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Buckling of laser-welded sandwich panels: ultimate strength and experiments

Hans Kolsters and Dan Zenkert

*Department of Aeronautical and Vehicle Engineering,
Royal Institute of Technology (KTH), SE-100 44 Stockholm, Sweden.
E-mail: hansk@kth.se, danz@kth.se*

Abstract

This is the last of three companion papers which examine the elastic buckling and collapse of laser-welded sandwich panels with an adhesively bonded core and uni-directional vertical webs. By evaluation of the buckling stress in the first two papers it has been found that the buckling stress in compression parallel and normal to the webs typically reaches the proportional limit of the face plate and web material well before elastic buckling occurs. Hence, this paper presents an extension of the buckling model into the elasto-plastic regime, with the aim to determine the ultimate (local) strength of the sandwich and to allow experimental verification of the results. Using tangent modulus theory to ‘plasticise’ the elastic buckling model, the ultimate strength is evaluated for a sandwich configuration with high-strength steel face plates and a broad range of core moduli. The critical load predicted by the inelastic buckling model agrees well with non-linear finite element results and experimental values obtained from compression testing.

Keywords: Sandwich; Laser welding; Web; Core; Ultimate strength; Non-linear FE; Compression testing

1. Introduction

1.1. Background

This is last of three companion papers which examine the elastic buckling and collapse of laser-welded sandwich panels with an adhesively bonded core and uni-directional vertical webs. In the first paper [1] it has been shown that the critical load in compression parallel to the webs can be accurately predicted by using a Pasternak foundation to model the core and by using a simple clamping factor to describe the boundary conditions at the laser weld. To be able to use the same model in compression normal to the webs, the second paper [2] has

outlined a ‘constrained wrinkling’ approach which accounts for the fact that the face plate can only deform in certain mode shapes. In both cases it has been found that for low modulus core materials the critical load is equal to the bare plate buckling load and for high modulus materials the solution converges toward anti-symmetrical wrinkling. However, it has also been shown that buckling is typically inelastic, as the proportional limit of the face plate and web material is usually reached well before elastic buckling occurs.

Dependent on the direction of the applied load inelastic buckling of the face plate can have different consequences (assuming the face plate and webs have similar material behaviour). In compression parallel to the webs, as shown in Fig. 1(a), the zone affected by yielding is likely to remain confined to only a few adjacent cells because the webs are relatively sturdy and can redistribute the load to a considerable degree. On the other hand, in bending or compression normal to the webs yielding of the face plate can create a plastic hinge that can extend sideways and thereby affect the global stability of the panel, Fig. 1(b). Thus, while inelastic buckling in one direction is basically a *serviceability limit state*, where the panel is damaged but has some post-buckling reserve, in the other direction it constitutes a *collapse limit state*, in which it is no longer able to maintain its primary load carrying role.

Thus, an extension of the buckling model into the elasto-plastic regime is necessary for three reasons. First of all, rule requirements and the present production window of the laser-welding process limit the possibilities to design sandwich panels that are completely elastic. Secondly, when yielding and/or inelastic buckling occurs it depends on the direction of the applied load whether there is a margin of safety or a direct risk of failure, and it is necessary to know how to deal with each particular case. Especially the ability of the core to keep the face plate straight and delay the building up of a plastic hinge is hereby of considerable importance. And last but not least, inelastic effects simply have to be accounted for if the results in [1,2] are to be verified by experiment.

1.2. Practical considerations

Because the possibilities for experimental verification of elastic buckling in laser-welded sandwich panels are limited it is vital that the choice of materials and sandwich dimensions are carefully considered. As can be seen in Fig. 2 the buckling stress parallel to the webs is always above the proportional limit of NV-27 structural steel and exceeds the limit for NV-40 for $E_c > 15$ MPa approximately, while in compression normal to the webs the behaviour remains elastic until $E_c > 25$ MPa and $E_c > 50$ MPa for NV-27 and NV-40 respectively. This means that for a standard sandwich configuration elastic buckling can only be achieved for very light core materials, of which the mechanical properties are generally poorly controlled (e.g., injected polyurethane foam).

If cellular PVC foam core is used, which is available in a variety of grades and densities and of which the mechanical properties are well controlled, the minimum elastic modulus of the core typically ranges from 30 to 300 MPa [4] and a yield strength of around 800 MPa is required if the transition from elastic to inelastic buckling is to fall somewhere in the middle. Alternatively, the buckling stress can be lowered by reducing the plate thickness or increasing the web-pitch, but as the minimum plate thickness required for laser welding is 1.5 mm and the buckling stress for $t_f = 1.5$ -2.0 mm and $2p \geq 120$ mm is almost constant for $E_c > 30$ MPa this is not of any practical use.

Fig. 1. In-plane compression of a laser-welded sandwich panel parallel and normal to the webs.

