NASA TECHNICAL NOTE

NASA TN D-3780

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NASA TN D-3780

COMPRESSION TESTS OF OPEN-FACE TRUSS-CORE SANDWICH PANELS

by George W. Zender

Langley Research Center

Langley Station, Hampton, Va.

NATIONAL AERONAUTICS AND SPACE ADMINISTRATION • WASHINGTON, D. C. • JANUARY 1967

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NATIONAL AERONAUTICS AND SPACE ADMINISTRATION

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SUMMARY

The results of compression tests of 15 open-face truss-core sandwich panels are presented. The experimental buckling stresses are compared with those obtained by the use of orthotropic plate theory. The comparison shows that the open-face truss-core sandwich panel is substantially less efficient than indicated by orthotropic plate theory for panels with design stresses in the region of the local buckling stress. The inefficiency appears to be associated with the reduced stiffnesses near the edges of the panel where the supporting structure interrupts the ribs and core. Better performance might be obtained if the core and ribs of the open-face panel were extended continuously over the supporting structure. In addition, initial imperfections appear to have a significant effect upon the performance of open-face sandwich panels.

INTRODUCTION

The truss-core sandwich type of construction is often the choice of designers for structures in the moderately high range of structural indices. For lower loadings, however, the truss-core sandwich is less attractive because of the thin gages involved, and designers look for other efficient types of construction which involve heavier gages. An interesting structure that evolved from such consideration is known as the open-face sandwich, wherein one of the face sheets of the conventional sandwich is omitted or replaced with transverse ribs appropriately spaced on one side of the panel. The open-face sandwich considered herein consists of a face sheet, a truss core, and transverse ribs.

In order to study the behavior of open-face sandwich panels when subjected to edge compressive loads, tests were performed on a series of 15 panels. The experimental buckling stresses of the panels were compared with computed results obtained with the use of orthotropic plate theory. This paper presents the detailed experimental information and theoretical results used in making the comparison.

SYMBOLS

The units used for the physical quantities defined in this paper are given both in the U.S. Customary Units and in the International System of Units (SI). Conversion factors pertinent to the present investigation are presented in the appendix and in reference 1.

b	width, inches (meters)
D	bending stiffness per unit width or length, inch-kips (meter-newtons)
d	core depth, inches (meters)
E	Young's modulus, kips/inch^2 (newtons/meter^2)
G	shear modulus, kips/inch^2 (newtons/meter^2)
h	equivalent (smeared) thickness of core and face material, inches (meters)
I	moment of inertia, inches^4 (meters^4)
J	torsion constant, inches^4 (meters^4)
k_x	local buckling coefficient (ref. 4)
L	length, inches (meters)
r	number of corrugations
s	rib spacing, inches (meters)
t	thickness, inches (meters)
w	amplitude of buckling wave, inches (meters)
x, y, z	coordinates (see fig. 1)
ϵ	axial strain
θ	corrugation angle, degrees

λ	half-wave length, inches (meters)
μ	Poisson's ratio
σ	stress, kips/inch^2 (newtons/meter^2)

Subscripts:

c	core
e	experimental
f	face
l	local
p	panel
r	rib
x	refers to axial direction
y	refers to transverse direction

TEST SPECIMENS AND METHOD OF TESTING

Details of the 15 panels are given in figure 1 in conjunction with table I. The panels were assembled by spotwelding, and the type of transverse rib stiffening used for each panel is shown in figure 1. Panels 1 to 7 were fabricated inhouse, whereas the remainder of the panels were obtained from an industrial manufacturer. The ends of the panels were finished flat, square, and parallel in an effort to achieve uniform loading by the heads of the testing machine during tests. Approximately 3/8 inch (0.95 cm) of each end of the panels was cast in a plastic material before finishing to aid in obtaining a smooth end surface.

The panels were subjected to compressive loads as shown in figures 2 and 3. The unloaded edges of the panels were supported by edge fixtures of the types shown in figures 4 and 5. The edge fixtures were made slightly shorter than the panels in order to prevent the testing machine from loading the fixtures. Thus, the testing machine load was supported by the panels alone except for friction loads, which were assumed negligible.

