Compressive Tests on Long Continuous Stiffened Panels

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Results of eight tests on long stiffened panels under axial compression until collapse are presented. The specimens are three-bay panels with associated plates made of very high tensile steel S690. Four different configurations are considered for the stiffeners, which are made of mild or high tensile steel for bar stiffeners and mild steel for L and U shape stiffeners. The influence of the stiffener's geometry on the ultimate strength of the stiffened panels under compression is analyzed. This series of experiments belongs to an extended series of tests that include short and intermediate panels, which allows analyzing the effect of space framing on the strength of stiffened panels.
[DOI: 10.1115/1.4004517]

Introduction

There is a trend in maritime transportation for the use of non conventional materials that allow having the same hull strength with a lighter ship structure. There are several solutions to achieve that purpose, which are the use of composite materials on small boats, aluminum hull structures on small and medium size ships, and high strength steel on large size ships. All these solutions increase the strength to weight ratio, which allows a direct increase of the ship's deadweight allowing for a better economical performance, or, alternatively leading to faster and more efficient ships. The application of very high tensile steel may be considered as a solution but it requires explicit consideration of the failure mechanisms, primarily fatigue and buckling [1].

Another alternative for the design of lighter ships is to look for different design solutions. These solutions must have better structural performance for the same weight, i.e., to have a better local strength against the mechanisms of failure like fatigue and buckling and to give an adequate contribution for the global strength of the hull.

The adoption of very high strength steels satisfies these requirements allowing the use of thinner plates, with the corresponding weight reduction, which is very important for high speed vessels. However thinner plating raises important concerns about the elasto-plastic buckling strength and to circumvent this constraint, new U shape stiffeners have been considered. In view of this novelty, a test program was planned so that the performance of the new configuration could be compared with the traditional solutions of bar and L stiffeners.

The tests of panels under compressive loads raise several problems related to their implementation in order to reproduce adequately the working conditions on a ship structure. Some of the more important ones are the boundary conditions on the loaded top edges and unloaded lateral edges, the control and measurement of out of plane eccentricity of the load, and the continuity of loads and moments to the panel under test.

Although several test programs have been made in the past on stiffened panels under compression [2–5], no results were found for the specific shape of U stiffeners. Most of the tests reported in the mentioned references have been made on one stiffened panel that in a real structure would be limited by transverse frames on the tops. However this approach raises difficulties in reproducing in the experiments adequate boundary conditions at the loaded edges. To circumvent this problem the test series was planned using specimens with three longitudinal bays. The use of three-bay panels instead of single-bay panels [2–4] allows for more realistic results by avoiding boundary conditions problems for the central plates related to eccentricity of load and for including the interference between adjacent panels [5].

The objective of these tests is to compare different structural solutions for panels under compression. Comparison between the performance of S690, mild steel, and hybrid solutions are made. The base geometry is the one used in the box girders tests [6]. In that regard, the results can be compared with those of similar stiffened plates belonging to much larger structures.

Four series of experiments were carried out using two different types of steel as follows: Fully S690 structure having S690 on plating and bar stiffeners (FS series); Hybrid bar structure with S690 on plating and mild steel on bars (BS); Hybrid L structure with S690 on plating and mild steel on L stiffeners (LS); and hybrid U structure made of S690 on plating and mild steel on U stiffeners (US).

This series of experiments belongs to an extended series of tests that include short and intermediate panels, which allows analyzing the effect of space framing on the strength of stiffened panels [7].

Description of the Models

The S690 steel was supplied by Dillinger Hütterwerke in 4 mm thick sheets and the mild steel was supplied by Lisnave Shipyard. The stiffened plates were manufactured at Lisnave Shipyard according to the standard techniques of the shipyard.

Figure 1 shows the geometry of the different panels.

The configuration of the panels allows for partially solving some test problems related to eccentricity of the applied load and the influence of adjacent structures on the collapse of a likely beam column panel. The first problem, related to the eccentricity of the applied load in the top ends of the panel, is reduced to very low values in the middle panel because of the lateral reaction on the supports of the intermediate frames. Thus the middle bay of the panel, where one expects the failure to be developed, is always under axial compression with virtually no eccentricity during the entire loading path, even when the plate effectiveness reduces and the neutral axis shifts.

The interference between adjacent panels may have different consequences, by reducing or increasing the strength of the panel when compared to a similar one made of a single bay. The continuity of a three-bay panel ensures that the supporting conditions near the transverse frames are as close as possible to the simply supported ones; but, on the other hand, some reaction is expected from the outer bays when the collapse begins to develop out of

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Contributed by the Ocean Offshore and Arctic Engineering Division of ASME for publication in the Journal of Offshore Mechanics and Arctic Engineering. Manuscript received April 25, 2010; final manuscript received January 30, 2011; published online December 5, 2011. Assoc. Editor: Xin Sun.