Fig. 2. Buckling stress for sandwich plate in compression parallel and normal to the webs. Pasternak foundation model with clamped boundary conditions at the laser weld [1], constrained wrinkling solution [2], and finite element results for 3D periodic model (ABAQUS). The material designations ‘NV-27’, ‘NV-40’ and ‘NV-620’ indicate structural steel with a proportional limit of no less than 265, 395, and 615 N/mm² respectively [3].

Thus, it has been decided for this investigation to refit the standard sandwich panel with Docol 800 DP high-strength steel face plates (thickness 1.8 mm). Docol DP is a cold-reduced, high strength steel manufactured by SSAB Swedish Steel that derives its properties from a dual-phase (DP) microstructure containing martensite and ferrite, that give the material its strength and formability [5]. Most importantly, DP steels are supplied with a guaranteed minimum tensile strength from 400 N/mm² and upward, and because the difference between the yield strength and tensile strength is fairly large this will result in a well controlled inelastic buckling trajectory. Besides, DP steels can be laser-welded to mild steel without difficulties, which has also been verified in production trials, and laser-welded joints typically show the same strength as the parent material.

1.3. Aims and scope

To make the findings in [1,2] more practically relevant this paper examines the (in)elastic buckling behaviour of laser-welded sandwich columns with high-strength steel face plates and an adhesively bonded PVC foam core. The elastic buckling models derived in [1,2] are extended into the elasto-plastic regime by using tangent modulus or Engesser theory and the (in)elastic buckling stress is evaluated for both compression cases and global buckling. The

3D finite element model that has been used before is adapted to experimental specifications, and non-linear material and geometrical behaviour are incorporated. A series of compression tests is performed and the load-displacement history is compared to analytical and finite element results. For the case of compression normal to the webs a new method is devised in which only one of the face plates is loaded (compression-bending), to avoid the risk of global (shear) buckling or adhesive failure.

2. Analytical modeling

Consider a laser-welded sandwich plate element with face thickness t_f , core thickness t_c , distance between the centroids of the faces d , web thickness t_w , web-pitch $2p$, and elastic moduli E_f , E_w , and E_c , as shown in Fig. 3.

2.1. Elastic buckling

Compression parallel to the webs

In compression parallel to the webs the buckling stress of the face plate is given by

$$\sigma_{cr} = k_{cr} \frac{\pi^2 D_f}{b^2 t_f} \quad (1)$$

where b is the width of the plate ($b=2p$), D_f is the bending stiffness and the critical buckling coefficient has been derived as [1]

$$k_{cr} = 2\sqrt{n^4 + f_W + n^2 f_P + f_P + 2n^2} \quad (\text{one wave in } y\text{-direction}) \quad (2)$$

Here, n^2 is the forementioned clamping factor which for clamped boundary conditions at the laser weld equals 1.745. The foundation parameters f_W and f_P are defined as

$$f_W = \frac{b^4 k_W}{\pi^4 D_f} \quad f_P = \frac{b^2 k_P}{\pi^2 D_f} \quad (3)$$

and the (reduced) foundation stiffness k_W and k_P for anti-symmetrical wrinkling are

$$k_W = \frac{1}{6} E_c t_c \quad k_P = \frac{1}{3} G_c t_c \quad (4)$$

Compression normal to the webs

In compression normal to the webs the plate buckling stress can be calculated from Eq.(1) as well, but according to the ‘constrained wrinkling’ approach outlined in [2] the critical buckling coefficient is determined by

Fig. 3. Sandwich plate element and characteristic dimensions.

$$k = n^2 \left(\frac{b}{l} \right)^2 + \frac{1}{n^2} f_w \left(\frac{l}{b} \right)^2 + f_P \quad (5)$$

where l is the effective buckling wavelength and k_{cr} is found by evaluating k for different values of n^2 and l and picking the smallest value. Instead of a fixed value for n^2 , the clamping factor is dependent on the critical mode shape and for clamped boundary conditions at the laser weld the first four mode shapes and the corresponding values of n^2 and l are shown in Fig. 4.

2.2. Inelastic buckling

If the buckling stress exceeds the proportional limit of the face plate material it is customary to replace the elastic modulus E by the tangent modulus E_{tan} and use the stress-strain relation of the material to determine the position of the stability point. This point is defined as where the inelastic buckling load, which by E_{tan} is a function of strain, equals the applied load, also as a function of strain.

Compression parallel to the webs

Assuming that both face plates are loaded equally and the webs remain straight, which is not unreasonable due to their low slenderness, the inelastic buckling load per unit width is equal to

$$P_{ib,x} = 2\sigma_{crf,x}t_f + \sigma_w t_c \left(\frac{t_w}{2p} \right) + \sigma_c t_c \left(1 - \frac{t_w}{2p} \right) \quad (6)$$

Fig. 4. Clamping factor and buckling wavelength for the solid-drawn part of the plate - constrained wrinkling.

where the first term is the ‘plasticised’ buckling load of both face plates. The buckling stress $\sigma_{crf,x}$ is thereby determined from Eq.(1) and (2), where the bending stiffness D_f is now no longer a constant but a function of the tangent modulus $E_{tan,f}$

$$D_f = \frac{E_{tan,f} t_f^3}{12(1 - \nu_f^2)} \quad (7)$$

The last two terms are the plastic loads in the webs and the core, where the stresses σ_w and σ_c can be written as a linear function of the strain in the face plate ($\sigma_{w,c} = E_{w,c} \varepsilon_f$) or by using engineering stress-strain curves if these are available, $\sigma_{w,c} = \sigma_{w,c}(\varepsilon_f)$.