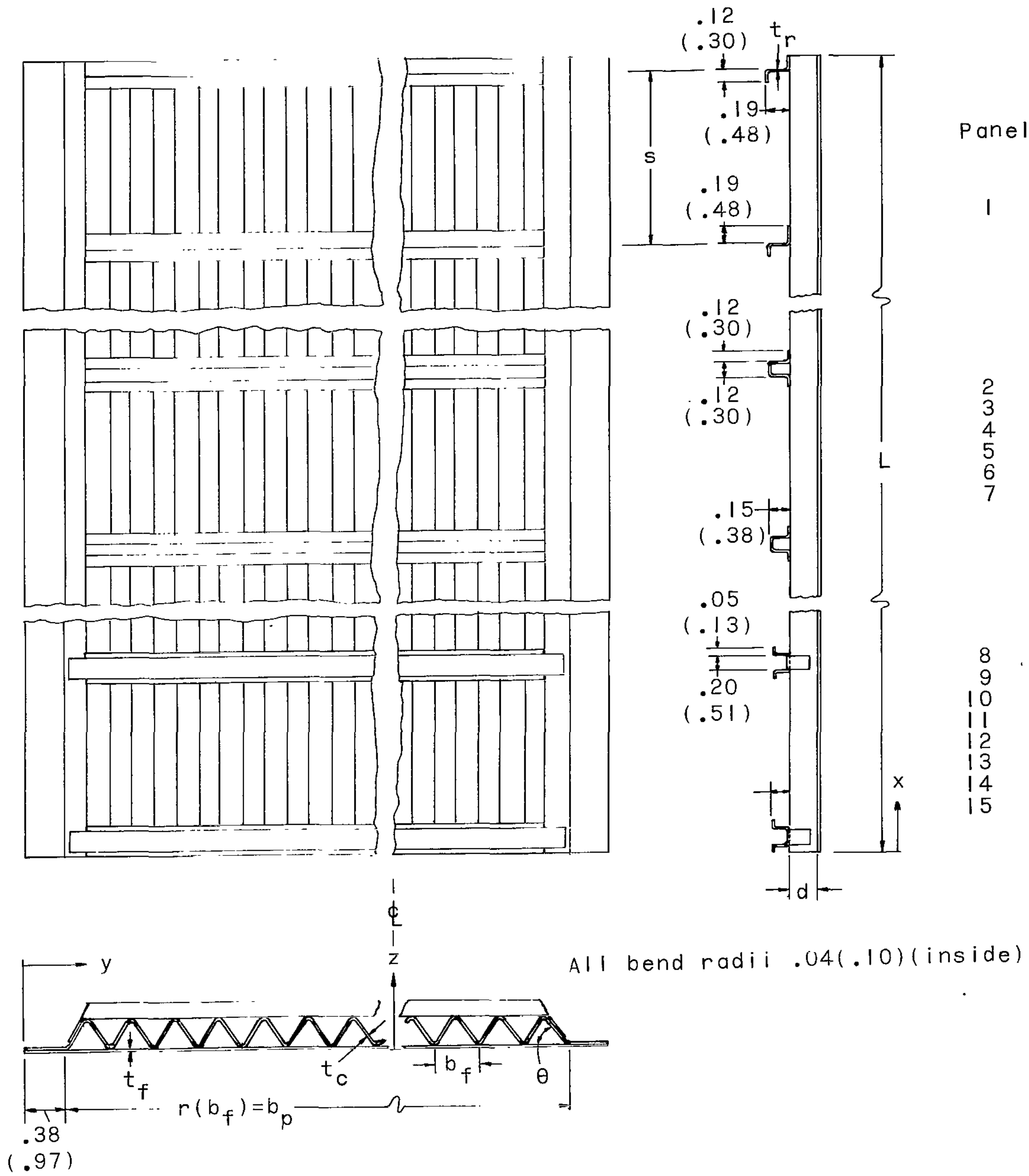
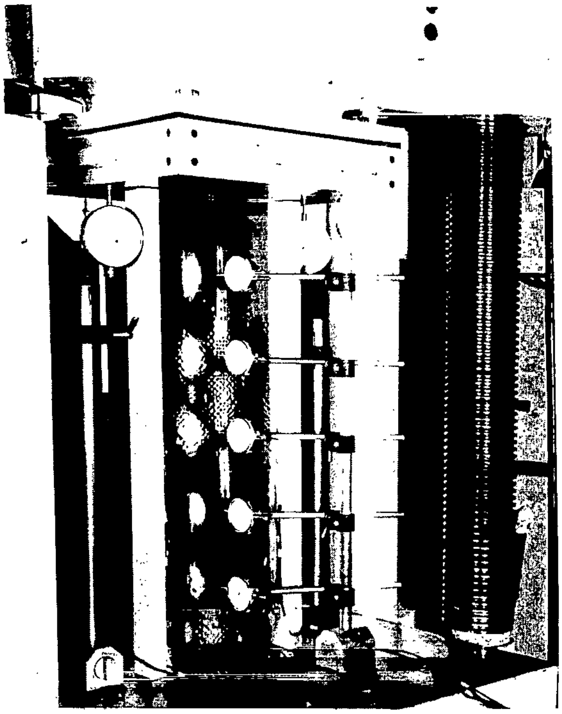
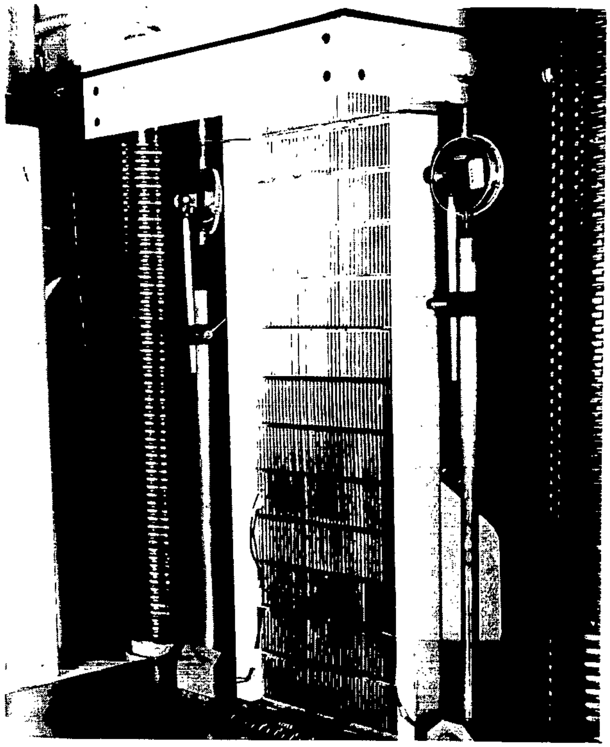


Figure 1.- Details of panels. Dimensions are given in inches and parenthetically in centimeters. See table I for symbol dimensions or numbers.



