Nov 13th, 12:00 AM

Evaluation of Elastically Restrained Compression Flanges of Cold-formed C-shaped Sections

G. Haaijer

K. H. Klippstein

J. H. Garrett Jr.

Seventh International Specialty Conference on Cold-Formed Steel Structures
St. Louis, Missouri, U.S.A., November 13-14, 1984

EVALUATION OF ELASTICALLY RESTRAINED COMPRESSION FLANGES OF COLD-FORMED C-SHAPED SECTIONS

J. H. Garrett, Jr.[1], G. Haaijer[2], and K. H. Klippstein[3]

ABSTRACT

Structural tests were performed at the United States Steel Research Center during the summer of 1983 on a flooring system commonly found in residential and light-commercial construction. The system was comprised of a 3/4-in.-thick plywood deck attached to the top flange of two two-span-continuous cold-formed sheet-steel C-shaped joists. The joists were 7.34-in. deep, 1.875-in. wide, having yield strengths of about 57 ksi. Each span was approximately 20 feet. The structural testing program was performed to establish if lateral-torsional buckling of the compression flange near the interior support was a critical design criterion.

Test results indicate that 3/4-in.-thick plywood connected as described provides composite action resulting in an increase of the joist-load-carrying capacity of about 15 percent. The joists in all three floor tests sustained local buckles of the lip, flange, and web near the interior support. None of the joist flanges failed in a torsional-flexural mode.

Considering the redistribution of the loads due to varying effective section properties of these light-gage steel joists and the composite action, the test results confirm the findings of a previous finite-element elastic-buckling study, which showed that the unbraced length that is used in utilizing a current design method for a uniform bending-moment distribution (Section 3, Part III of the American Iron and Steel Institute's *Cold-Formed Steel Design Manual*) could be less than the distance between the inflection point and the interior support if a plywood deck at least 3/4-inch thick is attached to the top flange. The effective unbraced length is approximately one-fourth of the distance between the inflection point and the interior support. When L/4 is used as the laterally unbraced length, lateral-torsional buckling is no longer a problem in the floor systems investigated, but combined bending and shear becomes critical.

[1] Research Assistant, Carnegie-Mellon University, formerly a Co-op Student, U. S. Steel

[2] Vice President - Director of Engineering, AISC, formerly a Senior Research Engineer, U. S. Steel

[3] Adjunct Professor, University of Pittsburgh, formerly a Research Engineer, U. S. Steel

INTRODUCTION

FLOOR JOIST BUCKLING MECHANISM

A common flooring system used in residences and light-commercial structures consists of a plywood deck attached to the top flange of cold-formed sheet-steel C-shaped joists as shown in Figure 1. When this floor supports a uniform gravity load over two continuous-beam spans, the bottom flange of the joist is compressed in the region adjacent to the interior support, as shown in Figure 2. If no gypsum ceiling is attached to the bottom flange, which is common for most first-level floors, lateral-torsional buckling of the bottom flange adjacent to the interior support may become a critical design consideration. Lateral-torsional buckling is a beam failure in which the compression flange of the beam moves laterally, vertically, and/or torsionally. However, these flange movements are elastically restrained by the web and the upper flange of the joist and the plywood.

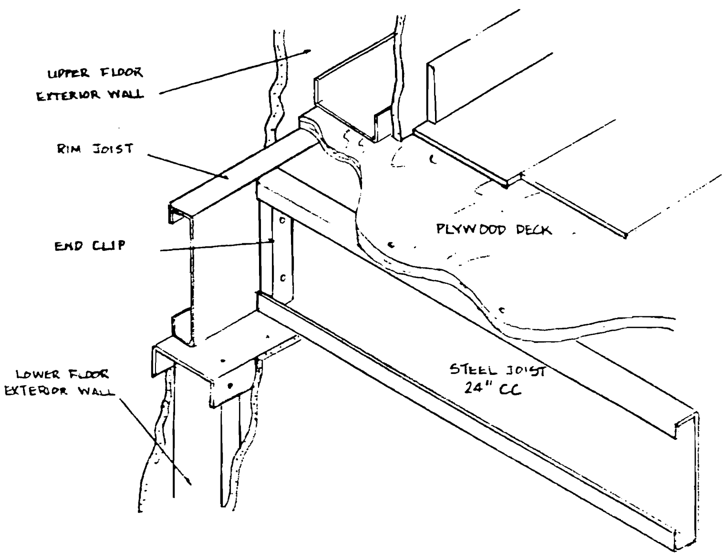


Figure 1: Cold-Formed Sheet Steel C-Shaped Joist/Plywood Flooring System

Sections 1 thru 5 of the current American Iron and Steel Institute (AISI) Specification [1][4] and its Commentary [2] do not contain any provisions on elastically restrained beams which may be subject to lateral-torsional buckling. Therefore, Section 6,

[4] See References

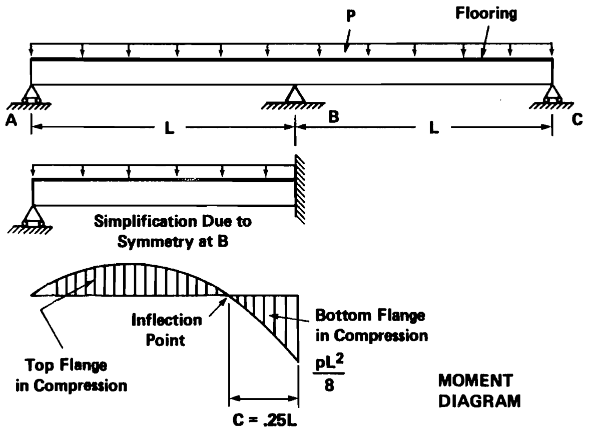


Figure 2: Bending Moment Diagram For A Uniformly Loaded Two-Span Continuous Beam

entitled "Tests for Special Cases", is applicable. However, costs for full-scale testing programs representative of the numerous joist sizes, spans, and spacings are prohibitive.

Therefore, alternate analysis methods were investigated by USSCR[5] with the objective to develop a conservative design method that could be used to (1) ensure the desired factor of safety against torsional-flexural buckling and (2) reduce the required testing to a few confirmatory tests.

As a first step, a conservative design concept now contained in Section 3 of Part III of the AISI Design Manual [3] was incorporated in an existing USSCR computer program that determines allowable uniform floor loads for joists. This design concept applies to the buckling of the *brims* (flanges) of a hat-shaped flexural member when the *brims* are in compression and the top flange of the hat is in tension. When the plywood of the floor system is compared with the top flange of the hat, then any two adjacent joists can be compared with the sides of the hat and the *brims*. Between the interior support and the inflection point (See Figure 2) of a two-span continuous floor system, the plywood floor (top flange) is also in tension and the bottom flanges of the joists (brims) are in compression.

