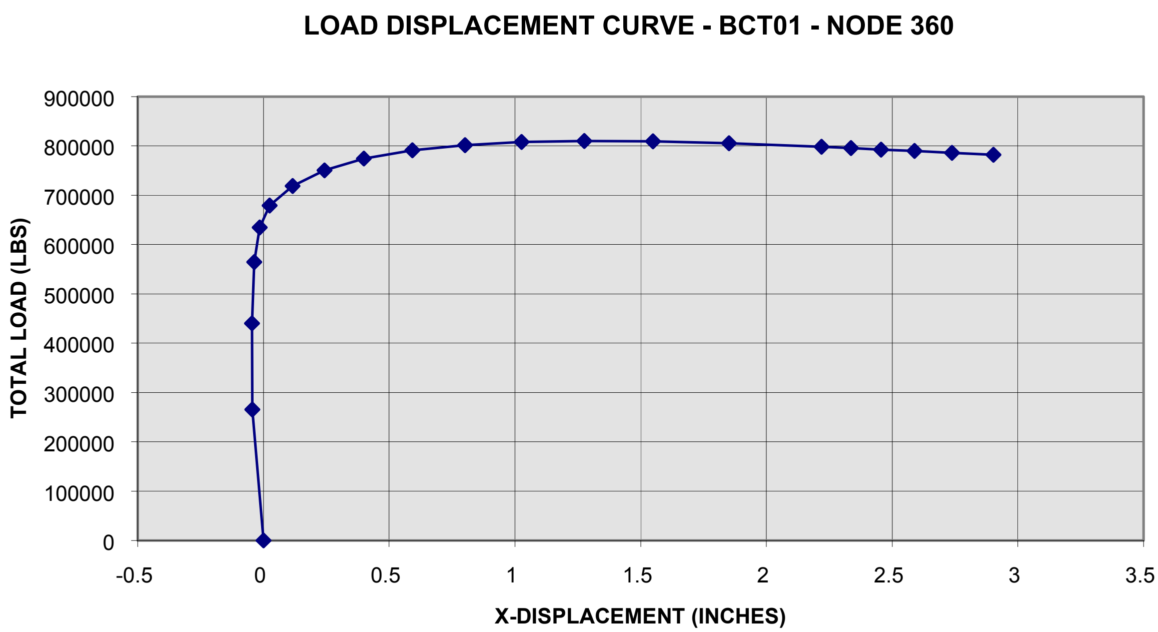


Figure D.11 Load Displacement Curve for Nonlinear Analysis of FE Model using the LDC Method