L-58-72
Figure 2.- Face side of panel 1. σ = 32 ksi (220 MN/m^2).



L-58-73
Figure 3.- Rib side of panel 1. σ = 32 ksi (220 MN/m^2).

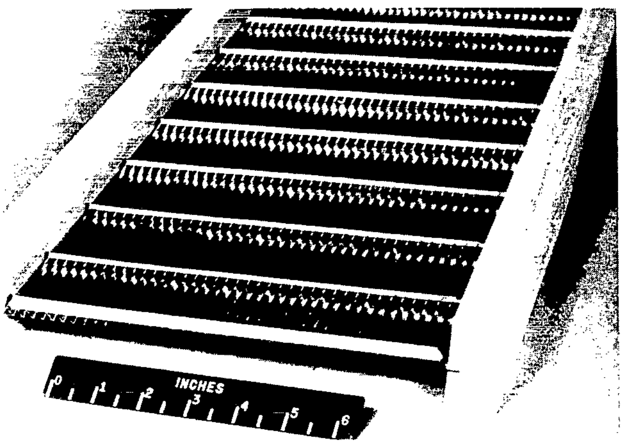


Figure 4.- Mahogany edge supports. L-58-22

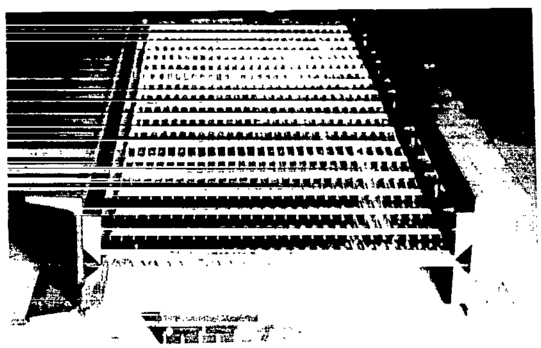


Figure 5.- Steel edge supports. L-62-8344

Panels 1 to 7 were supported along the unloaded edges by mahogany strips (fig. 4). The cross section of the mahogany strips was 1 inch by 2 inches (2.5 by 5 cm) for panels 1 to 5, and 2 by 4 inches (5 by 10 cm) for panels 6 and 7. A snug-fitting slot was provided in the strips to accommodate the projecting flange of the face and core material. Panels 8 to 15 were supported by steel knife-edge fixtures (fig. 5). The support line provided by the steel fixtures for panels 8 to 15 was just outboard of the outermost corrugations. Shortening strain was obtained from the average of the values of four A-9 type SR-4 strain gages located near the panel edges, and lateral displacements were obtained from dial gages, as shown in figure 2. Measurements were obtained for increasing values of load level until the panels were well into the buckling range.

EXPERIMENTAL RESULTS

Experimental results for the panels are shown in figures 6 to 11. The average stress σ on the panel is shown as the ordinate of each figure. The left-hand abscissa of each figure shows the amplitude of the buckles of the panel w divided by the core depth d. The amplitude was obtained from plots of the lateral deformations of the panel center line as indicated in figure 12. The right-hand abscissa of figures 6 to 11 shows the unit shortening or edge axial strain, and the initial slope was used as the modulus of elasticity for the panels as tabulated under E in table II. The value of E for panel 8 was assumed the same as for the similar panels 9 to 15 since strain data were not obtained for panel 8. The deviation of the strain plots from linearity is a result of the reduced axial stiffness of the panel caused by buckling and should not be confused with plasticity since the stress levels were well within the elastic range at buckling. Thus, the left-hand curves of each figure show the growth of buckles with increasing load or stress, and the right-hand curves show the growth of axial deformation.

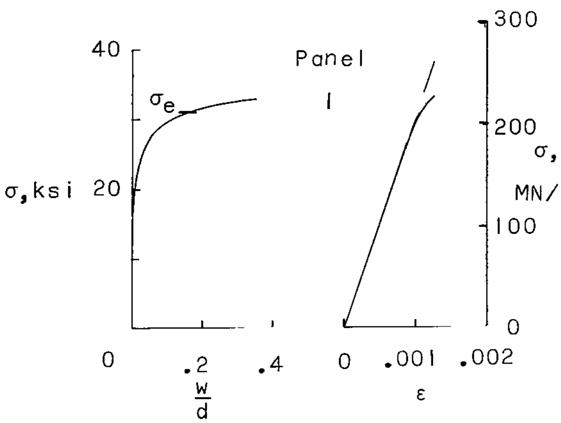


Figure 6.- Buckling amplitude and edge axial strain of panel 1.

