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BEHAVIOR OF CONTINUOUS SPAN PURLIN SYSTEMS

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ABSTRACT

Cold-formed steel structural shapes are the mainstay of metal building roof systems in the United States. Because of the measurable economic gains that can be derived from an optimum design of such a roof system, it is imperative that the synergism of the system, i.e., the combined structural resistance provided by the structural members, roof panels and their attachments, be considered in the structural design methodology.

This paper will discuss a test program conducted to provide insight into the behavior of a typical metal building roof system subjected to wind uplift loading. Both C and Z shaped structural members, continuous over three spans, were investigated. The tests were representative of a conventional through fastened roof assemblage. Results of the test program, as well as an easily applied design approach, is presented.

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INTRODUCTION

Cold-formed steel structural shapes, C and Z sections, are the mainstay of the metal building roof system in the United States. These shapes are generally designed as beams, continuous over the main frames, having the top flanges attached to metal roof sheets. However, when subjected to wind uplift, the top flange is the tension flange and the bottom flange functions as the compression flange in the positive moment region of the span. The tendency, in conventional steel design, is to assume that because the compression flange is not fully braced, the design of the member should assume a laterally unbraced condition. This is unduly conservative, and measurable economic gains can be derived from recognizing the synergism of the roof system, i.e., the combined structural resistance of the structural members, roof panels and their attachments.

To gain insight into the behavior of a typical metal building industry roof system, a series of full scale tests were conducted. The test program focused on the uplift capacity of both C and Z section roof systems as used by members of the Metal Building Manafucturers Association. This paper describes the test program, discusses the test results and presents a simplified design procedure for assessing the uplift capacity of a roof system.

TEST PROGRAM

The test program consisted of a total of 19 full scale tests. Fourteen test specimens used Z-section beams and five used C-section beams. The Z and C sections were selected to provide a test program that would envelope most of the section geometries commonly used in the metal building industry. The following summarizes the range of variation in the key section geometric parameters:

Flange width	- 2 to 3 in. (50.8 to 76.2 mm)
Web depth	- 6.5 to 10 in. (165 to 254 mm)
Thickness	- .056 to .101 in. (1.42 to 2.57 mm)
Edge Stiffener	- 42 to 90 degrees.
Lap length	- 30 in. to 72 in. (76.2 to 183 mm)
Span length	- 20 ft. to 30 ft. (6.1 to 9.1 m)

Table 1 provides a summary of each test specimen and it's geometry.

All test specimens used an industry standard galvanized roof panel, formed from 24 or 26 ga. thick sheet steel. The panels were 36 in. (91.4 cm) wide with 1.25" (31.8 mm) deep corrugations, 12 in. (30.5 cm) on center. The attachment of the roof panel to the beam section was accomplished by using commonly used self-drilling screws (No. 12 SDS). The screws were located at 12 in. (30.5 cm) centers along the length of the beam secion. Because lateral stability of a beam section is enhanced by the rotational restaint that is provided by the panel and it's attachment, the panel and fastener type are significant parameters.

TEST SETUP

All specimens were tested as continuous beams subjected to a uniformly distributed load. Each test specimen consisted of two beam sections (purlins), continuous over three spans and affixed to roof panels. Figure 1 is a schematic of the test setup, and Figures 2 and 3 are photographs of a typical test setup. A detailed discussion of the test setup is given in Reference 1.

As depicted in Figures 2 and 3, the test specimen was constructed in a pressure test chamber. A simulated wind uplift load was applied by pressurizing the test chamber. The applied load was recorded by use of manometers located at each end of the test chamber. The vertical deflection of each purlin was recorded in the end bays, and at the location of the maximum deflection based on linear elastic beam theory. Because of the unsymmetrical geometry of the purlin cross section, the compression flange will also displace horizontally under load; this deflection was also measured. Both the vertical and horizontal displacements were measured by utilizing a surveyor's level and targets (Ref. 1).

