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BUCKLING STRENGTH OF COLD-FORMED STEEL CURVED PANELS

by
James L. Jorgenson[1] and Asadul H. Chowdhury[2]

I. INTRODUCTION

This paper is the first of a two-paper sequence describing the results of an investigation of the structural behavior of Deep Cold-Formed Corrugated Steel Curved Panels. The present paper deals with the experimental results and the second paper will deal with the analytical investigation. A typical curved panel is shown in Figure 1.

Deep cold-formed corrugated steel curved panels are used in the construction of metal buildings. The buildings incorporating the curved panels are of arch shape construction and are used primarily for agricultural buildings and to some extent for commercial buildings. The metal panels serve as both a covering of the building and as a structural frame.

The AISI Specification for the Design of Cold-Formed Steel Structural Members(1) does not have bending strength criteria for curved panels. In the absence of any specification, the design of deep cold-formed corrugated steel curved panels has, until now, been based on the AISI Specification which is valid for straight panels only. It has been observed by Jorgenson and Chern(2) that the load carrying capacity of curved panels is significantly overestimated if predicted on the basis of allowable stresses for straight panels as provided by AISI Specification. The AISI Code limit for h/t ratio is 200. But the curved panels used in the construction of metal buildings have h/t ratio as high as 350. Hence, only through a laboratory test, analysis or a combination of laboratory test and analysis, can the bending strength of curved panels be evaluated.

The research described in this paper explored the structural behavior of deep cold-formed corrugated steel curved panels experimentally. The results of the curved panels were compared with those of the identical straight panels. Based on the experimental results, the elastic and inelastic buckling phenomena of curved panels were explained.

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II. EXPERIMENTAL METHODS

This section of the paper will describe the properties of the steel and the methods of testing the panels.

2.1 MEMBER SHAPE

The corrugated metal panels under study are fabricated from rolled sheet galvanized steel. The fabrication process consists of: unrolling, cutting, punching, and then going through a roll-former which permits the panels to take on the corrugated shape. The curved panels need an additional operation called stretch forming. This consists of placing the panel in tension and then stretching it around a mold with the desired radius.

A cross section of a metal panel is shown in Figure 2. This shape results from role forming a flat 36 inch (915 mm) wide sheet. The center of gravity of the section is at 3.15 inches (80 mm), as shown, while the moment of inertia of the section for a 0.052 inch (1.32 mm) thickness is 5.54 $in.^4$ (230.6 cm^4)

The length of the straight member varies with its intended use. The length of the curved members (radius of inside flange equals 30.5 feet (930 cm)) is about 14 feet (427 cm). As used in buildings, the panels are placed side by side with an overlap on the bolt lines.

2.2 MECHANICAL PROPERTIES OF STEEL

The mechanical properties of importance in this investigation are the yield strength, tensile strength and the percent of elongation at rupture of the steel. Tensile coupons were taken from the webs of a number of the test specimens. The coupons (3/4" X 8" (19 mm x 203 mm)) were oriented to the longitudinal direction of the panel and at least 3 coupons were taken at each location.

The testing was in accordance with ASTM A370. Coupon test averages are shown in Table 1. The column labeled source refers to the bending test from which the coupons were taken. The next column indicates the galvanized thickness of the test specimen. For each test specimen the following columns report the static yield strength, the tensile strength, and the percent of elongation at failure.

2.3 SELECTION OF TEST METHOD

The objective in the testing program was to determine in the laboratory the bending strength of the curved panels which would match the bending strength of the panels in the building under uniform loads. The problem then was to model the field conditions in the laboratory. Specimen size limited the testing to lengths of about 13 feet and width of from one to three panels. Decisions to be made in the selection of the test methods were: (1) support conditions for test specimen?, (2) how should the loads be applied?, (3) how many panels in a test specimen?, and (4) what should be done to model the effect of the adjacent panels in the building which are not a part of the test specimen. The following paragraphs will address each of these questions.

Test specimen support conditions were set so as to provide simple beam support conditions. One end of the test specimen rested on a flat surface and the other end on a roller. The support surface at each end was in a plane perpendicular to the direction of the applied loads.

A number of different load applications were tested. Uniform loads, line loads, and concentrated loads were used. A uniform load was applied by placing an air bag between the test specimen and a reaction specimen. The uniform load was considered ideal in that it was similar to the wind or snow loads that would be applied to the actual building. However, the air pressure necessary to cause failure on a short span (about 12 feet (366 cm)) is about 10 times that which the building would be subjected to by wind or snow. At these high air pressures, the panel section was significantly distorted causing a reduction in its bending strength. Therefore, the uniform load from the air bag was not used in the testing.

Transverse line loads located at third points on the span were used in the previously reported tests.(2) The line load was applied continuously over the width of the test specimens. Both the tension and compression flange of the specimens were loaded. Bending failure occurred by compression in the compression flange. There remains some question as to whether the presence of the line load on the compression flange initiated the failure and/or provided significant lateral support for the compression flange. To improve on that situation, it was decided to load only the tension flange, leaving the compression flange unloaded and free to deform.