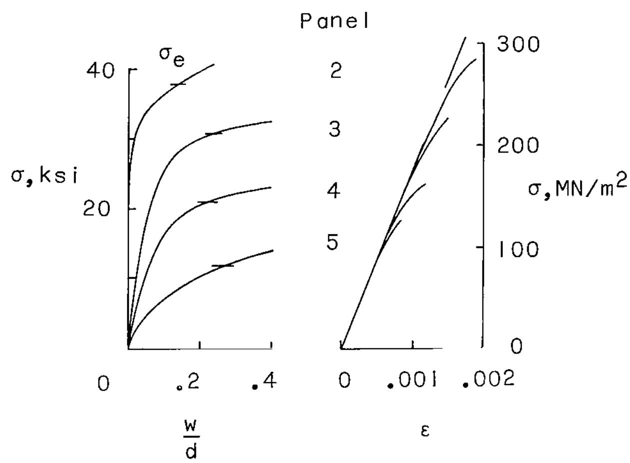


Figure 7.- Buckling amplitude and edge axial strain of panels 2, 3, 4, and 5.

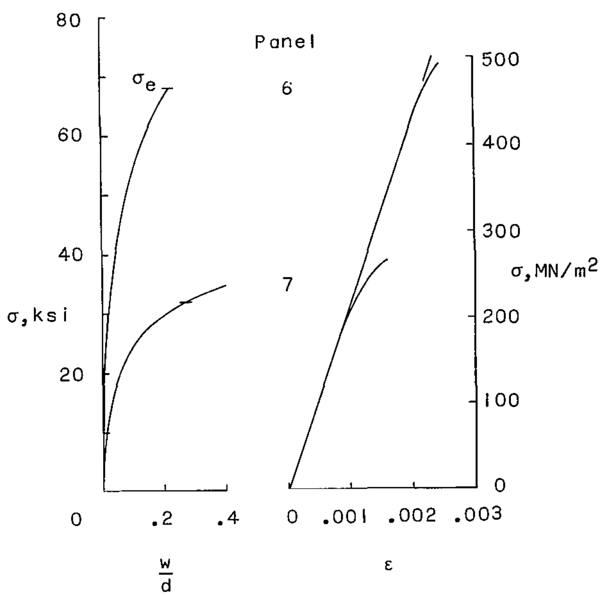


Figure 8.- Buckling amplitude and edge axial strain of panels 6 and 7.

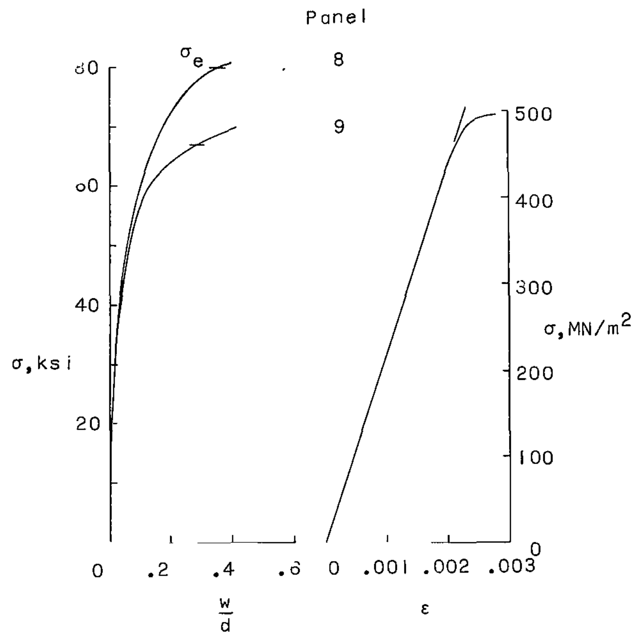


Figure 9.- Buckling amplitude and edge axial strain of panels 8 and 9.

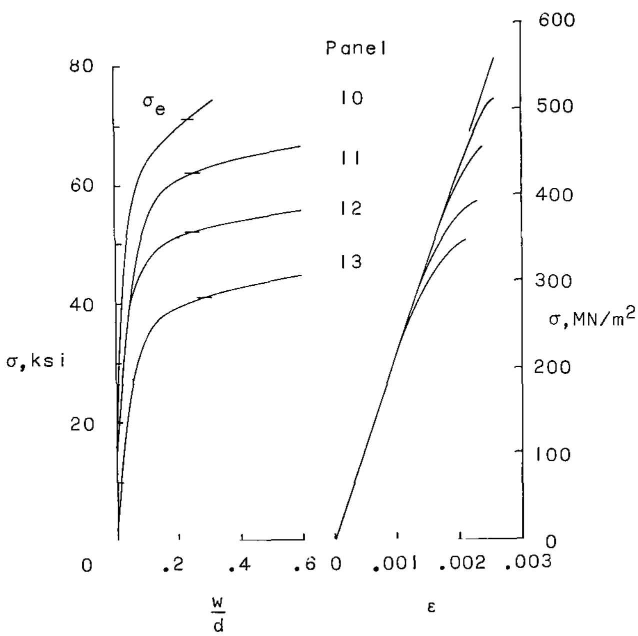


Figure 10.- Buckling amplitude and edge axial strain of panels 10, 11, 12, and 13.

