A comparative study on three different construction methods of stiffened plates-strength behaviour and ductility characteristics

(Estudo comparativo do comportamento e ductilidade de placas enrijecidas em três diferentes métodos de construção)

Mohammad Reza Khedmati

Amirkabir University of Technology, 424 Hafez Avenue, Tehran 15914, Iran
E-mail: khedmati@cic.aut.ac.ir

Khosrow Ghavami

Department of Civil Engineering, Pontifícia Universidade Católica (PUC-Rio), Rio de Janeiro, Brazil
E-mail: ghavami@civ.puc-rio.br

Mehran Rastani

Petropars Ltd. No. 10, North Naft Street, Mirdamad Blvd, Tehran, Iran

Resumo

O objetivo principal dessa pesquisa é a caracterização da carga última e ductilidade de placas enrijecidas com soldas não-contínuas, sujeitas à compressão axial. Para tanto, são realizadas análises numéricas detalhadas nas placas enrijecidas sob compressão axial com três tipos de solda. Para obtenção da resposta elasto-plástica não-linear, as placas são submetidas a carga de compressão até que haja o colapso. As placas enrijecidas selecionadas para análise, nesse estudo, são oriundas de estruturas da plataforma de navios de grande calado e de embarcações de canal de pequeno calado. São considerados três procedimentos diferentes para soldar o enrijecedor à placa, que são: solda contínua, solda intermitente e solda alternada por faixa. É dada uma atenção especial para a modelagem de elementos finitos das soldas. Alguns testes disponíveis, na literatura, foram analisados a partir da aplicação do método de elementos finitos FEM, mostrando a confiabilidade do programa.

Palavras-chave: Placas enrijecidas, solda intermitente, carga de flambagem, resistência última, ductilidade, elementos fintos.

Abstract

Strength and ductility characteristics of non-continuously welded stiffened plates under in plane axial compression are the main focus of this research. A series of detailed numerical analyses of stiffened steel plates subjected to in plane compressive load is performed. Complete equilibrium paths are traced up to collapse for non-linear elastoplastic response of stiffened plates. Stiffened plates are selected from the deck structure of real sea-going ships and inland waterway vessels. Three different stiffener-to-plate welding procedures are considered: continuous, chain intermittent and staggered intermittent fillet welding. Special attention is paid to the finite element modeling of the fillet welds in either of welding practices. Some available tests are simulated applying finite element method.

Keywords*: Stiffened plates, intermittent welds, weld gap, buckling load, ultimate strength, ductility, Finite Element Method (FEM).*

1. Introduction

Thin-walled structures are to be found in many of today's engineering disciplines and it is of particular note that they now formulate an increasing growing proportion of today's engineering construction. The quest for lighter more efficient optimised structural systems which provide high strength and stiffness in conjunction with low structural weight has served to promote this trend and also to increase the need for continued research and development for further advancement in the field. The range of application of thin-walled structures has become increasingly more diverse with a considerable deployment of thin-walled structural elements and systems found in a wide range of areas within the Civil, Mechanical, Chemical, Marine, Aeronautical and Offshore fields. Today's advanced structural design of thin-walled lightweight structural systems offers a considerable high degree of structural safety and reliability.

As an example of such thin-walled structures, ship structures may be exampled which can be modelled basically as a box girder consisting of a number of plates stiffened by different elements, Figure 1. In order to study the strength of ship hull girder under applied load conditions, as well as extreme and accidental limit states, having knowledge about the strength behaviour of different structural elements consisting of plate and stiffened plate is vital. One of the most important elements within ship structures as well as other thin-walled structures is stiffened plates. There are many research works concerning with the stiffened plates in which the plate and stiffeners have been assumed to be attached to each other using continuous fillet welds. The buckling and collapse behaviour of such stiffened plates under different loading conditions has been investigated both numerically [1-8] and experimentally [9-10].

On the other hand, the necessity to reduce the costs of labour and material and also to save the weight of ship hull and other thin-walled structures causes to apply other types of welding procedures or practices in their construction. Among the alternative welding methods or techniques, non-continuous welding is frequently applied for the attachment of plates and stiffeners. Unfortunately, there is very rare works on the response of the stiffened plates incorporating such a type of welding. The only work may be reported here is the one made by Miller et al. [11]. They focused their research on the mechanism of collapse in special type of non-continuously welded stiffened plates, namely intermittently welded serrated stiffened plates, which is applied in the construction of some inland tank barges operating in US waters. Serrated stiffeners are produced by cutting channels in half. As a result, two sections of serrated stiffeners are obtained, each of which is connected to the plate only where the weld exists. Some of the vessels constructed according to this practice, suffered catastrophic hull structural failures releasing thousands of gallons of heavy fuel oil into the water. Based on some experiments, Miller et al. concluded that the plate deflects into the serration just between the non-continuous weld lines, leading to significant reduction of the ultimate strength and subsequently much earlier failure of the whole stiffened plate.

The objective of this study is to investigate the sensitivity of the ultimate strength and failure mechanism of stiffened plates constructed by different welding methods in common. In line with this research, a series of elastoplastic large deflection analyses is performed on the stiffened plates, which are selected out of the structure of real seagoing ships and inland waterway vessels. Three different welding methods are compared with each other focusing

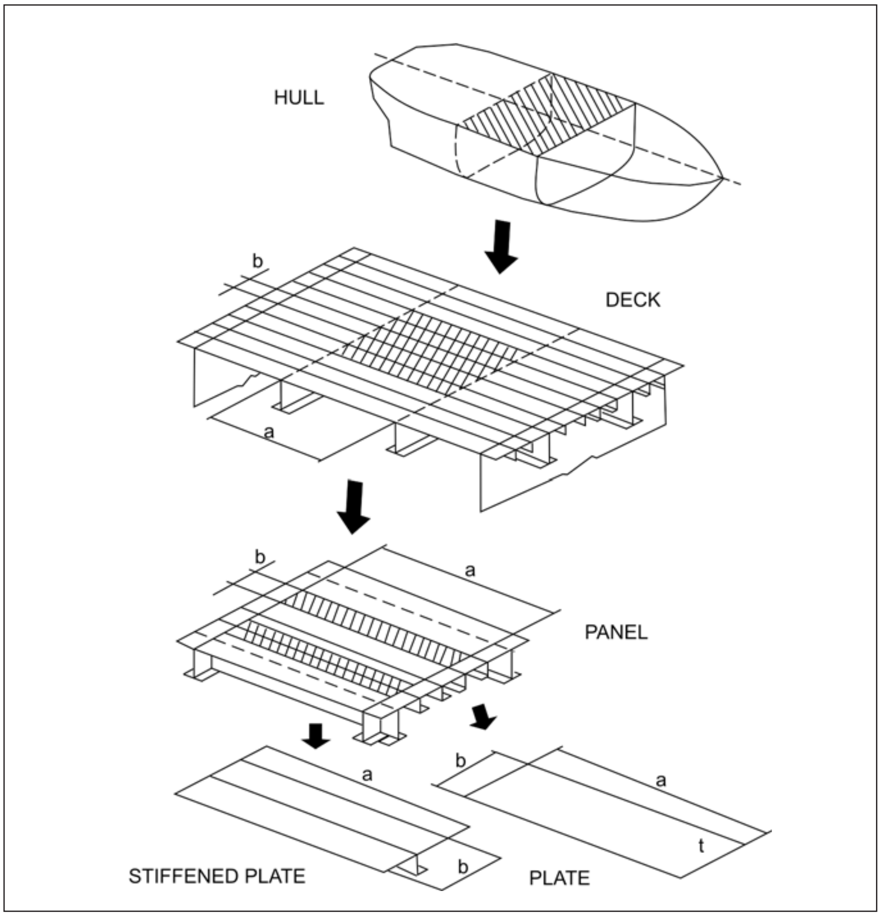


Figure 1 - Ship hull girder as a box-like thin-walled stiffened structure.

on their effect on the strength and ductility behaviour of stiffened plate. Special attention is made on the modelling of the fillet welds along continuous or intermittent weld lines.

2. Welding techniques used in ship and offshore construction

Continuous welding, Figure 2, may be used for attaching the stiffeners to plating. This technique requires the largest amount of weld, adds weight to the ship, and increases the construction costs. However, with the increased use of welding machines, this technique has become much faster and less expensive when compared to manual labour costs.

Typically, rather than using continuous welding, the longitudinal stiffeners are attached to the plating using intermittent fillet welds, in a staggered or chain arrangement, Figure 3. In addition to decreasing the construction costs, the magnitude of welding residual stresses is decreased from the amount present in continuous welding.

3. Failure mechanism of stiffened plates

The decks of ships are constructed using a series of cross-stiffened panels, as shown in Figure 4. The transverse members of the deck have substantially deeper webs and are more rigid than the longitudinal stiffeners, eliminating the possibility of grillage buckling. “Gross”, or “grillage” buckling occurs when the transverse members are not stiff enough to provide rigid support to the longitudinal stiffeners and they buckle together with the longitudinal stiffeners [12]. With the possibility of grillage buckling eliminated, if a compressive failure is going to occur in the deck, it will likely occur in the longitudinally stiffened sub-panels between transverse members. In general, longitudinally stiffened sub-panels subjected to an axial compressive load can buckle elastically, commonly referred to as “buckling”, or they can fail inelastically, known as “collapse” [12]. In “overall buckling”, the longitudinal stiffeners do not have sufficient lateral rigidity and they buckle together with the plating. “Local buckling” occurs when either the longitudinal stiffeners or plating between the longitudinal stiffeners buckles. If the longitudinal stiffeners buckle, the plate loses its lateral rigidity and overall buckling becomes imminent. Similarly, if the plating between the longitudinal stiffeners buckles, the stiffeners, behaving like individual columns, would eventually fail when forced to carry the entire load [12].