---

[5] United States Steel Corporation – Research

Thus, between the interior support and the inflection point, the floor system is analogous to the hat section with the exception that the top flange and the sides of the hat section are connected continuously, while the plywood and the floor joists are connected intermittently by screws (usually 12 inch on centers). However, the spring constant, β, which represents the elastic restraint for the compression flange and is usually determined by a virtual displacement analysis, may not be as accurate for the floor system as it is for the hat section because of the local effects around the fasteners that connect the plywood to the joists. In addition, the AISI method assumes a uniform moment distribution over the length of the beam, while the uniform load used in designing a floor system causes a parabolic moment distribution as shown in Figure 2. Also, composite action of the plywood deck is not considered. Thus, for the reasons stated above, the use of this design method was considered to be very conservative.

A subsequent analytical study based on an elastic-buckling finite-element analysis [4] confirmed that this design method was indeed conservative and that an effective length equal to about 1/4 of the unbraced length between the interior support and the point of inflection could be used. However, the USSCR joist-design computer program was changed using an effective length of 1/2 of the unbraced length until a further reduction could be confirmed by tests. Detailed results of the experimental study conducted during the summer of 1983 are described elsewhere [5]. This paper summarizes the experimental study and the conclusions based on the study's results.

TESTS OF COLD-FORMED C-SHAPED JOISTS

OVERVIEW OF TEST PROGRAM

The physical parameters of the test specimens, such as the type of joist and span, were chosen for a case in which buckling of the bottom flange adjacent to the interior support was critical, as determined by the original USSCR computer program. One small-scale 12-in.-wide, 24-in.-long floor specimen with one joist was chosen to determine the spring constant, β, which represents the elastic restraint provided by the plywood and the joist portions in tension to the lateral-torsional displacements of the joist portion in compression. Three full-scale 40-ft-long two-span continuous floor specimens were chosen. The first specimen was used for an exploratory test and the joist properties are different from the other two full-scale tests. The other two were chosen to determine the load at which the first buckle appeared on the specimens with and without composite action of the attached plywood deck. Particular emphasis was placed on developing a test program that would entail unique test set-ups and provide comprehensive test data and test procedures for future reference.

SPRING-CONSTANT TEST SPECIMEN AND SETUP

The specimen used to determine the spring constant of the elastic restraint provided to the free flange of the full-scale joists consisted of a 24-in. by 18-in. by 3/4-in.-thick piece of AC grade plywood attached to a 24-in.-long 7.25-in.-deep C-shaped section (see Figure 3). The plywood was fixed at one end over the entire length and the joist was attached to the plywood 12 inches above the face of the fixed support as shown in Figure 3. This specimen represents one half of the full-scale specimen with 24-in. spacing between the two steel C-shaped joists as shown in Figure 4. To determine the spring constant , the bottom flanges were loaded laterally (vertical in the test setup) and the lateral deflection, D, was measured by Gauge 1 (see Figure 4). The applied load of approximately 0.001 kip/linear inch of joint length was divided by the measured lateral deflection (Gauge 1) to obtain the spring constant. Gauges 2 and 3 were used to determine the actual contributions of the plywood and local deformations around the fasteners to the spring constant. All gauges permitted readings to 1/10000th of an inch. The joist was attached to the plywood with 2-in.-long self-drilling fasteners 12 in. on centers, which is representative of the spacing used in the full-scale floor test specimens, as well as in actual construction.

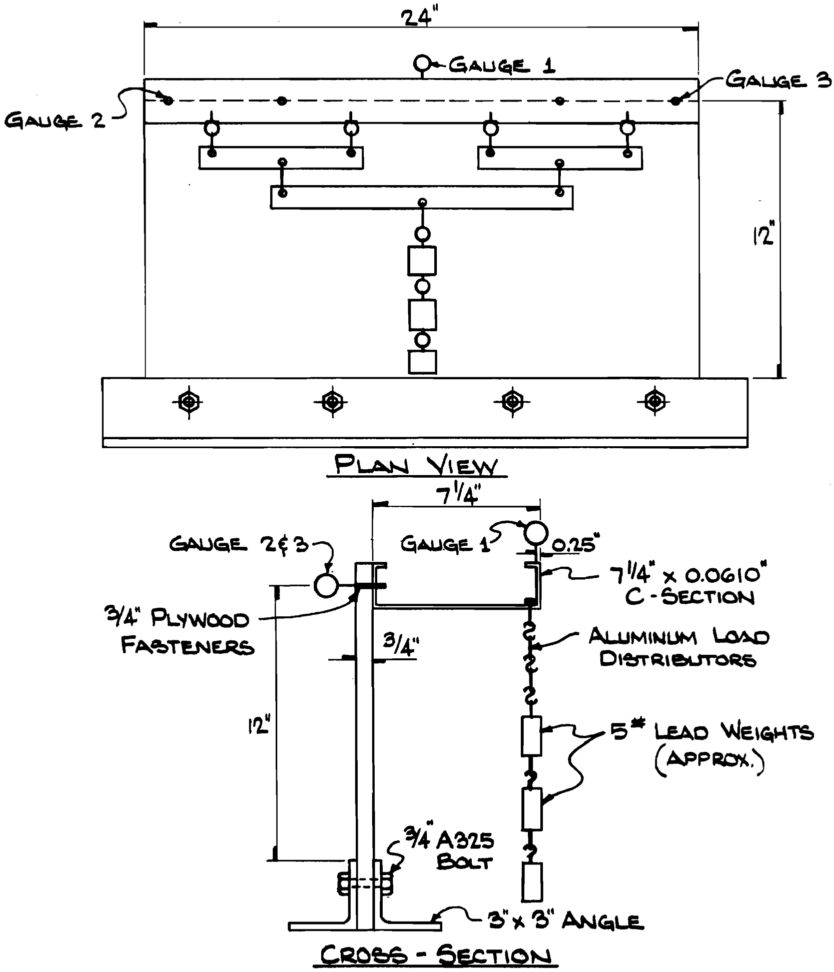
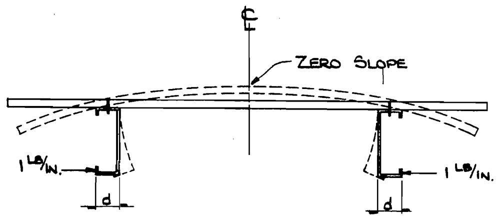


Figure 3: Spring-Constant Test Set-Up



β = 0.001 / D = (kips per linear inch) / (inches)

Figure 4: Determination of Spring Constant

FULL-SCALE STRUCTURAL FLOOR TEST SPECIMENS AND SETUP

Specimens. The large full-scale floor test specimens consisted of a 4-ft-wide by 3/4-in.-thick by 8-ft-long AC plywood panels attached to the top flange of two 40-ft-long cold-formed steel C-shaped joists as shown in Figure 5. The exact dimensions and yield strength of each set of joists used in each test are given in Figure 6. The joists for specimen 1 were from a different production run than those of specimens 2 and 3. In all three tests, the C-shaped joists were placed back to back to provide symmetry (lateral stability). Lateral spacing was 2-ft from the centers of the top flanges.