By writing the stress in the face plate as a function of strain as well, $\sigma_{f,x} = \sigma_{f,x}(\varepsilon_f)$ and the applied load per unit width is equal to

$$P_{a,x} = 2\sigma_{f,x}t_f + \sigma_w t_c \left(\frac{t_w}{2p} \right) + \sigma_c t_c \left(1 - \frac{t_w}{2p} \right) \quad (8)$$

Compression normal to the webs

Analogous to Eq.(6) the inelastic buckling load in compression normal to the webs is equal to

$$P_{ib,y} = 2\sigma_{crf,y}t_f + \sigma_c t_c \quad (9)$$

where the buckling stress $\sigma_{crf,y}$ is now determined from Eq.(1) and (5), and D_f is a function of $E_{tan,f}$ as before. Similarly, the applied load is equal to

$$P_{a,y} = 2\sigma_{f,y}t_f + \sigma_c t_c \quad (10)$$

Global buckling

Global buckling beyond the proportional limit can be treated in the same way as local buckling of the face plate alone, by using, e.g., the plate buckling solution in [6] and using the tangent modulus for the constituent materials to evaluate the stiffness properties in [7]. Although it is customary, when the sandwich panel or column first departs from the straight position, to account for the loading and unloading of either side (due to bending) by, e.g., a reduced modulus E_{red} , it is assumed here that by the action of the webs the panel remains perfectly straight. Besides, the tangent modulus is generally only slightly conservative with respect to the reduced modulus and considerably easier to use, as it only depends on the magnitude of the stress and not the shape of the cross-section.

2.3. Numerical evaluation

To find the stability point where the inelastic buckling load equals the applied load, the tangent modulus and stress for the constituent materials are expressed as a function of strain, using experimental stress-strain curves where necessary. Subsequently, both loads can be evaluated and plotted as a function of strain and the stability point can be found graphically.

2.3.1. Material models

Besides Docol 800 DP high-strength steel face plates, the sandwich consists of ST37K steel webs and Divinycell H-grade PVC foam core.

Core material - stress-strain relation

Divinycell H-grade is a partially cross-linked structural PVC foam core material with predominantly closed cells, which is manufactured by Diab [4] in a baking process yielding rectangular blocks with densities between 30 and 300 kg/m³. Although PVC foam core, much like other cellular foam core materials, behaves differently in tension and compression, the maximum strain of interest here is approx. 2% and thus the core material can be assumed to behave linearly elastic ($\sigma_c = E_c \varepsilon_f$).

Face plate and webs - stress-strain relation

In an earlier investigation [8] the stress-strain behaviour for Docol 600 DP and 800 DP was obtained by tensile testing of standard coupons and compressive testing of short sandwich columns with a 350 kg/m³ PVC foam core. The material behaviour was practically the same in tension and compression and hence only coupon tests have been performed to reestablish the stress-strain relation 15 years after. Similarly, only coupon tests have been used to obtain the stress-strain relation for ST37K steel.

Tensile tests have been performed on 250 mm x 25 mm coupons using an Instron 4505 universal testing machine, and strains have been recorded using both an extensometer and strain gauges. The measured stress-strain relation for the face plate and web material is shown in Fig. 6.

Face plate and webs - material behaviour

Structural steels typically demonstrate a linear elastic and strain-hardening plastic behaviour without a sharply defined initial yield point, which can be described by writing the total strain ε as the sum of a linear elastic component $\varepsilon_E = \sigma/E$ and a plastic component ε_P , e.g. [9]

$$\varepsilon = \frac{\sigma}{E} + 0.002 \left(\frac{\sigma}{\sigma_{0.2}} \right)^n \quad (11)$$

Here, $n = 0.301/\log(\sigma_{0.2}/\sigma_{0.1})$ and $\sigma_{0.1}$ and $\sigma_{0.2}$ are the nominal stress for 0.1% and 0.2% strain offset respectively; $\sigma_{0.2}$ also being the yield strength and proportional limit.

