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Design Methods for Stiffened Plates Under Predominantly Uniaxial Compression

C. Guedes Soares & J. M. Gordo

Unit of Marine Technology and Engineering, Instituto Superior Técnico, Universidade Técnica de Lisboa, Av. Rovisco Pais, 1096 Lisboa, Portugal

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ABSTRACT

The performance of three methods to design stiffened panels under predominantly in-plane uniaxial compressive loading is compared by referring to numerical and experimental results. The prediction of the tripping of the stiffeners is also included in the design method in order to complement its scope of application. Experiments on stiffened plates under compression and lateral pressure are also compared with the methods of strength prediction. A method is proposed in order to decrease the bias and uncertainty as results from the data set considered. © 1998 Elsevier Science Ltd.

NOTATION

A	= total sectional area ($=A_s+A_p$)
A_e	= effective sectional area of the plate-stiffener ($=A_s+b_e{\cdot}t$)
A_p	= sectional area of the associated plate ($=b{\cdot}t$)
A_s	= sectional area of the stiffener
a	= length of plate and stiffener between transverse girders
b	= plate breadth between stiffeners
b_e, b'_e	= effective and tangent or reduced width of plate
E	= Young modulus of elasticity
E_t	= tangent modulus of elasticity
e	= shift of neutral axis of plate-stiffener
h	= height of stiffener web
I	= moment of inertia of the stiffener with associate plate
I_e	= moment of inertia of effective plate and stiffener
J_e	= St. Venant torsional constant ($=(d_w t_w^3 + b_f t_f^3)/3$)
l	= span of the stiffener

Q_L = lateral pressure [MPa]
R_r, R_y, R_τ = reduction factors to account the effect of residual stresses, biaxial compression and shear stresses on plate strength
r = radius of gyration of the effective sectional area (stiffener and associated effective plating)
t = plate thickness
t_w = web thickness
W = section modulus of plate-stiffener
z_p = distance from the neutral axis to the midplane of the plate
z_s = distance from neutral axis to the extreme fibre of the stiffener
β = plate slenderness $\left(=\frac{b}{t}\sqrt{\frac{\sigma_0}{E}}\right)$
δ_0 = initial deflection amplitude in the plate
η = breadth of tension zone of welding residual stresses
γ = magnification factor in Perry-Robertson formula
λ = column slenderness $\left(\frac{l}{r}\sqrt{\frac{\sigma_0}{E}}\right)$
ϕ = efficiency factor $(=\sigma/\sigma_0)$; non-dimensional strength of a plate
ϕ_u = ultimate strength of the stiffened plate
ϕ_{ua} = ultimate strength of stiffener with associated effective plating
ϕ_{up} = effective width ratio, ultimate strength of the plate
σ_b, σ_{bu} = mean and ultimate bending stresses
σ_{cr} = critical buckling stress
σ_e = Euler stress $(=\pi^2 Er^2/a^2)$
σ_{JO} = Johnson-Ostenfeld stress $(=(1-\sigma_0/4\sigma_e)\cdot\sigma_0)$
σ_r = mean compressive residual welding stresses in the plate
σ_T = torsional buckling stress in x and y direction
σ_u = ultimate compressive stresses
σ_{uc} = ultimate compressive stress of the stiffener with associated effective plating
σ_x, σ_y = mean axial stress in x and y direction
σ_{xu}, σ_{yu} = mean ultimate stress in x and y direction
σ_0 = yield stress
τ, τ_u = mean and ultimate shear stress
ξ = imperfection factor in Perry-Robertson formula
Γ = longitudinal warping constant $(=(4d_w^3t_w^3+b_f^3t_f^3)/144)$.

1 INTRODUCTION

The description of the behaviour of stiffened plates under predominantly compressive loads is relatively complicated due to the large number of possible combinations of plate and stiffener geometry, boundary conditions and loading. It is possible to carry out accurate predictions of collapse load for any type of stiffened plate configuration using one of the finite element formulations available. More simplified formulations have also been developed, based on a beam-column concept, which have been used for strength assessment but most frequently for design purposes. The main feature of this type of approach is that one isolated stiffener with an associated width of plating is considered as representative of the whole panel behaviour.

Some of the approaches solve the equilibrium equations of the beam-column in an iterative way by accounting for the decreased contribution of the plate flange in the post-buckling region [1–3]. This is normally accounted for by using load-shortening curves for plate behaviour produced by finite difference or finite element methods [4–9]. Others, while using similar load-shortening curves to describe the plate contribution, use a finite-element formulation for the beam-column behaviour [10,11].

These theoretical studies together with various experimental programmes[11–14] have provided the understanding about the main features of stiffened plate behaviour which will be briefly reviewed here. The understanding of these aspects is a basic starting point for the development of design equations.

As categorised in Smith *et al.*[15] and Guedes Soares and Soreide,[16] one can consider three main types of collapse, namely plate collapse, interframe flexural buckling and overall grillage collapse, in addition to the tripping collapse of the stiffener.

The response of a short stiffened panel, with a length approximately equal to the width of the plate between stiffeners, is dominated by the plate failure. In ships, the panels are generally much longer than the stiffener spacing and therefore the possibility of having plate failure only exists in special cases, such as, for example, a panel with high strength stiffeners and with relatively low strength nearly perfect plates. Under these conditions the plates show a very steep unloading characteristic and the stiffeners are not able to accommodate the drop in load due to plate failure.

An overall column type of plate failure can occur in long uniaxially stiffened panels. In orthogonal stiffened panels the corresponding mode is the grillage collapse, which involves both longitudinal and transverse stiffeners. This collapse mode can be influenced by local buckling of the plate or of the stiffener and is generally not found in ships. However, it may be relevant for lightly stiffened panels found in superstructure decks.

Some design studies led to the conclusion that the optimum design of a compressed stiffened plate would be obtained whenever the strength of the overall buckling mode equals the strength of the local buckling mode. However, such panels show an interaction between local and global modes that makes them imperfection sensitive, with a violent collapse [17]. These characteristics are undesirable from a safety point of view and therefore stiffened plates are generally designed to exhibit an interframe type of collapse.

Interframe collapse is commonly stimulated by the presence of heavy transverse girders, often the case in ships. This is a typical case of interactive collapse [18] in which the overall collapse of the beam-column is triggered by local buckling of the plate or stiffener. It is possible to have a failure towards the stiffener outstands—plate induced failure, or towards the plate—stiffener induced [19]. The local failure of the stiffener may be due to flexural buckling[20] or to torsional buckling [21].

Tripping involves a rotation of the stiffener about one hinge which is usually considered to be located on the connection of the stiffener to the plating, associated with a vertical hinge along the web of the stiffener. This mode of panel failure is one of the most dangerous ones because it is always associated with a very quick shed of load carrying capacity of the column. Lateral-torsional instability may occur alone by twisting of the stiffener about its line of attachment to the plating, developing a partial or full hinge on the intersection, or may be induced by flexural buckling, particularly if the deflected shape of the column is towards the plate. In that case, the stiffener will be subjected to a higher stress than the average column stress and the critical tripping stress could be easily reached, followed by a pronounced load shedding.

A panel will be subjected to all failure modes and it will finish up collapsing in the mode which corresponds to its lowest strength. Therefore, all modes must be accounted for by a design method.

This work was initiated with the objective of assessing the design approach included in the ABS proposal for a set of rule requirements [22]. In addition to the evaluation of the general formulation, which results from a comparison with available methods, the study aims also to propose specific improvements based on detailed comparison with numerical and experimental results. In order to better assess the performance of this design method, two other methods are also calibrated with the same data set.

Section 2 deals with the formulations for plate induced collapse while Section 3 covers the stiffener induced collapse. Section 4 deals with the assessment of the performance of the methods as results from comparisons with available data, and Section 5 analyses the limited data available and the effect of lateral pressure.

Since 1992, when this study was completed, some new proposals have been made and some additional studies have been published as discussed in Section 6 of this paper.

2 DESIGN OF STIFFENED PLATES UNDER UNIAXIAL LOAD

Several simplified design methods have been proposed. Most of these techniques account for the interaction between local and global collapse and are specially adequate for interframe collapse where these effects are more important.