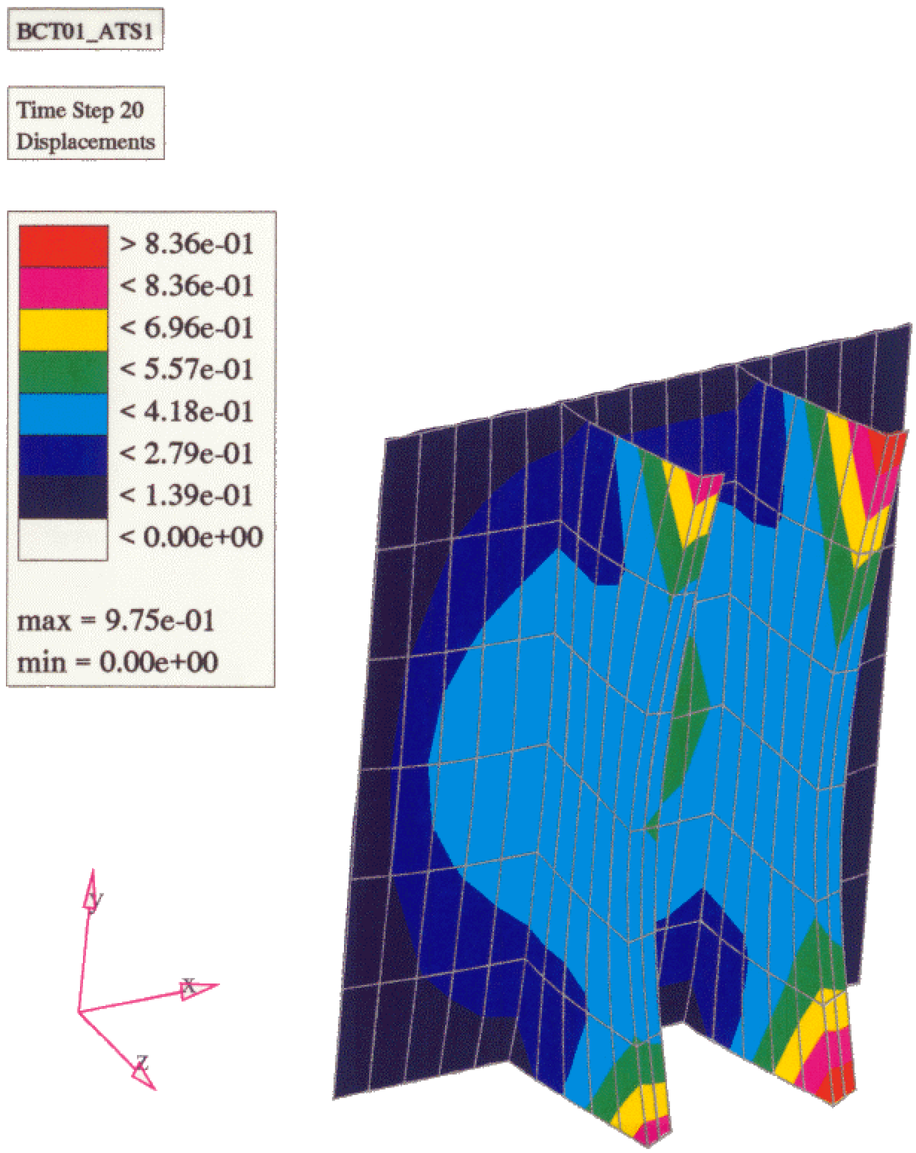


Figure D.12 Displaced Shape at Load Step 20 for Nonlinear Analyses of FE Model using the ATS Method

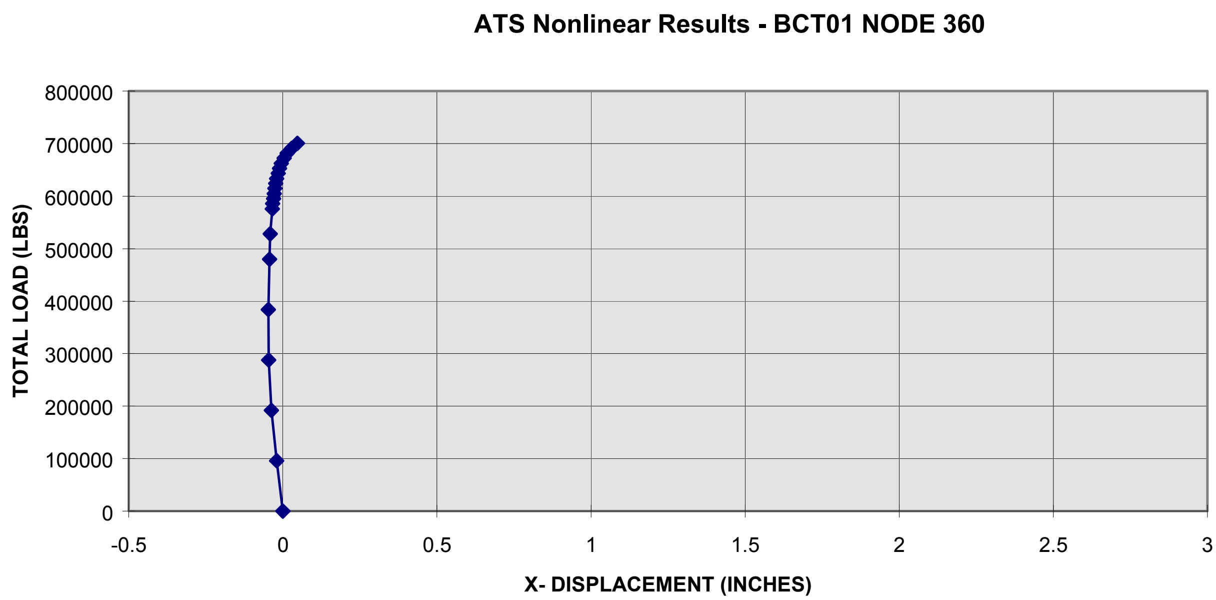


Figure D.13 Load Displacement Curve for Nonlinear Analysis of FE Model using the ATS Method