While Eq.(11) and similar expressions (e.g., [10]) give an accurate description of the stress-strain curve to slightly above the proportional limit, the accuracy decreases rapidly for higher stresses. To improve the accuracy of the solution for up to 2% strain it is possible to write the plastic strain as $\varepsilon_P = a_1(\sigma/a_2)^n$ and determine the material constants a_1 , a_2 and n from a curvefit of tensile test data. Although this works well for structural steel of average strength, e.g. ST37K, this does not work for high strength steel such as Docol 800 DP due to the considerable difference between the yield strength and the ultimate tensile strength of the material. Typically, a curvefit of tensile test data for high strength steel using Eq.(11) yields the correct ultimate strength but also a non-conservative estimate of the yield strength, and it has been found that a more accurate curvefit is obtained by using

$$\varepsilon = \frac{\sigma}{E} + a_1 \left(\frac{\sigma}{\sigma_{0.01}} \right)^{n_1} + a_2 \left(\frac{\sigma}{\sigma_{0.01}} \right)^{n_2} \quad (12)$$

where $\sigma_{0.01}$ marks the yield strength for 0.01% strain offset.

The resulting numerical fit of tensile test data for Docol 800 DP and ST37K respectively is shown in Fig. 5 and the corresponding mechanical properties are presented in Tables 1 and 2.

2.3.2. Tangent modulus and stability point

The tangent modulus is the slope of the stress-strain curve in Fig. 5, which for small strains is equal to the elastic modulus of the material and rapidly decreases to zero as the strains exceed

Fig. 5. Stress-strain relation for Docol 800 DP steel (face plates) and ST37K steel (webs).

Table 1
Mechanical properties of Docol DP high-strength steel *.

	E	$\sigma_{0.01}$	$\sigma_{0.1}$	$\sigma_{0.2}$	σ_{ult}^{**}
Docol 800 DP	[GPa]	[MPa]	[MPa]	[MPa]	[MPa]
Measured data	204.6	389	583	660	972
Numerical fit	204.6	419	599	666	956
Manufacturer*	205	-	-	720 ± 80	890 ± 80

*) In rolling direction, typical value.

**) At 5% strain.

Table 2
Mechanical properties for ST37K structural steel.

	E	$\sigma_{0.01}$	$\sigma_{0.1}$	$\sigma_{0.2}$	σ_{ult}^*
ST37K	[GPa]	[MPa]	[MPa]	[MPa]	[MPa]
Measured data	203	344	381	388	408
Numerical fit	203	373	382	389	408

*) At 5% strain.

the proportional limit and the material becomes fully plastic, as shown in Fig. 6(a). By substitution of the tangent modulus in Eqs.(6-10) the inelastic buckling load and the applied load can simply be evaluated and plotted as a function of strain, and the stability point can be found graphically, Fig. 6(b).

2.3.3. Inelastic buckling load

The position of the stability point is evaluated for one set of sandwich scantlings, clamped boundary conditions at the laser weld, and 41 core compressive moduli E_c ranging from 0.1 to 1000 MPa with equal spacing on a $\log(E_c)$ scale. For further discussion these 41 ‘specimens’ are denoted #1 to #41, and by means of example the corresponding core moduli for specimens 21 to 31 (most practically relevant) are presented in Table 3. With regard to the experimental investigation global buckling considers a short column with length $L=2*2p$.

Compression parallel to the webs

The inelastic buckling stress in compression parallel to the webs is presented in Fig. 7(a). Although the elastic buckling stress at 291 MPa is well below the yield onset $\sigma_{0.01}$ for both the face plate and web material inelastic buckling already occurs at a slightly lower stress (281 MPa, $\varepsilon_f = 0.14\%$) due to the downward slope of the tangent modulus in Fig. 6(a). As the core modulus increases and the elastic buckling stress passes the point of yield onset and the proportional limit, inelastic buckling occurs at increasingly higher stress until for $E_c=316$ MPa (#36) the buckling stress exceeds the ultimate strength and no stability point can be

Fig. 6. (a) Tangent modulus for Docol 800 DP high-strength steel and ST37K structural steel. (b) Graphical solution of stability point.

Table 3.
Specimen naming convention and core modulus.

Specimen	#21	#22	#23	#24	#25	#26	#27	#28	#29	#30	#31
E_c [MPa]	10	12.6	15.8	20.0	25.1	31.6	39.8	50.1	63.1	79.4	100

Fig. 7. Compression parallel to the webs. (a) Elastic and inelastic buckling solution. (b) Inelastic local and global buckling.

found. However, before that occurs extensive plastic deformation of the face plate has already occurred and as the webs have become fully plastic around $E_c = 25\text{--}32$ MPa (#25-26) it is not surprising that for high core modulus global collapse becomes predominant, Fig. 7(b).

Compression normal to the webs

The case of compression normal to the webs is more pedagogic, as the face plate is very wide and the problem is basically that of a sandwich column. Also, the elastic buckling stress of the face plate is significantly lower than in compression parallel to the webs and the elastic and inelastic buckling solutions overlap up to $E_c = 20\text{--}25$ MPa (#24-25), as shown in Fig. 8(a). As the core modulus increases the inelastic buckling solution gradually diverges from the elastic solution until once more for $E_c = 316$ MPa (#36) the buckling stress exceeds the ultimate strength and no stability point can be found.