The methods proposed by Faulkner *et al.* [23] and by Carlsen [24] are oriented towards marine structures and while the first one is based on a Johnson-Ostenfeld formulation, the second one builds upon a Perry-Robertson approach. Therefore, it is most interesting to analyse the background and the performance of these two methods. Other existing proposals such as Dwight and Little [25], Horne and Narayanan [26], Chatterjee and Dowling [27] and Murray [28], have been developed for steel box girders in bridges. Some of these methods have already been compared [16] with the experimental results currently available.

2.1 Johnson-Ostenfeld formulation

The method proposed by Faulkner *et al.* [23] is based on a Johnson-Ostenfeld type of formulation for elasto-plastic behaviour together with the effective width approach for a plate. When a plate or a strut has a very high elastic buckling stress it happens that the unstable failure does not occur before the development of a certain degree of plastic deformation. This phenomenon obviously changes the critical stress and an empirical way of accounting for that effect is due to Johnson and Ostenfeld. According to them, whenever the Euler buckling stress (σ_e) is higher than half the yield stress (σ_0), the critical buckling stress is given by $\sigma_{JO}=[1-(\sigma_0/4\sigma_e)]\cdot\sigma_0$, assuming that the proportional limit is $0.5\sigma_0$.

The effective width formulation is a way of expressing the diminishing of strength that a plate exhibits in the post-buckling regime. This weakening effect is expressed by a reduction of the width that effectively resists the compressive loads [29].

According to the method of Faulkner *et al.* [23], the ultimate strength of a stiffened plate modelled as a stiffener, with an associated width of plate, is given by:

$$\phi_u = \frac{\sigma_u}{\sigma_0} = \frac{\sigma_{cr}}{\sigma_0}\left[\frac{A_s b_e t}{A_s + bt}\right] \tag{1}$$

where

$$\frac{\sigma_{cr}}{\sigma_0} = \left\{ \begin{array}{ll} 1 - \dfrac{1}{4}\left(\dfrac{a}{\pi r_{ce}}\right)^2 \dfrac{\sigma_0}{E}, & \sigma_e \geq 0.5\sigma_0 \\ \dfrac{\sigma_e}{\sigma_0} = \left(\dfrac{\pi r_{ce}}{a}\right)^2 \dfrac{E}{\sigma_0} & \sigma_e < 0.5\sigma_0 \end{array} \right\} \tag{2a,b}$$

$$r_{ce}^2 = \frac{I_e'}{A_s + b_e t} \tag{2c}$$

and EI'_e is the buckling flexural rigidity of the stiffener. The tangent effective width of the plate b'_e is given by:

$$\frac{b'_e}{b} = \frac{1}{\beta}\sqrt{\frac{\sigma_0}{E}}. \tag{3}$$

The effective width of the plate is related to the slenderness as follows:

$$\frac{b_e}{b} = \frac{2}{\beta} - \frac{1}{\beta^2} \tag{4}$$

which accounts implicitly for some degree of initial deflections. The effective widths should be reduced by the factors R_r, R_y and R_τ, which represent, respectively, the effects of residual stresses, biaxial loading and shear stresses:

$$R_r = \begin{Bmatrix} 1 - \left(\dfrac{2\eta}{b/t - 2\eta}\right)\left(\dfrac{\beta^2}{2\beta - 1}\right)\dfrac{E_t}{E}, & \beta \geq 1 \\ 1 & \beta < 1 \end{Bmatrix} \tag{5a}$$

$$R_y = 1 - \left(\frac{\sigma_y}{\sigma_{yu}}\right)^2, \ \sigma \leq 0.25\sigma_0 \tag{5b}$$

$$R_\tau = \left\{1 - \left(\frac{\tau}{\tau_0}\right)^2\right\}^{\frac{1}{2}} \tag{5c}$$

where

$$\frac{E_t}{E} = \begin{Bmatrix} \dfrac{3.62\beta^2}{13.1 + 0.25\beta^4} & \text{for } 0 \leq \beta \leq 2.7 \\ 1 & \text{for } \beta > 2.7 \end{Bmatrix}. \tag{6}$$

The width η of the residual tension zones is suggested to be typically between 3 to 4.5. The method requires an iterative procedure to calculate the correct value of σ_{cr}/σ_0 but usually two to four iterations are sufficient.

Although eqn (6) was the initial proposal for the tangent modulus of elasticity [23], Guedes Soares and Faulkner [30] have indicated that the simpler expression:

$$\frac{E_t}{E} = \begin{Bmatrix} \dfrac{\beta - 1}{1.5} & \text{for } 0 \leq \beta \leq 2.5 \\ 1 & \text{for } \beta > 2.5 \end{Bmatrix} \tag{7}$$

would have an adequate accuracy.

Equation (4) accounts implicitly for some level of initial distortions while eqn (5a) corrects for some explicit level of residual stress. The problem with the widespread use of that formula is that the representative types of imperfections for a given type of structure may be different from the one that was included in the data set that was used to derive eqn (4).

This problem has been identified by Guedes Soares [31] who developed an alternative expression for the strength of perfect plates which could then be corrected explicitly for the effect both of initial distortions and of residual stresses. Guedes Soares [31] has also demonstrated how one should derive one equation of the type of eqn (4), which depends explicitly only on plate slenderness, by incorporating the effect of initial distortions and residual stresses which are representative of the type of structural under consideration.

For some applications, it may be preferable to have simpler equations that account implicitly for the effect of residual stresses and initial distortions. However, when these equations are developed they are specific of one type of structure in that the implicit levels of initial imperfection are the ones representative of these structures. The concept behind this approach, which was presented in [32,33], is to realise that the strength of a specific plate will depend on the precise imperfections that it has. Since these are unknown at the outset, but can be described in probabilistic terms, the aim is to predict the expected value of the plate strength accounting for these probability distributions.

Denoting the strength predicted explicitly as a function of slenderness, residual stresses and slenderness by $\phi_p(\beta,\eta,\delta)$ and $\phi_a(\beta)$, that predicted by the approximate expression depending only on β, they can be related by a modelling factor $B(\beta,\eta,\delta)$ given by:

$$\phi_p(\beta,\eta,\delta) = \phi_a(\beta)B(\beta,\eta,\delta). \tag{8}$$

The expected value of this modelling factor can be used to form a single equation depending only on β:

$$\phi(\beta) = \phi_a(\beta)\bar{B} \tag{9}$$

where

$$\bar{B} = \int\int\int B(\beta,\eta,\delta) f_{\beta,\eta,\delta}(\beta,\eta,\delta)\,\mathrm{d}\beta\,\mathrm{d}\eta\,\mathrm{d}\delta \tag{10}$$

and $f_{\beta,\eta,\delta}(\beta,\eta,\delta)$ is the joint probability density function of the three parameters that govern plate strength.

It was argued that this distribution could be described by the product of the three marginal distributions because they could be considered independent. Histograms were presented in [32,33] of data collected in different ships and it was shown that, while not many differences resulted among some merchant ships [33], they were significant between tankers and warships [32]. The general form of the equations obtained with that procedure were:

$$\frac{b_e}{b} = \frac{a_1}{\beta} - \frac{a_2}{\beta^2}$$

where a_1 and a_2 are coefficients that depend on the type of ship and on the level of safety to be introduced in the design equation.

2.2 Perry-Robertson formulation

Dwight and Little [25] based their formulation on the European multiple column curves which are given as a function of initial imperfections. The strength of the plate elements is predicted by curves based on a Perry-Robertson formulation. They disagree with the generally adopted formulation of effective width using instead an effective yield stress. Thus, the resulting plate strength curves give a reduced yield stress as a function of b/t. This reduced yield stress is then used together with λ for entering the column curves. The effect of simultaneous presence of shear stress is also incorporated. Another interesting aspect of the formulation of Dwight and Little is the consideration of two classes of plate curves, depending on the level of residual stresses.

Carlsen [24] based his proposal on a Perry-Robertson formulation together with an effective width approach for the plate strength. The average stress in a plate-stiffener combination is given by:

$$\phi = \frac{\sigma_u}{\sigma_0} = \frac{A_e}{A_t} \frac{(1+\gamma+\varepsilon) - \sqrt{(1+\gamma+\varepsilon)^2 - 4\gamma}}{2\gamma} \tag{11a}$$

where

$$\gamma = \frac{\sigma_0}{\sigma_e} \tag{11b}$$

$$\varepsilon = \frac{z_c \delta_0}{i_e^2} \tag{11c}$$

where z_c is the distance from the neutral axis of the effective stiffened plate to the most compressed fibre. Thus, in the case of plate induced failure this distance is z_p and in stiffener induced failure it is z_s. Due to its small influence on strength of the assembly, the plate may be considered to be fully effective when calculating σ_e and the radius of gyration i_e.