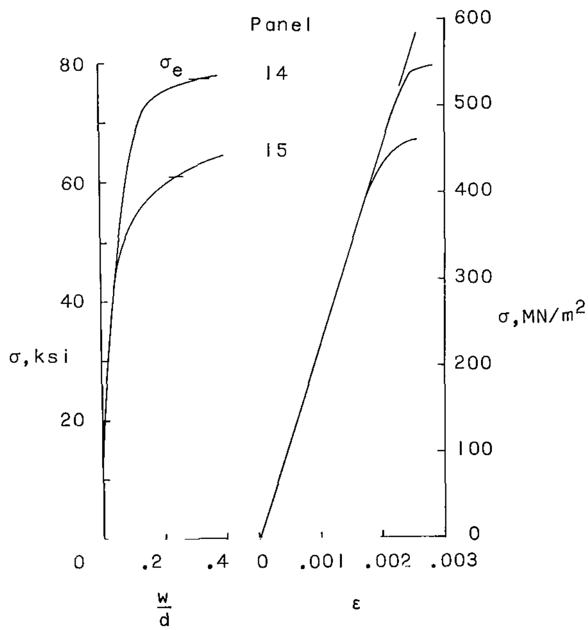


Figure 11.- Buckling amplitude and edge axial strain of panels 14 and 15.

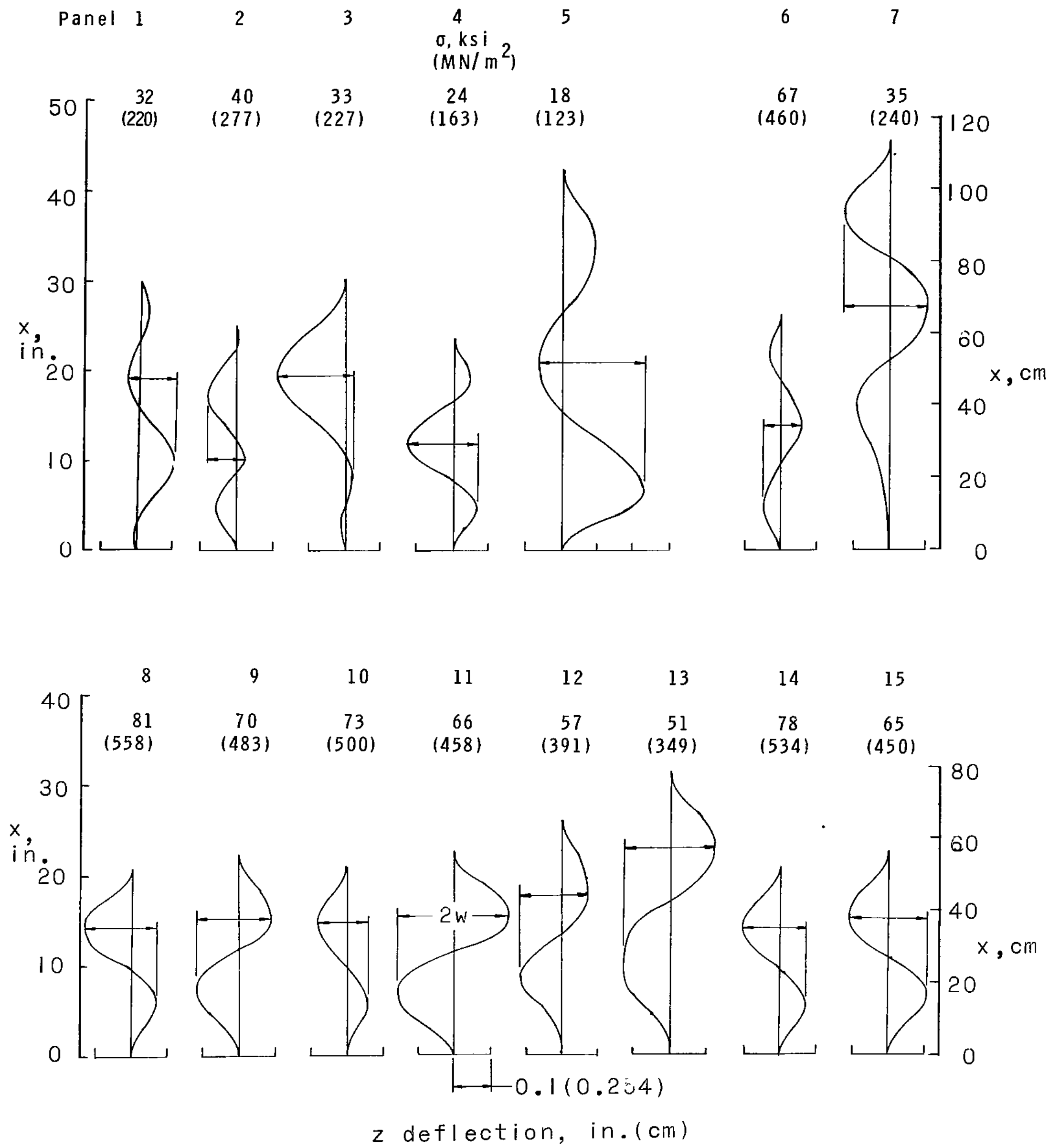


Figure 12.- Buckling modes of panels.