The selected support and loading system is shown in Figure 3. Span length is about 13 feet (396 cm). Transverse line loads were located at the one-third points on the span. Various size loading blocks were used, small blocks produced failure at the block location. The block size selected is shown in Figure 4. The lower part of the block is two 24 inch (610 mm) long 2" x 4" (51 mm x 102 mm) members on edge with a 1" x 4" (25.4 mm x 102 mm) spacer between them. To provide a uniform surface area for load application, a layer of sand was placed between the block and metal panel.

The number of panels in a test specimen varied from one to three. For the narrow flange in compression, two panels were used for the final testing (Figure 5). Using one panel permitted the panel to twist and fail at a low load. Using three panels required three loading points on each transverse line of loading. With three load points, the transverse loading beam becomes statically indeterminate making it difficult to apply equal loads to each of the panels. For the wide flange in compression most of the tests were run with a single panel as shown in Figure 6. A few tests were run with two and three panels. The advantage of more than one panel was to determine the influence of the adjacent panels while the disadvantage was the possible unequal loads on individual panels.

The final consideration in testing methods was to model the effect of the adjacent panels in the building which are not a part of the test specimen. For the wide flange in compression, the only added influence was to place a 1" x 1" x 16 gage (25.4 mm x 25.4 mm x 1.52 mm) angle across the tension flange at each line of loading. The function of the angle was to maintain the distance between bolt lines. Without the angles, the distance between bolt lines increased causing the section to decrease in depth. In the building, the adjacent panels would provide a compressive force which would tend to maintain the 27 inch (686 mm) bolt line distance.

For the narrow flange in compression, the main concern was in providing lateral support for the outside flanges of the test specimen and to do that without providing the additional strength that would come from adding an additional panel. For the straight panels, this was accomplished by adding a narrow flange to each of the outside flanges. The added flange was the same as a regular flange except that the web was bent into a horizonal plane as shown in Figure 7a.

Figure 8 shows the two flanges on the outside of the test specimen. The holes in the added flange were large and the bolts only hand tightened hence the added flange did not provide additional compressive force in the flange. With the web of the added flange bent to a horizontal plane, the added flange provided little additional bending resistance. The width of the added flange and its close fit to the narrow flange under test provided lateral support for the flange.

A similar added flange was used for curved test specimens. However, it was not possible to bend the web in a horizontal plane, hence, a shallower web in the normal position was used as shown in Figure 7(b).

Figure 5 illustrates the use of the added flange for curved test specimens. Some deeper webs, up to 6 inches (153 mm), were used in a few tests. These gave slightly higher failure loads and no failures took place on the outer flanges. The stiffening effect of the added flange was reduced by sawing slots in the web of the stiffener as shown in Figure 9. These tests resulted in slightly lower loads and failure was initiated by buckling of the outside flanges of the test specimen. Based on the tests with the different sizes of added stiffeners, the stiffener with the 2 inch (51 mm) web as shown in Figure 7(b) was used on most of the tests.

2.4 TESTING PROCEDURE

The loading system used in this testing program consisted of a set of four Enerpac hydraulic jacks (Model No. 22-092), connected to a Riehle pumping and indicating unit (Model M-type Pumping Unit). The hydraulic jack has the effective piston area of 1.77 square inches (11.42 cm^2). The Riehle pumping unit is equipped with two 'M'-type gage indicators. The gage indicator used has a range from 0 to 4,000 psi (0 to 27.6 MPa) with the scale of 10 psi (0.069 MPa) per division. Figures 3 and 10 provide views of the 4 hydraulic jacks and the 2 pair of 2" x 6" (51 mm x 153 mm)timber members used to provide the loads for the test specimens.

Two dial gages were placed at midspan of the test specimen, one gage was on each side of the specimen. The dial gages were used to measure the vertical deflection of the panels. The dial gages are shown in Figure 4.

The testing procedure started by setting the deflection gage readings to zero. Then a certain increment of the load (from 50 to 200 psi (0.34 MPa to 1.38 MPa)) was gradually applied to the test speciman. The load was held at this value until the readings on the deflection gages were stabilized, read, and recorded. This procedure was repeated for each additional load increment until the deflection was observed increasing without an increase in the applied load. This load was called the failure load.

III. EXPERIMENTAL RESULTS

A typical load-deflection diagram for curved panels with wide flange in compression is shown in Figure 10. The test specimen consists of three panels as shown in Figure 3. Deflection at the midspan of the panels is plotted on the horizontal axis with the total load, 4P, on the three panels of the specimen on the vertical axis. The technique of applying the total load, 4P, on the three panels as concentrated loads and the locations of the dial gages are also shown in Figure 10. Since the deflection at failure could not be read, a dashed line is used to plot the deflection curve beyond the last deflection reading.

The curved panel with wide flange in compression failed by elastic buckling of the compression flange as shown in Figure 11. The failure mode was marked by a significant increase in the vertical deflection and a reduction of applied loads. In the elastic failure mode the panel changed cross sectional shape but when the specimen was unloaded the panel returned to its original cross section without any permanent wrinkles or marks in the metal. This behavior was observed in curved specimens while testing one, two or three panels when the wide flange was in compression.