For the first two test specimens, the plywood deck was constructed with five sheets of AC grade plywood. Each sheet was butted to adjacent sheets leaving no gaps between the five pieces of plywood. The plywood was attached to the steel joists with 2-in.-long self-drilling fasteners. The fasteners were placed on 12 in. centers, starting 6 in. from the ends. A hand drill, equipped with a phillips-head screwdriver and a clutch attachment, was used for ease of fastener placement and uniformity of fastener tightness.

The third specimen was constructed so that the plywood supplied lateral and torsional restraint to the steel joists, but supplied no tensile or compressive load resistance (composite action) in the direction of the longitudinal joist axis. The elimination of composite action was achieved by using 11-3/4-in.-long plywood strips and placing a 1/4-in. gap between each strip (see Figure 7). Each strip was attached to the joists with one fastener placed in the center of the strip such that the fastener spacing was again 12-in. on centers.

Test Setup. The floor specimens were supported in an inverted position as shown in Figures 8 and 9. The plywood deck faced the floor (test base) and the steel joists were above the deck. Three floor supports (two exterior and one interior) held the floor specimen down as an airbag pushed up on the plywood deck from below. The airbag reactions on their bottom side were resisted by the test base described later. The three supports simulated the support given by the exterior and interior load bearing walls commonly found in residential and light-commercial buildings.

The exterior supports (upper beams of reaction frames) consisted of a hot-rolled 3.5-in. by 3.5-in. angle section held in place by 3/4-in. diameter threaded rods at each end of the support. The interior support consisted of a welded 5-in. by 5-in. structural tubing held in place by 1-in. diameter threaded rods at each end of the support. Load cells were

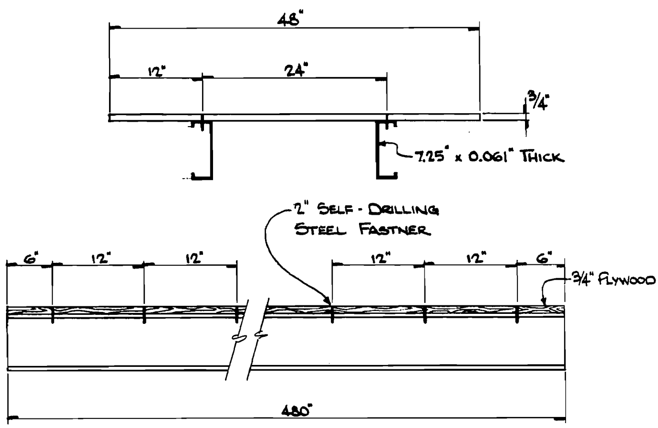


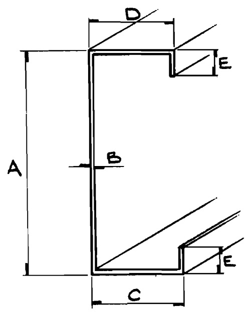
Figure 5: Typical Test Specimen

attached to the ends of all six rods and then to the test base. The load cells monitored the load in each of the rods.

The interior support was placed over the mid-length of the joists. The exterior supports were positioned so that the spans measured 19-ft-9-3/4 in. from the center of the exterior support to the center of the interior support. Bearing stiffners made of wood, having the same width as the supports, were fitted inside each joist under each support. These stiffners were added to prevent crippling of the joist webs at the supports (see Figure 9).

The test base, which served as a reaction to the airbags and the support rods, was constructed of two 40-ft-long HP8x36 hot-rolled I-sections and 1-in.-thick plywood supported by three 6-ft-long HP8x36 steel sections (lower part of reaction frame) placed on the concrete floor. After the test base was assembled, the airbag was placed on top of the plywood deck, followed by the membrane (Test 3 only), and the inverted specimen. The support rods with load cells were attached to the top and bottom support members with a "uniball" connection to ensure that a moment would not be transfered to the load cell. Before each test, the support rods were set plumb with a level.

To prevent lateral movement of the third specimen due to lateral buckling in the positive bending region, a membrane of fiber-reinforced waterproofed paper, 4-ft wide by 0.012-in. thick by 40-ft long, was placed between the plywood and the airbags. The paper was



SPECIMEN	DIMENSIONS (in) A	B	C	D	E	F_y (ksi)
1	7.344	0.061	1.813	1.688	0.531	41.5
2	7.344	0.063	1.875	1.750	0.531	59.7
3	7.344	0.063	1.875	1.781	0.531	58.5

Figure 6: Cold-Formed Steel Joist Dimensions

held in place by the friction of the airbag and the plywood. This friction increased as the uniform floor load increased. The airbag alone may have been sufficient to prevent the plywood's lateral

Two custom-made 4-ft-wide by 3-in.-deep by 20-ft-long matress airbags were used to load each specimen. The airbags could safely be inflated to a thickness of about 3 in. and still maintain full contact with the plywood deck, thus ensuring a uniform load distribution throughout the tests. The airbags were contructed of 10-mil-thick clear plastic by heat-sealing the top and bottom pieces to the side pieces with lap seams. Two fittings, one for air intake/exhaust and one for a manometer, were placed on each airbag. The intake fittings of both bags were connected to the same airline connected to an air regulator; the manometer fittings of both bags were connected to the same airline connected to a water filled manometer. The air pressure needed for filling the bags was provided by a hydraulic compressor capable of producing a maximum pressure of 9 psi or 1,296 psf.

<u>Instrumentation</u>. The loads placed on the full-scale floor specimens were measured in two ways: by six load cells attached to the support rods and by the manometer readings of the airbag pressure. Also, the vertical displacements of the floor and the horizontal displacements of the bottom flanges of the joists were recorded.

The exterior support loads were measured with 8-kip load cells and the interior support loads with 20-kip load cells. These load cells contained strain gages which measured strain as the cells were loaded. These strains were calibrated for known loads prior to testing.

