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INELASTIC MEMBRANE ANALYSIS OF CONTINUOUS STEEL PLATES

by

James Bond P.Eng

B.A.Sc. University of Waterloo

**A thesis submitted to the Faculty of Graduate
Studies and Research in partial fulfilment of
the requirements for the degree of
Master of Engineering**

**Department of Civil and Environmental Engineering
Carleton University
Ottawa, Ontario
Canada**

**The Master of Engineering in the Civil and Environmental
Engineering Program is a joint program with the University of Ottawa,
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INELASTIC MEMBRANE ANALYSIS OF CONTINUOUS STEEL PLATES

Submitted by
James Bond, BAsC.

in partial fulfilment of the requirements for
the degree of Master of Engineering

For Dr. J.L. Humar, Department of Civil and
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26 April 1995

ABSTRACT

One of the objectives of a naval architect or naval structural engineer, when designing marine structures, is to determine the proper distribution of material in the plating and stiffening elements, to resist both normal operating loads and extreme loads. The source of the extreme loads could be severe ice forces (collision or constant pressure), a collision for a commercial ship or blast and missile impact for a warship.

Stiffened plate panels will carry large transverse loads if they are allowed to deform to the extent to which membrane action in the plate is the primary means of support. Such deflections, as much as one fifth of the span, are obviously unacceptable under normal operational load conditions, but must be utilized to economically resist extreme loads.

This report summarizes the experimental work by Young (1959), McDermott et al. (1974), Ratzlaff and Kennedy (1985), Kennedy and Cheng (1987), Ratzlaff (1995) and others on the behaviour of single and double steel membrane plates. Also presented is an analytical model which describes the load deflection characteristics, the deflected shape, internal force distributions and failure load for one-dimensional plate panels with variable loads (magnitude, direction, location), variable material properties (parent plate, heat affected zone material and weld material), and flexible boundaries. Comparisons are made between the analytical model predictions and experimental data. Steel panels, as typically stiffened, are analyzed for extreme load response. Through parametric studies the sensitivity of the load and deflection at the end of the elastic response and at the ultimate capacity of the plate (rupture) to changes in geometric and material parameters are investigated. Finally, the results of the parametric studies are utilized to make rational changes to a typically stiffened panel. This panel is analyzed and the results of the analysis are compared to the baseline structure. The cost of constructing both the baseline and the revised structure are determined to illustrate the economic ramifications of the changes.

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1.0 INTRODUCTION

1.1 General

The goal of a naval architect or naval structural engineer is to determine the proper distribution of material in hull plating and stiffening so that, not only the normal operating loads will be adequately supported, but so that the vessel can also survive extreme loads. The source of these extreme loads could be severe ice forces against an ice breaker or arctic oil platform, a collision for a commercial ship or blast and missile impact for a warship.

Stiffened plate panels will support very high pressures if allowed to deform to the extent that membrane action in the plate is the primary support method. Such deflections, as much as one fifth of the plate span, are clearly unacceptable in the support of normal operational loads but must be utilized to economically resist extreme loads.

Considerable experimental work has been performed by Young (1959), McDermott (1974), Ratzlaff and Kennedy (1986), Kennedy and Cheng (1987), Brown and Joubert (1990) and Ratzlaff (1995) to characterize the load versus deflection behaviour of plates subjected to high pressures or normal loads. Analytic methods of predicting this behaviour have been developed by Young, McDermott and Kennedy. However, a recognized and fully verified design approach and analysis method does not exist. The objective of this thesis and the investigation presented herein is to advance towards the goal of such an approach and method.

This work utilizes the work of others as a starting point to advance the overall subject further. This starting point of confirmed assumptions and a wealth of experimental data are acknowledged as invaluable to providing a manageable scope for this study.

The analysis and mathematical computer model that was developed is limited to the one-dimensional membrane action of a plate panel. The model utilizes the material stress versus strain characteristics, incorporates flexible boundaries and can simulate behaviour of point loads, uniformly distributed loads (UDL's) or hydrostatic pressure.

Variable material properties, such as parent plate, weld metal and a heat affected zone, biaxial state of stress and strain rate effects, are also included in the model. The ultimate load carrying capability (plate rupture load) and the value of the maximum deflection are sought in the analysis.

1.2 Outline

The report is structured to present chronologically the process that was followed to achieve the results. The results of a literature search are presented, with the previous publications of others that have been utilized discussed and summarized. The structural response of a plate panel under load is described. The behaviour of the material in the plate during the load application is characterized. The theoretical approach used in the mathematical computer model is presented. The computer model, its structure and features, are described. Validation of the model and/or discrepancies between its predicted behaviour and the response of plates tested experimentally by others is demonstrated. Parametric studies are presented to provide insight into the effect of varying the plate geometric and physical properties on the plate load capacity and deflection. These parametric studies are performed on the structural configurations of stiffened panels as found in three representative baseline vessels.

The last sections contain a discussion of the possible uses for the work presented in this thesis.

2.0 LITERATURE REVIEW

2.1 General

A literature review has been conducted to disclose work by others on the same or similar topic. The articles and papers reviewed cover a time span from 1959 through 1995. They include the discussion of the utilization of membrane action to support transverse loads.

2.2 Young A.G.; SHIP PLATING LOADED BEYOND THE ELASTIC LIMIT; Transactions of the Institute of Naval Architects, 1959

Summary

The author conducted an experimental program to study the behaviour of plate panels loaded beyond the elastic limit. The effect of membrane forces at large deflections were taken into account. With support of both theoretical considerations and the experimental work, simple formulae were developed to determine the "practical" load carrying capability of elastically restrained plates. Young considered "practical" to be a serviceability condition with a maximum central deflection to plate thickness ratio of approximately three. The plates were not tested to rupture.

Objective

The aim of the work was to develop a plastic design method for ship plating loaded beyond the elastic limit.

Experimental Program

Young performed small scale and full scale tests to establish a load deflection relationship.

The full scale tests were performed on plate strips, two each, of 3/16" (4.76 mm), 1/4" (6.35 mm) and 1/2" (12.7 mm) thickness. The strips each had widths of approximately 1" (25.4 mm) and lengths of approximately 25" (635 mm). Young thus had b/t ratios between 133 and 50 which are typical of ship structures. The strips were welded to relatively heavy plates which in turn were bolted against the machined faces of support joists. The strip ends were thus fully restrained against rotation. The test apparatus was designed to allow completely free horizontal movement of one support end or to rigidly maintain the original distance between support ends through the use of an adjustable length tube. Screws at the ends of the tube provided the adjustment capability. Four equal point loads were applied at increasing values to each test specimen.

The average geometric and material mechanical properties of the test specimens are presented in Table 2.1. The load versus deflection results for the full scale tests are presented in tabular form in Table 2.2 and graphically in Figure 2.1. Load versus deflection results reported by Young in Imperial units have been converted to SI units.

Table 2.1: Young's Full Scale Test Specimen Properties

Specimen Number	Thickness (inches)	Width (inches)	Yield Stress (tons/sq.in)	Pc ** (lbs)
1	0.184	0.990	19.0	200
2	0.252	1.244	19.2	476
3	0.504	1.010	18.2	1465

** where Pc is the collapse load for a simply supported specimen in the absence of axial restraint

Table 2.2: Young's Full Scale Test Experimental Results

Load Ratio **	Young's Test Results					
	1/2" Specimen		1/4" Specimen		3/16" Specimen	
	Total Load (kN)	Defl'n (mm)	Total Load (kN)	Defl'n (mm)	Total Load (kN)	Defl'n (mm)
0.5	3.2	3.2	1.1	4.5	0.4	4.7
1.0	6.5	5.1	2.1	6.4	0.9	6.4
1.5	9.8	11.5	3.2	9.6	1.3	8.9
2.0	13.0	15.4	4.2	11.5	1.8	9.3
2.5	16.3	17.9	5.3	12.8	2.2	10.7
3.0	19.5	21.8	6.4	14.1	2.7	11.7
3.5	22.8	24.3	7.4	15.4	3.1	12.6
4.0	26.1	27.5	8.5	16.6	3.6	13.6
4.5	29.3	29.4	9.5	17.9	4.0	14.5
5.0	32.6	33.3	10.6	18.9	4.4	15.4
5.5	35.8	35.8	11.6	19.8	4.9	16.4
6.0	39.1	38.4	12.7	23.0	5.3	17.3
7.0			14.8	25.6	6.2	18.2
8.0					7.1	19.2
9.0					8.0	21.5
10.0					8.9	23.4
11.0					9.8	25.7
12.0					10.7	29.0

Where ** Load Ratio is the ratio of applied load to P_c load as indicated in Table 2.1

Figure 2.1: Load versus Deflection Results for Young's Full Scale Tests

The small scale tests were performed using a hydrostatic pressure loading on 9" square panels. The panels were securely clamped around their perimeter to a frame such that the area exposed to the load measured 6" by 6". This area was split into three panels, measuring 2" by 6", by "support beams". Three different configurations which allowed differing support conditions at the support beams were examined. The first configuration (Arrangement A) provided rigid support against the load, the second (Arrangement B) allowed the strength of the beams in bending to provide the support and

the third (Arrangement C) had support as in the second but in addition the plate panel was soldered to the support beam. The small scale testing arrangements are illustrated in Figure 2.2.

Figure 2.2: Young's Small Scale Test Arrangement
(After Young 1959)

The load versus deflection results for the three arrangements are presented in Table 2.3.

Table 2.3: Young's Small Scale Test Experimental Results

Load Ratio **	Deflection Ratio ***		
	Arrangement A	Arrangement B	Arrangement C
0.5	0.25	0.20	0.15
1.0	0.70	0.50	0.55
1.5	1.00	1.05	1.00
2.0	1.30	1.15	1.45
2.5	1.60	-	1.80
3.0	1.90	-	2.10
3.5	2.05	-	-
4.0	2.20	-	-
4.5	2.40	-	-
5.0	2.60	-	-
5.5	2.80	-	-
6.0	3.00	-	-
6.5	3.20	-	-
7.0	3.50	-	-

where ** Load ratio is the ratio of applied load to P_c the load which causes collapse of a simply supported plate with no edge restraint

*** Deflection ratio is the ratio of maximum central deflection under the applied loading to the plate thickness

Behaviour

Young considered the load deflection curve to be governed by four different situations. The four parts of the load deflection curve, each of which Young developed equations to predict the load deflection for, are:

- 1). Purely Elastic
- 2). Elasto-Plastic with plastic hinges at the supports, curve section valid up to the formation of a centre hinge
- 3). After the formation of the centre plastic hinge
- 4). Pure Plastic Membrane

Figure 2.3 illustrates these four regions and the equations Young developed to describe them.

Figure 2.3: Young's Predicted Load versus Deflection Behaviour (After Young 1959)

The meaning of each symbol presented in Figure 2.3 is as follows:

- σ_t = membrane stress
- σ_p = equivalent yield stress of material in an elemental strip of plate
- w = displacement
- h = plate thickness
- q = intensity of the uniformly distributed load
- q_p = collapse load
- γ = edge fixity parameter

Young also investigated the effect of non-rigid in-plane support at the plate edges. By introducing a variable "k", the edge elasticity coefficient in the analysis, the variable edge restraint could be accounted for. The effect of decreasing the edge restraint was to allow the deflection at the end of the elastic response phase to increase.

Relevance

The principal use of Young's work will be in the development of a new design method. The experimental results can be used for validation.

**2.3 McDermott, Kline, Jones, Maniar and Chiang; "TANKER
STRUCTURAL ANALYSIS FOR MINOR COLLISIONS" SNAME
Transactions Vol 82, 1974.**

Summary

McDermott et al (1974) studied the behaviour of a longitudinally framed oil tanker during a minor collision with another vessel. The various phases of structural collapse were examined with the absorbed energies summed to determine the total energy absorbed.

Study Objective

The objective of this study was to develop an analytical procedure to evaluate oil tanker configurations from the point of view of maximizing the absorbed energy capability before rupture during a minor collision. With maximized energy absorption, the risk of cargo spillage during the collision or grounding is minimized.

The study involved inspection of actual collision damage, an experimental program to confirm hypotheses and finally the development of a mathematical model.

Tanker Structural Response to Collision with Another Vessel

The analytical investigation considered the collision of two tankers. The movement of one vessel with an infinitely stiff bow was studied as it collided into the side of a second longitudinally framed oil tanker. This second tanker was analyzed in both single and double hull configurations. The striking vessel was considered to have either a vertical or a raked bow. Perpendicular and oblique angle collision geometry was studied.

The scenario of collision geometry and structural response, as demonstrated by the mathematical model, that resulted in the maximum amount of energy being absorbed was the following:

Colliding Vessel - infinitely stiff vertical bow

Collision Geometry - 90° angle collision

- striking bow impacts at midspan of longitudinals between frames and transverse bulkheads

Struck Vessel - The following sequential structural collapse process occurs:

- (1) Longitudinal plastic bending of stiffened hull plate
- (2) Buckling of longitudinal stiffeners
- (3) Stiffened hull plates unload in bending and immediately reload in plastic membrane tension
- (4) Web frames flanking the strike yield or buckle
- (5) With constant resisting forces from web frames as they yield or buckle the hull plates continue to strain in plastic membrane tension until rupture.

Experimental Program

To confirm and consolidate the assumptions and structural response predictions an experimental program consisting of ten 1:5 scale tests was conducted.

The tested geometries included unstiffened and angle stiffened plate specimens. The specimens were supported laterally at their ends and at the mid-span vertical support points. These midspan support points separated the overall panel into three 30 inch (762 mm) spans (McDermott reported data in Imperial units which have been converted to SI units for use herein). The load was applied to the middle span. An external box-shaped frame served to provide downward restraint on the jacking system applying the load and upward restraint to the horizontal supporting frame. A horizontal test frame anchored the specimen so that membrane tension could be developed. Each test specimen simulated a portion of a ship hull extending over three frame spacings.

The steel used in the tests had a yield strength of 241 MPa, an ultimate tensile strength of 448 MPa and a ductility of 32% elongation in a 203mm specimen.

Experimental Results

The results of Test #1 are presented in Table 2.4. This test was a flat plate with

no stiffening, an unsupported span of 762 mm and a point load at midspan.

Table 2.4: McDermott et al - Experimental Results Test #1

Load (kN)	Deflection (mm)	Load (kN)	Deflection (mm)
11	28	227	155
27	38	245	168
58	53	271	178
76	66	289	180
98	76	316	193
116	97	334	203
138	104	360	218
160	124	378	226
187	135	409	246
205	147	418	Rupture

Failure Criteria

Failure, in a real (ship) sense, was defined as rupture of the hull plate forming a tank boundary, thus causing leakage of the vessel contents. The failure criterion used in the mathematical model were either:

- (i) overall stretching during the membrane tension phase was limited to 0.10 mm/mm strain
- (ii) the critical bend angle leading to maximum tensile strain was exceeded

$$\epsilon_m = \frac{4}{3} \frac{\sigma^*}{\sigma_u - \sigma^* \cos \Theta} \sin \Theta \cos \Theta \quad [2.3.1]$$

where, ϵ_m = membrane - tension strain at hull rupture

$$\sigma^* = (\sigma_y + \sigma_u)/2$$

σ_y = yield stress

σ_u = ultimate stress

Θ = one half of the critical bend angle, where

$$\Theta = \tan^{-1} \delta / 0.5L$$

δ = maximum deflection

L = distance between support at time of rupture

McDermott suggests that ϵ_m is approximately equal to 1.5 times the tension test ductility in a two inch gauge length.

Mathematical Model

McDermott et al (1974) developed two equations for the plate load versus deflection response and one to estimate the deflection at rupture.

The equations for load versus deflection are:

Simple Form:

$$P = T \left(\frac{\delta}{L'} + \frac{\delta}{L''} \right) \quad [2.3.2]$$

where, P = total load

L' = horizontal distance from load to nearest support

L'' = horizontal distance from load to other support

δ = maximum deflection at load

T = area * $(\sigma_y + \sigma_u)/2$

σ_y = yield strength

σ_u = tensile strength

Complex Form:

$$P = P_{bc} + T * \left(\left(\frac{L'}{(\delta + \delta_{bc})} \right)^2 + 1 \right)^{-1/2} + T * \left(\frac{L''}{(\delta + \delta_{bc})} \right)^2 \quad [2.3.3]$$

where, T = Area * σ_u

δ_{bc} = deflection at instant when bending phase is assumed to terminate and membrane tension phase begins

P_{bc} = load at δ_{bc}

The equation for the deflection at rupture is:

$$\delta_{bc} = (K * (L'\epsilon_r + L''\epsilon_l + \Delta_s) + \delta_{bc}^2)^{1/2} \quad [2.3.4]$$

where,

$$K = 2L'L''/L_1$$

$$L_1 = L' + L''$$

ϵ_r = reserve ductility available within L' during
membrane tension phase

ϵ_l = strain within L'' when strain within L' is ϵ_r

Δ_s = amount by which the span shortens during loading

Figure 2.4 illustrates the load versus deflection response for McDermott's Test #1. This test was for an unstiffened plate loaded by a single point load at mid-span.

Figure 2.4: McDermott Test #1: Load versus Deflection Results

Conclusions

Generally in a ship collision more energy can be absorbed without rupture if the ships frames are weak and if the hull is single skinned and not a double hull of the same structural weight.

Of the energy absorbed 2/3 to 9/10 is absorbed during the membrane tension phase of the structural response. The total energy figure (ie. maximum deflection and maximum load) is very sensitive to the value of the ultimate strain that the hull plating can withstand.

Relevance

This work by McDermott et al (1974) provides an excellent description of the stiffened plate panel response at ultimate loads. Additionally, it suggests the proper proportioning philosophy of scantlings to achieve maximum ultimate load carrying capability. Finally, experimental results are provided for use in the mathematical and computer model development and validation.

2.4 Jones N.; INFLUENCE OF IN-PLANE DISPLACEMENTS AT THE BOUNDARIES OF RIGID-PLASTIC BEAMS AND PLATES;

International Journal of Mechanical Science, 1973.

Summary

Jones studied the influence of small in-plane displacements at the boundaries of various laterally loaded beams. The theoretical predictions indicate that an extremely small in-plane support displacement can change the response from that of an axially restrained beam to that of a freely supported beam. Jones concluded that it is extremely important for designers to know precisely the amount of axial restraint at the supports of beams or plates.

Numerical Analysis

Jones considered the load versus deflection response of beams or infinitely long plates loaded with a uniform pressure or a centrally located concentrated load. The boundary conditions incorporated into the analysis are for rotationally free and rotationally fixed supports with in-plane boundary (displacement) stiffness ranging from full restraint to no restraint.

Figure 2.5 illustrates the extreme sensitivity of the load/deflection relationship to changes in the support in-plane restraint. When axial restraint is not present the plate collapses once a three hinge collapse mechanism is formed. Providing restraint allows the plate to carry considerably more load. In Figure 2.5 β indicates the amount of axial restraint and δ is the central deflection ratio of the plate, defined as the ratio of the central deflection to plate thickness. $\beta = 0$ indicates full axial restraint and $\beta = -1/2$ indicates no axial restraint. Jones concluded that at central deflection ratios greater than three the plate load deflection curves for different values of β greater than $-1/2$ start to become coincident, as in the figure the lines are starting to come together as δ increases.

P = load
 p = uniformly distributed pressure
 P_c = static collapse load
 p_c = static collapse pressure
 δ = central deflection ratio
 $= W / H$
 W = central deflection
 H = plate thickness
 β = in-plane boundary stiffness indicator

Figure 2.5: Sensitivity of Load vs Deflection
Relationship to Changes to In-Plane Restraint
(After Jones 1973)

Jones presented an example to demonstrate the sensitivity of the load versus deflection relationship to boundary in-plane stiffness. With this example he demonstrated that for a fully axially restrained plate 100 inches long the load required to achieve a 1 inch midspan deflection is four times greater than the load required to achieve the same deflection if no restraint is provided.

Relevance

Jones provides an understanding of the sensitivity to in-plane stiffness at load levels resulting in deflection to plate thickness ratios of three or less.

2.5 Ratzlaff K. and Kennedy D.J.L.; ANALYSIS OF CONTINUOUS STEEL PLATES SUBJECTED TO UNIFORM TRANSVERSE LOADS; Canadian Journal of Civil Engineering Vol 12, 1985.

Summary

Ratzlaff and Kennedy recognized, as have others, that for plated structures where large deformations under load do not cause serviceability concerns, utilizing the inelastic membrane forces generated in the plate to support the load is a way to create structures more economically. An extensive literature search by the authors revealed that a satisfactory solution does not exist for the load-deflection response of transversely loaded flat steel plates with clamped edges beyond the elastic limit when both flexural and membrane action are taken into account.

Study Objectives

The objectives of the work presented by the authors was to analyze the load-deflection response of initially flat rectangular plates with clamped edges under uniform transverse loads. They also sought to establish an analytic procedure for predicting this response including inelastic behaviour.

Results

Through analysis and a literature search the authors established that three regions of behaviour exist for the transversely loaded plate:

- (1) Elastic flexural-membrane action
- (2) Inelastic flexural-membrane action
- (3) Inelastic-membrane action.

The authors have established the boundaries of these regions to provide an overall domain in which the load-deflection response lies. This response domain is illustrated in Figure 2.6.

**Figure 2.6: Ratzlaff and Kennedy - Behaviour Domain
(After Ratzlaff and Kennedy 1986)**

The load value is expressed non-dimensionally as the ratio of the uniform load that would cause plastic collapse in the absence of elastic restraint. Similarly the deflection value is expressed non-dimensionally as the ratio of the deflection to the plate thickness. The curves labelled in circles which define the boundaries of the admissible response domain were established as follows:

- Curve E, from points O to A, is based on Timoshenko's elastic analysis extended beyond the point when yielding due to bending at the support begins at the extreme fibres to the point of full plastic moment across the section which is reduced to account for the tensile membrane forces. Point A was established assuming that the moment-curvature relationship was elastoplastic.
- Curve F, formed by three straight line segments, is the response of the plate when flexural action only occurs. The curve is established by allowing one edge support to move freely towards the other. It changes

slope once plastic hinges form at the plate edges and maintains a constant slope until a mid-span hinge forms at the collapse load. At this load the plate will continue to deflect without any increase in load. Curve F establishes the lower boundary of the combined membrane and flexural domain.