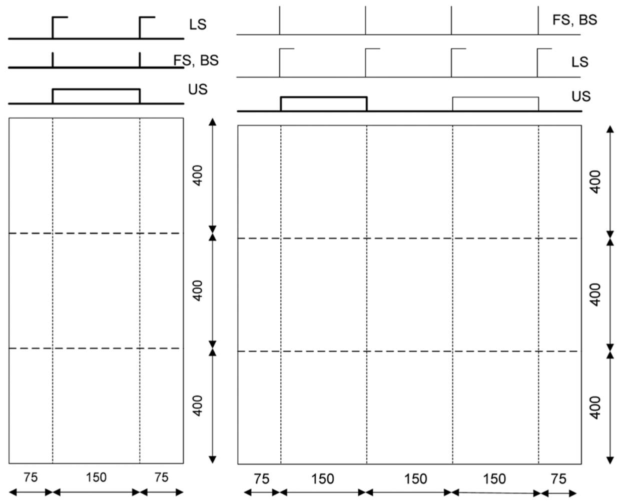

Fig. 1 Geometry of stiffened panels for fully S690 steel (FS), mild steel bar stiffeners (BS), and L and U mild steel stiffeners for narrow and wide panels

plane deformations on the middle bay panel. These reactions reduce the out of plane deformations on the middle panel for the same load, reducing the bending moment due to the induced deformations at every point of the middle panel. However, if the plate induced failure and the stiffener induced failure loads are very close for a particular geometry of the panel, then the overall collapse may be very sudden due to statistical weakness on one of the bays (local high levels of imperfections or residual stresses, for instance), promoting a premature collapse of the whole panel.

These FS panels are similar to those used in the box girder tests reported in [6]. The spacing between longitudinal stiffeners, b, which are bars of B20*4 mm, is 150 mm. The spacing between supporting points (frames) is 400 mm. The number of spans is 3. The material is 4 mm thick S690 steel. The panels have the overall dimensions of 300 mm and 600 mm wide, with two and four stiffeners, respectively; and 1200 mm in length. The yield stress of S690 steel is 690 MPa. The total cross sections are 1360 mm^2 and 2720 mm^2, respectively, for the narrow (A series) and the wide (B series) panel. The corresponding squash loads are 938 and 1877 KN.

The BS panels have the same overall dimensions but are reinforced with mild steel stiffeners of B30*8 mm. The yield stress of the stiffeners is 343 MPa. The areas of the stiffeners are 480 mm^2 and 960 mm^2, respectively, for A and B specimens, resulting in a stiffener to plating area rating of 0.4. The squash loads of the panels are 993 KN (A series) and 1985 KN (B series).

The LS series models have L shape stiffeners (mild steel, L $38 \times 19 \times 4$) with 38 mm of web height, 19 mm of flange width, and 4 mm of thickness. The spacing between supporting points (frames) is 400 mm. The number of spans is 3. The plating is 4 mm S690 steel. The panels have the overall dimensions of 300 mm and 600 mm wide, with two and four stiffeners, respectively, and 1200 mm in length. The yield stress of the stiffeners is 296 MPa. The corresponding squash loads for the two panels are 963 and 1926 KN.

The US series models have U shape stiffeners (mild steel, U $(40+150+40) \times 2$ mm), as shown in Fig. 1, with a thickness of 2 mm, a web height of 40 mm and a flange 150 mm wide. The spacing between supporting points (frames) is 400 mm. The number of spans is 3. The plating is 4 mm thick S690 steel. The panels have the overall dimensions of 300 mm and 600 mm wide, with two and four stiffeners, respectively, and 1200 mm in length. The yield stress of the stiffeners is 200 MPa. The narrow panel has a squash load of 920 KN and the wide one has a squash load of 1840 KN.

The initial imperfections of the panels were measured before they were mounted in the setup device without any restrain at the

Table 1 Geometric and mechanical characteristics of panels

	Narrow Panels—A series				Wide Panels—B series			
	FS	BS	LS	US	FS	BS	LS	US
σ_{Yp}(MPa)		690				690		
σ_{Ys}(MPa)	690	343	296	200	690	343	296	200
σ_{Yeq}(MPa)	690	591	582	554	690	591	582	554
A_p (mm^2)		1200				2400		
A_s (mm^2)	160	480	456	460	320	960	912	920
A_t (mm^2)	1360	1680	1656	1660	2720	3360	3312	3320
A_s/A_t (%)	11.8	28.6	27.5	27.7	11.8	28.6	27.5	27.7
$B \rightarrow \Phi_p$		$2.20 \rightarrow 0.702$				$2.20 \rightarrow 0.702$		
L/r	103.5	52.1	27.8	23.2	103.5	52.1	27.8	23.2
σ_E (MPa)	184	728	2554	3667	184	728	2554	3667
F_{sq} (KN)	938	993	963	920	1877	1985	1926	1840

edges. However, the mounting process of three span panels requires the application of transverse forces in order to maintain the transverse frames and supports in the same plane. Thus, the panels are not free of internal initial bending stresses and the measured free initial imperfections become not relevant for calculations.

Design Characteristics

The main characteristics of interest for design are summarized in Table 1 where σ_{Yp} is the yield stress of the plating, σ_{Ys} is the yield stress of the stiffeners, σ_{Yeq} is the equivalent yield stress of the panel, F_{sq} is the squash load of the panel, A_p, A_s, and A_t denote the plating, stiffeners, and total areas of the panels, respectively; β is the plate slenderness between longitudinal stiffeners; Φ_p is the plating effectiveness [8]; L is the column span between adjacent frames; r is the radii of gyration of the cross section; and σ_E is the Euler stress of the column.