To prevent these elastic buckling failures, the longitudinal stiffeners must have sufficient torsional stability to prevent tripping and sufficient lateral rigidity to prevent overall buckling. Tripping occurs when a stiffener buckles by twisting about its line of attachment to the plating [12]. The stiffeners usually found in ship deck construction are large enough to prevent both types of elastic buckling. Therefore, a more complicated inelastic failure, or collapse, is more likely. Similar to elastic buckling, inelastic collapse of a gross panel can occur in two different ways. “Interframe” collapse occurs when a longitudinally stiffened sub-panel between any of the transverse stiffeners fails, while “gross panel” collapse is the

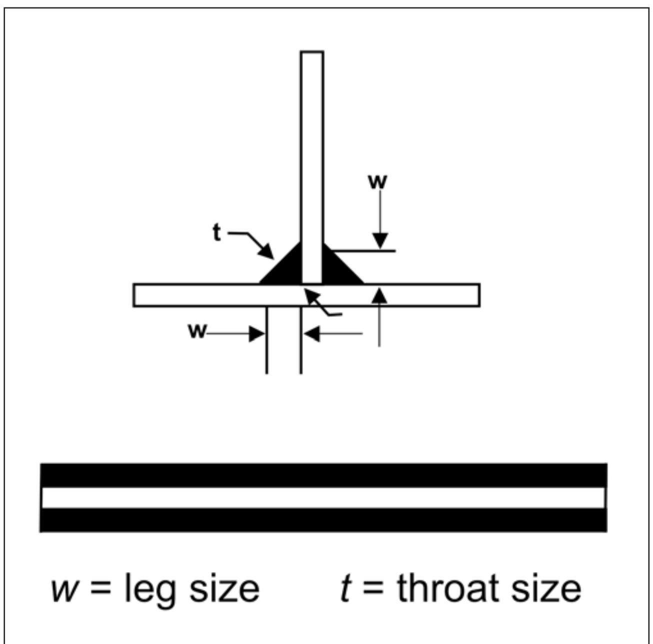


Figure 2 - Continuous welding of attaching the stiffeners to plating.

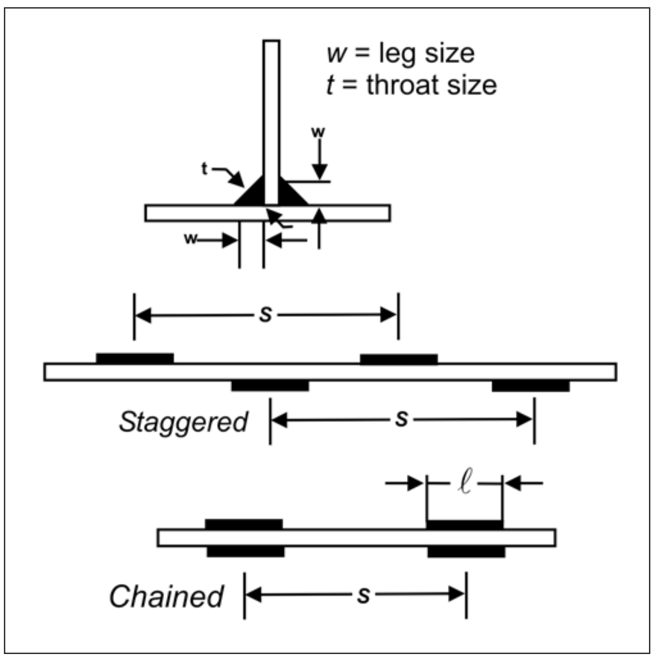


Figure 3 - Intermittent welding of attaching the stiffeners to plating.

failure of both the longitudinal and transverse members. Due to the minimum scantling requirements in the rules of classification societies, for example [13], the risk of gross panel collapse is minimized, making inelastic interframe collapse the likely failure mode for a typical longitudinally framed decks.

4. Characteristics of finite element analysis

4.1 FEA code

All FEA calculations have been performed applying ADINA commercial FEA code [14], which has adequate ability and accuracy to perform a nonlinear finite element analysis in elastoplastic range with due allowances for large deflections and initial imperfections. The algorithm of ADINA code is based on the automatic-step incremental solution of nonlinear finite element equations of the static analysis using the load displacement control (LDC) method. Such a method can be used to find the nonlinear equilibrium path of a model from pre- and post-buckling steps until its collapse. The LDC method of the ADINA code automatically selects the proper iteration method to achieve the quick convergence.

4.2 FE models

4.2.1 Stiffened plate under longitudinal in plane compression

Figure 5 shows a typical stiffened panel and the FE model (shaded area). Both the plate and the stiffener are modelled with eight-nodded isoparametric quadrilateral shell elements (Figure 6). This element is found to be the most effective one for the linear and nonlinear analysis of general shell structures with a high predictive capability, [16-17]. Fillet welds are also modelled with eight-nodded shell elements with the thickness equal to the throat of the fillet weld. For any element, 3 x 3 Gauss integration points in the r-s plane, Figure 7, and two points through the shell element thickness are taken into account. Also at the plate-to-stiffener junctions, it is assumed that the shell elements of the fillet welds are connected to those of the plate panel and the stiffener with some rigid elements, Figure 8.

The material behaviour of all shell elements is assumed to be elastic-perfectly plastic. Von Mises yield

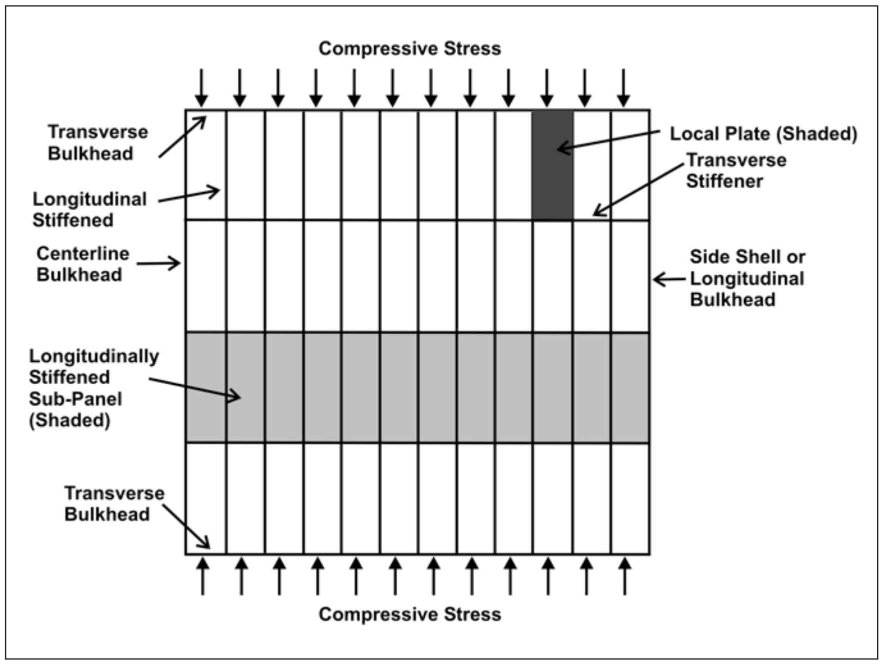


Figure 4 - Deck structural elements.

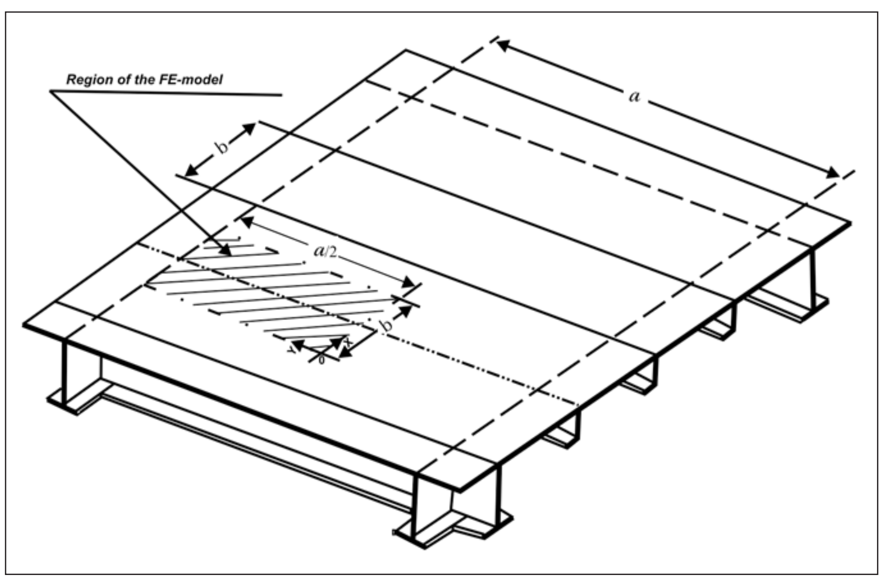


Figure 5 - Typical longitudinally stiffened panel and the extent of the FE model.

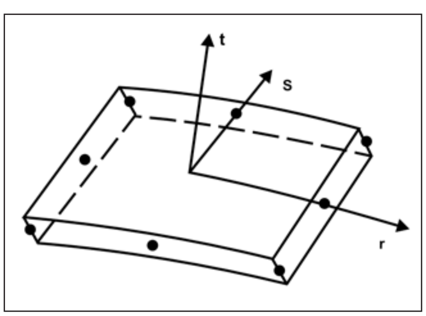


Figure 6 - Eight-nodded isoparametric quadrilateral element for thick and thin shells.

criterion is assumed in the FE analyses. The mesh division of the model is fine enough to capture the buckling and ultimate strengths of the stiffened panels. However, finer meshes are used near welded parts of the FE model. A typical FE model with the mesh divisions is shown in Figure 9. Table 1 demonstrates the state of the boundary conditions of the model.