The response considering membrane action only was examined to establish the upper boundary of the behaviour domain. This analysis results in the curves N, M_i , and M_e which are more clearly seen in Figure 2.7.

Figure 2.7: Ratzlaff and Kennedy - Load - Deflection Behaviour; Membrane Action Only (After Ratzlaff and Kennedy 1986)

- Curve M_e is established considering elastic membrane action. The deflection is a result of axial straining as support only comes from axial forces. In the elastic region the straining is uniform along the length of the plate. The corresponding load deflection curve is a smooth non-linear curve constantly increasing in slope. This curve is valid up to point B when the membrane load at the edges equals the yield load. After this point the deflection and load carrying capability continues until yielding spreads completely from the edges to the centre of the span. This is

curve M_i . Both M_e and M_i have been constructed using the elastic value of Poisson's ratio; however, beyond yield Poisson's ratio changes from the elastic value to the fully plastic value of 0.5. If this value is used in the equation used to generate curve M_i the result is curve N. Above yield, it would be expected that the load versus deflection value would migrate from M_i to N as Poisson's ratio increased and as the extent of yielding increased across the plate.

- Curve Y, on Figure 2.6, is established to determine the upper boundary between curves N and M_e . The curve is based on full yielding of the cross-section first in flexure and then in tension. The curve thus necessarily starts at q/q_c equal to one and becomes tangent to curve M_i (which is membrane only at yield). It must represent an upper limit to the behaviour as it is based on full cross-section yield.
- Curve R connects point A and B assuming that the flexural contribution decreases parabolically. The flexural contribution was taken to be the difference between the ordinate of point A on curve E and the ordinate of point C on curve M_e .

From the analysis under which these curves have been created their union and intersection points as shown in Figure 2.6 represent the domain of possible load versus deflection response.

The authors prepared a non-linear finite element analysis, using ADINA, of a plate and compared the predicted load versus deflection response to the admissible response domain. At higher loads the deflection predicted was lower than anticipated, although immediately after membrane edge yielding (point B on Figure 2.7) the response did start to migrate towards curve N as expected, but then fell back. It was anticipated that this was due to the bilinear stress versus strain material curve that was used in the ADINA program and thus the beneficial stiffening affects due to strain hardening were not realized.

The experimental work performed by Young in 1959, see Section 2.2, was also compared to the admissible domain with good agreement. Young's data (with reference to Figure 2.6) followed curve E, then curve R and then gradually curved up to follow

curve N.

Conclusions

The admissible response domain established is valid, with further experimental or FEA work required to establish more exact behaviour. Measured stress versus strain curves are required. Also a predicted value of the load/deflection at rupture must be determined.

2.6 Ratzlaff K. and Kennedy D.J.L.; BEHAVIOUR AND ULTIMATE STRENGTH OF CONTINUOUS STEEL PLATES SUBJECTED TO UNIFORM TRANSVERSE LOADS; Canadian Journal of Civil Engineering Volume 113, 1986.

This paper is a continuation of work published in 1985 in which Ratzlaff and Kennedy established the admissible load versus deflection response domain of an initially flat rectangular steel plate, clamped on all four edges and subjected to an uniform transverse load (from hydrostatic pressure).

Study Objective

The objective of the continued study was to provide analysis that predicts accurately the load-deflection response, the failure mode and load, and to confirm these values by tests.

Analysis

Ratzlaff and Kennedy continue from their 1985 work and publication by preparing an improved finite element (ADINA) analysis and performing experimental work.

The improved finite element analysis (FEA) utilized a linearized uniaxial stress - strain material property curve. Using this curve strain hardening effects are included in the analysis. This analysis illustrated that the load versus deflection curve did move above the hypothesized curve N of the admissible response domain (Ratzlaff and Kennedy (1985), see Figure 2.6), as predicted, as tensile stresses at the plate edge exceeded the yield stress.

For their experimental verification Ratzlaff and Kennedy tested two 1.47 mm thick steel plates under hydrostatic pressure. The plates had a short axis length to thickness ratio of 103 and an aspect ratio (long to short axis dimension ratio) of 3. With this aspect ratio the plate was in a plane stress condition with zero strain in the long direction. The

plates were fully restrained on all four sides against rotation and translation.

Test Parameters

Geometric Properties of the Plate

Thickness = 1.47 mm

Width = 152 mm

Length = 456 mm

Material Properties of the Steel

$E = 197100 \text{ MPa}$

$\sigma_y = 270 \text{ MPa}$

$\sigma_u = 355 \text{ MPa}$

$\epsilon_{st} = 0.017$

$\mu_{elastic} = 0.30$

$\mu_{failure} = 0.50$

The average results of the two tests are listed in Table 2.5. In this table "Load Ratio" is defined as the ratio of the applied load to the value of load that would cause a plastic mechanism to form in a similar plate of zero aspect ratio. "Deflection Ratio" is defined as the ratio of actual deflection to plate thickness. Figure 2.8 illustrates the load versus deflection curves for the experimental work.

Table 2.5: Ratzlaff and Kennedy Test Results

Load (kPa)	Deflection (mm)
540	3.3
810	5.0
1080	6.6
1345	8.8
1615	10.5
1885	11.8
2155	13.2
2420	14.7
2690	16.2
2960	17.6
3230	19.3
3500	21.1
3670	21.7
3670+	Rupture

Ratzlaff and Kennedy presented two failure criteria based on a shear limit and a maximum strain limit for the material. These limits were developed as follows:

Shear Limit:

Metal forming operations show that when a plate is fully restrained, both in bending and tension, at the edges and subject to a sufficiently large hydraulic pressure it will shear at the edges similar in fashion to a high strength bolt shearing in a lap joint. Based on this evidence, through extension of the Von Mises-Huber-Hencky yield criterion to ultimate strength conditions, the ultimate shear strength was taken as $\sigma_u/(3)^{1/2}$. This determination is supported by the fact that CSA Standard S16.1-M84 "Steel Structures for Buildings - Limit States Design" ... gives $0.60A_bF_u$ as the ultimate shear resistance of such a high strength bolt.

Maximum Strain Limit:

The limiting strain can result from any combination of axial and bending strains and is set to the ultimate strain from the experimentally determined stress-

Figure 2.8: Ratzlaff and Kennedy - Load versus Deflection Test Results

strain curve. The maximum strains occur at the plate edge, midway along the long side, where the strain gradient is very high. The gauge length for any experimentally determined maximum permissible strain should be essentially zero. To establish the strain at failure with a zero gauge length several tensile tests were performed at varying gauge lengths and the results were extrapolated to the zero gauge length condition. The final results suggest that for zero gauge length the maximum strain (true fracture strain) in a plate under biaxial tensile stress is equal to 1.31, calculated as:

$$\epsilon_f = \ln(A_o/A_f) \quad [2.6.1]$$

where A_o = net cross-sectional area

A_f = area of the fracture surface

Predicted Behaviour

Based on a hypothesized plate edge localized geometry, where the deformed shape at the edge is a cubic function, the authors related the membrane stress, lateral hydrostatic pressure and edge inclination to determine a set of equations by which the maximum deflection for a given hydrostatic load, stress and strain value is calculated. The process described is iterative. By solving this set of equations a load versus deflection curve can be generated.

Relevance

The principal use of this work will be in the development of a new design method. The experimental results can be used for validation and the developed prediction process can be compared to the admissible domain of response values.

2.7 Ratzlaff K.P.; EXPERIMENTAL STUDY OF TRANSVERSELY LOADED STEEL PLATES; Ph.D Thesis University of Alberta, March 1995.

Summary

Ratzlaff conducted tests on steel plates to determine the load/deflection behaviour of the plates under transverse hydrostatic pressure. Finite element analyses, using ADINA, of the plates were performed and these analytical results were compared to the test results.

Comparisons of the test results and analytical results were also made to the response domain described in Ratzlaff and Kennedy (1985). Comparison was also made between the failure load and displacement found in the test and those predicted by the two failure criteria, presented in Ratzlaff and Kennedy (1986), based on the edge effects relating to a maximum tensile strain and to ultimate shear loading.

Experimental Program

Ratzlaff tested four plates, two approximately 7 mm thick, one approximately 13 mm thick and one approximately 16 mm thick. The plates were welded, around their perimeter, to a 150 mm thick steel plate. Fluid was pumped between the plates to create a hydrostatic load. The plates were loaded to rupture, indicated by leaking of the hydraulic fluid.

Ratzlaff conducted ancillary tests to determine the material properties of each of the tested plates.

Test Results

The experimental results for the tests are summarized below. The plate geometry and the material properties for the test results presented in tabular form in Table 2.6 and graphically in Figure 2.9 are as follows:

PLT-6 Test 1:

Plate Thickness = 6.8 mm
Plate Breadth = 1043 mm
Initial Deflection = 8.0 mm
 $\sigma_y = 294.4$ MPa
 $\epsilon_y = 0.00144$
 $\epsilon_{st} = 0.0147$
 $\sigma_u = 477.2$ MPa
 $\epsilon_u = 0.191$
 $\epsilon_f = 0.295$

PLT-6 Test 2:

Plate Thickness = 7.1 mm
Plate Breadth = 1043 mm
Initial Deflection = 2.5 mm
 $\sigma_y = 292.5$ Ma
 $\epsilon_y = 0.00140$
 $\epsilon_{st} = 0.0230$
 $\sigma_u = 453.4$ MPa
 $\epsilon_u = 0.215$
 $\epsilon_f = 0.315$

PLT-12:

Plate Thickness = 13.4 mm
Plate Breadth = 1028 mm
Initial Deflection = 5.2 mm
 $\sigma_y = 318.9$ MPa
 $\epsilon_y = 0.00154$
 $\epsilon_{st} = 0.0175$
 $\sigma_u = 521.8$ MPa
 $\epsilon_u = 0.190$
 $\epsilon_f = 0.263$

PLT-16:

Plate Thickness = 16.5 mm
Plate Breadth = 1016 mm
Initial Deflection = 6.8 mm
 $\sigma_y = 307.4$ MPa
 $\epsilon_y = 0.00149$
 $\epsilon_{st} = 0.0160$
 $\sigma_u = 512.3$ MPa
 $\epsilon_u = 0.184$
 $\epsilon_f = 0.268$

Table 2.6: Ratzlaff (1995) Plate Tests conducted 1989-02-14

PLT-6 Test 1		PLT-6 Test 2		PLT - 12		PLT - 16	
q (kPa)	W _{max} (mm)	q (kPa)	W _{max} (mm)	q (kPa)	W _{max} (mm)	q (kPa)	W _{max} (mm)
0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
1.6	0.1	3.2	1.3	8.9	0.5	46.4	1.1
18.0	4.3	6.0	2.2	47.3	2.3	112.6	2.6
39.2	7.1	9.5	3.1	95.8	4.3	188.6	4.2
60.4	9.2	12.7	4.0	139.0	6.0	227.4	5.0
87.3	11.3	18.5	5.0	183.9	7.6	262.4	5.7
123.8	13.9	24.1	6.0	222.9	8.9	310.9	6.6
153.3	15.4	30.8	7.0	268.9	10.3	342.5	7.2
208.1	18.1	37.6	7.9	317.9	11.8	408.7	8.5
249.0	19.9	42.8	8.5	358.8	13.1	471.5	9.6
293.8	21.9	54.0	9.7	405.6	14.6	520.9	10.8
336.1	23.4	61.8	10.4	455.7	16.3	553.3	11.6
366.1	24.7	72.8	11.3	496.2	17.7	598.3	12.8
408.8	26.5	54.6	12.3	543.3	19.2	639.3	14.0
454.6	28.5	97.0	13.2	587.9	20.6	684.6	15.3
493.2	30.3	113.2	14.3	634.9	22.1	746.6	17.0
556.0	33.9	128.9	15.4	679.6	23.4	798.7	18.3
590.6	36.0	142.6	16.4	720.3	24.5	872.6	20.2
630.6	38.4	159.5	17.4	778.7	26.0	936.6	21.7
673.4	41.6	174.3	18.2	835.5	27.5	997.1	23.1
719.3	44.5	215.8	20.5	894.7	29.0	1074.6	24.8
742.7	47.0	273.9	23.1	949.1	30.3	1134.6	26.1
790.4	49.2	321.3	25.0	1010.2	31.6	1178.0	27.0
824.1	51.4	373.5	26.9	1067.6	33.0	1238.2	28.2
867.1	54.2	422.5	28.9	1119.8	34.2	1265.0	29.3
904.7	57.0	479.5	31.5	1177.7	35.4	1344.3	30.2
952.7	59.8	535.4	34.2	1234.6	36.7	1400.6	31.2
1004.8	63.2	593.1	37.6	1238.9	37.6	1455.0	32.2
1023.7	64.0	622.5	39.8	27.2	24.8	1518.1	33.3
1057.0	66.3	686.8	43.2	88.8	25.7	1594.2	34.6
1101.9	68.7	745.0	47.0	290.3	28.5	1668.3	35.9
1142.6	70.8	809.6	50.9	690.2	32.8	1742.6	37.2
1151.5	71.4 *	857.9	53.9	1111.2	36.5	1803.7	38.2
		913.0	56.8	1227.8	37.4	1864.5	39.3
		974.0	60.5	1351.5	39.3	1867.2	40.0
		1029.9	63.7	1468.5	42.0	1966.4	41.0
		1100.0	67.9	1581.3	44.5	2036.5	42.2
		1143.3	70.4	1674.2	46.6	2111.3	43.6
		1201.7	73.8	1784.6	49.1	2191.2	45.0
		1259.1	77.1	1889.1	51.8	2263.1	46.2
		1307.2	79.6	1908.7	53.6	2338.1	47.6 *
		1364.4	82.3	2414.5	49.1		
		1424.0	85.1	2475.9	50.4		
		1410.7	85.4 *	2489.8	51.4 *		

* indicates plate rupture

Figure 2.9: Ratzlaff - Plate Test Results

Ratzlaff predicted the loads that would cause failure, based on a maximum strain limit and a maximum shear loading, of the test specimens. These failure predictions, listed in Table 2.7, were not accurate.

Tabl. 2.7: Ratzlaff - Test to Predicted Failure Ratios

Test Specimen	Test to Predicted Failure Ratios	
	Maximum Strain Limit	Shear Limit
PLT 6 Test 1	0.25	0.32
PLT 6 Test 2	0.29	0.40
PLT 12	0.18	0.24
PLT 16	0.17	0.24

Parent plate material properties were considered in the prediction of the failure loads, heat affected zone (HAZ) properties were not considered. Failure (rupture of the plate) initiated in the relatively coarse grain HAZ.

2.8 Kennedy S.J. and Cheng J.J.; BEHAVIOUR OF TRANSVERSELY LOADED CONTINUOUS STEEL-CONCRETE COMPOSITE PLATES; University of Alberta Structural Engineering Report No 150; July 1987.

Summary

Kennedy and Cheng conducted tests on concrete/steel composite plates to establish the load/deflection behaviour of the plates under transverse loading with axial restraint. Ancillary tests were performed to determine the materials properties of the steel and concrete and to establish local behavioural characteristics such as the coefficient of friction between the steel and concrete. Analytic models were proposed and developed which predict the response of the panels under load given the various parameters of geometry, boundary stiffness, material properties and load geometry.

Experimental Program

Six large scale tests were performed to establish the panel deflection behaviour. Five of the panels were steel/concrete composite construction, consisting of two steel plates with a concrete core. Mechanical shear interconnection was not provided. One test was conducted with two steel plates one on top of the other, again with no mechanical shear interconnection provided. The length and width of the panels tested were 2850 mm and 500 mm respectively. The thickness of the steel plates and the concrete core varied from 3.18 mm to 6.35 mm and from 120 mm to 203 mm respectively. Four equal magnitude point loads were applied to each panel with the spacing between loads equal to 610 mm. The distance between the supports and adjacent load was 510 mm. Axial restraint was provided by composite bulkheads in flexure, a horizontal test frame comprised of hollow structural sections and box girders with a combined stiffness of 550.8 kN/mm.

Material Tests

Five coupons from both the longitudinal and transverse direction of rolling of each of the four steel plates were tested as per ASTM E8-85a (1985) to determine their uniaxial stress-strain characteristics.

From a comparison of the initial and final dimensions of the steel plates in the composite panel tests it was apparent to the authors that the material had experienced a state of stress lying between uniaxial stress and plane strain. Plane strain being defined as a condition where σ_x , τ_{yz} , τ_{zx} , and ϵ_x are zero. The x, y, and z directions taken to be along the length, through the thickness and across the width of the plate respectively.

Because of the complexity of performing a plane strain test on the plate, a series of tests were conducted to give a semi-empirical transformation model to transform the uniaxial stress strain relationship to plane strain conditions and to provide data to develop an analytic model. The testing was conducted on three specimens taken from the same piece of seamless 1026 steel tube. One cylindrical specimen was tested in axial tension (uniaxial stress) and the other two were tested under plane strain conditions (axial tension combined with internal pressure).

From these tests Kennedy established an analytic method for transforming a uniaxial stress strain curve to a plane strain stress strain curve, refer to Section 3.3.

Plate Test Results

The results of the all steel test and the test with a 152 mm concrete core and approximately 5 mm plate are presented in Table 2.8. The plates were loaded to rupture or to a load level slightly less than the rupture load. Severe cracking at the steel plate upper edge signified eminent rupture.

Table 2.8: Kennedy and Cheng - Experimental Results

COMPOSITE TEST C152T4 Steel 2 * 4.87 mm Concrete 152 mm		ALL STEEL TEST Top Plate 6.90 mm Bottom Plate 6.96 mm	
Total Load (kN)	Deflection (mm)	Total Load (kN)	Deflection (mm)
0	0	0	0
25	8	25	30
50	11	50	40
100	20	100	55
200	70	200	80
300	103	300	98
400	118	400	109
500	141	500	117
600	176	600	126
700	205	700	135
800	228	800	145
900	247	900	162
1000	266	1000	182
1100	283	1200	219
1200	300	1400	255
1300	320	1600	295
1400	340	1800	319
1500	360	2000	347
1600	381	2200	376
1700	401	2400	397
1800	420	2600	426
1900	440	2800	442
2000	480	3000	469
2200	497	3118 *	485
2268 *	510		

* indicates plate rupture

Analysis

The behaviour of axially restrained plates was analyzed and two models for use in the prediction of load versus deflected behaviour were developed. The first model, a flexural model, is applicable for small length to depth (thickness) ratios. The second model, of membrane behaviour, is only applicable at large length to depth ratios.

The membrane model is of primary interest here since over 90% of the load

resistance of a plate loaded to rupture is membrane action. Kennedy's membrane model has been built upon and is the basis for the model developed in this work. The analytical procedure which describes the membrane response is an iterative process.

For any applied load the following steps are used to determine the corresponding centre span deflection:

- (1) a small value of centre span deflection, y , is assumed;
- (2) based on the distribution of loads the horizontal force, H , is calculated;
- (3) the internal forces, corresponding strains (based on linearized stress strain curves) and extensions of each segment are calculated;
- (4) the horizontal projection of each extended segment is calculated;
- (5) the sum of these projections is compared to the original length between the flexible boundary (support) and centre span. If the projected length of the membrane is greater than the original length, the centre span deflection, y , is increased and steps 2 to 4 are repeated until the lengths are in close agreement;
- (6) the resulting horizontal force, H , is applied to the boundary and the corresponding x -displacement is determined;
- (7) if the x -displacement of the boundary due to the load, H , is greater than the displaced position of the boundary, then the boundary is displaced by an amount Δ_x , and a new length between the boundary and centre span is calculated;
- (8) steps 1 to 7 are repeated until the x -displacement of the boundary due to the horizontal load is in agreement with the displaced boundary position;
- (9) a larger load is assumed and the analysis continued until failure, defined as rupture of the steel plates, occurs.

Failure Criterion

The Von Mises-Huber-Hencky yield criterion extended to ultimate for plane strain

stress conditions was used for predicting the shear tension failure of the plate at the support (midway along the long side of the panel). Refer to Section 3.4.1 for details.

Conclusions

From this work the following conclusions were drawn:

- (1) The transformation model developed for converting the longitudinal stress-strain curves of a conventional uniaxial tension test to one for plane strain yielded results which were in excellent agreement with the test results from the plane strain and the uniaxial stress tests.
- (2) The composite plates exhibit two distinct types of behaviour, flexural and membrane with a transition from one to the other.
- (3) The flexural stiffness of steel-concrete composite plates is chiefly a function of depth of the concrete core.
- (4) The ultimate load carrying capacity is a function of the steel plate thickness, support conditions and the load distribution.
- (5) In all tests, a shear tension failure occurred when a fracture strain or an ultimate stress of one or more of the plates at the support was exceeded.
- (6) In all tests, the ultimate failure mode, the centre span deflection and the deflected shape of the structure were about the same, regardless of plate thickness or section depth.
- (7) The average maximum centre span deflection at failure was 1 in 5.46 of the span length and the average maximum angular rotation at the support was 29.8 degrees.
- (8) Measurements of plate dimensions before and after the test showed that the critical section of the plate, where rupture occurred, was in a state of plane strain.
- (9) A membrane model was developed to describe the load deflection behaviour of steel plates of finite width. The model is in close agreement with the test results. The model is general and can be applied to other

load configurations and boundary stiffnesses and can be used to determine the energy absorption capacity of the plate and the strength and stiffness of the restraint system so that the required behaviour could be achieved.

- (10) If full membrane capacity is required, then the in-plane restraint mechanism must be able to develop a load equal to the yield load of the plate element divided by the cosine of the angle that the critical plate segment makes with the non-deformed position.
- (11) Variations in boundary stiffness alter the extent of the elastic membrane behaviour region. The less stiff the boundary system, the greater the extent of this region.
- (12) A failure criterion, based on the combined stress state at the critical region and on an extension of the Von Mises-Huber-Hencky yield criterion to ultimate, gives a mean test to predicted value of 1.128, for the membrane failures, with a coefficient of variation of 0.037. This criterion is also presented as a strength interaction equation in terms of the normal and shear loads on the plate cross section.