The plate slenderness is the same for all plates and it is defined as

$$\beta = \frac{b}{t}\sqrt{\frac{\sigma_{Yp}}{E}} \tag{1}$$

The plate elements have all the same spacing between longitudinal stiffeners ($b = 150$ mm), the same thickness ($t = 4$ mm) and the material was considered to have a Young's modulus (E) of 200 GPa. As a consequence, the effective width of the plate elements (Φ_p), according to Faulkner [8] is equal to 0.702 [Eq. (2)], which corresponds to an ultimate stress of 484 MPa for the plate elements made of high tensile steel.

$$\Phi_p = \frac{2}{\beta} - \frac{1}{\beta^2} \tag{2}$$

One should note that the same plate element made of normal steel with 240 MPa of yield stress has an estimated ultimate stress of 227 MPa, according to Eq. (2) with a β of 1.3. This means that the material yield stress ratio is 2.875 (= 690/240) but the efficiency of S690 compared to normal steel is reduced to 2.13 (= 484/227), due to the increase in plate slenderness and consequent reduction in the buckling stress.

The column's Euler stress is evaluated considering the whole plating as effective for the calculation the radii of gyration and is given by

$$\sigma_E = \frac{\pi^2 r^2 E}{L^2} \tag{3}$$

The Euler stress is only dependent on the geometric characteristics of the panel and is independent of the yield stress of the base material, which means that the critical elastic stress is the same for panels of the same geometry but made of high tensile steel, normal steel, or a combination of both, ensuring that the materials have the same Young's modulus. However, the application of the concept of a column's slenderness raises several difficulties when applied to hybrid panels. The column slenderness is defined as

$$\lambda = \sqrt{\frac{\sigma_Y}{\sigma_E}} \tag{4}$$

On hybrid panels the yield stress to be used in Eq. (4), σ_Y, may be the yield stress of the plating, of the stiffener, or the equivalent yield stress. The use of each one leads to completely different results and because of that, only the values of L/r and σ_E are presented in Table 1.

The squash load, F_{sq}, is given by

$$F_{sq} = \sigma_{Yp}A_p + \sigma_{Ys}A_s \tag{5}$$

The equivalent yield stress σ_{Yeq} for hybrid panels is defined using the concept of the squash load of the panel [7]

$$\sigma_{Yeq} = \frac{F_{sq}}{A_p + A_s} \tag{6}$$

Assuming that the Young's modulus of the material is the same for S690 and mild steel, one may define an equivalent yield strain as

$$\varepsilon_{Yeq} = \frac{\sigma_{Yeq}}{E} \tag{7}$$

Although both concepts cannot be applicable with high accuracy when used together, in the range of stresses from σ_{Ys} to σ_{Yeq} as shown in Fig. 2, they can be used to compare the normalized stress strain curves of different panels made of different materials.

The difference in the material behavior of the equivalent material and the overall material behavior of the hybrid BS panels may be observed in Fig. 2, where the structural modulus of the real panel reduces above the yield stress of the stiffeners and the equivalent yield strain is lower than the truly global yield strain, which is equal to the yield strain of the S690 steel, ε_{Yp}. The difference between these two strains is computed as:

$$\varepsilon_{Yp} - \varepsilon_{Yeq} = \frac{\sigma_{Yp} - \sigma_{Ys}}{E} \cdot \frac{A_s}{A_t} \tag{8}$$

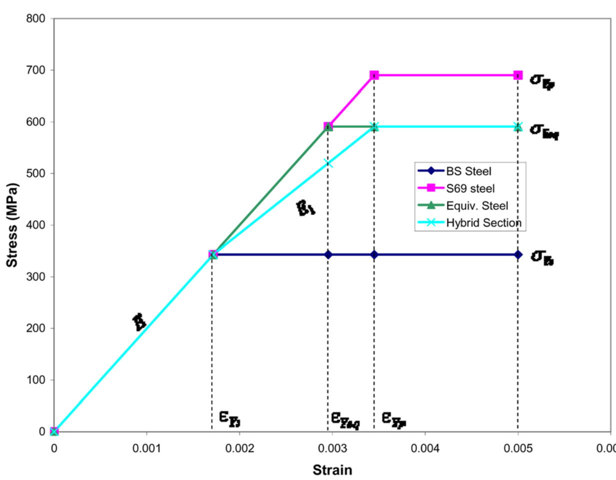

Fig. 2 Material behavior of mild, S690 steel and equivalent material of hybrid BS specimens

The ratio between the elastic modulus after the yielding of the stiffeners, E_1, and the initial elastic modulus, E, is simply given by:

$$\frac{E_1}{E} = \frac{\sigma_{Yeq} - \sigma_{Ys}}{\sigma_{Yp} - \sigma_{Ys}} \tag{9}$$

The equation may be expressed in terms of the sectional areas, applying Eq. (6) and for $\sigma_{Yp} \neq \sigma_{Ys}$, by

$$\frac{E_1}{E} = \frac{A_p}{A_t} \tag{10}$$

The BS panels have theoretical values of $E_1 = 0.714E$ or $E_1 = 143$ GPa. The LS and US panels have $E_1 = 145$ GPa.