4.2.2 Stiffened plate under transverse in plane compression

Figure 10 shows a typical stiffened panel with a transverse framing system, which is applied in the most inland waterways vessels. The FE model is shown in that figure by the shaded area. Boundary conditions imposed on the model are described in Table 2. A typical FE model with the mesh divisions is shown in Figure 11.

4.3 Initial geometric imperfections

In the ultimate strength calculations, the collapse load can significantly depend on shape and the amplitude of initial geometric imperfections as a result of post-welding distortion. In this study both the amplitude and geometric shape of the post-welding distortions including buckling deformation caused by residual stresses of the fillet welds due to fabrication process as an initial imperfection on the stiffened plate are taken in to consideration. However, this assumption for intermittent fillet welds leads to somewhat a conservative design, as this type of welds produce much smaller residual compressive stresses in plate panel than continuous fillet welds. In the ultimate strength calculations, the collapse load can significantly depend on initial geometric imperfections. Thus, an eigen-value analysis is carried out by ADINA. For this purpose, a linearised elastic buckling analysis is performed first, and then superimposed on the geometry of the FE model, as a coordinate perturbation proportional to the buckling mode shapes. Finally, the collapse analysis is effectively achieved by using the LDC solution scheme on the model of the geometrically imperfect stiffened panel.

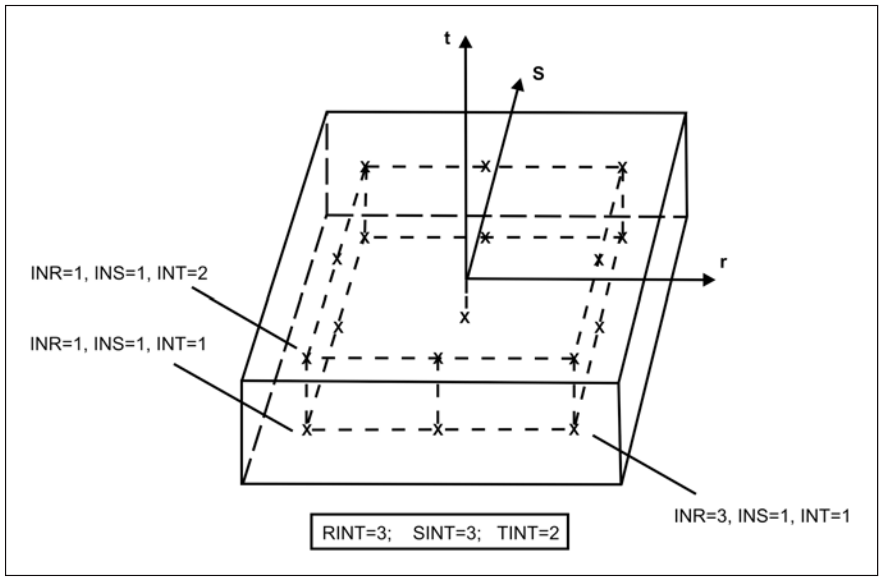


Figure 7 - Example of integration point labelling for rectangular shell element.

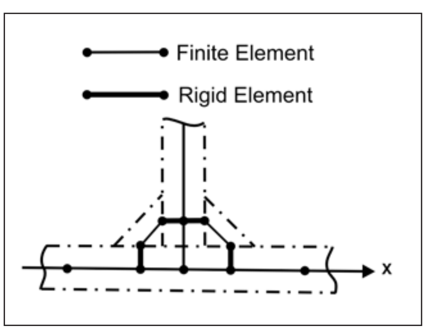


Figure 8 - Rigid elements at the plate-to-stiffener junction.

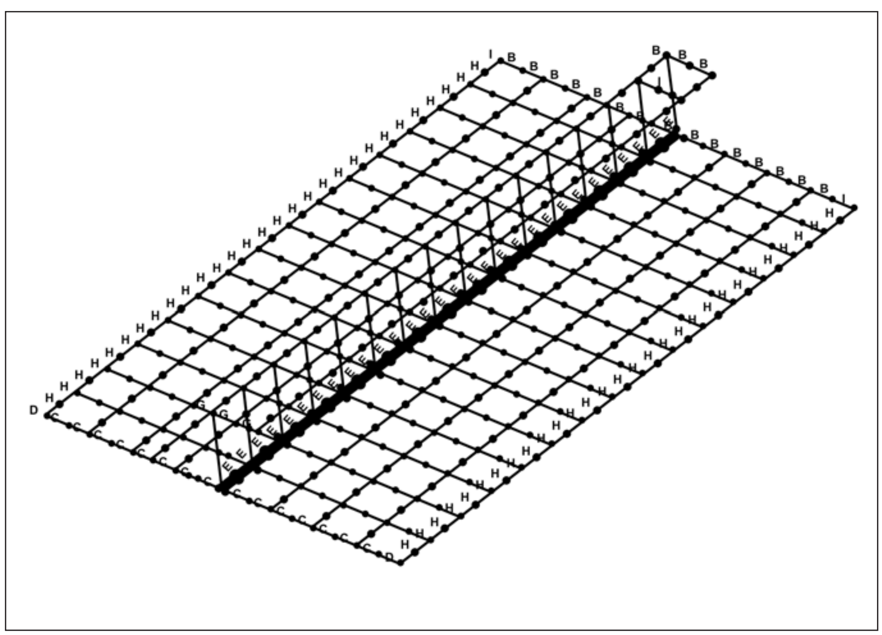


Figure 9 - Typical FE model with the mesh divisions for a longitudinally stiffened panel.

5. Important definitions

Buckling strength: the smallest load at which the equilibrium path fails to be stable as the load is slowly increased from zero value. This is commonly known as a critical load.

Ultimate strength: maximum load sustained by the stiffened panel, over which the collapse of the structure appears.

Longitudinal ductility: the ratio of the axial shortening at the ultimate strength state, δ_p, to the elastic shortening of the panel, δ_e; i.e. δ_p / δ_e. This quantity is the basis for comparison of the post-yield characteristics of stiffened panels.

6. Validation of the FEA code

In order to check the validity of the FEA code in use, some of the tests performed by Horne and Narayanan [18-19] are simulated using the ADINA code. Table 3 gives a summary of the test specimen geometries, test results and FE results. More details on the test specimens and procedure can be found in Refs. [18-19]. As it can be seen from Table 3, a good correlation exists among the FEA and test results.

Table 1 - Boundary conditions of the FE model of longitudinally stiffened panel.

Location of nodes	Boundary conditions
Along the longitudinal unloaded edge $x = 0$	$\theta_y = 0$, U_x = Coupled
Along the longitudinal unloaded edge $x = b$	$\theta_y = 0$, U_x = Coupled
Along the transverse edge $y = 0$	$\theta_x = \theta_z = 0$, $U_y = 0$; for panels which natural buckling mode is odd numbers of half waves
	$\theta_z = 0$, $U_y = U_z = 0$; for panels which natural buckling mode is even numbers of half waves
Along the transverse edge $y = a/2$	$\theta_x = 0$ (Just for clamped case), $U_z = 0$
Along the transverse edge $y = a/2$	ΔU_y = 0.0125mm As the first prescribed displacement, The code traces automatically the nonlinear response of the model up to its collapse

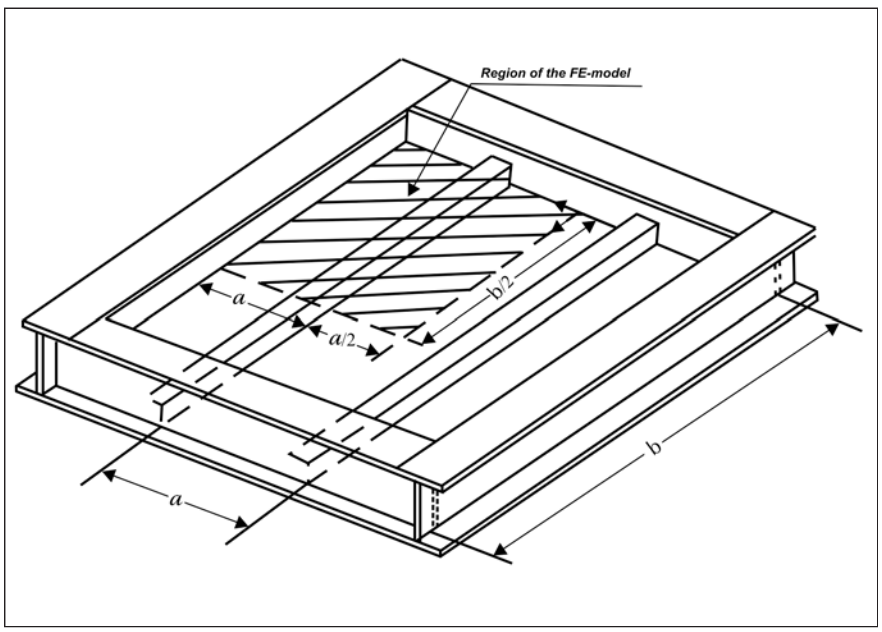


Figure 10 - Typical transversely stiffened panel and the extent of the FE model.

7. Parametric study on stiffened plates under longitudinal in plane compression

7.1 Parameters considered

A number of stiffened plates, the dimensions of which are given in Table 4, was analysed using FEM. Fourteen stiffened panels with various practical characteristics were chosen from the structures of real sea-going ships either designed according to or approved by PRS requirements [20]. Three types of welding were assumed along the plate-to-stiffener junction lines: double continuous, chain and staggered intermittent fillet welds. Both plate and stiffener materials are assumed to be of normal strength structural steel (NS), with a nominal yield strength of $\sigma_y = \sigma_0 = 235$ *MPa* and modulus of elasticity taken as $E = 2.06 \times 10^5$ *MPa*.