TEST RESULTS

All test specimens were loaded until failure of one of the purlins was obtained. Table 2 provides a summary of the failure load for each specimen. The failure load is given by the pressure at failure times the tributary width of roof panel that is supported by each purlin. Failure typically manifested itself as a local buckling of the web and flange at the location of maximum applied moment in one of the end spans. Figure 4 shows typical failure conditions.

Also listed in Table 2 is the tested yield strength for each purlin specimen evaluated in accordance with the procedures of ASTM A370.

EVALUATION OF TEST RESULTS

The continuity of the Z-sections is achieved by nesting of the purlin sections at the frame line, or intermediate support. The literature contains little information regarding the behavior of continuous span purlins, and the ability of the nested purlins to develop continuity. Reference 3 indicates that to achieve full continuity, the lap length should be at least 1.5 times the depth of the section. For similiar purlin sections in this test program, the assumption of 1.5 times the depth is a reasonable length to develop full continuity. This is based on the comparison of the failure load for tests No. 1 and 3 and tests No. 2 and 4. As given in Table 3, the failure load for these test specimens is virtually independent of the lap length, which suggests that linearly elastic beam theory can be used to evaluate the applied internal forces with sufficient accuracy.

Previous research (Ref. 2) presented an analytical technique for calculating the actual stresses that would be experienced in a purlin section, having an unbraced compression flange, subjected to wind uplift. However, to estimate the ultimate failure mode required an iterative calculation procedure that was not well suited for routine design. Therefore, a simple, straightforward empirical procedure was developed based on the test results for both continuous span purlins, as described previously, and simple span purlins as given in Reference 2 and unpublished tests conducted at Butler Research.

The following simplified design approach for evaluating the capacity of a C or Z beam section, attached to roof panels, and subjected to a wind uplift load, is based upon applying a reduction factor to the calculated fully braced moment capacity. The nominal moment capacity is determined based on the provisions given in the 1986 edition of the AISI Specification (4), assuming full lateral support along the length of the member. Recognizing the relationships between applied load and moment based on elastic beam theory, the nominal load was calculated and is listed in Table 4.

Also given in Table 4 is the tested failure load. Based on the given test results, the load capacity, of a C or Z beam section braced on its tension flange and having its compression flange unbraced, is a constant relationship to the fully braced member. This relationship is evident by the ratio of the tested failure load to the nominal load (Table 4).

For purpose of design, it is recommended that the following expressions be used to calculate the nominal capacity of C and Z beams:

$$M_n = R\, S_e\, F_y \qquad (1)$$

Where

R = 0.70 for continuous span Z sections

= 0.60 for continuous span C sections

S_e = the elastic section modulus of the effective section at F_y

F_y = yield stress.

Because the reduction factor, R, was experimentally determined, the use of Eq. 1 should be limited to through fastened roof assemblies and for members having the following geometric limits:

- Purlin depth less than 10 in. (25.5 cm)
- The free flange is a stiffened compression element
- $60 <$ web depth / thickness < 170
- $2 <$ web depth / flange width < 5
- $16 <$ flange flat width / thickness < 43

CONCLUSIONS

The rigourous, analytical evaluation of the strength of a Z or C beam roof system subjected to wind uplift requires an iterative calculation procedure, that does not lend itself to routine design. Therefore, an simple, easily applied empirical procedure has been developed.

REFERENCES

1. Perry, D., et.al., "Continuous Span Purlin Uplift Tests," Final Report, Metal Building Manufacturers Association, October 1987.

2. Pekoz, T., and Soroushian, D., "Behavior of C and Z Purlins Under Uplift," Proceedings of the Sixth International Specialty Conference on Cold-Formed Steel Structures, November 1982, University of Missouri-Rolla, Rolla, MO.

3. Robertson, G.W., and Kurt, C.E., "Behavior of Nested Z-Shaped Purlins," Proceedings of the Eighth International Specialty Conference on Cold-Formed Steel Structures, November 1986, University of Missouri-Rolla, Rolla, Mo.