To an extent Fig. 8(a) also resembles a plot of the compressive stress of a column as a function of its slenderness ratio L/r , which for a plate on elastic foundation can be freely interpreted as the ratio of buckling wavelength to face plate thickness, or as the ratio of foundation stiffness to bending stiffness. Thus, if the column is slender or the face plate is lightly supported, the buckling stress follows Euler's solution for elastic buckling until for a certain critical value the proportional limit is reached. Then, if the column becomes more sturdy or the foundation stiffness increases, the (wave)length becomes relatively small and buckling becomes inelastic until the ultimate strength is reached and the column or plate collapses.

However, as the buckling stress passes the onset of yield and the proportional limit of the face plates their bending and extensional stiffness decreases considerably and the sandwich can collapse globally, due to fully plastic Euler buckling. This remarkable result is shown in Fig. 8(b) where a double cell sandwich column first fails due to negligible shear stiffness ($P_{cr} = 2\pi^2 D_f / L^2$), then due to elastic-plastic wrinkling, and finally, as the webs and face plates

Fig. 8. Compression normal to the webs. (a) Elastic and inelastic buckling solution. (b) Inelastic local and global buckling for sandwich column with one and two cells.

are sufficiently plasticised, due to global (Euler) buckling once more. In the results and discussion this subject will be elaborated on further.

3. Finite element analysis

3.1. 3D buckling model and non-linearisation

In [1,2] the commercial finite element code ABAQUS [9] has been used to construct a fully periodic 3D finite element model, representative of a sandwich plate element which is two 'cells' wide and four cells long (Fig. 9). The model uses 8-node, second-order shell elements with reduced integration (S8R) for the webs and face plates, and 20-node brick elements (C3D20R) to model the core.

Choice of elements

Strictly speaking, the use of small-strain elements (e.g., S8R) in plasticity analysis can cause an error, as their thickness does not change during the analysis. Instead, finite strain shell elements (e.g., S4R) should be used to account for material and geometric nonlinearity, the spread of plasticity, changes in element thickness, and residual stresses for example. However, it has been verified that the use of S8R elements (instead of S4R) up to 2% global strain does not cause a significant error, which is not surprising as the face plates and webs are relatively thin. In addition, the investigation is mostly interested in the onset of plasticity and not large plastic deformation and hence the choice of elements has not been altered.

Material behaviour

The material behaviour of the face plates and webs has been modeled by an elastic-plastic constitutive model incorporating an isotropic strain-hardening flow rule, using the stress-

Fig. 9. Finite element model of periodic sandwich plate element with core removed [ABAQUS]

strain properties derived earlier. For definition of the plastic part of the material model the true stress and logarithmic plastic strain have been converted from the nominal stress and strain as follows

$$\sigma_{true} = \sigma_{nom}(1 + \varepsilon_{nom}) \quad (13)$$

$$\varepsilon_{ln}^{pl} = \ln(1 + \varepsilon_{nom}) - \frac{\sigma_{true}}{E} \quad (14)$$

Geometrical non-linearity

For simplicity, the finite element analysis has been divided in two steps; the first step being a displacement controlled linear (bifurcation) buckling analysis to determine the mode shape for the first buckling mode, and the second step being a modified Riks procedure that has used this mode shape as an initial imperfection field for limit load and post-buckling analysis. To verify the satisfactory convergence of the post-buckling solution the analysis has been performed for three levels of imperfection, with a maximum out-of-plane displacement equal to 0.01, 0.10 and 0.25 times the face plate thickness.

3.2. Simulation of experiments

Regarding the computational effort involved and the aim for experimental validation it has been decided to only simulate the planned experiments, i.e. compression of a sandwich column parallel to the webs and compression-bending of a single sandwich cell normal to the webs. For this purpose, only the relevant parts of the 3D model have been retained, all periodic boundary conditions have been removed, and test conditions have been modeled using displacement control (Fig. 10). The reason for choosing to test a single cell in compression-bending and not simply in uni-axial compression normal to the webs is explained in the experimental section.

4. Experimental investigation

4.1. Materials and specimen manufacture

As mentioned earlier it was decided for this investigation to refit the standard sandwich panel

Fig. 10. Finite element models for simulation of experiments. (a) Compression of sandwich column parallel to the webs. (b) Compression-bending of single cell normal to the webs. [ABAQUS]

with Docol 800 DP high-strength steel face plates (thickness 1.8 mm), and to use Divinycell H-grade PVC foam core, with densities between 30 and 300 kg/m³.

Core density sampling

As there is always a certain spread of core material properties and the compressive modulus varies accordingly the average density for each material grade was determined from 10 samples each, and used for extrapolation of the core modulus from nominal material data as presented in Table 4.

Adhesive

Because it was not possible to bond larger core blocks into the cavities before welding of the second face plate, it was decided to insert and bond smaller blocks of core material after welding using RenLam (Araldit) LY 5138-2, which is a fairly viscous epoxy adhesive that can easily be poured into narrow spaces and has good gap-filling properties. The failure strain of the adhesive varies from a minimum of 2.2% after a 7 day room temperature cure, to a minimum 7.3% after 8hrs at 80°C, which was considered adequate.