Relevance

The analytic model of the membrane behaviour of steel plates subjected to transverse loads, and the material behaviour and the failure criterion served as the starting point for the extended work presented in this thesis.

2.9 Brown K.C. and Joubert P.N.; PRESSURE LOADING OF ALUMINUM PLATES; Marine Technology, Vol 27, No. 6, November 1990

In this publication Brown and Joubert report on the findings of their examination and analysis of the damage to the yacht "INCH BY WINCH". The damage occurred during the 1984 Sydney - Hobart yacht race which was characterized by gale conditions lasting for 72 hours with 40 - 50 knot winds. It was speculated that the damage to the hull plating was caused by severe slamming.

Study Objective

The objective of the investigation was to estimate the slamming load pressure needed to cause the observed damage. This was to be achieved through experimental work in which a plate panel similar to the hull plating of the damaged yacht was subjected to a hydrostatic pressure in a test apparatus.

Experimental Work

The sample hull section tested was constructed to replicate as many material aspects as possible of a rectangular panel of INCH BY WINCH. The original bottom hull panels were of welded aluminum construction and consisted of plates, frames and stringers. The thickness of all components was nominally 5 mm and the frame and stringer spacing were 340 mm and 460 mm respectively. The authors constructed an array of nine similar panels to test. To compensate for the fact that the outer panels would have a rotationally free edge in the test, unlike the hull condition, for the outer panels the frame and stringer spacing was reduced to 240 mm and 350 mm respectively. Two arrays were constructed and attached together by half-tubes cut from 50 mm aluminum circular hollow sections, thus creating a 50 mm gap between the panels facing each other. This gap was filled with either oil or water and pumped to the desired pressure.

Five plate conditions were tested so that the global geometric effects of the hull (curved as well as flat sections) could be included. These five conditions were:

- (1) Flat
- (2) Convex 3000 mm radius
- (3) Convex 1500 mm radius
- (4) Concave 3050 mm radius
- (5) Concave 1550 mm radius

The aluminum alloy, A5036-T32, used was the same material as that found in the INCH BY WINCH. The material properties as given by a partial stress versus strain curve in the publication are listed in Table 2.9 below:

Table 2.9: Aluminum Alloy A5036-T32 Stress Strain Data

Stress (MPa)	Strain
0	0.000000
100	0.001440
300	0.002880
216	0.003610
225	0.004310
238	0.006600
247	0.010220
270	0.019080

The central deflection of the panels versus hydraulic load is illustrated in Figure 2.10 for the flat plate only and tabulated in Table 2.9.

Figure 2.10: Brown and Joubert - Flat Panel Central Deflection Under Load

Table 2.10: Brown and Joubert Flat Plate Load versus Deflection

Load (kN)	Deflection (mm)	Load (kN)	Deflection (mm)
0	0	280	7.8
20	1.0	300	8.3
32	1.4	320	8.6
50	2.0	340	9.1
70	2.7	360	9.4
100	3.6	380	9.8
120	4.1	400	10.3
131	4.4	420	10.7
146	4.6	440	11.2
160	5.1	460	11.7
191	5.8	480	12.2
223	6.6	500	12.6
253	7.2	520	13.2

Brown and Joubert determined, taking their test values at face value, that the pressures required to cause the Inch by Winch damage (unacceptable permanent deformation values) were approximately 600 kPa for flat panels, 200 kPa for the panels of 3000 mm radius and 350 kPa for the panels of 1500 mm radius. These pressures are compared to the 64 kPa American Bureau of Shipping (ABS) static design pressure for INCH BY WINCH.

The measured principal strains at the centre of each panel ranged up to a maximum of 8500×10^{-6} .

Brown and Joubert compare the results of a small deflection (beam theory) analysis and large deflection analysis to the test and damage results. Some of their conclusions are:

- (1) Rotational edge conditions, (fixed or free rotation) do not have much influence on the load/deformation relationship at loads near or exceeding the initial yield load.
- (2) Pressures causing deflection above twice the plate thickness are carried primarily by membrane action.
- (3) Pressures substantially higher than the initial yield pressures are

tolerable without noticeable permanent plate deformations.

Brown and Joubert also address the fact that their testing and analysis is static in nature while slamming is clearly an impact type of loading. They conclude that the static analysis is reasonable given the following considerations:

- (1) Measured strains are approximately 8000×10^{-6} .
- (2) Strain rates of 40 are required to have significant affect.
- (3) From (1) and (2) loading velocities of 150 m/s would be needed for the impact loading to be significant.
- (4) Slamming velocities for a 10 metre sailing yacht are in the range of 10 m/s.

Conclusions

- a. Tests indicate that ABS design pressures are underestimated by a factor between three and ten.
- b. The flat plates act primarily as a membrane.
- c. The deformation of convex singly curved panels involves elastoplastic buckling and failure occurs at substantially lower pressures than for flat plates.
- d. The current method (ABS) for design of hull plating is highly questionable and a more valid alternative should be sought.

Relevance

The publication provides test results using an aluminum alloy for use in model development. It is suggested that the design method presently used for hull plating is inadequate.

2.10 SUMMARY

The present state of research and analysis of the load versus deflection behaviour of continuous plates, restrained axially and transversely loaded until rupture occurs suggests:

- (1) there is a definable response domain
- (2) the load deflection behaviour is characterized by three distinct regions, an elastic flexural-membrane response region, an inelastic flexural-membrane response region and an inelastic membrane response region as shown in Figure 2.11. The elastic flexural-membrane response region is a small part of the overall behaviour for plates of length to thickness ratio in the practical range for stiffened plate structures.
- (3) at large deflections membrane action dominates to the extent that 90% of the total energy absorbed in the deflection process is due to the membrane action
- (4) the stiffness of the plate boundary plays a significant role in the elastic membrane response region.
- (5) shear-tension failure (rupture) of the plate occurs at the midpoint of the long side of the panel. The angle between the plate and the horizontal, at the long edge, is approximately thirty degrees for a symmetric load condition that simulates a full span uniformly distributed load.

Figure 2.11: Generalized Load versus Deflection Response

The load versus deflection response for the experimental work described in the preceding subsections of this section are summarized in Table 2.11. The work of Brown and Joubert (1990) is not included because their work examined the behaviour of welded aluminum while all others were steel. The aluminum was much less ductile and rupture occurred at a much lower deflection to thickness ratio than for the steel specimens although the load versus deflection curves for the aluminum and steel are generally similar. In Table 2.11 b/t is the ratio of plate span to thickness.

Table 2.11: Load versus Deflection Results of the Researchers Reviewed

P/P _c	Young * 3/16 Δ/t b/t=135.9	Young * 1/4 Δ/t b/t=99.2	Young * 1/2 Δ/t b/t=49.6	Kennedy All Steel Δ/t b/t=207.4	Ratzlaff & Kennedy Δ/t b/t=113.4	Ratzlaff PLT6 #1 Δ/t b/t=151.2	Ratzlaff PLT6 #2 Δ/t b/t=144.8	Ratzlaff PLT12 Δ/t b/t=76.7	Ratzlaff PLT16 Δ/t b/t=62.3	McDermott Test 1 Δ/t b/t=151.5
0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1	0.44	0.96	0.38	1.66	0.92	1.28	1.47	0.70	0.47	1.15
2	1.68	1.68	1.07	2.56	1.06	1.92	2.14	1.34	1.04	2.30
3	2.13	2.08	1.52	3.06	1.66	2.40	2.65	1.90	1.58	3.44
4	2.64	2.35	1.90	3.48	2.08	3.15	3.06	2.34	2.01	4.59
5	2.94	2.72	2.20	3.89	2.33	3.51	3.45	2.76	2.42	5.18
6	3.24	2.97	2.57	4.41	2.73	3.86	3.80	3.19	2.79	5.69
7	3.46	3.34	2.95	4.68	3.40	4.26	4.16			6.20
8	3.78	3.64		4.95	4.35	4.69	4.56			6.72
9	4.12	4.00		5.30	4.60	5.18	5.10			7.03
10	4.47	4.36		5.64	5.17	6.29	5.62			7.29
12	5.52			6.18	6.32	7.41	6.62			7.80
14				6.68	7.08	8.59	7.65			8.41
16				7.14	8.05	9.65	8.70			9.05
18				7.44	8.89		9.76			9.39
20				7.75	9.71		10.93			9.65
25				8.34	11.95					11.13
30				8.83	14.39					12.64
35				9.58						14.26
40				10.26						15.88
60				15.23						26.25
70				17.75						28.60
80				20.46						30.80
100				24.32						33.00
120				28.00						36.68
140				31.32						41.54
160				34.68						46.58

* plates not loaded to rupture

The values in Table 2.11 have been non-dimensionalized by expressing the deflection as a ratio of the plate thickness and the load as a ratio of the load which would cause a three hinge plastic collapse mechanism. These three hinge plastic collapse load values have been determined, depending upon load type, as follows:

(1) Hydrostatic Pressure:

$$q_c = \frac{4 \times b \times t^2 \times \sigma_p}{L^2} \quad [2.9.1]$$

where, q_c = collapse pressure
 t = plate thickness
 L = distance between plate supports
 b = plate width (typically unit width)
 σ_p = material yield stress for appropriate condition *
 * for plane strain condition

$$\sigma_p = \frac{\sigma_y}{\sqrt{1 - \nu_p + \nu_p^2}} \quad [2.9.2]$$

* for plane stress condition

$$\sigma_p = \sigma_y \quad [2.9.3]$$

where, σ_y = yield stress from uniaxial tension test
 ν_p = plastic value of Poisson's ratio

(2) Four Equally Spaced Equal Point Loads:

$$W_p = \frac{4 \times b \times t^2 \times \sigma_p}{1.216L} \quad [2.9.4]$$

where, W_p = total collapse load

(3) Single Central Point Load:

$$P = \frac{2 \times b \times t^2 \times \sigma_p}{L} \quad [2.9.5]$$

where, P = magnitude of the centrally placed load

Figure 2.12 graphically illustrates the results presented in Table 2.11 for values of P/P_c less than or equal to thirty. Figure 2.13 graphically illustrates the results of Kennedy and Cheng, and McDermott whose failure load ratios were much higher than those of the other researchers. The line representing the work of McDermott (Test #1) differs significantly from the other researchers at higher load levels because it is the only data for a centrally placed single point load.

The effect of variable in plane boundary stiffness was investigated by several of the researchers. It can be concluded that the minimum restraint capacity required at the boundary is equal to the membrane force at rupture in the deflected plate multiplied by the cosine of the angle that the plate makes with the horizontal at the support. Also, it can be hypothesized that to maximize the load carrying capability of the plate the transverse support capability must be equal to the rupture load. If the support fails (yields or buckles) at a lower load than the rupture load the deflection response area will spread to adjacent plate panels thus increasing the membrane length, the load carrying capability, and the overall region of plastically deformed plating.

Given the above behaviour characteristics a method of analysis that can accurately predict plate deflection for a given load geometry and boundary arrangement is sought. Additionally, since the nature of some extreme value loadings is not precisely known the ability to predict the load nature and value that would result in a given deformed shape would be invaluable.

Figure 2.13: Non-Dimensional Plot of Load versus Deflection Results of Young (1959), Ratzlaff & Kennedy (1985), and Ratzlaff (1995) (Complete) and for Kennedy and Cheng (1987) and McDermott et al (1974) (Load Ratios < 30)

Figure 2.14: Non-Dimensional Plot of Load versus Deflection Results of Kennedy & Cheng (1987) and McDermott et al (1974)

3.0 PLATE BEHAVIOUR UNDER TRANSVERSE LOADS

Given descriptions of the plate response to transverse loads it is reasonable to expect that an analytical method to predict the response can be developed. To accomplish this a clear understanding of the mechanics of the response is necessary.

3.1 Plate Deflection

The resistance of a plate panel, with edges restrained in the plane of the plate, to transverse load is initially provided by bending. Membrane forces develop in the plate as the load and deflection increase. At deflections greater than the thickness of the plate the membrane forces dominate. As the loaded plate deflects further plastic hinges (zones of inelasticity) quickly form at the plate edge, signifying the end of the elastic response phase. The stages of the load deflection response were illustrated as a curve in Figure 2.11. These inelastic zones spread across the plate with continued loading. Once the inelastic zones reach completely across the plate the combined inelastic flexure and membrane response ends. Additional load is carried by inelastic membrane action until rupture. This final portion of the response curve is marked by a distinct change in slope, caused by the onset of strain hardening in the material.

3.2 Material Behaviour

As the loaded plate of aspect ratio greater than two (typical of marine construction) deflects it carries load primarily across its short axis. The membrane stress that develops would be uniaxial in nature if the plate was formed of a multitude of unconnected strips of small width. Since the plate is continuous in both principal directions the effect is for restraint to be provided in the long direction. The critical zone

at rupture is at the edge of the plate midway along the long side. The failure at this location, which has been verified experimentally, is a shear tension failure. The material is in a state of plane strain. Fracture prediction must be determined for this stress state and critical load combination.

The response characteristics to load of any material are traditionally described with a stress versus strain curve derived from a uniaxial tensile test. This curve can be transformed to a curve describing the stress strain characteristics in the principal stress direction (across the width of the plate) for a plane strain stress state. The transformation model is based on an extension of the Von Mises-Huber-Hencky yield criterion extended to ultimate, octahedral shear stress - shear strain curves and measured values of strain energy density.

The elastic region transformation function for any given strain ϵ is:

$$\sigma_{xps} = \sigma_{xus} / (1 - \nu^2) \quad [3.2.1]$$

For $\nu = 0.30$ and therefore in the elastic region:

$$\sigma_{xps} = 1.0989 \sigma_{xus} \quad [3.2.2]$$

where, σ_{xps} = stress in x direction under plane strain conditions;

σ_{xus} = stress in x direction under uniaxial stress conditions; and

ν = Poisson's ratio.

The equations for transformation for stress and strain in the inelastic region are given by equations [3.2.3] and [3.2.6] respectively.

$$\sigma_{xps} = \frac{\sigma_{xus}}{\sqrt{1 - \nu + \nu^2}} \quad [3.2.3]$$

where, ν = Poisson's ratio based on the value of ϵ corresponding to σ_{xus}

for, $\epsilon < \epsilon_y$ $\nu = 0.3$ [3.2.4]

$\epsilon_y < \epsilon < 0.01$ use equation [3.2.5], Stang et al (1946)

$$\nu_p = \frac{1 - \sqrt{\frac{1+\delta}{1+\epsilon_x}}}{\epsilon_x} \quad [3.2.5]$$

where, ν_p = plastic value of Poisson's ratio;

ϵ = maximum principal strain;

δ = dilatation associated with the volume change that occurs during elastic straining;

$\delta = \epsilon_y (1-2\nu)$;

ϵ_y = yield strain; and

ν = elastic value of Poisson's ratio;

$0.01 < \epsilon < \epsilon_u$ At $\epsilon = 0.01$ $\nu = 0.455$

$\epsilon = \epsilon_u$ $\nu = 0.500$

For $0.01 < \epsilon < \epsilon_u$, ν_p varies linearly between the two values above.

$$\epsilon_{xps} = \frac{(1+\nu)}{\sqrt{1-\alpha+\alpha^2}} \epsilon_{xus} \quad [3.2.6]$$

where, ϵ_{xps} = strain in x direction under plane strain conditions;

ϵ_{xus} = strain in x direction under uniaxial stress conditions; and

ν = Poisson's ratio based on the value of ϵ_{xus} .

$$\alpha = -\frac{\nu}{1-\nu} \quad [3.2.7]$$

The yield point occurs at:

$$\sigma_{xps} = 1.1251 \sigma_{xus} \quad [3.2.8]$$

$$\epsilon_{xps} = 1.0238 \epsilon_{xus} \quad [3.2.9]$$

The area under the stress versus strain curve represents the energy of distortion per unit volume. The ratio of energy of local plastic distortion per unit volume for the plane strain to the uniaxial stress condition gives a measure of the reduced ductility. The maximum strain at rupture for the plane strain condition was determined such that the area under the transformed stress-strain curve was equal to the ratio of the strain energy

of the plane strain to uniaxial stress case multiplied by the area under the uniaxial stress-strain curve.

3.3 Failure Mechanism

Failure is defined as a shear tension failure (rupture) or tearing of the plate. Failure of a plate restrained against lateral edge support movement will generally occur at the centre of the long edge. If the load is a "sharp" point load, failure may occur at the point of load application.

Two criteria are used to determine the load at failure; one based on a limiting stress value or equivalent shear and tensile resistance, and the other based on a limiting strain value at the critical section. These are:

- (1) The combination of axial and shear stress exceeds the limiting value as defined by the Von Mises-Huber-Hencky yield criterion extended to ultimate for a plane strain stress state.
- (2) The ultimate strain capacity of the material, in a plane strain stress state, is exceeded.

3.3.1 Failure as Defined by the Von Mises-Huber-Hencky Yield Criterion Extended to Ultimate

The limiting value of stress as defined at ultimate by the Von Mises-Huber-Hencky stress criterion has the general form of:

$$(\sigma_x - \sigma_y)^2 + (\sigma_y - \sigma_z)^2 + (\sigma_z - \sigma_x)^2 + 6(\tau_{xy}^2 + \tau_{yz}^2 + \tau_{zx}^2) = 2\sigma_u^2 \quad [3.3.1]$$

where, σ_x , σ_y & σ_z are the normal stresses in an x, y, z coordinate system, with x parallel to the short axis of the plate, y through the thickness of the plate and z parallel to the long axis of the plate;

τ_{xy} , τ_{yz} & τ_{zx} are the shear stresses in an x, y, z coordinate system; and σ_u is the ultimate stress for the uniaxial stress condition.

The critical location for failure by this criterion will be at the support edge. Here the plate is in a state of plane strain. σ_y , the through thickness stress, τ_{yz} and τ_{zx} are equal to zero. Further, the transverse in plane stress, σ_z , is equal to $\nu\sigma_x$, Poisson's ratio multiplied by the longitudinal stress. Equation [3.3.1] simplifies to:

$$(1 - \nu + \nu^2)\sigma_x^2 + 3\tau_{xy}^2 = \sigma_u^2 \quad [3.3.2]$$

Figure 3.1 below illustrates the forces corresponding to the stresses that the plate is subjected to at its support point. Both of the forces shown in the figure act on the cross-sectional area of the plate (A_o) which is equal to the effective or unit plate width in the long direction multiplied by the plate thickness.

Figure 3.1: Free Body Diagram of Plate at Support Point

Thus from Figure 3.1:

The average normal stress $\sigma_x = P/A_o$

The average shear stress $\tau_{xy} = V/A_o$

where, P = horizontal component of the membrane force

V = vertical component of the membrane force

Substituting into equation [3.3.2] yields:

$$(1 - \nu + \nu^2) (P/A_o)^2 + 3 (V/A_o)^2 = \sigma_u^2 \quad [3.3.3]$$

The ultimate force that the material can support is equal to the ultimate stress

(engineering stress) multiplied by the original cross-sectional area. Let this force equal U so that:

$$U = A_o \sigma_u \quad [3.3.4]$$

Substituting into equation [3.3.3] and simplifying yields:

$$(1-\nu + \nu^2) (P/U)^2 + 3 (V/U)^2 = 1.0 \quad [3.3.5]$$

At failure (rupture), the material is fully plastic and $\nu_p = 0.5$ as discussed in Section 3.2.

Substituting this value into equation [3.3.5] yields equation [3.3.6] which is illustrated in Figure 3.2.

$$0.75 (P/U)^2 + 3 (V/U)^2 = 1.0 \quad [3.3.6]$$

Figure 3.2: Failure: Von Mises-Huber-Hencky Yield Criterion Extended to Ultimate

Failure can be predicted for a given load configuration, plate geometric and material properties.

3.3.2 Failure by Exceedence of Ultimate Strain Value

An alternate failure criterion is rupture of the material, which will occur when the material is strained (combination of axial and bending strains) beyond the ultimate strain

for a given stress state. The ultimate strain is defined as the engineering strain corresponding to the point of incipient necking.

Between load points or between the load and support points the plate carries the load in membrane action which corresponds to axial strain only. At the plate edge (support point) and at points of concentrated load the plate bends sharply which corresponds to concentrated changes in slope, small radius's of curvature and significant bending strains. For large deflections, changes in slope of 30° and a radius of curvature of 1.5 times the plate thickness have been measured. These in combination with axial strains suggest that the critical location of strain will be at the support or point of concentrated load as shown in Figure 3.3. Failure will occur when the summation of the axial and bending strains exceeds the material ultimate strain.

At Load Point

At Support

Figure 3.3: Critical Locations for Ultimate Strain

The axial strains on each side of the bent section can be calculated from the membrane forces. The corresponding axial stress can be determined by dividing the membrane force by the cross-sectional area of the unit or effective width of plate and the strain subsequently found from the material stress strain curve. The axial strain is assumed

to vary linearly across the region of the plate subjected to the discrete angle change.

The bending strains in this region occur over a small length and vary from zero at the ends where the radius of curvature is infinite to a maximum value somewhere in between. It can be expected that the bending stress distribution through the plate thickness is not uniform and that this nonlinear effect that extends the bending stresses an unknown distance into the straight portions of plate.

The shape of the bending stress distribution along the length over which bending occurs is unknown. Bending is zero at the ends and reaches a peak value at some point over the bending length. The radius of curvature which will determine the length of the bending distribution is also unknown.

Thus there are three unknown parameters in the bending strain portion of the ultimate strain exceedence criteria. To establish values for these unknowns the experimental work of Kennedy and Cheng has been utilized as a benchmark. This work was used because photographs of the plate at the support at failure were available, thus giving an indication of the radius of curvature. Figure 3.4 illustrates one of Kennedy and Cheng's test plates at failure.

Figure 3 4: Radius of Curvature at Support at Failure

In this figure the radius of curvature is approximately 1.5 times the plate thickness of the 6.35 mm thick plate illustrated.