The experimental buckling stress σ_e is indicated by a short line on each of the buckling plots and is tabulated in table II. The value was selected by using the "top-of-the-knee method." (See ref. 2.) Results of tests of open-face corrugated-core sandwich panels without transverse ribs are reported in reference 3 in which buckling stresses were obtained by the Southwell method (ref. 4.) As shown in reference 2, the Southwell method is not generally satisfactory for determining the experimental buckling stress of plates. In the tests reported herein, the Southwell method yields results in better agreement with theory, but at stresses far into the buckling range beyond those to which the panels were subjected.

THEORETICAL ANALYSIS

The theoretical buckling stresses for the panels are tabulated in table II under the headings σ_p for panel buckling and σ_l for local buckling. Panel buckling (general instability) was determined from the following relation (eq. (233) of ref. 4) for simply supported orthotropic plates:

$$\sigma_p = \frac{2\pi^2}{b_p{}^2 h}\left(\sqrt{D_x D_y} + D_{xy}\right) \tag{1}$$

where the stiffness constants were taken as

$$D_x = \frac{EI_x}{b_f} \qquad (I_x \text{ from face and core material over length } b_f)$$

$$D_y = \frac{EI_y}{s} \qquad (I_y \text{ from face and rib material over length } s)$$

$$D_{xy} = \frac{GJ}{2b_f} \qquad (J \text{ for torque box formed by face and core material})$$

The stiffness constants for the panels are tabulated in table II. Experimental values of the stiffness constants D_x and D_y were obtained in several instances from pure bending tests of the panels or specimens similar to the panels. Such tests verified the expression given for D_x but yielded transverse stiffnesses approximately 14 percent greater than those obtained by using the equation given for D_y, probably because of the truss-type stiffness provided by the core material which is not included in the expression for D_y. Since inclusion of this effect would add to the unconservatism of the theoretical results, as shown in a subsequent section, the more conservative expression for D_y as given was employed. The validity of the expression given for D_{xy} is indicated in reference 5.

Local buckling was determined by the method of reference 6 from the equation

$$\sigma_l = \frac{k_x \pi^2 E}{12(1 - \mu^2)} \left(\frac{t_f}{b_f}\right)^2 \tag{2}$$

For reasons of symmetry, the local buckling coefficient k_x (listed in table II) for the panels may be obtained directly from the results given in figure 6(a) of reference 6.

DISCUSSION

The experimental and theoretical buckling stresses are compared in figure 13. The experimental stresses σ_e are plotted as the ordinate and the panel buckling stress σ_p as the abscissa, both divided by the local buckling stress σ_l. The experimental data are shown by the symbols, and the solid lines indicate the conditions required for agreement of the experimental and theoretical buckling stresses. The vertical deviation of the symbols below the solid lines is therefore a measure of the amount by which the experimental buckling stress is less than the calculated stress. The deviation is largest in the range where $\sigma_p = \sigma_l$. Efficiency studies of panels are often based on designs in which the panel buckling stresses are set equal to the local buckling stresses. It is apparent that such studies would be overly optimistic in the case of the open-face sandwich panels because only about 70 percent of the local buckling stress would be developed by such designs. In the present case, the most stress sustained by any panel was 80 percent of the local buckling stress; this stress was attained for a panel in which the design panel buckling stress was 1.26 times the local buckling stress.