The stiffener deflection amplitude is always assumed to be $\delta_0 = 0.0015a$. For plate induced failure, account is taken for the shift of neutral axis due to loss of effectiveness of plate by modifying δ_0 as follows:

$$\delta_0 = 0.0015a + z_p\left(1 - \frac{A_e}{A_t}\right). \tag{12}$$

The effective width of plate used to calculate the effective cross sectional area A_e is given by:

$$\frac{b_e}{b} = \frac{1.8}{\beta} - \frac{0.8}{\beta^2} \text{ for } \beta > 1, \text{and } 1.0 \text{ otherwise} \tag{13}$$

$$\frac{b_e}{b} = 1.1 - 0.1\beta \text{ for } \beta > 1, \text{and } 1.0 \text{ otherwise.} \tag{14}$$

for the cases of plate induced and stiffener induced failures, respectively.

This formulation is based on initial deflections equal to 0.01b and residual stresses of $0.2\sigma_0$. To account for residual stresses in the stiffener the predicted strength is reduced by 5%.

This method is the basis of the DNV classification note [34] to predict the flexural buckling strength of continuous stiffeners with few modifications, i.e., for stiffener induced failure the yield stress is replaced by the tripping stress in very slender stiffeners and the ε parameter is increased if γ is high; for plate induced failure the shift of the neutral axis due to loss of effectivity of plate is reduced by 30% and thus the predicted strength increases in comparison with the original formulation; also a check of the tripping of the stiffener is required.

2.3 ABS method

2.3.1 General formulation

The method proposed in the Classification Note of the American Bureau of Shipping (ABS) [22] indicate that the buckling strength of longitudinals and stiffeners with associated effective plating may be treated as a beam-column between the two supporting points, i.e., transverses, girders or floors, subjected to axial compression and lateral loads.

The ultimate design limit state is determined by the expression.

$$\frac{\sigma_a}{\sigma_{ua} \cdot \frac{A_e}{A}} + m\frac{\sigma_b}{\sigma_{ub}} \leq S_m \tag{15}$$

where the stress index a, b and u denotes, respectively, axial compression, bending and ultimate.

In uniaxial compression of the panels only the first term of the first member is relevant, and so, the ultimate predicted strength according to the ABS proposed procedure will be:

$$\phi_u = \frac{\sigma_{ua}}{\sigma_0} \cdot \frac{A_e}{A} = \phi_{ua}\frac{A_e}{A} \tag{16}$$

where the variables are defined in the notation section.

2.3.2 Plate strength

The ultimate strength of the plate, σ_{up}, is determined by the Frankland equation for long plates

$$\frac{\sigma_{up}}{\sigma_0} \equiv \phi_{uFr} = \frac{2.25}{\beta} - \frac{1.25}{\beta^2}(\beta > 1) \quad (17)$$

which was also adopted by the US Navy. This expression has the same general form as the one due to Faulkner [29] and to Guedes Soares [32, 33] but the coefficients are different, leading to a more conservative prediction.

For wide plates, ABS recommends modifying the Valsgaard equation [35], by replacing Faulkner equation by Frankland equation in the first term of that expression:

$$\sigma_{up} \equiv \frac{1}{\alpha}\phi_{uFr} + 0.08\left(1 - \frac{1}{\alpha}\right)\left(1 + \frac{1}{\beta^2}\right)^2 \leq 1.0. \quad (18)$$

The ultimate strength of plate elements (eqn (17) or eqn (18)) should be always used when the applied stress exceeds the critical buckling stress. The critical buckling stress is defined elastically by the normalised Euler stress, $_e$, in conjunction with an Ostenfeld-Bleich parabola, in order to account for elasto-plastic behaviour.

Defining the Euler stress ratio by:

$$\phi_e \equiv \frac{\sigma_e}{\sigma_0} = k_i \frac{\pi^2}{12(1 - \nu^2)} \frac{1}{\beta^2} \quad (19)$$

the critical buckling stress is:

$$\left\{ \begin{array}{ll} \phi_c = \phi_e & \text{if } \phi_e \leq p_r \\ \phi_c = 1 - p_r(1 - p_r)\frac{1}{\phi_e} & \text{if } \phi_e \geq p_r \end{array} \right\}. \quad (20)$$

The recommended proportional limit is $p_r = 0.6$. The buckling coefficient, k_i, varies with the load conditions and the type of stiffeners. For long plates, k_i must be taken equal to 4.0 or 4.4 depending on the torsional rigidity of the stiffeners. Flat bar and bulb-bar are considered torsionally weak stiffeners ($k_i = 4.0$), tee and angle stiffeners are considered to be torsionally strong ones.

For wide plates, k_i is also dependent of the aspect ratio and is defined as:

$$k_i = \left(1 + \frac{1}{\alpha^2}\right)^2 \cdot c_2 \quad (21)$$

where

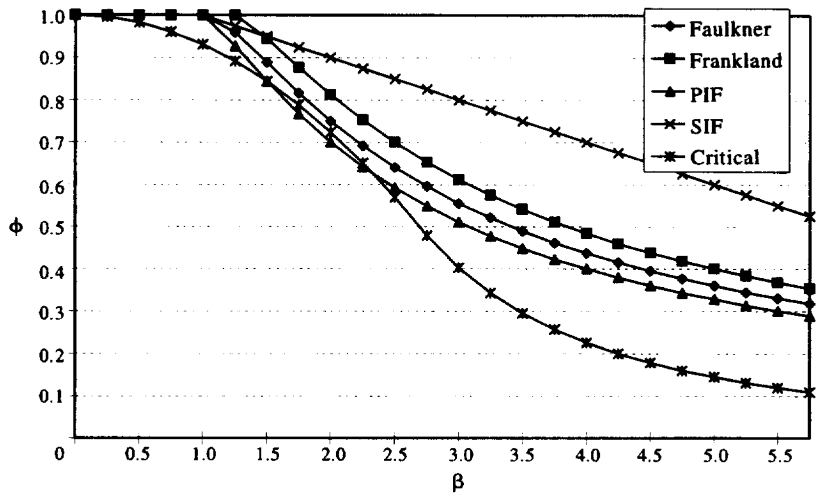

Fig. 1. Comparisons between plate strength prediction used on Faulkner method, Carlsen's method for plate induced failure (P.I.F.), stiffner induced failure (S.I.F.) and ABS proposed rules (Frankland and critical).

$c_2 = 1.1$ for flat bar and bulb-bar
$c_2 = 1.2$ for angle and the stiffeners.

However, in uniaxial tests on stiffened plates, the critical buckling of the plate elements are always exceeded and so only eqn (17) and eqn (18) must be used.

Figure 1 compares the predicted plate strength for the mentioned formulations, showing that the Faulkner formula is, approximately, the mean value of the ultimate plate strength proposal from ABS (Frankland equation) and Carlsen method for plate induced failure. The critical plate strength is well below the others in the elastic region as expected. The formula for the stiffener induced failure is not an ultimate prediction of plate strength but a prediction of the effectivity of the plate when the stiffener fails.

The large differences between these predictions and their relative position have a decisive importance in the prediction of the stiffened plate panel strength and may be compared with the conclusions of the analysis of stiffened plate panels.

2.3.3 Axial compression of stiffeners

The ultimate strength of the beam-column, stiffener with associated effective plating, subjected to axial compression, may be obtained from eqn (20), considering that the Euler stress ratio of the beam-column is given by:

$$\phi_e \equiv \frac{\sigma_e}{\sigma_0} = \frac{\pi^2}{\lambda^2} \text{ and } \frac{\sigma_{ua}}{\sigma_0} = \phi_c \tag{22}$$

where the column slenderness is:

$$\lambda = \frac{l}{r}\sqrt{\frac{\sigma_0}{E}}. \tag{23}$$

The strength of the panel should not be taken greater than the tripping limit of the stiffener. The determination of the critical tripping stress ratio follows the Ostenfeld-Bleich procedure in order to account for elasto-plastic behaviour when the elastic tripping stress ratio is greater than the proportional limit.