One distinct buckling mode characterized the buckling behavior of curved panels with wide flange in compression as shown in Figure 12. A bending moment acting on a curved beam causes circumferential stresses which are the normal stresses on plane sections passing through the center of curvature perpendicular to the plane of curvature. The circumferential stresses cause the longitudinal waves in the wide compression flange of the curved panel. The circumferential stresses also cause the flange to deflect radially. Accompanying the radial deflection of the flange there are transverse bending stresses in the flange which are flexural stresses on planes at right angles to the radius of curvature. The transverse buckling waves in the wide compression flange of a curved panel (Figure 12) are caused by the transverse compressive stresses.

The radial deflection of the flange of the curved panels results in the distortion of the cross section of the beam, the effect of which is to decrease the stiffnesses of the member and to increase the maximum circumferential stresses in the beam. This increase in stresses decreases the load carrying capacity of the curved panel with wide flange in compression. This is reflected in Table 2 which shows that for the wide flange in compression the curved panel has about 70% of bending capacity of the straight panel.

Table 2 gives a summary of the panel test results. Column 3 of this table specifies which of the panels flanges are in compression. The number of panels composing the test specimen are given in column 5. The experimental bending moment at failure per foot of panel width is provided in column 7. Since the yield strength and thickness of the specimen varied from panel to panel, the corrected bending moment at failure is given in column 10. The corrected failure moment is based on the anticipated minimum yield strength of 33 ksi (227.4 MPa) and the anticipated minimum thickness of 0.039" (0.99 mm) for the 20 gage and 0.052" (1.32 mm) for the 18 gage panels. In the case of elastic buckling failure, the failure moment is independent of the yield strength of the steel, hence the failure moment should not be corrected for yield strength. Those values without the yield correction are shown in parenthesis. The comparison between the bending strength of curved and straight panels is based on the corrected failure moment.

Figure 13 shows the typical load-deflection diagram for curved panels with narrow flange in compression. The test specimen consists of two panels. Deflection at the midspan of the panels is plotted on the horizontal axis with the total load, 4P, on the two panels of the specimen on the vertical axis. The technique of applying the total load, 4P, on the two panels as concentrated loads and the locations on the dial gages are given in Figure 13. The deflection at failure could not be recorded, so a dashed line is used to plot the deflection curve beyond the last deflection reading.

The curved panel with narrow flange in compression failed by plastic yielding of the compression flange. The plastic yield in the compression flange occurred at the extreme fiber at the flange-web connection point along the transverse line of loading (Figure 14). The amount of deformation increased with the increase of applied loads and the failure mode was marked by a significant increase in vertical deflection and a reduction in applied load. When the load was released, the specimen remained in the deformed condition with sharp bends or wrinkles in the metal. The failure of the flange of curved panels with narrow flange in compression is similar to the failure of the flange of curved I-beam in which the maximum circuferential stress in the flange occurs at the extreme fiber in the plane of the web(3).

IV. CONCLUSIONS

The purpose of this research was to determine experimentally the bending strength of the 18 gage and 20 gage panels and to determine the best testing method for obtaining those results. The results can be summarized in the following statements:

1. The selected method consisted of a 13 foot (396 cm) long simple span beam from one to three panels wide and loaded at third points on the span with two foot (610 mm) long line loads on the tension flanges.

2. In all cases, bending failure occurred by buckling of the compression flange. For the wide flange in compression on the curved member the buckling was elastic while for all other tests the buckling was plastic.

3. For the narrow flange in compression, the curved panel has about 80% of the bending capacity of the straight panel. The similar ratio for wide flange in compression is 70%

APPENDIX I - REFERENCES

1. American Iron and Steel Institute, SPECIFICATION FOR THE DESIGN OF COLD-FORMED STEEL STRUCTURAL MEMBERS, 1968 Edition, AISI, New York, 1970.

2. Jorgenson, J. L., and Chern, C, Bending Strength of Deep Corrugated Steel Panels, Proceedings, Second Specialty Conference on Cold Formed Steel Structures, St. Louis , Missouri, October, 1973.

3. Seely, Fred B., and Smith, James O., Advanced Mechanics of Materials, Second Edition, John Wiley & Sons, Inc., 1952.

SOURCE	THICKNESS, IN.* (MM)	STATIC YIELD, ksi (MPa)	TENSILE STRENGTH KSI (MPa)	PERCENT ELONGATION
TI	.039 (0.99)	29.5 (203.4)	43.8 (302.0)	38.1
T4	.054 (1.37)	40.8 (281.3)	47.9 (330.0)	36.9
T23	.053 (1.34)	32.1 (221.3)	46.3 (319.2)	35.3
T24	.052 (1.32)	31.7 (218.3)	47.3 (326.1)	38.8
T25	.053 (1.34)	31.6 (217.8)	48.3 (333.0)	35.0
T26	.039 (0.99)	34.0 (234.4)	47.3 (326.1)	34.7

TABLE 1 MECHANICAL PROPERTIES OF STEEL

* Steel thickness not corrected for surface galvanization.