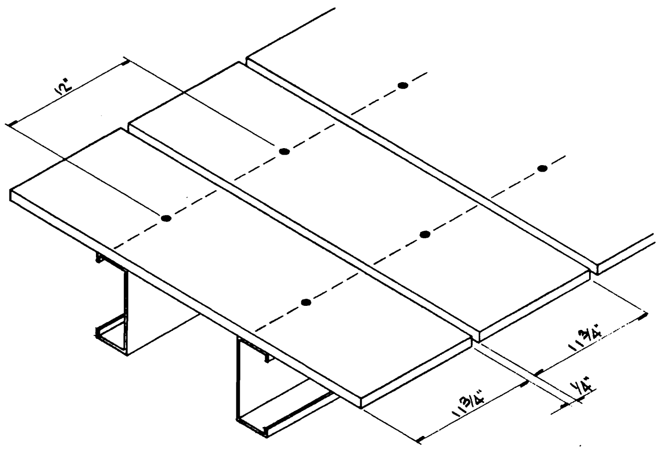


Figure 7: Third Full-Scale Test Specimen's Discontinuous Plywood Deck

The airbag pressure was measured with a water-filled manometer which was capable of reading a maximum airbag pressure of 2 psi (288 psf) or 55.4 inches of water. Because the weight of the floor assembly was resting on the airbag during testing, the manometer readings included the weight of the test specimen while the load cell readings did not.

All displacements were monitored with Direct Current Displacement Transducers (DCDT). All DCDTs were calibrated prior to testing. A total of twenty-six DCDTs were used to monitor displacements of the test specimen as it was being loaded. Fourteen were used to measure vertical displacements of the floor (adjacent to the joists) and twelve were used to measure the horizontal displacement of the bottom (free) flanges of the joists. Two vertical DCDTs were placed at each support, one at each joist location (6 total). Also, vertical DCDTs were placed at each joist at the center of both spans and at 60 in. from each side of the interior support (8 total). Horizontal DCDTs were placed at 60 in., 120 in., and 180 in. from the interior support for both spans and joists (12 total). A diagram of the DCDT locations is given in Figure 10. All DCDTs were attached to a separate floor supported framing system as seen in Figure 8.

Data Acquisition. Two data acquisition systems were used to read and process the test data of the full-scale floor tests: Acurex's Autodata Twenty and Digital Equipment Corporation's MINC. Both systems were needed because neither had the capacity to handle a total of 26 DCDTs alone. The Autodata Twenty was programmed to read 20 DCDTs.

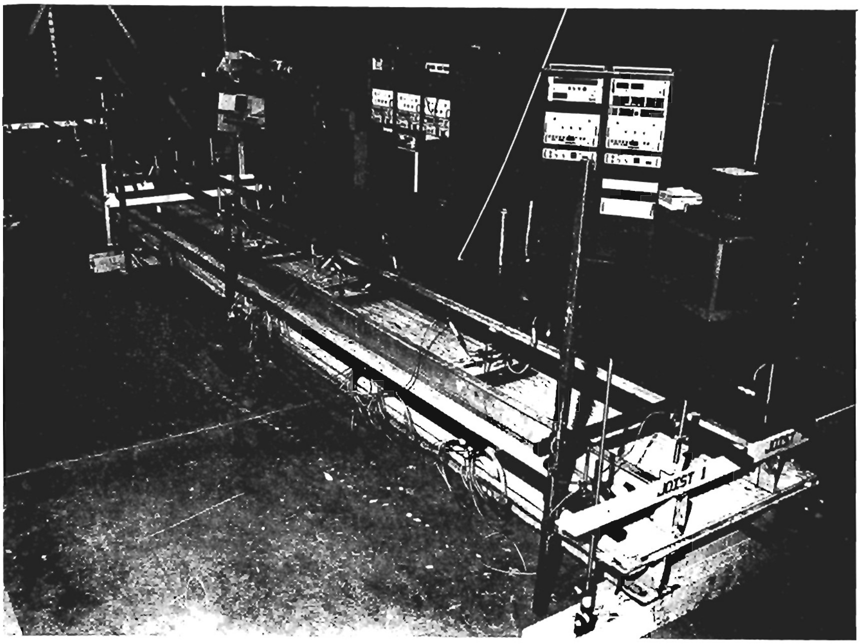


Figure 8: Full-Scale Test Setup

The MINC was programmed to read 6 DCDTs, accept DCDT data from the Autodata Twenty over an ASCII serial character data line, organize this data and the data read from the 6 DCDTs, produce vertical and horizontal displaced shapes of both joists on the screen, print the data out in a readable format on the printer device, and store the data on a floppy disk for later use. The displaced shapes were valuable for seeing the amplified vertical and horizontal deflection patterns of the joists during testing. The BASIC program used to control MINC operations is listed elsewhere (Appendix D of [5]).

Testing Procedure. The procedure used for testing each floor specimen was as follows:

1. The specimen was lifted off the test base by the airbag with the upper supports totally removed from the test set-up. This was done to determine the dead load of the floor specimen by measuring the bag pressure with the manometer connected to the airbag.

2. Then, the supports were placed on the floor but not fastened to the test base. The increase in bag pressure read by the manometer was taken to be the weight of the supports.

3. The supports were then fastened, but not tightened, to the test base.

4. All load cells and DCDTs were zeroed.

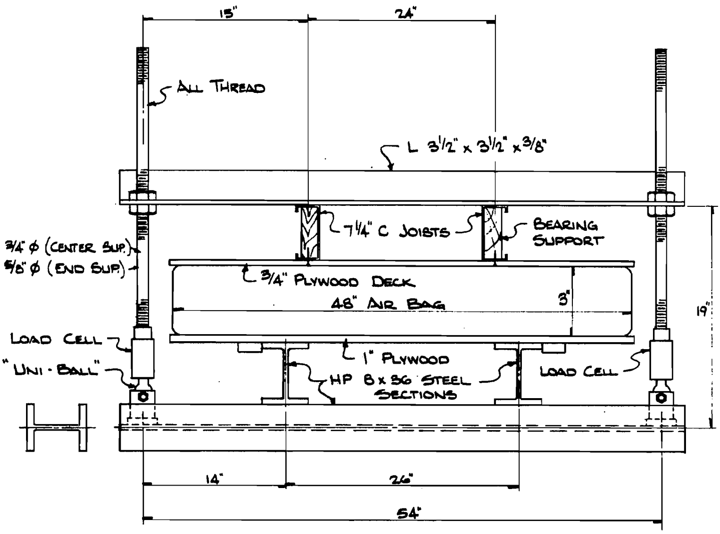


Figure 9: Typical Cross-Section of Floor Supports

5. The pressure in the bag was increased in increments of 1 in. of water (5.189 psi). For each load increment, the load cells and DCDTs were read and the data were printed out and stored.

6. At each load increment, the area of each joist adjacent to the interior support was visually inspected for buckles.

7. When a buckle was found, the airbag pressure was recorded and the load cells and DCDTs were read and stored.

8. For Test 3, loading was continued until one of the airbags failed.

TEST RESULTS

The results for the spring-constant determination test and the three full-scale floor tests are summarized below. The displacement data for the spring-constant test, the load cell data for each full-scale test, and the graphs of the centerspan vertical and horizontal displacements are displayed in detail elsewhere (Appendix F of [5]).