Experimental Results

A 300 t hydraulic press was used to perform tests of the panels under uniaxial compression. Figure 3 shows the general arrangement of the tests and a view of the support for the framing systems on the narrow A series, which intends to reproduce simply supported boundary conditions.

The lateral edges of the panels are totally free to move out of plane and to rotate. This means that large panels (B series) should be less affected by the lack of effectiveness at the lateral plating edges during buckling. In fact, the percentage of the total cross-section area with reduced effectiveness due to unsupported lateral edges is lower in the wide panels than in the narrow ones and, thus, the expected ultimate load is higher for the wide panels. The transverse framing system is simply supported in a U bar in each side, allowing longitudinal displacement and in-plane rotation but avoiding out of plane displacement from the initial plane of load. The loaded top edges have full contact with the steel beds, corresponding to nearly clamped conditions, at least until collapse, due to the bi-dimensional geometry of the cross section of the panels.

The hydraulic flow was controlled manually due to limitations on the control device which means that the shortening rate was not constant during the tests.

FS Panels. The panel FS4A was tested in two cycles: first it was loaded until collapse and during the shedding of load the

Fig. 3 Setup of the 200 series test of stiffened plates

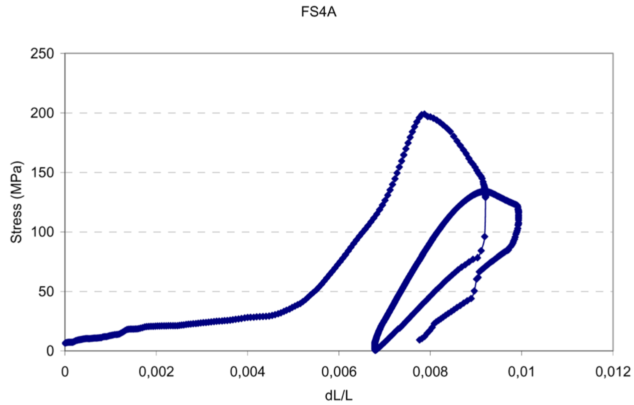

Fig. 4 Average stress versus average shortening curve of FS4A panel

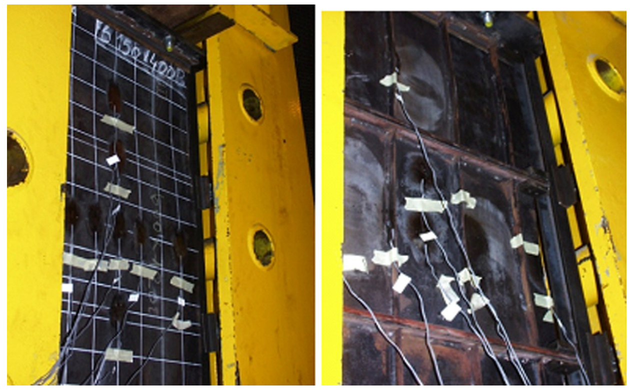

Fig. 6 FS4B at collapse

shortening was reduced until a complete discharge; then the buckled panel was reloaded until reaching a new shedding of load with increasing shortening. The ultimate load was reached at 199 MPa, 8% above the estimated elastic Euler stress. The buckling was marked by a sudden and deep change in the tangent modulus, but it occurs in a smooth and continuous discharge of load. This ultimate load is only 29% of the nominal yield stress of the material.

After the total discharge of load on the first cycle, large permanent deformations were present but the reload showed that the slope of the average stress strain curve is not so different during reloading. In fact, the slope is between the values found for the intact panel at slow and high rate of loading, as it can be seen in Fig. 4 for stresses below 120 MPa and above this value. The loading rate may be evaluated by the spacing between the ticks in the graphic.

The new maximum of the collapsed panel was achieved at the same point as the maximum displacement of the previous cycle, but the slope of the curve changed and the panel showed a lower shedding of load with increasing displacement. One possible reason for this behavior may be attributed to different configurations of the deformed shape of the panel in the two cycles of load, due to the stabilization of the permanent deformations during the discharge on the first cycle.

The form of residual deformations was in three half waves, the middle one toward the stiffeners and outer ones toward the plating. The collapse was of column type mode.

The wide panel FS4B was loaded in three cycles, the first two at low stress levels, 75 and 150 MPa, and the last one until collapse, as shown in Fig. 5.

The average collapse stress was reached at 240 MPa, 20% higher than the one measured in the similar narrow panel FS4A. It is expected that wide panels are stronger than narrow ones, but this large difference can be justified by different levels of initial imperfections. Unfortunately, it was not possible to measure the initial imperfections after mounting the panels.

Three points where the stiffness of the plate changes markedly can be noted from the average stress shortening curve of the final cycle: at 167 MPa there is a discontinuity of the curve associated with a slight reduction in the slope (structural modulus); at 189 MPa, there is a reduction in the slope of the curve; and at 220 MPa, non linear effects become very important due to the out of plane deformation of the panel. Non linearities originated by local plasticity are not present due to low level of stresses relative to the yield stress of the material. Thus, these changes should be originated by rearrangements of the deformed shape of the panel at those points.

The collapse was very sudden and led to an immediate discharge of one third of the load. The type of collapse in the experiment was a column failure type mode in the middle bay of the panel and deformations towards the stiffeners in the outer bays, as may be seen in Fig. 6.