1	2	3	4	5	6	7		8		9		10	
GAGE	SHAPE	FLG. IN COMPR.	TEST NO.	NO. OF PANELS	FAIL TYPE	FAIL MOMENT		SAMPLE YIELD STRESS		SAMPLE THICKNESS		*CORRECTED FAIL MOMENT	
						IN-K/FT	M-KN/M	KSI	MPA	IN	MM	IN-K/FT	M-KN/M
18	STR	W	T-11	1	PL	74.3	27.5						
		W	T-13	1	PL	69.9	25.9						
		W	T-2,3,4	2	PL	71.7	26.5						
		ave.				71.3	26.4	40.8	281.3	0.054	1.37	55.5	20.6
		n	T-30	2	PL	52.1	19.3						
		n	T-31	2	PL	51.3	19.0						
		n	T-32	2	PL	50.0	18.5						
		ave.				51.1	18.9	31.8	219.3	0.0526	1.33	52.2	19.5
	CURV	W	T-6	1	EL	45.0	16.7						
		W	T-7	3	EL	40.5	15.0						
		W	T-38	2	EL	38.3	14.2						
		W	T-40	3	EL	39.6	14.7						
		W	T-41	3	EL	39.6	14.7						
		ave.				40.6	15.0	38.68	266.7	0.054	1.37	34.8 (39.1)	12.9 (14.5)
		n	T-24	2	PL	43.8	16.2						
		n	T-25	2	PL	41.7	15.42						
		n	T-22,23	2	PL	50.7	18.75						
		ave.				45.4	16.80	34.8	239.8	.053	1.34	41.8	15.5
20	STR	W	T-14	1	PL	39.9	14.8						
		W	T-16	1	PL	40.5	15.0						
		W	T-17	1	PL	40.9	15.2						
		ave.				40.5	15.0	29.5	203.4	0.039	0.99	41.8	15.5
		n	T-28	2	PL	33.0	12.2						
		n	T-29	2	PL	30.9	11.4						
		n	T-20	2	PL	30.6	11.32						
		ave.				31.5	11.6	34.0	234.4	0.039	0.99	28.2	10.4

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1	2	3	4	5	6	7		8		9		10	
GAGE	SHAPE	FLG. IN COMPR.	TEST NO.	NO. OF PANELS	FAIL TYPE	FAIL MOMENT IN-K/FT	M-KN/M	SAMPLE YIELD STRESS KSI	MPA	SAMPLE THICKNESS IN	MM	*CORRECTED FAIL MOMENT IN-K/FT	M-KN/M
20	CURV	W	T-8	1	PL	25.1	9.3						
		W	T-9	1	EL	23.8	8.8						
		W	T-10	1	EL	22.9	8.5						
		ave.				23.9	8.8	29.5	203.4	.039	0.99	24.7 (22.1)	9.2 (8.2)
		n	T-27	2	PL	33.4	12.4						
		n	T-26	2	PL	30.0	11.1						
		n	T-21	2	PL	34.9	12.9						
		ave.				32.8	12.1	34.0	234.4	.039	0.99	29.3	10.8

* Corrected for steel yield of 33 ksi (227.4 MPa) and panel thickness of .036" (.91 mm) for 20g and .052" (1.32 mm) for 18g. The values in parenthesis were not corrected for yield strength.

W = Wide Flange
n = Narrow Flange
PL = Plastic Failure
EL = Elastic Failure

Examples for correction: (20 gage curved panel with wide flange in compression) Corrected Failure Moment = (Uncorrected failure moment) x (correction for yield strength) x (correction for thickness) = $23.9 \left(\frac{33}{29.5}\right) \left(\frac{.036}{.039}\right) = 24.7$ in-k/ft.

TABLE 2 SUMMARY OF PANEL TESTS

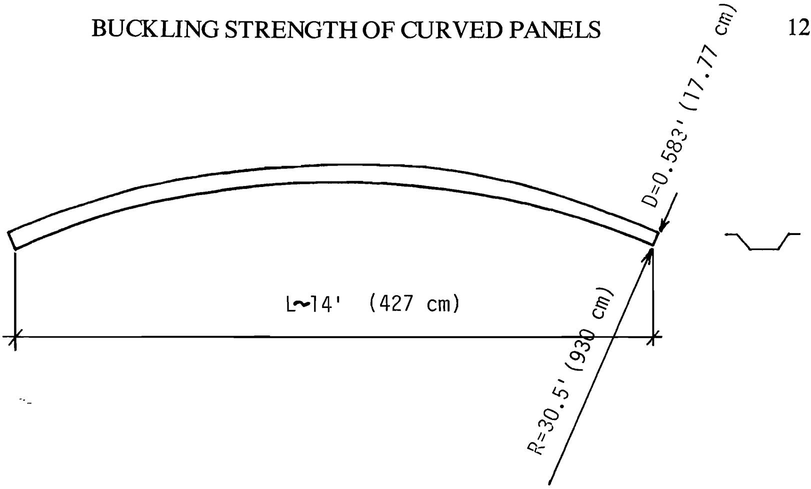
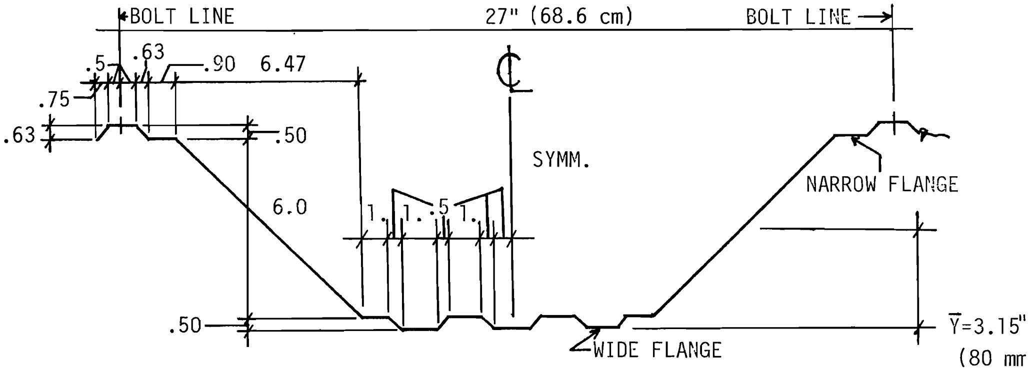