3 TRIPPING OF STIFFENERS

Most of the design methods do not account for the tripping failure of the stiffeners. The approach generally adopted is to design the stiffeners so that tripping failure is always avoided. This tripping induced failure shows a very rapid unloading and should be avoided. For plain flat stiffeners this implies that the ratio of the height to thickness should satisfy:

$$\frac{h}{t} \leq c\sqrt{\frac{E}{\sigma_0}} \tag{24}$$

where $c = 0.35$–0.37.

Another aspect that has led to designs which intend to be safe in relation to tripping failure is the lack of theoretical formulations that account properly for their collapse in the elasto-plastic range.

3.1 Elastic tripping stress

There are not so many studies about tripping and most of the present work has followed the approach presented by Faulkner [36,37] and Adamchak [38], using them to determine the tripping stress and to estimate a pattern for load shedding after tripping. Their approach is based on the Rayleigh's principle in order to obtain the elastic critical stress for tripping. Corrections to the linear derived tripping stresses are presented which are intended to incorporate nonlinear behaviour of both plate and stiffener.

The proposed approach balances the torsional, sideways bending, warping, and spring strain energies with the elastic tripping stress energy:

$$I_p \sigma_T = GJ + \frac{m^2 \pi^2 E T_p}{a^2} + \frac{C_s a^2}{m^2 \pi^2} \tag{25}$$

where J is the St Venant torsional constant, T_p is an appropriate tripping parameter that includes both sideways bending, $I_z \bar{z}^2$, longitudinal warping contributions, Γ and is defined as:

$$T_p = I_z z^{-2} + \Gamma \tag{26}$$

C_s is the elastic rotational spring stiffness per unit length of the toe which can be determined as:

$$C_s = \frac{Et^3}{2.73b}. \tag{27}$$

Equation 25 is applicable when considering a constant constraint along the toe, but a more accurate approach can be obtained if one accounts for the destabilising moments induced by the plate's loaded shape. Faulkner [37] proposed that the rotational constraint might be approximated by a linear interaction based on the analysis of dynamic behaviour ship grillages [39]:

$$\frac{C}{C_s} + \frac{\sigma}{\sigma_{cr}} = 1 \text{ for } \frac{\sigma}{\sigma_{cr}} \leq 2 \tag{28}$$

where σ_{cr} is the elastic critical stress of a simply supported plate:

$$\sigma_{cr} = \frac{\pi^2}{12(1-\nu^2)} E \left(\frac{t}{b}\right)^2 \left[\frac{m_0 b}{a} + \frac{a}{m_0 b}\right]. \tag{29}$$

Introducing eqn (28) eqn (29) into eqn (25), the tripping stress may be determined as:

$$\sigma_T = \frac{GJ + \frac{m^2 \pi^2 E T_p}{a^2} + \frac{C_s a^2}{m^2 \pi^2}}{I_p + \frac{C_s a^2}{m^2 \pi^2 \sigma_{cr}} k}. \tag{30}$$

The coefficient k is an attempt to include the interaction between the plate and the stiffener and Faulkner recommended the values 1.0, 0.0, 0.5 and 0.0, respectively, for $m_0/m = 1,2,3$ and more than 3. Theoretically, two remarks must be made about this approach; first the third value must be 0.33 because only one third of the plate is destabilising the stiffener; on the other hand if the plate is destabilising the stiffener then the stiffener is stabilising the plate and some degree of rotational constraint is applied to the plate, so the assumed simply supported plate's boundary conditions might seem to be a little conservative; also any consideration about the plastification of the toe is not taken into account which might be very important from the point of view of load-end shortening curves of columns since if some plastification occurs than the toe's spring constant will be reduced.

3.2 Inelastic effects

In order to estimate inelastic effects, Faulkner [37] had recommended the use of $\sqrt{E_t E}$ instead of the E_t used on column flexural buckling, due to local bending effects, and a tangent modulus defined by the Ostenfeld-Bleich quadratic parabola:

$$\frac{E_t}{E} = \frac{\phi \cdot (1 - \phi)}{p_r \cdot (1 - p_r)}. \tag{31}$$

The inelastic tripping stress of the stiffener with effective associated plate will become:

$$\begin{aligned} \phi_{Ti} &= \frac{\phi_T^2}{\phi_T^2 + p_r(1 - p_r)} \quad & if\ \phi_T > p_r \\ \phi_{Ti} &= \phi_T & if\ \phi_T < p_r \end{aligned} \tag{32}$$

where p_r is recommended to be taken as 0.8 because of the presence of largely tensile stress state due to residual stresses. However, for ships in service, $p_r = 0.5$ may be more relevant because the residual stress level drops quickly in the early stage of ship's life.

The ultimate average tripping stress of the panel will finally be:

$$\phi_{Ti} = \phi_{Ti} \frac{A_s + b_e t}{A_s + bt} \tag{33}$$

where the effective width b_e must be calculated for a slenderness $\beta = \beta_0 \cdot \sqrt{\varepsilon_{Ti}}$ using eqn (4), eventually corrected by the reduction factors (eqn (5a–c)); the approach used to determine b_e implicitly considers that the stiffener behaves elastically until σ_{Ti} is reached and so, naturally, the correspondent strain will be $\varepsilon_{Ti} = \sigma_{Ti}/E$ which is also the plate's average shortening.

4 CALIBRATION WITH EXPERIMENTAL RESULTS

4.1 Available experimental results

The models tested by Faulkner [14] were of a single bay construction, representing approximately one-quarter scale inter-frame ship panels. The stiffeners were mainly T-sections and the remainder were flat bars. All panels had five longitudinal stiffeners except two that were single stiffened plates.

Continuous manual single pass fillet welds were used, but different levels residual stresses were included in order to determine their effect.

The models were simply supported on the loaded edges and free to deflect out-of-plane on the unloaded edges. The material was mild steel in sheet thickness from $t=2.5$ to 4.9 mm and stiffened depths from $d=23.6$ to 63.5 mm. Special care was taken due to the fact that mean yield stress in the plate is 24% lower than in the stiffener. A series of 13 similar models was tested to assess the level of systematic errors presented. A value of 10.9% was found.

Horne [12,26] has tested 44 stiffened plates in end-long compression. A parametric study was conducted, involving plate slenderness, column slenderness, residual stresses and initial imperfections. Also, different boundary conditions were imposed on the loaded edges but unloaded edges were free to deflect out of plane.

Most part of the tests were designed for the specimens to collapse by plate induced failure while five of them were expected to have a stiffener induced failure. The stiffeners were welded to the plating in two different modes: continuous and intermittent. Sheet plates of 10 mm, 9.5 mm and 6.5 mm, associated with different sizes of stiffeners (152.5 mm × 16 mm, 152.5 × 9.5 mm and 80 mm × 12 mm), were combined in order to cover the range of slendernesses. Two grades of steel were used: grades 43 and 50. The panels were of single span, 1830 mm long.

At Glengarnock [40], the models were of two bay construction, free to deflect out-of-plane along their unloaded edges. The plates of the models had a slenderness β of about 3.5. The column slenderness, λ, had covered the range of 1.0 to 4.6, i.e., including very slender panels. Boundary conditions on loaded edges were fixed and intermittent welding was used to fix stiffeners to plating, apart from in one plate.

In the tests performed at Imperial College [41], the panels were supported laterally along all four edges. Pinned end condition models of four and nine longitudinal stiffeners were chosen. Continuos welding was used to attach stiffeners to plating. The number of stiffeners and the boundary conditions ensured the models were representative of real structures.

Seven full scale welded steel grillages representative of warships bottom, deck and superstructures were tested by Smith [11]. An extensive coverage with measurements of initial and plate deformations is available. A special care was taken on the reproduction of shipyard welding procedures.

4.2 Analysis of the experimental results

The analysis and comparisons of tests with design formulae is performed on the basis of the ratio of the predicted strength from each method and the experimental strength. Thus, the column identified as Faulkner in Table 1 represents the ratio between the expected strength given by Johnson-Ostenfeld formulation, eqn (1), and the experimental strength. For the column identified as Carlsen, eqn (11) was used, supported by eqn (13) for plate induced failure and eqn (14) for stiffener induced failure. In the case of ABS, the prediction is based on eqn (16) using the Frankland equation for plate strength and the auxiliary eqns. 19, 20, 22.