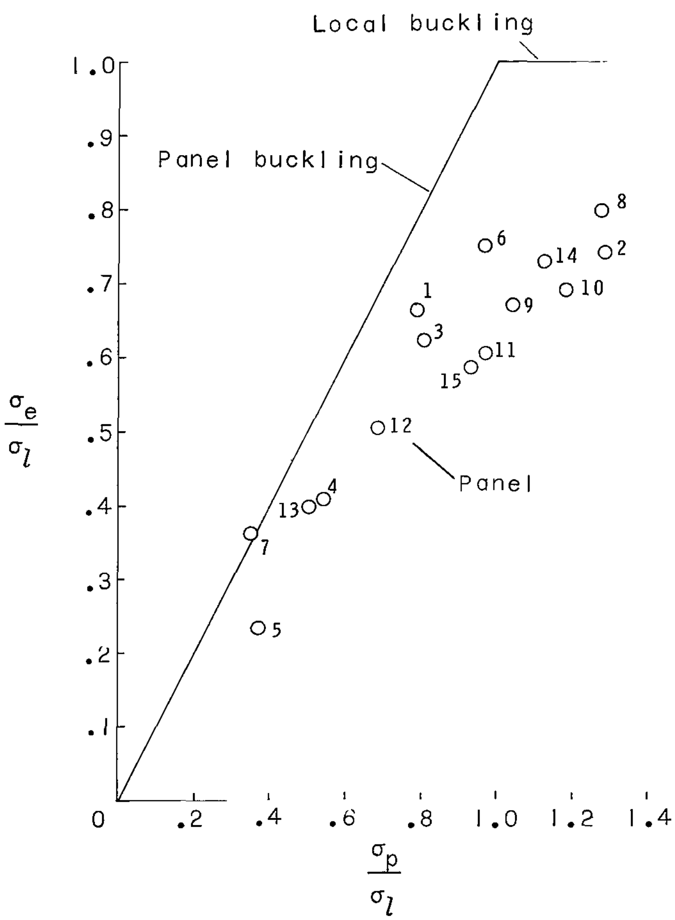
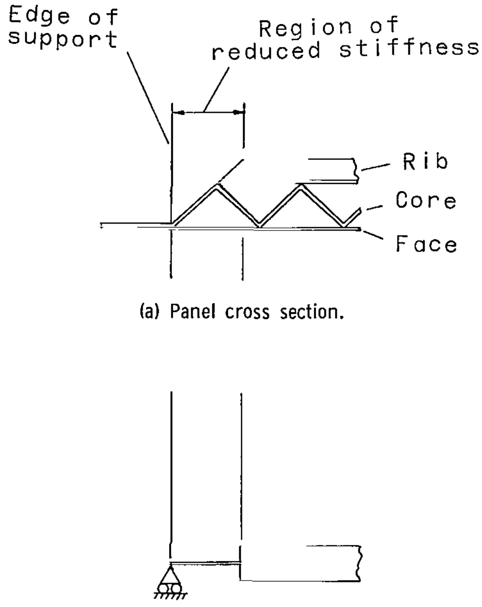


Figure 13.- Comparison of experimental and theoretical buckling stresses.

Several limited studies were performed in an attempt to resolve the discrepancy of experiment and theory indicated in figure 13. Calculations which included the effect of eccentricities by the method of reference 7 resulted in theoretical buckling stresses substantially the same as those presented herein. Similar results were obtained when the effect of shear deformations were included by using the method of reference 8 with the orthotropic shear stiffnesses as defined in reference 9.



(a) Panel cross section.

(b) Substitute structure.

Figure 14.- Edge detail of actual and substitute structures.

The effect of the reduced transverse bending stiffnesses near the edges of the panels was studied briefly and appears to be the most likely explanation for the deviation of theory and experiment. As indicated in the sketch of figure 14(a), the stiffnesses are reduced from the theoretically assumed value in the region between the edge of the rib and the support line. Approximate calculations of the buckling stresses of the substitute structure shown in figure 14(b) yielded results which indicate that the simply supported edge conditions (buckling stress coefficient = 4) were not achieved experimentally. Such results indicate the desirability of extending the core and ribs of open-face sandwich panels continuously over the supporting structure in design practice rather than interrupting the ribs and core. The importance of edge stiffness, which has been recognized previously for dynamic loadings (ref. 10), also appears significant in the case of buckling of open-face sandwich panels subjected to static loads.

A large effect of initial imperfections on the performance of open-face sandwich panels is suggested by the irregular mode shapes shown in figure 12. In this respect, more uniform mode shapes appear for panels 8 to 15, which were fabricated by an experienced industrial manufacturer. It should be noted, however, that although all the panels were designed to buckle into mode shapes containing three half-waves in accordance with the equation $\frac{\lambda}{b} = \sqrt[4]{\frac{D_x}{D_y}}$, the mode shapes for panels 8 to 15 contain only two half-waves.

CONCLUSIONS

The results of compression tests of 15 panels indicate that the open-face truss-core sandwich type of construction is substantially less efficient than indicated by orthotropic plate theory for panels with design stresses in the region of the local buckling stress. Approximately 70 percent of the calculated buckling stress was achieved for open-face sandwich panels designed such that the calculated panel buckling stress was in the range of the local buckling stress. The deviation of the experimental from the calculated buckling stresses appears to be associated with the reduced stiffnesses near the edges of the panel where the rib and core continuity is interrupted. Better performance might be obtained if the core and ribs of the open-face panel were extended continuously over the supporting structure. In addition, initial imperfections appear to have a substantial effect on the performance of compressively loaded open-face sandwich panels.

Langley Research Center,
National Aeronautics and Space Administration,
Langley Station, Hampton, Va., September 22, 1966,
124-08-03-03-23.

APPENDIX

CONVERSION OF U.S. CUSTOMARY UNITS TO SI UNITS

The International System of Units (SI) was adopted by the Eleventh General Conference on Weights and Measures held in Paris, October 1960 in Resolution No. 12 (ref. 1). Conversion factors for the units used herein are given in the following table:

Physical quantity	U.S. Customary Unit	Conversion factor (*)	SI Unit
Length	in.	0.0254	meters (m)
Stress, modulus	kips/in.^{2}	6.895×10^6	newtons/meter^2 (N/m^2)
Specific stiffness	in.-kips	113	meter-newtons (m-N)

*Multiply value given in U.S. Customary Unit by conversion factor to obtain equivalent value in SI Unit.