Two possible bending strain distribution profiles were examined, a symmetric triangular distribution and a symmetric sine distribution ($-\pi/2$ to $3\pi/2$) as illustrated in Figure 3.5.

Figure 3.5: Bending Strain Profiles Examined at Critical Locations

The total strain, refer to Figure 3.6, was determined by summing the bending and axial strains at several locations along the length of the bend. The maximum strain was found to be approximately the same for the two distribution profiles but at slightly differing locations. The length affected by the non-linear straining was varied until rupture was predicted for the load which caused rupture in Kennedy's test plate. The length of the non-linear extension was found to be approximately twice the plate thickness at each end of the bend length.

Figure 3.6: Total Strain at Support

The ability of this analysis to predict the plate failure of the other researchers tests is discussed in Section 5.0 "IMASP" Model Verification/Comparisons to Experimental Data.

4.0 "INELASTIC MEMBRANE ANALYSIS OF STEEL PLATES" (IMASP) MODEL DEVELOPMENT

Section 3 of this report provides an explanation of the primary mechanics of the load versus deflection response of a plate panel while Section 2.10 provides a summary of the influence of secondary factors on this relationship. This section describes the analytic model and program that have been developed to predict the understood behaviour. The program is named "IMASP" which is an acronym for Inelastic Membrane Analysis of Steel Plates.

4.1 IMASP Objectives

The objectives of the IMASP program are to predict the load versus deflection behaviour of a plate panel and to predict the value of the load that would cause the plate to rupture when given details of the plate geometry, details of the support conditions and values for the stress versus strain characteristics of the plate material. The panel may be a single plate or a composite plate in which two plates are rigidly held a fixed distance apart by a solid core like concrete or a rigid polymer.

4.2 Assumptions

Based on the conclusions of the Literature Review, presented in Section 2, and the response mechanics as presented in Section 3, simplifying assumptions can be made about the behaviour of the plate to be modelled.

Firstly, it is assumed that the plate panel being analyzed has an aspect ratio of

three or more with the aspect ratio defined as the ratio of long to short panel dimensions.

The flexural resistance to transverse load is very small compared to the membrane contribution to the overall plate load carrying capability. Plastic hinges or yield lines form readily at low load levels, approximately 5 to 10 percent of the ultimate strength of the plate. The plate essentially behaves as a cable and the deflection response can be accurately predicted assuming this type of behaviour. Following from this assumption it is further assumed that the deflection response can be predicted with sufficient accuracy by using straight line deflection profiles between the points of load application.

The plate is rigidly supported at either end against transverse displacement and plate deflections are measured relative to the fixed support.

4.3 IMASP Input Variables

It is intended to keep the program input variables to a minimum and limit them to the physical properties of the plate to be modelled. Thus, the input variables to the IMASP program are:

(1) a description of the plate global configuration:

- single panel or double panel composite construction
- an indication of the presence of welding in the construction
- type of load
- type of horizontal boundary stiffness

(2) the plate geometry:

- plate length (short axis length)
- effective plate width
- plate thickness
- horizontal boundary stiffness values (series and parallel)
- location of weld seam if present
- plate separation distance if composite

(3) the boundary conditions:

- type of load of horizontal boundary stiffness
- horizontal boundary stiffness values (series and parallel)

(4) the load data:

- depending upon the type of load the input parameters are a pressure, point load values, locations and angle or a uniformly distributed load value

(5) the material properties of the plate:

- linearized stress versus strain values for the parent plate, heat affected zone (HAZ) and weld metal if present

4.4 Modelled Response Process

The IMASP program determines the deflection profile of a plate under a given load through the use of static equilibrium, material stress versus strain compatibility and geometric compatibility. For a given plate geometry (length, width and thickness), under a known load condition there is only one deflection profile that can exist so that the internal stresses (forces) balance the external reactions.

The unique deflection profile can be found through the use of the following iterative analytical procedure:

- (1) Input plate and problem definition variables as listed in Section 4.3
- (2) Divide the plate into segments over its length, with a segment being the distance between points of load and the distance between a load point and the support. Figure 4.1 illustrates a typical plate under analysis. This plate is supporting eight point loads (thus the plate is broken into nine segments) and it has flexible boundaries.

**Figure 4.1: Typical IMASP Plate Analysis
(After Kennedy and Cheng (1987))**

- (3) Assume a small deflection value at the first load point.
- (4) Use statics to determine the vertical and horizontal reaction forces.
- (5) From the horizontal reaction forces determine the elastic support translation and thus the revised horizontal distance between supports.
- (6) Calculate the internal forces, stresses, and the corresponding strains (from the linearized stress strain curves) in each plate segment.
- (7) Calculate the extension of each segment and determine the horizontal projection of each segment.
- (8) Sum these horizontal projections and compare them to the distance between supports from Step 5.
- (9) If the total projected length is greater than the distance between supports the deflection value at the first load point needs to be increased and steps 4 through 8 repeated. Similarly if the total projected length is less than the distance between supports the deflection value needs to be decreased. The process used to iterate to obtain the value of deflection at the first load point such that the actual and projected lengths are in close agreement is as

follows:

- upper and lower bounds on the deflection value at the first load point are initialized to be equal to zero and fifty percent of the plate width for the lower and upper bound values respectively.
- depending on the difference between the actual and projected lengths the upper and lower bounds are revised to be equal to the assumed deflection.
- the assumed deflection value is set to be equal to the average of the upper and lower bound.

4.5 IMASP Features

The IMASP program will determine the load versus deflection profile for single or double composite plates subjected to point loads, uniformly distributed loads or hydrostatic pressure. A more detailed description of these basic features is as follows:

(1) Double Plates with a Rigid Core

- the program allows for the modelling of two plates which are rigidly held a fixed distance (input variable) apart. The two plates can have different thicknesses, effective plate widths and material properties.
- the program assumes that the outside plate is loaded directly while the inside plate shares this load with an assumed spread from the point load to three point loads separated by one third of the plate separation distance. The central load of the three loads is assumed to be fifty percent of the total load carried by the inside plate and the two sides loads each carry twenty-five percent. Figure 4.2 illustrates the load distribution.

Figure 4.2: Load Distribution From Outside (Loaded) Plate to Inside Plate

- the distribution of the total load between the outside and inside plates is determined by the program and is found through an iterative process with the final distribution causing the same vertical displacement in each plate relative to its support. This follows from the fact that the plates are rigidly held a fixed distance apart.

(2) Point Loads

- the program allows for up to forty point loads to be specified as a loading condition
- each point load is assigned a load magnitude, the distance that the load is applied relative to the plate left support is specified and the angle that the line of action of the load makes with the plane of the plate is specified (measured counter clockwise from the plane of the plate).

(3) Hydrostatic and Uniformly Distributed Loads

- the program models uniformly distributed loads (UDL's) which cover the full length of the plate by changing them to forty equally spaced equal magnitude point loads.
- UDL's are modelled with the assumption that the line of action of the load remains normal to the undeformed plate.
- hydrostatic pressures are modelled such that the line of action of the load remains normal to the plate as it deforms.

In addition to the above basic features the IMASP program considers the effect on the load versus deflection profile due to the following:

(4) Elastic Edge Supports in the Plane of the Undeformed Plate

- horizontal stiffness restraint can be modelled as a set of springs combined in series or in parallel to model the adjacent plate panels and the flexural stiffness of the bulkheads. The stiffness of the horizontal restraint at either end and/or the restraint provided to the outside or inside plates of a double membrane can be different.

(5) Line of Action for Point Loads

- the program is set-up to recognise that certain types of point loads will be of sufficient mass inertia that their line of loading action will remain at a fixed distance from the plate support. Other lighter load sources however make contact with a point on the plate and will move horizontally with the point on the plate as the plate deflects vertically.

(6) Strain Rate Sensitivity

- materials loaded at high rates of strain (the result of collision or impact loads) behave differently than materials loaded quasi-statically (tension coupon test). In general, under high rates of strain a material exhibits higher values of yield and ultimate stress and is less ductile.
- the program provides the ability to model high strain rate loadings through a transformation of the material characteristic stress versus strain curve (based on data available from Marsh and Campbell (1963)).

(7) Parent Plate Material Properties

- the material properties (stress versus strain) are a significant factor in determining the load versus deflection response. Unfortunately, rarely is a full stress versus strain curve for a material available to a structural designer or analyst. Quite often the only material property provided is the yield stress. Based on a sample of full stress versus strain curves (Kennedy and Cheng (1987), Ratzlaff and Kennedy (1986), Bowker (1991) and Ratzlaff (1995)) for steel the following

assumptions have been made to estimate a full engineering stress versus engineering strain curve.

$$E = 207000 \text{ MPa}$$

$$\epsilon_y = \sigma_y / E$$

$$\sigma_u = 1.45 \sigma_y$$

$$\epsilon_u = 0.175$$

$$\text{at } \epsilon = 0.01 \quad \sigma = \sigma_y + 1 \text{ MPa}$$

$$\epsilon = 0.05 \quad \sigma = 0.90 \sigma_u \text{ MPa}$$

$$\epsilon = 0.10 \quad \sigma = 0.95 \sigma_u \text{ MPa}$$

$$\epsilon = 0.15 \quad \sigma = 0.99 \sigma_u \text{ MPa}$$

these values can be requested for use in the IMASP program with the material yield stress the only required input value.

- the effect of varying these values is demonstrated in the Parametric Studies section of this report.

(8) Multiple Material Properties

- the program can be used to model the effects of having a weld seam in the plate and the heat affected zones (HAZ) associated with the welding process.
- it has been assumed that the width of a weld seam (butt weld) is the same as the plate thickness and that the width of the HAZ is equal to 1.5 times the plate thickness.
- the program prompts for the material stress versus strain curve values for the weld metal and heat affected zone if present.
- in recognition of the fact that there is limited stress versus strain data for weld metals and especially for HAZ the program contains a library of σ vs ϵ characteristics which can be selected for these materials.

Weld: Data has been obtained (Bowker (1991)) for an E480XX electrode. This electrode is used for welding steels in the 300 to 350 MPa yield strength range. The curve built into the program is presented in Table 4.1.

Table 4.1: IMASP Weld Material (480xx Electrode) Property Library

Stress (MPa)	Strain (mm/mm)
81	0.001000
162	0.002000
249	0.004000
291	0.006000
317	0.008000
340	0.010000
404	0.015000
453	0.020000
483	0.025000
506	0.030000
521	0.035000
536	0.040000
549	0.045000
562	0.050000
577	0.060000
581	0.070000

HAZ: There is limited data available on the stress versus strain characteristics of HAZ material. One set of stress versus strain data, for a simulated HAZ, using "moderate" weld heat input, in a nominally 350 MPa yield strength steel, was obtained (Bowker(1991)). The tensile testing of this material was performed at -50°C. The parent plate of the same 350 MPa material was also tested at 25°C (apparent yield 355 MPa) and at -60°C (apparent yield 428 MPa). The values of stress versus strain for the simulated HAZ material at 25°C were estimated by prorating from the results of the two parent plate tests. This adjusted to "room temperature" simulated HAZ stress versus strain data is plotted in Figure 4.3. The programming for the IMASP library HAZ σ vs ϵ curve has been set up so that the HAZ curve is created by taking the average of the weld metal stress and the parent plate stress at a given strain value. The validity of this approach can be justified by comparing the library IMASP HAZ for a 350 MPa parent plate material, created using 350 MPa parent plate material based on Section 4.5

Paragraph (7) and the IMASP library weld data, to the temperature prorated data of Bowker (1991). The comparison values are shown in Figure 4.3. The approach reflects the lack of data available.

Figure 4.3: Comparison IMASP HAZ Stress Strain Curve to Bowker (1991) HAZ Stress Strain Data

Figure 4.3 indicates that the approach used to generate HAZ material curves is reasonable. For strains between 20000 microstrain and 120000 microstrain the difference between the IMASP curve and that of the limited available data is not large.

- (9) Stress State Conditions within the plate (i.e. Plane Stress versus Plane Strain)
- the material characteristic stress versus strain properties which the program utilizes are from uniaxial tensile tests. Most plate structures are restrained on all four sides and the material under load is subjected to a biaxial stress

state of plane strain.

- the program allows the user to transform the stress strain curve from uniaxial plane stress condition (tension coupon) to a biaxial plane strain condition. The transformation is not performed automatically in the program because some of the experimental research results that are used for program/model verification are for a plate loaded in the plane stress condition.

(10) Failure Analysis

- the ability to predict the load that will cause the plate to rupture is a key element in the IMASP program. Two failure criteria as described in section 3.3 are utilized. Both criteria have been benchmarked to the experimental work performed by Kennedy and Cheng (1987).

4.6 IMASP Program

The IMASP program has been written in Fortran for use on a PC. The program has been written in a manner such that the main program acts as an organiser for the subroutines which house the computational functions. The sixteen subroutines contain input functions, the steps for the solution iteration process and failure analysis. Output file preparation is performed in the main program.

5.0 MODEL VERIFICATION / COMPARISONS TO EXPERIMENTAL DATA

5.1 General

Comparison to the experimental work of various researchers, summarized in Section 2, has been used as a means of demonstrating the validity and accuracy of the IMASP model. The purpose of the model is to predict the membrane behaviour load deflection curve. Poor correlation with the tests is expected at very low load levels where flexural and flexural membrane behaviour dominate the response. This is especially true for double membranes with rigid cores which will have a larger bending resistance than a single steel plate due to their effective increase in plate thickness.

It is necessary to non-dimensionalize the load and deflection values to make comparisons across the data sources for the various researchers since differing plate thicknesses, plate lengths, loading type and materials were used. The deflection and load are non-dimensionalized by dividing by the plate thickness and by dividing by the load that causes a plastic collapse mechanism to form for the specific load type (as described in Section 2.10) respectively.

5.2 Young (1959)

Young tested three steel strip specimens, each loaded with four near equally spaced point loads of the same magnitude as described in Section 2.2. These specimens were not loaded to rupture.

The three specimens, with dimensions as listed in Table 5.1, had yield strengths of 262 MPa, 264.8 MPa and 262 MPa and a Modulus of Elasticity of 201330 MPa, 195100 MPa, and 195100 MPa, respectively. Young did not provide further details of the material stress versus strain characteristics. The remaining material stress versus strain characteristics assumed in the IMASP computer model were developed as per Section 4.5 and are presented in Table 5.2.

Table 5.1: Young - Dimension of Test Specimens

Specimen #	Length (mm)	Width (mm)	Thickness (mm)
1	615.95	25.146	4.674
2	615.95	31.598	6.400
3	615.95	25.564	12.800

Table 5.2: Young - Specimen Material Properties

Specimen 1 (3/16")		Specimen 2 (1/4")		Specimen 3 (1/2")	
σ	ϵ	σ	ϵ	σ	ϵ
000.0	0.00000	000.0	0.00000	000.0	0.00000
262.0	0.00130	264.8	0.00136	251.0	0.00134
263.0	0.01000	265.8	0.01000	252.0	0.01000
341.9	0.05000	345.6	0.05000	327.6	0.05000
360.9	0.10000	364.8	0.10000	345.8	0.10000
376.9	0.15000	380.2	0.15000	360.4	0.15000
379.9	0.17500	384.0	0.17500	364.0	0.17500

Young's test set up is illustrated in Figure 5.1. Figure 5.2 illustrates the structural idealization of the test that was assumed for the IMASP model.

Figure 5.1: Young's Full Scale Test Apparatus

Figure 5.2: Young - Idealization for Computer Model

IMASP was used to predict the plate deflection response up to the maximum load that Young applied in his experimental work. He did not load his plate specimens to failure. In the IMASP analysis the horizontal edge restraint stiffness was assumed to be infinite.

Tables 5.3 through 5.5 and Figures 5.3 through 5.5 provide the results of the IMASP predicted deflections and measured deflections from Young's test for prescribed levels of load (to the maximum that Young tested) for the 3/16", 1/4" and 1/2" thick plate test specimens. Also given in the tables are the test to predicted ratios of those deflections. These values (Tables 5.3 through 5.5) are for a full parent plate specimen. Young's test specimens were welded to the horizontal reaction frame with fillet welds. Therefore, there will be a heat affected zone (HAZ) at the specimen edges. IMASP was used to model the effect of the HAZ, for the 1/2" specimen only, by using the library HAZ feature as described in Section 4.5. Table 5.6 presents the predicted deflection for given loads for both the full parent plate and combined parent plate and HAZ analysis case. In this table the load versus deflection results are presented for loads increasing to the level at which IMASP predicts failure due to the ultimate strain of the material being reached.

**Table 5.3: Comparison Experimental and Predicted Load versus Deflection
Values for Young's 3/16" Specimen**

	Total Load (kN)	Load/ W_p	Young Defl'n Δ_y (mm)	Δ_y/t	Model Defl'n Δ_m (mm)	Δ_m/t	Defl'n Ratio Test: Model
Specimen # 1 $t = 4.674$ mm $W_p = .769$ kN	0.0	0.00	0.00	0.00	0.0	0.00	-
	0.5	0.65	1.2	0.26	6.5	1.39	0.18
	1.0	1.30	2.8	0.60	8.1	1.73	0.35
	1.5	1.95	7.7	1.65	9.2	1.97	0.84
	2.0	2.60	9.8	2.10	10.1	2.16	0.97
	2.5	3.25	10.3	2.20	11.0	2.35	0.94
	3.0	3.90	12.2	2.61	11.6	2.48	1.05
	3.5	4.55	13.1	2.80	12.2	2.61	1.07
	4.0	5.20	14.0	3.00	12.7	2.72	1.10
	4.5	5.85	15.0	3.21	13.3	2.85	1.13
	5.0	6.50	15.6	3.34	14.7	3.15	1.06
	5.5	7.15	16.3	3.49	16.2	3.47	1.01
	6.0	7.80	17.3	3.70	17.7	3.79	0.98
	6.5	8.45	18.5	3.96	19.2	4.11	0.96
	7.0	9.10	19.4	4.15	20.7	4.43	0.94
	7.5	9.75	20.5	4.39	22.2	4.75	0.92
	8.0	10.40	21.5	4.60	23.7	5.07	0.91
	8.5	11.05	22.9	4.90	25.1	5.37	0.91
	9.0	11.70	24.1	5.16	26.5	5.67	0.91
	10.0	13.00	26.4	5.65	29.5	6.31	0.89
	10.5	13.65	27.6	5.91	31.0	6.63	0.89

Figure 5.3: Young - Comparison Experimental and Predicted Load versus Deflection Values - 3/16" Specimen

**Table 5.4: Comparison Experimental and Predicted Load versus Deflection
Values for Young's 1/4" Specimen**

	Total Load (kN)	Load/ W_p	Young Defl'n Δ_y (mm)	Δ_y / t	Model Defl'n Δ_m (mm)	Δ_m / t	Defl'n Ratio Test : Model
Specimen # 2 $t = 6.40 \text{ mm}$ $W_p = 1.83$ kN	0	0.00	0.00	0.00	0.0	0.00	-
	1	0.55	1.9	0.30	6.7	1.05	0.28
	2	1.09	7.0	1.09	8.5	1.33	0.82
	3	1.64	9.6	1.50	9.6	1.50	1.00
	4	2.19	11.3	1.77	10.6	1.66	1.07
	5	2.73	12.8	2.00	11.5	1.80	1.11
	6	3.28	13.8	2.16	12.1	1.89	1.14
	7	3.83	14.6	2.28	12.8	2.00	1.14
	8	4.37	16.0	2.50	13.6	2.13	1.18
	9	4.92	17.3	2.70	15.3	2.39	1.13
	10	5.46	18.5	2.89	17.0	2.66	1.09
	11	6.01	19.0	2.97	18.7	2.92	1.02
	12	6.56	20.2	3.16	20.4	3.19	0.99
	13	7.10	21.6	3.38	22.1	3.45	0.98
	14	7.65	22.6	3.53	23.8	3.72	0.95
	15	8.20	23.7	3.70	25.4	3.97	0.93
	16	8.74	25.0	3.91	27.1	4.23	0.92
	17	9.29	26.2	4.09	28.8	4.50	0.91
	18	9.84	27.5	4.30	30.5	4.77	0.90

Figure 5.4: Young - Comparison Experimental and Predicted Load versus Deflection Values - 1/4" Specimen

**Table 5.5: Comparison Experimental and Predicted Load versus Deflection
Values for Young's 1/2" Specimen**

	Total Load (kN)	Load/ W_p	Young Defl'n Δ_y (mm)	Δ_y / t	Model Defl'n Δ_m (mm)	Δ_m / t	Defl'n Ratio Test : Model
Specimen # 3 $t = 12.8\text{mm}$ $W_p = 5.614$ kN	0	0.00	0.00	0.00	0.0	0.00	-
	2	0.36	1.6	0.13	7.4	0.58	0.22
	4	0.71	2.6	0.20	9.0	0.70	0.29
	6	1.07	5.3	0.41	10.4	0.81	0.51
	8	1.43	8.3	0.65	11.4	0.89	0.73
	10	1.78	11.8	0.92	12.3	0.96	0.96
	12	2.14	14.8	1.16	13.2	1.03	1.12
	14	2.49	16.4	1.28	15.4	1.20	1.06
	16	2.85	18.4	1.44	17.6	1.38	1.05
	18	3.21	20.7	1.62	19.8	1.55	1.05
	20	3.56	22.0	1.72	22.1	1.73	1.00
	22	3.92	24.0	1.88	24.3	1.90	0.99
	24	4.28	25.2	1.97	26.4	2.06	0.95
	26	4.63	27.2	2.13	28.6	2.23	0.95
	28	4.99	28.2	2.20	30.8	2.41	0.92
	30	5.34	30.2	2.36	33.0	2.58	0.92
	32	5.70	31.9	2.49	35.1	2.74	0.91
	34	6.06	33.2	2.59	37.3	2.91	0.89
	36	6.41	35.5	2.77	39.4	3.08	0.90
	38	6.77	36.7	2.87	41.4	3.23	0.89
	40	7.13	38.4	3.00	43.2	3.38	0.89

2 of/de 2

PM-1 3½"x4" PHOTOGRAPHIC MICROCOPY TARGET
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Figure 5.5: Young - Comparison Experimental and Predicted Load versus Deflection Values - 1/2" Specimen

Table 5.6: Comparison Full Parent Plate Model versus Combined Full Parent Plate and HAZ Model for Young's 1/2" Specimen

	Total Load (kN)	Load/ W_p	Full Parent Plate Defl'n Δ_{fpp} (mm)	Δ_{fpp} / t	Parent Plate + HAZ Defl'n $\Delta_{fpp+HAZ}$ (mm)	$(\Delta_{fpp+HAZ}) / t$
Specimen # 3 $t = 12.8\text{mm}$ $W_p = 5.614\text{ kN}$	0	0.00	0.00	0.00	0.0	0.00
	2	0.36	7.4	0.58	7.4	0.58
	4	0.71	9.0	0.70	9.3	0.73
	6	1.07	10.4	0.81	10.5	0.82
	8	1.43	11.4	0.89	11.5	0.90
	10	1.78	12.3	0.96	12.5	0.98
	15	2.67	16.5	1.29	16.5	1.29
	20	3.56	22.1	1.73	22.1	1.73
	25	4.45	27.5	2.15	27.5	2.15
	30	5.34	33.0	2.58	33.0	2.58
	35	6.23	38.3	2.99	38.3	2.99
	40	7.13	43.2	3.38	43.0	3.36
	43	7.66	46.0	3.59	45.7 *	3.57 *
	45	8.02	47.7	3.73		
	50	8.91	52.1	4.07		
	55	9.80	56.2	4.39		
	60	10.69	60.2	4.70		
	65	11.58	64.1	5.01		
	70	12.47	67.8	5.30		
	75	13.36	71.3	5.57		
	80	14.25	74.8	5.84		
	85	15.14	78.1	6.10		
	90	16.03	82.3	6.43		
	95	16.92	86.6	6.77		
	100	17.81	90.8	7.09		* Plate Rupture (Ultimate Strain Exceeded)
	105	18.70	95.0	7.42		
	111	19.77	100.2 *	7.83 *		

Figures 5.3, 5.4 and 5.5 clearly illustrate that the experimental and predicted load versus deflection behaviour are in good agreement in the region of inelastic membrane behaviour. All three experimental result curves follow the same general response of a straight line steep slope elastic region followed by a straight line portion at a lesser slope than the final portion of the elastic region response. The predicted response through the elastic region is a curve constantly increasing in slope as the membrane resistance builds and begins to dominate the response. Variations in stiffness (slope) or magnitude can be attributed to estimates in material properties and boundary stiffness and the influence of the heat affected zone. The influence of these items on inelastic membrane behaviour will be explored in Section 6.3 Parametric Studies.