The results were presented by the sources of experiments in order to enable the identification of one of the possible sources of bias, which is connected with the way how the experiments are conducted; a compilation of results by type of welding procedure and boundary conditions is also included separately in Table 1.

For all data available the Faulkner method seems to be the most reliable one except for one source, as indicated in Table 1. The overall mean value of the prediction normalised by the experimental strength is 1.05 with a coefficient of variation of 13%. This relatively high value of the scatter is mainly due to the unusual mean value of the one source of the tests series conducted by Faulkner, which is 20% higher than the others for all the methods. In this case the models showed a very low strength that may be a result of an incorrect procedure during the tests, or due to relatively high difference between the plate and stiffener yield strengths; because of this it was felt the need to define clearly the yield stress to be used in the prediction methods.

The choice was to consider a yield stress resulting from the weighting of areas and the respective yield stress:

$$\sigma_0 = \frac{A_s \sigma_{0s} + A_p \cdot \sigma_{0p}}{A_s + A_p}. \tag{34}$$

When this source, Faulkner a) in Table 1, is removed from the data base all strength prediction formulas have better values of the coefficient of variation. However, the Faulkner method continues to be the most reliable one, having an average value of 1.02 and COV of 10%. It is also significant the agreement between all of the sources of data when this method is used.

This method does not show any marked skewness when the results are plotted against β and λ, Fig. 2 and Fig. 3.

Carlsen method has a higher scatter with an overall coefficient of variation of 16% when one considers the minimum value between plate and stiffener induced failure, denoted as P.I.F. and S.I.F. in the table. It is a conservative method with a mean value of 0.842 when the minimum of P.I.F. and S.I.F. is used, which seems to mean that a safety factor of about 1.15 is used. However the uncertainty associated with the large scatter cancels the safety margin provided by the bias of the method. The analysis of the results for the plate induced failure and stiffener induced failure separately shows that the

TABLE 1

Comparisons Between Several Methods of Strength Predictions Organised by Source of Tests, Welding Procedure (Continuous and Intermittent) and Boundary Conditions on Loaded Edges (Clamped and Simply Supported). ABS-UPS Prediction of Beam-column Strength Uses Ultimate Plate Strength (eqn 17) and ABS-CPS Prediction of Beam-column Strength Uses Critical Plate Strength (Eqn 20)

Mean Values

Method Source	*Faulkner*	*Carlsen*		*ABS-UPS*		*ABS-CPS*		*no. obs*
		P.I.F.	*S.I.F.*	*w/out trip.*	*w/trip.*	*w/out trip.*	*w/trip.*	
Faulkner[14]a	1.222	1.039	1.140	1.457	1.413	1.131	1.093	18
Faulkner[14]b	1.003	0.817	0.957	1.294	1.194	0.882	0.824	24
Mathewson[40]	1.028	0.731	0.961	1.416	1.292	0.847	0.753	17
Rutherford[41]	0.990	0.953	0.851	1.027	1.027	0.933	0.933	5
Horne[12,26]	1.030	0.944	0.935	1.148	1.037	0.987	0.887	44
Smith[11]	1.022	0.874	1.043	1.138	1.112	0.889	0.760	7
Continuous Welding	1.061	0.924	0.982	1.265	1.183	0.971	0.915	79
Intermitent Welding	1.020	0.829	0.949	1.274	1.125	0.921	0.798	36
All-clamped	1.003	0.815	0.972	1.250	1.065	0.901	0.748	33
All-simply supported	1.066	0.926	0.972	1.274	1.204	0.978	0.931	82
All-data	1.052	0.897	0.978	1.275	1.170	0.959	0.881	115
All-data*	1.020	0.871	0.948	1.242	1.125	0.927	0.841	101

Coefficient of Variation

Method Source	*Faulkner*	*Carlsen*		*ABS-UPS*		*ABS-CPS*	
		P.I.F.	*S.I.F.*	*w/out trip.*	*w/trip.*	*w/out trip.*	*w/trip.*
Faulkner[14]a	0.116	0.162	0.128	0.198	0.216	0.108	0.125
Faulkner[14]b	0.062	0.181	0.183	0.224	0.171	0.112	0.187
Mathewson[40]	0.119	0.181	0.188	0.189	0.245	0.134	0.206
Rutherford[41]	0.080	0.047	0.151	0.197	0.197	0.108	0.108
Horne[12,26]	0.107	0.145	0.120	0.150	0.213	0.100	0.174
Smith[11]	0.161	0.271	0.062	0.164	0.164	0.270	0.470
Continuous Welding	0.131	0.193	0.154	0.217	0.222	0.158	0.219
Intermitent Welding	0.095	0.183	0.169	0.181	0.249	0.123	0.183
All-clamped	0.099	0.165	0.149	0.216	0.305	0.118	0.160
All-simply supported	0.126	0.195	0.163	0.203	0.194	0.154	0.205
All-data	0.126	0.196	0.163	0.211	0.238	0.153	0.222
All-data*	0.103	0.190	0.151	0.204	0.222	0.138	0.213

* Data analysed without [14]

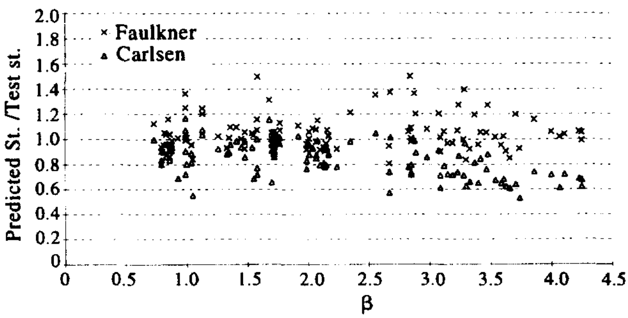

Fig. 2. Beam-column strength predictions normalised by tests results against plate slenderness β.

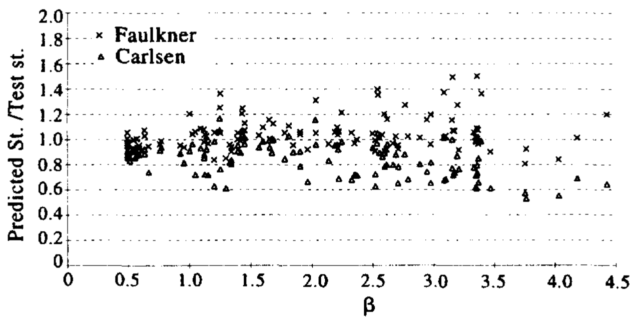

Fig. 3. Beam-column strength predictions normalised by tests results against column slenderness λ.

last one has a bias closer to the unity and a lower scatter than the plate induced failure but the coefficient of variation still remains very high (COV = 16%).

On the other hand, ABS method is very optimistic, 27% higher than experimental results without considering tripping and 17% with tripping formulations. The COV are 21% and 24%, respectively. This means that the tripping formulation predicts very low strength when flexural/torsional buckling is predominant, Fig. 4 and Fig. 5.

Since this method is similar to the Faulkner method except that, in the plate strength prediction, it replaces the Faulkner equation by Frankland one, it may be said that the 22% difference is a result of the magnification of the 10% difference between the two plate strength formulations in Fig. 1.

If the effective width of the plate is defined by the critical strength (eqn (20)), in order to calculate the effective cross section area then the ABS method becomes a little conservative (mean value = 0.96) associated with a COV of 15%. In this case, the use of tripping formulation turns the prediction very conservative, from 4.1% to 11.0%, and the scatter increases

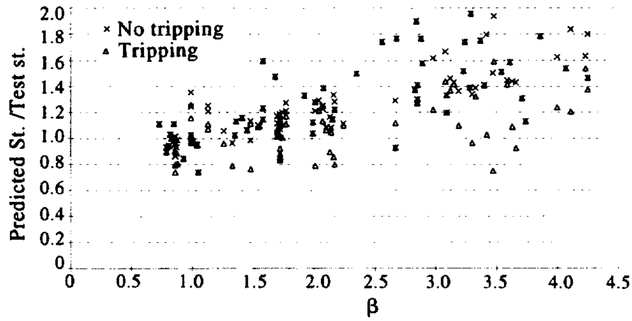

Fig. 4. ABS beam-column strength prediction normalised by tests results ϕ_p/ϕ_{exp} against plate slenderness β, using ultimate plate strength.