Specimen manufacture

After laser welding the void sandwich panels were cut and milled to testing specifications, ensuring that the loaded edges were as perfectly parallel as possible. The face plate of each compression-bending specimen was given a 3mm overhang beyond the web for load-introduction. Surface preparation prior to bonding consisted of degreasing, gritblasting and sanding, after which each cavity was sealed at one end to allow the adhesive and small blocks of core material to be inserted from the other. The core blocks were therefore cut to give a 1mm gap on all sides, which proved satisfactory. After bonding the core was prevented from 'floating' to the surface by weights and any visible voids were filled. A 7 day room temperature cure was completed prior to further handling.

4.2. Experimental setup, instrumentation and test procedure

All compression tests have been carried using a 2MN Arboga hydraulic press and a prescribed displacement of 0.2mm/min, with the load and cross-head displacement being

Table 4.

Nominal and sampled material properties for Divinycell H-grade PCV foam core.

Grade	$\rho_{c,nom}$ [kg/m ³]	$E_{c,nom}$ [*] [MPa]	$\rho_{c,sampled}$ [kg/m ³]	$E_{c,extrapol.}$ [MPa]
H30	36	20	41.3	28.5
H60	60	60	58.6	57.5
H80	80	85	79.4	92.4
H100	100	125	90.6	111.1
H130	130	175	115.1	152.1
H160	160	230	141.9	197.1
H200	200	310	181.9	264.0
HCP70	300	600	292.6	584.9

*) $E_{c,nom}$ from extensometer ASTM D 1621.

logged by the press itself and additional dial and strain gauges being logged by a data acquisition system. Inconsistencies in the measurement of the cross-head displacement, due to the compliance of the supporting structure and play in the drive system, have been eliminated by calibrating the displacement reading for a number of specimens with different axial stiffness. The onset of buckling has been identified visually, by touch, and by inspection of load-displacement as well as strain readings (Fig. 11).

Compression-bending normal to the webs

Based on the results in [2] it was recognised early on that a special set-up is required to evaluate the buckling behaviour in compression normal to the webs, without the risk of global and/or shear buckling. Thus, two separate tests have been carried out; the first a compression-bending test of a single cell, Fig. 12(a) and 12(b), and the second a pure compression test of a sandwich column with two cells, Fig. 12(c). While the latter test is more realistic and can be used, e.g., to capture the cross-over from global buckling to local buckling as shown in Fig. 8(b), the compression of a single face plate has particular advantages.

As has been shown in [2] the (elastic) cross-over from shear buckling of a single cell to constrained wrinkling does not occur before $E_c \geq 125$ MPa; in other words, for standard sandwich configurations shear buckling is predominant for all material grades below H100. Besides, dependent on the applied load and the quality of the adhesive bond premature debonding can take place as well and thus the range of material combinations that can trigger constrained wrinkling becomes fairly limited. However, both shear buckling and bondline failure can practically be eliminated by only compressing a single face plate, which by its attachment to the web induces a small bending moment (hence ‘compression-bending’). Basically, this bending moment should be sufficient to force the face plate to buckle inward, into the core, rather than outward, with the risk of it ripping away from the core.

A drawback of only compressing one face plate is that the opposite plate remains more or less straight, which increases the foundation stiffness (compared to anti-symmetrical wrinkling) and slightly raises the buckling stress for intermediate core moduli. Also, due to type of loading only symmetric/uneven buckling modes can occur (see Fig. 4), but this does not outweigh the advantage of knowing how and where buckling is likely to occur.

Fig. 11. Positioning of strain gauges. (a) Compression parallel to the webs. (b) Compression-bending normal to the webs.

Fig. 12. Compression tests normal to the webs. (a) Compression-bending of a single cell. (b) Load introduction and position of strain gauges. (c) Pure compression of a sandwich column with two cells.

5. Results and discussion

5.1. Selection of results

In linear (bifurcation) buckling analysis, the critical load is typically compared to the load corresponding to the first eigenvalue (finite elements) or to the load corresponding to strain reversal (experiment), both of which can be determined fairly simply.

The situation changes for inelastic buckling analysis as the definition of the stability point is not based on a specific amount of plastic deformation. Strictly speaking, it should be possible to evaluate the stability point analytically and determine the critical strain, and use this particular strain level to define a prescribed displacement which can be used to find the

‘exact’ critical load using finite elements and/or experiment. Another possibility is to simply scour the results database of each individual finite element calculation for the load corresponding to a number of physically relevant criteria, e.g. the first onset of yield, divergence of the load-strain relation, strain reversal at an outward buckle, etc.

However, during the course of this investigation no consistent way has yet been found to determine the appropriate critical load for comparison with the inelastic buckling load and hence only the ultimate load is used as a reference. In compression parallel to webs this may overestimate the inelastic buckling load due to a certain amount of post-buckling reserve but in compression-bending normal to the webs the ultimate load should basically be the same.