BS panels. The hybrid bar panels, BS4A and BS4B, presented very similar behaviors, shown in Figs. 7 and 8. The ultimate strength was achieved almost at the same average stress, 436 MPa for the narrow panel and 461 MPa for the wide one, which represents a difference of +5.7%. The collapse of both panels was very sudden with sharp decrease of load. As may be seen from Fig. 7, the narrow panel lost almost two-thirds of the compressive load instantaneously and the collapse occurred without much elasto-plastic precollapse deformation.

The wide panel, BS4B, had the same behavior with respect to the nonlinear behavior before buckling and shedding after buckling, as

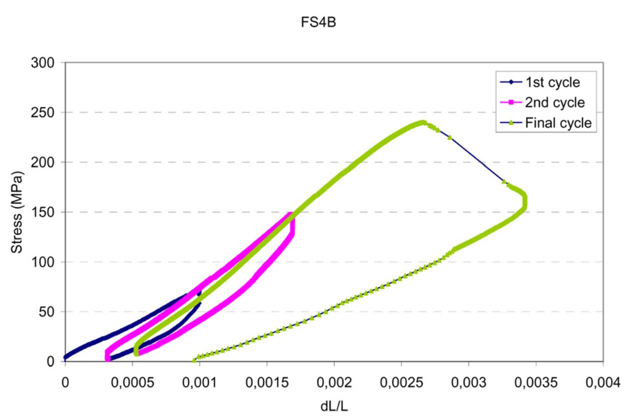

Fig. 5 Average stress versus average shortening curve of FS4B panel

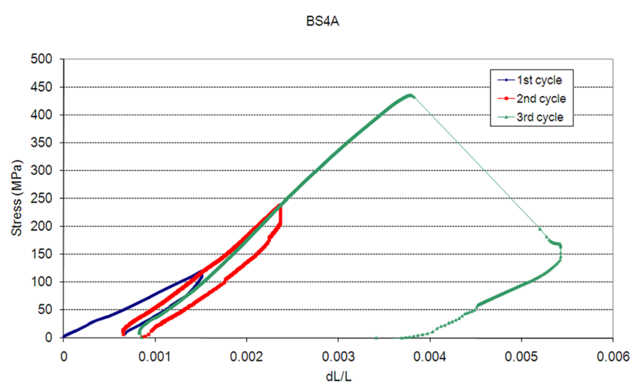

Fig. 7 Average stress versus average shortening curve of BS4A panel

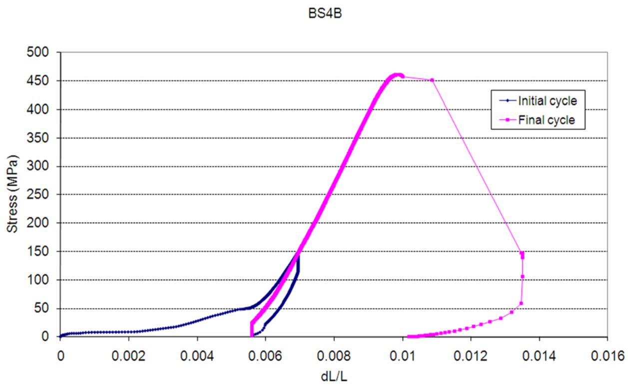

Fig. 8 Average stress versus average shortening curve of BS4B panel

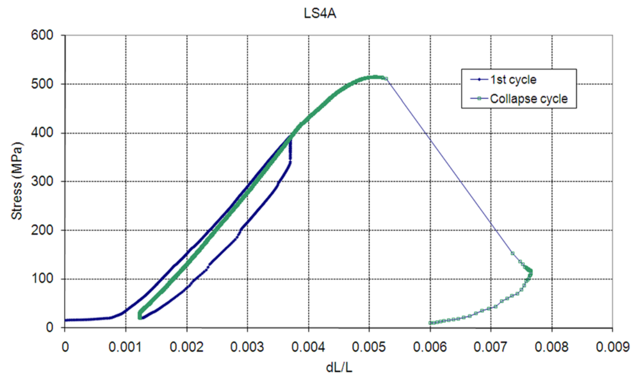

Fig. 10 Average stress versus average shortening curve of LS4A panel

shown in Fig. 8. Even the form of the curve at collapse is very similar, leading to a column type collapse immediately after out of plane deformations have initiated development. This very large shedding is justified by the fact that the average collapse stress greatly exceeds the yield stress of the stiffeners (343 MPa). Thus, stiffeners have no strength reserve when the development of out of plane deformations toward the stiffeners occurred in the middle bay of the panel and toward the plating in the outer bays, leading to an instantaneous creation of plastic hinges in all stiffeners. As result, the residual axial strength of the wide hybrid panel is very low.

Figure 9 shows the residual form of both panels after having been dismounted from the setup.

They present similar deformations due to collapse well visualized in the right side of Fig. 9. The inner bay is deformed towards the plating, meaning that one had large compressive plastic deformations in the stiffeners. The middle bay is permanently deflected towards the stiffeners, but there is not, apparently, any plate buckling deformation. The upper bay remains virtually flat, indicating there was not much spreading of plasticity in that region.