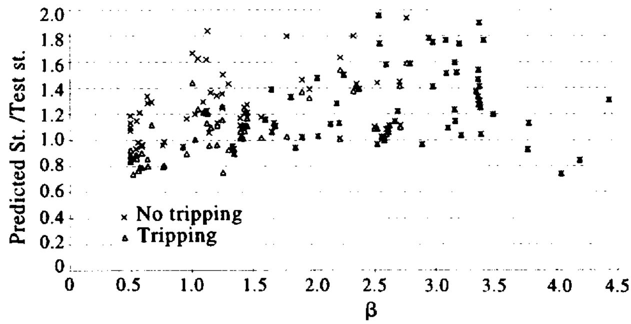

Fig. 5. ABS beam-column strength prediction normalised by tests results ϕ_p/σ_{exp} against column slenderness λ, using ultimate plate strength.

substantially (COV increases to 22%), which confirms that torsional/flexural buckling formulation needs more investigation, particularly concerning the elasto-plastic effects and the interaction with associated plating.

From Figs 6 and 7, it is evident that the tripping formulation only affects the low column slenderness and it is independent of plate slenderness. For these slendernesses the collapse phenomena is mainly due to elasto-plastic effects, due to flexural buckling without tripping relevance.

One solution for this may consist of considering tripping only when *elastic* tripping stress is lower than flexural bending stress. Two main reasons may be pointed out to justify it: the first deals with the nature of usual ship panels which are designed in such a way that flexural-buckling occurs normally earlier than tripping buckling of stiffeners; the second is directly related to the nature of tripping of lateral-torsional weak stiffener. For this kind of stiffeners, let us say bar stiffeners, tripping occurs prematurely, on the elastic range of stress and in this situation plastic effects are irrelevant.

Figures 8–11 plot the distribution of strength predictions normalised by tests.

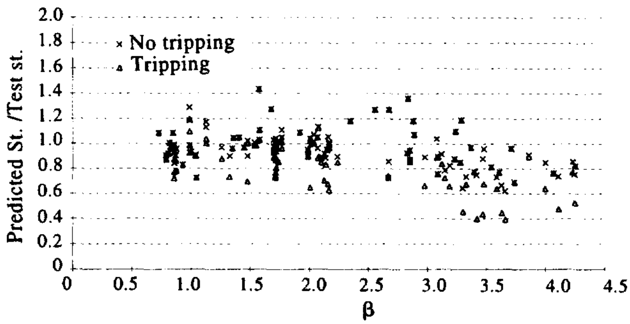

Fig. 6. ABS beam-column strength prediction normalised by tests results ϕ_p/ϕ_{exp} against plate slenderness β, using critical plate strength.

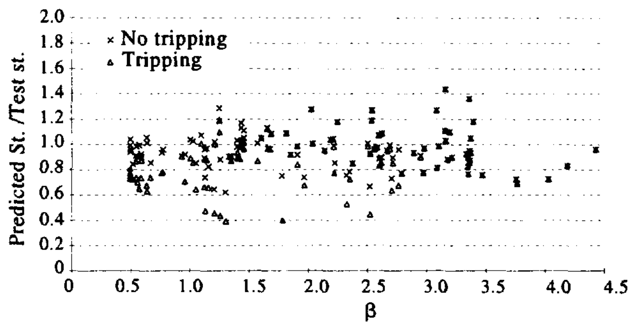

Fig. 7. ABS beam-column strength prediction normalised by tests results ϕ_p/ϕ_{exp} against column slenderness λ, using critical plate strength.

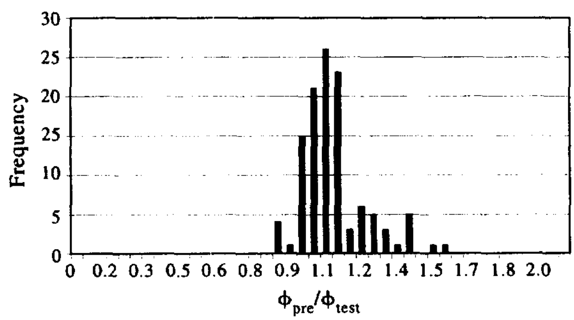

Fig. 8. Distribution of results of Faulkner method normalised by tests.

They confirm the concentration of results in the range of 0.95 to 1.1 when the Faulkner method is used. Also, the ABS prediction using critical plate strength, Fig. 11, shows an approximate Gaussian distribution around 1.0.

In order to investigate the overall skew of the different methods, the predictions normalised by tests results are plotted against the tests results

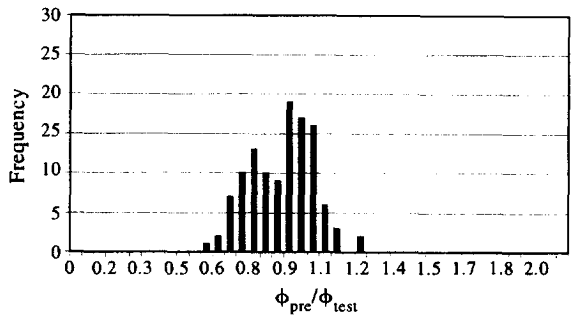

Fig. 9. Distribution of results of Carlsen method normalised by tests.

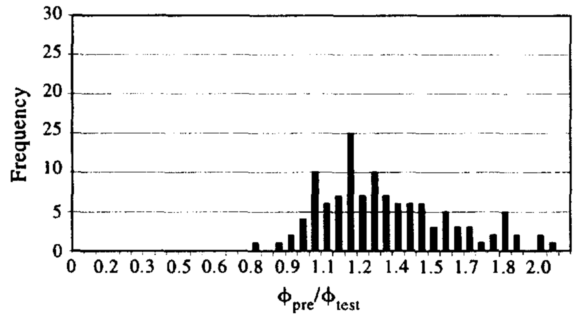

Fig. 10. ABS prediction for beam-column model using Frankland equation normalised by tests.

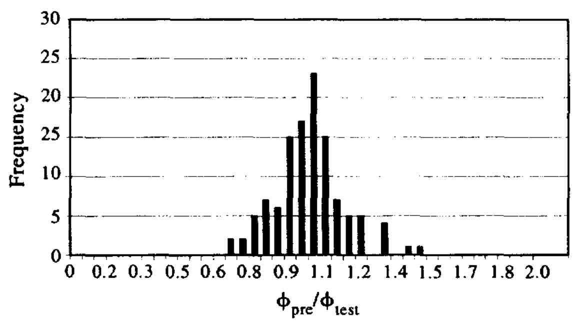

Fig. 11. ABS prediction for beam column model using critical plate strength normalised by tests.

themselves, because the tests strength showed by the model incorporated always the influence of the two main parameters: plate slenderness and column slenderness.

Figures 12–15 cover all the methods analysed showing that Faulkner method has not any marked skew when the strength decreases and the scat-

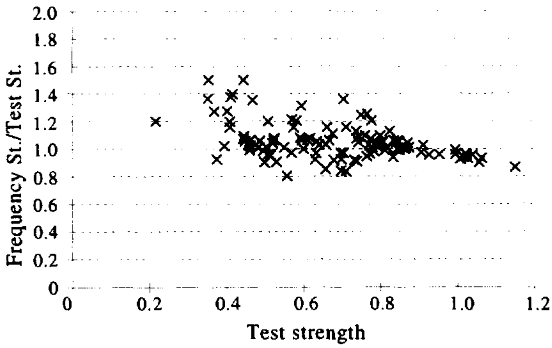

Fig. 12. Faulkner method.

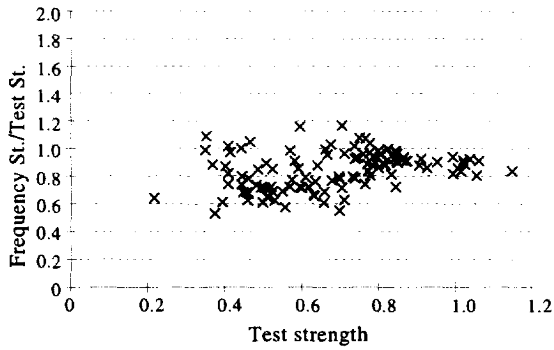

Fig. 13. Carlsen method.