Spring-Constant Test Results. The vertical deflection of the free flange was 0.422 in. for an applied load of 26.076 lbs, or 1.087 lb/in. Thus, the spring constant, β, was experimentally found to be 2.57 lb/in./in.

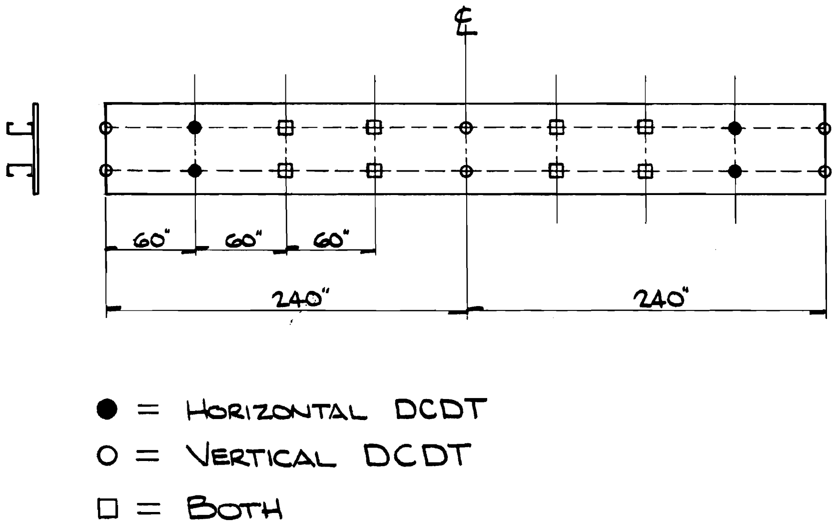


Figure 10: DCDT Locations

Full-Scale Test Results. The detailed incremental deflection measurements during loading of the three full-scale floor tests will be presented in the next chapter. All three tests ultimately failed in an interactive buckling mode between the web and the free flange adjacent to the interior support, as typified by Figure 11 for Test Specimen 3. The net loads when buckling first occured, along with the concomitant average vertical and horizontal deflections at mid span, are shown in Table 1.

COMPARISON AND DISCUSSION OF RESULTS

The test results described in the previous chapter are compared with the analytical results for the spring-constant specimen and the full-scale floor joists.

SPRING-CONSTANT TEST

The spring constant, β, determined from the test (2.57 lb/in./in.) is significantly less than that determined from a virtual work analysis[6] (4.39 lb/in./in.). Two major factors contribute

---

[6] Details of the virtual work analysis are shown in Appendix E of Reference [5]

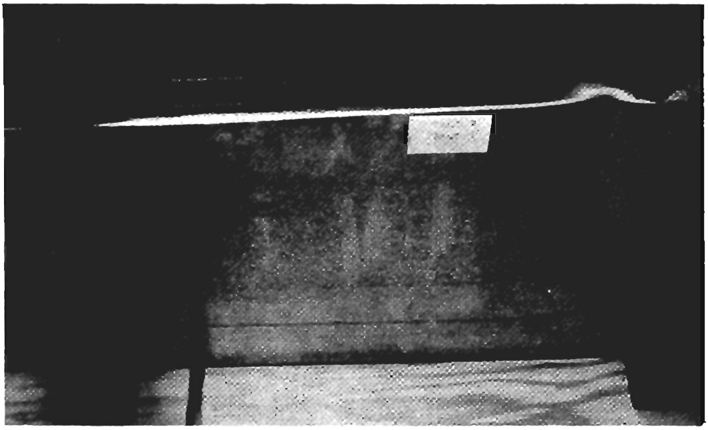


Figure 11: Local Buckle of Web and Compression Flange of Specimen 3

SPECIMEN NUMBER	LOAD (PSF)	DEFLECTIONS (in.) VERTICAL	HORIZONTAL
1	64.4	0.730	0.416
2	82.6	1.892	0.646
3	71.6	1.503	0.654

The vertical deflection listed is the average centerspan deflection of the floor and the horizontal deflection is the average centerspan deflection of the bottom flanges.

Table 1: Structural Test Results Summary at First Buckle

to this difference: (1) the actual modulus of elasticity of the plywood[7] (about 675 ksi) is much smaller than the theoretical value reported in the Plywood Design Specification [6]

[7] Calculated in Appendix E of Reference [5]

(about 1350 ksi), and (2) the connection between the plywood and joists is by screws spaced 12-inches on centers, which causes additional local deformations at and around the fasteners, while the analysis assumes a uniform continuous connection without considering such local deformations.

For a floor assembly with C-shaped joists faced in the same direction and not back-to-back, as assumed in the USSCR joist program and most commonly found in actual construction, the theoretical spring constant calculated within the USSCR joist program is 3.42 lb/in./in. Therefore, based on the spring-constant test, a value of 2.00 lb/in./in. (3.42 * 2.57/4.39) appears more reasonable for floor joists facing in the same direction and spaced 24-inches on centers with 3/4-in.-thick plywood flooring.

FULL-SCALE FLOOR TESTS

The ultimate loads from the full-scale floor tests are compared with the results from the original USSCR joist program (Appendix B of [5]), and with the findings of a previously-conducted finite-element elastic-buckling analysis [4]. The results are summarized in Table 2.

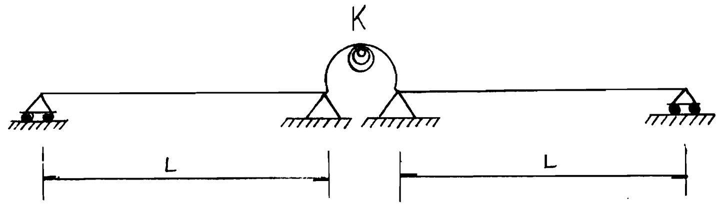
The finite-element elastic-buckling analysis [4] indicated that the effective buckling length of the compression flange may be reduced to approximately one-fourth of the distance between the interior support and the inflection point. Therefore, four effective lengths (L_{eff}) were used in the USSCR joist program to analytically determine the buckling load of each test specimen: L, L/2, L/4, L/10000, where L is the distance between the interior support and the point of inflection. The results of the USSCR joist program are based on the actual dimensions and properties of each test specimen and on the spring-constant determined from the spring-constant test. Also, the results of the computer program were multiplied by a safety factor of 1.67 to obtain the ultimate loads shown. The numbers in parentheses indicate the critical design rule (the design rule code is defined in Table 3). The actual and analytical results compare as follows.