4. Cold-Formed Steel Design Manual, American Iron and Steel Institute, Washington, D.C., 1986 Ed.

TABLE 1
GENERAL SECTION GEOMETRY

Test No.	Section Type	Section Depth (In.)	Flange Width (In.)	Material Thickness (In.)	Span Length (Ft.)	Lap Length (Ft.)
1	Z	9.5	3.00	0.0701	30	6.0
2	Z	9.5	3.00	0.1013	30	6.0
3	Z	9.5	3.00	0.0696	30	2.5
4	Z	9.5	3.00	0.1002	30	2.5
5	Z	9.5	3.00	0.0695	20	2.5
6	Z	6.5	2.00	0.0587	20	4.0
7	Z	6.5	2.00	0.0613	20	4.0
8	Z	6.5	2.00	0.0600	20	4.0
9	Z	6.5	2.00	0.0590	20	4.0
10	Z	8.5	2.00	0.0847	24	5.0
11	Z	8.5	2.00	0.0845	24	5.0
12	Z	10.0	2.75	0.0833	24	5.0
13	Z	8.5	2.50	0.0613	24	4.0
14	Z	9.5	3.00	0.0660	20	2.5
15	C	9.0	2.38	0.0607	24	4.5
16	C	7.0	2.50	0.0598	24	4.5
17	C	8.5	2.50	0.0897	24	5.0
18	C	8.5	2.50	0.0561	20	4.0
19	C	8.5	2.50	0.0895	30	6.0

Notes: 1. For definition of lap length see Fig. 5.
2. 1 in. = 25.4 mm.
3. 1 ft. = 0.3048 m.

TABLE 2
TEST RESULTS

Test No.	Tested Fy (ksi)	Failure Load (lb/ft)
1	63.1	138
2	58.1	234
3	61.6	136
4	58.2	218
5	61.3	280
6	55.2	109
7	57.4	124
8	56.1	121
9	55.8	119
10	60.2	181
11	57.8	193
12	61.5	286
13	58.5	134
14	63.9	241
15	56.5	128
16	54.6	98
17	58.3	217
18	65.3	156
19	58.3	135

TABLE 3
LAP LENGTH COMPARISON

Test No.	Required Lap Length (ft.)	Test Lap Length (ft.)	Failure Load (lb/ft.)
1	2.375	6.0	138
3	2.375	2.5	136
2	2.375	6.0	234
4	2.375	2.5	218

Notes:
1. Required lap length based upon Ref. 3, and shown on Fig. 5 as greater than 1.5 D.
2. 1 ft = 0.3048 m
3. 1 lb = 4.448 N.

TABLE 4
EVALUATION OF DATA

Test No.	Nominal Calculated Load (lb/ft)	Test Failure Load (lb/ft)	Test / Calculated
1	193	138	0.72
2	283	234	0.83
3	189	136	0.72
4	274	218	0.80
5	421	280	0.67
6	170	109	0.64
7	190	124	0.65
8	177	121	0.68
9	171	119	0.70
10	294	181	0.62
11	281	193	0.69
12	400	286	0.72
13	186	134	0.72
14	406	241	0.59
Mean			0.70
Standard Dev.			0.06
15	213	128	0.60
16	145	98	0.68
17	337	217	0.64
18	288	156	0.54
19	222	135	0.61
Mean			0.61
Standard Dev.			0.05

Notes: 1. 1 ft = 0.3048 m
2. 1 lb = 4.448 N.