The size and configuration of fillet welds have been designed in accordance with the PRS rules [20]. Each stiffened plate has an initial deflection mode whose shape is proportional to the linearised elastic buckling mode, obtained through an eigen-value analysis separately. The maximum amplitude of initial deflection of the plate is taken, in accordance with the PRS rules [20], to be equal to 0.5*t*. The last character in the "Panel No." indicates the type of welding, i.e. C, S and P respectively correspond to the continuous, staggered and chain intermittent fillet welds.

7.2 Results

The numerical results obtained from the analysis of six groups of stiffened plates are given in Table 4. The spread of plasticity and the deflected mode at ultimate strength for the models EX1C, EX1S and EX1P have been shown, as an example, in Table 5. Figure 12 shows the average stress-average strain relationship of these models in comparison with each other.

Table 2 - Boundary conditions of the FE model of transversely stiffened panel.

Location of nodes	Boundary conditions
Along the unloaded edge $y = 0$	$U_z = 0$, $U_x = 0$, U_y = Couplet
Along the symmetric edge $x = 0$	$U_x = \theta_y = \theta_z = 0$
Along the symmetric edge $y = b / 2$	$U_y = \theta_x = \theta_z = 0$
Along the loaded edge $x = 3a / 2$	$U_z = 0$
Along the loaded edge $x = 3a / 2$	ΔU_x = 0.02mm As the first prescribed displacement. The code traces automatically the nonlinear response of the model up to its collapse

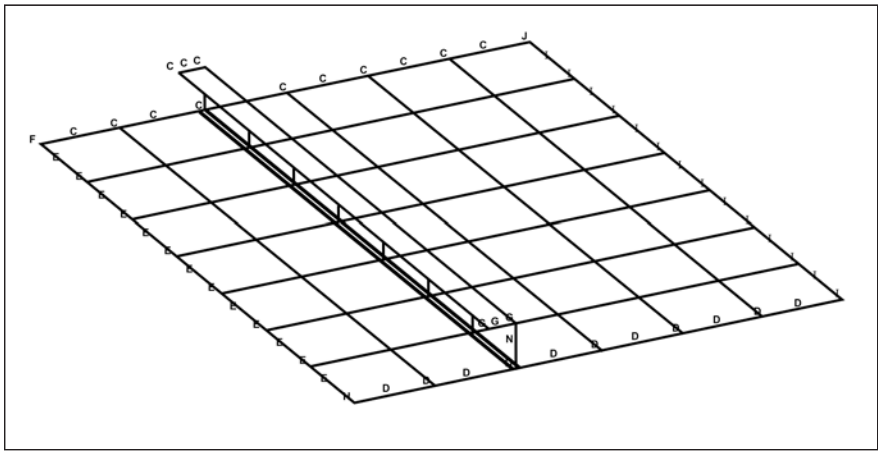


Figure 11 - Typical FE model with the mesh divisions for a transversely stiffened panel.

Table 3 - Comparison of experimental and FEA results.

Specimen	Panel Characteristics							Welding type	$\frac{\sigma_{Exp.}}{\sigma_{Yp}}$	$\frac{\sigma_{FEA}}{\sigma_{Yp}}$	Error
No.	$\frac{l}{r}$	$\frac{b}{t}$	*t* [mm]	*l* [mm]	*s* [mm]	Stiff. Size [mm]	Geo. Cond.				%
1	19	48	9.5	915	457	152.5x16	Dished Plate	Continuous	0.87	0.89	+2.3
2	19	48	9.5	915	457	152.5x16	Dished Plate	Intermittent	0.83	0.85	+2.4
4	19	48	9.5	915	457	152.5x16	Straight Plate	Intermittent	0.90	0.95	+5.6
8	38	48	9.5	1830	457	152.5x16	Straight Plate	Intermittent	0.85	0.90	+5.9
13	38	48	9.5	1830	457	152.5x16	Dished Plate	Intermittent	0.75	0.79	+5.3
14	38	48	9.5	1830	457	152.5x16	Dished Plate	Continuous	0.83	0.81	-2.5

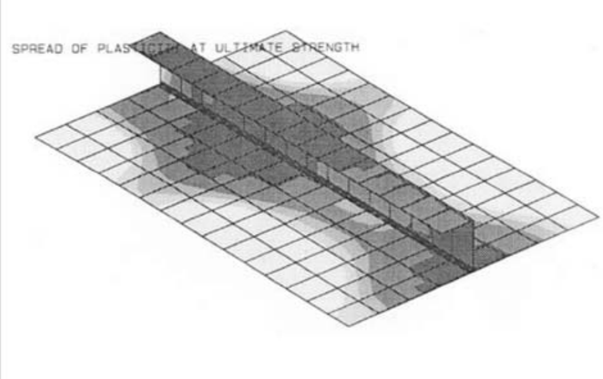
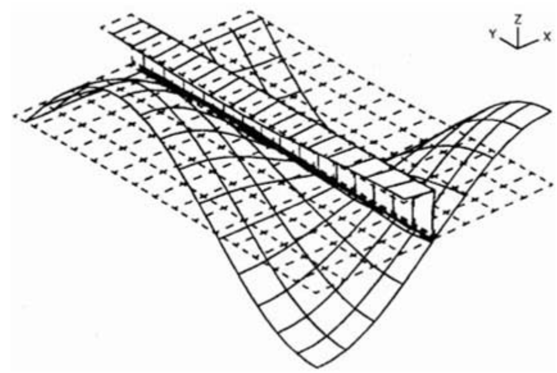
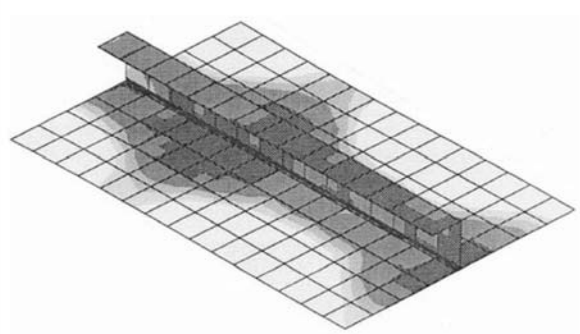
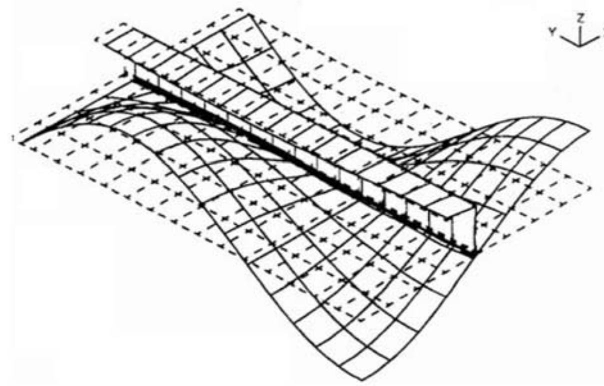
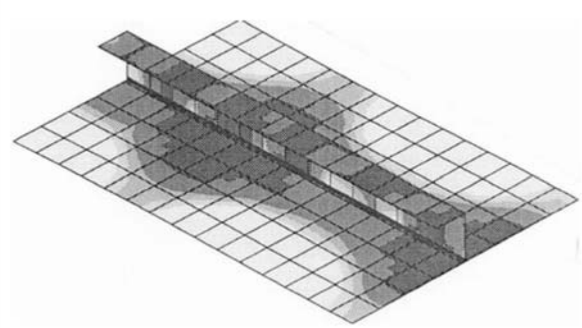
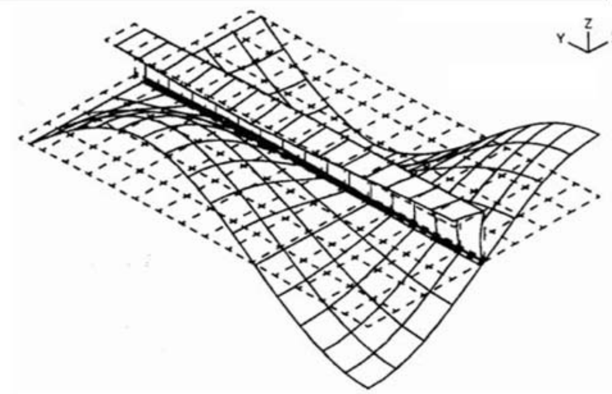
l - Plate length *s* - Stiffener spacing *b* - Plate breadth *r* - radius of gyration of stiffener
t - Plate thickness σ_{yp} - Yield stress of plate material

Table 4 - Geometric characteristics and summary of results obtained through FEA for longitudinally stiffened panels.