For full-scale test 1, the web and compression flange adjacent to the interior support buckled at 64.4 psf. The USSCR joist program predicted a lateral torsional buckling load of 43.4 psf for L_{eff} = L and 51.8 psf for L_{eff} = L/2. The program predicted a combined bending and shear failure at 52.6 psf when L_{eff} was equal to, or less than, L/4. The test confirmed that lateral-torsional buckling was not the critical mode of failure. The actual failure mode was local flange and web buckling. The actual failure is 22.4 percent greater than that predicted by the USSCR joist program if L_{eff} = L/4 were used. Some of this percentage is attributable to the composite action of the plywood deck which is ignored in the USSCR program.

For test 2, the web and compression flange adjacent to the interior support also buckled, but at a load of 82.6 psf. This raise in buckling load is largely attributable to the higher yield strength (59.7 ksi vs. 41.5 ksi) of the joists in specimen 2. The USSCR program predicted lateral-torsional buckling loads of 55.1 psf and 72.7 psf when L and L/2 were used as the laterally unbraced length (L_{eff}), respectively. The program again predicted a combined bending and shear failure at a load of 74.3 psf when an unbraced length equal to, or less than, L/4 was used. For test 2, the actual failure was 11.2 percent greater than that predicted by the USSCR program, using L_{eff} = L/4, therefore, lateral-torsional buckling is not critical, but web buckling due to shear/bending interaction is critical.

For test 3, the web and compression flange adjacent to the interior support buckled in the same manner as the first two tests. The ultimate load was 71.6 psf. Because the composite action between joists and plywood could not be developed in the third test, the third specimen failed at a lower load than specimen 2. The yield strengths and dimensions of the joists in the second and third specimens were nearly identical. Therefore, the difference in the load carrying capacity of tests 2 and 3 is directly attributable to the composite action between joists and plywood (approximately 15 percent additional strength). The USSCR program predicted lateral-torsional buckling at loads of 54.3 psf and 71.81 psf when L and L/2 were used for the laterally unbraced length (L_{eff}), respectively. The program predicted a combined bending and shear failure at a load of 73.5 psf when an unbraced length equal to, or less than, L/4 was used. The difference between the USSCR predictions for tests 2 and 3 are solely due to the slight differences in section dimensions and yield strength. Thus, for test 3, the actual load was 2.5 percent *less* than that predicted by the USSCR program, using L_{eff} = L/4. The test's failure mode agrees with the USSCR prediction that lateral-torsional buckling is not critical, but that web buckling due to shear-bending interaction is critical.

For all three tests, the actual reactions and displacements, shown elsewhere (Appendix F Tables F-5 through F-7 of [5]), were compared with those predicted by classical two-span-continuous and simply-supported beam theories. It was found that the floor system was not acting as a true two-span-continuous beam because : (1) the experimental loads were redistributed from the interior support to the exterior supports, and (2) the experimental displacements were higher than the theoretical displacements of a two-span continuous beam and lower than those of a simply-supported beam. These findings lead to the conclusion that a rotational spring may be envisioned at the center support rather than a fully rigid connection (see Figure 12).



Center Rotational Spring

Figure 12: Two Spans Connected By Center Rotational Spring

A detailed analysis of the deflections and reactions (Appendix F and G of Reference [5]) indicated that the load redistribution commenced immediately with the first load increment and remained proportionally the same for all subsequent load steps, except for the deflections of test 1. For this exploratory test, the DCDTs used to measure the vertical displacements were located too close to the free flanges of the joists and were soon pushed sideways by the laterally moving free flanges. This contact between these DCDTs and the bottom flanges resulted in smaller-than-actual vertical displacements at the midspans. The reactions recorded by the load cells were used to determine the effective spring rigidity at each load increment. Since the load redistribution was nearly linear, the rigidity values for the load increments up to the observation of the first buckle were used to determine the average effective spring rigidity, K, for each test. The resulting K values were 23.78, 14.77, and 19.20 EI/L for tests 1, 2, and 3, respectively. The spring rigidity for a fully rigid or a hinged connection at the interior support would be ∞ (infinite) and 0 (zero), respectively.

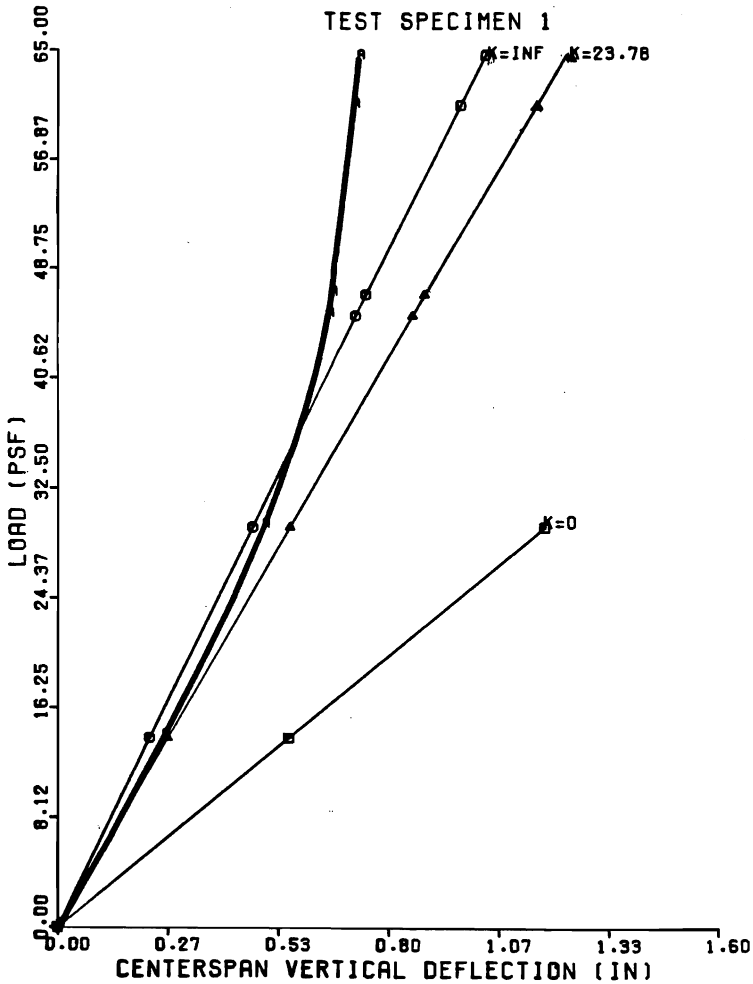
The average effective spring rigidity of each test specimen, along with K = ∞ and K = 0, were used to determine three load-displacement curves for tests 1, 2, and 3, as shown in Figures 13, 14, and 15, respectively. Also shown in each figure is the average midspan vertical deflection for each test, based on the four deflection measurements taken at each load increment. As seen from Figure 13, the measured deflections for test 1 initially coincide with those predicted with the average effective spring rigidity but soon divert from it for reasons stated above. However, the measured displacements for tests 2 and 3 are in excellent agreement with the linear load-displacements predicted with the effective spring rigidity and fall appropriately between those for K = ∞ and K = 0.