LS panels. The LS4A panel (the L stiffener narrow panel), presents a similar behavior to the BS series panels but with differences with respect to nonlinear behavior after 410 MPa and slower spread of plasticity during collapse, leading to a smoother load shortening curve during collapse, as seen in Fig. 10. One has to note; however, that the shedding after collapse is of the same magnitude of the ones on BS panels, reducing the average stress from 515 MPa at collapse to 130 MPa immediately after collapse.

It is also very interesting to note that the first cycle of load was carried out to an average stress higher than the yield stress of the stiffeners, which means that residual stresses due to manufacturing have been shaken out. In fact the slope of the last cycle's curve is the same as the slope of the first cycle's curve between approximately 210 MPa and the previous maximum of stress, confirming that no residual stress effects were present in the panel after 210 MPa of compressive average stress.

Figure 11 shows the panel after collapse. The residual deformations are, in general, the same type as those of the BS panels. It is very interesting to note the large permanent local deformations on the stiffeners, leading to completely deformed flanges and webs on the L stiffeners, normally called tripping. It is not so usual to have tripping on 'L' stiffeners but one has to bear in mind that the stiffener had already been in the plastic domain at the collapse stress and, thus, large deformations may be easily reached at constant stress.

However, the results obtained with the LS4B wide panel were of different qualitative nature compared to the BS series and LS4A panel. The panel presented a lower ultimate stress (461 MPa) than the narrow one (515 MPa), which is an unexpected result, but it is above the ultimate strength of the BS series panels. The stress shortening curve is presented in Fig. 12 and one may identify several differences from the previous experiments: a sudden increase in deformations after the initiation of the shedding of load, but not as intense as the previous ones; a slower decrease of load after the maximum load has been reached; a marked change of structural tangent modulus at approximately 370 MPa, which should be related to the yielding of the stiffeners.

The lower ultimate stress and the smoother collapse indicate that the initial imperfections of the wide panel were higher than those of the narrow LS panel.

US panels. The US series panels presented almost equal results for both panels. The ultimate compressive stress was 403 MPa for

Fig. 9 BS series panels after collapse; detail of residual deformations on BS4B at right

Fig. 11 LS4A panel after collapse

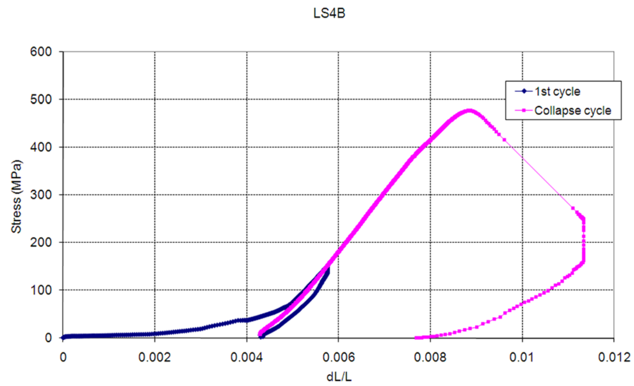

Fig. 12 Average stress versus average shortening curve of LS4B panel

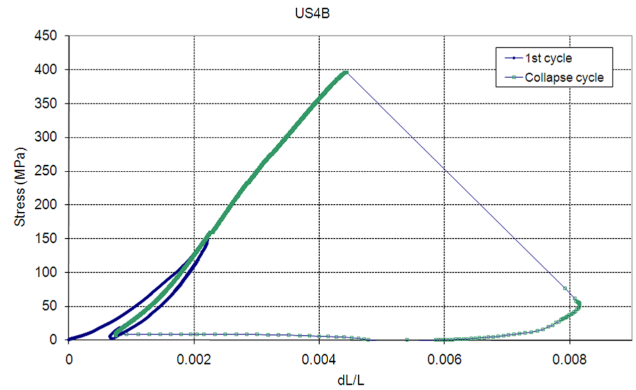

Fig. 14 Average stress versus average shortening curve of US4B panel

US4A panel, Fig. 13, and 396 MPa for the wide panel, Fig. 14, which is less than 2% difference. This level of strength is well above the ultimate stress of the flange plating of the stiffener, which is 133 MPa according to Eq. (2) and the yield stress of the stiffeners, which is 200 MPa.

Both collapses lead to totally inefficient panels, having virtually no structural rigidity. In the narrow panel the stress felt below 140 MPa and the wide one deformed completely without any ability to support a load after collapse. The loading device read 50 MPa after deformation of the panel.

The loading side of the average stress shortening curve showed the same behavior for both panels. On average the curves are almost linear but locally there are some small variations in the slope, which are the result of the premature collapse of the flange of the U stiffeners, developing large out of plane flange deformations during the loading path. The buckling of the flange plating is documented in Fig. 15 and leads in the end to the total failure of the panel. The figure also shows that the collapse is local and the rest of the panel retains the initial configuration.

Local irregularities in the stress shortening curve of the last cycle of loading may be observed for both tests around the ultimate buckling stress of the flange of the U stiffener (133 MPa); close to 220 MPa, that is a little above the yield stress of the stiffener; and 270 MPa of the average panel stress.

For US4A panel, one may identify a marked softening of the structure above 370 MPa of average stress, which may be responsible for the little difference in the shape of the curves at collapse between A and B experiments.

In Table 2 the summary of results in these series of tests is presented and comparison with the equivalent and stiffener yield stresses is made.