1	2	3	4	5	6	7	8	9	10	11	12
Group No.	Panel No.	$l=a$ [mm] / b [mm] / t [mm]	a/b	b/t	l/r	λ	Stiffener size, mm	Weld type	$\frac{\sigma_{ult}}{\sigma_{Yp}} = \frac{\sigma_m}{\sigma_{Yp}}$	Ultimate Strength Cont./Int.	$\frac{\delta_p}{\delta_e}$
1	EX1C	2250	3	62.5	46	0.49	Angle 150x100x10	Cont.	0.838	-	1.92
	EX1S	750						Stagg.	0.823	1.018	1.82
	EX1P	12						Chain	0.822	1.020	1.76
	EX4C	2250	3	62.5	49	0.52	Bulb 160x38x10	Cont.	0.838	-	2.00
	EX4S	750						Stagg.	0.822	1.020	1.92
	EX4P	12						Chain	0.802	1.045	2.50
	EX5C	2250	3	62.5	47	0.51	Flat 200x10	Cont.	0.786	-	1.47
	EX5S	750						Stagg.	0.776	1.013	1.45
	EX5P	12						Chain	0.776	1.013	1.42
	EX9C	2250	3	62.5	27.5	0.3	Bulb 230x60x12	Cont.	0.807	-	1.47
	EX9S	750						Stagg.	0.796	1.014	1.72
	EX9P	12						Chain	0.770	1.048	1.28
2	EX2C	3000	4	62.5	61	0.65	Angle 150x100x10	Cont.	0.810	-	1.82
	EX2S	750						Stagg.	0.795	1.019	1.65
	EX2P	12						Chain	0.776	1.044	2.00
	EX3C	3000	4	62.5	36.5	0.39	Bulb 230x60x12	Cont.	0.822	-	1.76
	EX3S	750						Stagg.	0.808	1.017	1.80
	EX3P	12						Chain	0.782	1.051	1.90
3	EX14C	1500	2	47	20	0.22	Bulb 230x58x10	Cont.	0.821	-	2.10
	EX14S	750						Stagg.	0.817	1.005	2.00
	EX14P	16						Chain	0.810	1.014	1.93
	EX16C	1500	2	47	35	0.38	Bulb 160x38x10	Cont.	0.820	-	2.10
	EX16S	750						Stagg.	0.812	1.010	2.00
	EX16P	16						Chain	0.807	1.016	1.98
4	EX11C	2250	3	47	30	0.32	Bulb 230x58x10	Cont.	0.875	-	1.78
	EX11S	750						Stagg.	0.864	1.013	1.81
	EX11P	16						Chain	0.853	1.026	2.19
	EX15C	2250	3	47	53	0.57	Bulb 160x38x10	Cont.	0.872	-	1.73
	EX15S	750						Stagg.	0.861	1.013	1.92
	EX15P	16						Chain	0.849	1.027	2.35
5	EX10C	3000	4	47	40	0.43	Bulb 230x58x10	Cont.	0.853	-	2.10
	EX10S	750						Stagg.	0.833	1.020	1.68
	EX10P	16						Chain	0.821	1.035	1.67
	EX17C	3000	4	47	71	0.76	Bulb 160x38x10	Cont.	0.845	-	1.56
	EX17S	750						Stagg.	0.827	1.022	1.48
	EX17P	16						Chain	0.815	1.037	1.62
6	EX12C	1100	1.6	56	23	0.25	Bulb 160x38x10	Cont.	0.801	-	2.00
	EX12S	670						Stagg.	0.789	1.015	1.86
	EX12P	12						Chain	0.784	1.022	1.80
	EX13C	1100	1.6	42	25	0.27	Bulb 160x38x10	Cont.	0.824	-	2.20
	EX13S	670						Stagg.	0.816	1.010	2.16
	EX13P	16						Chain	0.812	1.015	2.00

$$\lambda = \frac{1}{\pi r}\sqrt{\frac{\sigma_{Yp}}{E}}$$

Table 5 - Plotted FEA results for the models EX1C, EX1S and EX1P under longitudinal compression.

Model	Spread of plasticity at ultimate strength	Deflected mode at ultimate strength (Magnification Factor = 5)
EX1C		
EX1S		
EX1P		

7.3 Evaluation of effects of different quantities on the ultimate strength

7.3.1 Effect of *b / t*

Figure 13 represents the relationship between non-dimensional ultimate strength of the stiffened plates having a / b = 4, b / t = 47&65.2 and column slenderness, λ. As it can be observed, the variation in the ratio b / t has a negligible effect (less than 5%) on the average decrease of the strength of the stiffened plates in any type of welding. The same result is observed for the other values of b / t.

7.3.2 Effect of *a / b*

As can be seen from Figure 14, for the stiffened plates with a / b = 3, for which the elastic buckling mode coincides with the stable mode, the highest amount of ultimate strength is obtained for various types of welding in comparison with the other aspect ratios of stiffened plates (a / b = 2 & 4).

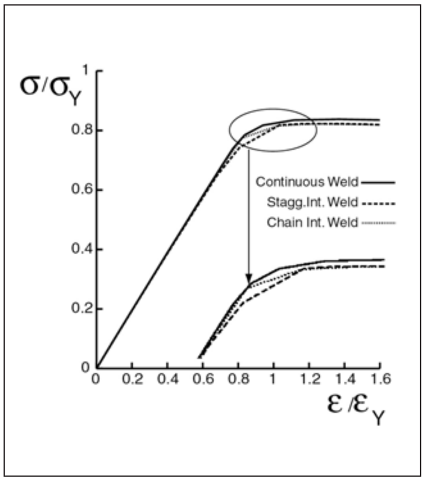


Figure 12 - Average stress-average strain relationship of the models EX1C, EX1S and EX1P under longitudinal compression.

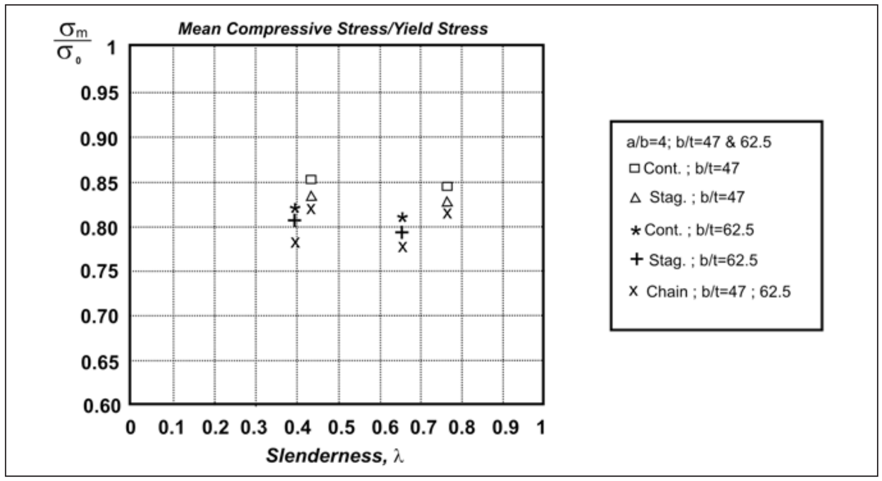


Figure 13 - Effect of *b* / *t* on the ultimate strength of the stiffened plates with various types of welding (*a* / *b* = 4, *b* / *t* = 47 & 62.5).

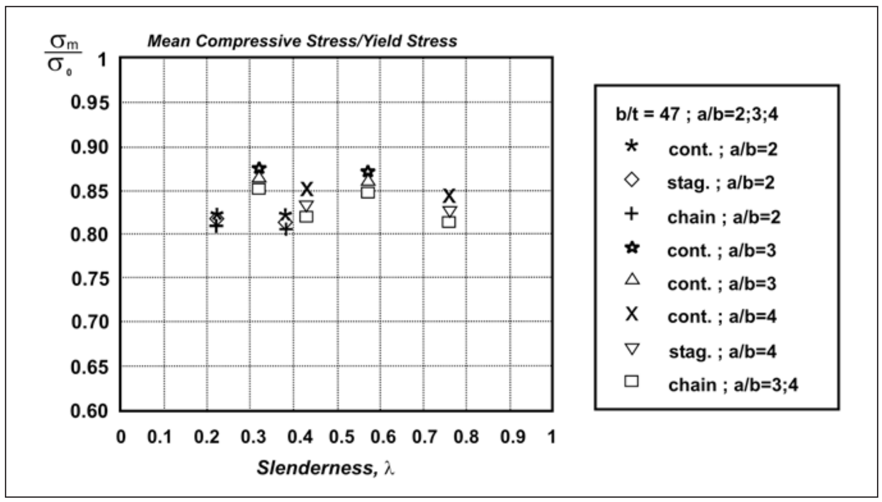


Figure 14 - Effect of *a* / *b* on the ultimate strength of the stiffened plates with various types of welding (*b* / *t* = 47, *a* / *b* = 2,3,4).

7.3.3 Effect of stiffener section

Comparison of the results in Table 4 indicates that the stiffened plate with flat bar stiffener gives a lower ultimate strength value than those panels with the angle and bulb flat stiffeners due to low torsional rigidity of this type of cross-section, for any type of welding.

7.4 Sensitivity analysis on the ultimate strength of stiffened plates

7.4.1 Ultimate strength versus column slenderness

It can be seen from Figure 15 that the difference in the ultimate strength between continuously welded plates staggered intermittently welded ones is lower (0.7-2.7%) in general, than that between continuously welded plates staggered intermittently welded ones (1.7-4.5%). Also with the increase in the values of column slenderness, the stiffened plates with chain intermittent type of welding exhibit less strength than those with staggered intermittent type of welding, both in regard to the same cases but with continuous type of welding.

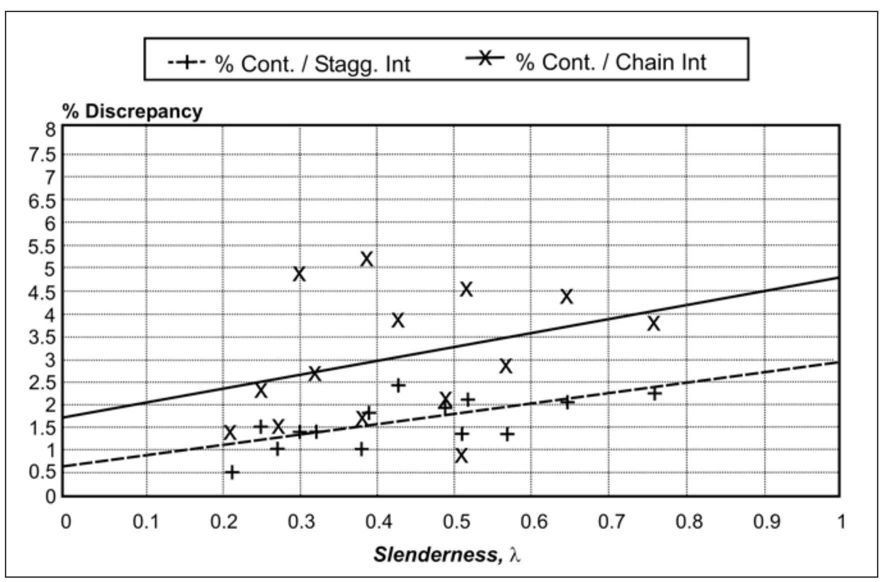


Figure 15 - Differences in the ultimate strength of the stiffened plates with various types of welding as a function of λ.