The final load value applied by Young is small when compared to the predicted rupture load of the specimens (Load/W_p approximately equal to 19.8 for the 1/2" plate with parent plate material only modelled). Thus, the portion of the total response behaviour investigated by Young was small. The nature of the IMASP program is to have improved accuracy at higher loads because the flexural and flexural membrane portions of the response (which IMASP ignores) become less important.

The IMASP prediction incorporating a HAZ is virtually identical to the full parent plate analysis up to the point of rupture for the HAZ included analysis. The plate with the HAZ material is predicted to fail at a considerable lower load (approximately 60% lower) than the full parent plate material specimen. The lower ultimate strain capability of the HAZ material restricts the response. Young did not test his plates to failure and thus the validity of the HAZ model cannot be evaluated.

5.3 McDermott et al (1974)

McDermott et al (1974) conducted several tests on stiffened plate combinations and unstiffened plates under different point load conditions. Only the results for the single unstiffened plate loaded at midspan are used for comparison to the IMASP model. The geometric properties are listed in Table 5.7.

Table 5.7: McDermott - Unstiffened Plate Specimen Dimensions

Length (mm)	Width (mm)	Thickness (mm)
762	152.4	5.03

The material properties (stress versus strain curve) as listed in Table 5.8 were generated by IMASP from McDermott's reported values for yield strength and ultimate tensile strength of 302 MPA and 463.3 MPA respectively.

Table 5.8: McDermott - Material Properties

Stress (MPa)	Strain (mm/mm)
0.0	0.000000
302.0	0.001466
303.0	0.010000
417.0	0.050000
440.1	0.100000
458.7	0.150000
463.3	0.175000

Figure 5.6 illustrates McDermott's test set-up. The in-plane horizontal restraint was composed of an external reaction frame made of a hollow structural section at each end (acting in flexure) separated by compression struts made of W25x6's. WT4x8.5's tees and a six inch wide 1/4" thick plate connect the test specimen to the reaction frame.

**Figure 5.6: McDermott's Experimental Test Set-Up
(After McDermott et al (1974))**

Figure 5.7 illustrates the structural idealization assumed for the IMASP model. The plate length is taken as the distance between the two solid round supports. The reaction frame, hollow structural sections and the plate and tee frame to specimen connectors were modelled in series and the corresponding elastic stiffness (flexible in plane boundary restraint) was incorporated in the analysis. The resultant stiffness of the boundaries are listed on the figure.

Figure 5.7: McDermott - Computer Model Idealization

Table 5.9 presents McDermott's experimental central deflection results and the IMASP predicted central deflection at various levels of load. Figure 5.8 illustrates the data of Table 5.9. In the IMASP solution, failure was predicted due to the near simultaneous exceedance of the material ultimate strain and the Von Mises criterion being reached.

Table 5.9: Comparison Experimental and Predicted Load versus Deflection Values for McDermott Test #1

	Load (kN)	Load/ W_p	McDermott Defl'n Δ_{McD} (mm)	Δ_{McD} / t	Model Defl'n Δ_m (mm)	Δ_m / t	Defl'n Ratio Test : Model
McDermott Test # 1 $t = 5.03 \text{ mm}$ $W_p = 2.428$ kN	0	0.00	0.00	0.00	0.0	0.00	-
	10	4.12	24	4.77	20.8	4.14	1.15
	20	8.24	34	6.76	26.2	5.21	1.30
	30	12.36	40	7.95	29.9	5.94	1.34
	40	16.47	46	9.15	33.0	6.56	1.39
	50	20.59	49	9.74	41.2	8.19	1.19
	60	24.71	56	11.13	49.5	9.84	1.13
	70	28.83	62	12.33	57.9	11.51	1.07
	80	32.95	69	13.72	65.5	13.02	1.05
	90	37.07	75	14.91	72.6	14.43	1.03
	100	41.19	81	16.10	79.4	15.79	1.02
	120	49.42	99	19.68	92.4	18.37	1.07
	140	57.66	108	21.47	104.5	20.78	1.03
	160	65.90	124	24.65	115.8	23.02	1.07
	180	74.14	134	26.64	126.5	25.15	1.06
	200	82.37	145	28.83	137.1	27.26	1.06
	220	90.61	153	30.42	147.2	29.26	1.04
	240	98.85	165	32.80	156.8	31.17	1.05
	260	107.08	175	34.79	166.1	33.02	1.05
	280	115.32	180	35.79	174.9	34.77	1.03
	300	123.56	188	37.38	186.5	37.08	1.01
	315	129.74	197	39.17	195.9	38.95	1.01
	320	131.80	200	39.76	Rupture		
	340	140.03	209	41.55			
	360	148.27	218	43.34			
	380	156.51	232	46.12			
	400	164.74	238	47.32			
	418	172.16	Rupture

Figure 5.8: Comparison Experimental and IMASP Predicted Load versus Deflection Values - McDermott's Test #1

The shape of the experimental and predicted curves shown in Figure 5.8 are in excellent agreement. The experimental and predicted curves change from elastic behaviour to elasto-plastic behaviour, a marked decrease in slope, at points A (Load = 55 kN) and E (Load = 70 kN) respectively of Figure 5.8. The beginning of the fully plastic region response behaviour is marked by the increase in slope at a load of approximately 100 kN (Point B) for the predicted response and at a load of 160 kN (Point F) for the

experimental response. Strain hardening occurs at Points C and G and rupture occurs at points D and H on Figure 5.8. The variation in deflection (ie. increased deflections for the test) over the elastic membrane response region are indicative of more flexible boundary conditions. This effect will be shown in Section 6.3 Parametric Studies. In McDermott's test set-up a possible source of flexibility, not accounted for in the IMASP prediction model, could be bolt slip at the connection of the tee to the hollow structural section reaction frame. The remaining portions of the curves exhibit similar characteristics. Other variations may be attributed to differences in the actual material characteristics and those calculated in IMASP.

Rupture of the plate appears to be prematurely predicted by the IMASP program. This does not necessarily indicate a flaw in the IMASP failure criteria. There are three possible sources causing this difference. Firstly, McDermott's plate edge (vertical support point) can rotate over the top of the solid round supports causing a rotational flexibility which is unaccounted for in the IMASP model. Therefore, the upper edge of the plate at the support with the allowed rotation in the experiment will have a larger curvature and therefore smaller bending strains than predicted. Thus, the maximum material strain will be reached at a higher deflection and magnitude of load in the experiment than in the predicted response. Secondly, the ultimate strain value for McDermott's material (unknown) may have been higher than that assumed for IMASP input leading to a larger load carrying capability and larger deflection at rupture. The final source of difference could be the fact that IMASP was benchmarked to four point loads and not a single central point load.

5.4 Ratzlaff and Kennedy (1986)

Ratzlaff and Kennedy conducted two identical tests on plates 152mm wide, 456mm length and 1.47mm thick. The plates were loaded under hydrostatic pressure, monotonically to failure. The test set-up, detailed in Figure 5.9, consists of a thin steel test plate clamped at the edges between two one inch thick plates by high strength bolts. A small gap and epoxy sealing strip was left at the edge as shown. Figure 5.10 illustrates the structural idealization used in IMASP. The boundaries were modelled as being infinitely stiff.

The material properties used in the IMASP model are listed in Table 5.10. These consist of the material yield stress, 270 MPa, the Modulus of Elasticity, 197100 MPa, the ultimate tensile stress, 355 MPa and the strain hardening strain, 0.017 as reported by Ratzlaff and Kennedy and values interpolated from the given stress strain curve. The ultimate strain was taken as 0.175 as per Section 4.5.

Table 5.10: Ratzlaff and Kennedy - Material Properties

Stress (MPa)	Strain (mm/mm)
0.0	0.000000
270.0	0.001370
271.0	0.017000
309.5	0.025000
327.4	0.050000
348.2	0.100000
351.8	0.150000
355.0	0.175000

Figure 5.9: Ratzlaff and Kennedy's Experimental Test Set-Up
(After Ratzlaff and Kennedy (1986))

Figure 5.10: Ratzlaff and Kennedy - IMASP Model Idealization

Table 5.11 presents Ratzlaff and Kennedy's experimental central deflection results and the IMASP predicted central deflection at increasing levels of hydrostatic pressure. Figure 5.11 illustrates the data of Table 5.11. Ratzlaff and Kennedy loaded the plate until it ruptured causing hydraulic fluid to leak.

**Table 5.11: Comparison Experimental and Predicted Load versus Deflection
Values for Ratzlaff and Kennedy's Flat Plate Test**

	Total Load (kPa)	Load/ W_p	Ratzlaff & Kennedy Defl'n Δ_{RK} (mm)	Δ_{RK} / t	Model Defl'n Δ_m (mm)	Δ_m / t	Defl'n Ratio Test : Model
Average of Two Tests $t = 1.47$ mm $W_p = 116.7$ kPa	0	0.00	0.0	0.00	0.00	0.00	-
	50	0.43	0.4	0.24	1.74	1.18	0.21
	100	0.86	0.6	0.42	2.10	1.43	0.29
	200	1.71	1.3	0.85	2.66	1.81	0.47
	300	2.57	2.2	1.47	3.04	2.07	0.71
	400	3.43	2.7	1.84	3.34	2.27	0.81
	500	4.28	3.2	2.20	3.71	2.52	0.87
	600	5.14	3.5	2.36	4.44	3.02	0.78
	700	6.00	4.0	2.73	5.19	3.53	0.77
	800	6.86	4.9	3.31	5.92	4.03	0.82
	900	7.71	5.6	3.84	6.65	4.52	0.85
	1000	8.57	6.4	4.35	7.38	5.02	0.87
	1200	10.28	7.8	5.33	8.83	6.01	0.89
	1400	12.00	9.3	6.32	10.26	6.98	0.91
	1600	13.71	10.2	6.94	11.68	7.95	0.87
	1800	15.42	11.5	7.79	12.91	8.78	0.89
	2000	17.14	12.6	8.55	14.09	9.59	0.89
	2200	18.85	13.6	9.22	15.21	10.35	0.89
	2400	20.57	14.6	9.96	16.28	11.07	0.90
	2600	22.28	15.9	10.79	17.32	11.78	0.92
	2800	23.99	17.0	11.56	17.83	12.13	0.95
	3000	25.71	18.0	12.22	19.29	13.12	0.93
	3200	27.42	19.2	13.07	Rupture		
	3400	29.13	20.7	14.08			
	3600	30.85	21.6	14.70			
	3670	31.45	21.7	14.76			
			Rupture

Figure 5.11: Comparison Experimental and Predicted Load versus Deflection Values - Ratzlaff and Kennedy

Figure 5.11 reveals excellent agreement between the experimental and predicted response shape. The experimental and predicted response curves reveal the change from elastic behaviour to elasto-plastic behaviour through a distinct reduction in the slope of the load versus deflection curve at points A (Load = 500 kPa) and E (Load = 675 kPa) of the experimental and predicted response curves respectively. Similarly, an increase in the slope of the curves at points B (Load = 1500 kPa) and F (Load = 1500 kPa) on the

experimental and predicted response curves respectively signal a change to full plastic behaviour. Points C (Load = 2700 kPa) and G (Load = 3200 kPa) indicate the onset of strain hardening in the plate. Rupture occurs at points D (Load = 3000 kPa) and H (Load = 3670 kPa) on the experimental and predicted curves respectively. IMASP predicted failure due to the exceedance of the material ultimate strain value. The predicted curve is to the right of the experimental results curve. This is indicative of a boundary flexibility. A possible source of this flexibility is slip at the high strength bolt connection.

IMASP predicts a lower rupture load than was experimentally determined. This difference could be due to the epoxy seal in the experimental test set-up. This seal would allow some rotation at the edge so that the radius of curvature would be increased and therefore the bending strain and thus the total strain at the support (critical section) would be lowered. This bending strain reduction would allow larger load application with resulting increased deflections prior to the critical (ultimate) strain value and rupture prediction being reached.

5.5 Ratzlaff (1995)

Ratzlaff conducted tests on four steel plates. Table 5.12 lists the specific geometry of the four plates. A small gap was left between the plates. The loaded plate was edge prepped and the gap filled with weld metal as shown in Figure 5.12. This test set up resulted in the welded side of the plate being loaded which is opposite to the situation that exists in typical stiffened plate structures. Each plate was welded around its perimeter to a 150 mm thick steel backing plate. The welding process caused the plates to curve away from the backing plate, resulting in the initial plate central deflections as indicated in Table 5.12. The test plates were loaded with hydrostatic pressure, monotonically, by pumping fluid between the test and backing plates.

Table 5.12: Ratzlaff - Test Specimen Dimensions

Specimen	Panel Breadth (mm) **	Thickness (mm)	Initial Deformation (mm)
PLT6 Test 1	1043	6.80	8.0
PLT6 Test 2	1043	7.10	2.5
PLT12	1028	13.40	5.2
PLT16	1016	16.50	6.8

** Short axis dimension

Ratzlaff conducted tests to determine the tensile properties of the plates. Specifically, the yield stress and strain, the strain at the onset of strain hardening, and the ultimate stress and strain values were determined. The parent plate material properties (stress versus strain curve) used in the IMASP model are shown in Table 5.13. The values that were determined by Ratzlaff are marked with asterisks, the remainder of the values were assumed as per Section 4.5.

Table 5.13: Ratzlaff - Parent Plate Material Properties

Test PLT6 #1 6.8 mm Thick Plate		Test PLT6 #2 7.1 mm Thick Plate		Test PLT12 12 mm Thick Plate		Test PLT16 16 mm Thick Plate	
E = 204600 MPa		E = 208600 MPa		E = 207078 MPa		E = 206309 MPa	
σ	ϵ	σ	ϵ	σ	ϵ	σ	ϵ
000.0	0.000000	000.0	0.000000	000.0	0.000000	000.0	0.000000
294.4 *	0.001440*	292.5*	0.001400*	318.9*	0.001540*	307.9*	0.001490*
295.4	0.014700*	293.5	0.023000*	319.9	0.017500*	308.4	0.016000*
429.5	0.050000	407.1	0.050000	469.6	0.050000	461.1	0.050000
453.3	0.100000	429.7	0.100000	495.7	0.100000	486.7	0.100000
472.4	0.150000	447.8	0.150000	516.6	0.150000	507.2	0.150000
477.2 *	0.191000*	453.4*	0.215000*	521.8*	0.190000*	512.3	0.184000*

Figure 5.13 illustrates the IMASP engineering idealization of Ratzlaff's test set-up. The boundaries of the plate were assumed rigid. A heat affected zone (HAZ) portion of plate was assumed at the edge. The IMASP HAZ calculated properties were used in the analysis. The HAZ material properties (stress versus strain curve) used are shown in Table 5.14.

Table 5.14: IMASP Generated - Ratzlaff - Material Heat Affected Zone Properties

Test PLT6 #1		Test PLT6 #2		Test PLT12		Test PLT16	
σ	ϵ	σ	ϵ	σ	ϵ	σ	ϵ
0.00	0.000000	0.00	0.000000	0.00	0.000000	0.00	0.000000
267.98	0.001000	264.19	0.001000	297.15	0.001000	282.93	0.001000
308.54	0.002000	304.69	0.002000	337.67	0.002000	323.50	0.002000
352.15	0.004000	348.20	0.004000	381.22	0.004000	367.15	0.004000
373.27	0.006000	369.20	0.006000	402.26	0.006000	388.30	0.006000
386.39	0.008000	382.20	0.008000	415.31	0.008000	401.45	0.008000
398.00	0.010000	393.70	0.010000	426.85	0.010000	413.10	0.010000
430.30	0.015000	425.71	0.015000	458.96	0.015000	445.48	0.015000
455.09	0.020000	450.22	0.020000	483.58	0.020000	470.35	0.020000
470.38	0.025000	465.22	0.025000	498.69	0.025000	485.73	0.025000
482.18	0.030000	476.73	0.030000	510.30	0.030000	497.60	0.030000
489.97	0.035000	484.24	0.035000	517.91	0.035000	505.48	0.035000
497.76	0.040000	491.75	0.040000	525.53	0.040000	513.35	0.040000
504.55	0.045000	498.25	0.045000	532.75	0.045000	520.23	0.045000
511.35	0.050000	504.76	0.050000	538.75	0.050000	527.10	0.050000
519.43	0.060000	512.27	0.060000	546.47	0.060000	535.35	0.060000
522.02	0.070000	514.29	0.070000	548.70	0.070000	538.10	0.070000

Tables 5.15 through 5.18 and Figures 5.14 through 5.17 present Ratzlaff's experimental central deflection results and the IMASP predicted central deflection at various levels of load.