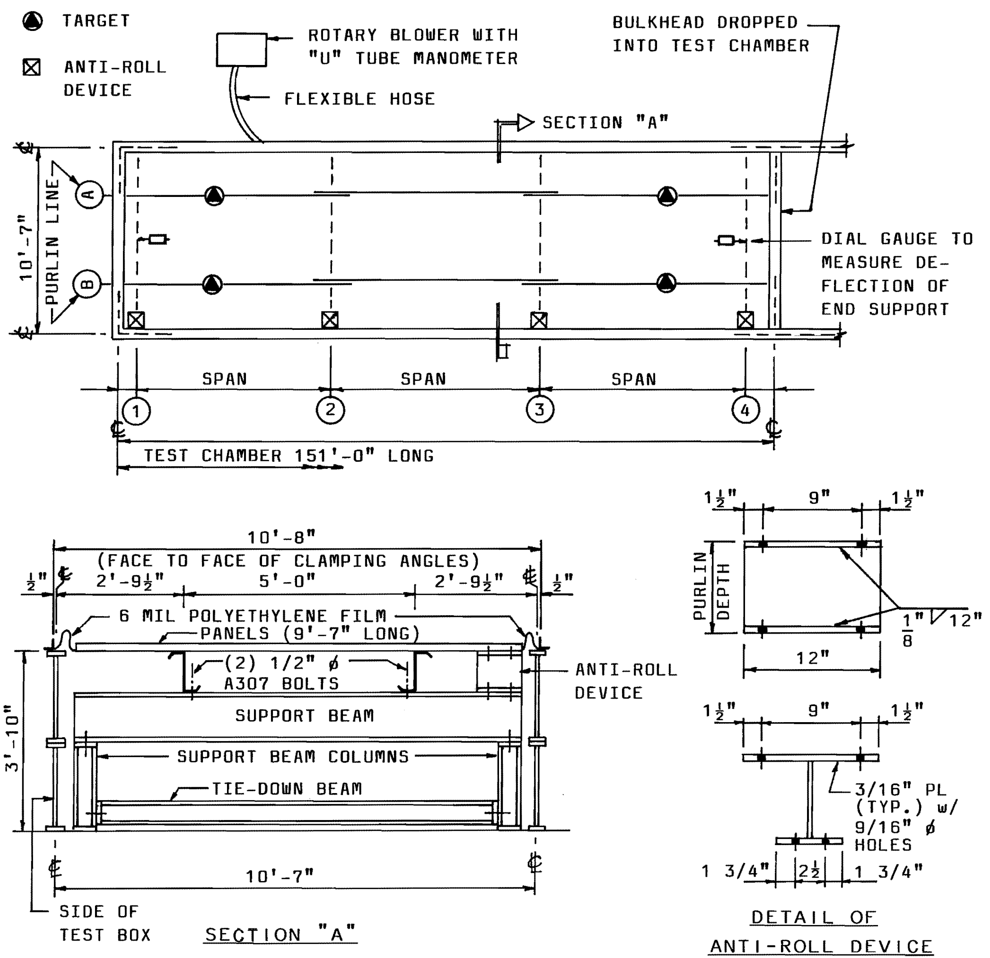


FIG. 1 TEST SET-UP FOR PURLIN UPLIFT TESTS

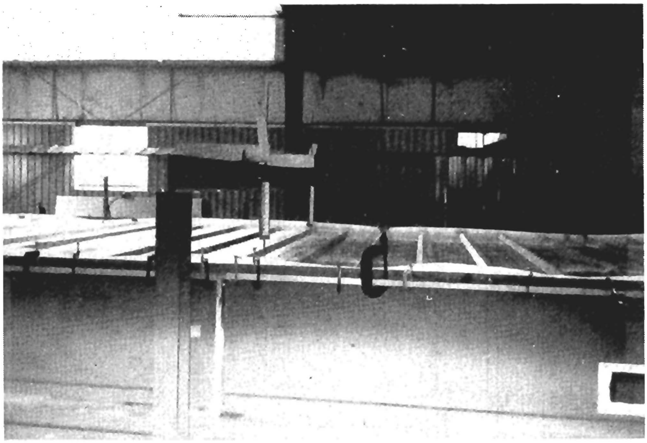


FIG. 2 PHOTOGRAPH OF TEST SETUP

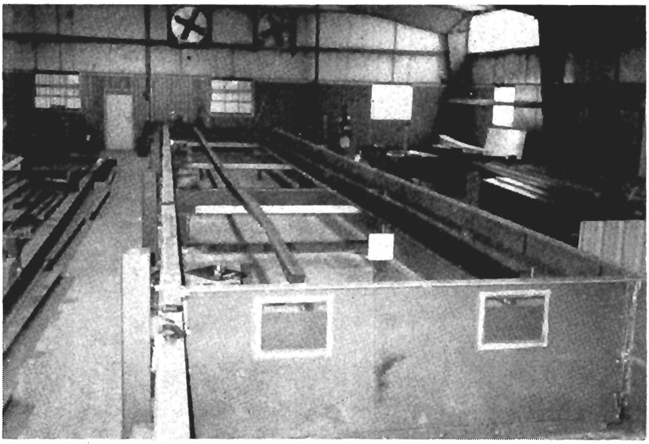