The main conclusion from this table can be drawn from the analysis of the last column, indicating that the total yield of the weakest panel material does not lead to the collapse of the structure, because the ratio between the ultimate stress and minimum yield stress of panel material is well above 1, reaching 2 for US panels.

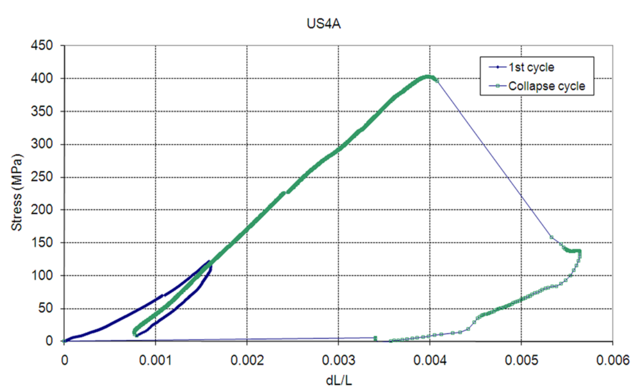

Fig. 13 Average stress versus average shortening curve of US4A panel

Comparison With Design Formulas

An upper limit for the ultimate strength of stiffened panels can be estimated by considering only the buckling of the plating and assuming that the stiffener may sustain the load until its yielding. This upper limit can be estimated bt modifying Eq. (5) by affecting the term related to the plating squash load with a reduction factor. The ultimate strength of the plating given by Eq. (2) was used, and the upper limit becomes

$$\phi_{ul} = \frac{\sigma_{ul}}{\sigma_o} = \frac{\phi_p \cdot \sigma_{Yp} A_p + \sigma_{Ys} A_s}{\sigma_{Yp} A_p + \sigma_{Ys} A_s} \tag{11}$$

As mentioned before, ϕ_p is 0.702 for all plates, leading to ultimate plating stress of 484 MPa, because they have the same geometry and are made of the same material. However, the stiffener's cross-sectional area varies for different stiffener geometries, according to Table 1, and the material properties of the different stiffeners are different.

The ultimate strength may be predicted by modifying the Euler stress for columns to account for plastic effects, applying the Johnson-Ostenfeld approach, which may be expressed by Eq. (12) when the Euler stress [Eq. (3)] is higher than the proportional stress, normally taken as 50% of the yield stress, σ_o, or equal to the Euler stress when it is lower than that value. There is not much reference in literature about the yield stress to be used when

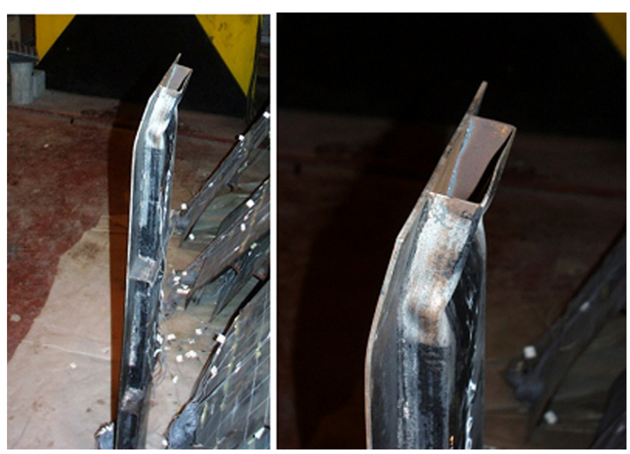

Fig. 15 US4A panel after collapse

Table 2 Summary of results

Panel	Ultimate load (KN)	Ultimate stress (MPa)	Equivalent stress (MPa)	Minimum stress (MPa)	Ultimate stress/ equivalent stress	Ultimate stress/ minimum stress
FS4A	271	199	690	690	0.29	0.29
BS4A	732	436	591	343	0.74	1.27
LS4A	853	515	582	296	0.88	1.74
US4A	669	403	554	200	0.73	2.02
FS4B	653	240	690	690	0.35	0.35
BS4B	1551	462	591	343	0.78	1.35
LS4B	1582	478	582	296	0.82	1.61
US4B	1317	397	554	200	0.72	1.99

dealing with hybrid panels; the use of lower yield stress gives a conservative estimation of the column strength and the use of the high strength steel yield stress originates an upper limit [9]. Thus the authors have used the equivalent yield stress for the calculations, resulting in σ_o equal to σ_{Yeq} in the formulas.

$$\begin{aligned} \phi_{cr} &= \frac{\sigma_{cr}}{\sigma_o} = 1 - \frac{\sigma_o}{4\sigma_E} && \Leftarrow \sigma_E > 0.5\sigma_o \\ \phi_{cr} &= \frac{\sigma_{cr}}{\sigma_o} = \frac{\sigma_E}{\sigma_o} && \Leftarrow \sigma_E \leq 0.5\sigma_o \end{aligned} \tag{12}$$

It is assumed that the radii of gyration, r from Eq. (3), is not very sensitive to the effective width of the associated plate; thus, this critical stress is the average critical stress of the effective stiffened plate at collapse and it should be corrected for the effectiveness of the associated plating. Finally one has the expression for the ultimate strength of stiffened plates given by

$$\phi_{uc} = \frac{\sigma_{uc}}{\sigma_o} = \frac{\sigma_{cr}}{\sigma_o} \cdot \left[\frac{\phi_p \cdot A_p + A_s}{A_p + A_s}\right] \tag{13}$$

Table 3 presents the comparison of the successful tests and the ultimate stress predictions according to the above formulas. The test results are highlighted in italics and the best predictions in bold. Also shown is the nominal column slenderness λ using the equivalent yield stress of the panel, given by

$$\lambda = \frac{L}{\pi r}\sqrt{\frac{\sigma_{Yeq}}{E}} \tag{14}$$

Table 3 summarizes the test results and compares them with different analytical approaches.