Fig. 1 CURVED PANEL, ELEVATION VIEW



All dimensions are in inches unless specified otherwise.

Fig. 2 SECTION THROUGH ONE PANEL

IN.	MM
0.5	12.7
0.63	16.00
0.75	19.05
0.90	22.86
1.0	25.40
6.0	152.4
6.47	164.34

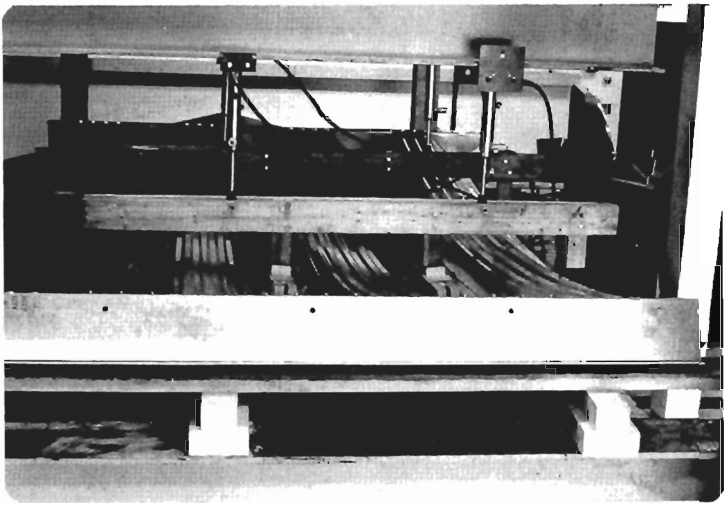


FIG. 3 SUPPORT AND LOADING SYSTEM

FIG. 4 BLOCK FOR LOAD APPLICATION

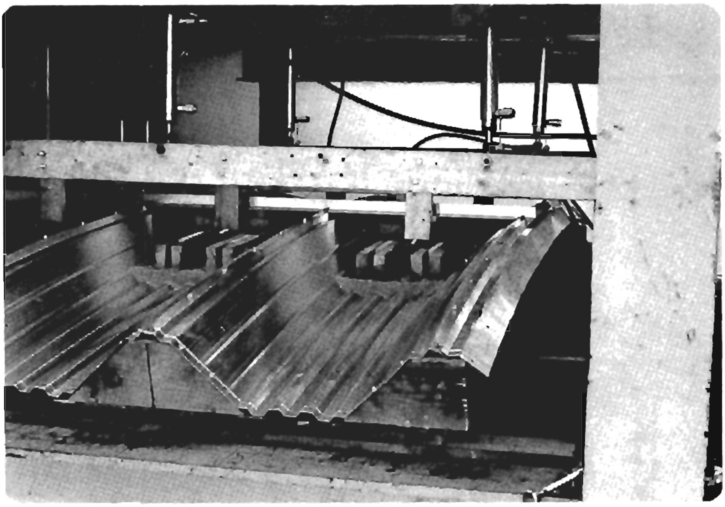


FIG. 5 TWO PANELS, NARROW FLANGE IN COMPRESSION

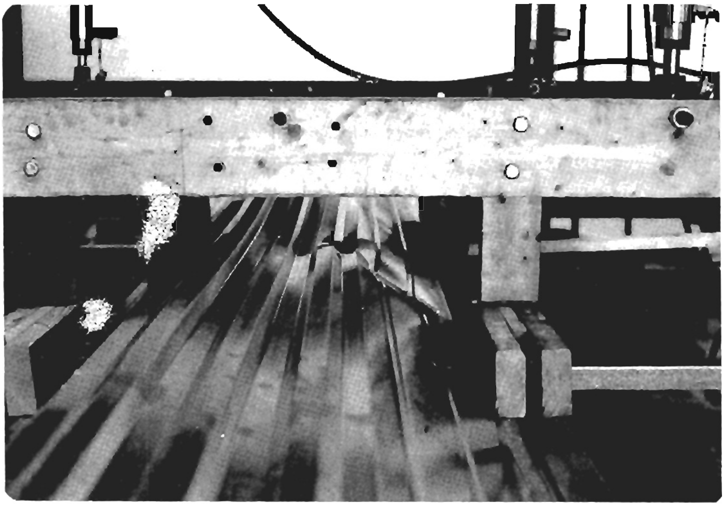
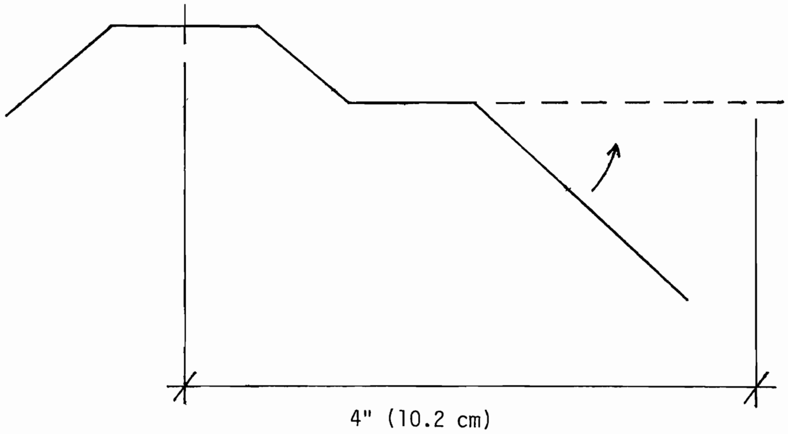
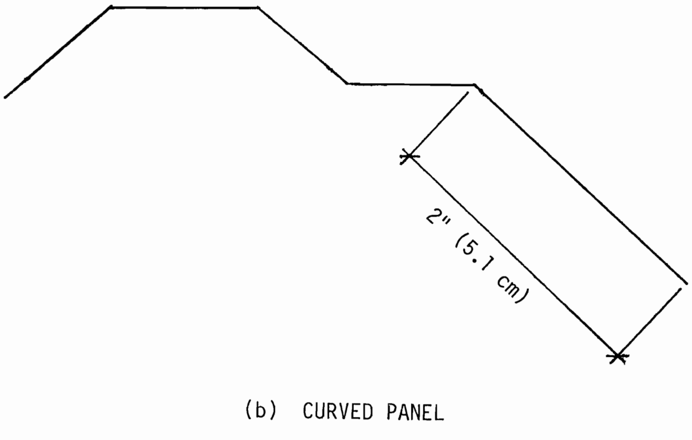