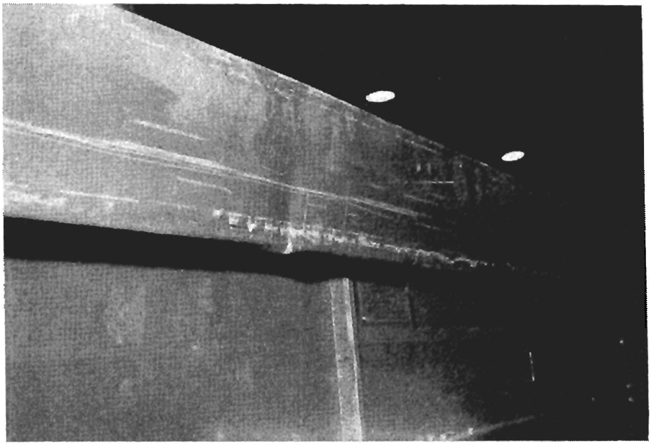


FIG. 3 - PHOTOGRAPH OF PURLINS AND SUPPORTS



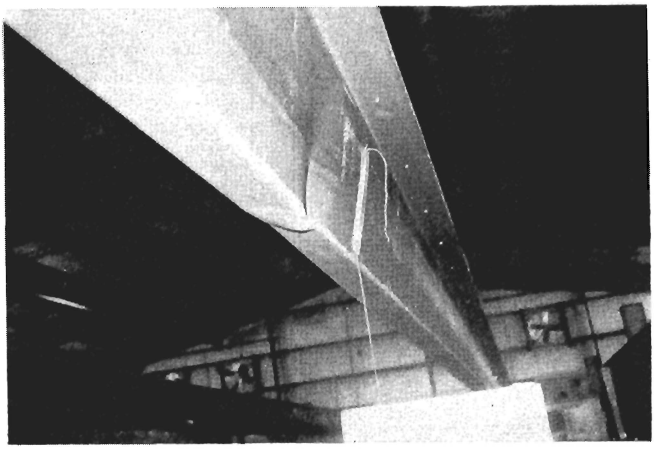


FIG. 4 TYPICAL PURLIN FAILURE MODE

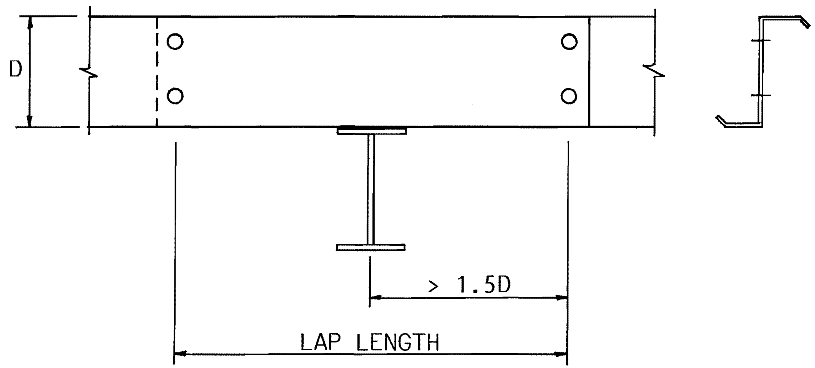


FIG. 5 TYPICAL Z-BEAM LAP