Prefixes to indicate multiples of units are as follows:

Prefix	Multiple
centi (c)	10^{-2}
mega (M)	10^6
giga (G)	10^9

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TABLE I.- DIMENSIONS OF PANELS

(a) U.S. Customary Units

Panel	Material	θ, deg	b_p, in.	L, in.	b_f, in.	t_f, in.	t_c, in.	t_r, in.	h, in.	d, in.	s, in.	r
1	Type 302 stainless steel	60	9.25	30.00	0.231	0.005	0.005	0.005	0.0133	0.20	2.50	40
2	Type 301 stainless steel	45	8.24	25.25	0.458	0.010	0.008	0.010	0.0195	0.20	2.06	18
3			10.07	30.75							2.52	22
4			11.92	36.13							2.98	26
5		↓	13.74	41.75	↓	↓	↓	↓	↓	↓	3.44	30
6	Inconel X	45	8.49	26.75	0.566	0.018	0.010	0.010	0.0313	0.25	1.88	15
7		↓	14.15	45.38	↓	↓	↓	↓	↓	↓	↓	25
8	Type 301 stainless steel	62	6.74	21.28	0.355	0.012	0.010	0.010	0.0286	0.25	1.75	19
9			7.46	23.04		↓			↓			21
10			6.76	21.30		.015			.0318			19
11			7.46	23.03								21
12			8.88	26.58								25
13			10.33	31.80		↓			↓			29
14			6.77	21.31		.016			.0333			19
15		↓	7.45	23.06	↓	↓	↓	↓	↓	↓	↓	21

TABLE I.- DIMENSIONS OF PANELS – Concluded

(b) International System of Units

Panel	Material	θ, deg	b_p, cm	L, cm	b_f, cm	t_f, cm	t_c, cm	t_r, cm	h, cm	d, cm	s, cm	r
1	Type 302 stainless steel	60	23.50	76.20	0.587	0.013	0.013	0.013	0.0338	0.51	6.35	40
2	Type 301 stainless steel	45	10.93	64.14	1.163	0.025	0.020	0.025	0.0495	0.51	5.23	18
3			25.58	78.11							6.40	22
4			30.28	91.77							7.57	26
5		↓	34.90	106.05	↓	↓	↓	↓	↓	↓	8.74	30
6	Inconel X	45	21.56	67.95	1.438	0.046	0.025	0.025	0.0795	0.64	4.78	15
7		↓	35.94	115.27	↓	↓	↓	↓	↓	↓	↓	25
8	Type 301 stainless steel	62	17.12	54.05	0.902	0.030	0.025	0.025	0.0726	0.64	4.45	19
9			18.95	58.52					↓			21
10			17.17	54.10					.0808			19
11			18.95	58.50								21
12			22.56	67.51								25
13			26.24	80.77					↓			29
14			17.20	54.13					.0846			19
15		↓	18.92	58.57	↓	↓	↓	↓	↓	↓	↓	21

TABLE II.- PANEL PROPERTIES AND BUCKLING STRESSES

[μ = 0.3]

(a) U.S. Customary Units

Panel	E, ksi	D_x, in-kips	D_y, in-kips	D_{xy}, in-kips	k_x	σ_e, ksi	σ_p, ksi	σ_l, ksi
1	30 000	1.814	1.703	0.360	4.95	31	36.7	46.5
2	25 600	3.105	4.660	0.612	4.64	38	65.8	51.2
3			3.968			31	41.2	
4			3.457			21	27.7	
5	↓	↓	3.072	↓	↓	12	19.9	↓
6	31 000	6.70	9.64	2.00	3.20	68	87.7	90.6
7	↓	↓	↓	↓	↓	32	31.6	↓
8	31 500	7.40	6.35	1.41	3.08	80	127.7	100.0
9		↓	↓	↓	↓	67	104.2	↓
10		8.03	6.58	1.50	2.02	71	121.4	102.7
11						62	99.7	
12						52	70.4	
13		↓	↓	↓	↓	41	52.0	↓
14		8.22	6.64	1.53	1.64	77	117.3	104.1
15	↓	↓	↓	↓	↓	61	69.9	↓

TABLE II.- PANEL PROPERTIES AND BUCKLING STRESSES – Concluded

$[\mu = 0.3]$

(b) International System of Units

Panel	E, GN/m^2	D_x, m-N	D_y, m-N	D_{xy}, m-N	k_x	σ_e, MN/m^2	σ_p, MN/m^2	σ_l, MN/m^2
1	206.9	205	192	41	4.95	214	253	321
2	176.5	351	527	69	4.64	262	454	353
3			448			214	284	
4			391			145	191	
5	↓	↓	347	↓	↓	83	137	↓
6	213.7	757	1089	226	3.20	469	605	625
7	↓	↓	↓	↓	↓	221	218	↓
8	206.9	823	705	159	3.08	552	880	690
9		↓	↓	↓	↓	462	718	↓
10		893	731	170	2.02	490	837	708
11						427	687	
12						359	485	
13		↓	↓	↓	↓	283	359	↓
14		914	739	173	1.64	534	809	718
15	↓	↓	↓	↓	↓	420	668	↓

NASA-Langley, 1967 L-4571

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—NATIONAL AERONAUTICS AND SPACE ACT OF 1958

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