FIG. 6 SINGLE PANEL, WIDE FLANGE IN COMPRESSION



(a) STRAIGHT PANEL



(b) CURVED PANEL

FIG. 7 LATERAL SUPPORTS

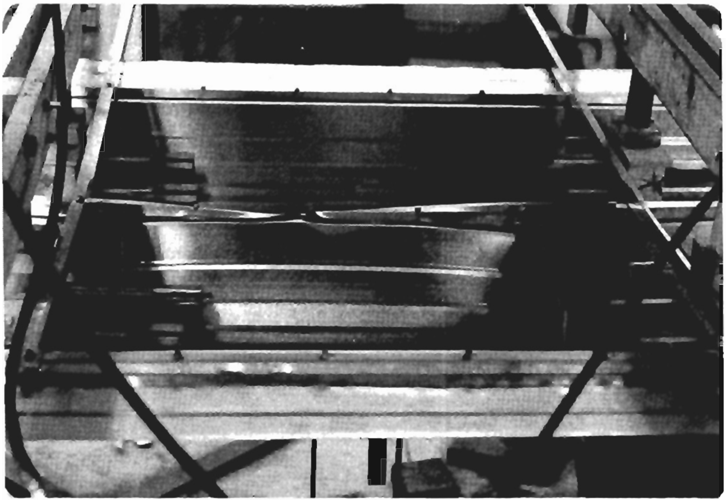


FIG. 8 STIFFENER FLANGES ON EDGES OF STRAIGHT PANELS

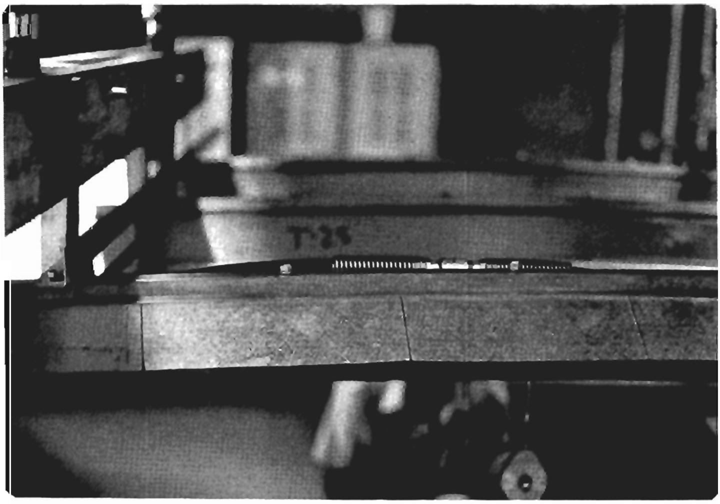