The average effective spring rigidity values were also used to determine the effective bending moment at the interior support. The results are shown in Table 4. The actual peak negative moment at the interior support was about 14 percent less than that expected using classical two-span continuous beam theory.

At first, this seems suprising in view of test 3, which failed slightly below the predicted load based on the classical two-span continuous beam theories. A smaller negative moment than that calculated should have resulted in higher test loads than experienced. However, a more detailed analysis of the effective section properties (moment of inertia and section properties) as a function of the compressive stresses in the flange and web elements, produces a conceptional section-properties distribution as shown in Figure 16. This distribution, in effect, shows that the moment of inertia is least at the interior support, where the moment initially is greater than at any other point along the beam. This results in a shift of the reactions of the interior supports towards the exterior supports, with effects similar to those of an elastic spring at the interior support, as discussed above. However, as evidenced by test 3, the resulting reduction of the bending moment at the interior support does not appear to provide for a significant increase in the ultimate load. A possible explanation is that the compressed portions of the joist (web, flange, and/or lip) are weakened more severely at the interior support because of the increased rotations which may reduce the overall stability of the compressed web, flange, and/or lip.

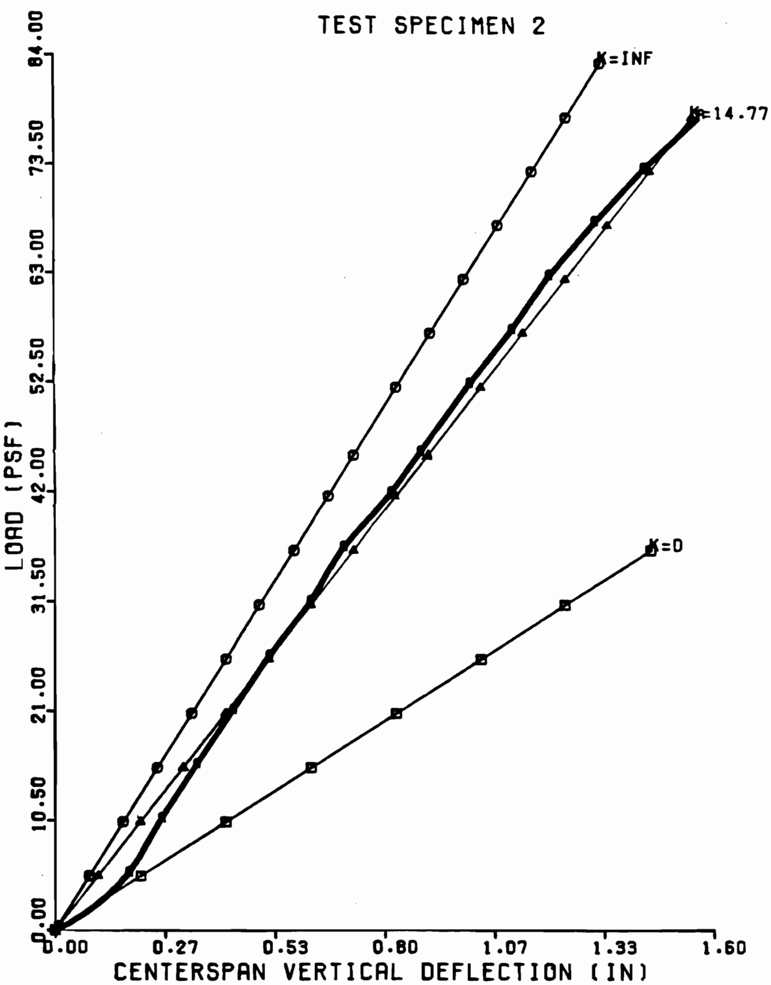
CONCLUSIONS AND RECOMMENDATIONS

Three full-scale floor tests were performed to determine the buckling characteristics of flooring systems using cold-formed sheet-steel joists continuous over two spans and 3/4-in-thick plywood. The floor specimens were uniformly loaded until the joists failed. These failure loads were compared with those predicted by a USSCR joist-design computer program which is based, conservatively, on the AISI Design Specifications and Supplementary Information. The laterally-unbraced length of the compression flange adjacent to the interior support was varied and the predicted analytical failure loads were compared with the actual failure loads to determine the effective laterally-unbraced length. The test results and the computer program results showed that *a plywood deck of 3/4-in thickness attached to the top flange of a two-span-continuous steel C-shaped floor joist, reduces the laterally-unbraced length of the unbraced compression flange to one-fourth of the distance between the interior support and the inflection point using current AISI design information for beams with uniform bending moments.* When L/4 is used as the laterally unbraced length, lateral-torsional buckling is no longer controlling failure mode of the floor systems investigated, but combined bending and shear becomes critical. The buckling load predicted by the USSCR joist program, when L/4 is used, is about 15 percent conservative because, in part, composite action is not considered.



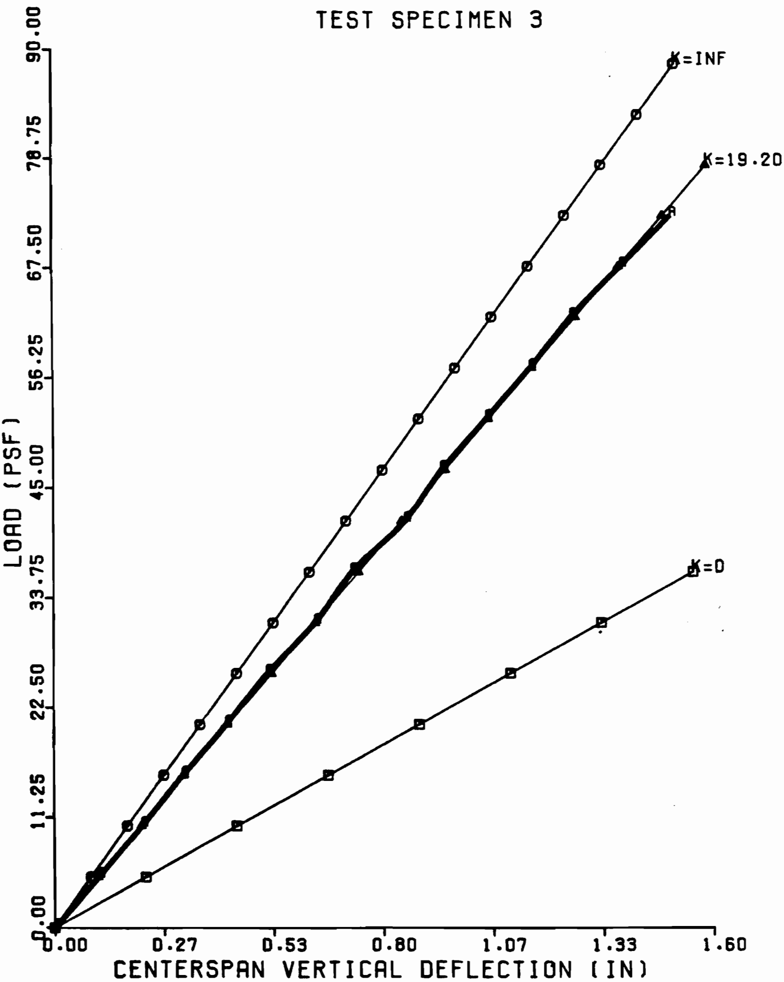
A = Average of four centerspan deflections