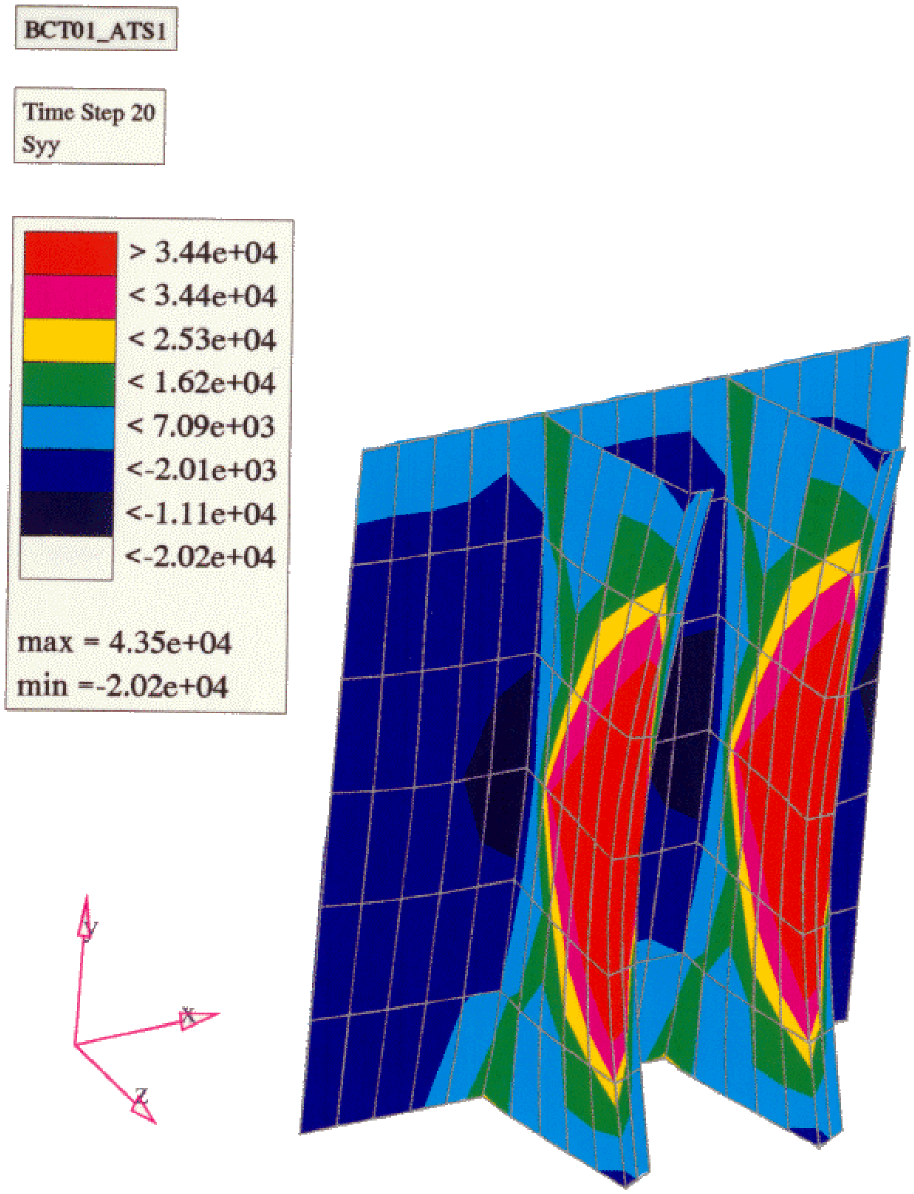


Figure D.14 S_{yy} Stresses in the FE Model at Load Step 20 using the ATS Method