FIG. 9 EDGE STIFFENERS WITH SLOTS IN THEIR WEBS

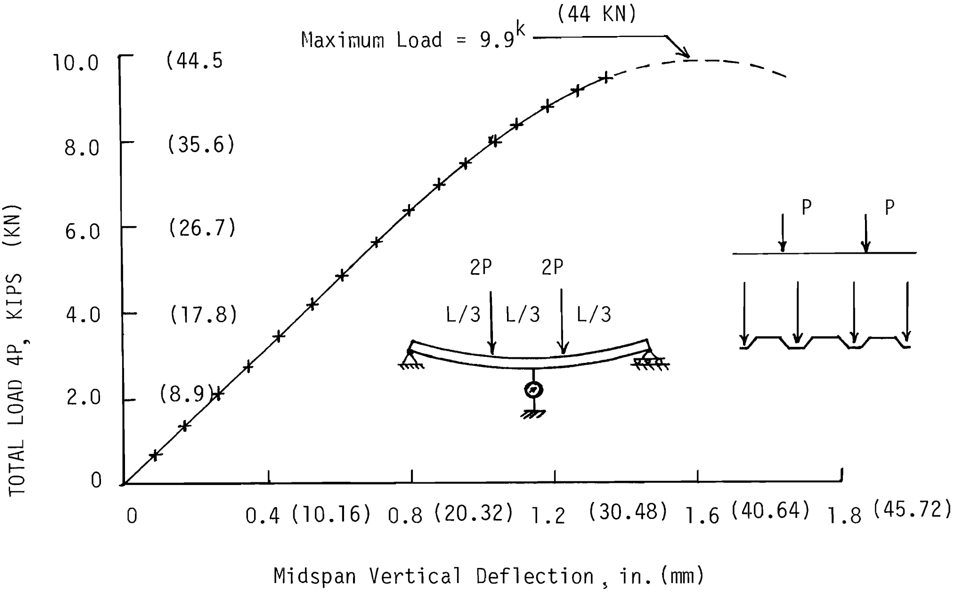


FIG. 10 LOAD-DEFLECTION CURVE, TEST NO. T-40

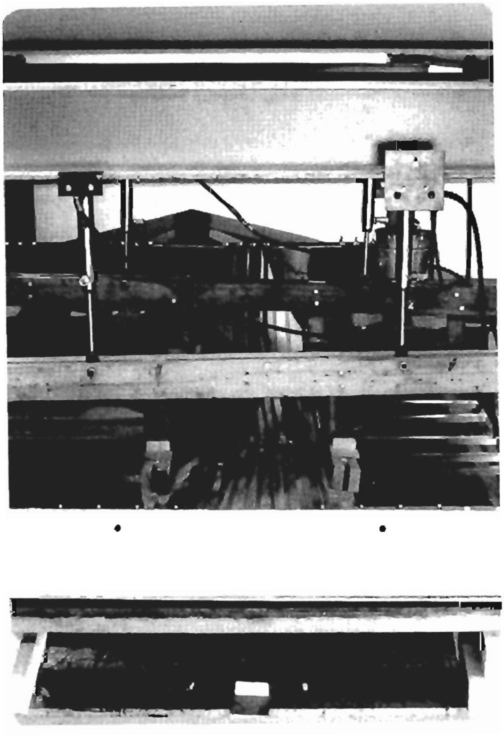


FIG. 11 ELASTIC BUCKLING OF WIDE FLANGE

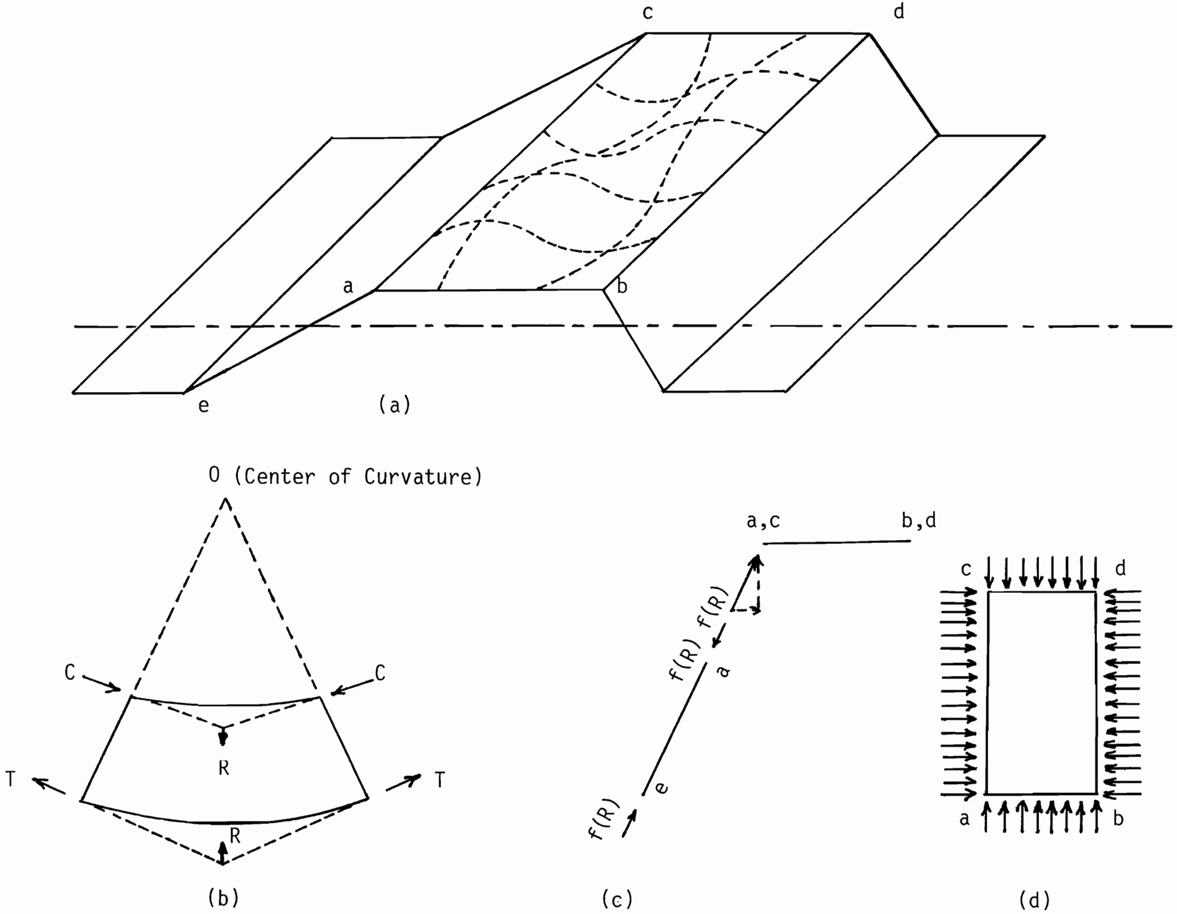