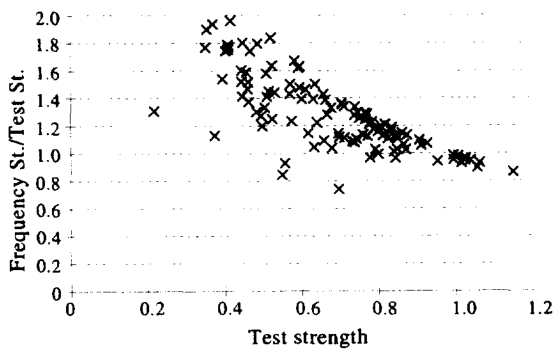

Fig. 14. ABS method using ultimate plate strength without tripping formulation, normalised by tests strength compared with test strength.

ter does not increase substantially. On the other hand, the ABS method (ultimate plate strength) demonstrates the importance of the equation for plate strength on the prediction of column strength: there is a very large skew with tests strength, also, the scatter increases drastically.

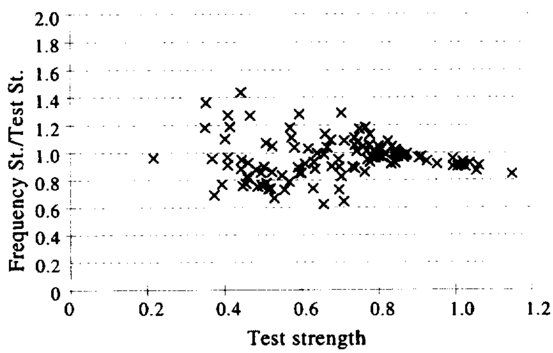

Fig. 15. ABS method using critical plate strength without tripping formulation, normalised by tests strength compared with test strength.

Good agreement with theories is achieved when the collapse stress is greater than 80% of the yield stress. This represents a plastic behaviour of the column, but even in this situation Johnson-Ostenfeld based methods predict better results, i.e., closer to 1, than those obtained with Perry-Robertson formulations.

5 EFFECT OF LATERAL PRESSURE

Lateral loading from sea water pressure or cargo is always present on plates and stiffened plates elements. Thus, it is important to investigate its influence on the ultimate axial compression strength of those elements.

Some of the methods available do not account for this influence because they considered it irrelevant when the pressure level is low, which is the case of sea water pressure on usual ships. Furthermore, in some cases, the presence of lateral pressure may induce an increase in strength, as in some plates of high aspect ratio, where lateral pressure develops deformations on the first mode which is a high strength mode for plates with aspect ratio higher than 1. This increase on plate strength may cancel the negative contribution of initial bending moment on the beam-column due to the lateral pressure. This global effect may be detected on beam-columns in compression that collapse by plate induced failure but the positive contribution of the plate is less well noticed when collapse is due to stiffener induced failure.

Partially because of this duality of behaviour, the Faulkner method does not incorporate any correction for lateral pressure while the Carlsen formulation may be corrected introducing an fictitious initial imperfections of the stiffener equal to the beam deflection.

ABS method uses a limiting eqn (16) which accounts for bending due to lateral pressure by the second term of the first member of the equation, where m is a magnification factor and σ_{ub} is the ultimate bending stress. Since this limit equation is linear, a reduction on axial compression is predicted even when the pressure is low. This seems to be a little conservative.

5.1 Available experimental results

Smith [11] used lateral pressure in four of the eleven experiments that he has carried out. The geometry of these grillages were similar to those without pressure, allowing for comparison and analysis of the effect of lateral pressure. The levels of pressure applied were of the same magnitude of those found on ship shell.

Kondo and Ostapenko [42] tested two small simply supported panels with the same lateral pressure but different plate and column slendernesses. The material used was steel of about 275 MPa yield stress and the panels were fabricated by welding.

Dean and Dowling [43] tested 3 simply supported panels reinforced by 8 stiffeners. The panels were fabricated of mild steel by welding. Transversely, there were two frames spaced by 742 mm and end bays were stiffened to force failure in the middle bay. Two levels of pressure were used.

Dubois [44] conducted 5 tests, two of them on transversely stiffened panels. Each series used geometrically similar panels where different levels of lateral pressure were applied. The yield stress of the material was of 295 MPa and the panels were fabricated by welding.

5.2 Analysis of the experimental results

The analysis of stiffened plates subjected to uniaxial compression and lateral pressure is quite complicated because three main parameters are involved: plate slenderness, column slenderness and lateral pressure. Furthermore, there are very few experimental results available. Only some qualitative indications can be obtained.

All methods of strength prediction give good results of the mean value and coefficient of variation, Table 2. The overall mean value of the predicted stress normalised by the experimental stress varies from 0.97 (Carlsen-stiffener induced failure) to 1.12, (Carlsen-plate induced failure). The overall COV is similar in all the cases being around 11%. The partial standard deviation of the sources is irrelevant due to the small number of experiments involved in each one.

Figures 16–18 plot the experimental uniaxial stress of the column normalised by the predicted stress against, respectively, the normalised lateral pres-

TABLE 2
Comparisons of Several Methods of Strength Predictions Organised by Source of Tests

Method Source	*Faulkner*	*Carlsen*		*ABS Lat. Pressure*	*No. Obs*
		Mean Values			
		P.I.F.	*S.I.F.*		
Kondo	1.119	1.023	1.091	1.038	2
Dubois	1.149	0.943	1.255	1.169	3
Smith	1.055	0.961	1.005	0.933	4
All data	1.101	0.968	1.115	1.035	9

Method Source	*Faulkner*	*Carlsen*		*ABS Lat. Pressure*	*No. Obs*
		Mean Values			
		P.I.F.	*S.I.F.*		
Kondo	0.188	0.273	0.098	0.056	2
Dubois	0.092	0.092	0.092	0.054	3
Smith	0.073	0.154	0.078	0.073	4
All data	0.102	0.149	0.123	0.119	9

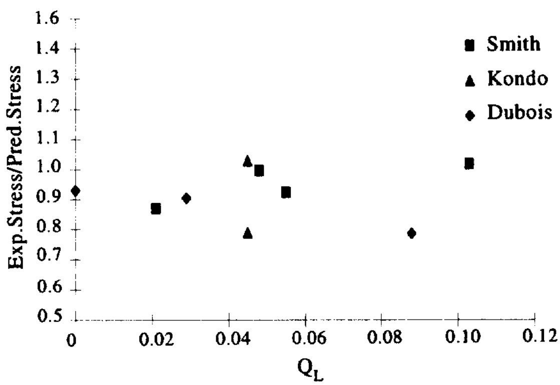

Fig. 16. Tests with lateral pressure [MPa] and uniaxial compression normalised by Faulkner method.

sure, the plate slenderness and column slenderness using the Faulkner method.

Dubois experiments have the same β and λ in order to investigate the influence of three levels of lateral pressure. In these conditions the predicted stresses are coincident, thus one may conclude that lateral pressure has a weakening effect on strength, Fig. 16.

Smith experiments seem, apparently to identify an increase on strength with lateral pressure. However this tendency cannot be totally confirmed because the other two parameters (β and λ) are varying: in Fig. 18 it is well

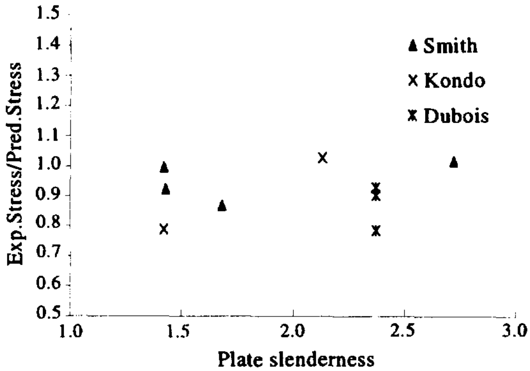

Fig. 17. Tests with lateral pressure and uniaxial compression normalised by Faulkner method.

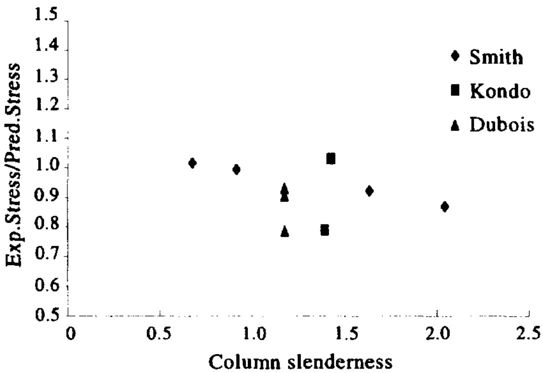

Fig. 18. Tests with lateral pressure and uniaxial compression normalised by Faulkner method.

evident a skew with the column slenderness, which does not lead to any conclusions about the effect of lateral pressure.