Figure 5.12: Ratzlaff's Experimental Test Set-Up

Figure 5.13: Ratzlaff - IMASP Model Idealization

Table 5.15: Comparison Experimental and Predicted Load versus Deflection
Values for Ratzlaff's 6 mm (Test #1) Steel Plate Test

	Total Load (kN)	Load/ W_p	Ratzlaff Defl'n Δ_R (mm)	Δ_R / t	Model Defl'n Δ_m (mm)	Δ_m / t	Defl'n Ratio Test : Model
Test PLT6 Test #1 $t = 6.80 \text{ mm}$ $q_p = 57.86 \text{ kPa}$	0	0.00	0.0	0.00	0	0.00	-
	50	0.86	8.0	1.18	12.3	1.81	0.65
	100	1.73	12.1	1.78	15.8	2.32	0.77
	150	2.59	15.2	2.24	17.9	2.63	0.85
	200	3.46	17.6	2.59	19.6	2.88	0.90
	250	4.32	19.9	2.93	21.3	3.13	0.93
	300	5.18	21.8	3.21	22.4	3.29	0.97
	350	6.05	24.0	3.53	23.6	3.47	1.02
	400	6.91	26.0	3.82	24.9	3.66	1.04
	450	7.78	28.4	4.18	27.9	4.10	1.02
	500	8.64	30.6	4.50	31.0	4.56	0.99
	550	9.51	33.7	4.96	34.1	5.01	0.99
	600	10.37	36.4	5.35	37.1	5.46	0.98
	650	11.23	39.7	5.84	40.2	5.91	0.99
	700	12.10	43.2	6.35	43.3	6.37	1.00
	750	12.96	47.0	6.91	46.3	6.81	1.02
	800	13.83	49.7	7.31	49.3	7.25	1.01
	850	14.69	53.3	7.84	52.3	7.69	1.02
	900	15.55	57.0	8.38	55.4	8.15	1.03
	950	16.42	59.8	8.79	58.3	8.57	1.03
	1000	17.28	63.2	9.29	61.3	9.01	1.03
	1050	18.15	66.1	9.72	64.3	9.46	1.03
	1100	19.01	68.6	10.09	67.3	9.90	1.02
	1150	19.88	71.3	10.49	70.2	10.32	1.02
	1151	19.89	71.4	10.50			
			Rupture				
	1200	20.74			73.2	10.76	
	1206	20.84			73.5	10.81	
					Rupture

Figure 5.14: Comparison Experimental and Predicted Load versus Deflection Values - Ratzlaff PLT6 Test #1

Table 5.16: Comparison Experimental and Predicted Load versus Deflection
Values for Ratzlaff's 6 mm (Test #2) Steel Plate Test

	Total Load (kN)	Load/ q_p	Ratzlaff Defl'n Δ_R (mm)	Δ_R / t	Model Defl'n Δ_m (mm)	Δ_m / t	Defl'n Ratio Test: Model
Test PLT6	0	0.00	0.0	0.00	0.0	0.00	-
Test #2	50	0.80	9.5	1.34	12.3	1.73	0.77
	100	1.60	13.3	1.87	15.5	2.18	0.86
	150	2.40	17.1	2.41	17.8	2.51	0.96
$t = 7.1$ mm	200	3.19	19.4	2.73	19.6	2.76	0.99
	250	3.99	21.7	3.06	21.0	2.96	1.03
$q_p = 62.62$ kPa	300	4.79	24.0	3.38	22.1	3.11	1.09
	350	5.59	26.0	3.66	23.3	3.28	1.12
	400	6.39	27.9	3.93	24.6	3.46	1.13
	450	7.19	30.0	4.23	26.9	3.79	1.12
	500	7.98	32.4	4.56	29.9	4.21	1.08
	550	8.78	35.4	4.99	32.9	4.63	1.08
	600	9.58	38.2	5.38	35.9	5.06	1.06
	650	10.38	41.4	5.83	38.9	5.48	1.06
	700	11.18	44.3	6.24	41.8	5.89	1.06
	750	11.98	47.0	6.62	44.8	6.31	1.05
	800	12.78	49.6	6.99	47.8	6.73	1.04
	850	13.57	52.3	7.37	50.7	7.14	1.03
	900	14.37	56.0	7.89	53.7	7.56	1.04
	950	15.17	59.0	8.31	56.7	7.99	1.04
	1000	15.97	61.7	8.69	59.6	8.39	1.04
	1050	16.77	64.5	9.08	62.6	8.82	1.03
	1100	17.57	67.9	9.55	65.5	9.23	1.04
	1150	18.36	70.5	9.93	68.4	9.63	1.03
	1200	19.16	73.7	10.38	71.4	10.06	1.03
	1238	19.77			73.6	10.37	
					Rupture		
	1250	19.96	77.0	10.94			
	1300	20.76	79.5	11.20			
	1350	21.56	82.2	11.58			
	1424	22.74	85.1	11.99			
			Rupture

Figure 5.15: Comparison Experimental and Predicted Load versus Deflection Values - Ratzlaff PLT6 Test #2

Table 5.17: Comparison Experimental and Predicted Load versus Deflection Values for Ratzlaff's 12 mm Steel Plate Test

	Total Load (kN)	Load/ q_p	Ratzlaff Defl'n Δ_R (mm)	Δ_R / t	Model Defl'n Δ_m (mm)	Δ_m / t	Defl'n Ratio Test : Model
Test PLT12 $t = 13.4 \text{ mm}$ $q_p = 250.33 \text{ kPa}$	0	0.00	0.00	0.00	0	0.00	-
	50	0.20	2.5	0.19	9.9	0.74	0.25
	100	0.40	4.6	0.34	12.1	0.90	0.38
	150	0.60	6.5	0.49	14.2	1.06	0.46
	200	0.80	8.0	0.60	15.6	1.16	0.51
	250	1.00	9.4	0.70	16.6	1.24	0.57
	300	1.20	11.0	0.82	17.6	1.31	0.63
	350	1.40	12.7	0.95	18.6	1.39	0.68
	400	1.60	14.5	1.08	19.4	1.45	0.75
	500	2.00	17.9	1.34	20.8	1.55	0.86
	600	2.40	21.1	1.57	21.9	1.63	0.96
	700	2.80	24.1	1.80	23.1	1.72	1.04
	800	3.20	26.6	1.99	24.4	1.82	1.09
	900	3.60	29.2	2.18	25.4	1.90	1.15
	1000	3.99	31.4	2.34	28.3	2.11	1.11
	1100	4.39	33.8	2.52	31.1	2.32	1.09
	1200	4.79	35.8	2.67	33.9	2.53	1.06
	1300	5.19	38.1	2.84	36.7	2.74	1.04
	1400	5.59	40.4	3.01	39.4	2.94	1.03
	1500	5.99	42.7	3.19	42.2	3.15	1.01
	1600	6.39	44.9	3.35	45.0	3.36	1.00
	1700	6.79	47.2	3.52	47.8	3.57	0.99
	1800	7.19	49.6	3.70	50.5	3.77	0.98
	1900	7.59	52.0	3.88	53.3	3.98	0.98
	1909	7.63	53.6	4.00		0.00	
			Rupture				
	2000	7.99					
	2200	8.79			61.5	4.59	
	2400	9.59			67.0	5.00	
	2600	10.39			72.4	5.40	
	2700	10.79			75.1	5.60	
	2727	10.89			75.8	5.66	
					Rupture

Figure 5.16: Comparison Experimental and Predicted Load versus Deflection Values - Ratzlaff PLT12

Table 5.18: Comparison Experimental and Predicted Load versus Deflection Values for Ratzlaff's 16 mm Steel Plate Test

	Total Load (kN)	Load/ q_p	Ratzlaff Defl'n Δ_R (mm)	Δ_R / t	Model Defl'n Δ_m (mm)	Δ_m / t	Defl'n Ratio Test : Model
Test PLT16 $t = 16.5 \text{ mm}$ $q_p = 374.56 \text{ kPa}$	0	0.00	0.0	0.00	0	0.00	-
	50	0.13	1.2	0.07	9.1	0.55	0.13
	100	0.27	2.3	0.14	11.4	0.69	0.20
	200	0.53	4.6	0.28	14.2	0.86	0.32
	300	0.80	6.3	0.38	16.0	0.97	0.39
	400	1.07	8.2	0.50	17.6	1.07	0.47
	500	1.33	10.2	0.62	19.1	1.16	0.53
	600	1.60	12.9	0.79	20.3	1.23	0.64
	700	1.87	15.7	0.95	21.4	1.30	0.73
	800	2.14	18.3	1.11	22.3	1.35	0.82
	900	2.40	21.0	1.27	23.3	1.41	0.90
	1000	2.67	23.1	1.42	24.0	1.45	0.98
	1100	2.94	25.5	1.55	25.6	1.55	1.00
	1200	3.20	27.6	1.67	27.9	1.69	0.99
	1300	3.47	29.7	1.80	30.2	1.83	0.98
	1400	3.74	31.2	1.93	32.5	1.97	0.98
	1500	4.00	33.1	2.01	34.8	2.11	0.95
	1600	4.27	34.6	2.10	37.1	2.25	0.93
	1700	4.54	36.2	2.19	39.4	2.39	0.92
	1800	4.81	38.2	2.32	41.7	2.53	0.92
	1900	5.07	40.5	2.45	44.0	2.67	0.92
	2000	5.34	41.8	2.53	46.2	2.80	0.90
	2100	5.61	43.6	2.64	48.5	2.94	0.90
	2200	5.87	45.1	2.74	50.8	3.08	0.89
	2300	6.14	47.0	2.85	53.1	3.22	0.89
	2338	6.24	47.6 Rupture	2.88			
	2400	6.41			55.3	3.35	
	2600	6.94			59.8	3.62	
	2800	7.48			64.3	3.90	
	3000	8.01			68.7	4.16	
	3200	8.54			73.1	4.43	
	3390	9.05			76.5 Rupture	4.64

Figure 5.17: Comparison Experimental and Predicted Load versus Deflection Values - Ratzlaff PLT16 Test

Figures 5.14 through 5.17 reveal good general agreement between the experimental and predicted load versus deflection values. Figures 5.14 and 5.15 are in excellent agreement. The accuracy of the prediction appears to be decrease as Ratzlaff's plates increased in thickness. In each of the experimental curves there is a noticeable absence of the elastic response section. This is most pronounced in the 12mm and 16mm

tests. In all of the experimental curves there is no distinct change in the slope of the curve where the elastic region would be exhausted. This absence is most likely due to residual, near yield, stresses caused by the heat input of the welding process (revealed by the presence of initial plate deflections). If the elastic response portion of the curve is removed (IMASP prediction curve shifted down by moving point A in Figure 5.17 to the origin to create Figure 5.18) in the Ratzlaff's 16mm test better (excellent) agreement between the curves is realized. In Figure 5.18 rupture of the plate is shown to occur at nearly the same load levels.

Figure 5.18: Comparison of Experimental Load versus Deflection Curve and IMASP Predicted Load versus Deflection with Elastic Portion Removed - Ratzlaff 16mm Plate

5.6 Kennedy and Cheng (1987)

Kennedy and Cheng conducted a series of tests, one of which was an all steel specimen and the other five were steel plates held a fixed distance apart by concrete infill. The specimens were loaded with four nearly equally spaced point loads of the same magnitude. In the case of the all steel test the plates merely laid on top of each other with no connection. For the other composite specimens the only connection between the steel and concrete was that provided by the bonding between the steel and concrete surfaces. The dimensions of the test specimens are listed in Table 5.19.

Table 5.19: Kennedy and Cheng - Test Specimen Dimensions

Test	Length (mm)	Width (mm)	Top Plate Thickness (mm)	Bottom Plate Thickness (mm)	Concrete Thickness (mm)
All Steel	2850	500.3	6.96	6.90	0
C121T6	2850	500.6	6.86	6.82	109
C152T6	2850	500.0	6.70	6.83	140
C152T4	2850	500.7	4.87	4.87	143
C152T3	2850	500.6	3.13	3.15	146
C203T6	2850	500.5	6.77	6.91	191

All of the steels used had yield strengths near 350 MPa. Kennedy provided a polynomial to predict the stress strain curve for the steels used in each test. The material properties (stress versus strain curve) assumed in the IMASP model of Kennedy and Cheng's All Steel test are listed in Table 5.20.

Table 5.20: Kennedy and Cheng - Steel Material Properties

Stress (MPa)	Strain (mm/mm)
347.90	0.001725
347.91	0.004000
347.92	0.008000
348.25	0.028000
374.42	0.035000
405.60	0.045000
460.66	0.110000
472.73	0.150000
474.30	0.177000

Kennedy's test set up is illustrated in Figure 5.19. Figures 5.20 illustrates the structural idealization of the test that is assumed in the IMASP model.

Figure 5.19: Kennedy and Cheng's Test Set-Up

Plate width = 500.3 mm

Bottom Plate thickness = 6.90 mm

Top plate thickness = 6.84 mm

Figure 5.20: Kennedy and Cheng - IMASP Model Idealization

Kennedy and Cheng's experimental results (deflection for a given total load) and the IMASP deflection prediction for the same load level for the researchers' All Steel test are presented in Table 5.21.

Table 5.21: Comparison Experimental and Predicted Load versus Deflection Values for Kennedy's All Steel Test

	Total Load (kN)	Load/ W_p	Kennedy Defl'n Δ_K (mm)	Δ_K / t	Model Defl'n Δ_m (mm)	Δ_m / t	Defl'n Ratio Test: Model
All Steel Test $t = 13.86$ mm $W_p = 19.29$ kN	0	0.00	0	0.00	0	0.00	-
	25	1.30	30	2.16	40	2.89	0.75
	50	2.59	40	2.89	49	3.54	0.82
	100	5.18	55	3.97	61	4.40	0.90
	200	10.37	80	5.77	78	5.63	1.03
	300	15.55	98	7.07	89	6.42	1.10
	400	20.74	109	7.86	98	7.07	1.11
	500	25.92	117	8.44	105	7.58	1.11
	600	31.10	126	9.09	112	8.08	1.13
	700	36.29	135	9.74	120	8.66	1.13
	800	41.47	145	10.46	137	9.88	1.06
	900	46.66	162	11.69	155	11.18	1.05
	1000	51.84	182	13.13	173	12.48	1.05
	1200	62.21	219	15.80	206	14.86	1.06
	1400	72.58	255	18.40	239	17.24	1.07
	1600	82.94	295	21.28	274	19.77	1.08
	1800	93.31	319	23.02	304	21.93	1.05
	2000	103.68	347	25.04	332	23.95	1.05
	2200	114.05	376	27.13	353	25.47	1.07
	2400	124.42	397	28.64	377	27.20	1.05
	2600	134.78	426	30.74	402	29.00	1.06
	2800	145.15	442	31.89	416	30.01	1.06
	3000	155.52	469	33.84	442	31.89	1.06
	3118 *	161.64	485	34.99	457	32.97	1.06
	Rupture						

The results listed in Table 5.21 are presented graphically in Figure 5.21.

Figure 5.21: Comparison Experimental and Predicted Load versus Deflection Values - Kennedy and Cheng - All Steel Test

The shape of the experimental and predicted curves shown in Figure 5.21 are in excellent agreement. The predicted and experimental curves change from elastic behaviour to elasto-plastic behaviour, a marked decrease in slope, at points A (Load = 650 kN) and D (Load = 800 kN) respectively of Figure 5.21. Strain hardening occurs at Points B (Load = 1900 kN) and E (Load = 2100 kN) and rupture occurs at points C and F (both at a Load = 3118 kN) for the experimental and predicted response curves

respectively. The variation in deflection (ie. increased deflections for the experimental response as compared to the predicted response) over the elastic membrane response region are indicative of more flexible boundary conditions in the test than was modelled. The reason for the discrepancy between the predicted and experimental behaviour is not known. This effect will be shown in Section 6.3 Parametric Studies. The remaining portions of the curves exhibit similar characteristics. Plate rupture is predicted by IMASP (Von Mises criteria at ultimate and ultimate strain) at the same load as occurred in the test. This is as expected given that the IMASP failure prediction was benchmarked to this experimental test.

5.7 Brown and Joubert (1990)

Brown and Joubert conducted a series of tests on five aluminum plates. Two of the plates tested were curved in a concave manner, two were curved convex and one was flat. All of the plates were approximately five millimetres thick. The plates were loaded hydrostatically by fluid pumped between two identical nine panel arrays for the flat plate test specimen. The flat plate test specimens had a length of 340 mm and a breadth of 460 mm.

The aluminum used had a yield strength of 216 MPa. Given that the test apparatus and test specimens were of welded construction the IMASP HAZ option was selected to determine the material properties and results. The aluminum parent plate material properties (stress versus strain curves) are listed, in Table 5.22. The parent plate value for strain at yield was calculated from an assumed Young's Modulus of 70000 MPa.

Table 5.22: Brown and Joubert - Aluminum Material Properties

Aluminum - Material Properties	
σ	ϵ
000.0	0.000000
216.0	0.001725
225.0	0.004000
238.0	0.008000
245.0	0.028000
260.0	0.035000
270.0	0.045000
400.0	0.110000

Brown and Joubert's flat plate test set up is illustrated in Figure 5.22. Figure 5.23 illustrates the structural idealization of the test that is assumed in the IMASP model.

**Figure 5.22: Brown and Joubert
- Test Apparatus**

Figure 5.23: Brown and Joubert - IMASP Model Idealization

Brown and Joubert's experimental results (deflection for a given total load) and the IMASP deflection prediction for the same load level are presented in Table 5.23. The results listed in Table 5.23 are illustrated in Figure 5.24.

Table 5.23: Comparison Experimental and Predicted Load versus Deflection Values for Brown and Joubert's Flat Plate Test

	Total Load (kPa)	Load/ W_p	Brown & Joubert Defl'n Δ_{BJ} (mm)	Δ_{BJ} / t	Model Defl'n Δ_m (mm)	Δ_m / t	Defl'n Ratio Test : Model
Flat Plate Test $t = 5.22$ mm $W_p = 215.8$ kPa	0	0.00	0.0	0.00	0	0.00	-
	25	0.12	1.3	0.26	4.95	0.99	0.26
	50	0.23	2.0	0.42	6.21	1.24	0.34
	75	0.35	2.9	0.58	7.11	1.42	0.41
	100	0.46	3.6	0.72	7.83	1.57	0.46
	125	0.58	4.2	0.84	8.36	1.67	0.50
	150	0.70	4.9	0.98	8.87	1.77	0.55
	175	0.81	5.5	1.10	9.40	1.88	0.59
	200	0.93	6.1	1.22	9.76	1.95	0.63
	225	1.04	6.6	1.34	10.17	2.03	0.66
	250	1.16	7.1	1.42	10.54	2.11	0.67
	275	1.27	7.7	1.54	10.90	2.18	0.71
	300	1.39	8.3	1.66	11.21	2.24	0.74
	325	1.51	8.6	1.72	11.49	2.30	0.75
	350	1.62	9.1	1.82	11.80	2.36	0.77
	375	1.74	9.7	1.94	12.03	2.41	0.81
	400	1.85	10.3	2.06	12.33	2.47	0.84
	425	1.97	10.8	2.16	12.53	2.51	0.86
	450	2.09	11.4	2.28	12.78	2.56	0.89
	475	2.20	12.6	2.42	13.03	2.61	0.93
	500	2.32	12.8	2.56	13.23	2.65	0.97
	520	2.41	13.2	2.64	13.42	2.68	0.98
			Rupture				
	600	2.78			14.06	2.81	
	700	3.24			14.79	2.96	
	800	3.71			15.48	3.10	
	900	4.17			16.11	3.22	
	1000	4.63			16.65	3.33	
	1250	5.79			17.95	3.59	
	1500	6.95			19.05	3.81	
	1850	8.57			21.04	4.21	
					Rupture		

In Figure 5.24 the predicted response is not illustrated up to the point of predicted

rupture. The predicted rupture load value is approximately two and a half times the load value at which plate rupture occurred in the experimental work.

Figure 5.24: Brown and Joubert - Comparison Experimental and Predicted Load versus Deflection Values

The curve, shown in Figure 5.24, for the experimental work shows a very small elastic response portion (origin to point A) on Figure 5.24.

There doesn't appear to be any correlation between the test and predicted results. It is suspected that the aluminum heat affected zone and residual stresses are the key sources of the differences. Insufficient information was reported in the paper to draw any further conclusions.

5.8 Summary

Throughout Section 5, load versus deflection results as predicted by the IMASP program have been compared to the experimental works of several researchers. Excellent agreement between the predicted and experimental results has been found. In general, for loads which result in load/deflection response in the inelastic range the IMASP predictions are within 5% of the evaluated experimental results. Table 5.24 below summarizes the features of the IMASP program that have been validated against experimental work. In this table the experimental work is briefly described, the average deflection ratio (experimental test to IMASP prediction) in the inelastic response region is stated and remarks with respect to the success of the validation are given.

Table 5.24: Summary of IMASP Response and Features Validity

Behaviour Response Prediction or IMASP Feature	Experimental Work for Comparison	Description	Remarks
Load Type			
• Point Load	McDermott et al	Single point load at midspan	Average deflection ratio = 1.056 (excellent)
• UDL	Kennedy & Cheng	4 equally spaced point loads	Average deflection ratio = 1.059 (excellent)
	Young	4 equally spaced point loads	
• Hydrostatic Pressure	Ratzlaff & Kennedy	hydrostatic pressure	Average deflection ratio = 0.875 (good)
	Ratzlaff	hydrostatic pressure	Average deflection ratio = 0.995 (excellent)

Table 5.24 cont'd: Summary of IMASP Response and Features Validity

Behaviour Response Prediction or IMASP Feature	Experimental Work for Comparison	Load Description	Remarks
Failure (Plate Rupture) Prediction	Kennedy and Cheng	All steel test loaded to rupture	IMASP benchmarked to this experimental work
	Ratzlaff	Four (4) tests modelled with HAZ at supports	Average ratio (for four tests) of test rupture load to predicted rupture load = 0.788 (good)
	Ratzlaff	16.5 mm Test with elastic response portion of prediction removed	Ratio of Test rupture load to Predicted rupture load = 0.978 (excellent)
Response with Multiple Material Properties	Ratzlaff	Four (4) tests modelled with HAZ at supports	Average ratio (for four tests) of test rupture load to predicted rupture load = 0.788 (good)
Elastic Plate Boundaries	McDermott et al	Single point load at midspan	Ratio of Test to Predicted Load Level @ end of elastic response = 1.27 (good) Ratio of Test to Predicted Deflection Level @ end of elastic response = 1.55
	Kennedy and Cheng	4 equally spaced point loads	Ratio of Test to Predicted Load Level @ end of elastic response = 1.23 (good) Ratio of Test to Predicted Deflection Level @ end of elastic response = 1.67

The experimental set up of several researchers (McDermott, Kennedy & Cheng, Brown & Joubert) had in-plane restraint systems that had some flexibility. This elastic flexibility has been identified and modelled. The effect of the flexibility is to extend the elastic membrane response portion of the overall behaviour to give increased deflection

for a given load level.

Features that have been incorporated in IMASP that have not been verified with experimental data are the response for "high" strain rate loads and the difference caused to the response by maintaining the load trajectory (fixed distance for the support). The effect of these features will be investigated in Section 6.3 Parametric Studies.

None of the physical tests, to which IMASP has been compared were for the case of a plate, restrained on all edges, loaded with a point load. To analyze this case with IMASP it is necessary to assume an effective width of plating. It is likely that a semi-empirical relationship exists based on the aspect ratio of the plate and the length of the load parallel to the long axis of the plate. The lack of experimental data prevents further development of this hypothesis.

6.0 APPLICATION IN MARINE VESSELS AND OFFSHORE STRUCTURES

6.1 General

Stiffened steel plate structures are used extensively in marine vessels and offshore structures. The ultimate load carrying capability (survivability) of these structures has become increasingly important. Large oil tankers have run aground (Exxon VALDEZ (1989), MEGA BORG (1990) and BRAER (1993)) causing extensive environmental damage. There have been ship collisions involving passenger ferries that have resulted in the vessel capsizing leading to loss of life. Warships have faced escalating threat levels (anti-ship missiles and increasingly smart and undetectable mines) that have resulted in heavy damage in hostile area (HMS SHEFFIELD (1982), USS STARK (1986) and USS PRINCETON (1991)). Additionally, numerous smaller (less damaging in terms of loss of life, environmental or property damage) incidents have occurred suggesting that the structural systems presently used should be revisited and possible new systems proposed. Reinforcing this need is general public awareness of the potential for damage due to failure of ship or offshore structural systems in environmentally sensitive areas. Specifically, this includes the Arctic with respect to icebreakers and offshore structures, Canadian coastal waters containing fish stocks and numerous other wildlife with respect to offshore structures (oil fields in the Hibernia and Sable Island areas) and general ship cargo included oil, refined petrochemical products and passengers.