5.2. Compression parallel to the webs

As can be seen in Fig. 13(a) the ultimate load P_{ult} overestimates the inelastic buckling load as long as buckling is elastic, up to $E_c \sim 20$ -30 MPa, but as soon as the 0.01% yield strength is exceeded the ultimate load is roughly equal to the inelastic buckling load. In Fig. 13(b) the experimental results, which are all in the inelastic range, are close to the inelastic buckling load as well, while the results for high core moduli are due to debonding and extensive plasticity in the webs. ($\sigma_{0.01}$ of the face plates $> \sigma_{0.2}$ webs).

The scatter of non-linear finite element results for $E_c \sim 20$ -100 MPa is due, among others, to an (elastic) buckling mode shift from mode I to mode II between #25 and #26, as shown in Fig. 14(a). For #26 the face plate buckles in four half-waves (Fig. 15) and the core modulus is 31.6 MPa, which is close to the sampled modulus of H30 in Table 4 ($E_c = 28.5$ MPa). Thus, a minimum of four half-waves can be expected to show up in all compression tests parallel to the webs. More importantly, according to the linear-elastic model the mode shift from mode II (4 half-waves) to mode III (5 half-waves) takes place between #31 and #32, where the local buckling stress is close to $\sigma_{0.2}$. The reduced load-carrying capacity of the face plate at this point means, however, that it is less likely to ‘hop’ into a higher buckling mode and a probable outcome of the experiments is therefore that the face plate buckles in four half-

Fig. 13. Inelastic buckling load per unit width in compression parallel to the webs. (a) Linear and non-linear finite element results. (b) Experimental results from compression testing.

Fig. 14. Inelastic buckling behaviour in compression parallel to the webs. (a) Range of core materials that result in mode I, II, and III type buckling. (b) Load-strain relation from non-linear finite element analysis (initial imperfections defined by first linear eigenmode with amplitude equal to 10% of face plate thickness).

Fig. 15. Buckling mode shapes in compression parallel to the webs for #1 (mode I), #26 (mode II), #32 (mode III), and #35 (mode IV). (a) Linear elastic finite elements. (b) Non-linear finite elements, deformed shape shown for maximum strain [ABAQUS].

waves for all H-grade materials. With a specimen length of 240 mm (twice the web-pitch) this means a buckling wavelength of around 60 mm, which matches with the deformed shape after compression of two specimens with H30 and HCP70 core material, as shown in Fig. 16. On the other hand, despite a similar buckling wavelength, a comparison of the corresponding load-strain relations in Figs. 14(b) and Fig. 17 quickly shows that the structural response and ultimate load are indeed very different, with the H30 column being fairly soft and HCP70 being able to stabilise the face plate up to a much higher strain level.

Fig. 16. Deformed shape of test specimens after compression parallel to the webs. Upward arrows indicate the top of an outward buckle and downward arrows indicate the bottom of an inward buckle. (a) Divinycell H30 ($E_c=28.5$ MPa, ~#26). (b) Divinycell HCP70 ($E_c=584.9$ MPa, ~#39).

Fig. 17. Load-strain relation for test specimens in compression parallel to webs. (a) Divinycell H30. (b) Divinycell HCP70.

5.3. Compression-bending normal to the webs

Because the face plate in compression-bending normal to the webs does not have any post-buckling reserve, apart perhaps from that which can be derived from the compressive strength of the core, the ultimate load should basically be the same as the inelastic buckling load, which Fig. 18 shows to be correct. The reason why the ultimate load is actually less than the predicted inelastic buckling load and the linear-elastic finite element result is due to two things: one being that the ultimate load in the non-linear finite element model is determined for a perturbed structure with an initial imperfection amplitude of 10% of the face plate thickness (~ 0.2 mm), which is relevant for small E_c ; and the other being that in the inelastic buckling model compression-bending is described only by means of the appropriate choice of foundation stiffness and the buckling mode shape of the face plate can be unsymmetric (e.g., mode II). Thus, while the finite element model and test specimens are more or less forced to jump from mode I to mode III, because of the natural tendency of the face plate to buckle inward, the analytical model can assume mode II and thereby exhibit a lower buckling stress.

Looking at the buckling mode shapes in Fig. 19 and the failure modes in Fig. 20 the trend during testing was that specimens with H100 or less failed in mode I, with relatively little

Fig. 18. Inelastic buckling load per unit width in compression-bending normal to the webs. (a) Inelastic buckling solution, non-linear finite elements and experiment. (b) Range of core materials that result in mode I and mode III type buckling.