7.4.2 Ultimate strength versus plate slenderness

It is interesting to note, from Figure 16, that with the increase in the column slenderness, the ultimate strength of stiffened plates with chain intermittent type of welding drops rapidly respect to the same stiffened plates with continuous type of welding. Also it is observed that the ultimate strength does not change much as a function of column slenderness, for the cases of stiffened plates with staggered intermittent and continuous types of welding.

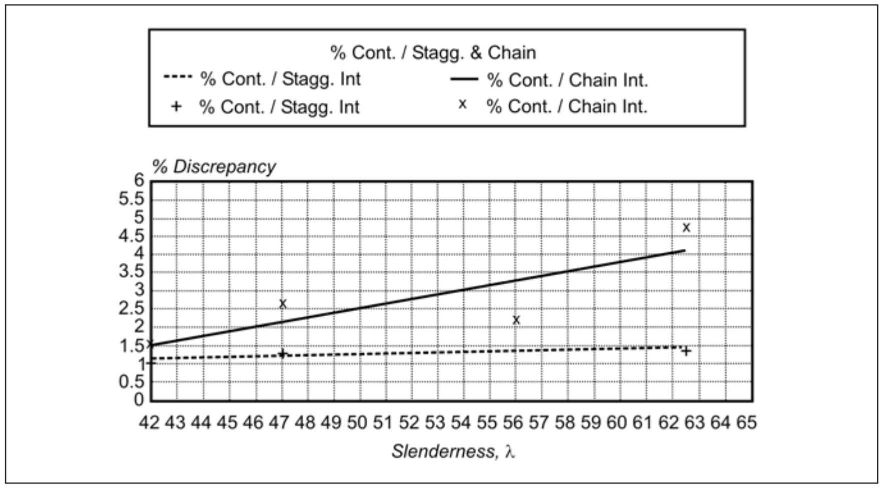


Figure 16 - Differences in the ultimate strength of the stiffened plates with various types of welding as a function of *b / t*.

7.4.3 Comparisons with PRS rules

Figure 17 demonstrates a comparison of all numerical results with strength curves proposed by PRS rules. Thick solid curve represents the Johnson-Ostenfeld parabola, which is used by PRS rules [20] as a basic criterion for the buckling control of longitudinally stiffened plates. The thin solid curve refers to the allowable buckling stress obtained by applying the factor $\eta = 0.85$ used for such stiffened plates. It seems reasonable to modify the allowable curve, as an acceptable design curve for stiffened plates with $\lambda < 0.4$, the value of the usage factor η, should be changed from 0.85 to 0.8. The dashed line in Figure 17 shows the modified design curve.

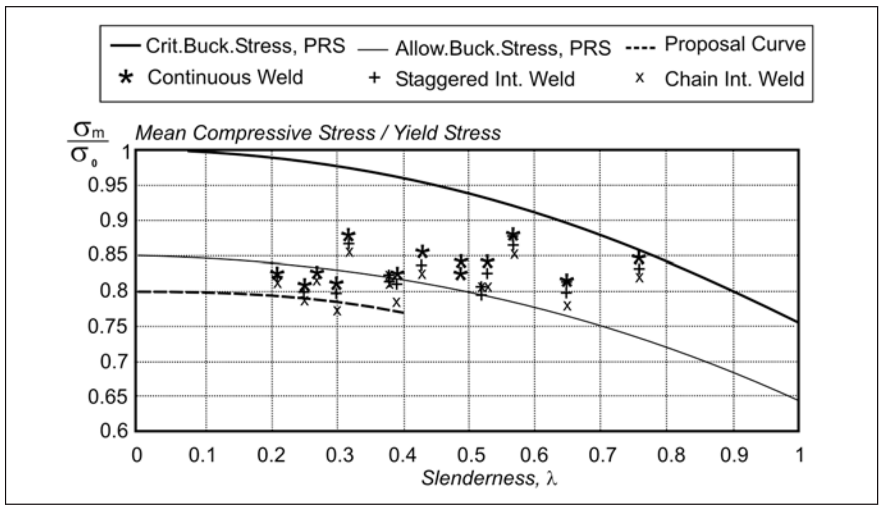


Figure 17 - Comparison of all numerical results with strength curves proposed by PRS rules.

7.5 Sensitivity analysis on the longitudinal ductility of stiffened plates

7.5.1 Effect of stiffener type

The values of longitudinal ductility for the plates stiffened with flat-bar are about 30% lower than those for the plates stiffened with bulb or angle-bar, for any type of welding, Table 4. Also this result indicates low post-yield reserve strength obtained by using flat-bar stiffeners.

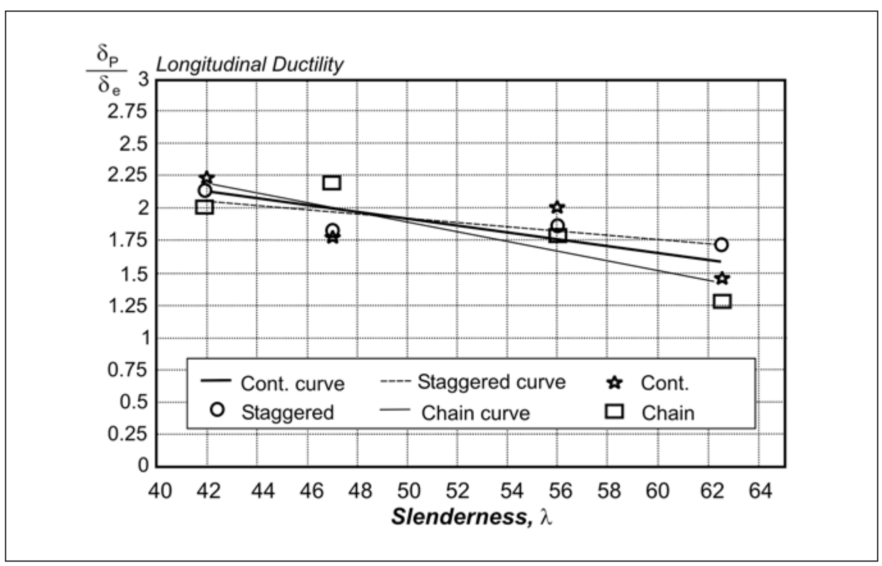


Figure 18 - Longitudinal ductility of stiffened plates with $\lambda \approx 0.3$ against the plate slenderness *b / t*.

7.5.2 Effect of *b / t*

It is observed from Figure 18 that with increase in *b*/*t*, the longitudinal ductility of the stiffened plates reduces for each type of welding. The longitudinal ductility of the stiffened plates with *b* / *t* within the range of 47-50 is almost equal for any type of welding. In regard to the stiffened plates with *b* / *t* < *47*, longitudinal ductility of the stiffened plates with chain intermittent welding is a little greater than that of corresponding plates with continuous and staggered intermittent welding. In the region *b* / *t* >*50*, the staggered intermittently welded plates have the highest ductility in comparison with the continuously and chain intermittently welded plates.

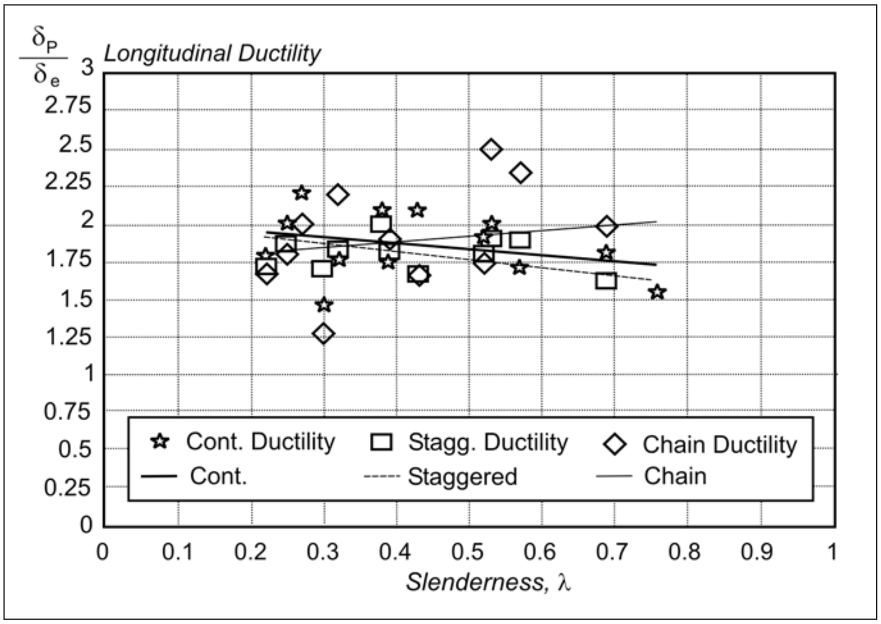


Figure 19 - Longitudinal ductility of stiffened plates with *b* / *t* = 42-62.5 against the column slenderness λ.