Although most of the above mentioned incidents have been attributed to human error (or direct human action) it is reasonable to assume that humans will continue to err and therefore such incidents will continue to happen. Thus highly survivable structures that remain able to perform satisfactorily at service load levels, without incurring large cost (acquisition or through life) increases, are desired and must be designed. Given the

successful validation of the IMASP model, presented in Chapter 5, stiffened plate structures can now be evaluated for their ultimate strength and inelastic membrane response behaviour under transverse loads and survivability assessed.

6.2 Typical Marine Structures

As a starting point to arriving at a structural system with improved ultimate strength characteristics, typical marine structures as presently constructed must be reviewed. Tables 6.3, 6.5 and 6.7 present typical structural systems of ships as they are presently built or have been built in the recent past. Table 6.3 presents the framing system for a Canadian medium duty icebreaker. Table 6.5 presents a typical framing arrangement for a frigate or destroyer size warship. Table 6.7 presents the framing system of an older, ice strengthened single hull oil tanker. These designs serve as a basis for evaluating newer double hull structural systems. Each of these tables contain a description of the framing system, scantlings of the system components, steel grades, weight, and the cost of constructing a panel bounded by decks and bulkheads. The cost figure incorporates both the cost of material and labour. Details of the cost for each panel are provided in a table accompanying each of the structural configuration description tables and are numbered 6.4, 6.6 and 6.8 for the icebreaker, frigate and tanker structures, respectively. The hydrostatic pressure to cause rupture of the plating has been determined based on the assumption that the stiffening elements which create the smallest plate do not collapse. The total load is calculated assuming that the panel is fully loaded.

6.2.1 Cost Calculations

The primary objective of any engineered structure is to provide a prescribed capability or function at minimum cost. Therefore, when evaluating different structural configurations for ultimate strength cost must be considered as a parameter in the

determination of the viability of the structural system.

Construction costs can be separated into their material and labour components. For marine structures, (welded stiffened panels) labour costs associated with welding are a significant proportion of the total labour cost and can be identified separately. In the following subsections details of the cost calculations for materials, welding labour and the remaining fabrication labour are presented.

6.2.1.1 Material Costs

The cost of materials presented herein are for the structural steel only. Welding consumables are incorporated into the labour rates used. Steel costs for marine structures are dependent upon the material yield strength and the level of toughness of the steel. Table 6.1 contains the values used to determine material costs.

Table 6.1: Structural Steel Costs

Steel Description	Cost \$ / tonne
$\sigma_y = 235$ to 300 No Specified Toughness	600
$\sigma_y = 300$ to 350	700
$\sigma_y = 350$ to 500 Notch Tough	800

6.2.1.2 Labour Costs

The labour rate used in the cost calculations is \$45 per hour. This figure is based upon a \$25 per hour wage combined with an 80% overhead.

Welding labour costs have been calculated by taking a basic configuration and its associated labour cost and then modifying this to account for complexities in specific structure. The cost of welding, in excess of the basic cost of laying down a single bead

of weld, is a function of the following:

- a). thickness of the pieces being joined;
- b). edge preparation
- c). weld type (butt or fillet); and
- d). welding process (manual or automatic).

Items a), b) and c) influence the weld volume, number of weld passes required and the rate of stick advance. Item d) influences the cost of the welding process per metre.

The basic configuration assumes that a single weld pass deposits a cross-sectional area of weld metal equal to 30 mm² and that 1.5 metres of this weld can be deposited by manual welding processes per manhour of labour. If automatic welding processes (machine welding) are used it is assumed that 6.0 metres of the 30mm² cross-sectional area of weld can be deposited per manhour of labour.

To determine the cost of labour for fillet welds the following process is followed:

1. Determine the length of fillet welds required, in metres, for each joint and record this value along with the thickness of the thinner plate at the joint;
2. Determine the weighted average plate thickness, based on the weld length;
3. Multiply the result of Step 2 by 80%;
4. Determine the number of weld passes required as:

$$\frac{(\text{Step 3})^2}{2} / 30$$

5. Determine cost as:

$$(\text{Step 1}) * \$45 * (\text{Step 4}) / 1.5 = \$$$

6. If automatic welding is used divide Step 5 by 4.

To determine the cost of labour for butt welds the following process is followed:

1. Determine the length of butt welds required, in metres, in the shell

plating as;

$$\text{Panel Perimeter} + (\text{Area of Shell Plate} * 14.5 / 30)$$

The second term in this function accounts for the number of plates required to cover the panel based on a plate size of 2.5m x 12m.

2. Determine the average plate thickness to be welded (weighted average based on area if more than one plate thickness is being used);
3. Determine the volume of weld required per metre of length as:

$$\frac{(\text{Step 2})^2 + 4.5}{2}$$

This assumes an edge preparation with a 45° bevel, a three millimetre nose and a three millimetre gap between the plates;

4. Determine the number of weld passes required as:

$$(\text{Step 3}) / 30$$

5. Determine cost as:

$$(\text{Step 1}) * \$45 * (\text{Step 4}) / 1.5 = \$$$

6. If automatic welding is used divide Step 5 by 4.

In addition to the labour costs associated with welding there are labour costs associated with handling of the steel, fit up for welding, general clean up, placing temporary services (scaffolding, lighting, heating, etc.), crane usage and general administration. To account for these costs a basic fabrication cost of 50 manhours per tonne of material has been assumed.

Due to the complexities in structural arrangements the basic resultant manhour figure is revised to account for the following:

- a). number of pieces to be handled;
- b). amount of curvature in the hull;
- c). number of intersections;
- d). access; and

e). thickness of the plating (edge straightening)

The number of pieces to be handled increases fabrication costs. To account for this, it is assumed that an extra 0.5 manhours is added to the basic cost for each piece of structure in a panel. A rule of thumb used to determine the number of pieces is to assume that plates are a maximum size of 2.5 metres by 12 metres and that the maximum length of any rolled section is 12 metres.

Curvature of the shell plate adds significantly to the amount of set up that is required during panel construction. Jigs must be constructed and set up to position the shaped plate and bent stiffeners. Double curvature further complicates the processes. Curvature, changing along the length of the hull, also complicates the process of joining assemble units into larger erection units and finally for the completed hull. Flat panels, relatively speaking, are easily and quickly set-up and constructed. To account for the incremental cost resulting from curvature the basic fabrication manhour figure of 50 manhours per tonne is increased by 10% for single curvature applications and by 30% for double curvature applications.

Intersections of stiffening elements have an appreciable impact of the cost of fabricating a panel. Cut outs for the passage of the smaller stiffening element through the larger element must be produced. In most cases these cutouts must be compensated for, to return the strength to the member. If the smaller member is not continuous through the larger member the number of pieces to be handled increases as does the amount of welding required. To account for these costs the fabrication manhour figure is increased by one manhour per intersection.

The spacing of the stiffening elements affects the ease of fabrication and drives the decision on the use of automatic welding machines. The closer the stiffeners are together the tougher the fabrication process becomes. Positioning of manual welding is more restrictive and if the spacing is less than 600 mm the use of welding machines is not feasible. To account for costs associated with stiffener spacing the following process is used:

1. Divide 1000mm by the smallest stiffener spacing (in mm);

2. Determine the number of panels with the minimum spacing;
3. Determine the additional fabrication manhours as:

$$\text{Step 1} * \text{Step 2} * 1 \text{ Hr}$$

Thin plates are prone to distortion from the heat of welding. Control of this distortion is important at the boundaries of construction units where these units are joined to other units to form larger assemblages. For plates thinner than 10mm plate edge distortion is a concern with progressively greater effort required to straighten the edges as the plate thickness decreases from 10mm. To determine the cost of performing the necessary edge straightening process the following equation is used:

$$(\# \text{ of metres of Edge plate for each thickness}) * Y = \text{hours},$$

where Y is obtained from Table 6.2

Table 6.2: Parameter for Edge Straightening Costs

Plate Thickness (mm)	Y (hours/m)
5	2.5
6	2.0
7	1.5
8	1.0
9	0.5
10	0
> 10	0

The manhours associated with each component of the fabrication process are added together to obtain the total number of fabrication manhours. This value is then multiplied by the labour rate of \$45/hour to obtain a cost figure. The material, welding labour and fabrication costs are then summed to arrive at the panel construction cost. The cost figures presented in each table are for comparison purposes only. They have not been validated against vessel "as built" cost data.

Table 6.3: Canadian Medium Duty Icebreaker - Midbody Side Shell Structure (CCG Type 1200)

General Parameters:		Hierarchy of Supports
Steel Grade: Lloyds EH36 ($\sigma_y = 355$ MPa)		- Web frames support stringers
Main Frame Spacing: 460 mm		- Stringers (longitudinal orientation) support main frames
Web Frame Spacing: 1840 mm		- Main frames support plating
Longitudinal Spacing: 1000 mm		
Distance Between Lower & Main Decks: 2895 mm		
Typical Distance Between Bulkheads: 17.480 m		
<u>Element</u>	<u>Scantlings</u>	<u>Weight (kgs)</u>
Shell Plating	35 mm	13904
Main Frames (28)	Flat Bars - 250 x 20	3182
Web Frames (9)	Angle - Web 700 x 20 - Flange 200 x 12.5	3375
Stringers (3)	44 Angle - Web 500 x 20 Flange 75 x 12.5	4482
Total Weight: 24943 kgs		
Cost of Panel Bounded by Decks and Bulkheads = \$ 161,395		
Cost of Panel (\$/m ²) = 3190		
Hydrostatic Pressure Required to Rupture Plate: 26500 kPa		
Total Load if Smallest Panel Fully Loaded = 26500 * 0.460 * 1.000 = 12190 kN

Figure 6.1: Canadian Type 1200 Icebreaker

Table 6.4: Construction Costs for Medium Duty Canadian Icebreaker

Component	Sub-Component	Manhours	Dollars
Material	\$/tonne	N/A	19,950
Welding Labour	Fillet Welds		37,970
	Butt Welds		11,315
Fabrication Labour	Basic	1247	56,115
	# of Pieces	75	3,375
	Curvature	374	16,830
	# of Intersections	111	4,995
	Access	241	10,845
	Edge Straightening	0	0
Totals			\$161,395

Table 6.5: Warship - Side Shell Structure (Canadian Patrol Frigate)

General Parameters:		Hierarchy of Supports
Steel Grade: CSA CAN3 G40.21M 350WT Cat 5 ($\sigma_y = 350$ MPa)		- Frames support longitudinals
Transverse Frame Spacing: 2000 mm		- Longitudinals support plating
Longitudinal Spacing: 550 mm		
Height between decks = 2750 mm		
Typical Distance Between Bulkheads: 10 m		
Element	Typical Scantlings	Weight (kg)
Shell Plating	10 mm	2160
Frames (4)	Tee - 300 x 10 Web 175 x 15 Flange	485
Longitudinals (4)	Tee - 122 x 5 Web 102 x 8 Flange	448
Total Weight: 3093 kg		
Cost of Panel Bounded by Decks and Bulkheads = \$ 18794		
Cost of Panel (\$/m ²) = 683		
Hydrostatic Pressure at End of Plate Elastic Response = 1400 kPa		
Hydrostatic Pressure Required to Rupture Plate = 8525 kPa		
Total Load if Smallest panel Fully Loaded = $8525 * 0.550 * 2.0 = 9377$ kN (IMASP Analysis)

Figure 6.2: Typical Frigate Structural Arrangement

Table 6.6: Construction Costs for Canadian Warship Structure

Component	Sub-Component	Manhours	Dollars
Material	\$/tonne	N/A	2,474
Welding Labour	Fillet Welds		2,315
	Butt Welds		2,125
Fabrication Labour	Basic	155	6,975
	# of Pieces	10	450
	Curvature	47	2,115
	# of Intersections	16	720
	Access	36	1,620
	Edge Straightening	0	0
Totals			\$18,794

Table 6.7: Single Hull Oil Tanker - Side Shell Structure (Enerchem J.C. PHILLIPS)

<u>General Parameters:</u>		
Steel Grade:	Lloyds Grade E ($\sigma_y = 235$ MPa)	Vessel is Ice Strengthened
Transverse Frame Spacing:	2743 mm (9')	
Main Frame Spacing:	686 mm (27")	
Intermediate Frame Spacing:	343 mm (13.5")	
Stringer Spacing:	1829 mm (6')	
Height between Bottom Structure & Main Deck = 6.858 m (22'6")		
Typical Distance Between Bulkheads: 13.716 m (45')		
<u>Element</u>	<u>Typical Scantlings</u>	<u>Weight (kg)</u>
Shell Plating	17.46 mm (11/16")	12893
Transverse Frames (4)	Tee - 533mm x 8.73mm Web - 102mm x 8.73mm Flange	5001
Main Frames (15)	Angles - 127mm x 9.52mm Web (Flanged) - 76mm x 9.52mm Flange	1560
Intermediate (Ice) Frames (20)	Angles - 102mm x 9.52mm Web (Flanged) - 76mm x 9.52mm Flange	1825
Longitudinal Stringer (3)	Angles - 533mm x 11.11mm Web (Flanged) - 152mm x 11.11mm Flange	2458
Total Weight: 23737 kg		
Cost of Panel Bounded by Decks and Bulkheads = \$ 150,515		
Cost of Panel (\$/m ²) = 1600		
Hydrostatic Pressure Required to Rupture Plate: 11900 kPa		
Total Load if Smallest Panel Fully Loaded = 11900 kPa * 1.715 * 0.343 = 7000 kN

Figure 6.3: J.C. Phillips - Single Hull Tanker

Table 6.8: Construction Costs for Single Hull Oil Tanker

Component	Sub-Component	Manhours	Dollars
Material	\$/tonne	N/A	14,250
Welding Labour	Fillet Welds Butt Welds		11,525 4,050
Fabrication Labour	Basic	1187	53,415
	# of Pieces	75	3,375
	Curvature	1190	53,550
	# of Intersections	117	5,265
	Access	113	5,085
	Edge Straightening	0	0
Totals			\$150,515

Tables 6.3, 6.5 and 6.7 present structural systems of different configuration, material strengths, plate thicknesses and spacing between structural components. It is expected that varying these parameters would alter the ultimate strength characteristics of the structure. Other factors that could influence the ultimate strength behaviour include load location and direction on a panel, the in-plane boundary stiffness, and rate of load application. The influence of all of these items is investigated in the next section - Parametric Studies.

6.3 Parametric Studies

With the confidence that the mathematical model (IMASP) provides a valid load/deflection response prediction of a plate panel subjected to transverse load parametric studies can be performed. The effects of varying the parameters that are expected to affect the response behaviour are investigated to establish parameter sensitivities and thus broaden the understanding of the response behaviour. The structures presented in Tables

6.3, 6.5 and 6.7 are used as a basis for these studies.

6.3.1 Plate Length

While keeping the load condition, support in-plane stiffness, plate thickness, plate width and the material properties (parent plate material only) constant the effect of varying the plate length (short axis of the stiffened plate panel) on the load deflection response has been investigated. The results of the investigation are presented, along with a schematic of the plate and loading arrangement, in Table 6.9. The plate was assumed to be in a biaxial stress state, representing a continuous plate condition.

Table 6.9: Effect of Varying Plate Length on Load/Deflection Response

- Plate Width = 550 mm
- Plate Thickness = 10.0 mm
- Material $\sigma_y = 350$ MPa
- L = Length

Plate Length (mm)	At End of Elastic Response		At Ultimate (Rupture) **	
	Load (kN)	Deflection (mm)	Load (kN)	Deflection (mm)
1000	345	31	2220	153
1500	345	47	2220	230
* 2000	345	58	2220	306
2500	345	76	2220	382

* Baseline structure described in Table 6.5

** Von Mises-Huber-Hencky yield criterion extended to ultimate

From Table 6.9 it can be seen that the load carrying capability of a plate is not affected by a change in length. The deflection, at rupture and at the end of the elastic response phase, is however directly proportional to a change in length. Large panels are better than small panels in absorbing energy through plate deformation. In structures comprised of large panels that are stiffened, which give the appearance of an array of small panels, large loads may be supported if the stiffeners defining the small panels are proportioned so that they will become unstable (trip) prior to rupture of the small panel. This instability allows adjacent panels to become part of the loaded membrane area thus increasing the energy absorption capacity of the system.

6.3.2 Plate Thickness

By keeping the loading condition, support in-plane stiffness, plate length and width, and the material properties constant, the effect of varying the plate thickness on the load deflection response can be established. Table 6.10 presents the predicted load and deflection values at the ultimate capacity for each plate. Failure as defined by the Von Mises-Huber-Hencky yield criterion extended to ultimate uses the shear stress and axial stresses in the plate. Failure defined as exceedence of the material ultimate strain uses the combined axial and bending stresses.

Table 6.10: Effect of Varying Plate Thickness on Load/Deflection Response

- Plate Width = 550 mm ● Plate Length = 2000 mm ● Material $\sigma_y = 350$ MPa

Plate Thickness (mm)	At End of Elastic Response		At Ultimate (Ultimate Strain Criterion)		At Ultimate (Von Mises Criterion)	
	Load (kN)	Deflection (mm)	Load (kN)	Deflection (mm)	Load (kN)	Deflection (mm)
2.5	80	58	658	356	540	306
5.0	155	58	1315	356	1110	306
7.5	240	58	1970	356	1665	306
* 10.0	345	58	2630	356	2220	306
15.0	490	58	3944	356	3330	306

* Baseline structure defined in Table 6.5

From Table 6.10 it can be seen that the plate deflection values at rupture and at the end of the elastic response phase are not affected by changes in plate thickness. The load carrying capability of the plate is however directly related to the plate thickness. Doubling the plate thickness will double the load carrying capability at rupture and at the end of the elastic response phase. The load at ultimate as defined by exceedence of the ultimate strain value is consistently higher (by 18.5%) than that predicted by the Von Mises criterion. For extremely thick plates the ultimate strain failure criteria may become important.

6.3.3 Boundary Stiffness

The effect of varying the elastic in-plane restraint at the support on the load deflection response of a plate, while holding all other parameters constant is illustrated in

Figure 6.4

Figure 6.4: Effect of Varying the In-Plane Support Restraint on the Load/Deflection Response

As shown in this figure, the ultimate (rupture) load and deflection are independent of the elastic in-plane support stiffness. The load and deflection at the end of the elastic response are, however, highly dependent on the elastic in-plane support stiffness. Softer in-plane support restraint appears to increase the elastic portion (initial straight line) of the load deflection response curve. Using statics, it can be shown that as the applied lateral load magnitude and therefore the lateral deflection increases the horizontal component of the membrane forces and therefore the horizontal thrust at each support lessens. As this

thrust lessens the amount of additional lateral support translation lessens. Thus, the load versus deflection response at higher load levels is less dependent upon the boundary in-plane stiffness. In addition, axial strains at the support are directly related to the membrane force in the plate. Support translation allows greater deflection and thus lower membrane (axial) forces in the plate. With this axial strain relief greater deflection is required, with accompanying load, to cause the material yield stress to be reached by combining the remaining axial stress with the bending stresses. Conversely stiffer boundaries reduce the deflection for a given load.

6.3.4 Material Properties

6.3.4.1 Parent Plate Yield Stress

The effect of varying the parent plate material yield stress, while holding all other parameters constant, on the load/deflection response of a plate is investigated in this subsection. The material stress versus strain curves have been generated using IMASP (refer to Section 4.5) and thus the ultimate strain values are identical for each case investigated. Different ultimate stress values, equal to 1.45 times the material yield stress, are used. The modulus of elasticity for each steel was assumed to be equal to 207000 MPa. The results are presented in Table 6.11 along with a schematic diagram of the panel under consideration.

Table 6.11: Effect of Varying Material Yield Strength on Load/Deflection Response

- Plate Width = 550 mm • Plate Length = 2000 mm • Plate Thickness = 10 mm

Yield Strength (MPa)	At End of Elastic Response		At Ultimate (Rupture) **	
	Load (kN)	Deflection (mm)	Load (kN)	Deflection (mm)
275	230	51	1750	306
300	270	55	1905	306
325	315	59	2070	306
350	345	58	2220	306
375	375	61	2380	306
400	405	62	2550	306

** Von Mises-Huber-Hencky yield criterion extended to ultimate

From Table 6.11 it can be seen that the plate deflection value at rupture is unaffected by an increase in the parent plate material yield strength while the ultimate load carrying capability increases with yield and ultimate tensile strength. The deflection and load values representing the end of the elastic response portion of the load deflection curve increase with increasing material yield strength as expected. Given that the ultimate strains have all been taken equal 0.175 the deflection at rupture is insensitive to increased yield strength.

6.3.4.2 Parent Plate Ultimate Strain

The effect of varying the parent plate material ultimate strain, while holding all other parameters constant, on the load/deflection response of a plate is investigated in this subsection. The material stress versus strain curves have been generated using IMASP

(refer to Section 4.5) except that the ultimate strain values for each case were varied having values of 0.160, 0.170, 0.175, 0.180, 0.190 and 0.200. The yield stress for the steel was taken as 300 MPa. The material stress strain curve is identical for each analysis case with the exception of the last segment where the slope of the stress strain curve was kept constant. The results are presented in Table 6.12 along with a schematic diagram of the panel under consideration.

Table 6.12: Effect of Varying Material Ultimate Strain on Load/Deflection Response

- Plate Width = 550 mm • Plate Length = 2000 mm • Plate Thickness = 10 mm
- Yield Stress = 300 MPa

Material Properties at Ultimate		At Ultimate (Rupture) **			
Stress (MPa)	Strain (mm/mm)	Load (kN)	Percentage of Base Case Load	Deflection (mm)	Percentage of Base Case Deflection
432.6	0.160	2604	82.7	532	100.4
434.2	0.170	3096	98.3	530	100.0
435.0	0.175 *	3150	100.0	530	100.0
435.8	0.180	3166	100.5	528	99.6
437.4	0.190	3173	100.7	525	99.1
439.0	0.200	3192	101.3	524	98.9

* IMASP library ultimate strain value base case

** Ultimate Strain criteria

From Table 6.12 it can be seen that increasing the ultimate strain value of the plate material has a negligible effect on the load or deflection at rupture. This is due to the fact that the material curve has a very low slope and thus a small change in load can result in a large change in strain.