Fig. 19. Buckling mode shapes in compression-bending normal to the webs. (a) #30, mode 3. (b) #34, mode 4.

plastic deformation, while specimens with H200 and over typically failed in mode IV. These last failures were typically rather explosive, where the face plate would remain straight due to its tendency to buckle inward, but would pop out as soon as a plastic hinge was formed. This is also reflected by the load-strain behaviour in Fig. 21, where the specimen with H100 is a textbook example of strain divergence and reversal, whereas HCP70 is able to support the face plate until sudden collapse.

Fig. 20. Failure modes in compression-bending normal to webs.

Fig. 21. Load-strain relation in compression normal to webs. (a) Divinycell H100. (b) Divinycell HCP70.

5.4. Global buckling - compression normal to the webs

In Section 2.3.3 it has been mentioned that in compression normal to the webs a double cell sandwich column can first fail due to negligible shear stiffness, then due to elastic-plastic wrinkling, and finally due to fully plastic global (Euler) buckling. Although it has not been possible to replicate by experiment the second cross-over from wrinkling back to global buckling, the first cross-over has been found to take place slightly later than indicated by the inelastic buckling solution, at $E_c \sim 100$ MPa. This conclusion is based on an evaluation of the experimental results presented in Fig. 22 and the failure modes in Fig. 23. Here, specimens with H30, H60, and H80 failed due to (elastic) global buckling; specimens with H200 and HCP70 failed to the same type of wrinkling failure as in the compression-bending case, and the specimens with H100 failed in global buckling as well as in shear buckling of a single cell.

Fig. 22. Inelastic buckling load per unit width in global buckling normal to webs. Local buckling, global buckling and experiment.

Fig. 23. Failure modes in global buckling normal to webs.

5.5. Design implications - structural steel face plates (NV-36)

Up to now this investigation has used high-strength steel face plates for the benefit of experimental verification, but laser-welded sandwich panels are typically manufactured with structural steel face plates (DNV category NV-36), with a minimum yield strength of 355 N/mm². Having determined the mechanical properties of the new face plate material (ST12) the inelastic buckling load has been numerically evaluated and is presented in Fig. 24.

By comparison of Fig. 24(a) with Fig. 7(b) and Fig. 24(b) with Fig. 8(b) it can be seen that the local and global buckling trend is virtually the same for both face plate materials, but that the actual cross-over point from local to global buckling shifts down, toward the yield strength of the material $\sigma_{0.2}$, and left, toward lower core moduli.

Table 3
Mechanical properties for ST12 mild steel (~NV 27)

ST12	E [GPa]	$\sigma_{0.01}$ [MPa]	$\sigma_{0.1}$ [MPa]	$\sigma_{0.2}$ [MPa]	σ_{ult}^* [MPa]
Measured data	204.4	296	369	385	465
Numerical fit	204.2	305	352	368	450

*) At 5% strain.

Fig. 24. Inelastic buckling load per unit width, local and global buckling. (a) Compression parallel to the webs. (b) Compression normal to the webs

6. Summary

The inelastic buckling and collapse of laser-welded a sandwich panels with an adhesively bonded core and uni-directional vertical webs has been investigated. Tangent modulus theory has been used to extend an elastic buckling model into the elasto-plastic regime, with the aim to determine the ultimate strength of the sandwich and to allow experimental verification of the results. The ultimate strength has been evaluated for a sandwich configuration with high-

strength steel face plates and a broad range of core moduli. The critical load predicted by the inelastic buckling model agrees well with non-linear finite element results and experimental values obtained from compression testing.

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References

- [1] Kolsters H., Zenkert D., “Buckling of laser-welded sandwich panels. Part I: elastic buckling parallel to the webs”, submitted for publication.
- [2] Kolsters H., Zenkert D., “Buckling of laser-welded sandwich panels. Part II: elastic buckling normal to the webs”, submitted for publication.
- [3] Det Norske Veritas, *Rules for Classification of High Speed and Light Craft*, Part 3 Chapter 2, Hull Structural Design - Steel, Norway, January 1991.
- [4] DIAB, *Divinycell H-grade*, Technical Manual, 2002.
- [5] SSAB Swedish Steel, *Docol UHS - Cold-reduced Ultra High Strength Steels*, 2002.
- [6] Robinson J.R., “The Buckling and Bending of Orthotropic Sandwich Panels with all Edges Simply-Supported”, *The Aeronautical Quarterly*, Vol. 6, No 2, 1955, pp. 125-148.
- [7] Kolsters H., Zenkert D., “Stiffness modeling of laser-welded sandwich panels with vertical webs and core material”, submitted for publication.
- [8] Zenkert D., Olsson K.-A., “DP-Sandwich - the utilization of thin high-strength steel sheets in compression”, *Thin Walled Structures*, Vol. 7, 1989, pp.99-117.
- [9] Hill H.W., “Determination of stress-strain relations from offset yield strength values”, *NACA Tech. Note 927*, 1944.
- [10] Ramberg W., and Osgood W.R., “Description of stress-strain curves by three parameters”, *NACA Tech. Note 902*, 1943.
- [11] Hibbit, Karlsson and Sorenson Inc., *ABAQUS Theory Manual*, 1998.