Figure 13: Theoretical and Actual Load–Displacement Curves – Test 1



A = Average of four centerspan deflections

Figure 14: Theoretical and Actual Load-Displacement Curves - Test 2



A = Average of four centerspan deflections

Figure 15: Theoretical and Actual Load–Displacement Curves – Test 3

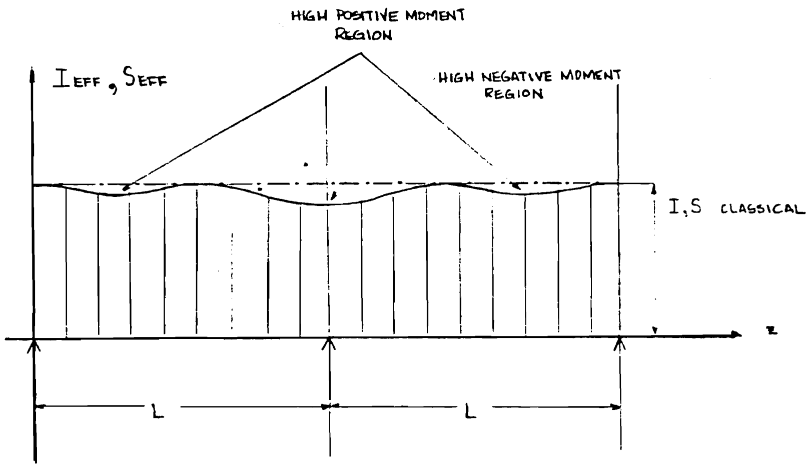


Figure 16: Conceptional Distribution of Effective Section Properties

SOURCE	ULTIMATE LOADS, PSF SPECIMEN 1	SPECIMEN 2	SPECIMEN 3
TEST	64.4	82.6	71.6
USSCR JOIST PROGRAM[8] WITH:			
(a) L_{eff} = L	43.4 (9)[9]	55.1 (9)	54.3 (9)
(b) L_{eff} = L/2	51.8 (9)	72.7 (9)	71.8 (9)
(c) L_{eff} = L/4	52.6 (6)	74.3 (6)	73.5 (6)
(d) L_{eff} = L/10000	52.6 (6)	74.3 (6)	73.5 (6)

Table 2: Comparison of Full-Scale Test Results

[8] Based on actual dimensions and mechanical properties of each test specimen and on the spring-constant test described in Chapter 2.

[9] Numbers is paranthesis define the critical design rule as defined in Table 3.

NUMBER	DESCRIPTION	AISI SECTION
1	Bending, basic design stress	Section 3.1
2	Bending, compression on unstiffened lip	Section 3.2.1
3	Buckling of top flange between fasteners	Section 3.3.1
4	Bending stress in web	Section 3.4.2.1
5	Shear at interior support	Section 3.4.1
6	Shear and bending combined	Section 3.4.3
7	Crippling at interior support	Section 3.5.1
8	Bending and crippling combined	Section 3.5.2
9	Buckling of bottom flange adjacent to interior support; spans fully loaded, no lateral bracing	Part III, Section 3
10	Buckling of bottom flange adjacent to interior support; spans partially loaded, no lateral bracing	Part III, Section 3

Table 3: Critical Design Rule Code

TEST	$K = \infty$ (wL^2)	K_{CRS} EI/L	$K = K_{CRS}$ (wL^2)
1	1/8.0	23.78	1/9.0
2	1/8.0	14.77	1/9.6
3	1/8.0	19.20	1/9.3
	AVG = 1/8.0	AVG = 1/9.3	

Table 4: Comparison of Peak Negative Moments

As seen from the load cell data and the vertical displacement data, the flooring system does not behave as a true two-span-continuous beam, but behaves as though its two spans were connected at the center support, by an elastic rotational spring. *This reduced fixity at the interior support results in a decrease of the peak negative moment to about 86 percent of the peak negative moment expected from two-span-continuous beam theory.* This reduction in the peak negative moment at a given load is believed to be offset by a loss of stability in the compressed elements due to the increased rotation of the section at this location. The redistribution of the bending moment is believed to be caused the reduction of the effective moment of inertia of the light-gage steel joists in high bending-moment regions.

A previously recommended reduction of the effective laterally-unbraced length of the bottom flange to one-fourth (1/4) the distance between the inflection point and the interior support [4] is consistent with the test results described. It is recommended that further research be conducted to substantiate these findings and investigate a wider range of joist dimensions such that appropriate design provisions can be developed for use by other designers. It is also recommended that a more detailed investigation of the rotational behavior of the joist sections over the interior support be conducted.

REFERENCES

[1] *'Specification for the Design of Cold-Formed Steel Structural Members', Cold-Formed Steel Design Manual--Part I*
American Iron and Steel Institute, 1980.

[2] *'Commentary on the 1980 Edition of the Specification for the Design of Cold-Formed Steel Structural Members', Cold-Formed Steel Design Manual--Part II*
American Iron and Steel Institute, 1983.

[3] *'Supplementary Information on the 1968 Edition of the Specification for the Design of Cold-Formed Steel Structural Members', Cold-Formed Steel Design Manual--Part III*
American Iron and Steel Institute, 1977.

[4] J. H. Garrett, Jr., G. Haaijer, K. H. Klippstein.
Effective Lengths for Laterally Unbraced Compression Flanges of Continuous Beams Near Intermediate Supports.
In Wei-Wen Yu and Joseph H. Senne (editors), *Recent Research and Developments in Cold-Formed Steel Design and Construction*, pages 179-210. Department of Civil Engineering, University of Missouri-Rolla, November, 1982.

[5] J. H. Garrett, Jr.
Evaluation of Elastically Restrained Compression Flanges of Cold-Formed C-Shaped Sections.
Master's thesis, Carnegie-Mellon University, September, 1983.

[6] *Plywood Design Specification*
American Plywood Association, 1976.