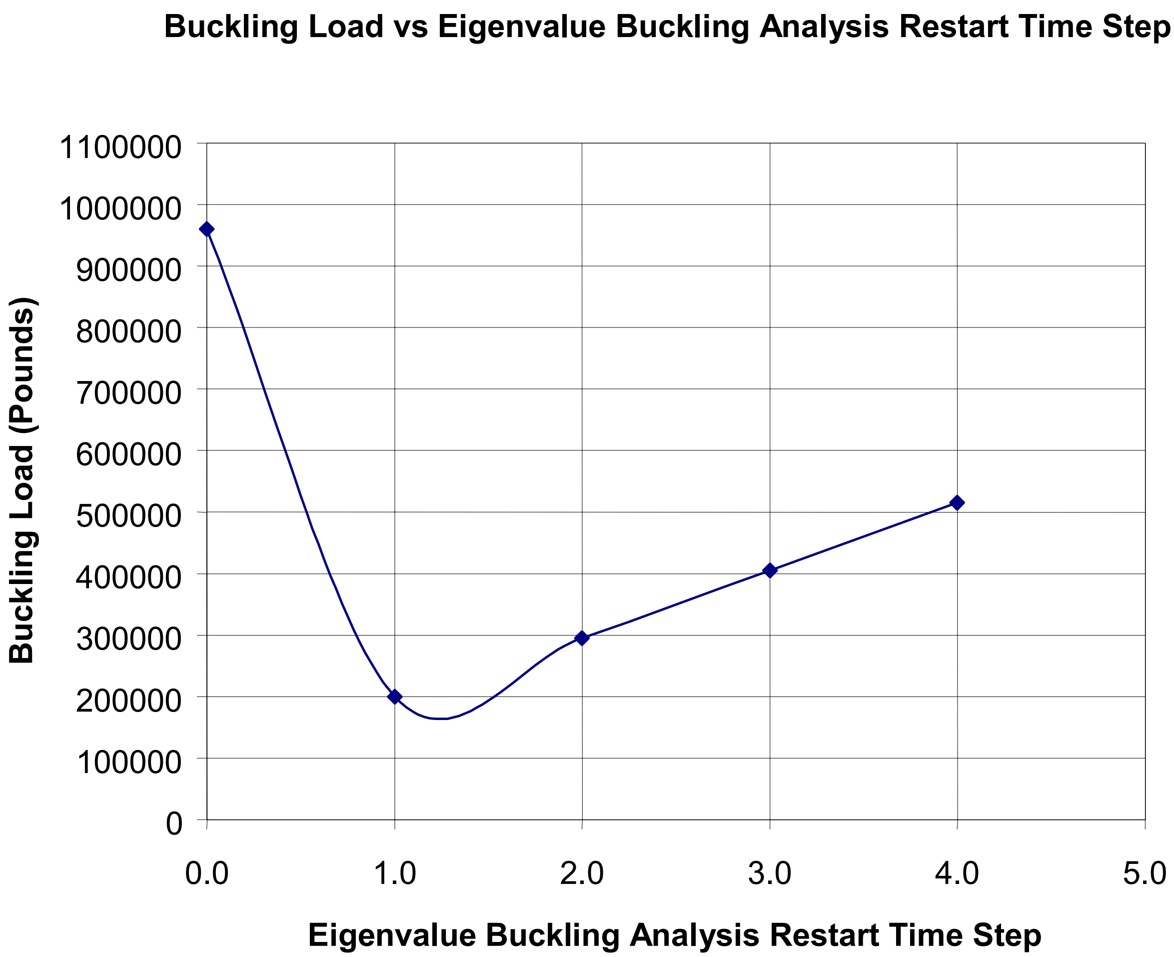


Figure D.15 Plot of Predicted Buckling Load vs Eigenvalue Buckling Analysis Restart Time Step for the ATS Method