7.5.3 Effect of column slenderness

Analyzing the ductility curves represented in Figure 19, the following statements can be drawn:

In stiffened plates with $\lambda < 0.37$, the ductility of continuously welded plates is higher than that of the intermittently welded ones. The difference between the ductility of continuously welded stiffened plates and chain intermittently welded ones grows with decreasing the column slenderness.

Although, for any column slenderness, the ductility of continuously welded stiffened plates is greater than that of staggered intermittently welded ones, the difference between them is rather small.

In stiffened plates with $\lambda > 0.37$, the ductility of chain intermittently welded plates is higher than that of the continuously or staggered intermittently welded ones. This indicates that the chain intermittently welded slender stiffened plates provide a more post-yield reserve strength than the others.

The ductility of stiffened plates with continuous or staggered intermittent welding decreases while the column slenderness value increases, whereas in respect with the chain intermittently welded, it accompanies with an increasing tendency.

8. Parametric study on stiffened plates under transverse in plane compression

8.1 Parameters considered

In order to analyse the stiffened plates with transverse framing system subjected to in plane compression parallel to the smaller dimension of the plate field, three panels with various characteristics have been chosen as given in the Table 6. These panels were chosen from the structures of three inland waterways vessels with transverse framing system, which have been designed according to PRS requirements [21]. Other parameters considered here are the same as those mentioned in Sec. 7.1.

8.2 Results

The numerical results obtained from the analysis of three groups of stiffened plates are given in columns 8 and 9 of Table 6. The spread of plasticity and the deflected mode at ultimate strength for the models T3C, T3S and T3P have been shown, as an example, in Table 7. Figure 20 shows the average stress-average strain relationship of these models in comparison with each other.

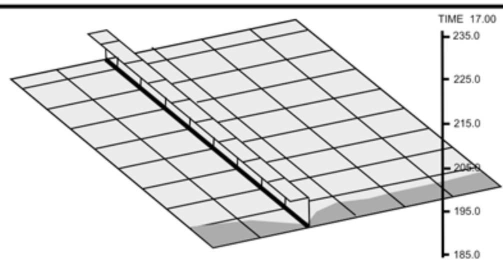
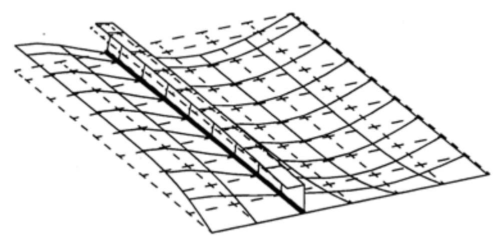
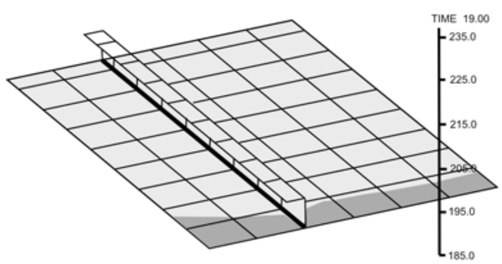
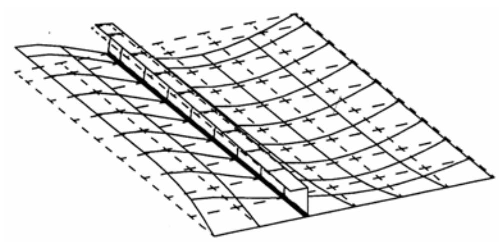
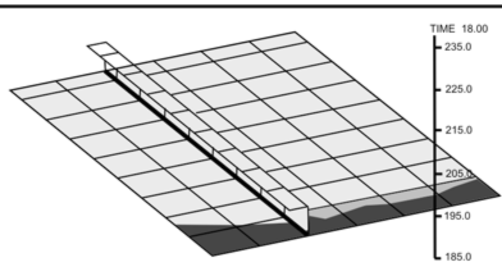
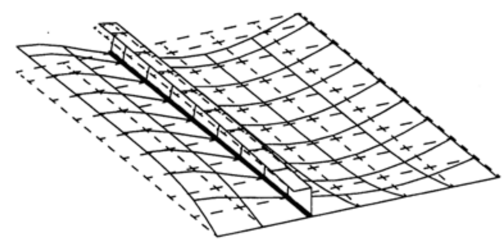
8.3 Dominant parameters on the ultimate strength

In transversely stiffened panels, the stiffener is applied in order to force a nodal line for the buckled state of the panel, and thus to prohibit the overall buckling of the stiffened panel. Since the aspect ratio of the panels in such a case is usually less than 1.0, the primary buckling mode consists of a single half-wave pattern. The middle portion of the plate receiving little support from the stiffened side buckles like a wide column, and the ultimate strength of such a panel may be estimated according to the ultimate strength of a wide free plate. The central part of the plate is similar to an eccentrically loaded wide column (b→∞), and with increasing load the stress in this part may be estimated according to the ultimate strength of a wide column. As the load is further increased, this part becomes wider, but the ultimate strength level remains constant. So only the stresses in the side

Table 6 - Geometric characteristics and summary of results obtained through FEA for transversely stiffened panels.

1	2	3	4	5	6	7	8	9	10	11
Group No.	Panel No.	$l=a$ [mm] / b [mm] / t [mm]	a/b	b/t	Stiffener size, mm	Weld type	Stiffener ends are <u>constrained</u> by welding		Stiffener ends are <u>not constrained</u> by welding	
							$\frac{\sigma_{ult}}{\sigma_{Yp}}=\frac{\sigma_m}{\sigma_{Yp}}$	$\frac{\delta_p}{\delta_e}$	$\frac{\sigma_{ult}}{\sigma_{Yp}}=\frac{\sigma_m}{\sigma_{Yp}}$	$\frac{\delta_p}{\delta_e}$
1	T1C	500	0.28	225	Angle 75x50x6	Cont.	0.528	1.21	0.575	1.5
	T1S	1800				Stagg.	0.522	1.18	0.589	1.4
	T1P	8				Chain	0.522	1.15	0.574	1.4
2	T2C	550	0.26	300	Angle 75x50x8	Cont.	0.558	1.31	0.567	2.1
	T2S	2100				Stagg.	0.546	1.16	0.586	1.3
	T2P	7				Chain	0.551	1.12	0.569	2.0
3	T3C	600	0.24	319	Angle 90x60x8	Cont.	0.564	1.31	0.577	1.9
	T3S	2550				Stagg.	0.576	1.17	0.585	1.3
	T3P	8				Chain	0.561	1.21	0.588	1.4

Table 7 - Plotted FEA results for the models T3C, T3S and T3P under transverse compression.

Model	Spread of plasticity at ultimate strength	Deflected mode at ultimate strength (Magnification Factor = 5)
T3C		
T3S		
T3P		

strips continue to increase and eventually these strips undergo inelastic buckling resulting in the panel collapse.

Figure 21 shows that with decreasing the aspect ratio *a*/*b* of the panel, the ultimate strength of transversely stiffened plates will decrease regardless of the type of welding. Also it can be observed from this figure that the method of welding does not affect significantly on the ultimate strength of transversely stiffened panels.

8.4 Ductility

The calculated ductility in all of the transversely stiffened panels with any type of welding given column 9 of Table 6 shows an average value of the order of 1.2. This indicates a small post-yield reserve of the strength in comparison with the longitudinally stiffened panels.

8.5 Effects of boundary conditions

The same analysis with different boundary conditions of stiffeners was performed for all stiffened plates. It was assumed that the stiffeners are free at the ends and not restrained by welding connection, Figure 22. The results (columns 10 and 11 of the table 6) show an average increase of about 6% in the values of the ultimate strength, as compared with the corresponding panels with stiffeners restrained at their ends. This comparison in the case of the

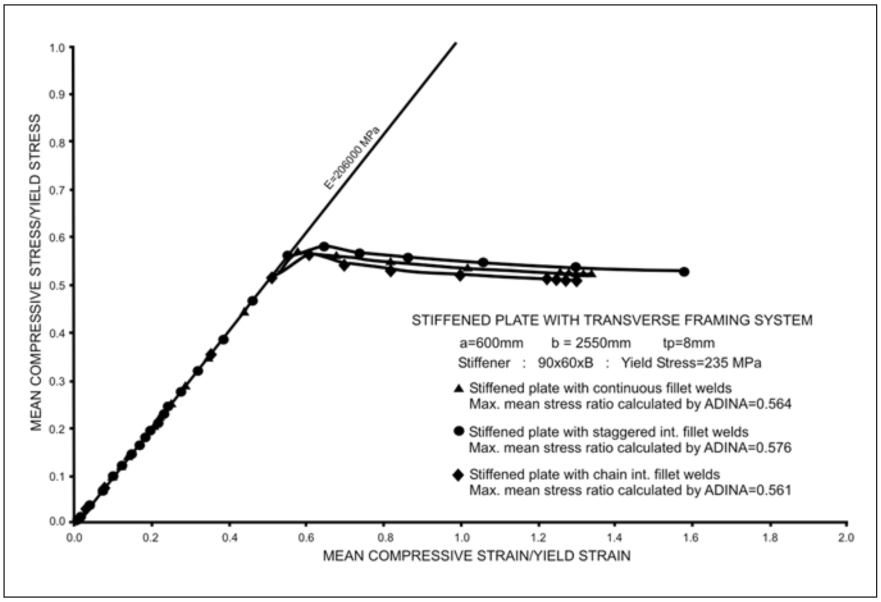


Figure 20 - Average stress-average strain relationship of the models T3C, T3S and T3P under transverse compression.