All panels have collapsed by the stiffener's induced failure and, thus, the predictions given by σ_{uc} are very conservative compared with the experimental results because the formula considers the reduction in the effective width of the associated plating and that is not correct for this range of column slenderness because the collapse is due to the stiffener's buckling and the level of stress in the plating is far below the yield stress of the plating when collapse occurs. The use of the equivalent yield stress of the hybrid panel in formulas (11) and (12) instead of the plating yield stress leads to a fair estimation of panel strength in the cases considered.

The FS panel presents the highest column slenderness and collapsed by stiffener induced failure in the elastic range at very low axial average stress. The critical stress approach prediction is close to the experimental values; in fact, it is 8% and 30% lower than the narrow and wide panel strength, respectively.

The ultimate stress of BS and LS panels may be predicted by Eqs. (11) or (12) and he experimental values are in between them. Nevertheless, the critical stress approach gives an optimistic prediction, overestimating the strength, and the ϕ_{ul} looks conservative with one exception, the BS4A panel.

Table 3 Comparison with formulas

Panel	λ	σ_u (MPa)	σ_{Yeq} (MPa)	σ_{ul} (MPa)	σ_{cr} (MPa)	σ_{uc} (MPa)	Ultimate strength σ_u/σ_{Yeq}
FS4A	1.934	***199***	690	509	**184**	136	0.288
BS4A	0.226	***436***	591	**444**	471	371	0.738
LS4A	0.120	***515***	582	433	**549**	430	0.885
US4A	0.097	***403***	554	**405**	533	418	0.727
FS4B	1.934	***240***	690	509	**184**	136	0.348
BS4B	0.226	***462***	591	433	**471**	371	0.778
LS4B	0.120	***478***	582	**444**	549	430	0.821
US4B	0.097	***397***	554	**405**	533	418	0.715

Conclusions and Final Comments

The criterion for the design of the panels was to have a similar squash load on all panels. With this criterion, hybrid panels have a better performance than full S690 panels because they have a higher sectional area and inertial moment than FS panels, leading to lower column slenderness and higher critical stress.

The use of S690 on the plating of the panels increases the average ultimate strength on the order of 2 or above when compared with mild steel plating. In this range of slenderness, all panels collapse by the stiffener's induced failure located in the middle bay.

The transverse forces generated by axial compression may reach very high values, which were identified by the noisy collapse, the residual plastic deformation of the frames, and the degradation of the supporting structure. Under longitudinal thrust, the state of stress near the frames is predominantly biaxial, inducing frame bending that may lead to collapse if the frames are not strong enough.

Multispan panel models are much more adequate for testing panels under compression and give more reliable results due to a better control of boundary conditions on the supports. The premature plasticity or buckling of the stiffeners did not originate the collapse of the panels, but in a single span model this is not necessarily true.

On hybrid panels the collapse is reached at much higher stress than the yield stress of the stiffeners. This means that most of the strength of the panels comes from the S690 plating no matter if the stiffeners have already yielded or not, ensuring that they still contribute to maintaining the global geometry.

The best results in terms of ultimate strength were obtained for the LS panels, which are, on average, 20% stronger than the US panels with approximately the same column slenderness; and an average of 10% stronger than the BS ones, but in this case with lower slenderness. The best predictions were obtained using the ultimate column strength approach, Eq. (11) or the Johnson-Ostenfeld approach, Eq. (12), associated with the equivalent stress concept, Eq. (6), columns (4) and (5), respectively, in Table 3. Nevertheless, further discussion is required about the normalizing stress to be used in design of hybrid panels.

The use of formula (13) is completely inadequate for this range of column slenderness. It underestimates the load carrying capacity of the panels due to accounting for the reduction of the effective area of the associated plate, which does not occur during stiffener induced failure.

Acknowledgment

The work presented in this paper was done under the project 'High Tensile Steel 690 in Fast Ship Structures (FASDHTS),' CEC BRITE/EURAM Programme (2000–2004), which was partially financed by the European Union, through the Contract No. GRD1-1999-10558. The partners were: TNO Building and Construction, Centre for Mechanical Engineering (NL), AF Industriteknik AB (S), AG der Dillinger Hüttenwerke (D), Alstom Chantiers de l'Atlantique (F), Bureau Veritas (F), Chalmers University of Techonology AB (S), Flensburger Schiffbau mbH&Co. KG (D), Germanischer Lloyd AG (D), Instituto Superior Técnico (P), Lisnave Estaleiros Navais (P), Royal Schelde group BV (NL), Technische Universität Hamburg-Harburg (D), and Van der Giessen-De Noord shipbuilding division BV (NL).

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