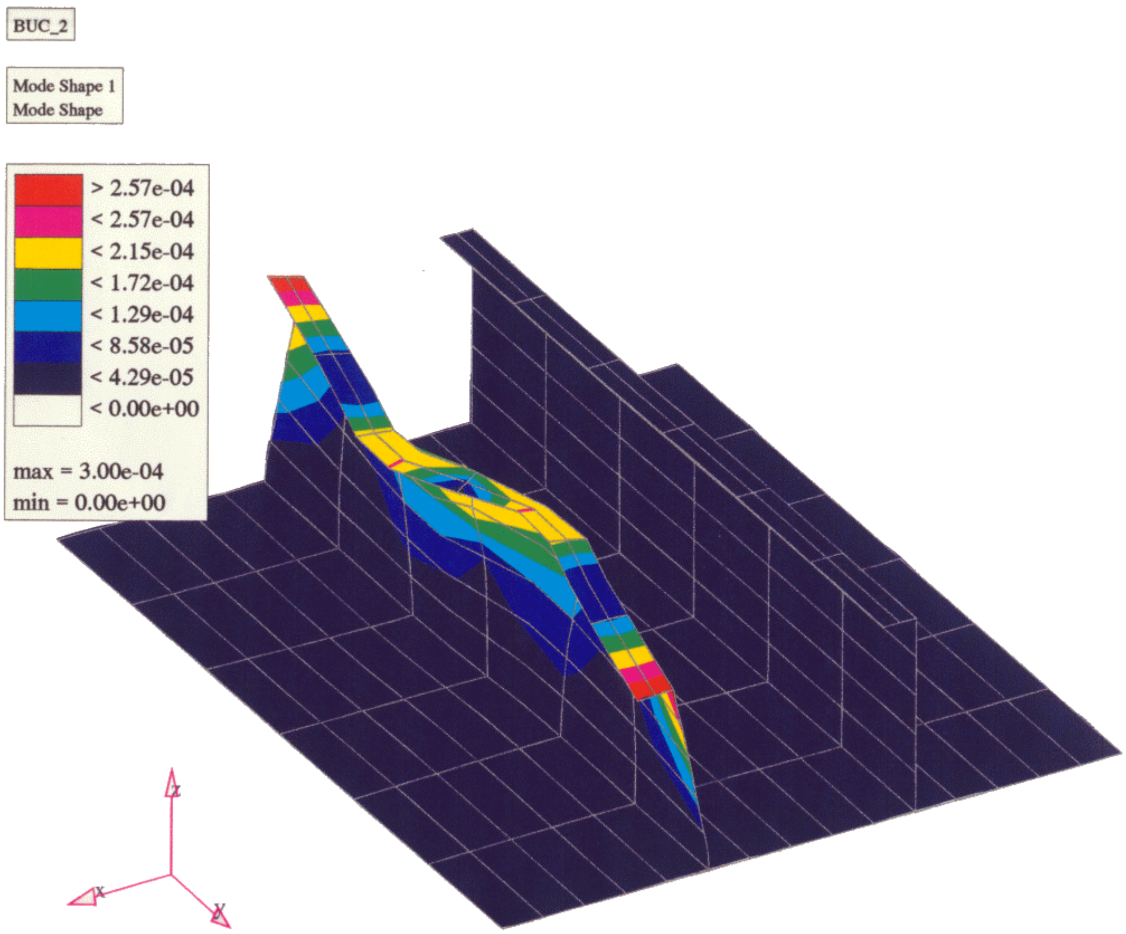


Figure D.16 Mode Shape for Restart Buckling Analysis at Load Step 4

APPENDIX E

FILES THAT CONTAIN LOAD-DISPLACEMENT AND SHEAR FORCE CURVES FOR MOST ANALYSIS OF THE TEST MATRIX

(CD-ROM INSIDE BACK COVER)

APPENDIX F

DEMONSTRATION OF HOW THE EQUIVALENT STANDARD EQUATIONS DO NOT ACCOUNT FOR YIELD STRENGTH

From Section 2.4.2 of the Equivalent Standards, frame members consisting of angle sections must satisfy the following tripping criterion:

$$\frac{LU}{WF} \le \frac{300 \times \mathbf{N}}{\mathbf{V}} \tag{1}$$

where:

"N" equals one when ∂ is equal to or greater than 85 degrees and (1-cos ∂) when ∂ is less than 85 degrees and,

$$\mathrm{V} = \frac{\sqrt{\mathrm{f_y} \times \mathrm{Z_p}}}{\mathrm{Z_p^{AF}}} \tag{2}$$

From the definition of V:

"f_y" is the nominal yield stress;

"Z_p" is the required frame section modulus in accordance with sections 18 or 19 and 20;

$$\begin{aligned} &= \quad \frac{CF \times AF \times P_{AV} \times S \times LB \times B \times R_2 \times 41670}{f_y} \\ &or \quad \frac{AF \times P_{AV} \times S \times LB \times B \times R_2 \times 41670 \times C}{f_y} \end{aligned} \tag{3}$$

"Z_p^{AF}" is the as-fitted modulus calculated in accordance with section 22,

$$= \mathrm{k} \times \left[\mathrm{A_f} \times \left(\mathrm{h_w} + 0.5 \times \left(\mathrm{t_f} + \mathrm{t_p}\right)\right) + \mathrm{A_w} \times 0.5 \times \left(\mathrm{h_w} + \mathrm{t_p}\right)\right] \tag{4}$$

Substituting (3) and (4) into (2) gives:

$$\mathrm{V} = \sqrt{\frac{\mathrm{f_y} \times \dfrac{(\mathrm{CF} \times \mathrm{AF} \times \mathrm{P_{AV}} \times \mathrm{S} \times \mathrm{LB} \times \mathrm{B} \times \mathrm{R_2} \times 41670)}{\mathrm{f_y}}}{\mathrm{k} \times \left[\mathrm{A_f} \times \left(\mathrm{h_w} + 0.5 \times \left(\mathrm{t_f} + \mathrm{t_p}\right)\right) + \mathrm{A_w} \times 0.5 \times \left(\mathrm{h_w} + \mathrm{t_p}\right)\right]}}$$

The f_y terms cancel which effectively removes the yield strength from the tripping criterion for angle main frames. The same condition applies to both flat bar and tee main frames.