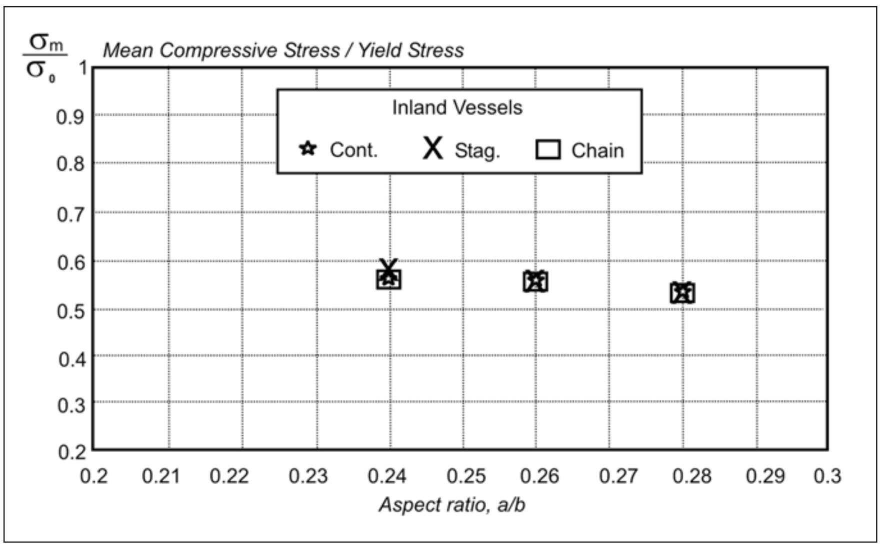


Figure 21 - Effect of *a* / *b* on the ultimate strength of the transversely stiffened plates with various types of welding.

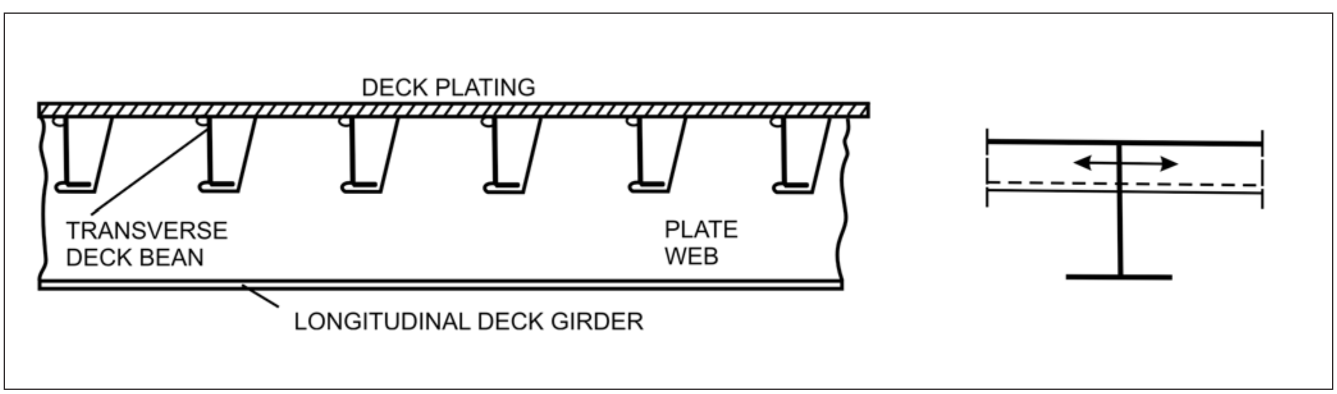


Figure 22 - Stiffeners with free ends at the transversely stiffened plates.

ductility values, exhibit a significant increase about 30%.

When the ends of the stiffeners are considered as either free or constrained, their fundamental behaviours are almost the same, but the component modes are different. The interaction between the component modes may accelerate or decelerate the plastification and this greatly influences the resulting ultimate strength and ductility values of the stiffened plates. Thus, apart from the practical processes and economic parameters, more ultimate capacity and ductility in case of transversely stiffened panels of inland waterways vessels, can be attained when the ends of the stiffeners are free. This can be achieved by passing them through the longitudinal girders without welding, as depicted in Figure 22.

8.6 Comparisons with the rules

In accordance with the requirements of the PRS rules [21], the extreme stresses in bottom and deck plating of the ship hull girder under overall bending should not exceed $0.6\sigma_{YP}$. From the calculated ultimate strength values (Table 6), it is noted that for any type of welding, the ratio σ_{ult}/σ_{YP} ranges from 0.5 to 0.6 and well below the permissible stress limit stated above. This leads to the conclusion that the permissible stress limit as proposed by PRS for transversely stiffened panels is slightly on the unsafe side and it seems that the limit of $0.5\ \sigma_{YP}$ would provide a more adequate margin of safety. This needs a further study.

9. Conclusions

A series of detailed numerical analyses of stiffened steel plates subjected to inplane compressive load were performed using ADINA commercial finite element code. Complete equilibrium paths were traced up to collapse for nonlinear elastoplastic response of stiffened plates. Analyzed stiffened plates were imperfect and their aspect ratio, plate slenderness and column slenderness are changed in a systematic manner. Different types of stiffener were chosen for stiffened plate models, selected from the deck structure of real sea-going ships and inland waterways vessels. Three different stiffener-to-plate welding procedures were considered: continuous, chain intermittent and staggered intermittent fillet welding.

It was found that, in comparison with continuously welded stiffened plates, the ultimate strength is much more reduced in case of chain intermittent fillet welds than the case of staggered intermittent fillet welds. Also the longitudinal ductility of stiffened plate with continuous and staggered intermittent welding is decreasing while the column slenderness increases, whereas in regard to stiffened plate with chain intermittent welding, the longitudinal ductility is increasing.

Study of the strength and ductility characteristics of stiffened plates constructed by different welding practices, under other in plane or out of plane loads, alone or in combination, may be found useful in safety and reliability assessment of this-walled structures. These remain as future works.

10. References

[1] SHERBOURNE, A.N., LIAW, C.Y., MARSH, C. Stiffened plates in uniaxial compression. ***IABSE***, 31, p.145, 1971.

[2] CARLSEN, C.A. Simplified collapse analysis of stiffened plates. ***Norwegian Maritime Research***, 4. p. 20-36, 1977.

[3] MOOLANI, F.M., DOWLING, P.J. Ultimate load behaviour of stiffened plates in compression. ***Steel Plates Structures, Crosby Lockwood Staples, London***, p. 51-88, 1977.

[4] GUEDES SOARES, C., SOREIDE, TH. Behaviour and design of stiffened plates under predominantly compressive loads. ***International Shipbuilding Progress***, p. 341, 1983.

[5] CHAPMAN, J.C., SMITH, C.S., DAVIDSON, P.C., DOWLING, P.J. Recent developments in the design of stiffened plate structures, ***Trans. RINA***, 1989.

[6] BONELLO, M.A., CHRYSSANTHOPOULOS, M.K., DOWLING, P.J. Ultimate strength design of stiffened plates under axial compression and bending. ***Marine Structures***, 6, p. 533-52, 1993.

[7] CHEN, Q., ZIMMERMAN, T.J.E., DeGEER, D., KENNEDY, B.W. Strength and stability testing of stiffened plate components. ***Ship Structural Committee Report SSC-399***, 1997.

[8] GRONDIN, G.Y., ELWI, A.E., CHENG, J.J.R. Buckling of stiffened plates- a parametric study. ***Journal of Constructional Steel Research***, 50, p.151-175, 1999.

[9] GHAVAMI, K. Experimental study of stiffened plates in compression up to collapse. ***Journal of Constructional Steel Research***, v. 28, n. 2, p. 197-222, 1994. (Special Brazilian Issue, Guest Editor Khosrow Ghavami).

[10] TANAKA, Y., ENDO, H. Ultimate strength of stiffened plates with their stiffeners locally buckled in compression. ***Journal of the Society of Naval Architects of Japan***, 1988; 164, (in Japanese).

[11] MILLER, M., NADEAU, J., WHITE, G.J. Longitudinally stiffened panels - a comparative analysis of the compressive strength of three common construction methods, Proceedings of SNAME Annual Meeting, 1999.

[12] HUGHES, O.F. (2. nd. ed.) Ship Structural Design, SNAME, Jersey City, NJ, 1988.

[13] ABS. Rules for Building and Classing Steel Vessels for Service on Rivers and Intracoastal Waterways, 2005.

[14] ADINA, Version 6.1.4, ADINA R & D, Inc., USA, 1992.

[15] BATHE, K.J., DVORKIN, E.N. On the automatic solution of nonlinear finite element equations. ***Computers and Structures***, v. 17, n.5-6, p.871-879. 1983.

[16] DVORKIN, E.N., BATHE, K.J. A continuum mechanics based four-node shell element for general nonlinear analysis. ***Engineering Computations***, p. 77-88. 1984.

[17] BATHE, K.J., DVORKIN, E.N. A formulation of general shell elements- the use of mixed interpolation of torsional components. ***Int. J. Num. Meth. In Eng.***, 22, p. 697-722, 1986.

[18] HORNE, M.R., NARAYANAN, R., Ultimate capacity of longitudinally stiffened plates used in box girders, ***Proc. Inst. Civ. Engrs***, v. 61, n. 2, p.253-280, 1976.

[19] HORNE, M.R., NARAYANAN, R., Influence on strength of compression panels of stiffener section, spacing and welded connection. ***Proc. Inst. Civ. Engrs***, v. 63, n. 2, p.1-20. 1977.

[20] POLISH REGISTER OF SHIPPING. Rules for classification and construction of sea-going ships. ***Pt.II***, Hull, 2002.

[21] POLISH REGISTER OF SHIPPING. Rules for classification and construction of inland waterways vessels. ***Pt.II***, Hull, 2002.

Artigo recebido em 07/12/2006 e aprovado em 07/12/2006.