The two tests of Kondo have similar λ and Q_L and different β: one using a moderate stocky plating and the other with a slender plating. An increase of strength with β is detected when Faulkner formulation is used.

More interesting is the comparison of similar panels with and without lateral pressure (Smith tests, Figs. 19–21). For this source of tests the skew of Faulkner formulation with λ is confirmed both for no lateral pressure and lateral pressure. However, the skew is more marked in the absence of lateral pressure, which may mean that lateral pressure reduces the scatter of the results. This may be justified by the stabilising effect of lateral pressure, because it induces similar shapes of deformations both on the plating and on the beam-column.

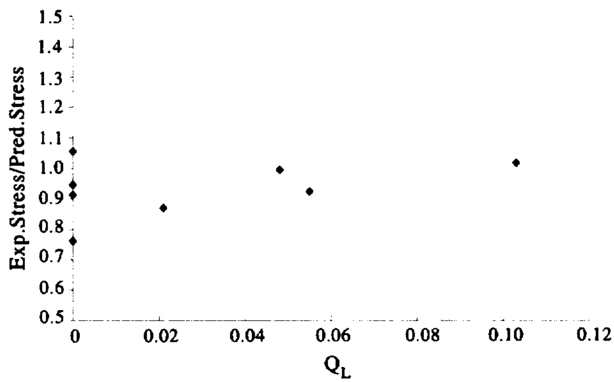

Fig. 19. Smith tests normalised by Faulkner formulation: comparison between tests without and with lateral pressure [MPa] plotted against lateral pressure.

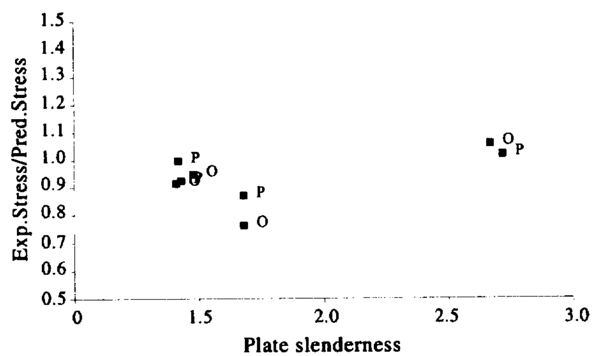

Fig. 20. Smith tests normalised by Faulkner formulation: comparison between tests without lateral pressure (O) and with lateral pressure (P) plotted against plate slenderness β.

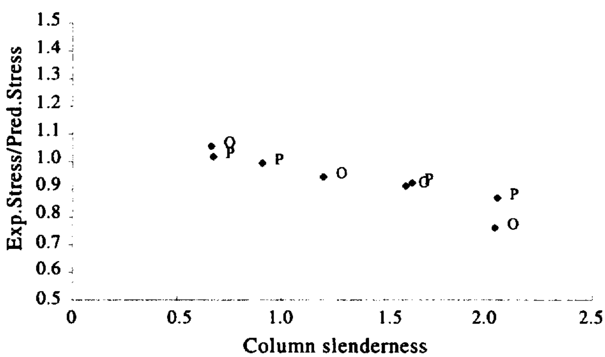

Fig. 21. Smith tests normalised by Faulkner formulation: comparison between tests without lateral pressure (O) and with lateral pressure (P) plotted against column slenderness λ.

The analysis of the available experimental results does not allow a definite conclusion about the degrading effect of lateral pressure. However, the scatter of the results seems to be reduced when lateral pressure is present compared with the scatter in the absence of pressure. Also the number of sources of these tests do not enable more conclusions, since the experimental techniques are different and the test results are very sensitive to that. For plates subjected to biaxial compression and even simultaneous effect of lateral pressure some recent proposals of design equations [45,46] can also be adopted.

6 CONCLUDING REMARKS

Since this study was concluded in 1992, some further results have been published and it is useful to review them and to compare their findings with the ones reported here.

Bonello *et al.* [47] compare predictions using several design codes against numerical results of stiffened plates under axial compressive load. The effect of lateral pressure is also investigated. A method based on the Perry equation is proposed to account explicitly for axial compressive loading and lateral pressure. The codes used in the comparison were modified adopting the same plate effective width formulation [48] for all of them:

$$\phi_p = 0.23 + \frac{1.16}{\beta} - \frac{0.48}{\beta^2} + \frac{0.09}{\beta^3} \tag{35}$$

in order to explore the differences in the column approach.

Panagiotopoulos [49] and Vayas [50] have studied the ultimate torsion of stiffeners attached to plating under axial compression. Panagiotopulos [49] uses, in his finite element analysis, plates with an aspect ratio of 3, with slenderness ranging from 40 to 90, and bar stiffener slenderness from 5 to 30. Both restrained and unrestrained imperfect flat outstands were analised and the results showed that the unrestrained outstands underestimate the strength of slender bars attached to associated plates.

A benchmark study of the ultimate strength of multi-span stiffened panels was conducted by the ISSC Committee V.I. [51]. Several classification societies and other organisations were invited to predict the ultimate capacity of 10 stiffened panels and comparisons between the codes and methods were summarised in [52]. It is interesting to note that the same method used by different contributors gave different predictions for the same examples. The main conclusion from this study was that most methods are pessimistic and have a model uncertainty always larger than 10%.

Pu *et al.* [53] have reassessed the method proposed by Guedes Soares [31] for the strength of plate elements, checking it against experimental and new numerical results and they confirmed that the method performs better than the original proposal of Faulkner (eqn (4)).

They also modified Faulkner's method for the design of stiffened plates, as described in Section 2.1 here, by substituting in that procedure Faulkner's plate strength formulation with the one of Guedes Soares and they have concluded that this method performs better, although it must be noted that the improvements are only at a level of 5%, since the method was already good.

The new classification note from DnV [34] introduced a few minor modifications to the flexural buckling of stiffened panels relative to the method of Carlsen described in Section 2.2.

In 1995, ABS published new rules for the design of ships including plate buckling formulations. ABS new rules are basically as described here, which are based on an Euler formulation modified by the Johnson-Ostenfeld criteria to account for plasticity, using a proportional limit of 0.6. The concept of effective area of the associated plate is adopted to calculate the radii of gyration of the cross section. The tripping of the stiffener is also covered in a similar way of that described in Section 3.

7 CONCLUSIONS AND RECOMMENDATIONS

A discussion has been provided on the way important parameters affect the strength of stiffened plate elements and some methods to quantify their effect have been advocated.

Although numerical methods were not discussed in detail, reference was made to the existing formulations and it was emphasised that at present several computer programs are available to provide a relatively accurate prediction of the load carrying capacity of a stiffened plate. Because these methods become inconvenient for systematic use in design of stiffened panels, an evaluation of simpler design methods was provided.

The comparisons that have been made with experimental results and with predictions of numerical codes, allow conclusions to be drawn about the relative accuracy of the different design methods.

One aspect which is worth noting is that some of the design methods have implicitly incorporated a safety margin (Carlsen method and the tripping formulation) and therefore they predict strengths lower than those found in actual tests. Other methods (Faulkner and ABS) predict the mean value of the strength and thus they can be directly compared with experimental results. In order to use them as a design method, an explicit safety factor should be used.

The ABS method was found to be generally unconservative with a bias of 1.27 (the collapse stress is overestimated by 27%) when the recommended

ABS ultimate plate strength is used. ABS formulation becomes slightly conservative (4%) when the critical plate strength replaces the former formulation of plate elements strength. However there is no justification to use critical strength instead of ultimate strength to design plates against collapse.

This means that a special care must be taken with the plate strength formulation, because it has a great impact on column strength prediction. It may also be concluded that the bias in relation to β which was detected with ABS formulations can be corrected with a different choice of plate strength formulae.

As a last remark about uniaxial compression strength it must be said that including the tripping formulation does not increase the accuracy of the results, which implies that more investigation is needed on this subject.

Relative to panels subjected to lateral pressure and uniaxial compression, it could not be proved of that lateral pressure had a weakening effect due to the small number of tests available. However the scatter of results seems to be reduced when lateral pressure is present, comparatively to the scatter in the absence of pressure.

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