FIG. 12 BUCKLED WIDE COMPRESSION FLANGE OF CURVED PANEL

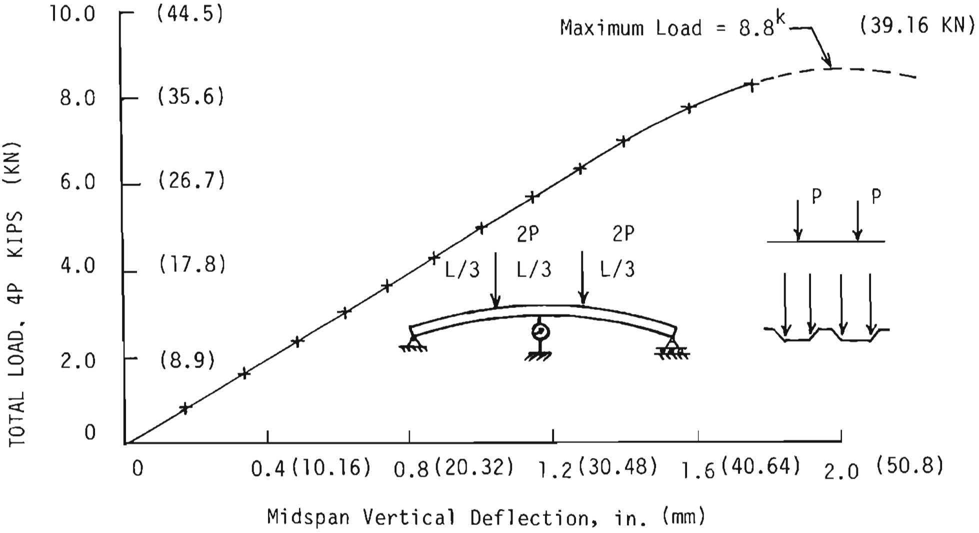


FIG. 13 LOAD-DEFLECTION CURVE, TEST NO. T-23

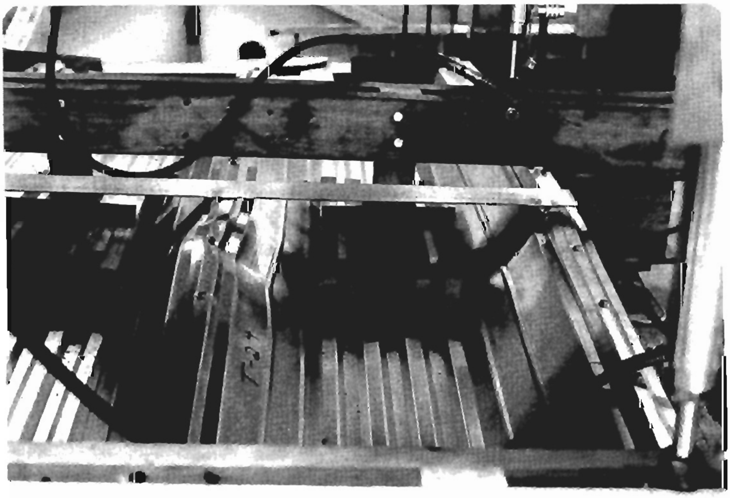


FIG. 14 PLASTIC FAILURE OF NARROW FLANGE