6.3.4.3 Heat Affected Zone (HAZ)

Since plated structures are typically of welded construction, the effect of the inclusion of weld seams and heat affected zones on ultimate strength and deflection capacity (ductility) is of considerable significance. Plate weld seams at panel edges are naturally avoided since this is where stiffening elements are welded to the plate. However, the welding of the stiffeners to the plate results in a heat affected zone in the plate. By holding all other parameters (plate length, width and thickness, load configuration and in-plane support stiffness) constant the effect of including a HAZ at the panel edge on the load/deflection response process can be investigated. Table 6.13 presents the results of the comparison between a panel comprised solely of parent plate and one with the same parent plate plus material with IMASP generated HAZ material properties at the panel edges. The length of the HAZ at each plate edge is 1.5 times the plate thickness (52.5 mm).

Table 6.13: Effect of Heat Affected Zone Material on Load/Deflection Response

- Plate Width = 460 mm
- Plate thickness = 35 mm
- Parent Plate $\sigma_y = 355$ MPa

Material	At End of Elastic Response		At Ultimate (Rupture) **	
	Load (kN)	Deflection (mm)	Load (kN)	Deflection (mm)
* Parent Plate (PP)	1030	30	6500	153
PP + HAZ ***	1060	31	3200	90

* Baseline structure defined in Table 6.3

** Ultimate Strain criteria

** HAZ at the panel edges (supports)

In Table 6.13 the dramatic reduction in the ultimate load carrying capability and in the maximum deflection due to the inclusion of HAZ is evident. The load deflection values at the end of the elastic response phase are unaffected by HAZ inclusion because the length of the HAZ relative to the length from the support to the first load point (first plate segment) is small. Thus, the smaller strain in the HAZ material for a given stress in the first plate segment does not influence the overall behaviour for strains up to the parent plate yield strain. If the distance from the support to the first load point is small the load and deflection values at the end of the elastic response may differ from the parent plate with HAZ inclusion. This effect is investigated in Section 6.3.5.

6.3.4.4 Weld Metal

Large welded construction plated structures must contain plate butt welds. These weld seams introduce both a HAZ material and the weld metal itself into the plate. The effect on the load/deflection response of introducing a seam can be investigated by holding all other parameters constant and introducing the weld. IMASP generated weld and HAZ material properties are used in the investigation. The results of the investigation are presented in Table 6.14. In this table the first line is the benchmark parent plate only analysis. The effect of varying the location of the weld seam is also investigated.

Table 6.14: Effect of Introducing Weld Material on Load/Deflection Response

- Plate Width = 460 mm • Plate thickness = 35 mm • Parent Plate $\sigma_y = 355$ MPa

Material	Weld Location Relative to the Left Support (mm)	At End of Elastic Response		At Ultimate ** (Rupture)	
		Load (kN)	Deflection (mm)	Load (kN)	Deflection (mm)
* Parent Plate (PP)	No Weld	1030	30	6500	153
PP + HAZ	HAZ @ Supports	1060	31	3200	89
PP + HAZ + Weld	125	1160	34	3200	89
PP + HAZ + Weld	400	1160	34	3200	89

* Baseline structure defined in Table 6.3

** Ultimate Strain criterion

As indicated in Table 6.14, the introduction of a weld seam with its associated HAZ provides a slight increased load carrying capability at the end of the elastic response as compared to the parent plate only or parent plate plus HAZ at the support points. The introduction of a weld between the supports has no effect on the ultimate load carrying capability of the plate as compared to the case with parent plate plus HAZ at the supports.

6.3.5 Effect of Varying Load Position and Angle

The effect of varying the location and direction of the load, while holding all other parameters constant, on the load/deflection response of a plate is illustrated in Figure 6.5.

By moving a normal load from midspan to a quarter point the ultimate load capacity of the plate is reduced by approximately 30%. Also the response is stiffer with less deflection resulting from a given load.

Figure 6.5: Effect of Varying the Load Position and Direction on the Load/Deflection Response

A load on a non-normal angle to the plate significantly decreases the ultimate load carrying capability of the plate. This is due to increased axial strain in the plate. The strains created by the horizontal component of the load is additive to the membrane strains developed to carry the vertical component of the load.

This study illustrates the need to balance the shear capability of the plate supports. At shear values greater than 50% of the ultimate load carrying capability of a plate loaded at midspan the support should start to yield and buckle thus allowing the plate to shed

load by increasing the size of the loaded panel. This would prevent premature rupture of the plate. For example referring to the two normal load cases in Figure 6.5, for the midspan load each support is carrying approximately 625 kN (Total Load carried = 1250 kN) prior to rupture and the left support of the point load at quarter span case is similarly carrying (vertically) approximately 675 kN (Total Load carried = 900 kN) at rupture. The ideal load under which the support would buckle would be 650 kN, thus ensuring that loads close to the support will not cause rupture before a large deflection and energy absorption occurs.

6.3.6 Effect of Varying Rate of Load Application

Table 6.15 illustrates the effect of the rate of load application and therefore the rate of material straining on the load/deflection response of a plate. All parameters except the strain rate have been held constant. The values given in the table indicate that increased rates of strain lead to higher load carrying capability at the end of the elastic response phase and markedly lower ultimate load carrying capacity. With increased strain rates less energy can be absorbed by the structure. Table 6.15 also indicates that, for steel, the response to loadings with strain rates less than 0.001 mm/mm/sec can be treated as static load cases.

Table 6.15 : Effect of Varying Strain Rate on Load/Deflection Response

- Plate Width = 500 mm • Plate thickness = 10 mm
- Material $\sigma_y = 350$ MPa (curve from Kennedy & Cheng - plane stress)

Strain Rate (mm/mm/sec)	At End of Elastic Response		At Ultimate (Rupture) **	
	Load (kN)	Deflection (mm)	Load (kN)	Deflection (mm)
1×10^{-5} (Static)	254	56	2216	413.2
0.001	254	56	2215	412.9
0.05	264	57	2182	400
0.1	280	58	2152	388
1.0	382	64	1892	306
10	418	68	1592	236

** Von Mises-Huber-Hencky yield criterion extended to ultimate

Examples of in-service strain rates that may be encountered by ships are given by Shoemaker (1981) and are summarized in Table 6.16.

Table 6.16: Examples of Ship In-Service Strain Rates

"Type" of Loading	Ship Service Example	Strain Rate (mm/mm/sec)
Static	- Hull Loading during Calm Seas	1×10^{-5}
Intermediate	- Hull Loading during Slamming Conditions on Rough Seas	1×10^{-3}
Extremely High	- Local Damaged Areas during Collision - Explosive Loading of Structure	10

It is of significance to note that the results presented in Table 6.15 when combined with the data of Table 6.16 indicate that for normal ship loads, including slamming, steel does not behave differently than it would under static conditions.

6.3.8 Summary / Conclusions from Parametric Studies

Table 6.17 provides a summary of the parametric studies that have been performed. For each parameter the effect of a change to the parameter on the load versus deflection response of the loaded plate at the end of the elastic response and at the ultimate (rupture) load, expressed in general terms, is given in the table.

Table 6.17: Parametric Study - Summary

Variation	Effect at End of Elastic Response Phase		Effect at Ultimate Load Capacity	
	Load Level	Deflection	Load Level	Deflection
Plate Length - Increased	No Change	Increased	No Change	Increased
Plate Thickness - Increased	Increased	No Change	Increased	No Change
In-Plane Restraint at Support - Increased	Decreased	Decreased	No Change	No Change
Parent Plate Yield Strength - Increased	Increased	Increased	Increased	No Change
Parent Plate Ultimate Strain - Increased	Not Applicable	Not Applicable	Increased	No Change
Inclusion of Heat Affected Zone	No Change	No Change	Decreased	Decreased
Point Load Moved Closer to Support	No Change	Decreased	Decreased	Decreased
Point Load at an Angle - Directed at Closest Support	Decreased	Decreased	Decreased	Decreased
Point Load at an Angle - Directed Away from Closest Support	No Change	No Change	Decreased	Decreased
Increased Strain Rate	Increased	Increased	Decreased	Decreased

6.4 Design Implications

The summary of the parametric studies can be used to determine the effect of making changes to a given structural configuration to achieve specific types of load deflection behaviour at the end of the elastic response region or at the ultimate (rupture) state. The following paragraphs contain discussion relating to the proportioning of structure, with reference to the baseline structures described in Section 6.2 for ultimate load scenarios.

As a prelude to the discussion of proportioning structure to achieve maximize load carrying capability prior to rupture, the ship structure as presented in Table 6.5 will be analyzed. This structure has 10mm thick shell plating, 550mm spacing between longitudinals, 2000mm spacing between frames, 2750mm between deck height and 10 metres between bulkheads. The load imparted to the side shell structure is assumed to be from another ship colliding at a right angle. The bow of the striking ship is assumed to be non-rigid so that the contact area between the ships is increasing (bow of the striking vessel is crushing) as the deformation of the struck vessel side shell increases. This load scenario will effectively act as a hydrostatic pressure. The response up to the ultimate (rupture) load level could be as follows:

Load Level (1)

Development of inelastic membrane action in the plate spanning between longitudinals:

Plate Length = 550 mm (between longitudinals)

Plate Thickness = 10 mm

Plate Yield Strength = 350 MPa

Panel Aspect Ratio = 3.63:1

Maximum Pressure (Rupture) = 8525 kPa

Maximum Deflection (Rupture) = 69 mm

If it is assumed that the initial width of the contact area between the two vessels is 1000 mm, and that the loaded area is equal to the contact width multiplied by the plate length, the total applied load is 4690 kN.

Load Level (2)

At a load slightly less than the rupture loads of (1), stiffeners B and C trip, and the panel continues to carry load with the following geometry:

Plate Length = 1650 mm (three longitudinal spacings)

Plate Thickness = 10 mm

Panel Aspect Ratio = 1.21:1

Maximum Load (Rupture) = 2865 kPa

Maximum Deflection (Rupture) = 206 mm

If it is assumed that bow of the striking ship continues to crush such that the width of the load contact area has increased to 2000 mm, and with the plate length of 1650 mm, the total applied load is 9455 kN.

Load Level (3)

At a load slightly less than the rupture loads of (2), stiffeners A and D trip, the panel continues to carry load, but with a 90° change in loading response orientation (the dominant membrane action will be across the panel short axis) with the following geometry:

Plate Length = 2000 mm (between frames)

Panel Aspect Ratio = 1.375:1

Loaded panel between Frames 2 and 3.

**Plate Thickness (smeared thickness which includes the area of the tripped stiffening elements) = $[10 \cdot (2750) + 4 \cdot (1426)] / 2750$
= 12.07 mm**

Maximum Load (Rupture) = 2865 kPa

Maximum Deflection (Rupture) = 250 mm

If it is assumed that the contact area width is 2000 mm, and with the plate length of 2000 mm, the total applied load is 11400 kN.

Load Level (4)

At a load slightly less than the rupture load of (3), Frames 2 and 3 trip. The panel continues to carry load, but, again with a 90° change in loading response orientation (dominate response across the short distance between decks) and panel with the following geometry:

Plate Length = 2750 mm (between decks)

Panel Aspect Ratio = $6000 / 2750 = 2.18:1$

**Smeared Plate Thickness = $[10 \cdot (6000) + 2 \cdot (5625)] / 6000$
= 11.875 mm**

Maximum Load (Rupture) = 2035 kPa

Maximum Deflection (Rupture) = 343 mm

If it is assumed that the contact area width has increased to 6000 mm, and with the plate length of 2750 mm, the total applied load is 33580 kN.

The above response process involves the gradual expansion of a relatively small loaded panel into a larger responding panel. The collapse progression described can result in a large amount of energy being absorbed prior to plate rupture. It is this absorbed energy (combination of load level and deflection) that is important in the resistance of minor ship collisions or large unlikely (> 100 year recurrence) ice loadings on an offshore structure. The sequence could be carried farther than that described above if the members providing overall lateral support (reaction points) for load level (4), the decks and bulkheads, buckled allowing the response panel to grow across these boundaries. The buckling (tripping) process of the structural members (stiffeners) in the above analysis is a key part of the overall response. If the first level of support to the plate (longitudinals) had not tripped, plate rupture and the loss of watertight integrity would have occurred at a load of 4690 kN or approximately 14% of the calculated ultimate load. Many traditional structural philosophies have sought to avoid buckling and have associated its occurrence with failure and loss of the ability to support additional load. This loss of ability is due to the lack of inclusion of the membrane support mechanism.

The progressive strength analysis presented above requires buckling to achieve higher levels of ultimate load. Buckling is a sought after and welcome part of the overall process of designing to achieve the maximum ultimate load support response. This implies that the designer must predict the buckling load with some certainty. Since bifurcation buckling is often difficult to predict accurately, the use of non-symmetric stiffeners with their webs oriented at an angle other than 90° with respect to the shell would assist in achieving the desired buckling type response. This type of stiffener will gradually trip (lateral distortion under load due to eccentricities), it will not have a bifurcation type of buckling response.

The panel maximum load value obtained above for the ship structure is an upper bound. Several factors could influence the response behaviour to lower the maximum value. These factors include; one of the longitudinal stiffeners or transverse frames not tripping resulting in plate rupture; the longitudinal stiffeners or transverse frames not

tripping in pairs which would modify the panel geometry, or a stiffening element tripping prematurely most likely due to construction imperfections.

The parametric studies performed in Section 6.3 provide an indication of steps that could be taken to increase the amount of energy that could be absorbed by the vessel analyzed above. With reference to Table 6.17, for ultimate load conditions, an increase in plate thickness, yield strength, ultimate tensile strength, and ultimate strain (ductility) will allow larger loads to be supported, while increasing the distance between supports will allow larger deflections. Implementing such changes would result in different weights and build costs for a vessel. The optimum design would enhance ultimate strength (same height between decks) with minimal impact on the vessel baseline build cost.

The structure as presented in Table 6.18, a rework of the structure described in Table 6.5 incorporating the above ultimate strength enhancing changes, has been analyzed to illustrate the progression of failure and the probable ultimate capacity. The shell plating thickness has been increased from 10 mm to 12 mm. The longitudinal spacing has been increased from 550 mm to 915 mm and the size of the longitudinal stiffeners has been increased from a 125x5 web and 102x8 flange to 142x5 web and 102x10 flange. The UCREW longitudinals have been sized such that the same elastic bending stress levels are realized in each of the structures for a load condition of a hydrostatic pressure on the panel defined by the longitudinal stiffener spacing. The steel yield strength has been increased from 350 MPa to 420 MPa. The corresponding construction costs for this revised design are given in Table 6.19.

Table 6.18: Ultimate Collision Resistance Enhanced Warship (UCREW)
- Side Shell Structure

General Parameters:		Hierarchy of Supports
Steel Grade: Lloyds Grade EH42 ($\sigma_y = 420$ MPa)		- Frames support longitudinals
Transverse Frame Spacing: 2000 mm		- Longitudinals support plating
Longitudinal Spacing: 915 mm		
Height between decks = 2750 mm		
Typical Distance Between Bulkheads: 10 m		
Element	Typical Scantlings	Weight (kgs)
Shell Plating	12 mm	2590
Frames (4)	Tee - 300 x 10 Web 175 x 15 Flange	485
Longitudinals (2)	Tee - 142 x 5 Web 102 x 10 Flange	271
Total Weight: 3346 kgs		
Cost of Panel Bounded by Decks and Bulkheads = \$18,875		
Cost of Panel (\$/m ²) = 686		
Hydrostatic Pressure at End of Plate Elastic Response = 1400 kPa		
Hydrostatic Pressure Required to Rupture Plate = 7395 kPa (calculated as per Table 6.2.5)		
Total Load if Smallest Panel Fully Loaded = 7395 * 0.915 * 2.0 = 13533 kN		

Table 6.19: Construction Costs for UCREW Structure

Component	Sub Component	Manhours	Dollars
Material	\$/tonne	N/A	2,677
Welding Labour	Fillet Welds		1,560
	Butt Welds		2,983
Fabrication Labour 2 2222	Basic	167	7,515
	# of Pieces	8	360
	Curvature	50	2,250
	# of Intersections	12	540
	Access	22	990
	Edge Straightening	0	0
Totals			\$18,875

Applying the same analysis technique to describe the response of the structural panel up to ultimate (rupture), yields the following:

Load Level (1)

Development of inelastic membrane action in the plate spanning between longitudinals:

Plate Length = 915 mm (between longitudinals)

Plate Thickness = 12 mm

Plate Yield Strength = 420 MPa

Panel Aspect Ratio = 2.19:1

Maximum Pressure (Rupture) = 7395 kPa

Maximum Deflection (Rupture) = 113 mm

If it is again assumed that the initial contact width is 1000 mm, the total applied load is 6766 kN.

Load Level (2)

At a load level slightly less than the rupture loads of (1), the longitudinal stiffeners trip and the panel continues to carry load with a 90° change in loading response orientation (the dominant membrane action will be across the panel short axis) with the following panel geometry:

Plate Length = 2000 mm (between frames)

Panel Aspect Ratio = 1.375:1

Loaded panel between Frames 2 and 3.

Plate Thickness (smeared thickness which includes the area of the tripped stiffening elements) = $[12 \cdot (2750) + 2 \cdot (1730)] / 2750$
 $= 13.26 \text{ mm}$

Maximum Load (Rupture) = 3738 kPa

Maximum Deflection (Rupture) = 249 mm

If it is again assumed that the contact width has increased to 2000 mm at this load level, the total applied load is 14952 kN.

Load Level (3)

At a load slightly less than the rupture load of (2), Frames 2 and 3 trip. The panel continues to carry loads but, again with a 90° change in loading response orientation (dominate response across the short distance between decks) with the following panel geometry:

$$\text{Plate Length} = 2750 \text{ mm (between decks)}$$

$$\text{Panel Aspect Ratio} = 6000/2750 = 2.18:1$$

$$\begin{aligned} \text{Smeared Plate Thickness} &= [12*(6000) + 2*(5625)] / 6000 \\ &= 13.875 \text{ mm} \end{aligned}$$

$$\text{Maximum Load (Rupture)} = 2845 \text{ kPa}$$

$$\text{Maximum Deflection (Rupture)} = 342 \text{ mm}$$

If it is again assumed that the contact width has increased to 6000 mm, the total applied load is 46942 kN.

The results of the two ultimate strength analysis performed clearly demonstrate the increase in ultimate capacity (40% increase) as a result of the panel modifications. The cost of the panel is essentially the same and there is not a radical change in the structural layout.

6.5 Summary

In Section 6 the need for improved ultimate strength of marine vessels was discussed, typical structural arrangements were described, various structural parameters were varied to examine the impact on the ultimate strength of a panel within these typical structural arrangements and finally a full analysis of a warship structural panel was performed to describe the progression of failure and the ultimate load.

7.0 SUMMARY, CONCLUSIONS AND FUTURE WORK

Structures, in which the IMASP mathematical model can be used for portions of the design process or analysis, include commercial ships (oil tanker, chemical carriers and icebreakers), warships, and offshore structures (fixed or mobile rigs).

Oil tankers can be designed using the model to design an arrangement of structure to provide the maximum energy absorption capability during a collision or to analyze internal explosions using the high strain rate transformation feature. Offshore structures do not need fair plating as a service condition performance requirement and therefore their plating could deform appreciably to support high ice pressures. The model could be used to design the plate panels of these structures. Also, as an analysis tool the model can be used to predict the load that caused a large deformation in a plate panel of a structure.

Specific results and conclusions that can be drawn from this work include:

- (1) The program IMASP (Inelastic Membrane Analysis of Continuous Steel Plates), and the analysis methodology that it incorporates, gives results that correlate with data from tests by Young (1959), McDermott et al (1974), Ratzlaff and Kennedy (1986), Kennedy and Cheng (1987), Brown and Joubert (1990) and Ratzlaff (1995).
- (2) The program has the capacity to predict the elastic and inelastic membrane response for plates subjected to multiple load (varying magnitude and direction), varying in-plane restraint, varying strain rates and incorporates parent plate, weld and HAZ stress strain characteristics.
- (3) The program has the capacity to predict the ultimate load carrying capacity, based on either ultimate strain or Von Mises yield stress (extended to ultimate) failure criterion.
- (4) Costing algorithms for marine structures have been developed.
- (5) The general effect on the membrane response of a plate of varying plate length; thickness; in-plane restraint; yield strength; ultimate strength; inclusions of weld

and heat affected zones; location, orientation and rate of loading has been presented.

- (6) Application of the IMASP program, to describe the progressive failure of a typical structural panel and of a panel with similar dimensions but with scantlings and structural layout modified as suggested by the results of parametric studies, indicates that it is possible to improve ultimate load capacity without increasing cost.

The model and the structural support philosophy IMASP embraces can be developed further. Items to be considered in this future work are a continuation of the work on double membranes separated by a dense core (with inclusion of a bending phase model to improve relatively low load level predicted response accuracy), extension of the model to two panels so that multiple hits on adjacent panels can be analyzed and the addition of a process to track accumulated plastic damage. In addition a review of present design procedures for ships and offshore structures with the purpose of incorporating what can be learned from the membrane response prediction, and thus advancing these codes (Canadian and US naval design codes, Lloyds Registry of Shipping, American Bureau of Shipping, the Arctic Shipping Pollution Prevention Regulations and CSA S473) with a better understanding of extreme load behaviour would be useful. It is necessary to establish appropriate design loads for extreme loads due to collisions, explosions and low probability ice loads for comparison to predicted structural capability.

Additional experimental verification of the present extended features and future work additions of the mathematical model are recommended. Specifically, the inclusion of actual stress versus strain values for a heat affected zone material or a refined prediction methodology developed through a series of tests and validation of the high strain rate material property transformation through high strain rate material coupon testing are recommended. The development of a relationship allowing for the determination of the effective width of plating to be used for the analysis of a point load on a plate is recommended.

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